UNIVERZITET U NOVOM SADU
FAKULTET TEHNIČKIH NAUKA U NOVOM SADU

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COMPARATIVE ANALYSIS OF DURABILITY AND RANKING OF CONCRETE BRIDGES IN HOT CLIMATES

DOCTORAL THESIS

КОМПАРАТИВНА АНАЛИЗА ТРАЈНОСТИ И РАНГИРАЊА БЕТОНСКИХ МОСТОВА У ТОПЛИМ КЛИМАТИМА

ДОКТОРСКА ДИСЕРТАЦИЈА

Novi Sad, 2022.

УНИВЕРЗИТЕТ У НОВОМ САДУ **ОБРАЗАЦ – 5а**
ФАКУЛТЕТ ТЕХНИЧКИХ НАУКА

КЉУЧНА ДОКУМЕНТАЦИЈСКА ИНФОРМАЦИЈА[1]

Врста рада:	Докторска дисертација
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Наслов рада:	КОМПАРАТИВНА АНАЛИЗА ТРАЈНОСТИ И РАНГИРАЊА БЕТОНСКИХ МОСТОВА У ТОПЛИМ КЛИМАТИМА
Језик публикације (писмо):	Енглески (латиница)
Физички опис рада:	Унети број: Страница - 617 Поглавља - 14 Референци - 37 Слика - 563 Табела - 104 Графикона - Дијаграма - Хистограма – Алгоритама - Шема - Прилога -
Научна област:	Грађевинско инжењерство
Ужа научна област (научна дисциплина):	Трајност и процена стања бетонских конструкција
Кључне речи / предметна одредница:	Бетон, дефект, оштећење, детериорација, санација, одржавање, систем за управљање мостовима
Резиме на језику рада:	У докторској дисертацији су приказани резултати сопственог истраживања трајности армиранобетонских мостова у топлим климатима. Урађена је процена стања седам армиранобетонских мостова у Либији, прорачун њиховог рејтинга и њихово рангирање са аспекта врсте и степена оштећења, у складу са немачком БМС методологијом. Формиран је каталог карактеристичних дефеката и оштећења и дефинисани су основни узроци њиховог настанка. Постављени циљеви су испуњени, а

[1] Аутор докторске дисертације потписао је и приложио следеће Обрасце:

5б – Изјава о ауторству;

5в – Изјава о истоветности штампане и електронске верзије и о личним подацима;

5г – Изјава о коришћењу.

Ове Изјаве се чувају на факултету у штампаном и електронском облику и не кориче се са тезом.

	анализа резултата истраживања показала је да се специфичности топлих климата морају узети у обзир приликом процене стања, одређивања рејтинга мостова и предлога стратегије њиховог одржавања.
Датум прихватања теме од стране надлежног већа:	12.07.2018.
Датум одбране: (Попуњава одговарајућа служба)	
Чланови комисије: (титула, име, презиме, звање, институција)	Председник: проф. др Бојан Матић, ванредни професор, Факултет техничких наука, Универзитет у Новом Саду Члан: др Ксенија Јанковић, научни саветник, Институт ИМС, Београд Члан: проф. др Иван Лукић, ванредни професор, Факултет техничких наука, Универзитет у Новом Саду Ментор: проф. др Мирјана Малешев, редовни професор, Факултет техничких наука, Универзитет у Новом Саду Ментор: проф. др Властимир Радоњанин, редовни професор, Факултет техничких наука, Универзитет у Новом Саду
Напомена:	

UNIVERSITY OF NOVI SAD
FACULTY OR CENTER

KEY WORD DOCUMENTATION[2]

Document type:	Doctoral dissertation
Author:	SAEEDA OMRAN SALEH FURGAN
Supervisor (title, first name, last name, position, institution)	prof. dr Mirjana Malešev, full professor, Faculty of Technical Sciences, University of Novi Sad prof. dr Vlastimir Radonjanin, full professor, Faculty of Technical Sciences, University of Novi Sad
Thesis title:	COMPARATIVE ANALYSIS OF DURABILITY AND RANKING OF CONCRETE BRIDGES IN HOT CLIMATES
Language of text (script):	English language (latinic)
Physical description:	Number of: Pages - 617 Chapters -14 References - 37 Illustrations - 563 Tables - 104 Graphs - Diagrams - Histograms - Algorithms - Schemes – Appendices -
Scientific field:	Civil Engineering
Scientific subfield (scientific discipline):	Durability and assessment of concrete structures
Subject, Keywords:	Concrete, defect, damage, deterioration, repair, maintenance, bridge management system

[2] The author of doctoral dissertation has signed the following Statements:

5б – Statement on the authority,

5в – Statement that the printed and e-version of doctoral dissertation are identical and about personal data,

5г – Statement on copyright licenses.

The paper and e-versions of Statements are held at the faculty and are not included into the printed thesis.

Abstract in English language:	The doctoral dissertation presents the results of own research into the durability of reinforced concrete bridges in warm climates. An assessment of the condition of seven reinforced concrete bridges in Libya, the calculation of their rating and their ranking from the aspect of type and degree of damage, in accordance with the German BMS methodology, was done. A catalog of characteristic defects and damage was created and the basic causes of their occurrence were defined. The set goals were met, and the analysis of the research results showed that the specificities of warm climates must be taken into account when assessing the condition, determining the rating of bridges and proposing a strategy for their maintenance.
Accepted on Scientific Board on:	12.07.2018.
Defended: (Filled by the faculty service)	
Thesis Defend Board: (title, first name, last name, position, institution)	President: prof. dr Bojan Matić, associate professor, Faculty of Technical Sciences, University of Novi Sad Member: dr Ksenija Janković, full research professor, IMS Institute, Belgrade Member: prof. dr Nebojša Radović, full professor, Faculty of Technical Sciences, University of Novi Sad Member: prof. dr Ivan Lukić, associate professor, Faculty of Technical Sciences, University of Novi Sad Member, Mentor: prof. dr Mirjana Malešev, full professor, Faculty of Technical Sciences, University of Novi Sad Member, Mentor: prof. dr Vlastimir Radonjanin, full professor, Faculty of Technical Sciences, University of Novi Sad
Note:	

ACKNOWLEDGEMENTS

I would like to show my sincere gratitude to my supervisors, **Prof. Mirjana Malešev** and **Prof. Vlastimir Radonjanin,** for their constant assistance, patience, tolerance, and support, as well as for their valuable directions and academic advices.

In addition, I would like to thank everyone in Novi Sad University who facilitated my work in this study.

Finally, I want to express my deep thanks to all professors who ever taught me.

КОМПАРАТИВНА АНАЛИЗА ТРАЈНОСТИ И РАНГИРАЊА БЕТОНСКИХ МОСТОВА У ТОПЛИМ КЛИМАТИМА

ПРОШИРЕНИ ИЗВОД ДОКТОРСКЕ ДИСЕРТАЦИЈЕ НА СРПСКОМ ЈЕЗИКУ

Објекти саобраћајне инфраструктуре, првенствено мостови, су од суштинског значаја за континуирани развој привреде сваке земље. Одржавање оваквих објеката у оптималним условима и у сваком тренутку је тежак задатак за службе које су за то одговорне. Да би се помогло у обављању таквог задатка, развијени су системи за управљање инфраструктуром (IMS) за ефикасно управљање имовином. Међу различитим IMS-овима, за примену у овом истраживању одабран је систем управљања мостовима (BMS), јер су мостови најважнији објекти у саобраћајној мрежи. BMS такође захтева различите врсте улазних података за одређивање рејтинга мостова. Најзначајнији BMS подаци су записи о претходним прегледима мостова јер без ових података пректично је немогуће да се прецизно предвиди будући статус моста, укључујући и његову трајност.

Процена трајности постојећих армирано-бетонских мостова, као и њихово одржавање, однедавно је тема интензивних истраживања у грађевинарству. Студија о процени трајности армиранобетонских мостова може открити потенцијални ризик у конструкцији и пружити тачне информације за доношење правовремених одлука за поправку, ојачање или уклањање мостова, како би се избегле тешке незгоде. Проблем одржавања и санације мостова је питање од великог значаја, посебно у развијеним земљама, као што су Сједињене Америчке Државе, Канада, Јапан, Аустралија, Европска Унија и итд. И друге земље покушавају да прате ове трендове. У циљу ефикаснијег одржавања мостова, посебно за планирање финансијских средстава за обезбеђивање њихове функционалности и безбедности, развијени су бројни експертски системи. Такође, уочено је да постоји мали број студија, у којима је анализирана применљивост постојећих БМС за мостове у топлим климатима. Стога ће анализа стања либијских бетонских мостова помоћи да се дефинишу типични дефекти и оштећења, који су карактеристични за топле климате. На овај начин ће се добити поуздани подаци који ће, уз остале неопходне информације (значај моста у обиму путне мреже, расположива средства за реконструкцију и др.), дати тачно рангирање мостова у погледу оптималног одржавања путне мреже.

Системи цивилне инфраструктуре, посебно путеви и мостови играју суштинску улогу у економији нација и њихова вредност у већини земаља је изузетно велика. У Северној Америци, на пример, укупна вредност инфраструктурних система процењује се на 33 трилиона долара. Просечна годишња потрошња на инфраструктурне системе се процењује на 303 милијарде долара у USA. Стога је одржив рад ових инфраструктурних објеката од кључног значаја. Велики проценат постојећих инфраструктурних објеката пропада због старости, агресивних услова околине и недовољног саобраћајног капацитета.

Због величине и цене инфраструктурних мрежа, одржавање таквих мрежа је изазован задатак, посебно у светлу ограничених буџета доступних за одржавање инфраструктуре. Сходно томе, земље (општине и транспортне агенције) су под све већим притиском да развију нове стратегије за управљање јавном инфраструктурном имовином на начин који обезбеђује дугорочну одрживост под ограниченим буџетима.

Овај проблем није специфичан само за земље са добро развијеном инфраструктурном мрежом, већ је веома важан и за земље са лошом мрежом путева. У овим земљама, у случају прекида саобраћаја, јављају се бројни проблеми јер постоји само један пут између суседних градова, без алтернативних саобраћајница. Зато је одговарајућа стратегија управљања путевима и мостовима веома важна и за мање развијене земље.

Мостови су интегрална инфраструктурна компонента и од великог су значаја за функционисање саобраћаја, те су као такви били предмет обимних истраживачких пројеката везаних за перформансе ових конструкција. Међутим, било је мало студија о перформансама безбедности саобраћаја на мостовима, који имају веома различите физичке и оперативне карактеристике.

Потреба за истраживањем

Постоји много дефиниција за појам трајност бетона. Неке од њих су:

- Способност бетона да издржи утицаје за које је пројектован без оштећења током дугог периода година позната је као трајност.
- Трајност бетона је способност бетона да се одупре атмосферским утицајима, хемијским дејствима и абразији, уз задржавање пројектованих својстава.

Обично се односи на трајање или животни век објекта без потребе за већим улагањима. Различити бетони захтевају различите степене трајности у зависности од изложености окружењу и жељених својстава. На пример, бетон изложен морској води имаће другачије захтеве од бетона у затвореном простору.

Бетон ће остати трајан ако:

- Је структура цементне пасте густа и ако има малу пропустљивост.
- У екстремним условима ниских температура, има довољно увученог ваздуха, да би се одупрео циклусима смрзавања-одмрзавања.
- Направљен је од сепарисаног агрегата који има добре механичке карактеристике и који је хемијски инертан.
- Састојци у мешавини агрегата садрже минималне нежељене примесе, као што су алкалије, хлориди, сулфати и муљ.

Трајност бетона зависи од следећих фактора:

- *Садржај цемента*: мешавина мора бити пројектована тако да обезбеди кохезију и спречи сегрегацију и издвајање воде. Ако се количина цемента смањи, онда ће при константном водоцементном односу (mv/mc) обрадивост бити смањена што доводи до неадекватног збијања. Међутим, ако се дода вода ради побољшања обрадивости, однос (mv/mc) се повећава и резултира порознијим бетоном, односно високопропусним материјалом.
- *Збијање*: бетон може садржати нежељене поре и шупљине, које најчешће настају услед неадекватног збијања. Обично се поступак комактирања свежег бетона бира у складу са расположивом опремом за збијање, димензијама бетонског елемента, врсти оплате и распореду арматуре.
- *Очвршћавање*: веома је важно да се омогући одговарајуће несметано очвршћавање бетона, за шта је потребно обезбедити довољну количину влаге, како би се процес хидратације обавио у потпуности.

– *Дебљина заштитног слоја бетона*: Дебљина заштитног слоја бетона мора бити у складу са препорукама за пројектовање, које су дате у одговарајућим стандардима/правилницима.

– *Пропустљивост*: сматра се најважнијим фактором трајности. Може се приметити да је већа пропустљивост директна последица повећања порозности цементног камена. Стога, правилно очвршћавање, довољна количина цемента, правилно збијање и одговарајући заштитни слој бетона могу обезбедити бетон ниске пропустљивости.

Постоји много врста трајности, али главне врсте трајности бетона су:

– Физичка трајност,

– Хемијска трајност и

– Биолошка трајност.

Физичка трајност се обезбеђује преко:

– Отпорности на циклусе замрзавања и одмрзавања,

– Смањене филтрације и пропустљивости воде и

– Отпорности на термичка напрезања (нпр. услед високе топлоте хидратације).

Хемијска трајност се обезбеђује преко отпорности на:

– Алкално-агрегатну реакцију,

– Дејство сулфата,

– Продор хлорида,

– Одложено формирање етрингита,

– Корозију арматуре.

Биолошка трајност је отпорност бетона на деловање живих организама као што су, на пример, биљке, сунђери, шкољке, маховине и лишајеви.

Узроци смањења трајности бетона класификују се на спољашње и унутрашње.

Спољни узроци су:

– Екстремни временски услови,

– Екстремна температура,

– Екстремна влажност,

– Абразија,

– Дејство електролита и

– Дејство природних или индустријских течности или гасова.

Унутрашњи узроци су:

– Физички

  ▪ Запреминске промене услед разлике у термичким особинама агрегата и цементне пасте,

  ▪ Замрзавање и одмрзавање

– Хемијски

- Алкално агрегатне реакције (алкално-силиката реакција и алкално-карбонатна реакција)
- Корозија челика

Постоје две кровне дефиниције детериорације бетона:

- физичка манифестација оштећења материјала (на пример, пуцање, раслојавање, љускање, крњење, отпадање слојева бетона, мрље итд.) узрокована спољашњим или унутрашњим аутогеним утицајима на очврсли бетон, као и на друге материјале;
- распадање материјала током тестирања или излагања агресивним утицајима током коришћења, или промене у боји, текстури, чврстоћи, хемијском саставу или других својстава природног или вештачког материјала услед дејства временских услова.

Одржавање мостова:

Објекти саобраћајне инфраструктуре су од есенцијалног значаја за континуирани развој економског и друштвеног благостања сваке земље. Одржавање оваквих објеката у оптималном стању у сваком тренутку је тежак задатак за органе служби. Да би се помогло у обављању таквог задатка, развијени су системи за управљање инфраструктуром (IMS) за ефикасно управљање имовином. Основни задатак ових система је да минимизирају укупне оперативне трошкове за сервисне службе, а да при томе максимизирају користи за јавне кориснике.

Да би се добила права одлука применом IMS-a, неопходно је имати висококвалитетне информације за различите аналитичке процесе система. Да би IMS исправно предвидео будуће потребе за одржавањем и поправкама, записи о периодичним прегледима су кључни ресурси између осталих неопходних информација. Међутим, многи инфраструктурни објекти, првенствено мостови, су већ постојали много пре него што је развијена IMS технологија. Због тога недостају евиденције о претходним прегледима мостова. Недостатак таквих историјских записа који су потребни као улазни подаци за IMS је веома чест оперативни проблем у њиховој имплементацији.

Међу различитим ИМС-овима, за примену у овом истраживању одабран је систем управљања мостовима (BMS), јер су мостови најважнији објекти у саобраћајној мрежи. BMS такође захтева различите врсте података за рад. Најзначајнији BMS подаци су записи о претходним прегледима мостова, јер без ових података систем није у могућности да прецизно предвиди будући статус моста, укључујући и његову трајност. Као што је дефинисано у BIM приручнику, „Систем инспекције и одржавања мостова (BIM) је свеобухватан систем управљања ресурсима са могућношћу обраде података о прегледу мостова и осталих информација потребних за програме одржавања, планирање буџета, стратешком планирању и планирању животног циклуса, тако да се оптимизује безбедност путника и улагање у конструкције мостова”.

Од средине 1980-их, многи истраживачи су спровели опсежне студије о процени трајности конструкција моста кроз фазе истраживања од материјала до компоненти конструкције. Такве студије су увелеу примену многе методе процене, укључујући метод вероватноће, вештачку неуронску мрежу (ANN), аналитички хијерархијски процес (AHP) итд.

Мостови су током свог радног века изложени разним механичким, физичким, хемијским и биолошким утицајима који убрзавају процес пропадања, угрожавају функционалност и смањују њихову трајност. У случају пројектовања нових мостова, наведени проблеми се решавају адекватним пројектом (performance based design), док се за постојеће мостове развијају адекватне стратегије одржавања (BMS).

Одржавање мостова је обично ограничено на текуће радове које систематски изводе службе за одржавање како би се обезбедило нормално и безбедно коришћење мостовских конструкција. Ови радови се углавном састоје од прегледа, одржавања, поправке и замене, ако је потребно, дилатационих спојева, коловозних плоча, дренажног система, ограда и баријера, лежишта моста и др., као и антикорозивне заштите појединих елемената, углавном фарбањем.

Појам одржавања може се, шире, посматрати и као: вишекомпонентни процес који води ка испуњењу свих услова везаних за безбедно коришћење постојећих мостова у предвиђеном периоду њиховог будућег рада.

У последње две деценије, брзо пропадање мостовских конструкција постало је озбиљан технички и економски проблем у многим земљама, како високоразвијеним, тако и ниско развијеним. То се посебно односи и на бетонске мостове, који су се дуги низ година сматрали трајним и са минималним трошковима за одржавање, док су само челичне конструкције захтевале антикорозивну заштиту која се примењује сваких неколико година. Овакав став је довео до озбиљне детериорације (пропадања) постојећих бетонских мостова.

Главни разлози за убрзано пропадање бетонских мостова су:

- повећање интензитета саобраћаја и тежине возила, посебно њихових осовинских оптерећења, у односу на период када су мостови пројектовани и изграђени,
- штетан утицај загађења животне средине, посебно атмосферског, на перформансе конструкцијских материјала (CO_2, SO_2, HCl , H_2S итд.)
- уобичајена употреба средстава за одмрзавање у земљама умерене и оштре климе,
- конструкцијски материјали лошег квалитета, као и елементи мостовске опреме, као што су дилатационе направе, хидроизолационе мембране итд.,
- ограничен програм одржавања или недовољан стандард одржавања,
- лоша конструкцијска и материјална решења посебно осетљива на оштећења изазвана саобраћајним оптерећењем и факторима средине.

Историја управљања мостовима започела је касних шездесетих година 20. века, након урушавања мостова у USA. Године 1967. срушио се Сребрни мост између Point Pleasant-a, WV и Callipolis-a, OH. Затим је 28. јуна 1983. године урушио се део моста на реци Mianus услед тренутног лома у споју гредних носача. Епилог оваг инцидента је неколико смртних случајева и поремћаја саобраћаја на североистоку САД на неколико месеци. Тада нису постојали програми систематског одржавања за праћење стања мостовске мреже.

Да би решила овај проблем, Федерална управа за аутопутеве (FHWA)) је креирала национални програм инспекције мостова (NBIP), који је наложио свакој држави да каталогизира и прати стање мостова на главним аутопутевима. Подаци прикупљени као део MBIP-а достављани су након сваког периода инспекције и одржавани су од стране FHWA у бази података националног инвентара мостова (NBI). Намера је била да се поправе мостови пре него што детериорација достигне критично стање. Од 1980-их, интересовање за развој BMS-а је порасло и на државном и на савезном нивоу. 1985. године, национални кооперативни програм истраживања аутопутева (NCHRP) покренуо је програм са циљем развоја модела за ефикасан BMS. Крајем 1980-их, FHWA је уз подршку неколико државних одељења за транспорт спонзорисала развој Pontis система (Pontis, 2001). Године 1991. Закон о ефикасности интермодалног површинског транспорта (ISTA) препознао је потребу за превентивним одржавањем инфраструктуре. ISTA је дала мандат сваком државном одељењу за транспорт (DOT) да имплементира BMS који максимално користи ресурсе за планирање одржавања.

После USA и друге развијене земље (Аустралија, Канада, Јапан, Француска, Немачка, Швајцарска, итд.) успоставиле су сопствени BMS или BIM.

Либијски транспортни ресори троше знатну количину средстава да би мостови функционисали на ефикасан начин. Агенције одговорне за мостове у Либији немају процедуру за рангирање мостова за одржавање, рехабилитацију и замену.

Оправданост истраживања

Прегледом цитиране литературе дошло се до закључка да је проблем одржавања и санације мостова питање од великог значаја, посебно у развијеним земљама, као што су Цједињене Америчке Државе, Канада, Јапан, Аустралија, Европска Унија итд. Остале земље, због значаја мостова у путној инфраструктури, такође покушавају да прате ове трендове. У циљу ефикаснијег одржавања мостова, посебно за планирање финансијских средстава за обезбеђивање њихове функционалности и безбедности, развијени су бројни експертски системи. Такође, уочено је да постоји мали број студија, у којима је анализирана применљивост постојећих БМС за мостове у топлим климатима. Системи управљања мостом засновани су на анализи података добијених прегледом и проценом појединих елемената моста. Да би се добили поуздани подаци за ову анализу морају се утврдити механизми детериорације јер зависе од климатских подручја.

Стога ће анализа стања либијских бетонских мостова помоћи да се дефинишу типични дефекти и оштећења, узимајући у обзир утицај топле климе. На овај начин ће се добити поуздани подаци који ће, уз остале неопходне информације (значај моста у обиму путне мреже, расположива средства за реконструкцију и др.), дати тачно рангирање мостова у погледу оптималног одржавање путне мреже.

Предмет истраживања

Предмет овог истраживања су седам бетонских мостова - надвожњака у Триполију који су изграђени пре више од 50 година.

Триполи има суптропску степску/полусушну топлу климу (Koppen-Geigerova класификација: BSh). Просечна годишња температура је 20,3^{0}C.

За Триполи је карактеристично просечно 251 мм падавина годишње, односно 20,9 мм месечно. У просеку има 43 дана годишње са више од 0,1 мм падавина или 3,6 дана са кишом, суснежицом или снегом месечно. Најсушније време је у јулу када у просеку падне 0 мм падавина. Највлажније време је у децембру када у просеку падне 74 мм падавина.

Просечна брзина ветра у Триполију има благе сезонске варијације током године. Ветровитији део године траје 195 дана, од 10. новембра до 24. маја, са просечном брзином ветра већом од 5,9 миља на сат. Најветровитији дани у години су у децембру, са просечном брзином ветра од 7,0 миља на сат.

Мирније доба године траје 170 дана, од 24. маја до 10. новембра. Најмирнији дани су у августу, са просечном брзином ветра од 4,9 миља на сат. Преовлађујући просечан смер ветра у Триполију варира током целе године.

Када се разматра трајност мостова у Триполију, мора се узети у обзир утицај локалне климе на процесе детериорације. Као што је раније речено, Триполи спада у регион са суптропском степском климом, па нема потребе за анализом/проучавањем свих набројани процеса који нарушавају трајност бетонских конструкција.

Најважнији узроци детриорације бетонских мостова су:

- Пропустљивост и транспортни процеси
- Корозија арматуре у бетону
- Карбонизација
- Продор хлорида
- Хемијска агресија: сулфати.

Интеракција између услова околине, својстава материјала и конструкцијских фактора мора се узети у обзир када се оцењује стање грађевинских конструкција. Веома често више механизам детериорације делују истовремено, а детаљна анализа уочених оштећења је од посебног значаја да би се идентификовали сви узроци који су допринели појави оштећења. Многе материје – физичке, хемијске и биолошке природе – које потичу из окружења, одговорне су за појаву оштећења и материјала и елемената конструкција. Највећи „непријатељ“ материјала и елемената конструкција је вода у облику течности, паре или леда. Вода је носилац је штетних материја, ствара услове за хемијске процесе и одржава биолошка дејства. Никада не треба потцењивати улогу влаге у пропадању грађевинског материјала, а први корак у процесу санације обично би био отклањање услова који су дозвољавали продирање влаге у грађевинске елементе.

На одабраним мостовима спровене су следеће активности:

- визуелни преглед видљивих делова мостова пре и после санације ради утврђивања дефеката и оштећења, као и ефикасности примењених мера санације,

- Контрола квалитета уграђених материјала пре и после санације,

– Процена трајности, носивости, стабилности и употребљивости мостова пре и после санације.

Добијени резултати су коришћени у даљим анализама које се односе на:

– утврђивање карактеристичних дефеката и оштећења АБ мостова - надвожњака у топлим колиматима,
– рангирање мостова,
– Каталог карактеристичних оштећења армиранобетонских елемената маостова у топлим климатима.

Хипотезе истраживања

Истраживање се заснива на следећим хипотезама:

– Основни климатски параметри (сезонске температуре, влажност ваздуха, брзина ветра, просечне падавине, итд.) имају велики утицај на врсту оштећења која се појављују на армиранобетонским мостовима током њиховог животног века,
– Прецизно дефинисани врста и степен оштећења, као узроци њиховог настанка, су од изузетне важности за поуздано рангирање мостова применом система за управљање мостовима (BMS),
– Ранг мостова у великој мери зависи од примењеног BMS-a

садржај докторске тезе

Докторска дисертација је структуирана кроз следећа поглавља:

1. Увод
2. Трајност бетона (теоријска разматрања)
3. Испитивање бетона уграђеног у конструкције
4. Преглед мостова и системи за управљање мостовима (БМС)
5. Преглед стања у области трајности и одржавања бетонских мостова
6. Процена стања седам мостова у Триполију пре санације
7. Рејтинг и рангирање мостова пре санације
8. Санационе мере за седам мостова у Триполију
9. Контролни преглед седам мостова у Триполију 6 година након санације
10. Рејтинг и рангирање мостова након санације
11. Анализа и дискусија
12. Закључна разматрања
13. Научни допринос и правци даљег истраживања
14. Литература

Студија случаја

Студија случаја је обухватила седам армирано-бетонских мостова (надвожњака) у Триполију, који су изграђени у исто време (педесете године двадесетог века). Сви анализирани мостови налазе се на главним градским саобраћајницама.

Мостови имају различите конструктивне системе. Пуне дужине мостова су у распону од 17 до 54 m. Процена стања одабраних мостова је урађена пре санације и 6 година након санације За процену стања мостова коришћени су подаци добијени детаљним визуелним прегледом и теренским испитивањима. Детаљним визуелним прегледом су, поред носећих елемената АБ конструкције, обухваћени и системи за одвођење воде, заштитне ограде, вертикална и хоризонтална сигнализација и сви остали елементи, који су битни за безбедно одвијање саобраћаја на мосту и испод њега. За теренска испитивања бетона уграђеног у елементе носеће конструкције мостова пре санације коришћене су стандардне методе испитивања без разарања и са делимичним разрањем структуре материјала, како би се добили поуздани подаци за процену мостова. Одабране су следеће методе:

- Мерење дубине карбонизације,
- Садржај хлоридних јона,
- Испитивање бетона узимањем-вађењем језгара,
- Испитивање бетона на лицу места мерењем индекса склерометра и
- Испитивање бетона методом Pull-off.

Теремска испитивања 6 година након санације су обухватила мерење дубине карбонатизације.

Сви прикупљени подаци су коришћени и за анализу (БМС) за потребе дефинисања рејтинга мостова. Евалуација стања седам мостова у Триполију урађена је у складу са немачком БМС методологијом. Сва претходно регистрована оштећења, разврстана су по деловима моста, којих према немачком БМС-у може да буде највише 14. Сваки део моста са припадајућим оштећењима, анализиран је са аспекта утицаја на стабилност/носивост конструкције, безбедност саобраћаја и трајност. Рејтинг мостова је урађен на бази података прикупљених у току процене стања пре санације и 6 година након санације.

Анализирани су следећи мостови:

- Souk Athulatha 1 (лучни мост укупне дужине 39m, главног распона 22,4m);
- Souk Athulatha 2 (лучни мост укупне дужине 39m, главног распона 27,6m);
- Alsseka Road (лучни мост укупне дужине 40,1m, главног распона 28,5m);
- Bab Bin Gheshir Road (плочасти мост укупне дужине 54,3m, главног распона 21,7m);
- Al Sreem Road (гредни мост укупне дужине 21,1m, главног распона 2x9,7m);
- Alshaab Port (гредни мост укупне дужине 17,4m, главног распона 13,4m);
- Abdul Salam Aref (гредни мост укупне дужине 19,7m, главног распона 8,5m).

Ови мостови су одабрани за проучавање у докторској тези јер су сви мостови од армираног бетона, изложени сличним саобраћајним оптерећењима и у истом климатском окружењу, што је омогућило примену компаративне анализе у циљу дефинисања карактеристичних оштећења.

У наставку текста за свих седама мостова укратко су приказани најважнији закључци који су изведени на основу прикупљених података у току визуелних прегледа, као и рејтинзи мостова, пре и 6 година након санације.

Мост Souk Athulatha 1 је био стар око 50 година када је први пут прегледан. Закључци процене стања овог моста пре санације су:

- Карактеристичан дефект лучних плоча и бочних греда је недовољна дебљина заштитног слоја. Овај дефект је безуспешно "решен" малтерисањем видљивих површина обичним цементним малтером.
- Карактеристично оштећење је корозија арматуре и пратеће пуцање и отпадање заштитног слоја. Главни узрок појаве оштећења је карбонизација бетона. Скоро сви прегледани елементи имали су проблем са карбонизацијом, посебно доња страна лучних плоча. У неким случајевима, фронт карбонизације је чак прошао иза шипки арматуре.
- Други узрок појаве описаних оштећења је неадекватна дренажа воде са коловозне плоче. Овај проблем је проузроковао цурење воде кроз дилатације и преливање воде преко ивице конзолних плоча. Као последица тога, дошло је до корозије шипки арматуре на доњој страни лучних плоча и конзолним плочама.
- Анализом чврстоће при притиску бетона која је одређена на језгрима може се уочити да је разлика између минималне и максималне вредности за сваки испитивани елемент велика и варира од 12 до 22MPa. Ово је довело до закључка да је уграђени бетон веома неуједначеног квалитета и да се чврстоћа при притиску разликује од локације до локације.
- Резултати чврстоће бетона при притиску добијени методом склерометра показују веома велику дисперзију за сваки анализирани елемент конструкције.
- На основу резултата добијених методом Pull-off може се закључити да је чврстоћа бетона на затезање веома ниска и мања од минималне захтеване вредности.
- Садржај хлорида у бетону у конструкцији моста није опасан за уграђене арматурне шипке.

Главни закључак са аспекта трајности, носивости, стабилности и употебљивости гласи:

- Трајност свих конструктивних елемената је смањена, због бројних оштећења која су настала у протеклом времену.
- Носивост конструктивних елемената није смањена јер нема озбиљних оштећења или деформација АБ елемената.
- Глобална стабилност и стабилност сваког елемента конструкције нису угрожени.
- Функционалност моста је делимично смањена, због оштећења површинских слојева асфалта и локалне нестабилности испуцалих и одвојених бетонских комада, на доњој страни лучних плоче, бочних греда, конзолних плоча и ивичних греда.

Рејтинг моста Souk Athulatha 1 пре санације је износио 2,8.

Закључци процене стања овог моста 6 година након санације су:

- Карактеристично оштећење доње стране бочних греда и плоче моста су мрежасте пукотине настале услед сушења репаратурног малтера, док је карактеристично оштећење ослоначких зидова љускање површинског заштитног премаза.
- Карбонизација је већ почела на доњој страни мостовске плоче и ослоначким зидовима. Највећа дубина карбонизације измерена је на ослоначком зиду и износи 5мм.
- Трајност, носивост стабилност и употребљивост нису угрожени.

Рејтинг моста Souk Athulatha 1 6 година након санације је износио 2,2.

Мост Souk Athulatha 2 је такође био стар око 50 година пре првог визуелног прегледа и на њему су регистровани исти дефекти и оштећења. Узроци њиховог настанка су описани код претходног моста.

Анализом резултата испитивања чврстоће бетона при притиску која је добијена испитивањем бетонских језгара може се видети да је разлика између минималне и максималне вредности за сваки испитивани елемент велика и варира од 14 до 21 MPa тј. уграђени бетон има велику дисперзију квалитета од веома високог (~47 MPa) до веома ниског (~16 MPa). Чврстоћа на притисак разликује од локације до локације.

Резултати чврстоће бетона при притиску, добијени методом склерометра показују умерену дисперзију за сваки анализирани елемент конструкције, осим унутрашњег зида за који бетон показује велику дисперзију.

На основу резултата добијених методом Pull-off може се закључити да је чврстоћа бетона на затезање веома ниска и мања од минималне захтеване вредности.

Садржај хлорида у бетону у конструкцији моста није опасан за уграђене арматурне шипке.

Главни закључак са аспекта трајности, носивости, стабилности и употебљивости гласи:

- Трајност свих конструктивних елемената је смањена, због бројних оштећења која су настала у протеклом времену.
- Носивост конструктивних елемената није смањена јер нема озбиљних оштећења или деформација АБ елемената.
- Глобална стабилност и стабилност сваког елемента конструкције нису угрожени.
- Функционалност моста је делимично смањена, због оштећења површинских слојева асфалта и локалне нестабилности испуцалих и одвојених бетонских комада, на доњој страни лучних плоче, бочних греда, конзолних плоча и ивичних греда.

Рејтинг моста Souk Athulatha 2 пре санације је износио 2,8.

Закључци процене стања овог моста 6 година након санације су:

- Карактеристично оштећење доње стране бочних греда и плоче моста су мрежасте пукотине настале услед сушења репаратурног малтера, док је карактеристично оштећење ослоначких зидова љускање површинског заштитног премаза.

- Карбонизација је већ почела на доњој страни мостовске плоче и ослоначким зидовима. Највећа дубина карбонизације измерена је на доњој страни мостовске плоче и износи 12.5мм.
- Трајност, носивост стабилност и употребљивост нису још увек угрожени.

Рејтинг моста Souk Athulatha 2 6 година након санације је износио 2,1.

Мост Alsseka Road је био стар око 50 година када је први пут прегледан.

Закључци процене стања овог моста пре санације су:

- Карактеристичан дефект лучних плоча и бочних греда је недовољна дебљина заштитног слоја бетона.
- Карактеристично оштећење је корозија арматуре и пратеће пуцање и отпадањезаштитног слоја бетона. Главни узрок појаве оштећења је карбонизација. На свим прегледаним елементима регистрована је карбонизацијачија је дубина варирала је од 10 mm до 90 mm. Највеће дубине карбонатизације су добијене код унутрашњих зидова и доње површине лучних плоча где је фронт карбонизације прошао иза шипки арматуре.
- Други узрок појаве оштећења је неадекватна дренажа воде са коловозне плоче.
- Најоштећенији елемент је јужна бочна греда, која је претрпела механичка оштећења због удара камиона. Уздужне шипке су се деформисале и изгубиле атхезију са бетоном, прекинуто је неколико узенгија, а танки заштитни слој бетона је испуцао и отпао.
- Разлика између минималне и максималне вредности чврстоће при притиску испитаних на бетонским језгрима је изразито велика. То је довело до закључка да је уграђени бетон веома неуједначаног квалитета. Добијена вредност чврстоће бетона при притиску је мала (~22MPa).
- Резултати добијени методом склерометра за унутрашње зидове су исувише ниски за армирани бетон.
- На основу резултата добијених методом Pull-off може се закључити да је затезна чврстоћа бетона веома ниска и мања од минималне захтеване вредности.
- Садржај хлорида у бетону уграђеном у конструкцију моста није опасан за арматурне шипке.
- Просечна вредност запреминске масе очврслог бетона је 2180 kg/m^3. Ова вредност је мања од очекиване (~2300 kg/m^3) јер бетон није довољно збијен.

Рејтинг моста *Alsseka Road* пре санације је износио 2,9.

Закључци процене стања овог моста 6 година након санације су:

- Карактеристично оштећење доње стране бочних греда и плоче моста су мрежасте пукотине настале услед сушења репаратурног малтера, док је карактеристично оштећење ослоначких зидова љускање површинског заштитног премаза.

- Карбонизација је већ почела на доњој страни мостовске плоче и ослоначким зидовима. Највећа дубина карбонизације измерена је на доњој страни мостовске плоче и износи 12.5мм.
- Трајност, носивост стабилност и употребљивост нису још увек угрожени.

Рејтинг моста Alsseka Road 6 година након санације је износио 2,0.

Мост Bab Bin Ghashir је такође био стар око 50 година пре првог визуелног прегледа. Закључци изведени након визуелног прегледа пре санације су:

- Трајност свих конструктивних елемената је смањена, због бројних дефеката који су настали током изградње овог моста.
- Плоча моста је изведена са веома танким заштитним слојем бетона на доњој страни (~1cm)
- Уграђени бетон има малу чврстоћу на притисак (C16/20) и малу запреминску масу (~2180 kg/m^3)
- Карбонизација постоји у свим бетонским елементима.
- Носивост јужне бочне греде је угрожена због оштећења главних арматурних шипки.Носивост осталих конструктивних елемената није угрожена јер нема озбиљних пукотина или деформација АБ елемената.
- Глобална стабилност и стабилност сваког елемента конструкције нису угрожени
- Функционалност моста је делимично смањена, због оштећења површинских слојева асфалта и локалне нестабилности раслојаних бетонских комада који су настали на доњој страни плафонске плоче, бочних греда конзолних плоча и ивичних греда.

Рејтинг моста Bab Bin Ghashir пре санације је износио 2,9.

Закључци процене стања овог моста 6 година након санације су:

- Најоштећенији елемент је бочна греда на којој је регистровано озбиљно механичко оштећење и подужна пукотина на њеној доњој страни. На месту механичког оштећења бетон и репаратурни малтер су издробљени, главна арматура је деформисања, а узенгије прекините и деформисане. Описана оштећења су озбиљна јер смањују носивост и трајност бочне греде.
- Типична оштећења елемената доњег строја су мрежасте пукотине у репаратурном малтеру због скупљања приликом сушења.
- Карбонизација је већ почела на доњој страни мостовске плоче и ослоначким зидовима. Највећа дубина карбонизације измерена је на ослоначким зидовима и износи 18mm.
- Општа стабилност, носивост, функционалност и трајност још нису угрожени, али је могуће локално смањење носивости бочне греде на месту механичког удара.

Рејтинг моста Bab Bin Ghashir 6 година након санације је износио 2,2.

Мост Al Sreem је био стар око 50 година када је први пут прегледан. Закључци изведени након визуелног прегледа пре санације су:

- Главни узрок оштећења су удари камиона.

– Горњи строј овог моста је армиранобетонски, а доњи је зидани. Закључено је да је карбонизација карактеристична за АБ греде и АБ плочу горњег строја моста. У случају АБ греда, фронт карбонизације је прошао иза шипки арматуре

– Други узрок појаве оштећења је неадекватна дренажа воде са коловозне плоче.. Последично су се појавиле тамне и беле мрље на површинама ових елемената, као и љуштење малтера.

– Зидани конструктивни елементи немају пукотине или друге врсте озбиљних оштећења. Регистрована су само површинска оштећења у виду ломљења малтера и пуцања на унутрашњим зидовима) .

– Уграђени бетон има велику дисперзију квалитета од веома високог $\sim$39MPa до веома ниске $\sim$15МПа.

– На основу резултата добијених методом Pull-off може се закључити да је затезна чврстоћа бетона веома ниска и мања од минималне захтеване вредности.

– Садржај хлорида у бетону уграђеном у конструкцију моста није опасан за арматурне шипке.

– Запреминска маса очврслог бетона је близу 2300 kg/m^3, па се претпоставља да је уграђени бетон довољно збијен.

Главни закључак са аспекта трајности, носивости, стабилности и употебљивости гласи:

– Трајност свих АБ конструктивних елемената је смањена, због карбонизације бетона.

– Носивост неколико подужних греда у горњем строју моста је смањена јер је главна арматура деформисана и извијена. Такође, велики део бетонског попречног пресека греда је издробљен на истим локацијама.

– Глобална стабилност моста није угрожена и

– Функционалност моста је делимично смањена, због оштећења површинских слојева асфалта и локалне нестабилности издробљених бетонских комада која су настала на доњој страни греда плоча.

Рејтинг моста Al Sreem пре санације је износио 2,6.

Закључци процене стања овог моста 6 година након санације су:

– Карактеристично оштећење АБ греда у горњем строју моста су мрежасте пукотине настале услед сушења репаратурног малтера

– Карбонизација је већ почела на доњој страни мостовске плоче. Највећа дубина карбонизације износи 6 mm.

– Трајност, носивост стабилност и употребљивост нису још увек угрожени.

Рејтинг моста Al Sreem 6 година након санације је износио 2,3.

Мост Alshaab port је био стар око 50 година када је први пут прегледан. Закључци изведени након визуелног прегледа пре санације су:

– Карактеристично оштећење АБ елемената је корозија арматуре и пуцање и отпадање заштитног слоја бетона.

- Главни узрок појаве оштећења је недовољна дебљина заштитног слоја бетона. Измерена дебљина заштитног слоја бетона у елементима горњег строја моста (АБ греде и плоче) је само 5mm.
- Други узрок појаве оштећења је карбонизација бетона. Дубина карбонизације је варирала од 20 mm до 80 mm и на свим испитиваним местима фронт карбонизације је пролазио иза арматурних шипки.
- Следећи узрок појаве оштећења је неадекватна дренажа воде са коловозне плоче.
- Анализом чврстоће бетона при притиску добијене на језгрима може се уочити да је разлика између минималне и максималне вредности велика и варира од 19 до 44МПа. Уграђени бетон је веома неуједначеног квалитета.
- На основу резултата добијених методом Pull-off може се закључити да је затезна чврстоћа бетона веома ниска и мања од минималне захтеване вредности.
- Садржај хлорида у бетону уграђеном у конструкцију моста није опасан за арматурне шипке.

Главни закључак са аспекта трајности, носивости, стабилности и употебљивости гласи:

- Трајност свих елемената горњег строја моста је смањена, због бројних оштећења која су настала у протеклом времену.
- Носивост конструктивних елемената није смањена.
- Глобална стабилност и стабилност сваког елемента конструкције није угрожена.
- Функционалност моста је делимично смањена, због оштећења површинских слојева асфалта и локалне нестабилности одвојених бетонских комада, која су настали на доњим странама главних греда и конзолних плоча у горњем строју моста.

Рејтинг моста Alshaab пре санације је износио 2,6.

Закључци процене стања овог моста 6 година након санације су:

- Карактеристично оштећење доње стране бочних греда и плоче моста су мрежасте пукотине настале услед сушења репаратурног малтера, док је карактеристично оштећење ослоначких зидова љускање површинског заштитног премаза.
- У првобитном пројекту моста крајњи зидови су пројектовани као зидане камене конструкције, али су у пројекту санације овог моста пројектанти предложили ојачање ових зидова извођењем новог додатног АБ слоја. Карактеристична оштећења крајњих зидова су вертикалне и хоризонталне пукотине. Претпоставља се да је главни разлог за појаву вертикалних и хоризонталних пукотина на зидова корозија арматуре.
- Карактеристична оштећења подужних носећих греда су подужне пукотине. Појавиле су се на бочним и доњим површинама ових греда. Подужне пукотине су веома дугачке и понекад веома широке, а узроковане су корозијом арматурних шипки
- Карбонизација је већ почела на доњој страни мостовске плоче и ослоначким зидовима. Највећа дубина карбонизације и износи 4 mm.

- Носивост стабилност и употребљивост овог моста нису још увек угрожени. Будући да је процес корозије арматуре регистрован у свим АБ елементима, проблем носивости би могао бити врло брзо актуелан. Пошто је регистровано много прслина и пукотина, трајност целе конструкције је смањена.

Рејтинг моста Alshaab 6 година након санације је износио 2,2.

Мост Abdul Salam Aref је био стар око 50 година када је први пут прегледан. Закључци изведени након визуелног прегледа пре санације су:

- Сви прегледани елементи имали су проблем са карбонизацијом. Дубина карбонизације је варирала од 20 mm до 100 mm. Карбонизација је најизраженија код унутрашњих зидова и просечно износи 75 mm. На основу измерених резултата дубине карбонизације и положаја арматуре у АБ стубовима и АБ зидовима, може се закључити да фронт карбонизације није још увек стигао до арматуре, али се у неким случајевима приближио шипкама.
- Недовољна дебљина заштитног слоја је карактеристичан дефект за бочне стране ивичних греда, доњу страну главних греда горњег строја моста у и горње плоче моста.
- Карактеристично оштећење је пуцање, одвајање и отпадање заштитног слоја бетона у зони кородиралих шипки арматуре
- Главни узрок настанка оштећења, посебно на стубовима и опорцима је неадекватан одвод воде са коловозне плоче и са саобраћајних трака испод моста, као и неодржавање система за прикупљање и одвођење атмосферилија са и испод моста.
- Најоштећенији елементи су АБ стубови и опорци. Раслојавање и одвајање и отпадање бетона је захватило је велику површину стубова, посебно у угловима, као и велику површину опораца. Огољене шипке су у изгубиле адхезију са бетонским језгром. Приликом визуелног прегледа уочен је и неадекватан распоред узенгија у стубовима и неадекватан распоред хоризонталних и вертикалних арматурних шипки у опорцима.
- Уграђени бетон је веома неуједначеног квалитета јер се чврстоћа бетона при притиску, одређена на бетонском језгрима, мења не само од елемента до елемента, већ и по дубини истог елемента. Уграђени бетон има чврстоћу на притисак која одговара класама C20/25 до C25/30.
- Резултати мерења површинске тврдоће методом склерометра указују на веома лош квалитет бетона у површинском слоју на стубовима и опорцима.
- Просечна вредност запреминске масе очврслог бетона је 2323 kg/m3. Ова вредност одговара очекиваној вредности (~2300 kg/m3) и може се закључити да је уграђени бетон добро збијен.
- На основу резултата добијених методом Pull-off може се закључити да је затезна чврстоћа бетона веома ниска и мања од минималне захтеване вредности.
- Садржај хлорида у бетону уграђеном у конструкцију моста није опасан за арматурне шипке.

Главни закључак са аспекта трајности, носивости, стабилности и употебљивости гласи:

- Трајност свих конструктивних елемената је смањена, због бројних недостатака који су настали током изградње овог моста.
- Носивост потпорних стубова и опораца је смањена због смањења попречног пресека бетона и губитка адхезије између арматуре и околног бетона.
- Носивост осталих конструктивних елемената није угрожена.
- Глобална стабилност и стабилност сваког елемента конструкције нису угрожени
- Функционалност моста је делимично смањена, због оштећења површинских слојева асфалта и локалних поплава саобраћајних трака испод моста током обилних падавина.

Рејтинг моста Abdul Salam Aref пре санације је износио 2,9.

Закључци процене стања овог моста 6 година након санације су:

- АБ стубови и опорци су у одличном стању. Нису примећене пукотине, крућење бетона или отпадање заштитног слоја. На површини опораца регистроване су само врло мале зоне са љуштењем заштитне боје и неколико танких прслина.
- Сви видљиви делови греда и плоча горњег строја моста су у добром стању. Регистроване прслине су веома танке и још увек нису смањиле трајност тих елемената.
- Карбонизација је већ почела на свим испитиваним елементима моста Највећа дубина карбонизације је измерена на доњој страни мостовске плоче и износи 9 mm.
- Носивост, трајност, стабилност и употребљивост овог моста нису још увек угрожени. Сва оштећења су у почетном стању и могу се успорити неким мерама попут импрегнације. Исте мере се предлажу за АБ елементе захваћене карбонизацијом.

Рејтинг моста Abdul Salam Aref 6 година након санације је износио 2,3.

Закључци

Најважнији резултати истраживања, као и закључци спроведених анализа, дати су у наставку у оквиру следећих целина: процена стања мостова пре санације, процена стања мостова након санације, рејтинг и рангирање мостова пре и после санације.

Процена стања армиранобетонских мостова пре санације

Карбонатизација бетона је регистрована у свим елементима носеће конструкције, на којима је рађено мерење. Подједнако је изражена у елементима горње и доње конструкције моста. На више од 50% мерних места у горњој конструкцији моста и на око 40% мерних места у доњој кострукцији моста дубина карбонатизације је већа од 40mm. За њу је карактеристична и велика дисперзија резултата мерења (10mm-79mm), како у оквиру елемената једног моста, тако и између свих анализираних мостова. Основни узроци за већу брзину напредовања фронта карбонатизације су неповољни термохигрометријски услови, односно релативна влажност ваздуха у границама од 40-60%, релативно лош квалитет заштитног слоја бетона због високих температура, брзог исушивања и прекинуте хидратације.

Резултати испитивања чврстоће бетона при притиску (на језгрима извађеним из елемената конструкције мостова) указују на неуједначен квалитет бетона са аспекта механичких карактеристика. Чврстоће бетона при притиску су у границама од 22MPa до 38.5MPa. Бетон уграђен у елементе горњег строја мостова има чврстоћу бетона при притиску већу од 30MPa само код два моста од седам анализираних (< 30% укупног броја мостова). Када је у питању доњи строј мостова, код само једног моста од четири анализирана, остварена је чврстоћа бетона при притиску већа од 30MPa. Анализом свих резултата испитивања чврстоће бетона при притиску изведен је општи закључак да бетон уграђен у носећу конструкцију седам одабраних мостова има нижу чврстоћу при притиску од пројектоване.

Чврстоћа бетона на затезање, која је одређена применом pull-off методе, је мања од минималне захтеване вредности (1,5MPa) у свим испитиваним елементима одабраних мостова. Овај закључак указује да је у свим испитиваним мостовима површински слој бетона лошег квалитета. Главни узроци недовољне чврстоће бетона на затезање су недовољна и неадекватна нега бетона у фази очвршћавања и/или употреба прашњавог агрегата.

Анализом резултата испитивања садржаја јона хлора у бетону закључено је да је садржај хлорида у бетону мањи од усвојеног критеријума и да не представља опасност за корозију уграђене арматуре.

Запреминска маса цементних бетона са природним агрегатом је уобичајено већа од 2300kg/m^3 за добро збијене бетоне, Резултати испитивања запреминске масе бетона уграђеног у конструкцију седам одабраних мостова, налазе се у границама од 2111 до 2356kg/m^3. Средња вредност запреминске масе за бетон уграђен у горњу конструкцију мостова је 2245kg/m^3, док за бетон уграђен у доњи строј мостова 2269kg/m3. Закључено је да уграђени бетон има нешто нижу вредност запреминске масе од 2300kg/m^3.

Дебљина заштитног слоја бетона измерена је на бетонским језгрима која су извађена из елемената носеће конструкције мостова. Дебљина заштитног слоја бетона је променљива и креће се од 0mm до чак 100mm, што указује да постављању арматуре није посвећено довољно пажње и да у неким елементима нису коришћени дистанцери за арматуру или арматурни кош није био довољно причвршћен. На елементима горњег строја мостова дебљина заштитног слоја бетона је у границама од 0mm до 70mm, а најчешће је од 10mm до 20mm. У елементима доњег строја дебљина заштитног слоја бетона је од 20mm до 100mm и без обзира на велики распон измерених дебљина, просечна вредност износи 35mm.

Корозија арматуре је регистрована и у елементима горњег строја и у елементима доњег строја конструкције и представља карактеристично оштећење мостова. По интензитету корозија се креће од површинске до јаке са листањем челика и нарушеном атхезијом са околним бетоном. Најчешће су корозијом захваћене арматурне шипке уграђене у доњу зону мостовских плоча и греда, као и у конзолне делове мостова. Иако у мањем обиму, корозија арматуре је регистрована и на стубовима, крајњим и унутрашњим ослоначким зидовима. Корозија арматуре у елементима горњег строја је узрокована недовољном дебљином заштитног слоја и узнапредовалом карбонатизацијом бетона.

Пуцање одвајање и отпадање бетона је најчешће оштећење бетона које настаје због повећане запремине продуката корозије шипки арматуре. На свих седам прегеданих мостова пуцањем, одвајањем и отпадањем бетона захваћен је заштитни слој бетона, а у појединим случајевима и зона кородиралих шипки арматуре – матрица пресека.

Процена стања армиранобетонских мостова 6 година после санације

Карбонатизација је регистрована на елементима горњег и доњег строја мостова. На доњој површини мостовских плоча дубина карбонатизације се креће од 1мм до 12.5мм, док је на елементима доњег строја измерена од 0 до 20мм. Ако се узме у обзир да је највећа очекивана вредност дубине карбонатизације за период од 5 година 5мм, лако се може закључити да карбонатизација напредује брже од претпостављене, иако су за репрофилацију коришћени фабрички произведени репаратурни материјали, а сви санирани елементи су додатно третирани заштитним премазима. Ова констатација још једном потврђује да је карбонатизација основни узрок нарушавања трајности АБ мостова у топлим климатима.

Карактеристична оштећења на доњој страни санираних мостовских плоча и попречних греда су мрежасте прслине, које су узроковане скупљањем услед сушења репаратурних малтера, који се по правилу наносе у танким слојевима. У појединим зонама на доњој страни мостовских плоча појавила су се и оштећења у виду пуцања, одвајања и отпадања танког површинског слоја репаратурног малтера. На носећим елементима доњег строја појавила су се само оштећења естетског карактера, у виду љускања завршног премаза.

Рејтинг и рангирање мостова пре санације

Рејтинг мостова је одређен према методологији која је предвиђена немачким БМС-ом. Рејтинг мостова пре санације се креће у границама од 2,6 – 2,9. За прорачун рејтинга свих седам мостова, меродавна су била оштећења на конструкцији горњег строја моста, а карактеристична оштећења су била корозија арматуре са или без редукције попречног пресека. На основу вредности рејтинга одређена је категорија оштећења моста као целине. Закључено је да свих седам мостова пре санације припадају истој категорији (2,5-2,9), која се описује као „довољна“ (sufficient condition) и за коју је у немачком БМС дат следећи опис:

- Осигурана је стабилност моста.
- Безбедност саобраћаја може бити нарушена.
- Стабилност и/или трајност најмање једне групе компоненти може бити нарушена.
- Трајност носеће конструкције моста може бити смањена. Може очекивати ширење оштећења конструкције, што у средњерочном довести до значајног угрожавања стабилности и/или безбедности саобраћаја.
- Потребно је текуће одржавање.
- Захтева се санација оштећења у кртакторочном периду.
- У краткорочном периоду могу бити потребне мере за отклањање оштећења или упозорења за одржавање безбедности на путевима.

Рејтинг и рангирање мостова 6 година након санације

Рејтинг мостова након санације се креће у границама од 2,0 – 2,3. За прорачун рејтинга свих седам мостова, меродавни су били недостатак вертиклане саобраћајне сигнализације или запушена атмосферска канализација. Закључено је да свих седам мостова након санације припадају истој категорији (2,0-2,4), која се описује као „задовољавајућа“ (satisfactory condition), а коју је у немачком БМС дат следећи опис:

– Обезбеђени су стабилност и безбедност саобраћаја конструкције.
– Стабилност и/или трајност најмање једне групе компоненти може бити нарушена.
– Трајност конструкције у дужем временском периоду може бити смањена. Могуће је ширење оштећења конструкције, што дугорочно доводи до значајног нарушавања стабилности и/или безбедности саобраћаја.
– Потребно је текуће одржавање.
– Захтева се санација оштећења у средњерочном периду.
– У краткорочном периоду могу бити потребне мере за отклањање оштећења или упозорења за одржавање безбедности на путевима

Оцена стања мостова која је урађена применом немачке БМС методологије, поклапа се са оценом стања мостова, која је изведена на основу анализе резултата детаљног визуелног прегледа мостова. Овај закључак указује на чињеницу да је коришћена немачка БМС методологија добро коципипирана, али да оцена стања у великој мери зависи од квалитета улазних података, првенствено података добијених визуелним прегледом мостова, за чије је прикупљање неопходно практично искуство и теоријско знање. За реализацију фазе визуелног прегледа, велику помоћ представљају „каталози карактеристичних оштећења“.

Главни допринос истраживања био је: идентификација дефеката и оштећења по елементима мостовских конструкција, типичних за топле климате; каталог типичних оштећења на елементима АБ мостова за поузданију процену мостова током визуелног прегледа и процена стања и прикупљање података о БМС-у и побољшање система одржавања мостова у Либији.

SUMMARY

Transportation infrastructure facilities are the essential components for the continuous development of the economy of any country. Maintaining such facilities and keeping their assets in optimal conditions at all times is a difficult task for service authorities. To assist with conducting such a task, infrastructure management systems (IMSs) have been developed for effective asset management. Among different IMSs, a bridge management system (BMS) was selected for implementation in this research, because bridges are the most important facilities in transportation network. BMS also requires various types of data for operation. The most significant BMS data are past bridge inspection records because without these data it is unable to accurately predict future bridge status, including its durability.

Durability assessment for existing reinforced concrete bridges, as well as their maintenance, has recently become a research topic in civil engineering. The study on the theory and application of durability assessment for Reinforced Concrete Bridge can reveal the potential risk in the structure and provide the correct information to make timely decisions for repairing, strengthening or removing bridges to avoid severe accidents. The problem of maintenance and rehabilitation of bridges is issue of high importance, especially in developed countries, such as USA, Canada, Japan, Australia, EU etc. Other countries are also trying to follow these trends. In order to obtain more efficient maintenance of bridges, especially for planning financial resources for ensuring their functionality and safety, many expert systems have been developed. Also, it was observed that there are a small number of studies, in which the applicability of existing BMSs for bridges in a hot climates has been analysed.

Therefore, the analysis of the condition of Libyan concrete bridges will help to define the typical defects and damages, taking into account the impact of the warm climate. In this manner, reliable data will be obtained, which will provide, together with other necessary information (the importance of the bridge in the scope of the road network, available funds for reconstruction, etc.), an accurate ranking of bridges in terms of optimal maintenance of the road network.

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INTRODUCTION

1. BACKGROUND

Civil infrastructure systems, especially roadways and bridges play essential roles in the economy of nations and their value in most countries is significant. In North America, for example, the total value of the infrastructure systems is estimated to be $33 trillion. The yearly average expenditure on the infrastructure system is estimated to be $303 billion in the United States (USA). Therefore, the sustained operation of these infrastructure assets is crucial. A large percentage of existing infrastructure assets are deteriorating due to age, harsh environmental conditions, and insufficient capacity [2].

Due to the large size and cost of infrastructure networks, maintaining such networks is a challenging but crucial task, particularly in light of the limited budgets available for infrastructure maintenance. Consequently, countries (municipalities and transportation agencies) are under increasing pressure to develop new strategies for managing public infrastructure assets in a way that ensures long-term sustainability under constrained budgets [2].

This problem is not specific just for countries with well-developed infrastructure networks, but also is very important for countries with poor network of roadways. In these countries, in a case of traffic interruption, a numerous problems appear because there is only one roadway between neighbouring cities, without alternative traffic ways. That’s why appropriate strategy for managing roadways and bridges is also very important for less developed countries.

Bridges are an integral infrastructure component and are of great importance for functioning of traffic, as such, have been the subject of extensive research efforts related to structural performance. However, there has been little study on the traffic safety performance of bridges, which have very different physical and operational characteristics [3].

2. NEED FOR RESEARCH

Description of problem

Durability of concrete

There are a lot of definitions for term durability of concrete. Some of them are:

- The ability of concrete to withstand the conditions for which it is designed without deterioration for a long period of years is known as durability.

- Durability of concrete is the ability of concrete to resist weathering action, chemical attack, and abrasion while maintaining its desired properties.
- Durability is defined as the capability of concrete to resist weathering action, chemical attack and abrasion while maintaining its desired engineering properties.

It normally refers to the duration or life span of trouble-free performance. Different concretes require different degrees of durability depending on the exposure environment and properties desired. For example, concrete exposed to tidal seawater will have different requirements than indoor concrete.

Concrete will remain durable if:

- The cement paste structure is dense and of low permeability.
- Under extreme condition, it has entrained air to resist freeze-thaw cycle.
- It is made with graded aggregate that are strong and inert.
- The ingredients in the mix contain minimum impurities such as alkalis, chlorides, sulphates and silt.

Durability of concrete depends upon the following factors [1]:

- Cement content: mix must be designed to ensure cohesion and prevent segregation and bleeding. If cement is reduced, then at fixed w/c ratio the workability will be reduced leading to inadequate compaction. However, if water is added to improve workability, water / cement ratio increases and resulting in highly permeable material.
- Compaction: the concrete as a whole contain voids can be caused by inadequate compaction. Usually it is being governed by the compaction equipments used, type of formworks, and density of the steelwork.
- Curing: it is very important to permit proper strength development aid moisture retention and to ensure hydration process occur completely.
- Cover: thickness of concrete cover must follow the limits set in codes.
- Permeability: is considered as the most important factor for durability. It can be noticed that higher permeability is usually caused by higher porosity. Therefore, a proper curing, sufficient cement, proper compaction and suitable concrete cover could provide a low permeability concrete.

There are many types of durability, but the major Concrete Durability types are:

- Physical durability,
- Chemical durability and
- Biological durability.

Physical durability is against the following actions

- Freezing and thawing action,
- Percolation / Permeability of water and
- Temperature stresses i.e. high heat of hydration.

Chemical durability is against the following actions:

- Alkali Aggregate Reaction,
- Sulphate Attack,
- Chloride Ingress,
- Delay Ettringite Formation,
- Corrosion of reinforcement.

Biological durability is against the actions of live organisms such as plants, sponges, boring shells, or marine borers, mosses and lichens.

Causes for the lack of durability in concrete classify on external and internal causes.

External causes are:

- Extreme Weathering Conditions
- Extreme Temperature
- Extreme Humidity
- Abrasion
- Electrolytic Action
- Attack by a natural or industrial liquids or gases

Internal causes are:

- Physical
  - Volume change due to difference in thermal properties of aggregates and cement paste
  - Frost Action
- Chemical
  - Alkali Aggregate Reactions (alkali silica reaction, alkali silicate reaction and alkali carbonate reaction
  - Corrosion of Steel

There are two umbrella definitions of concrete deterioration:

- physical manifestation of failure of a material (for example, cracking, delamination, flaking, pitting, scaling, spalling, and staining) caused by environmental or internal autogenous influences on rock and hardened concrete as well as other materials;

- decomposition of material during either testing or exposure to service, or changes in colour, texture, strength, chemical composition or other properties of a natural or artificial material due to the action of the weather.

Maintenance of bridges:

Transportation infrastructure facilities are the essential components for the continuous development of the economic and community well-being of any country. Maintaining such facilities and keeping their assets in optimal conditions at all times is a difficult task for service authorities. To assist with conducting such a task, infrastructure management systems (IMSs) have been developed for effective asset management. The main function of these systems is to minimise total operation costs for service authorities while maximising the benefits for public users.

To obtain the right decision from an IMS, it is necessary to have high quality asset information for the system′s various analytical processes. For an IMS to correctly predict a mixture of future maintenance and repair needs, periodic inspection records are the key resources amongst other information requirements. However, many infrastructure facilities were already in existence long before the IMS technology was developed. Thus many years of past inspection records for those structures was already lacking. In particular the lack of such historical records which are required as inputs to IMSs is a very common operational problem in their implementation [5].

Among different IMSs, a bridge management system (BMS) was selected for implementation in this research, because bridges are the most important facilities in transportation network. BMS also requires various types of data for operation. The most significant BMS data are past bridge inspection records because without these data it is unable to accurately predict future bridge status, including its durability. As defined in the BIM Manual, the “Bridge Inspection and Maintenance System (BIM) is a comprehensive inventory management system with the ability to process bridge inspection and component information for use in inspection management, maintenance programming, budget development, strategic planning, and life cycle planning so that the safety of the traveling public and the investment in bridge structures is optimized”.

Durability assessment for existing reinforced concrete bridges, as well as their maintenance, has recently become a research topic in civil engineering. The study on the theory and application of durability assessment for reinforced concrete bridge can reveal the potential risk in the structure and provide the correct information to make timely decisions for repairing, strengthening or removing bridges to avoid severe accidents [4]. In addition, research results in this field can be directly applied to guide the design and repair of bridges to extend their service life and minimize repair and strengthening costs.

Since the middle of the 1980s, many researchers have extensively conducted in-depth studies on the durability evaluation of bridge structure through research stages from material to component of structures. Such studies have provided many assessment methods, including Probability Method, Artificial Neural Network (ANN), Analytic Hierarchy Process (AHP), Grey Associated Analysis, and Comprehensive Weight Variation Evaluation [4].

During their service life bridges are subjected to a variety of mechanical, physical, chemical and biological influences, which accelerate the deterioration process, jeopardize the functionality and reduce their durability. In case of designing new bridges the above-mentioned problems are solved by an adequate design (performance based design), while for the existing bridges adequate maintenance strategies are developed (BMS).

The Term of Maintenance is usually limited to the current works performed systematically by maintenance services to ensure normal and safe utilization of bridge structures. These works consist mainly of inspection, maintenance, repair and replacement, if necessary, of expansion joints, bridge deck, drainage system, railings and barriers, pavement, bridge bearings, etc., as well as anti-corrosive protection of some elements, mostly by painting.

The term maintenance may also be considered, more widely, as: a multi-component process leading to the fulfilment of all conditions related to the safe utilization of existing bridges in the anticipated period of their future service.

In the recent two decades, rapid deterioration of bridge structures has become a serious technical and economic problem in many countries, including highly developed, as well as low developed ones. It concerns also the concrete bridges, which for many years have been considered as durable and requiring minimum maintenance cost, while only the steel structures demand anti- corrosive protection being applied every few years. These viewpoints led to serious deterioration of the existing concrete bridges.

Main reasons for accelerated deterioration of concrete bridges are:

- increase in traffic flows and weight of vehicles, especially their axle loads, compared to the period when the bridges have been designed and constructed,
- harmful influence of environmental pollution, especially atmospheric ones, on the performance of structural materials (CO_2, SO_2, HCl, H_2S etc.)
- common use of de-icing agents in countries of moderate climate,
- low quality structural materials as well as bridge equipment elements, such as expansion joints, waterproofing membranes, etc.,
- limited maintenance program or insufficient standard of maintenance,
- poor structural and material solutions particularly sensitive to damage produced by both traffic loads and environmental factors.

The history of bridge management started in the late sixties of the 20th century, after t collapses of bridges in USA. In 1967, the Silver Bridge between Point Pleasant, WV and Callipolis, OH collapsed. Then on June 28, 1983, a section of the Mianus River Bridge catastrophically failed due to the instantaneous fracture of a pin and hanger connection. This failure resulted in several fatalities and disrupted commerce in north-eastern USA for several months. No systematic maintenance programs were yet in place for monitoring the condition of bridge networks [2].

To address this problem, the Federal Highway Administration (FHWA) created the national bridge inspection program (NBIP), which ordered every state to catalogue and track the condition of bridges on principal highways. The data collected as part of the NBIP were submitted after each inspection period and maintained by the FHWA in the national bridge inventory (NBI) database. The intention was to repair bridges before deterioration reached a critical state. Since the 1980s, interest in the development of BMSs has increased at both the state and the federal levels. In 1985, the national cooperative highway research program (NCHRP) initiated a program with the objective of developing a model for an effective BMS. In the late 1980s, the FHWA with the support of several state departments of transportation sponsored the development of the Pontis system (Pontis, 2001). In 1991, the Intermodal Surface transportation efficiency act (ISTA) recognized the need for the preventive maintenance of infrastructure. ISTA mandated that each state department of transportation (DOT) to implement a BMS that maximizes the use of resources for maintenance planning [2].

After USA other developed countries (Australia, Canada, Japan, France, Germany, Switzerland, etc.) established own BMS or BIM. A short review of BMSs is given in Table 1.1[6].

3. JUSTIFICATION OF RESEARCH

By reviewing the cited literature it was concluded that the problem of maintenance and rehabilitation of bridges is issue of high importance, especially in developed countries, such as USA, Canada, Japan, Australia, EU etc. Other countries, because of the importance of bridges in the road infrastructure, are also trying to follow these trends. In order to obtain more efficient maintenance of bridges, especially for planning financial resources for ensuring their functionality and safety, many expert systems (Table I.1) have been developed. Also, it was observed that there are a small number of studies, in which the applicability of existing BMSs for bridges in warm climate have been analysed. Bridge management systems are based on analysis of data obtained by inspection and assessment of the individual bridge elements. In order to obtain reliable data for this analysis the mechanisms of deterioration must be determined because they depend on the climatic areas.

Therefore, the analysis of the condition of Libyan concrete bridges will help to define the typical defects and damages, taking into account the impact of the warm climate. In this manner, a reliable data will be obtained, which will provide, together with other necessary information (the importance of the bridge in scope of the road network, available funds for reconstruction, etc.), accurate ranking of bridges in terms of optimal maintenance of the road network.

Table I.1. Bridge Management Systems used worldwide

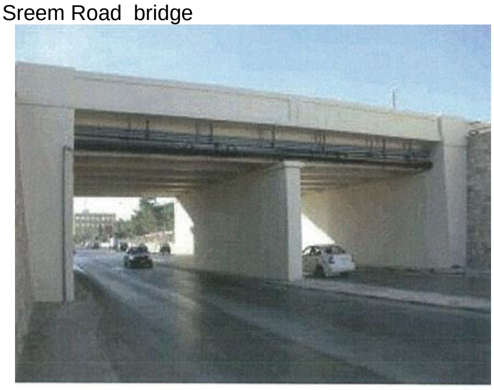
No.	Country	System Name	System abbreviation	First version
1	Canada	Ontario Bridge Management System	OBMS	2002
2	Canada	Quebec BMS	QBMS	2008
3	Canada	EBMS	EBMS	2006
4	Canada	PEIBMS	PEIBMS	2006
5	Denmark	DANBRO BMS	DANBRO	1975
6	Finland	The Finish BMS	FBMS	1990
7	France	Quality Image of Engineering Structures	IQUOA	1994
8	Germany	Bauwerk Management System	GBMS	N/A
9	Ireland	Eirspan	Eirspan	2001
10	Italy	Autonomous Province of Trento BMS	APTBMS	2004
11	Japan	Regional Planning Institute of Osaka BMS	RPIBMS	2006
12	Korea	Korea Road Maintenance Business System	KRBMS	2003
13	Latvia	Lat Brutus	Lat Brutus	2002
14	Netherland	DISK	DISK	1985
15	Poland	SMOK	SMOK	1997
16	Poland	SZOK	SZOK	2001
17	Spain	SGP	SGP	2005
18	Sweden	Bridge and Tunnel Management System	BaTMan	1987
19	Switzerland	KUBA	KUBA	1991
20	USA	Bridgit	Bridgit	1993
21	USA	Pontis	Pontis	1992
22	USA	AASHTO Ware Bridge Management software	BrM	Early 1990's
23	Vietnam	Bridgeman	Bridgeman	2001

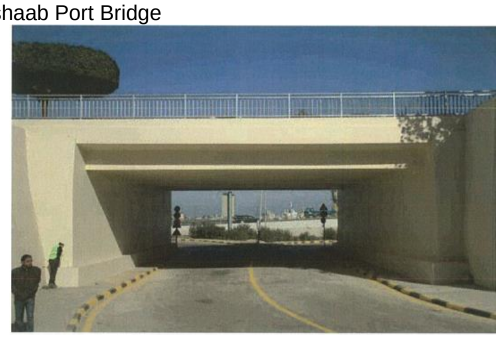
4. OBJECT OF RESEARCH

Object of this research are seven concrete bridges - overpasses in Tripoli that were constructed more than 50 years ago. Basic data of chosen bridges are given in Table I.2.

Table I.2: Basic data of chosen bridges

Type	Description	Name and appearance of the bridge
Simple Arch Bridge	Made of reinforced concrete. According to the style of construction this bridge is classified as semi-prefabricated, cantilever bridge, with two cantilever beams and prefabricated simple supported slab in the middle of span. This bridge has the same shape and dimensions like This bridge was built in the middle of XX centuries. Is located in the west part of the capital Tripoli, about 360 meters from the sea to the north. It is considered as one of major bridge to the capital Tripoli. It connects several main roads leading to the center of the capital. The coordinates for this bridge are 320 52'45.5" N 130 09'19.8"E.	Souk Athulatha 1 

Simple Arch Bridge	Made of reinforced concrete. This bridge was built in the middle of XX centuries. Is located in the west part of the capital Tripoli, about 360 meters from the sea to the north. It is considered as one of major bridge to the capital Tripoli. It connects several main roads leading to the center of the capital. The coordinates for this bridge are320 52'45.2" N 130 09'26.2"E.	Souk Athulatha 2 
Simple Arch Bridge	Made of reinforced concrete. This bridge was built in the middle of XX centuries. Is located in the east part of the capital Tripoli, about 2.66km from the sea to the north. It is considered as a major bridge to the capital Tripoli. It connects the city centre and the university and connects the roads leading to the collection of state institutions buildings. The coordinates for this bridge are 32052'22" N 130 11'55"E.	Alsseka bridge 
Overpas with three spans supported Bridge	Is an overpass with three spans supported by reinforced concrete support walls and abutments. This bridge was built in the middle of XX centuries. Is located in the east part of the capital Tripoli, about 2.66km from the sea to the north. It is considered as a major bridge to the capital Tripoli. It connects several main roads leading to the centre of the capital. The coordinates for this bridge are 320 52'22.2" N 130 11'45.1"E.	Bab Bin Gheshir road bridge 
Beam Bridge	Is two span beam bridge. It is also overpass with RC bridge superstructure which is supported by masonry bridge substructure. Masonry bridge substructure consists of one stone support wall and two stone abutments. All masonry elements were covered by plastering. RC bridge superstruc-ture consists of two decks which are supporting on longitudinal and transversal beams. This bridge was built in the middle of XX centuries. Is located in the south part of the capital Tripoli, about 2.21km from the sea to the north. It connects several main roads leading to the center of the capital and road to the airport. The coordinates for this bridge are: 320 53'03.3" N 130 10'29.5"E.	Al Sreem Road bridge 

Simple beam bridge	Is designed as simple beam bridge made of RC. This bridge was built located in the north part of the capital Tripoli, about 125.77 meters from the sea to the north. It connects several main roads leading to the centre of the capital. in the middle of XX centuries. The coordinates for this bridge are 320 53' 48.7" N 130 12' 02.3" E.	Alshaab Port Bridge 
I beam Bridge	Is designed as I beam bridge with a row of columns supporting the deck in the centre. Made of RC. This bridge was built in the middle of XX centuries. Is located in the west part of the capital Tripoli, about 567 meters from the sea to the north. It is considered as a major bridge to the capital Tripoli. It connects several main roads leading to the canter. Close to the rapid transit station. The coordinates of bridge are:320 53' 56.4" N 130 12' 47.0" E.	Adbassalam Aref Square Bridge 

Tripoli has a subtropical steppe/low-latitude semi-arid hot climate (Köppen-Geiger classification: BSh). The annual average temperature is 20.3^{0}C. Annual average temperatures chart is given in Fig I.1. and precipitation data in Table I.3.

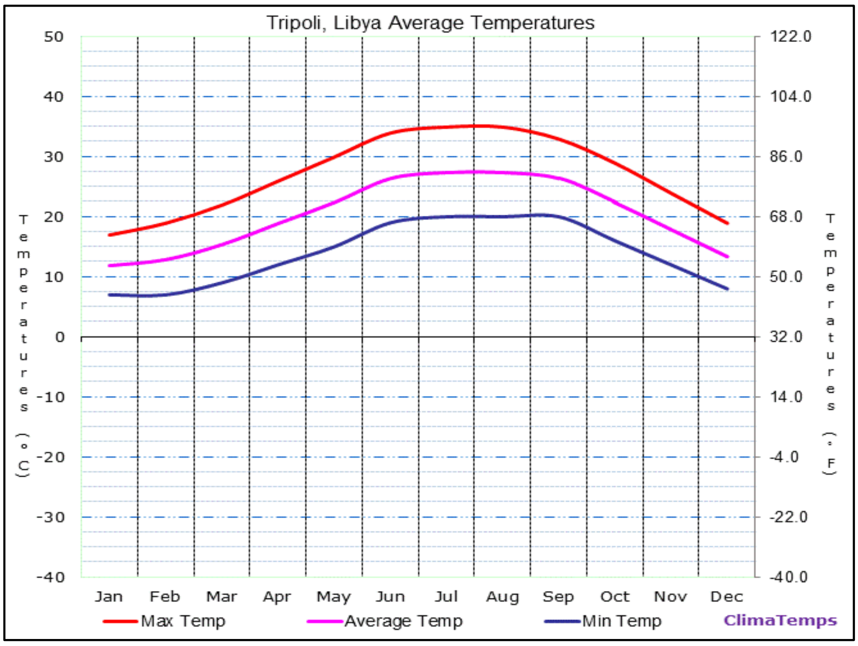


Figure I.1. Tripoli average temperatures chart

Table I.3. Precipitation data

		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec		Annual
	Average Precipitation mm (in)	46 (1.81)	27 (1.1)	12 (0.5)	16 (0.6)	2 (0.1)	1 (0)	0 (0)	1 (0)	6 (0.2)	27 (1.1)	39 (1.5)	74 (2.9)		251 (9.9)
	Precipitation Litres/m² (Gallons/ft²)	46 (1.13)	27 (0.66)	12 (0.29)	16 (0.39)	2 (0.05)	1 (0.02)	0 (0)	1 (0.02)	6 (0.15)	27 (0.66)	39 (0.96)	74 (1.82)		251 (6.16)
	Number of Wet Days (probability of rain on a day)	8 (26%)	5 (18%)	5 (16%)	3 (10%)	1 (3%)	0 (0%)	0 (0%)	0 (0%)	2 (7%)	5 (16%)	6 (20%)	8 (26%)		43 (12%)
	Percentage of Sunny (Cloudy) Daylight Hours	66 (34)	64 (36)	65 (35)	63 (37)	74 (26)	71 (29)	87 (13)	86 (14)	72 (28)	71 (29)	66 (34)	65 (35)		73 (2

Tripoli is entrusted with an average of 251 mm of rainfall per year, or 20.9 mm per month. On average there are 43 days per year with more than 0.1 mm of rainfall (precipitation) or 3.6 days with a quantity of rain, sleet, snow etc. per month. The driest weather is in July when an average of 0 mm of rainfall (precipitation) occurs. The wettest weather is in December when an average of 74 mm of rainfall (precipitation) occurs.

This section discusses the wide-area hourly average wind vector (speed and direction) at 10 meters above the ground. The wind experienced at any given location is highly dependent on local topography and other factors, and instantaneous wind speed and direction vary more widely than hourly averages.

The average hourly wind speed in Tripoli experiences mildly seasonal variation over the course of the year. The windier part of the year lasts for 195 days, from November 10 to May 24, with average wind speeds of more than 5.9 miles per hour. The windiest day of the year is December 23, with an average hourly wind speed of 7.0 miles per hour.

The calmer time of year lasts for 170 days, from May 24 to November 10. The calmest day of the year is August 5, with an average hourly wind speed of 4.9 miles per hour. The predominant average hourly wind direction in Tripoli varies throughout the year. The wind is most often from the north for 33 days, from March 13 to April 15; for 28 days, from July 10 to August 7; and for 14 days, from October 18 to November 1, with a peak percentage of 44% on July 26. The wind is most often from the east for 86 days, from April 15 to July 10 and for 72 days, from August 7 to October 18, with a peak percentage of 46% on June 10. The wind is most often from the west for 132 days, from November 1 to March 13, with a peak percentage of 56% on December 26.

The average of mean hourly wind speeds (dark gray line), with 25th to 75th and 10th to 90th percentile bands.

The average annual relative humidity is 57,4% and average monthly relative humidity ranges from 41% in June to 69% in December.

When we consider of durability of bridges in Tripoli, we have to take into a count effect of local climate on deterioration processes. As it is discussed before Tripoli belongs to the region with subtropical steppe climate, since no need to analyse/study all numbered processes that impair durability of concrete structures.

Most important causes of deterioration of concrete bridges are:

- Permeability and transport processes
- Corrosion of reinforcement in concrete
- Carbonation
- Chloride ingress
- Chemical attack: sulphates.

Interaction between environmental conditions, material properties and structural factors must constantly be considered when evaluating the failure of building structures. More often than one disruptive mechanism may operate and a detailed analysis of the affected structure is imperative to identify all the causes that contributed to the deterioration of the material.

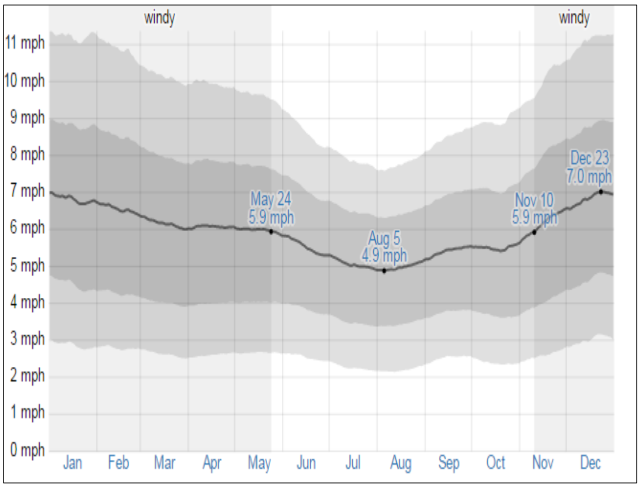


Figure I.2. Average wind speed

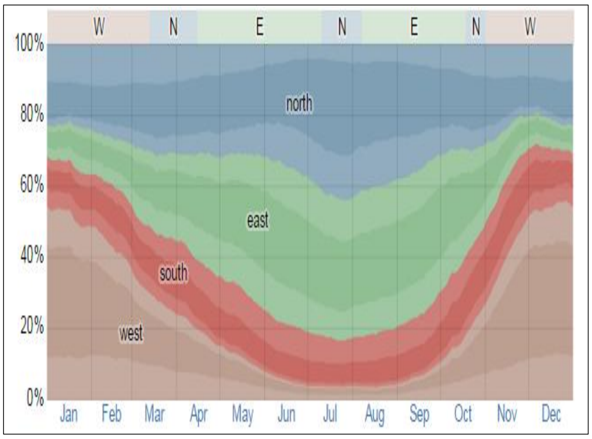


Figure I.3. Wind direction

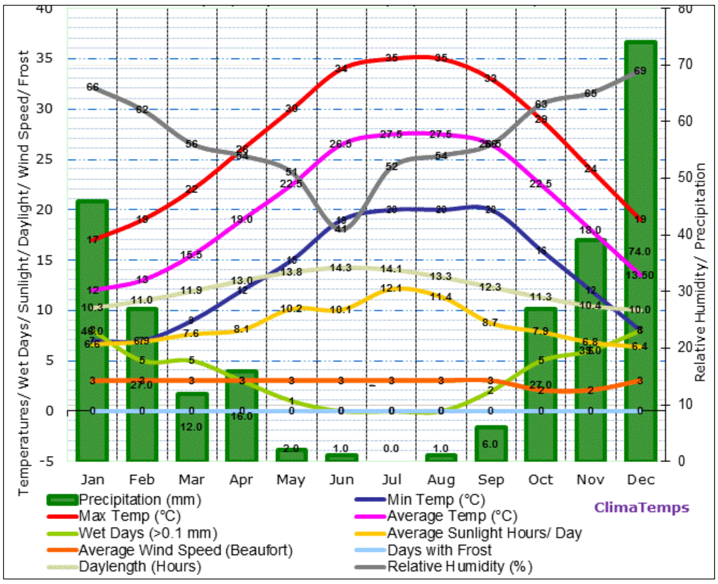


Figure I.4. Tripoli climate graph (altitude: 81m)

Many agencies-physical, chemical and biological-originating from environmental impact are responsible for the deterioration of building materials. The central “enemy” in most material failures is water, in vapour, liquid or solid from. It is the carrier of harmful contaminants, creates conditions for chemical processes and sustains biological actions. The role of moisture in the deterioration of building material should never be underestimated, and the first step in the remedial process would usually be to rectify conditions that allowed the ingress of moisture into the building elements.

Following activities are planning to be done on these bridges:

– Visual inspection of visible parts of the bridges before and after repair in order to determine defects and damages, as well as efficiency of applied repair measures,

– Quality control of built-in materials before and after repair,

- Assessment of the bridges durability, bearing capacity, stability and functionality before and after repair.

Obtained results will be used in further analyses which are related to:

- determination of characteristic defects and damages of RC bridges - overpasses in hot climate,
- ranking of bridges.

5. HYPOTHESIS OF RESEARCH

The research is based on following hypothesis:

The basic climate parameters (season temperature, humidity, wind speed etc.) have great influence on type of concrete damages that will appear during service life of RC bridges;

Proper defined causes, type and extent of damages are data of crucial importance for reliable ranking of bridges by bridge management system;

Ranking of bridges depends on chosen BMS.

6. AIMS OF RESEARCH

The main objectives of the research are:

- To define defects and damages by the elements of bridge structures, which are typical for hot climate, on the basis of theoretical analysis and the analysis of the in-situ results obtained through examination of seven RC bridges in Tripoli.
- To establish a catalogue of typical damages of RC bridge elements for a more reliable assessment of bridges during the control survey and collection of data for BMS.
- To improve the system of maintenance of bridges in Libya.

CHAPTER II

Durability of concrete (theoretical consideration)

CHAPTER II

Durability of concrete (theoretical consideration)

1. DETERORATION OF CONCRETE

There are two umbrella definitions of deterioration:

- physical manifestation of failure of a material (for example, cracking, delamination, flaking, pitting, scaling, spalling, and staining) caused by environmental or internal autogenous influences on rock and hardened concrete as well as other materials;
- decomposition of material during either testing or exposure to service, or changes in colour, texture, strength, chemical composition or other properties of a natural or artificial material due to the action of the weather.

Interaction between environmental conditions, material properties and structural factors must constantly be considered when evaluating the failure of building structures. More often than not more than one disruptive mechanism may operate and a detailed analysis of the affected structure is imperative to identify all the causes that contributed to the deterioration of the material. Many agencies-physical, chemical and biological-originating from environmental impact are responsible for the deterioration of building materials. The central villain in most material failures is water, in vapour, liquid or solid from. It is the carrier of harmful contaminants, creates conditions for chemical processes and sustains biological actions. The role of moisture in the deterioration of building material should never be underestimated, and the first step in the remedial process would usually be to rectify conditions that allowed the ingress of moisture into the building elements [14].

Of the most damages in bridges are:

- Permeability and transport processes
- Corrosion of reinforcement in concrete
- Carbonation
- Chloride ingress
- Chemical attack: sulphates

2. PERMEABILITY AND TRANSPORT PROCESSES

The durability of concrete is essentially influenced by processes that involve the passage, into or through the material, of ions or molecules in the form of liquids and gases. The service life will be dependent on the rate at which these species may move through the concrete. The passage of these potentially aggressive agencies is primarily influenced by the permeability of the concrete. Permeability may be defined as the ease with which an ion, molecule or fluid may move through the concrete. This definition is somewhat imperfect because the processes involved in fluid and ion migration include the distinct mechanisms of capillary attraction, flow under a pressure gradient and flow under a concentration gradient. These mechanisms are characterized by the material properties of sorptivity, permeability and diffusivity respectively. The term 'permeability' has often been popularly used, however, in an all-embracing manner to refer to properties that influence ingress.

The permeability of concrete to a given agent, for example carbon dioxide, is a function of the pore structure, the degree of interconnection of the pore structure and the moisture content of the permeable pore structure. The diameter of most ions and gas molecules are smaller than the pores in concrete so even the highest quality concrete will be permeable to some extent. The permeability of a concrete will be predominantly influenced by the permeability of the cement paste, especially the quality of paste in the cover concrete and at the interface with aggregate particles. The capillary pore structure is particularly significant. Permeability is a function of the degree of interconnection between the pores, the pore size distribution and its tortuosity. The moisture state is also important and can be beneficial or deleterious. Pores that are water-filled reduce the permeability to gases but may allow ionic diffusion. The significant relationship between permeability and the rate at which durability-threatening mechanisms proceed in concrete is apparent from a brief consideration of the phenomena. The most common problem is corrosion of reinforcement. The rate of corrosion is related to ease of ingress of moisture and oxygen.

Transfer of ions through the concrete is also a rate-controlling feature. Corrosion is preceded by depassivation of the reinforcement. This may be caused by carbonation or chloride ingress. Carbonation rates are a function of both physical and chemical phenomena but clearly the ease of ingress of carbon dioxide is a key feature. Depassivation due to chloride ingress is caused by the buildup of chlorides to a critical level, which is related to the ease of ingress of chloride ions from external sources.

Sulfate attack is caused by the ingress of sulfate ions, typically from groundwater. Deterioration by freeze/thaw behavior is a function of the number of freeze/thaw cycles and is related to flow of water and its distribution within the pore structure of the concrete. Alkali-silica reaction can occur in many concretes but it only becomes damaging when sufficient moisture can be imbibed from the permeable structure to cause the production of gel in amounts which cause expansion.

The transport processes involved in the passage of potentially harmful agencies through concrete are:

- gaseous diffusion (oxygen, carbon dioxide);
- vapour diffusion (moisture movement);
- ionic diffusion (chlorides, sulfates);
- absorption and capillary rise (chlorides dissolved in water);
- Pressure-induced flow (aggressive groundwater, freeze/thaw).

3. PORE STRUCTURE AND THE HYDRATION PROCESS

Permeability is obviously related to the pore structure but it is important to differentiate between porosity and permeability. Porosity is a measure of the proportion of a material represented by voids. Permeability is a measure of the ability of one material to move through another. Although permeable materials always require a pore network there is not necessarily a direct link. Lightweight concrete and air-entrained concrete, for example, may have relatively high porosity but can have low permeability. Poorly cured, high water/ cement ratio concretes will have both high porosity and high permeability. The extent to which a concrete is permeable derives from a number of factors. The most important of these, assuming well-proportioned materials and good compaction, are the water/cement ratio and the degree of hydration. It is the reaction between the un hydrated cement grains and the water that produces gel, which is particulate in nature with a pore structure that predominantly influences the permeability. The aggregate is usually impermeable or is surrounded by a layer of gel that is sufficiently impermeable to isolate the aggregate from the permeable network. Hardened concrete may be considered as a three-phase material consisting of solid matter, water and air. The relative proportions of each phase depend on the age of the concrete and the nature of the environment to which the element is exposed.

The greatest change occurs during hydration with some of the water becoming chemically bound and some evaporating. The water-filled spaces found in fresh cement paste become filled, partially or totally, by hydration products. The production of sufficiently impermeable concrete is made possible by the fact that the gel formed during hydration has a greater bulk volume than the parent cement grains. An excellent insight into the process of hydration may be gained from the work of researchers such as Powers (1958). The un hydrated cement grains may be assumed, in a slight simplification, to be formed of silicates, aluminates, and aluminoferrites. The dominant compounds are tricalcium silicates ($3CaO.SiO2$) and dicalcium silicates ($2CaO.SiO2$). These silicates are predominantly granular in nature. The silicates, on hydration, produce calcium silicate hydrates (for example $3CaO.2SiO2.3H2O$) and calcium hydroxide.

The calcium silicate hydrates have varying physical properties but it is now thought that most of them are fibrous in nature with straight edges and lengths up to ten times their width. The calcium hydroxide is more clearly crystalline. The calcium silicate hydrate crystals interlock and form both physical and chemical bonds. It is thought that the physical bonds are more significant in giving concrete its structural properties. The calcium silicate hydrate crystals are so small that the product is regarded as a gel. Cross linking of fibres leads to a particulate network with interstitial spaces. The clusters of gel particles will have spaces within them known as gel pores. Gel pores, sometimes characterised as ‘microspores’, exist as interlayer spaces between the calcium silicate hydrate sheets. The gel pores occupy about one third of the gel volume. Larger spaces are formed by the boundaries of the clusters and these are known as the capillary pores (Fig 2.1). Capillary pores may be described as the space originally occupied by the mix water. Hydration of cement grains can continue in the water-filled capillary pores.

The resulting gel will be porous but it forms at the expense of the capillary pore volume and therefore the overall effect is one of reduction in pore volume. This is due to the fact that hydrated grains occupy about twice the space of un hydrated grains. Clearly the state of the capillary pore structure at the end of the hydration stage is critically dependent on the water/cement ratio and the quality of curing. (Fig 2.2) indicates schematically the influence of water/cement ratio and curing. The graph shows the growth in gel at the expense of water during hydration. It also shows that at higher water/cement ratios the remaining free water creates a large capillary space while poor curing has the same effect by reducing the volume of gel produced.

The production of durable concrete therefore requires the water/cement ratio to be:

- low enough to limit the initial volume of the capillary pore network produced by the mix water;
- high enough to provide a water-filled capillary pore network with an initial volume at least twice that of the unhydrated cement;
- combined with a curing regime which enables the capillary network to remain water-filled long enough to ensure that the hydration process is not stopped through lack of water.

Consideration of the water/cement ratio requirements leads to a requirement in the range 0.4 to 0.5. Regarding a lower limit, (Neville, 1995) demonstrates that below a water/cement ratio of 0.38 the capillary pore volume would be insufficient to allow complete hydration of the cement grains. As the water/cement ratio increases above 0.38 the space available becomes progressively greater than that required. However the production of impermeable concrete does not demand that the capillary network should become completely filled by gel during hydration - it is sufficient if the gel partially fills the network in a manner which makes the capillary pore network discontinuous or tortuous.

At the upper end of the range, the use of a water/cement ratio in excess of 0.7 is unlikely to produce a pore network of acceptable impermeability for structural purposes.

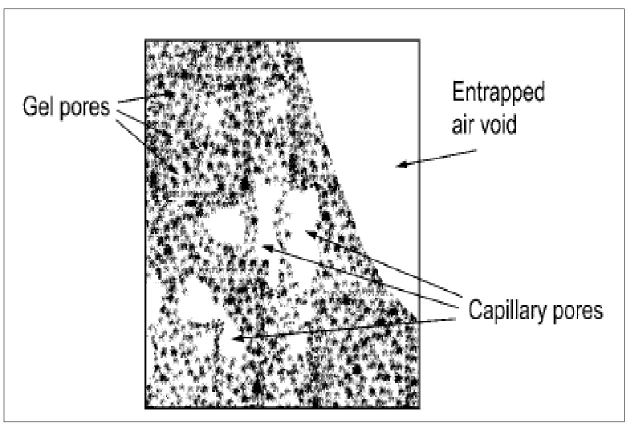


Figure II.1. Representation of the pore structure in concrete

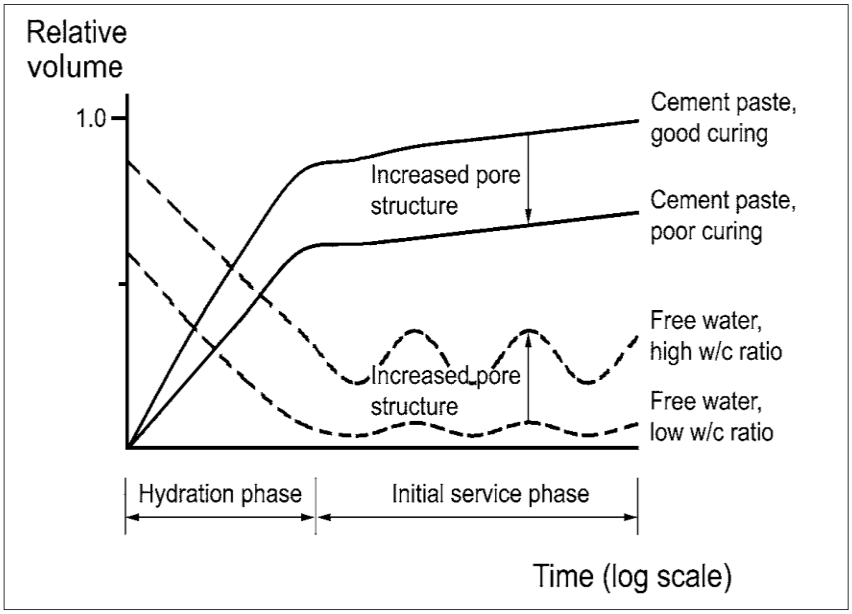


Figure II.2. Influence of water/cement ratio and quality of curing on the pore structure of concrete

4. CORROSION OF REINFORCEMENT IN CONCRETE

4.1 Nature of corrosion damage

Corrosion of reinforcement is the greatest cause of durability failure in the world today. This situation came about because designers and specifiers of works constructed in earlier decades were reliant on codes that did not fully take account of the phenomenon. The drafting of codes involves reliance on experience and there was insufficient evidence and understanding of the corrosion phenomenon in concrete to better inform the drafters. The emerging generation of standards will improve matters by forcing us to explicitly consider the mechanism of failure. It is important therefore to appreciate some fundamental aspects of the corrosion phenomenon. Corrosion is an electrochemical process whereby a metal undergoes a reaction with chemical species in the environment to form a compound. The chemical species are principally oxygen and water. Steel reinforcement has a natural tendency to corrode if access to oxygen is possible in a moist environment.

This is because it is formed of metals found naturally occurring as ores to which they wish to revert. The durability of reinforced concrete requires conditions in which the dissolution of metal atoms is not supported and that the reinforcement be inaccessible to oxygen and moisture. Two self-defense mechanisms are employed to achieve this. The first involves a naturally occurring protective film on the reinforcement, which requires certain conditions for its survival. The second involves cover concrete of sufficient depth and impermeability. Regarding the first issue, the high pH level of fresh concrete leads to the formation of a passive skin on the surface of reinforcement.

This skin prevents corrosion occurrence by preventing contact with oxygen and moisture. The passive film may be broken down in time through carbonation or chloride ingress reaching the steel. Regarding cover, the rate of corrosion depends on the rate at which oxygen and moisture may penetrate the cover. This has a two-fold influence. First, oxygen and moisture are required to feed the process. Second, the concrete must be moist enough to have an electrical resistance that is low enough to allow the creation of an electrochemical cell.

4.2 Electrochemical process

Measures to control corrosion of reinforcement require an understanding of the processes involved. Corrosion is an electrochemical process and therefore a basic understanding of electrochemistry and its application to the particular case of reinforced concrete is required

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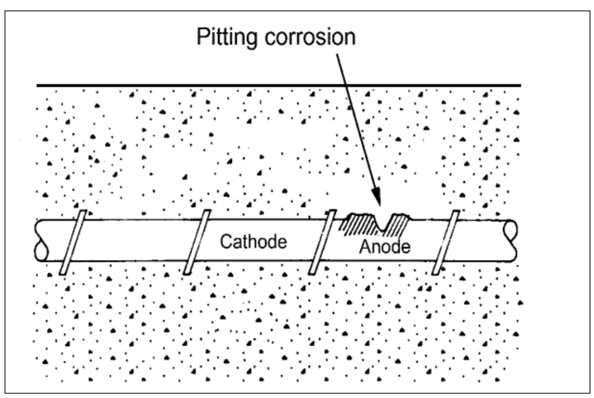


Figure II.3. Pitting corrosion

5. CARBONATION

5.1 Carbonation and corrosion

Carbonation is the term used to describe the effect of carbon dioxide on a material. The phenomenon rarely leads to structural problems but carbonation-induced corrosion can lead to unsightly spalling on structures. The repair costs, for example to multi-storey office development facades, can be considerable. Carbonation of cementitious materials results in a lowering of the pH -making the material less alkaline - and hence the term ‘neutralisation’ is also sometimes used in the literature. Reinforcement in concrete is embedded in an oxygenated alkaline solution. The reinforcement will not corrode if the protection afforded by the passive film - a thin layer of oxide deposited on the steel - remains substantially intact.

This insoluble oxide film prevents oxygen reaching the steel and inhibits corrosion. The reinforcement is said to be ‘passive’ when it is in this state. Corrosion of reinforcement can commence however if the passive oxide film protecting the reinforcement is destroyed, the cover concrete is sufficiently permeable to oxygen and moisture, and the concrete is moist enough to serve as an electrolyte. The lowered pH in zones of carbonated concrete may threaten the continuity of the passive film. It is important therefore to specify cover concrete that is capable of resisting the penetration of the carbonation front as far as the reinforcement during the service life of the structure.

5.2 Chemistry of carbonation

Initially carbon dioxide diffuses through the surface of the concrete due to the concentration difference between the atmosphere and the concrete pore structure. A thin skin of carbonated concrete develops which may be less than a millimeter in thickness. Further penetration is primarily a function of the concrete permeability and the amount of calcium hydroxide available for reaction. Carbon dioxide passes unhindered through the carbonated layer and is available for reaction with the next layer of calcium hydroxide. It may progressively penetrate further into the concrete over time and ultimately part of the carbonation front may reach the reinforcement and cause depassivation (Fig 2.4).

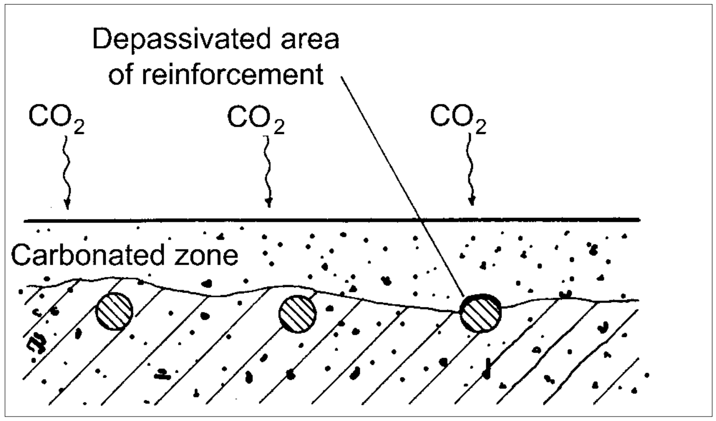


Figure II.4. Ingress of the carbonated zone to the reinforcement

5.3 Detection of the carbonation front

There are many different methods for locating the carbonation front. These include acid/base indicators, mineralogical analysis, differential thermal analysis, thermogravimetry, x-ray diffraction, neutron radiography, infra-red spectroscopy, and chemical analysis. The simplest method to use is the acid/base indicator phenolphthalein. This indicator solution method is the basis of almost all carbonation studies, especially due to its usefulness in field tests. The indicator changes color at a pH of approximately 9. Below this figure it remains colorless but above pH 9 it turns purple. It requires little skill in use and gives reproducible results. The test is carried out on freshly broken surfaces brushed free of dust and sprayed with indicator solution.

Readings on structures in service are best carried out on cores. Alternatively on-site testing may be carried out by drilling a 20 mm diameter hole and exposing the edges of the hole with a hammer and chisel. Phenolphthalein may then be sprayed onto the freshly broken surface. The smooth drilled surface is not amenable to testing. Concrete that is difficult to expose may be examined by drilling closely spaced holes and breaking out the concrete in between. The indicator solution is usually prepared from a gramme of phenolphthalein powder per 70 ml of ethanol and 30 ml distilled/ion exchanged water.

The depth of carbonation may therefore be regarded as the average distance from the surface of the concrete element to the zone where phenolphthalein indicator solution changes color to purple, indicating that carbon dioxide has not reduced the alkalinity of the hydrated cement in that zone.

The depassivation threat from carbonation is adequately assessed by locating the front with phenolphthalein even if the indicator does not mark the border between calcium hydroxide-dominated and calcium carbonate-dominated zones with absolute precision.

The carbonation front does not always advance at a constant rate due to in homogeneities in the concrete and dense aggregates are not colored by phenolphthalein. Thus it may be necessary to record both the average and the maximum depth of carbonation (Fig 2.5). The average depth gives an indication of the quality of the concrete and the influence of the local environment.

The maximum depth is important from the point of view of durability being threatened if a sufficient number of peaks on the front reach the reinforcement. The phenolphthalein test may also be carried out on site by breaking off pieces of concrete and spraying the exposed concrete. This may be misleading if readings are concentrated on corners of columns and beams.

These areas are the easiest to break off but they may have high carbonation depths that may not be representative of the general structure. The corners will experience biaxial penetration of carbon dioxide, as illustrated in (Fig 2.6), and the permeability of concrete placed in the corners of shutters may not be representative of the member due to compaction difficulties.

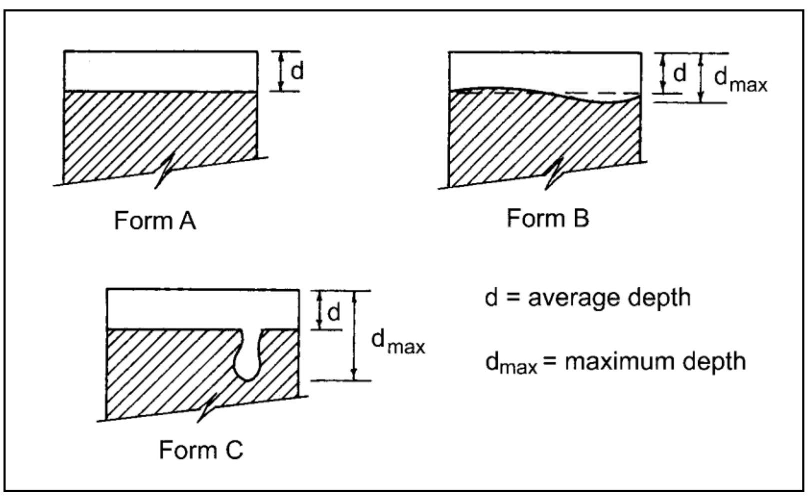


Figure II.5. Forms of carbonation profile encountered in practice

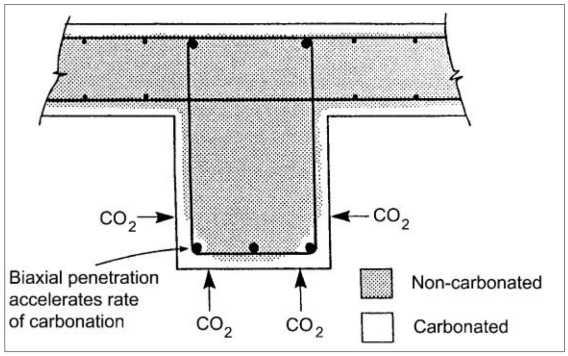


Figure II.6. Influence on carbonation profile of biaxial penetration of carbon dioxide

5.4 Primary factors influencing carbonation rate

Carbonation rate is significantly influenced by several factors that interact and the combined effect of these may exacerbate or ameliorate the process. The primary factors, determined from both field observations and theoretical considerations, are as follows:

- diffusivity/permeability;
- reserve alkalinity;
- the environmental carbon dioxide concentration;
- the exposure condition.

6. SULFATE ATTACK

Concrete may be damaged in a number of ways through contact with salts. Salts are chemical compounds typically formed by reactions between acids and bases and which dissociate in water into their constituent ions. The problems of reinforcement corrosion associated with sodium chloride are well documented but deterioration of the concrete itself may also arise through contact with sulfate salts. The term ‘sulfate attack’ has traditionally been associated with durability failure through disruptive expansion of concrete in contact with sulfate-bearing soils or groundwater. However, the effect of sulfate may manifest itself in a number of ways. This chapter considers three phenomena involving sulfate and concrete interaction that can be differentiated through their chemical or chronological characteristics:

- ‘classical’ sulfate attack;
- Thaumasite sulfate attack;
- Delayed ettringite formation.

The classical form of sulfate attack is a problem generally associated with buried concrete exposed to soils or groundwater containing soluble sulfates. Deleterious conditions may also arise in sewers. Significant durability problems can occur where concrete elements are exposed to such salts in solution above a critical concentration level. Durability failure occurs through a combination of expansive disruption and deterioration of the cement paste.

Tricalcium aluminate hydrate and calcium hydroxide can react with sulfate ions in solution to form solid products that occupy a larger volume than the source constituents. Ettringite and gypsum are the principal compounds formed. The disruptive expansion can be accompanied by strength loss consequent on the chemical deterioration of cement paste and damage to the aggregate interface bond. Concrete may crack parallel to the surface or become friable.

The related but rarer phenomenon of thaumasite formation has received increased attention lately. Its potential deleterious effect has been known for some decades but recent cases of damage in some United Kingdom motorway bridges has heightened awareness. Thaumasite is formed through a sulfate reaction involving limestone in aggregate, filler, or groundwater. It is a complex mineral that can attack the vital calcium silicate hydrates thus rendering the concrete soft, weak, or mushy. Delayed ettringite formation is a form of internal sulfate attack which may occur at an advanced age in particular concretes. Portland cements contain internal sulfates and added gypsum to influence setting and early strength characteristics.

Without the gypsum the reaction between tricalcium aluminate and water would lead to a flash set. Ettringite, an expansive compound involving tricalcium aluminate and gypsum, is normally formed at the hydration stage while the material is plastic and can accommodate the resultant strains. In particular conditions this internal sulfate may cause a phenomenon later in service.

This phenomenon is quite distinct from the classical and thaumasite forms of sulfate attack. The problem arises if ettringite is not allowed to develop at the plastic stage due to high temperature curing conditions. Subsequent wet conditions in service may encourage ettringite formation in mature concrete with consequent expansion of the cement paste and cracking.

High temperatures during curing may arise either through steam curing of pre-cast concrete products or through the significant effect of the heat of hydration in large pours. The European standard EN 206-1 includes sulfate attack as a subset of the multifaceted phenomenon of chemical attack.

The standard includes three exposure classes in respect of chemical attack - slightly, moderately and highly aggressive - based on the wider chemical composition of the environment.

6.1 factors influencing sulfate attack

Influences on classical forms of sulfate attack

Research has identified the following factors that have an influence on the intensity of sulfate attack:

- Sulfate concentration;
- Solubility of sulfates;
- Groundwater mobility;
- Concrete permeability;
- Wetting and drying cycles;
- Evaporation;
- Degree of carbonation prior to exposure.

Influences on thaumasite sulfate attack

The primary factors that must simultaneously be present for thaumasite sulfate attack are as follows (Hartshorn and Sims 1998, DETR 1999):

- Sulfates and/or sulfides in the ground;
- Mobile groundwater;
- Presence of carbonate;
- Low temperature.

The sulfates may pre-exist in the soil but thaumasite may also occur through the oxidation of pyrite. An abundant supply of water is required. Cold conditions are required with temperatures at least below 15Co. An alumina content is required, even if only at a low level. Indeed alumina contents encourage ettringite formation whereas lower amounts may facilitate reaction with the carbonate and calcium silicates.

Influences on delayed ettringite formation

The potential occurrence of delayed ettringite formation is predicated on the temperature during hydration and the availability of moisture in service. The principal influences on delayed ettringite formation are:

- Air temperature during hydration;
- Size and geometry of pour;
- Cement content;
- Cement chemistry and fineness.

CHAPTER III

TESTING OF CONCRETE IN STRUCTURES

CHAPTER III

TESTING OF CONCRETE IN STRUCTURES

1. SURFACE HARDNESS METHODS

One of many factors connected with the quality of concrete is its hardness. Efforts to measure the surface hardness of a mass of concrete were first recorded in the 1930s; tests were based on impacting the concrete surface with a specified mass activated by a standard amount of energy. Early methods involved measurements of the size of indentation caused by a steel ball either fixed to a pendulum or spring hammer, or fired from a standardized testing pistol. Later, however, the height of rebound of the mass from the surface was measured. Although it is difficult to justify a theoretical relationship between the measured values from any of these methods and the strength of a concrete, their value lies in the ability to establish empirical relationships between test results and quality of the surface layer. Unfortunately these are subject to many specific restrictions including concrete and member details, as well as equipment reliability and operator technique [15].

2. REBOUND TEST EQUIPMENT AND OPERATION

The Swiss engineer Ernst Schmidt first developed a practicable rebound test hammer in the late 1940s, and modern versions are based on this. (Fig 3.1) shows the basic features of a typical type N hammer, which weighs less than 2 kg, and has an impact energy of approximately 2.2 Nm. The spring- controlled hammer mass slides on a plunger within a tubular housing. The plunger retracts against a spring when pressed against the concrete surface and this spring is automatically released when fully tensioned, causing the hammer mass to impact against the concrete through the plunger [15].

When the spring-controlled mass rebounds, it takes with it a rider which slides along a scale and is visible through a small window in the side of the casing. The rider can be held in position on the scale by depressing the locking button. The equipment is very simple to use (Fig 3.2), and may be operated either horizontally or vertically, either upwards or downwards [15].

The plunger is pressed strongly and steadily against the concrete at right angles to its surface, until the spring-loaded mass is triggered from its locked position.

After the impact, the scale index is read while the hammer is still in the test position. Alternatively, the locking button may be pressed to enable the reading to be retained, or results can be recorded automatically by an attached paper recorder. The scale reading is known as the rebound number, and is an arbitrary measure since it depends on the energy stored in the given spring and on the mass used [15].

This version of the equipment is most commonly used, and is most suitable for concretes in the 20–60 N/mm^2 strength range. Electronic digital reading equipment with automatic data storage and processing facilities is also widely available (Fig 3.3). Other specialized versions are available for impact sensitive zones and for mass concrete. For low strength concrete in the 5–25 N/mm^2 strength range it is recommended that a pendulum type rebound hammer as shown in(Fig 3.4 is used which has an enlarged hammer head (Type P) [15].

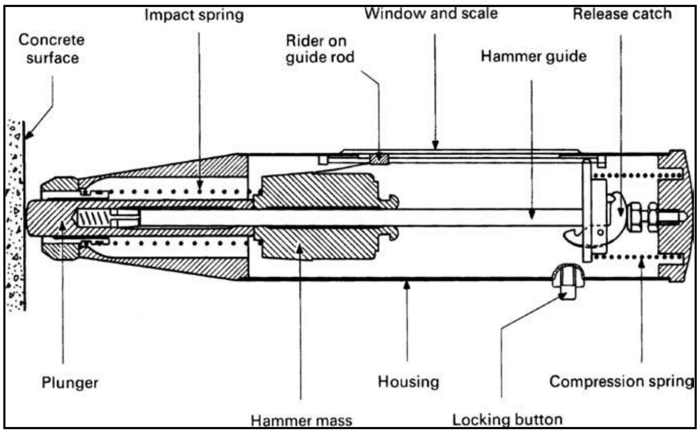


Figure III-1. Typical rebound hammer

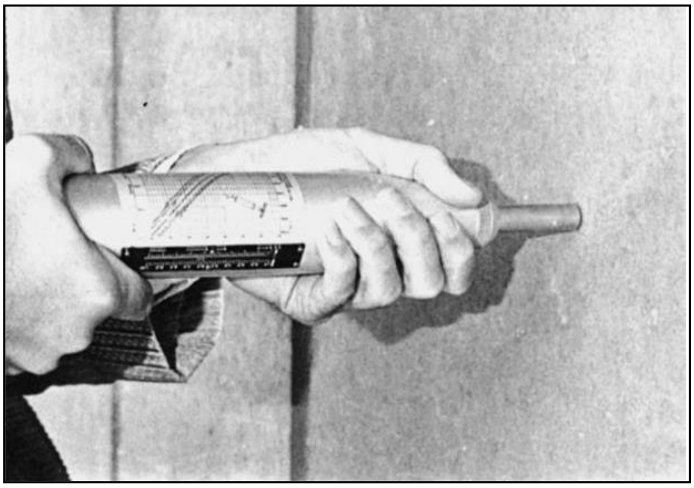


Figure III-2. Schmidt hammer in use

Figure III.3. Digi-Schmidt (photograph by courtesy of Proceq)

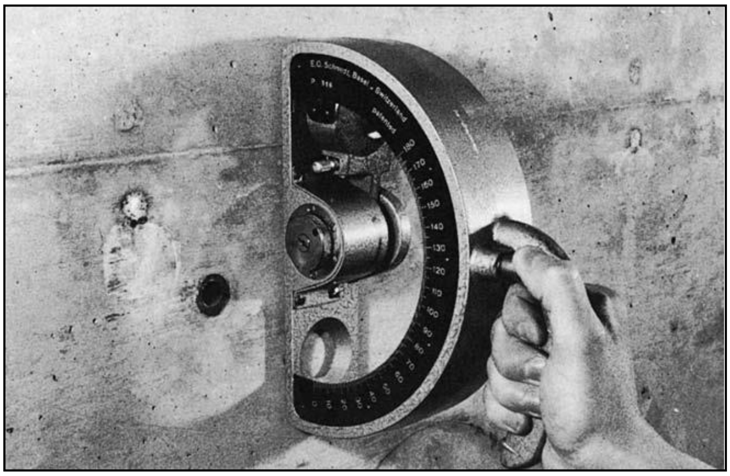


Figure III.4. Pendulum hammer

The reading is very sensitive to local variations in the concrete, especially to aggregate particles near to the surface. It is therefore necessary to take several readings at each test location, and to find their average. Standards vary in their precise requirements, but BS EN 12504-2 recommends not less than nine readings taken over an area not exceeding 300mm square, with the impact points no less than 25mm from each other or from an edge [15].

The use of a grid to locate these points reduces operator bias. Prior to testing, the equipment should be operated at least three times to ensure proper functioning and checked on the steel reference anvil with adjustment as necessary [15].

Temperature should be in the range 10-35 C^{o}. Any measurements where the surface has crushed or broken through a near surface void should be discounted, whilst if more than 20% of results are more than 6 units from the median the whole set should be discarded. ASTM C805 requires ten readings to be taken. The surface must be smooth, clean and dry, and should preferably be formed, but if trowel led surfaces are unavoidable they should be rubbed smooth with the Carborundum stone usually provided with the equipment. Loose material can be ground off, but areas which are rough from poor compaction, grout loss, spalling or tooling must be avoided since the results will be unreliable [15].

The test is based on the principle that the rebound of an elastic mass depends on the hardness of the surface upon which it impinges, and in this case will provide information about a surface layer of the concrete defined as no more than 30mm deep. The results give a measure of the relative hardness of this zone, and this cannot be directly related to any other property of the concrete. Energy is lost on impact due to localized crushing of the concrete and internal friction within the body of the concrete, and it is the latter, Which is a function of the elastic properties of the concrete constituents that makes theoretical evaluation of test results extremely difficult (Akashi et all, 1984) many factors influence results but all must be considered if rebound number is to be empirically related to strength [15].

Results are significantly influenced by all of the following factors:

1. Mix characteristics
   - Cement type
   - Cement content
   - Coarse aggregate type
2. Member characteristics
   - Mass
   - Compaction
   - Surface type
   - Age, rate of hardening and curing type
   - Surface carbonation
   - Moisture condition
   - Stress state and temperature.

Since each of these factors may affect the readings obtained, any attempts to compare or estimate concrete strength will be valid only if they are all standardized for the concrete under test and for the correlation specimens. These influences have different magnitudes. Hammer orientation will also influence measured values although correction factors can be used to allow for this effect [15].

The three mix characteristics listed above are now examined in more detail.

(i) Cement type. Variations in fineness of Portland cement are unlikely to be significant -their influence on strength correlation is less than 10%. Super-sulfated cement, however, can be expected to yield strengths 50% lower than suggested by a Portland cement correlation, whereas high alumina cement concrete may be up to 100% stronger.

(ii) Cement content. Changes in cement content do not result in corresponding changes in surface hardness. The combined influence of strength, workability and aggregate/cement proportions leads to a reduction of hardness relative to strength as the cement content increases (Kolek, 1970). The error in estimated strength, however, is unlikely to exceed 10% from this cause for most mixes.

(iii) Coarse aggregate. The influence of aggregate type and proportions can be considerable, since strength is governed by both paste and aggregate characteristics. The rebound number will be influenced more by the hardened paste. For example, crushed limestone may yield a rebound number significantly lower than for a gravel concrete of similar strength which may typically be equivalent to a strength difference of 6-7N/mm^2. A particular aggregate type may also yield different rebound number/strength correlations depending on the source and nature, and (Fig3.5) compares typical curves for hard and soft gravels. These have measured hardness expressed in terms of the Mohs' number of 7 and 3 respectively.
Lightweight aggregates may be expected to yield results significantly different from those for concrete made with dense aggregates, and considerable variations have also been found between types of lightweight aggregates (Bungey et all, 1994).

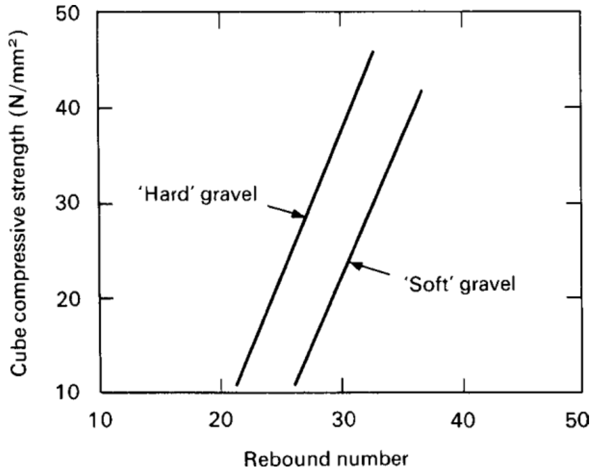


Figure III.5. Comparison of hard and soft gravels – vertical hammer

The member characteristics listed above are also to be discussed in detail.

(i) Mass. The effective mass of the concrete specimen or member under test must be sufficiently large to prevent vibration or movement caused by the hammer impact. Any such movement will result in a reduced rebound number. For some structural members the slenderness or mass may be such that this criterion is not fully satisfied, and in such cases absolute strength prediction may be difficult. BS EN 12504-2 requires that a member is at least 100mm thick and fixed within a structure. Strength comparisons between or within individual members must also take account of this factor. The mass of correlation specimens may be effectively increased by clamping them firmly in a heavy testing machine [15].

(ii) Compaction. Since a smooth, well-compacted surface is required for the test, variations of strength due to internal compaction differences cannot be detected with any reliability. All calibrations must assume full compaction [15].

(iii) Surface type. Hardness methods are not suitable for open-textured or exposed aggregate surfaces. Trowel led or floated surfaces may be harder than moulded surfaces, and will certainly be more irregular. Although they may be smoothed by grinding, this is laborious and it is best to avoid trowel led surfaces in view of the likely overestimation of strength from hardness readings. The absorption and smoothness of the mould surface will also have a considerable effect. Calibration specimens will normally be cast in steel moulds which are smooth and non-absorbent, but more absorbent shuttering may well produce a harder surface, and hence internal strength may be overestimated. Although moulded surfaces are preferred for on-site testing, care must be taken to ensure that strength correlations are based on similar surfaces, since considerable errors can result from this cause [15].

(iv) Age, rate of hardening and curing type. The relationship between hardness and strength has been shown to vary as a function of time (Kolek, 1970), and variations in initial rate of hardening, subsequent curing and exposure conditions will further influence this relationship. Where heat treatment or some other form of accelerated curing has been used, a specific calibration will be necessary. The moisture state may also be influenced by the method of curing. For practical purposes the influence of time may be regarded as unimportant up to the age of three months, but for older concretes it may be possible to develop reduction factors which take account of the concrete's history [15].

(v) Surface carbonation. Concrete exposed to the atmosphere will normally form a hard carbonated skin, whose thickness will depend upon the exposure conditions and age. It may exceed 20mm for old concrete although it is unlikely to be significant at ages of less than three months. The depth of carbonation can easily be determined as described. Examination of gravel concrete specimens which had been exposed to an outdoor 'city-centre' atmosphere for six months showed a carbonated depth of only 4 mm. This was not sufficient to influence the rebound number/strength relationship in comparison with similar specimens stored in a laboratory atmosphere although for these specimens no measurable skin was detected. In extreme cases, however, it is known that the overestimate of strength from this cause may be up to 50%, and is thus of great importance. When significant carbonation is known to exist, the surface layer ceases to be representative of the concrete within an element [15].

(vi) Moisture condition. The hardness of a concrete surface is lower when wet than when dry, and the rebound/strength relationship will be influenced accordingly. This effect is illustrated by (Fig3.7), based on early work by the US army (Willetts, 1958), from which it will be seen that a wet surface test may lead to an underestimate of strength of up to 20%. Field tests and strength calibrations should normally be based on dry surface conditions, but the effect of internal moisture on the strength of control specimens must not be overlooked [15].

(vii) Stress state and temperature. Both these factors may influence hardness readings, although in normal practical situations this is likely to be small in comparison with the many other variables. Particular attention should, however, be paid to the functioning of the test hammer if it is to be used under extremes of temperature, noting the limits of 10 to 35C^{o} in BS EN 12504-2 [15].

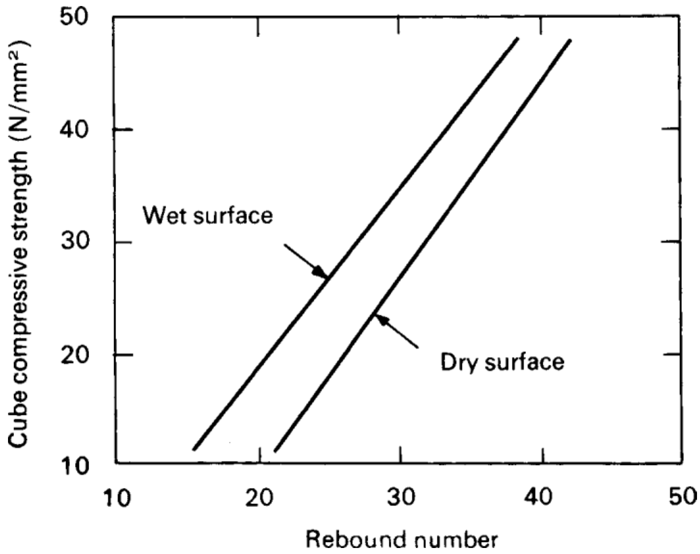


Figure III.6. Influence of surface moisture condition-horizontal hammer (Willetts, 1958).

Clearly, the influences of the variables described above are so great that it is very unlikely that a general calibration curve relating rebound number to strength, as provided by the equipment manufacturers, will be of any practical value. The same applies to the use of computer data processing to give strength predictions based on results from the electronic rebound hammer shown in Fig3.8, unless the conversions are based on case specific data. Strength calibration must be based on the particular mix under investigation, and the mould surface, curing and age of laboratory specimens should correspond as closely as possible to the in-place concrete. It is essential that correct functioning of the rebound hammer is checked regularly using a standard steel anvil of known mass. This is necessary because wear may change the spring and internal friction characteristics of the equipment [15].

Calibrations prepared for one hammer will also not necessarily apply to another. It is probable that very few rebound hammers used for in-situ testing are in fact regularly checked against a standard anvil, and the reliability of results may suffer as a consequence. The importance of specimen mass has been discussed above; it is essential that test specimens are either securely clamped in a heavy testing machine or supported upon an even solid floor. Cubes or cylinders of at least 150mm should be used, and a minimum restraining load of 15% of the specimen strength has been suggested for cylinders (Malhotra,1976), and not less than 7N/mm^2 is recommended for cubes tested with a type N hammer [15].

Some typical relationships between rebound number and restraining load are given in (Fig3.8), which shows that once a sufficient load has been reached the rebound number remains reasonably constant. It is well established that the crushing strength of a cube tested wet is likely to be about 10% lower than the strength of a corresponding cube tested dry. Since rebound measurements should be taken on a dry surface, it is recommended that wet cured cubes be dried in the laboratory atmosphere for 24 hours before test, and it is therefore to be expected that they will yield higher strengths than if tested wet in the standard manner [15].

Depending upon the purpose of the test programme it may be necessary to confirm this relationship, and the relative moisture conditions of the correlation specimens and in-place concrete must also be considered when interpreting the field results. The use of cores cut following in-situ hardness tests may help to overcome these difficulties in developing calibrations. If cubes are used, readings should be taken on at least two vertical faces of the specimen as cast, and the hammer orientation must be similar to that to be used for the in-place tests. The influence of gravity on the mass will depend on whether it is moving vertically up or down, horizontally or on an inclined plane. The effect on the rebound number will be considerable, although the relative values suggested by the manufacturer are likely to be reliable in this instance because this is purely a function of the equipment [15].

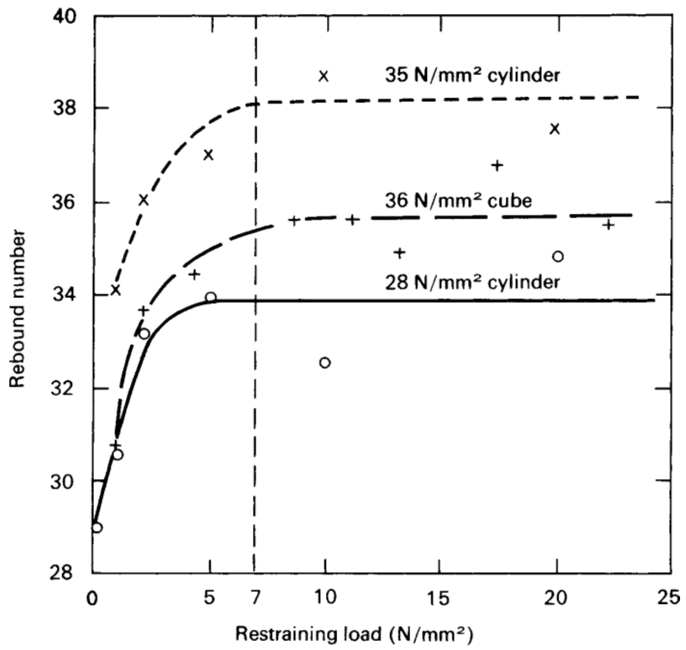


Figure III.7. Effect of restraining load on calibration specimen (Malhotra, 1976).

The interpretation of surface hardness readings relies upon a knowledge of the extent to which the factors have been standardized between readings being compared. This applies whether the results are being used to assess relative quality or to estimate strength. It will be apparent from Figure3.9 , which shows a typical strength calibration chart produced under 'ideal' laboratory conditions, that the scatter of results is considerable, and the strength range corresponding to a given rebound number is about ±15% even for 'identical' concrete. In a practical situation it is very unlikely that a strength prediction can be made to an accuracy better than ±25% (Malhotra, 1976). The scatter also suggests that even if a strength prediction is not required, a considerable variation of rebound number can be expected for 'identical' concrete, and acceptable limits must be determined in conjunction with some other form of testing. It is suggested that where the total number of readings -n- taken at a location is not less than ten, the accuracy of the mean rebound number is likely to be within ±15/√n% with 95% confidence. The results may usefully be presented in graphical form, and calculation of the coefficient of variation may yield an indication of concrete uniformity [15].

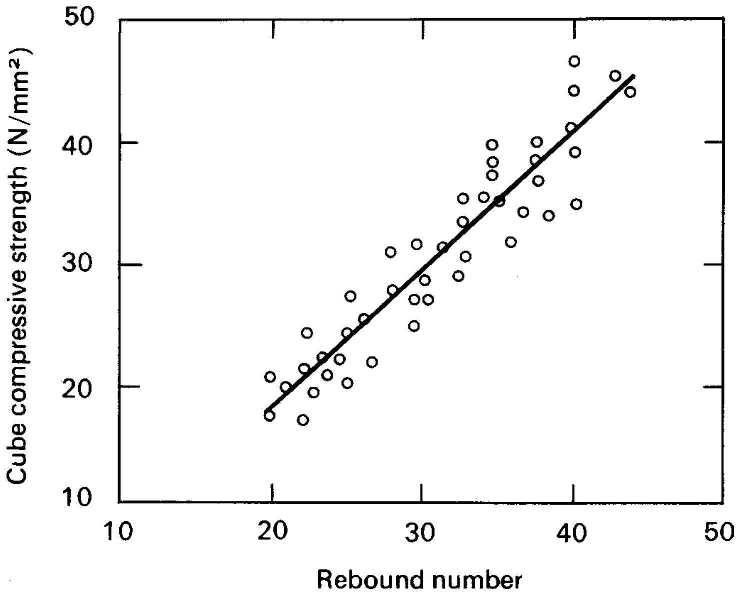


Figure III.8. Typical rebound number/compressive strength calibration chart

The useful applications of surface hardness measurements can be divided into four categories:

(i) Checking the uniformity of concrete quality
(ii) Comparing a given concrete with a specified requirement
(iii) Approximate estimation of strength
(iv) Abrasion resistance classification.

Whatever the application, it is essential that the factors influencing test results are standardized or allowed for, and it should be remembered that results relate only to the surface zone of the concrete under test. A further overriding limitation relates to testing at early ages or low strengths, because the rebound numbers may be too low for accurate reading and the impact may also cause damage to the surface

(Figure3.10). It is therefore not recommended that the method is used for concrete which has a cube strength of less than 10N/mm2 or which is less than 7 days old, unless of high strength [15].

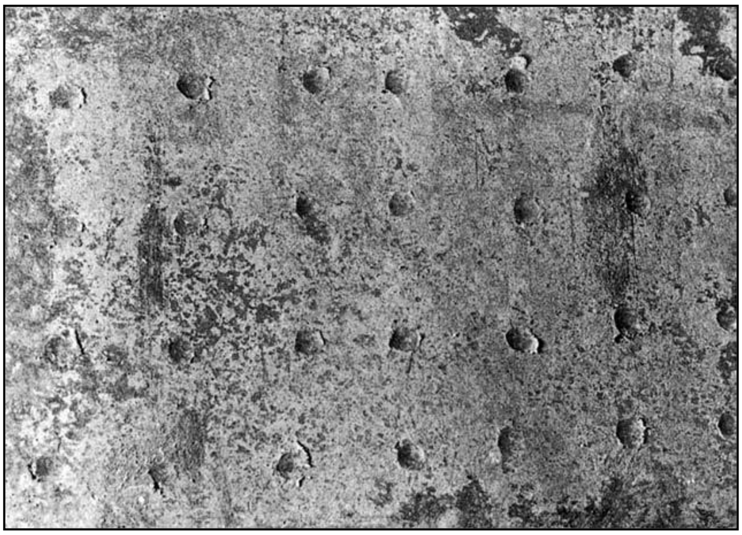


Figure III.9. Surface damage on green concrete

(i) Concrete uniformity checking. The most important and reliable applications of surface hardness testing are where it is not necessary to attempt to convert the results to some other property of the concrete. Although they do not detect poor internal compaction, results are sensitive to variations of quality between batches, or due to inadequate mixing or segregation. The value as a control test is further enhanced by the ability to monitor the concrete in members cheaply and more comprehensively than is possible by a small number of control specimens. For such comparisons to be valid for a given mix it is only necessary to standardize age, maturity, surface moisture conditions (which should preferably be dry), and location on the structure or unit. This approach has been extensively used to control uniformity of precast concrete units, and may also prove valuable for the comparison of suspect in-situ elements with similar elements which are known to be sound. A further valuable use for such comparative tests may be to establish the representation of other forms of testing, possibly destructive, which may yield more specific but localized indications of quality [15].

(ii) Comparison with a specific requirement. This application is also popular in the precasting industry, where a minimum hardness reading may be calibrated against some specific requirement of the concrete. For instance, the readiness of precast units for transport may be checked, with calibration based on proof load tests. The approach may also be used as an acceptance criterion, in relation to the removal of temporary supports from structural members, or commencement of stress transfer in prestressed concrete construction [15].

(iii) Approximate strength estimation. This represents the least reliable application and (unfortunately, since a strength estimate is frequently required by engineers) is where misuse is most common. The accuracy depends entirely upon the elimination of influences which are not taken into account in the calibration. For laboratory specimens cast, cured and tested under conditions identical to those used for calibration, it is unlikely that a strength estimate better than ±15% can be achieved for concrete up to three months old. Although it may be possible to correct for one or two variables which may not be identical on site, the accuracy of absolute strength prediction will decline as a consequence and is unlikely to be better than ±25%. The use of the rebound hammer for strength estimation of in-place concrete must never be attempted unless specific calibration charts are available, and even then, the use of this method alone is not recommended, although the value of results may be improved if used in conjunction with other forms of testing [15].

(iv) Abrasion resistance classification. Abrasion resistance is generally affected by the same influences as surface hardness, and Chaplin (Chaplin, 1980) has suggested that the rebound hammer may be used to classify this property. It is also reasonable to suppose that other durability characteristics that are related to a dense, well cured, outer surface zone may similarly be classified [15].

3. PULL OFF TESTING

Concrete pull-off testing is used to measure the direct tensile strength of a material or bond strength of an interface. The concrete pull-off testing equipment consists of a metal test disc, epoxy, core drill, draw bolt, and jack. First, a shallow core is drilled perpendicularly into the surface leaving the intact core still attached to the material at the area of interest. Next, a metal test disc of the same diameter as the core is epoxied/bonded to the surface of the attached core. Once the epoxy cures, a bolt is attached to the metal test disc and the jack, shown in Figure 1, pulls the bolt/disc until failure occurs. The load at failure as well as the location of the failure is recorded. Failure can occur in any one of the following planes: (A) epoxy, (B) overlay (if applicable), (C) interface, or (D) substrate, as shown in Figure 2.

Concrete pull-off testing is used to confirm the substrate strength prior to the installation of repair materials or as quality assurance for the installation of overlays or fiber wrap. Such testing follows ASTM C 1583, Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off Method). Using this type of testing for quality assurance ensures that clients receive an adequate, well-constructed repair.

Figure III-9. Concrete Pull-Off Testing Equipment

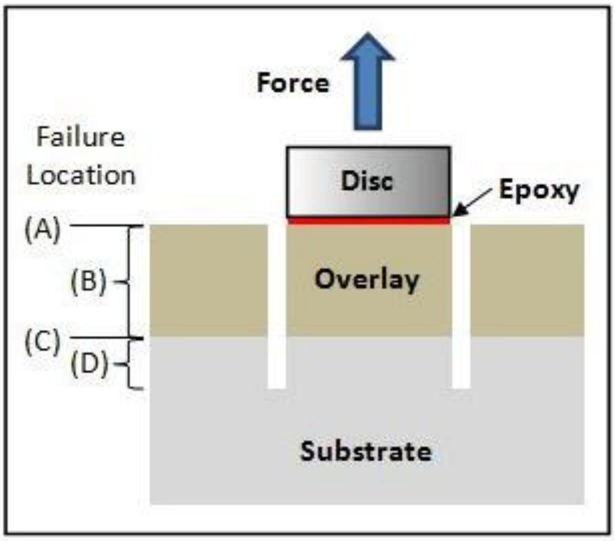


Figure III-10 Concrete Pull-Off Testing Schematic and Failure Planes

4. CORES

The examination and compression testing of cores cut from hardened concrete is a well-established method, enabling visual inspection of the interior regions of a member to be coupled with strength estimation. Other physical properties which can be measured include density, water absorption, and indirect tensile strength and movement characteristics including expansion due to alkali–aggregate reactions. Cores are also frequently used as samples for chemical analysis following strength testing. In most countries standards are available which recommend procedures for cutting, testing and interpretation of results; BS EN 12504-1 [30] in the UK, whilst ASTM C42 [31] and ACI 318 [32] are used in the USA. It must be noted however that the above new European Standard offers no guidance on planning or interpretation, although a further document dealing with this is in preparation. Extremely valuable and detailed supplementary information and guidance is also given by Concrete Society Technical Report 11 [29] and its addendum, which are related to the former British Standard (BS 1881: Part 201 - now withdrawn). A UK National Annex to BS EN 12504-1 is also in preparation dealing with allowances for voidage, reinforcement, maturity and direction of drilling, and this is likely to reflect the Concrete Society guidance. The Concrete Society have also published the results of extensive field experiments aimed at enhancing interpretation in terms of estimated cube strengths for different cement types, member types and construction conditions [33]. Interpretation is a potentially complex process and (Neville, 2001) has recently reviewed many of the issues involved including sampling and testing planning [15].

Core location will be governed primarily by the basic purpose of the testing, bearing in mind the likely [15].

serviceability assessment is the principal aim, tests should normally be taken at points of likely minimum strength, for example from the top surface at near mid span for simple beams and slabs, or from any face near the top of lifts for columns or walls. If the member is slender, however, and core cutting may impair future performance, cores should be taken at the nearest non-critical locations. Aesthetic considerations concerning the appearance after coring may also sometimes influence the choice of locations. Alternatively, areas of suspect concrete may have been located by other methods [15].

If specification compliance determination is the principal aim, the cores should be located to avoid unrepresentative concrete, and for columns, walls or deep beams will normally be taken horizontally at least 300mm below the top of the lift. If it is necessary to drill vertically downwards, as in slabs, the core must be sufficiently long to pass through unrepresentative concrete which may occupy the top 20% of the thickness. In such cases drilling upwards from the soffit, if this is feasible, may considerably reduce the extent of drilling, but the operation may be more difficult and may introduce additional uncertainties relating to the effects of possible tensile cracking. Reinforcement bars passing through a core will increase the uncertainty of strength testing, and should be avoided wherever possible. The use of a cover meter to locate reinforcement prior to cutting is therefore recommended [15].

A core is usually cut by means of a rotary cutting tool with diamond bits, as shown in Figure 3.25. The equipment is portable, but it is heavy and must be firmly supported and braced against the concrete to prevent relative movement which will result in a distorted or broken core, and a water supply is also necessary to lubricate the cutter. Vacuum-assisted equipment can be used to obtain a firm attachment for the drilling rig without resorting to expansion bolts or cumbersome bracing. Uniformity of pressure is important, so it is essential that drilling is performed by a skilled operator. Hand-held equipment is available for cores up to 75mm diameter. A cylindrical specimen is obtained, which may contain embedded reinforcement, and which will usually be removed by breaking off by insertion of a cold chisel down the side of the core, once a sufficient depth has been drilled. The core, which will have a rough inner end, may then be removed using the drill or tongs, and the hole made good. This is best achieved either by ramming a dry, low shrinkage concrete into the hole, or by wedging a cast cylinder of suitable size into the hole with cement grout or epoxy resin. It is important that each core is examined at this stage, since if there is insufficient length for testing, or excessive reinforcement or voids, extra cores must be drilled from adjacent locations. Each core must be clearly labeled for identification, with the drilled surface shown, and cross-referenced to a simple sketch of the element drilled. Photographs of cores are valuable for future reference, especially as confirmation of features noted during visual inspection, and these should be taken as soon as possible after cutting. A typical photograph of this type is shown in Figure…. Cores should be securely wrapped in several layers of ‘Clingfilm’ and then placed in a labeled polythene bag for return to the testing laboratory [15].

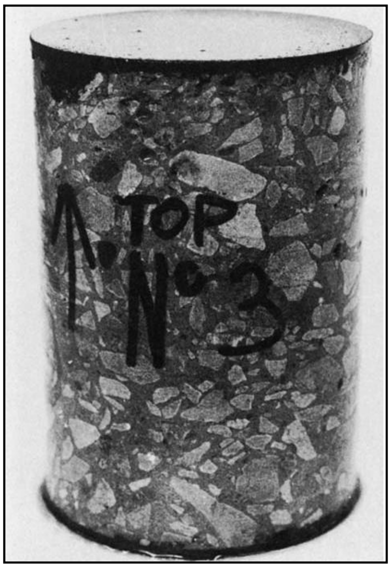


Figure III-11. Typical core

Each core must be trimmed and the ends either ground or capped before visual examination, assessment of voidage, and density determinations [15].

Aggregate type, size and characteristics should be assessed together with grading. These are usually most easily seen on a wet surface, but for other features to be noted, such as aggregate distribution, honeycombing, cracks, defects and drilling damage, a dry surface is preferable. Precise details of the location and size of reinforcement passing through the core must also be recorded. The voids should be classified in terms of the excess voidage by comparison with 'standard' photographs of known voidage provided by Concrete Society Technical Report 11 (1987). These reference photographs are based on the assumption of a fully compacted 'potential' voidage of 0.5%. This estimated value of excess voidage will be required when attempting to calculate the potential strength. If a more detailed description of the voids is required, this should refer to small voids (0.5-3 mm), medium voids (3-6 mm) and large voids (>6mm) with the term 'honeycombing' being used if these are interconnected. It is also helpful to describe whether voids are empty, or the nature of their contents, for example white gel from ASR [15].

Trimming, preferably with a masonry or water-lubricated diamond saw, should give a core of a suitable length with parallel ends which are normal to the axis of the core. If possible, reinforcement and unrepresentative concrete should be removed [15].

Unless their ends are prepared by grinding, cores should be capped with high alumina cement mortar or sulfur–sand mixture to provide parallel end surfaces normal to the axis of the core. (Other materials should not be used as they have been shown to give unreliable results.) Caps should be kept as thin as possible, but if the core is hand trimmed they may be up to about the maximum aggregate size at the thickest points [15].

This is recommended in all cases, and is best measured by the following procedure (Tech. Rept. 11,1987) [15] :

(i) Measure volume (Vu) of trimmed core by water displacement

(ii) Establish density of capping materials (Dc)

(iii) Before compressive testing, weigh soaked/surface-dry capped core in air

and water to determine gross weight W_t and volume V_t

(iv) If reinforcement is present this should be removed from the concrete

after compression testing, and the weight W_s and volume V_s determined

(v) Calculate saturated density of concrete in the uncapped core from

$$Da = \frac{Wt - Dc(Vt - Vu) - Ws}{Vu - Vs}$$

If no steel is present, W_s and V_s are both zero.

The value thus obtained may be used, if required, to assess the excess

voidage of the concrete using the relationship

$$\text{estimated excess voidage} = \frac{Dp - Da}{Dp - 500} \times 100\%$$

where D_p = the potential density based on available values for 28-day-old

cubes of the same mix. And Da is the actual density.

The standard procedure in the United Kingdom is to test cores in a saturated condition, although in the USA (ASTM C42) dry testing is used if the in-situ concrete is in a dry state. If the core is to be saturated, testing should be not less than two days after capping and immersion in water. The mean diameter must be measured to the nearest 1mm by caliper, with measurements on two axes at quarter- and mid-points along the length of the core, and the core length also measured to the nearest 1 mm. Compression testing will be carried out at a rate within the range 12–24N/(mm^2.min) in a suitable testing machine and the mode of failure noted. If there is cracking of the caps, or separation of cap and core, the result should be considered as being of doubtful accuracy. Ideally cracking should be similar all round the circumference of the core, but a diagonal shear crack is considered satisfactory, except in short cores or where reinforcement or honeycombing is present [15].

Although compression testing as described above is by far the most common method of testing cores for strength, recent research has indicated the potential of other methods which are outlined below. Two of these measure the tensile strength, although neither method is yet fully established. Tensile strength may also be measured by 'Brazilian' splitting tests on cores according to ASTM C42 [15].

These may be divided into two basic categories according to whether they are related to concrete characteristics or testing variables [15].

The moisture condition of the core will influence the measured strength

a saturated specimen has a value 10-15% lower than a comparable dry specimen. It is thus very important that the relative moisture conditions of core and in-situ concrete are taken into account in determining actual in-situ concrete strengths. If the core is tested while saturated, comparison with standard control specimens which are also tested saturated will be more straightforward but there is evidence (Bartlett,et all.1994) that moisture gradients within a core specimen will also tend to influence measured strength. This introduces additional uncertainties when procedures involving only a few days of either soaking or air drying are used since the effects of this conditioning are likely to penetrate only a small distance below the surface. The curing regime, and hence strength development, of a core and of the parent concrete will be different from the time of cutting [15].

This effect is very difficult to assess, and in mature concrete may be ignored, but should be considered for concrete of less than 28 days old. Voids in the core will reduce the measured strength, and this effect can be allowed for by measurement of the excess voidage when comparing core results with standard control specimens from the point of view of material specification compliance. Figure3.26, based on reference (Tech. Rept. 11,1987), shows the influence of this effect. Under normal circumstances an excess voidage of 0.5-1.0% would be expected. Higher values imply increasingly poorer compaction and should certainly be less than 2.5% [15].

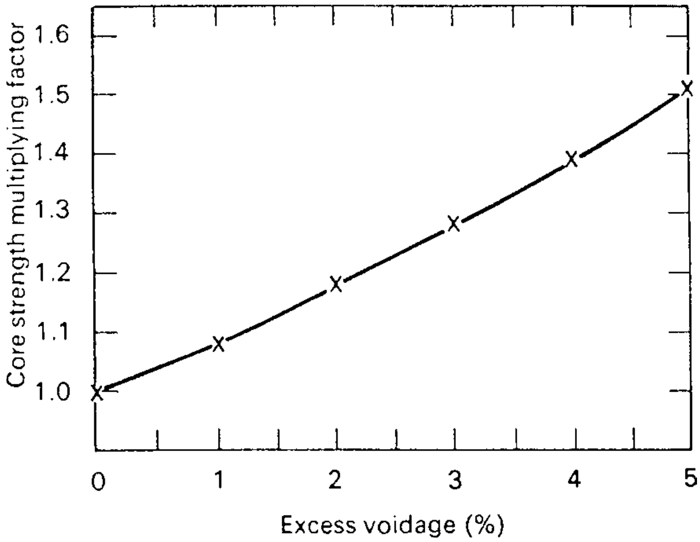


Figure III.12.. Excess voidage corrections

(i) Length/diameter ratio of core. As the ratio increases, the measured strength will decrease due to the effect of specimen shape on stress distributions whilst under test. Since the standard cylinder used in many parts of the world has a length/diameter ratio of 2.0, this is normally regarded as the datum for computation of results, and the relationship between this and a standard cube is established. Monday and Dhir (Monday,et all.1984) have indicated the influence of strength on length/diameter effects and this is confirmed by Bartlett and Macgregor (Bartlett,et all,1994) who also indicate the influence of moisture conditions. It is claimed that correction factors to an equivalent length/diameter ratio of 2.0 will move towards 1.0 for soaked cores and as concrete strength increases. The authors have also demonstrated the influence of aggregate type when lightweight aggregates are present (Bungey,et all,1994). This issue is widely recognized to be subject to many uncertainties, but the average values shown in Figure 5.5 are based on the Concrete Society recommendations (Tech. Rept. 11,1987). These differ from ASTM C42 suggestions which recognize, but do not allow for, strength effects and are also limited to cylinder strengths in the range 13-41N/mm^2 [15].

(ii) Diameter of core. The diameter of core may influence the measured strength and variability. Measured concrete strength will generally decrease as the specimen size increases; for sizes above 100mm this effect will be small, but for smaller sizes this effect may become significant.

However, as the diameter decreases, the ratio of cut surface area to volume increases, and hence the possibility of strength reduction due to cutting damage will increase. It is generally accepted that a minimum diameter/ maximum aggregate size ratio of 3 is required to make test variability acceptable [15].

(iii) Direction of drilling. As a result of layering effects, the measured strength of specimen drilled vertically relative to the direction of casting is likely to be greater than that for a horizontally drilled specimen from the same concrete. Published data on this effect are variable, but an average difference of 8% is suggested (Tech. Rept. 11,1987) although there is evidence that this effect may be influenced by concrete workability (Lesinskij,et all.2002) and is not found with lightweight aggregate concretes (Bungey, et all,1994). Whereas standard cylinders are tested vertically, cubes will normally be tested at right angles to the plane of casting and hence can be related directly to horizontally drilled cores [15].

(iv) Method of capping. Provided that the materials recommended, their strength is greater than that of the core, and the caps are sound, flat, perpendicular to the axis of the core and not excessively thick, the influence of capping will be of no practical significance [15].

(v) Reinforcement. Published research results indicate that the reduction in measured strength due to reinforcement may be less than 10%, but the variables of size, location and bond make it virtually impossible to allow accurately for this effect. Reinforcement must therefore be avoided wherever possible, but in cases where it is present the measured core strength may be corrected but treated with caution [15].

The likely coefficient of variation due to testing is about 6% for carefully cut and tested cores, which can be compared with a corresponding value of 3% for cubes. The difference is largely caused by the effects of cutting, especially since cut aggregate particles are only partially embedded in the core and may not make a full contribution during testing. It is claimed that the likely 95% confidence limits on actual strength prediction for a single core are ±12% when the Concrete Society calculation procedures (Tech. Rept. 11,1987) are adopted. It follows that for a group of n cores, the 95% confidence limits on estimated actual in-situ strengths are $\pm 12/\sqrt{n}\%$. Where the 'potential' strength of the concrete is to be assessed, a minimum of four cores is required and an accuracy of better than ±15% cannot be expected. This can only be achieved if great care is taken to ensure that the concrete tested is representative, by careful location and preparation of the specimens. Uncertainties caused by reinforcement, compaction or curing may lead to an accuracy as low as ±30% [15].

CHAPTER IV

INSPECTION OF BRIDGES AND BRIDGE MANAGEMENT SYSTEM

CHAPTER IV

INSPECTION OF BRIDGES AND BRIDGE MANAGEMENT SYSTEM

1. INSPECTION OF BRIDGES

1. 1 AMERICAN CLASSIFICATION OF TYPES OF INSPECTION OF RC BRIDGES

There are five general types of bridge inspections: Initial, Routine, Damage, In-depth, and Special Inspections. Additional specific access-related inspection types such as Confined Space, Fracture Critical, Underwater, High-water, etc. are discussed. The scope, intensity, and frequency of various types of general bridge safety inspections are discussed here to provide a better understanding of the purpose and use of each inspection type and to assist in the development of scope of inspection work for individual inspections [26].

An inspection event, particularly for large, complex, or deficient structures, often requires that a variety of inspection types be performed, using a variety of methodologies.

1.1.2 Initial inspections

An Initial Inspection is the first inspection of a new or existing structure, that is, when it becomes part of the bridge inventory. Additionally, reconstructed structures may also require an Initial Inspection to document modifications of the structure's type, size, or location.

The purpose of the Initial Inspection is to verify the safety of a bridge, in accordance with the NBIS and Department standards, before it is put into service. It also serves to provide required inventory information of the as-built structure type, size, and location for BMS (and the NBI) and to document its structural and functional conditions by:

- Providing all Structure Inventory & Appraisal (SI&A) data required by Federal regulations along with all other data required by Department standards and the local owner.
- Determining baseline structural conditions and eliminate deficiencies recorded under previous structural assessments.
- Clearance envelopes (for features carried and those intersected) and bridge waterway openings are to be documented at this time.
- Identifying maintenance needs, including preventative maintenance activities.

- Noting the existence of elements or members requiring special attention, such as fracture critical members, fatigue-prone details, and underwater members.
- Verify construction/rehabilitation contracts.

Documents, including but not limited to, photographs, drawings (design, as-built and shop drawings), scour analysis, foundation information, hydrologic and hydraulic data are to be inserted into the bridge file. Selected construction records (e.g., pile driving records, field changes, etc.) may also be of great use in the future and should be included. Include maintenance records for existing bridges [26].

The level of effort required to perform an Initial Inspection will vary according to the structure’s type, size, design complexity, and location. An Initial Inspection is to be a close-up, hands-on inspection of all members of the structure to document the baseline conditions. Traffic control and special access equipment may be required. Initial Inspections are performed for each structure after construction is essentially complete and before the bridge is put into service (or returned to service for bridges that have had a major reconstruction). Bridges open to traffic during construction operations are required to be inspected [26].

1.1.3 Routine inspections

Routine Inspections provide documentation of the existing physical and functional conditions of the structure. All changes to BMS inventory items that have occurred since the previous inspection are also to be documented and updated into BMS. The written report will include appropriate photographs and recommendations for major improvements, maintenance needs (preservation, preventative maintenance or On Demand repairs), and follow-up inspections. Load capacity analyses are re-evaluated only if changes in structural conditions or pertinent site conditions have occurred since the previous analyses.

Routine Inspections serve to document sufficient field observations/ measurements and load ratings needed to:

- Determine the physical and functional condition of the structure.
- Identify changes from the previously recorded conditions.
- Determine the need for establishing or revising a weight restriction on the bridge.
- Determine improvement and maintenance needs.
- Ensure that the structure continues to satisfy present service and safety requirements.
- Identifying and listing concerns of future conditions.
- Identify any inventory changes from the previous inspection.

The level of scrutiny and effort required to perform a Routine Inspection will vary according to the structure’s type, size, design complexity, existing conditions, and location. Generally, every element in a bridge does not require a hands-on inspection during each.

Routine Inspection to provide an acceptable level of assurance of the bridge’s on-going safety. The difficulty is that the areas not needing close-up scrutiny cannot always be absolutely determined until after the entire bridge has been inspected and non-critical areas identified. Accordingly, to provide a reasonable level of confidence in the safety of the bridge, knowledge of the structure and good engineering judgment are necessary when considering those portions that will not receive the close-up scrutiny with each inspection.

The following guidance is offered when determining the level of scrutiny needed for adequate inspection of individual bridges: Areas/elements that may be more difficult to access but that warrant hands-on inspection in each Routine Inspection, may include, but are not limited to:

- Load carrying members in Poor condition,
- Redundancy retrofit systems,
- Critical sections of controlling members on posted bridges,
- Scour critical substructure units,
- End regions of steel girders or beams under deck joints,
- Cantilever portions of concrete piers or bents in Fair or worse condition,
- Ends of Prestressed concrete beams at continuity diaphragms when warranted,
- Pin and Hanger / Hinge assemblies,
- Other areas determined by the Program Manager of the inspection to be potentially critical.

Routine Inspections are generally conducted from the deck, ground and/or water levels, ladders and from permanent work platforms or walkways, if present. Inspection of underwater members of the substructure is generally limited to observations during periods of low flow and/or probing/sounding for evidence of local scour.

Routine Inspections are regularly scheduled inspections performed once each calendar year. No routine inspection shall occur outside of an 18-month interval since the previous inspection [26].

1.1.4 In-depth inspections

An In-Depth Inspection is a close-up, hands-on inspection of one or more members and a close visual of all members above or below the water level to identify any deficiency not readily detectable using Routine Inspection procedures. An In-Depth Inspection may be limited to certain elements, span group(s), or structural units of a structure, and need not involve the entire structure. Conversely, In-Depth Inspections may include all elements of a structure. In-Depth Inspections can be conducted alone or as part of a Routine or other type of inspection.

In-Depth inspections serve to collect and document data to a sufficient detail needed to ascertain the physical condition of a bridge. This hard-to-obtain data is more difficult to collect than data collected during a Routine Inspection.

In-Depth Inspections should be routinely scheduled for selected bridges based on their size, complexity and/or condition. Major or complex bridges represent large capital investments and warrant closer scrutiny to ensure that maintenance work is identified and completed in a timely manner. These bridges tend to be more critical to local and area transportation because of the usual lack of suitable detours. It may be more difficult to provide a complete a snapshot of the bridge conditions when access difficulties limit the scope of Routine Inspections.

Scope and Frequency of In-Depth Inspections The level of effort required to perform an In-Depth Inspection will vary according to the structure's type, size, design complexity, existing conditions, and location. Traffic control and special equipment, such as under bridge cranes, rigging, or staging may be needed for In-Depth Inspections. Personnel with special skills such as divers and riggers may be required. Non-destructive field tests and/or material tests may be performed to fully ascertain the existence of or the extent of any deficiency. On small bridges, the In-Depth Inspection, if warranted, should include all critical elements of the structure.

For large or complex structures, these inspections may be scheduled separately for defined segments of the bridge or for designated groups of elements, connections or details that can be efficiently addressed by the same or similar inspection techniques. If the latter option is chosen, each defined bridge segment and/or each designated group of elements, connections or details should be clearly identified as a matter of record and should be assigned a frequency for re-inspection. The activities, procedures, and findings of In - Depth Inspections shall be completely and carefully documented more than those of Routine Inspections. Stated differently, In-Depth Inspection reports will generally be detailed documents unique to each structure that exceed the documentation of standard or routine inspection forms.

A structural analysis for load carrying capacity maybe required with an In-Depth inspection to fully evaluate the effect of the more detailed scrutiny of the structure condition.

An In-Depth Inspection can be scheduled in addition to a Routine Inspection, though generally at a longer interval, or it may be a follow-up to a previous inspection. An In-Depth Inspection that includes all elements of the structure will satisfy the requirements of the NBIS and take the place of the Routine Inspection for that cycle.

In-Depth Inspections do not reduce the level of scrutiny for Routine Inspections. Program Managers shall schedule In-Depth Inspection based upon condition and importance. For example, major bridges shall receive an In-Depth Inspection every five years when: the routine, fracture critical, damage, dive or special inspections determine that a more detailed evaluation is necessary. Increased intervals are up to the discretion of the Program Manager.

1.1.5 Damage inspections

Damage Inspections are performed following extreme weather-related events, earthquakes, vandalism and vehicular/marine traffic crashes, as directed by the District Bridge Engineer. When major damage has occurred, the Inspectors will need to evaluate fractured or failed members, determine the amount of section loss, take measurements for misalignment of members, check for any loss of foundation support, etc.

Damage Inspections serve to determine the nature, severity, and extent of structural damage following extreme weather-related events and vehicular and marine traffic collisions/accidents for use in designing needed repairs. Damage Inspection findings shall be used to determine the immediate need to place an emergency restriction on a bridge (e.g., weight restriction or closure) for vehicular traffic. If a bridge is closed to vehicular traffic, the need to close it to pedestrian traffic shall also be determined.

The findings of a Damage Inspection may be used to re-coup the costs of inspection and needed repairs or reconstruction from involved parties or other governmental agencies. Accordingly, documentation of the inspection may be critical in these efforts. For Department bridges, the extent of damage and estimated costs of repair should be reported to the district damage coordinator. Photographs, videos and sketches can be extremely helpful.

The amount of effort expended on this type of inspection will vary significantly depending upon the extent of the damage, the volume of traffic encountered, the location of the damage on the structure, and documentation needs. The scope of a Damage Inspection must be sufficient to determine the need for emergency load restrictions or closure of the bridge to traffic, and to estimate the level of effort necessary to accomplish repairs. The capability to make an on-site determination of the need to establish emergency load restrictions may be necessary.

A Damage Inspection is an unscheduled inspection to assess the structural damage resulting from environmental factors or human actions. Damage Inspections are performed on an as-needed basis.

1.1.6 Special inspections

Special Inspections are scheduled by the Bridge Owner to examine bridges or portions of bridges with known or suspected deficiencies. Special Inspections tend to focus on specific areas of a bridge where problems were previously reported or to investigate areas where problems are suspected. Special Inspections generally are not comprehensive enough to fulfill NBIS requirements for Routine Inspections. Special Inspections can be structured to fulfill the need for interim inspections between the 12-month routine inspections. Special Inspections are conducted until corrective actions remove critical deficiencies.

Special Inspections are used to monitor particular known or suspected critical deficiencies, fulfill the need for interim inspections (i.e., reduced inspection interval for posted bridges), and to investigate bridge conditions following a natural disaster or manmade emergency.

The level of effort required to perform a Special Inspection will vary according to the structure’s type, size, design complexity, existing conditions, and type of deficiency being investigated. The Program Manager defines the scope and frequency of the Special Inspections. The qualified Inspector performing a Special Inspection should be carefully instructed regarding the nature of the known deficiency and its functional relationship to satisfactory bridge performance. Guidelines and procedures on what to observe and/or measure must be provided. A timely process to interpret the field results by a Professional Engineer is required.

The determination of an appropriate scope and frequency for a Special Inspection frequency should consider the nature, severity and extent of the known deficiency, as well as age, traffic characteristics, public importance, and maintenance history. Special Inspections are typically at intervals shorter than 12 months.

1.1.7 Combined sewer system inspections

Culvert and drainage structures that meet the definition of a bridge will be considered a bridge culvert.

Combined sewer systems will be inventoried and inspected. The portion of the combined sewer defined as the bridge shall have an interior visual inspection required every five years. An annual inspection report (BR-86) will be required for each year. Note these structures are typically considered confined space.

Large-span multi-plate culverts, including box culverts, arches, pipe-arches, and circular pipes are relatively flexible soil interaction structures and more susceptible to failure when they lose their original global cross-sectional geometry. The inspection of these multi-plate culverts is to be sufficiently detailed to detect and monitor deformations (e.g., bulging; non-uniformity of the arch soffit, longitudinally or transversely; misalignment of plates; tearing; etc.) that could lead to a partial or complete collapse of the structure. Culverts under shallow earth fill are especially vulnerable to such deformations.

Bridge Inspectors will monitor the integrity of the culvert ‘s shape as the primary indicator of any structural distress. The bridge file shall contain sketches indicating the as-built geometry and subsequent measurements to monitor the structure ‘s performance at a minimum of two cross-section locations. Paint marks on the culvert will assist the Inspectors in ensuring measurements are taken at consistent locations.

1.1.8 Confined space inspection

NOTE: These are the Department guidelines for the treatment of confined space. Owner may elect to follow the department’s guideline. However, each agency shall be responsible for its own confined space program.

Entry of some bridge components (hollow piers, steel pier caps, box type superstructures) or culvert type bridges may pose OSHA requirements with regard to confined spaces. Therefore, entry of these items may include additional challenges with requirements for personal protective equipment and following the protocols of the Ohio Department of Transportation Confined Space Entry Program, and the Alternate Entry Procedures for bridge inspection.

Any bridge owner employee or consultant entering a confined space using Alternate Entry Procedures or the Confined Space Entry Procedures must have successfully completed a Confined Space training course.

Depending on their size and configuration, bridge components or culverts may meet the definition of being considered a confined space per OSHA (29CFR1910.46). Therefore, inspection procedures will vary with regard to the safety measures used. Entry Classes have been established for inventory requirements and to detail the entry requirements for the Inspector.

All structures classified as confined space by OSHA (29CFR1910.46) or this manual shall have documentation on entry types, dates, noted changes from last inspection, and atmospheric conditions. The Program Manager is responsible for maintaining a list of structures designated as confined space or components designated as confined space. Bridge files shall include all data of past entries and visual survey conducted by the inspector noting atmospheric conditions and physical hazards.

Some culverts qualify as Permit Required Confined Spaces because they may contain or have the potential to contain a hazardous atmosphere. Due to their stable nature, culverts generally do not contain physical threats such as the potential to trap or engulf an entrant. When the only hazard is atmospheric, alternate entry procedures may be followed.

No structure with confined space shall go without a visual inspection greater than 72 months. A bridge inspection report will be required on an annual basis. The inspection report shall document the last time the confined space was entered. Structures that are fully or partially collapsed or have significant infiltration of backfill material or water pose an additional physical threat and should not be entered. If entry is required, the full requirements of the Ohio Department of Transportation Confined Space Entry Program shall be followed.

Class A (Non-Entry Inspection) - Class A inspections involve gathering inventory and inspection information without entering the structure. The inspector will examine the structure from the openings, noting as much information as possible from a visual check. Class A inspections can be performed on any culvert; however, consideration should be given to extremely long structures or culverts with multiple bends which prohibit obtaining a good view of the entire barrel. An entry inspection is recommended for those culverts. If structural or other defects are noticed during the non-entry inspection, further investigation via manned-entry or video inspection may be required.

Class B (Non-Permit Required Entry) - Class B inspections are arms-length inspections performed on bridges/culverts that require no special provisions for confined space issues. An air monitor is required at all times while in the confined space.

Class C (Alternate Entry Permit Required) - Class C entry requires the structure to have no known history of atmospheric or physical hazards. Class C inspections are inspections performed on the structure that require Alternate Entry Procedures to be followed. The inspector should review the bridge file prior to each inspection. Contact the county maintenance forces to inquire about any potential problems or changes that may exist at the site. An air monitor is required at all times while in the confined space. See Ohio Department of Transportation Confined Space Entry Program and the Alternate Entry Procedures for bridge inspection for details.

Class D (Permit required) - Class D structures require the full use and implementation of the Ohio Department of Transportation Confined Space Entry Program.

1.1.9 Inspection of bridges over water

Nationwide, more bridges are lost each year due to scour than any other reason. Many times, these bridge losses occur during regional or localized flooding and their loss from the transportation system can make recovery from the original weather event even more difficult. One of the more effective ways of preventing the loss of a bridge due to scour failure is to identify those bridges most likely to be vulnerable to scour. With this determination, called a scour assessment, the bridge Inspectors and owners can concentrate inspection/monitoring efforts and remedial actions to mitigate conditions at bridges with critical vulnerability.

The main purpose of the scour assessment of an existing bridge is to determine whether the bridge is vulnerable to scour. A scour critical bridge is one whose foundation(s) has been determined to be unstable for the predicted scour conditions. To combat the loss of structures from the transportation system and protect our valued infrastructure, Ohio uses a tiered approach:

1. Assess and prioritize the bridges vulnerability to scour so that critical bridges can be identified for closer monitoring and possible implementation of scour countermeasures.
2. Complete a field review, including a scour vulnerability analysis, to verify the integrity of foundations and identification of structures requiring closer monitoring and anti-scour maintenance.
3. Complete a detailed scour analysis of bridges that are very susceptible to scour and where additional monitoring may be required.

1.1.10 Under water inspections

The purpose of underwater inspections is to provide information on underwater portions of a bridge to evaluate their overall safety and, especially, to assess the risk of failure due to scour. Underwater inspections are required in water >5’ deep at least once every 60 months.

Note: if a dive inspection is required and low flow allows the inspector to probe the entire substructure unit then the dive inspection date may be reset.

During periods of low flow, underwater members will be inspected visually and by feel using probing rods, sounding lines, or other hand tools. When the physical condition of the substructure members or the integrity of their foundations cannot be determined using the probing tools due to high water, high flow, turbidity, etc., inspection by divers is required. New technology, including ground sensing radar, ultrasonic techniques, remote video recorders, and others are useful aids for underwater inspections of substructure foundations for limited situations.

Key information to be determined in every underwater inspection (either by probing or diving) is the top of streambed relative to the elevation of the substructure foundations. Because scour can vary significantly from one end of a footing to the other, a single probing reading is not sufficient. Baseline streambed conditions should be established by waterway opening cross sections and by a grid pattern of probing readings around the face of a substructure unit. This baseline information is essential for future monitoring and assessment. The current streambed conditions and changes since the last inspection are critical inputs to the bridge scour assessment.

Each bridge should have local benchmarks established near each substructure unit to enable Inspectors to quickly and accurately determine the depth of adjacent scour. These benchmarks can be as simple as a painted line or PK survey nail driven into the wall in a place visible during high water. The location of these scour-monitoring benchmarks should be referenced in the inspection records and Bridge file. Use previously established benchmarks, when possible, to provide a long-term record of scour conditions. If new benchmarks need to be established, provide conversion from new to old datum.

During Routine Inspections, particular attention should be given to foundations on spread footings where scour or erosion can be much more critical than at deep foundations on piles or caissons. However, be aware that scour and undercutting of a pier or abutment on a deep foundation can also be quite serious. The foundation’s vertical support capacity normally will not be greatly affected unless the scour is excessively severe, but the horizontal stability may be jeopardized. This condition becomes particularly unstable when erosion has occurred on only one face of the substructure unit, leaving solid material on the opposite face. Horizontal loads may also have debris, or rock fills piled against or adjacent to substructure units whose loads were obviously not provided for in the original design. Such unbalanced loading can produce an unstable condition, requiring corrective action.

BMS and Underwater Inspections: The Bridge Management System uses inventory items to record each underwater inspection and to verify Ohio's compliance with the underwater inspection reporting requirements of NBIS. The date of the underwater inspection must be entered into the BMS or coded on a BR-87 and submitted to the Central Office, Office of Structural Engineering.

Underwater inspections are intended to investigate two critical issues regarding the condition of bridge substructures located in water:

- The condition of structural components (including pier shaft, abutment walls, footings, etc.) under water.
- The integrity of the substructure foundation (including underlying soil, piles, caissons, etc.) against scour at each substructure unit in water.

The inspection of the foundation of a substructure unit and the determination of its ongoing resistance to scour is critical for the overall safety of the bridge. Because the integrity of the foundation against scour can suddenly and dramatically change in a relatively short time (as compared to physical condition of the structure components), shorter intervals for inspection of the foundation are warranted. The recommended intervals for underwater inspection of the foundation of substructure units for bridges over water are based upon a scour assessment of each unit. The condition of the structural components can routinely be verified during the investigation of the foundation material. All bridges with substructure elements submerged greater than five feet in depth are to have an underwater inspection. The frequency of underwater inspection of a substructure unit is not to exceed 5 years (60 months).

1.1.11 High water inspections

The Program Manager is to establish an internal procedure to monitor scour critical bridges during or immediately after periods of high water. The following elements are recommended for consideration as part of the procedures:

- A list and, preferably a map, of scour critical bridges that are to be monitored during periods of high water. Other bridges that are not classified as scour critical but that may have scoured previously or that may be susceptible to debris and aggradation should be considered for inclusion.
- Because high stream flows can be very localized and information about its severity and extent may not be immediately available, a method of reporting the occurrence and extent of high water is needed. Many times, the first responders are maintenance forces, they can be trained to report high water events to the program manager. This method is useful for prioritizing structures to be checked by bridge Inspectors.
- Local benchmarks established at scour critical bridges can enable non-bridge Inspectors to record and report the height of water. The list of scour critical bridges could also indicate the location of the benchmarks and the water heights at which scour inspections are warranted. In addition, the benchmarks enable Inspectors to quickly gauge the progress of scour at a substructure.

- A high-water inspection plan can improve the Program Managers response, especially in times of area-wide flooding where inspection resources may be limited [26].

1.1.12 Non-highway bridges and structures over state routes

This Section is applicable to all non-highway bridges and structures, except railroad bridges and sign structures, over State Routes. For the purposes of this manual the term “overhead bridge” will be used to encompass all types of nonhighway bridges and structures.

Inventory Requirements: NBIS requires that all bridges or structures greater than 20ft. in length over Public Roads are to be inventoried and their data stored in the Department ‘s BMS. All bridges, regardless of their length, over State Routes are to be inventoried and their data stored in BMS [26].

The inspection of these non-highway bridges is similar to Routine Inspections of highway bridges.

Because of the many types and features of existing overhead bridges, this Section cannot list a complete set of specific inspection requirements.

Fracture critical inspections are not required for bridges not carrying “highway” traffic. Also load ratings for non-highway bridges are required for the type of loads the structure will be carrying. If appropriate, underwater inspection requirements for substructures should be included. Overhead bridge safety inspection reports must be signed and sealed by a Professional Engineer.

For longer bridges and structures, the inspection report to the Department may be limited to only those spans over the highway ROW and the substructure units supporting those spans. The District Bridge Engineer must approve the elimination of portions of a bridge from these inspection requirements. Bridge owners are encouraged, but not required, to inspect remaining portions with the same intensity. For building-to-building passageway bridges, the structural components may be covered by siding, masonry, etc. that would interfere with an inspection using normal bridge techniques. These architectural facades also prevent the deterioration normally suffered by bridge components exposed to the weather.

The scope of these inspections must be developed on a case-by-case basis. Safety inspection reports and data of all bridges over State Routes must be submitted to the Department for its review and acceptance. While this Section was developed for bridges over State Routes, other roadway owners are encouraged to adopt it for use for non-highway bridges over their roadways [26].

All bridges and structures, not including sign structures, over State Routes are to have a bridge safety Inspection every calendar year and should be on a frequency no greater than 18 months. The Program Manager may require inspections more frequently than 12 months if structure and/or site conditions warrant [26].

1.2. AUSTRALIAN CLASSIFICATION OF TYPES OF INSPECTION OF RC BRIDGES

The routine inspection is a diagnostic method with the greatest potential and is generally based on direct visual observation of a bridge's most exposed areas. It relies on subjective evaluations made by the bridge inspectors. During an inspection, no significant structural defect is expected and the work recommended falls within the range of maintenance [27].

A period of fifteen months between routine inspections is recommended so that the influence of the weather on the general condition and degradation of the bridge can be assessed (Andrey, 1987). A routine inspection must be planned in advance to facilitate the best assured conditions (e.g., weather conditions, traffic) that may permit detection of defects (Branco & de Brito, 2004).

Easy and fast nondestructive in situ tests are performed in detailed inspection in addition to direct visual observation as a way of exploring every detail that may potentially lead to future problems. There is a possibility that special means of access may be used if such is considered indispensable. The period recommended for a detailed inspection is five years and replaces a routine inspection if the inspector's calendars agree (Andrey, 1987). A preliminary visit to the bridge site may be useful to evaluate existing conditions. If there is a need to follow up the evolution of certain defects with greater frequency, however, the period between visits may be reduced to one year, especially for local areas of the bridge [27].

According to Branco and de Brito (2004) planning a detailed inspection includes a careful study of a bridge dossier to get to know the reasons and evolution of the defects detected in the previous inspections and the specific points to be assessed closely. Based on previous inspection forms and a preliminary visit to the site, the eventual special means of access needed are planned. The following files must be brought to the site and/or prepared beforehand: a list of all single points to be checked, schematics with reference grids of the most relevant elements, and the last periodic inspection form and the inspection manual [27].

According to the outcomes obtained, the inspection may possibly have one of the following consequences (Andrey, 1987): the organization of a structural assessment or of complementary surveillance measurements; the preparation of a list with particular aspects to follow especially carefully in the next inspection; the organization of maintenance work needed; and the establishment of a medium-term maintenance plan (Branco & de Brito, 2004).

A structural assessment is normally the consequence of the detection of a major structural or functional deficiency during a routine or detailed inspection. It may also be necessary if widening the deck or strengthening the structure is under consideration. The expected results from this inspection are: the characterisation of the structural shortcomings, the remaining service life estimation by using degradation mathematical models, and also evaluation of its present load-bearing capacity. It is not easy to predict the required means because a wide range of situations can initiate a structural assessment. The static and dynamic load tests and laboratory tests can be valuable complements to the information collected in situ. Nevertheless, they must be used with some parsimony since, as well as being expensive, they force the total interruption of traffic over the bridge for uncertain periods of time (Andrey, 1987). The final report of the structural assessment must include the index, structural identification form, schematic drawing of the bridge, structure general condition standard form, summary of the most significant results, equipment used and calibration sheets, photos and schematic representations of the cores, identification and description of the cores, identification and description of the asphalt surface samples, photos and drawings. All the data collected are dated and appended to the bridge dossier (OMT, 1988) [27].

1.3. GERMAN CLASSIFICATION OF TYPES OF INSPECTION OF RC BRIDGES

The Federal Department of Transportation, Construction, and Housing’s Office of Road Construction and Traffic (German Federal Ministry of Traffic, Building, and Urban Affairs) has issued two documents, DIN 1076: Engineering Structures in Connection with Roads-Inspection and Test and Directive for Uniform Determination Assessment, Recording, and Analysis of the Results of the Inspection of the Structures in Table 6.1. Classification of cracks in concrete structures and recommended repair procedures, from the Finnra Bridge Inspection Manual and damages classes from A to D [28].

Class A.

Surface treatment may be considered. A special inspection shall be undertaken to determine the degree of reinforcement corrosion as well as the chloride concentration and depth of carbonation. The surface treatment must be able to withstand minor structural deformation. A specification shall be drawn up.

Class B.

The cracks in the upper surfaces are soaked using capillary action. Other cracks are injected as needed. A leaking crack must always be injected. A specification shall be drawn up.

Class C.

A special inspection shall be undertaken to determine the cause of cracking. The cracks are injected using epoxy to restore original structural strength. Leaking cracks place special demands on the epoxy and the work method to be used. The effect of cracks on the condition of tendons in prestressed structures must be determined. A specification shall be drawn up.

Class D

The reason for cracking is determined through a special inspection. The cracks are injected using epoxy or cement slurry with filler added as needed. Calculations are used to determine the need for additional strengthening of structures and possible service limitations. A special inspection is carried out and a repair plan is drawn up. In the case of prestressed structures, the effect of the damage on tendons and cables must be determined.

Table IV-1. Classification of cracks in concrete structures and recommended repair procedures, from the Finnra Bridge Inspection Manual.

damage class	type of structure damage	superstructure		Other structure	special stress	
		normal reinforcement	prestressed reinforcement		Edge beam	water level range
1	Crack width is under 0.2 mm. Cracks are small, mainly surface cracks.	A	A	-	B	-
2	Crack width is 0.2 to 0.4 mm. Cracks are small structural cracks, generally due to shrinkage.	B	C	B	B	-
3	Crack width is 0.3 to l.0 mm. Structural cracks are generally due to deflection, exceeding of the shear capacity, or creep. Cracks are generally found in the superstructure.	C	D	C	C	C
4	Crack depth is over 1.0 mm. Structural cracks are due to uneven settlement or a large deformation. Cracks are often serrated and generally found in the substructure.	D	D	D	D	D

Accordance with DIN 1076. These documents provide detailed guidance on documentation of inspection and testing performed during bridge inspections. The scan team observed host nation inspectors with photographs from past inspections on site to use in current inspections. This practice allows the inspector to make more accurate observations of changes in bridge conditions since the last inspection. In Germany, digital photographs are imbedded in the final report, along with report text and associated sketches. Inspection vehicles in Germany were fully able to support activities at the inspection site. A maintenance repair and rehabilitation specialized truck was modified to incorporate office workspace for the inspector that included a desk, a laptop computer (including inspection program SIB-Bauwerke), a reference library complete with all pertinent inspection references, and a complete set of bridge records for the bridges being inspected [28].

Bridge inspections in Germany are defined as follows: Major inspections involve visual inspection and testing (material investigations) of all parts of a structure by inspection engineers. Generally, they are conducted every 6 years. Damage and condition assessment are performed according to RI-EBW-PRÜF, Directive for Uniform Determination, Assessment, Recording, and Analysis of the Results of the Inspection of the Structures in Accordance with DIN 1076. The first major inspection is performed before the structure is opened to traffic and the second major inspection is done before the end of the guarantee period. Minor inspections, conducted every 3 years, are visual inspections by inspection engineers to check the results of the major inspection. Ad hoc inspections are performed by engineers to obtain an in-depth view of a particular damage or deterioration process that has occurred at the bridge (accidents, flooding, etc.). Inspection in accordance with other regulations and standards may be required of machinery and electrical equipment forming part of highway structures, especially movable facilities and gantries.

Superficial inspections are performed by maintenance personal. These types of inspections require no special knowledge of highway structures. The objectives are to detect major visible faults, check the functionality of components on a quarterly basis (visual), and perform an annual inspection of all accessible parts. Routine safety monitoring is performed on an ongoing basis by maintenance personnel as part of their routine superficial inspection of the highway [28].

The assessment, repair and routine inspection of the seven bridges will be reviewed in the chapter 7 and chapters 8,9.

2. BRIDGE MANAGEMENT SYSTEM

A bridge management system (BMS) is a rational and systematic approach to organizing and carrying out all the activities related to providing programs for bridges vital to the transportation infrastructure. The activities include: (1) predicting bridge needs, (2) defining bridge conditions, (3) allocating funds for construction, replacement, rehabilitation, and maintenance actions, (4) identifying and prioritizing bridges for maintenance, rehabilitation, and replacement (MR&R) actions, (5) identifying bridges for posting, (6) finding cost-effective alternatives for each bridge, (7) recommending MR&R actions, (8) accounting of MR&R actions, (9) scheduling and performing minor maintenance, (10) monitoring and rating bridges, and (11) maintaining an appropriate data base of information. A BMS should assist decision-makers at all bridge management levels to select optimum solutions from an array of cost-effective alternatives for every action needed to achieve the desired levels of service within the funds allocated and to identify future funding requirements [37].

2.1 AMERICAN BRIDGE MANAGEMENT SYSTEM

The Federal Highway Administration (FHWA) requires each state to provide information about each bridge in their inventory as described in the FHWA's Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges. This information is used to generate a National Bridge Inventory (NBI). The FHWA categorizes bridges as structurally deficient or functionally obsolete based on their condition and appraisal ratings. Bridge eligibility for rehabilitation and replacement is determined by a sufficiency rating formula. The NBI does not provide information that can be used to predict a bridge's future condition or provide an estimate on future maintenance and repair needs of an agency bridge inventory.

Recognizing that a different strategy towards future bridge preservation was needed, the National Cooperative Highway Research Program (NCHRP) published a report (Report 300) in December 1987 to provide the framework for a BMS. The overall objective of this report was to develop a model bridge management system that could be implemented by a state or local transportation agency. BMS is intended to ensure the effective use of available funds and identify the effects of various funding levels on an agency bridge network [34] .

BMS is designed to provide information not currently available from NBI data.

FHWA recommends agency use of BMS to provide comprehensive management of their Bridge system. BMS can provide the following:

- Improvements in the type and quality of data that is collected, stored, managed, and used in a bridge system analysis.
- A logical method for setting priorities for current needs.
- Realistic and reliable forecasts of future needs.

- Ways to implement changes in management philosophies and goals.

2.1.1. Common BMS bridge elements

BMS elements are commonly used in highway bridge construction and encountered on bridge inspections. These elements are labeled “Commonly Recognized” (CoRe) structural elements because of their nationwide recognition and use. A single BMS element can incorporate only those components of a bridge that:

- Are made of the same material.
- In normal service be expected to deteriorate in a very similar fashion and at a similar rate.
- Can be inventoried with units that are easily assessed by the inspector.

The AASHTO Guide for Commonly Recognized (CoRe) Structural Elements provides basic definitions for CoRe elements. In general, all girders, trusses, arches, cables, floor beams, stringers, abutments, piers, pins and hangers, culverts, joints, bearings, railings, decks and slabs are identified as CoRe elements. AASHTO CoRe elements contain a description, definition, condition state language, a unit of measurement and a feasible action. The condition of a CoRe element is identified by condition states and corresponding condition state language. Each CoRe element has a range of 3 to 5 condition states. The first condition state (#1) would represent a like new condition while the last condition state would represent the worst condition. The AASHTO definitions have been significantly modified for use by the State of Washington.

“Smart Flags” are used to flag unique problems not identified by BMS elements.

A Smart Flag can have multiple condition states. Smart-Flags do not have feasible actions associated with their condition states since the deterioration rate is not predictable. Examples of Smart Flags are Steel Fatigue (cracks in steel elements), Scour, Pack Rust, etc.

Information from each CoRe element along with expert input to predict how the condition of that element will change over time is used in BMS computer programs. The BMS programs can estimate future network funding levels based on the predicted future bridge conditions and the corresponding costs to repair or replace them [34].

2.1.2. BMS inspections

As previously stated, a BMS inspection is intended to supplement but not replace an NBI type inspection. The following outline provides a short BMS summary for a typical inspection and is discussed in the following paragraphs.

A. Identify the BMS elements that apply to the structure.

B. Determine the total quantity for each element.

C. Inspect Bridge and record the deficient quantity for each element in the corresponding condition state.

WSDOT’s Bridge Preservation Office uses a laptop computer program to allow an inspector to record information from both an NBI and a BMS type inspection [34].

A. Identify the BMS Elements

Details about the design of the bridge are important when identifying the BMS elements. As-built plans provide the best resource for choosing the correct elements. Elements can be defined with as-built plans in the office prior to the inspection and then field verified. If as-built plans are not available, then the elements will have to be defined at the bridge site. For example, let’s say a bridge has a reinforced concrete bridge deck. In order to determine if the BMS coding for the concrete deck should be element No.12 or element No. 26 the type of steel reinforcing needs to be identified. Plans and special provisions would note if the reinforcing steel in a concrete deck was epoxy coated or not. A field inspection could not accurately determine if the steel was epoxy coated thus if as-built plans are not available then the type of element would need to be assumed. It should be noted that epoxy coated rebar in bridge decks became an industry standard in Washington State in the early 1980’s. An average bridge made of the same material will have six to ten elements. A large or complex bridge may have up to 20 elements. A typical bridge will have a bridge deck, possibly a deck overlay, bridge rails, a primary load carrying member like a prestressed concrete girder, primary substructure support like concrete columns, other elements like abutments, expansion joints and/or bearings [34].

B. Determine the Total Quantity of the Element

The units to be used for each element are defined with the corresponding element. The units are listed as “SF” (square feet), or “LF” (lineal feet), or “EA” (each).

The “SF” value is used to determine the area of a deck element and the area of steel for paint elements. For bridge decks use the curb-to-curb width of the deck by the length to determine the deck area. The “LF” value is used to determine the total length of an element. The length of an element is based on the way it was constructed. For example: A bridge may have been built using five “Prestressed Concrete Girders.” Each one was individually pre-cast and then put into place at the bridge site. If each girder were 100 feet in length then the total element quantity would be “500 LF.” If the same bridge was a “Concrete Box Girder” then the total quantity would be “100 LF” since the box girder was constructed as one unit. The quantity for the abutment elements is determined by estimating the length along the abutment. For example; if the abutment has integral wing walls then include them in the total length. If a retaining wall is being used for the abutment and the wall extends beyond the bridge then use the bridge out to out width value plus 40 feet for the total abutment length.

The “EA” value is used to determine the number of members in a condition state.

For example: A two span bridge may have been built with 5 piles at each support for a total of 15 piles in the bridge. The pile is inspected, evaluated, and recorded in the appropriate condition state.

C. Inspect the Element and Record the Quantity in the Corresponding Condition State

The first step is to review the condition state language for the elements to be inspected. A complete list of the condition state descriptions is provided in this chapter. Code the appropriate quantity of the element in the corresponding condition state or condition states. The total quantity for those with units of “EA” would be coded in one condition state while elements with units of “LF” or “SF” could have quantities in one or all of the condition states. Element condition state (CS) language is based on four condition states for all primary structural members, regardless of the materials. Similar to the NBI system of evaluation, the BMS requires the inspector to evaluate defects and also quantify the defect’s impact to the element or possibly the bridge. Different philosophies apply to the non-primary structural elements such as deck/overlays, joints, paint, and smart flags. The following summarizes the general BMS condition state philosophy for primary structural members. It must be noted that a defect could be CS1, CS3, or CS4 depending on the location and/or quantity.

Condition State 1: Most parts of a bridge will be in this condition state for all BMS elements. The element may have some defects, but is in good condition. Many times new bridges have insignificant defects and older bridges will acquire insignificant defects with time.

In order to determine if the defect is insignificant, the inspector must decide if the defect will impact the element load carrying capacity with time. Inspectors are cautioned to look at new construction that may not be CS1.

Condition State 2: This condition state documents repairs to structural members. Generally, these are easy to identify and report. Common repairs do not have the same integrity or longevity as original construction. Many times members are difficult to access and prohibit a good quality repair. Inspectors are cautioned to verify repairs to make sure they are functioning as intended. If a repair is not completed correctly or is not functioning properly, then the repair should be coded as CS3 or CS4.

Condition State 3: This condition state records any significant defect noticed by the inspector, but the defect does not significantly impact the capacity of the element. Capacity is not currently threatened, but if left unchecked, it could be threatened in the future. Repairs may apply to the elements in CS3 because the defects are more economical address now than to wait and repair later.

Condition State 4: This condition state documents members with defects that have impacted the structural capacity of the element. Based on the visual inspection, the owner of the bridge must address this deficiency in order to preserve or restore the capacity of the member and/or structure. Generally, these defects have reduced the structural capacity of the element, but are still within safe operating limits of design [34].

2.1.3. BMS computer programs

WSDOT currently uses the Bridgit computer program for bridge network analysis only. One of the many functions of this software is to provide guidance on how best to allocate funds in an agency bridge network. Bridgit software will allow quick answers to various "What If?" funding scenarios, providing immediate feedback needed in the budgeting and programming process. A BMS element for the environment state is controlled by the BMS Engineer and used for modeling the "What If". This element is not coded by the bridge inspectors for the Washington bridges [34].

2.2 GERMAN BRIDGE MANAGEMENT SYSTEM

There are about 36,000 bridges on the national road network in Germany. The bridge management system is based on the SIB-Bauwerke database. The database contains technical data on each bridge, load data, and proposed maintenance works. Bridges are divided into concrete, steel, and masonry. The permanent collection, storage, and evaluation of data on the behavior of bridges in use is carried out by state services according to the ASB instructions (instructions for the road database) using the SIB-Bauwerke software package [36].

The program is used to record the results of inspections and observed damages, implemented maintenance measures, and their costs. The program also contains tools for statistical data processing at the level of the complete road network. The program is planned to include a catalog of measures for maintenance and repair of structures, a catalog of costs, and a catalog of damage. It is also planned to add a model of the state of structural elements during their lifetime as well as an assessment of performance maintenance ie. occasional repairs. Based on this information, a priority list of individual constructions and optimization at the level of the entire network is determined. The development of the BISStra database for the analysis of the data that individual German provinces provide to the competent ministry is in progress.

Supervision, inspections and tests are carried out according to German standards (DIN 1076). Instructions for collecting, processing and analyzing data on the condition of the structure are given in the RIEBW-PRÜF guidelines. The guidelines enable a uniform assessment of the condition, and their content is detailed descriptions and criteria for evaluating the bridge. The impact of each damage on the stability of the structure, safety for the flow of traffic, the durability of the structure and the condition of the entire structure is evaluated in particular. All collected data is entered into the SIB-Bauwerke program. Bridge sections are graded from 1.0 to 4.0.

Based on the results of the review and additional analyses, the necessary documentation is created and annual works are defined. Priorities are determined based on the degree of damage, traffic conditions and available finances.

There are four types of reviews:

- main overview

- simple overview

- special review

- review according to special regulations

The main inspection of the bridge is carried out every six years. It consists of a detailed review of all structural elements from close range, an assessment of damage and deterioration in accordance with RI-EBW-PRÜF guidelines. The result is an assessment of the condition of the complete bridge. Based on the condition, maintenance work, cleaning, and necessary repairs are determined or the need for an additional inspection is determined with recommendations for additional inspections.

A simple inspection is also performed every six years, but it is performed three years after the main inspection. It includes a visual inspection of structural elements from the surface of the surrounding terrain and the surface of the roadway without access to hard-to-reach elements.

Damaged places and places with defects observed during the main inspection are inspected and analyzed.

Special inspections are carried out as needed. They are carried out after incidents that affect the condition of the structure. The content and scope of reviews vary from case to case.

The inspection according to the special regulations is carried out for the inspection of the electrical equipment and additional equipment of the bridge. It is carried out according to the requirements of other regulations and standards.

A bridge management system is being developed in Germany. The system will have the following characteristics:

- will enable insight into the current state of bridges in the state road network system

- will enable the assessment of the necessary financial resources

- will enable the development of a long-term bridge management strategy

The bridge management system will also have the ability to determine maintenance procedures and perform maintenance over time for individual elements based on deterministic models development of damage to structural elements. The best solution is chosen using a cost-benefit analysis. Optimization is done due to limited (available) financial resources.

Table IV-2 shows the parts of the bridge that are inspected.

Table IV-2. Bridge parts for condition assessment in Germany [36]

Part of bridge	Description of the bridge part
1	superstructure
2	substructure
3	facility in general
4	prestressed elements
5	foundations
6	anchors
7	kentledge
8	bearings
9	transition devices
10	insulation
11	road surfacing
12	box girder (corridors)
13	safety devices
14	other

Damage assessment tables for bridges in Germany are provided below.

Table IV-3. Damage Assessment „Stability or Load Bearing Capacity“ [36]

Rating	**Description**
0	The defect/damage has no effect on the stability of the component/structure
1	The defect/damage affects the stability of the component, but has no effect on the stability of the bridge. There are individual minor deviations in component condition, material quality, or component dimensions and minor deviations with regard to the planned stress still well within the permissible tolerances. Repair of damage as part of building maintenance.
2	The defect/damage affects the stability of the component, but has only a minor impact on the stability of the structure. The deviations in component condition, material quality or component dimensions or with regard to the planned stress from the use of the building have the tolerance limits reached or exceeded in individual cases. Damage repair required in the medium term.
3	The defect/damage affects the stability of the component and the structure.

	The deviations in component condition, material quality, component dimensions or with regard to the planned stress from the use of the bridge exceed the permissible tolerances. Required usage restrictions are not available or ineffective. A restriction of use must be implemented immediately if necessary. Damage repair required at short notice.
4	The stability of the component and the structure is no longer given. Required usage restrictions are not available or ineffective. Immediate action is required during the structural inspection. A restriction of use must be implemented immediately. The repair or renewal is to be initiated.

Table IV-4. Damage Assessment "Road Safety" [36]

Rating	Description
0	The defect/damage has no impact on road safety.
1	The defect/damage has little impact on traffic safety; road safety is guaranteed. Repair of damage as part of bridge maintenance.
2	The defect/damage slightly affects road safety; traffic safety is still given. Remedial action or warning required.
3	The defect/damage affects road safety; road safety is no longer fully guaranteed. Elimination of damage or warning required at short notice.
4	Due to the defect/damage, traffic safety is no longer given. Immediate action is required during the structural inspection. A restriction of use must be implemented immediately. The repair or renewal is to be initiated.

Table IV-5. Damage Rating "Durability" [36]

Rating	Description
0	The defect/damage has no effect on the durability of the component/structure.
1	The defect/damage affects the durability of the component, but has only a minor impact on the durability of the structure in the long term. A spread of damage or consequential damage to other components is not to be expected. Repair of damage as part of building maintenance.
2	The defect/damage affects the durability of the component and can also lead to an impairment of the durability of the structure in the long term. The spread of damage or consequential damage to other components cannot be ruled out. Damage repair required in the medium term.
3	The defect/damage affects the durability of the component and leads to an impairment of the durability of the structure in the medium term. A spread of damage or consequential damage to other components is to be expected.

	Damage repair required at short notice.
4	Due to the defect/damage, the durability of the component and the structure is no longer given. The spread of damage or consequential damage to other components requires immediate restriction of use, repair or building renewal.

A summary of damage categories is given in table IV-6.

Table IV-6. Assessment categories of individual damages for bridge management in Germany [36]

Rating	Criterion	Description of condition
0	S	No impact
	V	No impact
	D	No impact
1	S	It affects the bearing capacity of the element, but not the entire structure. Repair of damage is carried out as part of maintenance
	V	Minimal impact on traffic safety. Damage removal conducts within the framework of maintenance.
	D	Affects durability, but no increase in damage or consequences on other elements is expected. Damage removal is carried out within the framework of maintenance
2	S	It affects the load-bearing capacity of the elements and, to a lesser extent, the load-bearing capacity of the building. Deviations are within the permissible limits. Removal of damage within a reasonable time.
	V	Partial impact on traffic safety, but it exists. Removing damage or placing warning signs.
	D	It affects the durability of the element, and in the long term also the durability of the entire building. Removal of damage within a reasonable time.
3	S	It affects the bearing capacity of the element and the entire facility. Deviations pass permissible limits. Removal of damage immediately. Traffic restriction.
	V	It affects traffic safety. Damage removal or installation warning signs in the short term.
	D	It affects the durability of the element, and soon also the durability of the entire building. Damage removal in a short time.
4	S	Carrying capacity no longer exists. Immediate restriction of return traffic and initiation of repair or restoration.
	V	Traffic safety no longer exists. Current limit return traffic and starting repairs or renovations.
	D	Durability no longer exists. Instant fix, limitation traffic or reconstruction

S - bearing capacity

V - trafic safety

D - durability

In order to ensure objectivity when assessing the condition of bridges, a detailed procedure was developed with a damage catalog and inspection instructions. The following table shows an example of the analysis and assessment of damage to the superstructure of bridges made of concrete, reinforced concrete or prestressed concrete (according to German guidelines).

Table IV-7. Damage catalog according to German guidelines [36]

Description of damages and defects	S	V	D
Graffiti on visible surfaces	0	0	0
Visible changes on the concrete due to weather conditions	0	0	0
Minor wearing of concrete cover	0	0	1
Rust on the underside of the structure	0	0	1
Contamination of the interior of the box girders (formwork remains, etc.)	0	0	1
Contamination of the interior of box girders (bird droppings, etc.)	0	1	1
Concrete abrasion of main span structures	0	0	1
Insufficient concrete cover of auxiliary reinforcement	0	0	1
Insufficient concrete cover of the main reinforcement on the underside of the span structure (from 3.0 to 3.9 cm), but with good quality of concrete	0	0	1
Insufficient concrete cover of the main reinforcement on the underside side of the span structure (from 1.0 to 2.9 cm), but with good quality of concrete	0	0	2
Insufficient concrete cover of the main reinforcement on the underside side of the span structure (from 1.0 to 2.9 cm), but with poor quality of concrete	0	0	3
Insufficient concrete cover of the main reinforcement on the underside side of the span structure (less than 1.0 cm),	0	0	3
Carbonation penetrated to the main reinforcement	0	0	3
Spalling of a concrete cover close to the surface on the underside of the span structure (e.g. Icing)	0	0	2
Spalling of road surfacing	0	1	2
Visible main reinforcement on the underside of the span structure, slightly corroded bars (no significant reductions in cross-section)	1	0	3
The main reinforcement of the span structure lies in the area of carbonation and it is slightly corroded (does not	1	0	3

apply to prestressed elements)			
Visible main reinforcement on the underside side of the span structure, the reinforcement is slightly corroded (there are reductions in the cross-section).	2	0	3
Spalling of the concrete cover and matrix in the zone of heavily corroded main reinforcement on the underside of the span structure (advanced reduction of the cross-section)	3	0	3
Cracks parallel to the prestressing wires or post tensioned cables with a width of 0.1 to 0.2 mm in the wetting zone in the span structure	0	0	2
Surface cracks in the wetting area with a width of 0.2 to 0.4 mm in the reinforced concrete span structure	0	0	2
Cracks parallel to the prestressing wires or post-tensioned cables with a width of 0.2 to 0.4 mm in the wetting zone in the span structure	0	0	3
Cracks with width >0.4 mm in the wetting area in the reinforced concrete span structure	0	0	3
Surface cracks with width > 0.4 mm outside the wetting area in the span structure made of prestressed concrete	0	0	3
Surface cracks > 0.4 mm wide, located outside of the wetting area and outside of continuation zone of precast elements in the span structure made of prestressed concrete.	0	0	4
Cracks <0.2 mm wide, in the continuation zone of precast elements in the span structure made of prestressed concrete.	2	0	2
Cracks with width of 0.2 to 0.4 mm, in the continuation zone of precast elements in the span structure made of prestressed concrete.	2	0	3
Cracks >0.4 mm wide, in the continuation zone of precast elements in the span structure made of prestressed concrete.	2	0	4
Cracks > 0.4 mm that enlarge width under load	4	0	4

In the further evaluation process, the rating of carrying capacity, durability and traffic safety is determined based on the matrix. That grade is marked with Z1, and a positive or negative value ΔZ1 is added to it, which takes into account the spread of the load. ΔZ1 has the following values:

U= "small" → $\Delta Z1 = -0.1$

U= "average" → $\Delta Z1 = 0.0$

U= "large" → $\Delta Z1 = +0.1$

This is followed by an assessment of each of the 13 parts of the structure. The total Z_{BG} score for each individual part of the structure (listed in Table IV-2) will be the maximum of the corresponding Z1 scores with the addition of a positive or negative ΔZ2 value that takes into account the number of occurrences of damage within a group of elements that make up one part of the bridge. For the substructure of the bridge, ΔZ2 has the following values:

$n < 5 \rightarrow \Delta Z2 = -0.1$

$5 \leq n \leq 15 \rightarrow \Delta Z2 = 0.0$

$n > 15 \rightarrow \Delta Z2 = +0.1$

For other parts of the bridge from table 2.1.1 (all parts, except the substructure), ΔZ2 has the following values:

$n < 3 \rightarrow \Delta Z2 = -0.1$

$3 \leq n \leq 5 \rightarrow \Delta Z2 = 0.0$

$n > 5 \rightarrow \Delta Z2 = +0.1$

The following attachment shows the calculation matrix of the overall rating.

Table 2.2.7 Matrix for the calculation of the overall assessment of individual damage based on the grade of load capacity (S), traffic safety (V) and durability (D) for the assessment of bridges in Germany [36].

D=0

S / *V*	0	1	2	3	4
4	4,0	4,0	4,0	4,0	4,0
3	3,0	3,2	3,4	3,6	4,0
2	2,1	2,2	2,3	2,7	4,0
1	1,2	1,3	2,1	2,6	4,0
0	1,0	1,1	2,0	2,5	4,0

D=1

S / *V*	0	1	2	3	4
4	4,0	4,0	4,0	4,0	4,0
3	3,1	3,3	3,5	3,6	4,0
2	2,2	2,3	2,4	2,8	4,0
1	1,5	1,7	2,2	2,7	4,0
0	1,1	1,3	2,1	2,6	4,0

D=2

S / *V*	0	1	2	3	4
4	4,0	4,0	4,0	4,0	4,0
3	3,2	3,4	3,6	3,8	4,0
2	2,3	2,5	2,6	2,9	4,0
1	2,2	2,3	2,4	2,8	4,0
0	2,0	2,1	2,2	2,7	4,0

D=3

S / *V*	0	1	2	3	4
4	4,0	4,0	4,0	4,0	4,0
3	3,3	3,5	3,7	3,9	4,0
2	2,8	3,0	3,1	3,2	4,0
1	2,7	2,8	2,9	3,0	4,0
0	2,5	2,6	2,7	2,8	4,0

D=4

S / *V*	0	1	2	3	4
4	4,0	4,0	4,0	4,0	4,0
3	3,6	3,7	3,8	4,0	4,0
2	3,3	3,5	3,6	3,7	4,0
1	3,2	3,3	3,4	3,5	4,0
0	3,0	3,1	3,2	3,3	4,0

The assessment of the condition of the entire structure is determined as a function of several parameters:

$$Z_{ges} = f\ (S^V, S^S, S^D, U, n)$$

S^V - traffic safety

S^S - load bearing capacity

S^D - durability

U - total damage spread

n - number of damage occurrences

The assessment of the condition of the entire bridge Z, with the addition of a positive or negative value ΔZges, starts from the highest grade of the bridge part, max Z, which takes into account the spread of damage to several parts of the bridge:

1 to 3 parts of the bridge that are damaged → ΔZ_3=-0.1

3 to 7 parts of the bridge that are damaged → ΔZ_3=0.0

> 3 parts of the bridge that are damaged → ΔZ_3=+0.1

The final assessment of the condition of the entire bridge can be divided into six categories, which are shown in Table IV-8.

Table IV-8. Damage categories of the entire bridge structure in Germany [36]

Grade range	Description
1,0-1,4	**very good condition**
	The stability, traffic safety, and durability of the structure are ensured. Routine maintenance required
1,5-1,9	**good condition**
	The stability and traffic safety of the structure are ensured. The durability of at least one component group may be impaired. The durability of the structure may be slightly affected in the long term. Routine maintenance required.
2,0-2,4	**satisfactory condition**
	The stability and traffic safety of the structure are given. The stability and/or durability of at least one component group can be impaired. The durability of the structure can be affected in the long term. A spread of damage or consequential damage to the structure, which in the long term leads to significant impairment of stability and/or traffic safety or increased wear and tear, is possible. Ongoing maintenance required. Medium-term repair required. Measures to eliminate damage or warnings to maintain road safety may become necessary at short term.
2,5-2,9	**sufficient condition**
	The stability of the structure is ensured. The traffic safety of the structure may be impaired. The stability and/or durability of at least one component group can be impaired. The durability of the structure may be affected. A spread of damage or consequential damage to the structure, which in the medium term leads to significant impairments to stability and/or traffic safety or increased wear, is then to be expected. Ongoing maintenance required. Short-term repair required. Measures to eliminate damage or warnings to maintain road safety may be necessary at short term.
3,0-3,4	**insufficient condition**
	The stability and/or traffic safety of the bridge are impaired. The durability of the bridge can no longer be guaranteed. The spread of damage or consequential damage can lead to short-term stability and/or road safety no longer being guaranteed. Ongoing maintenance required. Immediate repair required. Measures to eliminate damage or warnings to maintain traffic safety or usage restrictions are required immediately.
3,5-4,0	**insufficient condition**
	The stability and/or traffic safety of the bridge are significantly impaired or no longer exist. The durability of the bridge can no longer be guaranteed. The spread of damage or consequential damage can, in the short term, result in the

	stability and/or traffic safety no longer being provided or in irreparable deterioration of the structure. Ongoing maintenance required. Immediate repair or replacement required. Measures to eliminate damage or warnings to maintain traffic safety or usage restrictions are required immediately.

CHAPTER V

STATE OF THE ART IN THE FIELDS OF DURABILITY AND MAINTENANCE OF CONCRETE BRIDGES

CHAPTER V

STATE OF THE ART IN THE FIELDS OF DURABILITY AND MAINTENANCE OF CONCRETE BRIDGES

Miyamoto et al. [7] in their study attempted to develop a new bridge management system (BMS) for deteriorated concrete bridges by evaluating the output results from a bridge rating expert system that is currently under development. The proposed BMS offered various maintenance plans based on a combination of maintenance cost minimization and quality maximization. Genetic algorithms (GAs) were adopted for solving the optimization problem. These algorithms, which were based on the theory of evolution, create a suitable individual solution through the repetition of three operators: selection, crossover, and mutation. Additionally, applications to several existing concrete bridges were presented to demonstrate the validity of the proposed bridge management system. The results of this study are summarized as follows:

- To clarify the difference between repairs and strengthening measures, they suggested load-carrying capability and durability as the respective main indexes of performance for bridge members.
- The deterioration curve was presented as a method of estimating the progressive deterioration of performance on existing bridge members. By assuming functional deterioration, the proposed BMS is able to estimate the deterioration of the repaired and/or strengthened bridge members.
- The proposed BMS was applied to an existing bridge. They verified that this BMS is able to estimate the deterioration of bridge members and present various maintenance plans based on cost minimization and quality maximization using genetic algorithms (Gas) and the ε-constraint method.

In the research of Enright and Frangopol [8] the effect of repair on time-variant failure probability was illustrated for several repair/replacement strategies. Time-variant reliability computations were performed using a combined technique of adaptive importance sampling and numerical integration. Several repair strategies are investigated for a typical bridge in Colorado. For this bridge, it was shown that for a single repair performed about halfway through the design life, the post-repair strength must be at least 80% of the original strength for the repair to have a significant influence on the system failure probability of the bridge during its remaining service life. Also, for maintenance using shotcrete, multiple repairs appear to have little influence on the lifetime system failure probability. It was shown that the optimum repair time is dependent on the failure cost of the bridge and that the discount rate influences the optimum repair time and life-cycle cost. As the discount rate increases, the optimum repair time increases until it reaches an upper bound that is dependent on the system failure probability constraint. The results can be applied to the prediction of optimal lifetime maintenance planning strategies for concrete bridges under corrosion.

In the research of Watanabe and Sakoi [9] the reinforced concrete bridge in actual environment at 55 years after constructed was inspected. The objectives of this study were to confirm the condition of deterioration of the bridge at actual environment and to obtain the fundamental data for maintenance and long-term management of bridges based on data obtained by survey. Mechanical properties and durability properties (carbonation, chloride ion penetration, scaling resistance and pore distribution) of concrete structure in cold region were tested and analysed. They concluded that beside mechanical properties, durability properties play important role in maintenance of bridges, especially for old bridges.

Stallings and Yoo [10] were inspected and load rated a total of twelve truss bridges in Alabama. The ratings were performed under the guidelines described by AASHTO (Manual 1983). Two types of rating methodologies are described: the allowable stress method and the load and resistance factor method. The allowable stress method is currently used by the Alabama Highway Department for establishing ratings for steel bridges. Hence, the allowable stress method was used in all rating calculations performed for steel bridge members under this project. The load and resistance factor method was used for reinforced concrete members where reinforcement details were available. The primary objective of this project was to provide a bridge inspection and load rating for twelve bridges for which a load rating had never been established. The specific project objectives were to perform a thorough field inspection of the bridges, to collect all pertinent field data on the geometry and details of the bridges, and to perform a structural analysis and load rating of each bridge based on the superstructure load carrying capacity. All twelve bridges exhibit signs of deterioration to varying degrees. The major causes of deterioration are: corrosion (loss of member cross sectional area), vehicle impacts due to overweight vehicles and narrow lanes, and fatigue fracture of members and rivets. Most of the bridges exhibited deterioration from all of these.

Cremona, C [11] noted that the management of structures is a very important economic issue. France has been aware of this for many years. In 2006, after the parliament vote of decentralization law, the State considered as critical to rationalize the maintenance and the management of the remaining national asset. Since the past 5 years, a lot of procedures and guidelines for bridge maintenance have been revised for the national bridge stock. In addition opportunity was given to introduce new concepts such focalized inspection, risk-based assessment. These changes or upgrades are made to take better account in the decision-making process of socio-economic aspects (disruption for road users in particular) and the effect of decisional choices and to introduce more elaborate structural condition assessment methods which will give a more reliable estimate of the current and predicted condition of the bridges asset. The Directorate for Transportation Infrastructure (DIT) has initiated a large review of the policies and procedures for bridge maintenance and management. It is certainly not exhaustive but highlights the actual procedures. Several developments are under process to improve the maintenance strategies applied to the national asset. Among them, it is important to note a new inspection level in the surveillance scheme: the focalized inspections. Focalized inspections constitute a new category of organized surveillance intended to improve preventive bridge maintenance. The project under development consists in introducing more non-destructive inspection techniques into periodical inspections. Focalized inspections are not applied to all bridges. Bridges with scores 3 and 3U are for instance not concerned (in-depth auscultations are generally scheduled for these structures) as well as vulnerable structures (these structures are analysed by a risk-based methodology). The remaining bridges are divided into two categories: the bridges concerned (stock 2) or not by detailed inspections (stock 1). He concluded that not all the bridges are subject to detailed periodical inspections.

Focalized inspection is a targeted inspection:

- It mainly concerns stock 1 bridges spread into bridge types, each type divided into predefined categories,

- It targets a sampling group inside each category,
- It focuses major degradation problems for each bridge type.

An aging index is assigned to each bridge, the average value giving the category aging index. The average aging index is assigned to all the bridges from a same category. Two aging indexes are given for a bridge: one for the exposed parts to aggressive environment, one for the other parts. A threshold aging index determines additional investigations if exceeded for a particular bridge. Each bridge is individually monitored by the focalized inspection. The focalized inspections are not periodic but depend on the results of the previous inspections. The results are collected and stored in the LAGORA Bridge Management Software. Armed with this information, bridge engineers will be able to make better decisions about repairs, to redesign details that will improve durability, and to use specialized repair techniques.

Wang stressed that reinforced concrete has been commonly used in China as the most popular structural material in many infrastructures [12]. However, since these structures have served for several decades, problems of concrete durability gradually arise due to severe service environment or air pollution of increasing CO2 concentration. He also said the durability of the reinforced concrete structures principally depends on the optimization of five factors. They are structure design, construction operation, management and maintenance, material properties and the external environmental conditions.

These five factors are closely correlated with each other so that the durability will be markedly reduced if one of them is poor. Without adequate durability, the reinforced concrete structure may deteriorate either due to concrete damages or due to reinforcement corrosion. The main reasons behind are concrete carbonation, chloride ion ingress, alkali-aggregate reactions and freeze-thaw cycles. In this study the influences of five factors on the concrete durability were analysed.

Further investigation was made regarding the deterioration of reinforced concrete. As a case study Jinan Yellow River Highway Bridge was inspected in the aspect of concrete durability. The result shows that most of the chambers inspected in Jinan Yellow River Highway Bridge are in a good condition. Concrete suffers slightly from carbonation but the thickness of carbonation depth is far less than the concrete cover. The reinforcement is not likely to be corroded. However, in some testing chambers there are problems with inadequate thickness of concrete cover and cracks. These problems can be treated by pasting cement mortar, which is for safety consideration.

The main objective of the research of Elbehairy [2], was to develop a practical and efficient framework for managing large bridge networks. The proposed framework is innovative in its ability to optimize decisions at the network level (which bridge should be repaired and when) as well as at the project level (best type of repair for bridge elements). This research resulted in the development of a practical, easy-to-use Markov chain deterioration model. The developed deterioration model builds on inspection data collected by municipalities.

The developed Markov chain model customizes the deterioration matrices to produce new ones that realistically describe the deterioration of different bridge elements in different environments. The main advantage of this research is the integration of the project-level and network-level decisions. This integration was simple in the case of only one component, for which both types of decisions are made at the same time in a single optimization process that considers all constraints on both levels. On the other hand, in the case of multiple bridge elements, the integration of the project-level and network-level decisions were made in two sequential optimization cycles. This methodology has been proven to arrive at good decisions on both the network and project levels. This research has investigated different techniques and methodologies for handling large-scale bridge networks, a typical infrastructure-asset-management problem. The performance of the optimization and the quality of the decisions are dependent to a great extent on the objective function, the problem size, and the formulation. The best strategy for optimizing the infrastructure problem is to prioritize the assets on a yearly basis while attempting to gain the maximum benefits from the repair.

Kenshel, O [6], adopted and developed a new BMS which supports the relative authorities in Libya by assisting maintenance planning as well as decision making of bridges to operate efficiently in a systematic way, also helps to identify and estimate the repairs required to keep bridges operate effectively besides ranking them to priority of work, and performing optimal maintenance and replacement actions. In this paper the data of two bridges were collected and they were inspected to compare and rank.

By comparing the Sufficiency Rating of the two bridges the results showed that the Mahary Bridge considered to be qualified for government replacement funding with a Sufficiency Rating of 23% then comes the Al Khazanat Bridge which qualifies for government rehabilitation funds with a percentage of 58%.

By reviewing the available literature, it was observed that a large number of researches in the field of durability of bridges and bridge maintenance management have been performed. In most of these papers the existing BMSs are analysed and several new are proposed. They are based on the contemporary mathematical programs and models including Probability Method, Artificial Neural Network (ANN), Analytic Hierarchy Process (AHP), Grey Associated Analysis, Comprehensive Weight Variation Evaluation, Fuzzy theory etc.

To obtain the right decision from an IMS or BMS, software packages must have high quality asset information for the system′s various analytical processes. Because of that periodic inspection records are the key resources amongst other information as historical bridge condition rating data can affect approximately 60% of BMS analysis models [5]. Some of new BMS software effort overcome problem of historical data record gaps by using non bridge factors as supplementary historical data, such as local climate, number of vehicles and population growth in the area surrounding the bridge [5]. Also, it is emphasized the importance of knowledge of deterioration mechanisms of concrete and reinforcement for obtaining quality input data.

In this doctoral dissertation problem of deterioration processes in hot climate will be particularly emphasized, because the analysis of the available papers showed that there is a lack of research in this field.

CHAPTER VI

ASSESSMENT OF 7 BRIDGES IN TRIPOLI (BEFORE REPAIR)

CHAPTER VI

ASSESSMENT OF 7 BRIDGES IN TRIPOLI

(BEFORE REPAIR)

This part contains a technical description, assessment and conclusion about state of the chosen and analysed bridges in Tripoli before repair.

1. SOUK ATHULATHA 1 BRIDGE

1.1. Technical Description

In this part, the location and type of the bridge are identified.

Location of bridge: Souk Athulatha 1

Bridge souk Athulatha1 is located in the west part of the capital Tripoli, about 360 meters from the sea to the north. It is considered as one of major bridge to the capital Tripoli. It connects several main roads leading to the center of the capital. In Figure VI-1, VI-2 and VI-3 are shown situation plan and views of bridge. The coordinates for this bridge are 320 52'45.5" N 130 09'19.8"E.

Figure VI-1. Souk Athulatha1 bridge location on goggle maps

Figure VI-2. Souk Athulatha1 Bridge, south side

Figure VI-3. Souk Athulatha1detail of ceiling and exterior wall

Type of bridge

Bridge souk Athulatha1 is designed as simple arch bridge made of reinforced concrete. According to the style of construction this bridge is classified as semi-prefabricated, cantilever bridge, with two cantilever beams and prefabricated simple supported slab in the middle of span.

This bridge was built in the middle of XX centuries. In Figure VI-4 and VI-5 north and south sides of the bridge are shown. The plan of the bridge is given in Figure VI-6.

The characteristic dimensional data of the Bridge are:

- Length: 39.00m
- Width: 25m
- Height: 5.58m
- Main span: 22.40m
- Sidewalk (right side): 3.24m
- Sidewalk (left side): 4.54m

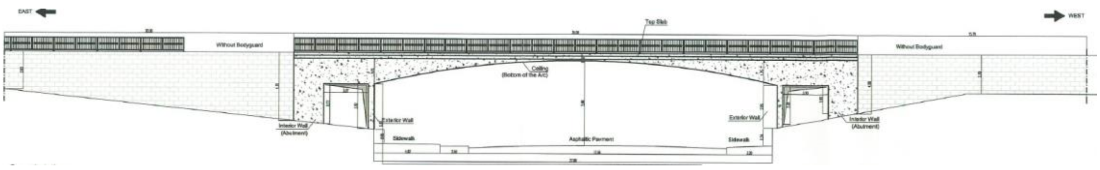


Figure VI-4. Longitudinal cross section of bridge (north side)

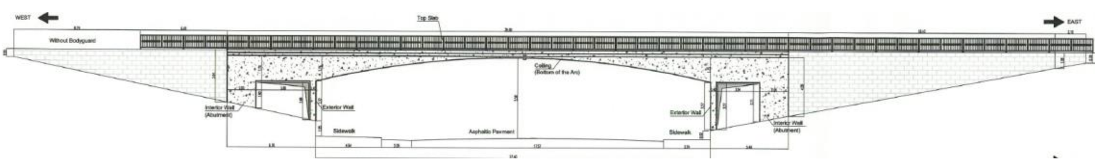


Figure VI-5. Longitudinal cross section of bridge (south side)

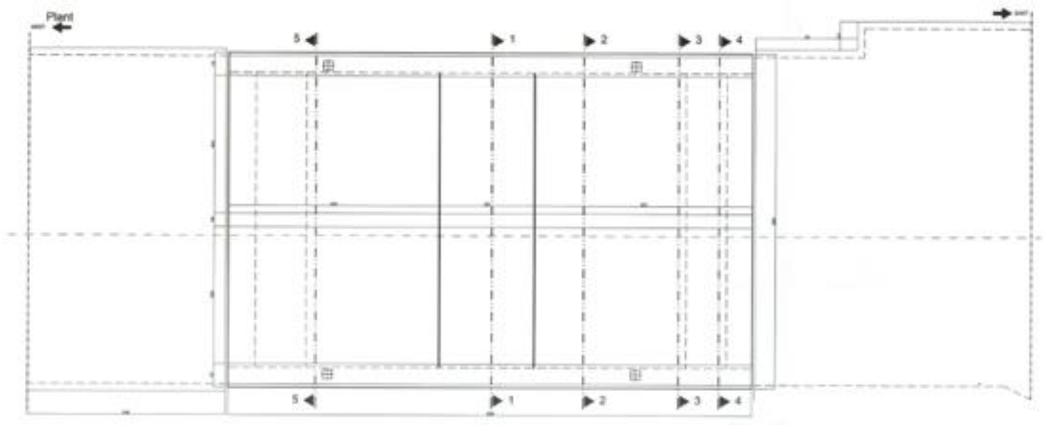


Figure VI-6. Plan of the bridge – upper side

Basic elements of the bridge are:

- Abutment
- Exterior wall
- Arc cantilever slab
- Simple Beam slab
- Cantilever side slab

Arc cantilever slabs and simple beam slab ware continuous during the construction.

Disposition of basic bridge elements are signed in Figures VI-7 and VI-8.

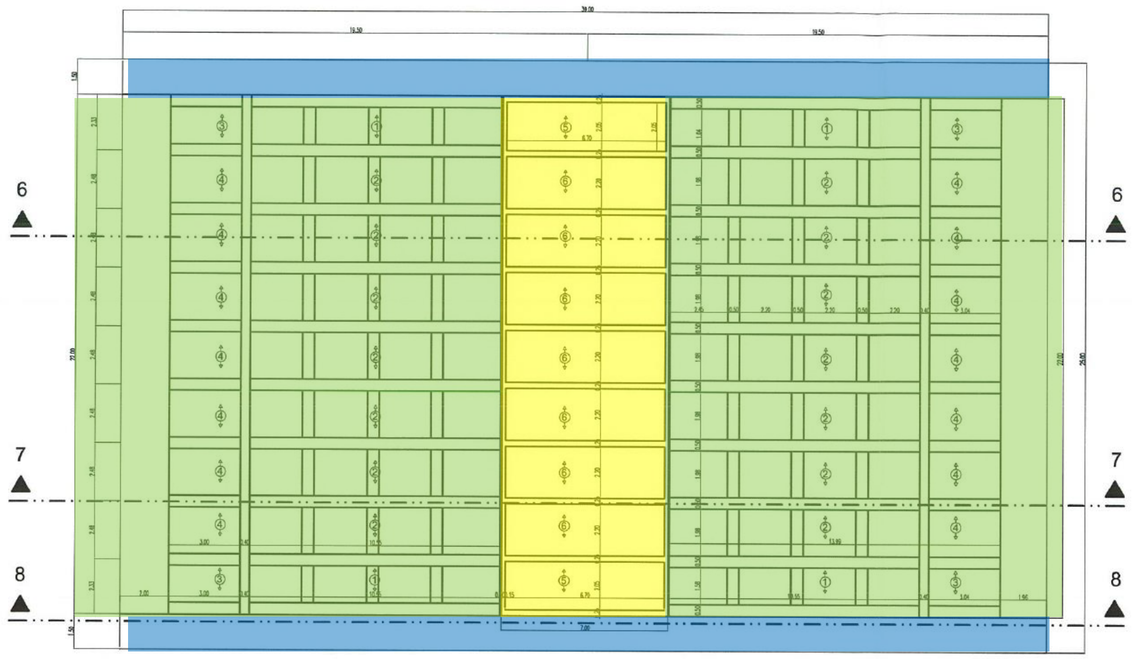


Figure VI-7. Disposition of arch cantilever slabs (green), simple beam slab (yellow)and cantilever side slab (blue) in plane of the bridge (bottom side)

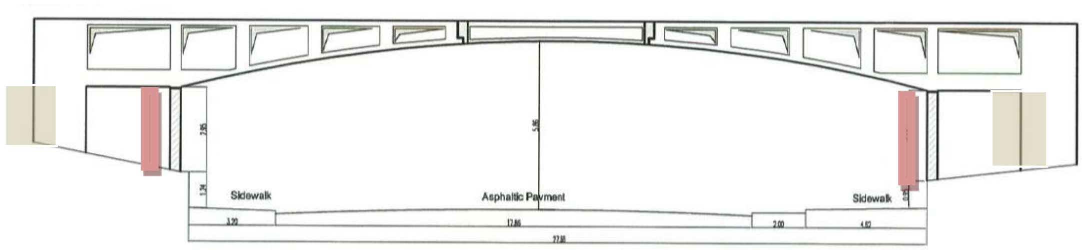


Figure VI-8. Disposition of exterior (brown) and Abutments of bridge (gray) (section 6-6)

The following text provides a brief description of basic elements of the bridge.

Bridge Souk Athulatha1 has two abutments. Both walls have no openings. The basic dimensions of each abutment are:

Abutment on east side:

- Length: 22.18m
- Height: 3.50m (visible part of total height)
- Depth: 2.1 m

Abutment on west side

- Length: 22.18m
- High:3.50m(visible part of total height)
- Depth: 2.03m

Figure VI-9 shows the general view of walls. Longitudinal view of abutments given in Figure VI-10.

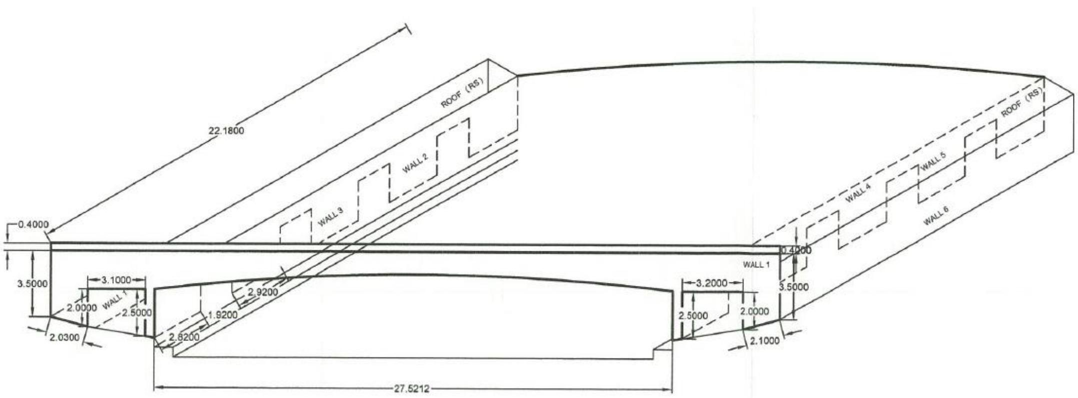


Figure VI-9. General view of walls

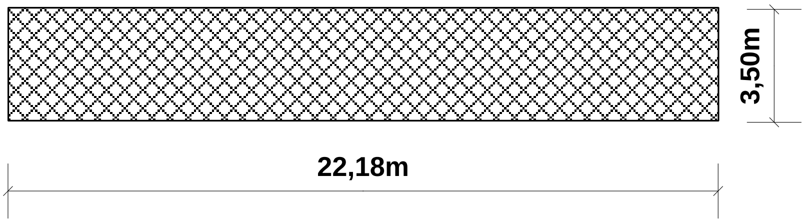


Figure VI-10. Longitudinal view of abutment

Bridge Souk Athulatha1 has two exterior walls. Both walls have four openings. The basic dimensions of each exterior wall are:

Exterior wall on west side:

- Length: total 22.18m (with four openings)
- Height: variable 2.5-3.27m (visible part of total height)
- Depth: 0.4m

Exterior wall on east side

- Length : total 22.18m (with four openings)
- Height : 2.5-3.27m (visible part of total height)
- Depth: 0.4m

Figure VI-9 shows the general view of walls. Longitudinal view of exterior walls given in Figure VI-11.

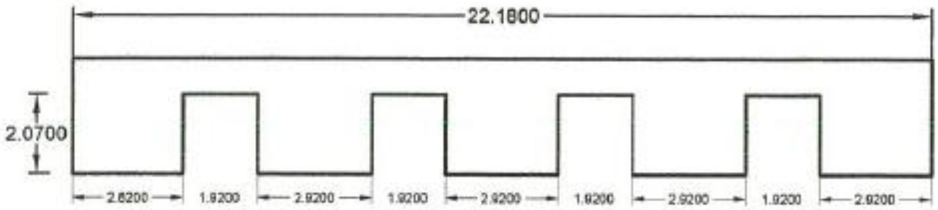


Figure VI-11. Longitudinal view of exterior wall

Upper (horizontal) part of the bridge is designed as arc slab. This slab consists of two arc cantilever slabs (Figure VI-12) and simple beam slab (Figure VI-13). Cantilever slabs have box cross section. The basic data of arc cantilever slabs are:

- Length: 14.97m
- Width: 22.18m
- Depth: variable from 50cm (hinge) up to the 205cm (fixed end)

In Figures VI-12, VI-13 and VI-14 the view of arch slab from bottom side and cross sections near hinge and fixed ends are shown.

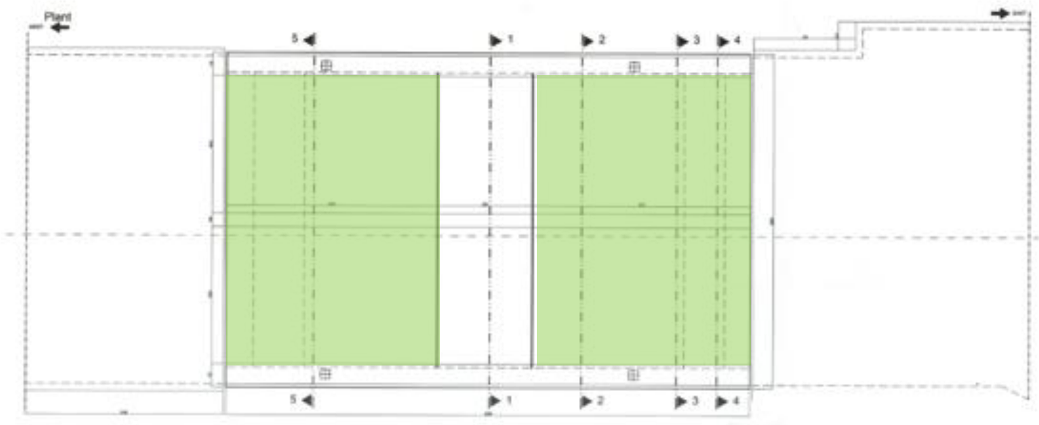


Figure VI-12. The location of cantilever arch slabs in plan of bridge

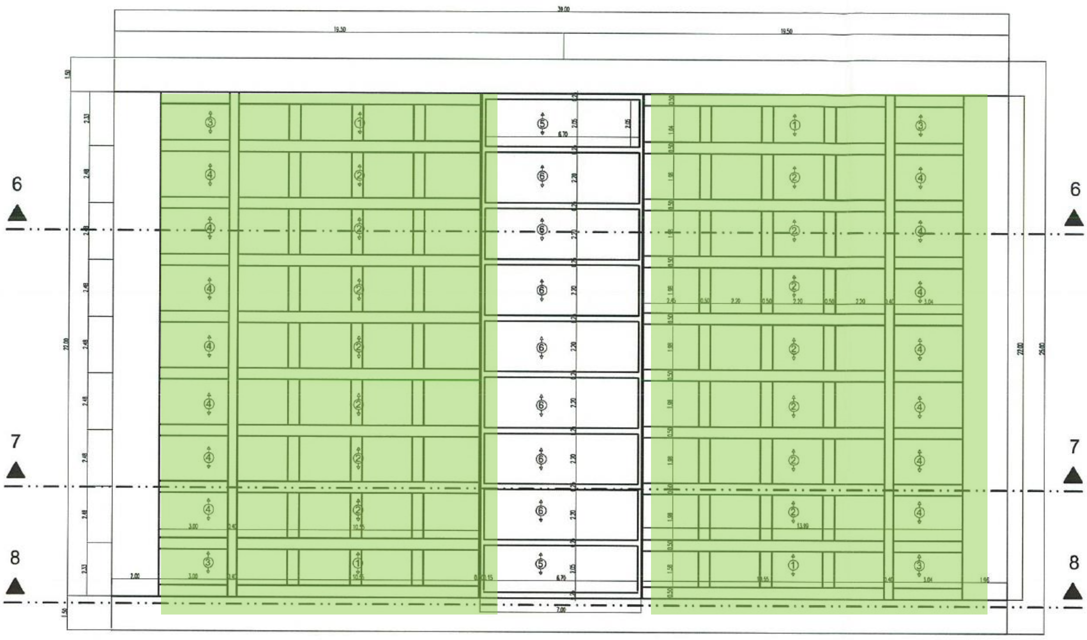


Figure VI-13. Arch slab view from bottom side and location of cantilever arch slabs

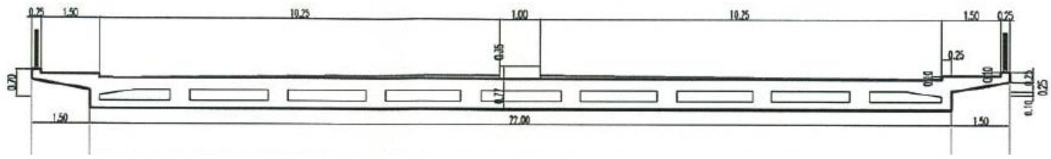


Figure VI-14 Cross section of arch slab near the hinge (section 2-2)

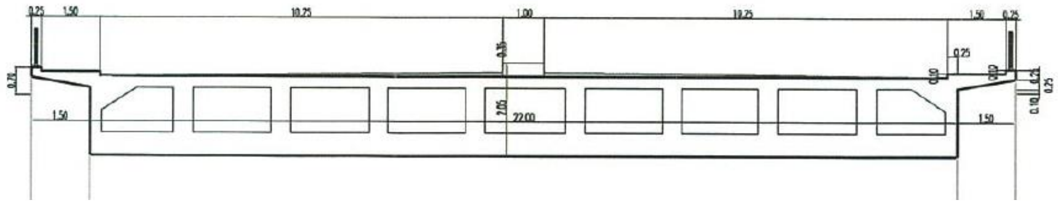


Figure VI-15. Cross section of arch slab near the fixed ends (section 4-4)

Simple beam slabs located in the middle of span. It has box cross section. Characteristic dimensions are:

- Length: 3.49m
- Width: 22m
- Depth: 50cm

Disposition of simple beam slabs and characteristic cross sections are given in Figures (VI-16, VI-17, VI-18, VI-19, VI-20) below.

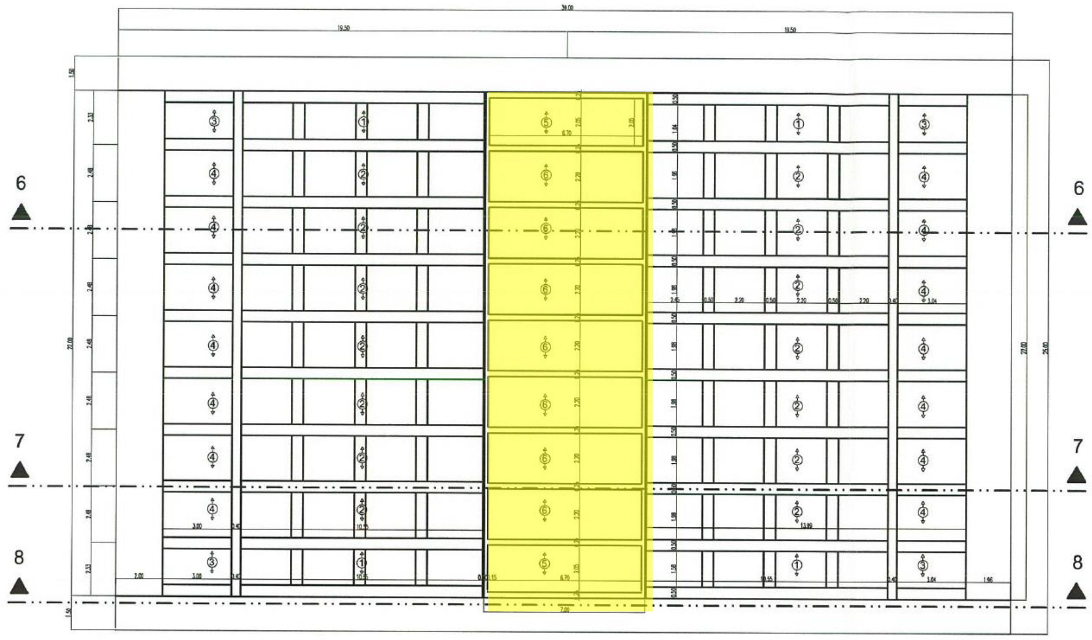


Figure VI-16. Simple beam slab view from bottom side

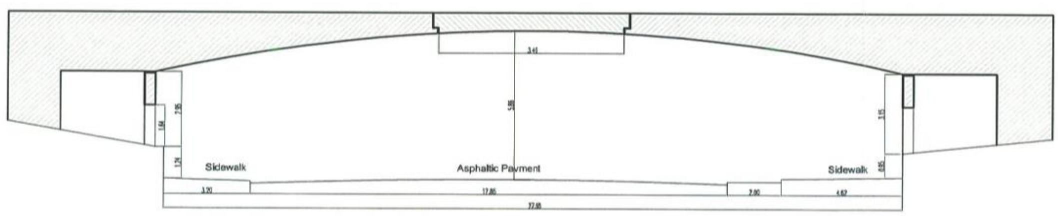


Figure VI-17. Disposition of simple beam slab in the span of bridge (section 7-7)

Figure VI-18. Dimensions of simple supported beam slab in cross section

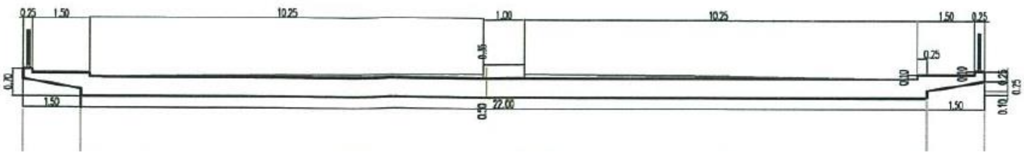


Figure VI-19. Longitudinal view of simple beam slab

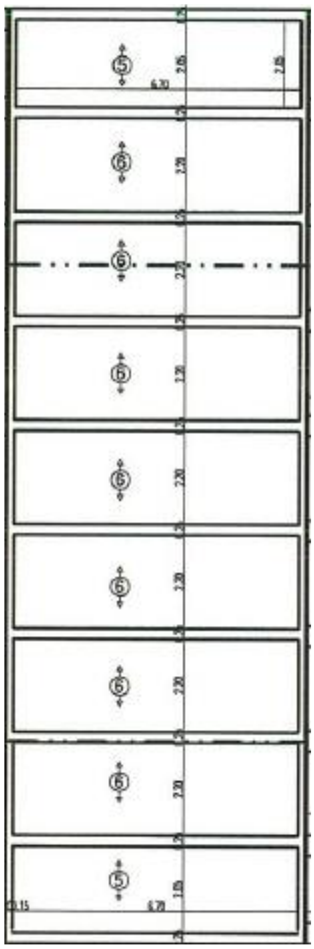


Figure VI-20. Plane of simple beam slab with characteristic dimensions

Bridge Souk Athulatha1 has two pedestrian paths that are designed as Cantilever side slabs with edge beam. The characteristic dimensions of side slabs (Figure VI-21) are:

- Length: 39.08m
- Width: 1.50m
- Depth:variable from 18cm (free end) up to the 40cm (fixed end)

and of edge beams (Figure VI-21) are:

- Length: 39.08m
- Cross section: 25cmx25cm

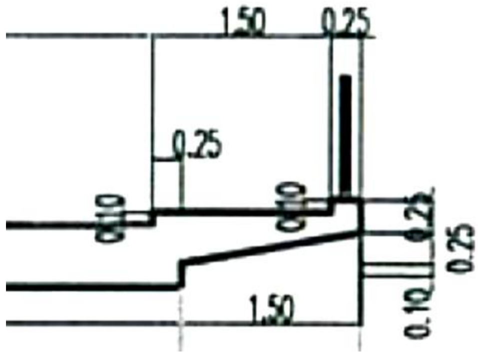


Figure VI21. Cantilevers

1.2. Assessment of Bridge Souk Athulatha1

In the aim of choose repair materials and technics for this bridge, next activities were planned:

- In-situ testing of concrete quality and
- Visual inspection of visible parts of bearing elements.

Numbered activities were done in 2009.

1.2.1 Testing of concrete in bearing elements of bridge

The testing of concrete quality encompassed the next activities:

- Measurement of carbonation depth,
- Chloride ion content,
- In-situ testing of concrete by taking of cores,
- In-situ testing of concrete by Schmidt Hammer test and
- In-situ testing of concrete by Pull-off method.

Carbonation depth

The extent of carbonation was assessed by treating with phenolphthalein indicator the fresh exposed surfaces of drilled cores, which were extracted from structure elements for testing concrete compressive strength or for testing carbonation depth.

All data of testing elements, measured depth of carbonation and rebar location are given in Table (VI-1).

Table VI-1. Data of testing elements, measured depth of carbonation and rebar location

Reference 1	Carbonization 2		Rebar Depth 3	Depth of carbonization 4	In Rebar plan 5	Element 6
	Yes	No	mm	mm		
05.01	X		20	20	Y	Lateral south wall
05.02	X		-	20	-	Abutment south side
05.03	X		20	20	Y	Exterior wall south side
05.04	X		20	20	Y	Exterior wall south side
05.05	X		-	30	-	Exterior wall in the middle
05.06		x	20	0	N	Exterior wall in the middle
05.07	X		-	20	-	Exterior wall north side
05.08		x	20	0	-	Exterior wall north side
05.09	X		20	40	Y	Lateral north wall
05.10		x	20	0	-	Exterior wall north side
05.11		x	40	0	-	In the slab
05.12		x	-	0	-	In the slab
05.13		x	-	0	-	In the slab
05.14	X		40	50	Y	Ceiling- south
05.15	X		60	60	Y	Ceiling- center
05.16	X		70	60	N	Ceiling- center
05.17	X		10	30	Y	Ceiling- center
05.18	X		60	50	N	Ceiling- north

For better analyzing of obtained results the Figure VI-22 is formed.

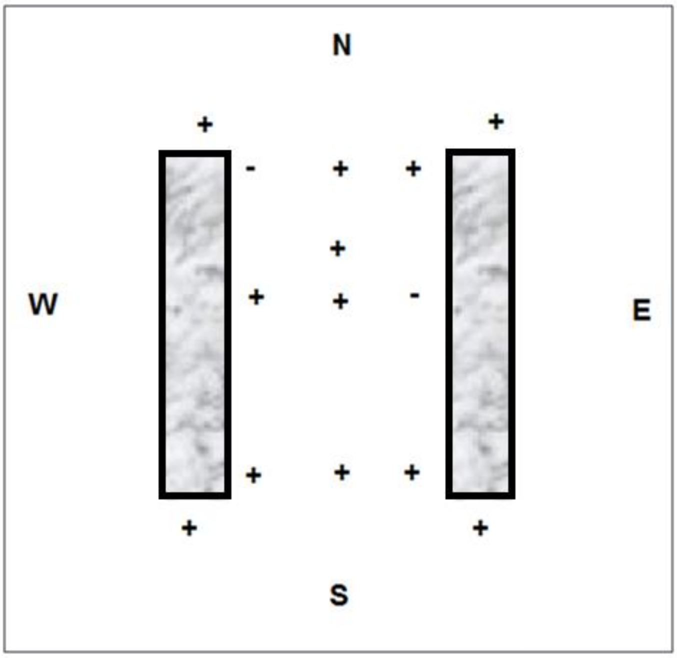


Figure VI-22. Carbonation test results for exterior walls and ceiling

After analyzing carbonation results, the next conclusions can be derivate:

- The carbonization is most expressed of ceiling.
- Both exterior walls have the same influence of concrete carbonation. (see Figure VI-22).
- Other concrete elements do not have problem with carbonation.
- The front of carbonization come up to reinforced bars, even passed behind the bars.

Chloride test

The content of ion chloride in concrete is checked by using small pieces of drilled cores which were pulverized and dissolved in acid liquid. The chloride ions react with acid in an electrochemical reaction. An electrode was inserted into the liquid and the change in voltage was measured. On the basis of measured voltage, the instruments showed the chloride content in concrete in %. The obtained results are given in Table VI-2.

Table VI-2 Chloride test result

Elements 1	Reference 2	% Chloride in concrete (Equipment reading) 3			% Chloride ion content by mass of cement 4		
		0-2cm	2-4cm	4-7cm	0-2cm	2-4cm	4-7cm
Support exterior wall(south)	05.01	0.0156	-	0.0037	0.002	-	0.000
Support Abutment (south)	05.02	0.0140	0.0051	0.0060	0.002	0.001	0.001

Ceiling in the middle	05.03	0.0690	0.0175	0.0179	0.009	0.002	0.002
Slab	05.04	0.0063	-	-	0.001	-	-
Ceiling-south-12/04	05.05	0.0340	0.0078	0.0047	0.004	0.001	0.001
Ceiling-center-12/04	05.06	0.0800	0.0216	0.0136	0.010	0.003	0.002
Ceiling-north-12/04	05.07	0.1036	0.0252	0.0125	0.013	0.003	0.002

For analyzing given results next criterion was used: The maximum of chloride ion content by mass of cement for reinforced concrete with ordinary carbon steel is 0.40% (class Cl 0.40) (BS 8500).

After comparing obtained results with specified criterion, the next conclusion was made:

- All testing results are smaller than criteria value.
- Chloride content in concrete in the bridge structure is not hazardous to imbedded reinforced bars.

Core test

For testing concrete compressive strength, the core tests were done. Cores were extracted from three different locations. In order to determine differences between surface and inner concrete quality, cores were taken out from whole depth of elements. The chosen locations for taking out cores were:

- Support walls -three cores,
- Lateral beam – two cores and
- Deck ceiling – two cores.

In the laboratory extracted cores were splitting in the next way:

- In three parts from support walls and
- In two parts for lateral beam.

Then, all obtained cores were visually inspected and prepared for testing compressive strength by cupping. Testing procedure for compressive strength is described in standard BS 1881: Part 120:1983. All obtained results of estimate in-situ compressive strength are given in table VI-3, and they represent cube compressive strength. For changing cylinder compressive strength to cube compressive strength, the factor of correction was used. This factor depends of dimensions of specimens and of direction of drilling.

On the basis of visual inspection, it was concluded that all specimens did not have reinforced bars and that all specimens were homogenous.

Table VI-3. Core test result

	Specimens 1	Direction of drilling [V/H] 2	Element 3	Density (kg/m3) 4	Cylindar strength (MPa) 5	Estimated In-situ cube Strength (MPa) 6
	N^{0}.1.1	H	Support wall	2347.44	43.10	45
	N^{0}.1.2	H	Support wall	2340.91	45.58	46
	N^{0}.1.3	H	Support wall	2251.61	37.55	39
	N^{0}.2.1	H	Support wall	2266.32	25.59	27
	N^{0}.2.2	H	Support wall	2281.54	22.39	24
	N^{0}.2.3	H	Support wall	2295.50	23.56	25
	N^{0}.3.1	H	Support wall	2265.32	25.59	27
	N^{0}.3.2	H	Support wall	2281.54	22.39	24
	N^{0}.3.3	H	Support wall	2295.50	23.56	25
	N^{0}.4.2	H	North lateral beam	2279.10	32.20	34
	N^{0}.4.1	H	North lateral beam	2216.35	42.72	45
	N^{0}.5.1	H	North lateral beam	2268.49	23.73	25
	N^{0}.5.2	H	North lateral beam	2324.93	23.12	23
	N^{0}.6	V	Deck ceiling	2117.96	27.15	28
	N^{0}.7	V	Deck ceiling	2104.35	16.85	16

In aim to make conclusion of concrete quality, the average value and the range of estimated in-situ cube compressive strength are calculated and shone in table VI-4.

Analyzing those results it can be seen that the difference between minimum and maximum value for each tested element is large and vary from 12 to 22MPa.

This led to the conclusion that built-in concrete has very unequal quality and compressive strength differ from one to another location. But at the same location the obtained compressive strengths are very similar and depth of wall and beam do not influence the quality of concrete.

Table VI-4. compressive strength test result

Compressive strength of concrete cores in souk Athulatha 1 bridge (MPa) - cube values				
	Element 1	Cube result 2	fck, average 3	Range of fckn 4
	Exterior walls	45	31.33	24 -46
		46		
		39		
		27		
		24		
		25		
		27		
		24		
		25		
	Deck ceiling	28	22.00	16 -28
		16		
	Lateral Beams	34	31.75	23 - 45
		45		
		25		
		23		

Schmidt hammer test

For getting more information of built-in concrete quality the Schmidt hammer test, as a nondestructive- surface hardness method is chosen. Data about tested elements and number of measure places are given in table VI-5.

Table VI-5. Tasted elements and number of measuring points

Element	Part of element	Number of measuring point	Total number of measuring point per element	Total number of measuring point
Arch slab	Ceiling-South side	5	19	30
	Ceiling – north side	4		
	Ceiling - center	4		
	Top slab	3		
	Ceiling-South side tunnel	3		
Exterior wall	South side	2	6	
	North side	4		
Abutment	South side	5	5	

On each test location 10 rebound readings were done. Prior to test the surface of concrete was smoothed by carbonudum stone and cleaned. Rebound number was calculated by using next rule: Each result from one test location is valid if it is in range of ±7 points of average value. For each reading the single compressive strength was calculated by using calibration curves and finally the average and standard deviation are calculated too. The calculate values of compressive strengths and standard deviations are given in table VI-6.

Table VI-6. Schmidt hammer test result

Element 1	Part of element 2	Wmed(MPa) 3	σ 4
Arch slab	Ceiling- south side	17.80	1.35
	Ceiling- south side	15.18	1.43
	Ceiling- south side	38.25	1.60
	Ceiling- south side	48.82	1.06
	Ceiling- south side	48.82	1.06
	Ceiling-north side	11.11	1.69
	Ceiling-north side	49.36	1.94
	Ceiling-north side	47.58	1.21
	Ceiling-north side	40.23	1.94
	Ceiling-center	38.53	1.22
	Ceiling-center	53.78	1.42
	Ceiling-center	17.59	1.56
	Ceiling-center	13.45	1.14
	Top slab	18.67	2.25
	Top slab	19.24	1.71
	Top slab	9.48	1.42
	Ceiling- south side(tunnel)	53.90	1.35
	Ceiling- south side(tunnel)	57.14	2.81
	Ceiling- south side(tunnel)	39.64	0.94
Exterior wall	Exterior wall- south side	35.65	1.15
	Exterior wall-south side	34.86	1.69
	Exterior wall-north side	38.79	2.02
	Exterior wall-north side	29.63	3.25
	Exterior wall-north side	36.44	1.64
	Exterior wall-north side	44.07	2.42
Abutment	Abutment-south side	19.88	1.09
	Abutment-south side	32.41	2.19
	Abutment-south side	30.77	1.39
	Abutment-south side	26.05	1.08

	Abutment-south side	24.58	1.34

In order to make some conclusion of concrete compressive strength, obtained by Schmidt hammer test, the individual results were grouped and mean value of compressive strength and standard deviation were calculated. The obtained data are shown in Table VI-7.

Table VI-7. Schmidt hammer test result analyse

Element 1	Part of element 2	Wmed (MPa) 3	σ 4	Carbonization test
Arch slab	Ceiling- south side	33.77	±16.38	Y
	Ceiling-north side	37,07	±17.75	Y
	Ceiling-center	30.84	±18.83	Y
	Top slab	15.80	±5.48	-
	Ceiling- south side(tunnel)	50.23	±9.31	-
Exterior wall	Exterior wall- south side	35.26	±0.56	Y
	Exterior wall-north side	37.23	±5.99	N
Abutment	Abutment-south side	26.74	±5.01	Y

The results given in previous table show a very large dispersion of compressive strength for each analyzed element of structure. Some results are too small for reinforced concrete (i.e., top slabs Wmed=9.48MPa, Ceiling-north side Wmed=11.11MPa). According to this analyze the next conclusion can be derived: the built-in concrete has very bad uniformity.

Some results given in Table VI-7 can be compared with results of compressive strength obtained by testing cores, Table VI-4. Comparing compressive strengths for exterior wall it is concluded that there is no significant difference, when carbonation reduction is taken account. The similar conclusion can be made for ceiling and lateral beams, because they belong to the same element (arch slab).

It is well known that the carbonation makes concrete surface layer to be harder. In extreme cases the overestimate of compressive strength from this cause may be up to 50%. On the base of carbonation test results the majority of tested elements are affected with process of carbonation and, according to previous comment, show the higher values of compressive strength from the real value. Therefore, the real estimated compressive strengths of tested elements are smaller from values given in table VI-7 for ceiling, exterior wall- south side and Abutment – south side.

The coefficient of correction is calculated by average of compressive strength obtained by core and average of compressive strength obtained by Schmidt hammer:

$f_{ck,av}$ = 30.02MPa (average compressive strength obtained by core)

$f_{ch,av}$ = 33.06MPa (average compressive strength obtained by Schmidt hammer)

$F_{c,\ comp}$ = 30.02/33.06=0.9080

In table VI-7a the average results of compressive strength before and after correction has been shown.

Table VI-7a. Schmidt hammer test result results of compressive strength before and after correction

Element	compressive strength before correction (MPa)	compressive strength after correction (MPa)
Abutment	26.74	24.43
Exterior wall (south side)	35.26	32.21
Exterior wall(north side)	37.23	34.01
Ceiling slab (south)	33.77	30.85
Ceiling slab (north)	37.07	33.86
Ceiling slab (centre)	30.84	28.17
Tunnel ceiling (south)	50.23	45.89
Top slab	15.80	14.43

After correction of results the next conclusion is made:

The highest value of concrete compressive strength in south tunnel ceiling is 45.89MPa

The smallest value of concrete compressive strength in top slab is 14.43MPa.

Pull off test

For measuring in-situ concrete tensile strength the pull-off method is used. The procedure is described in BS 1881: Part 207. The steel disks and epoxy resin glue were used. The tests were conducted in four places. Obtained results are given in table VI-8.

Table VI-8. Pull off test result

Reference 1	Element 2	W med (MPa) 3	Б 4	Failure mode (%) 5		
				concrete	Surface concrete	Epoxy glue
05.01/05.05	Exterior wall (East)	1.162	0.41	20%	46%	34%
05.06/05.10	Abutment	0.530	0.26	4%	84%	12%
05.11/05.15	Center ceiling (north)	0.537	0.27	92%	8%	0%

05.16/05.20	Center slab (north)	1.064	0.41	76%	20%	4%

On the bases of given results, it can be seen that all values of tensile strength are smaller than minimum require value and that build-in concrete has very bad quality.

Density

Calculation of density of hardened concrete is very good method for checking the quality of built-in concrete. For calculation of the concrete density, the mass of extracted cores is usually used. Obtained results are given in table VI-9.

Table VI-9. Density test result analyse

Element of structure	Density, Kg/m3	Average for each measuring place kg/m3	Average for element of structure, kg/m3
Exterior wall	2347.44	2313	2292
	2340.91		
	2251.61		
	2266.32	2281	
	2281.54		
	2295.50		
	2265.32	2281	
	2281.54		
	2295.50		
Deck ceiling	2117.96	2111	2111
	2104.35		
Lateral beams	2279.10	2272	2272
	2216.35		
	2268.49		
	2324.93		

On the bases of given results, it can be seen that values of densities for each tested element are uniform but they show differences when compared by elements. It can be concluded that concrete built in decks ceiling has the smallest value of density (~2100kg/m^3). This conclusion corresponds with conclusion of compressive strength – deck ceiling has the smallest compressive strength. Other two tested elements (exterior wall and lateral beam) have similar values of densities (~2280kg/m^3) and compressive strengths.

1.2.2. Visual inspection of Bridge Souk Athulatha1

First visual inspection of all visible bearing elements or part of bearing elements of bridge were done in 2009.

During the visual inspection it has been noticed that horizontal elements of structure were plastered by thin layer of ordinary cement mortar and covered by paint.

During the visual inspection o lot of damages were registered.

Characteristic damages are:

- Corrosion of reinforcement
- Damage of concrete due to reinforcement corrosion.
  - Falling down of cover (spalling).
  - Cracking of cover
  - Delamination and falling down of plaster layer
- White stains on concrete surface (water soluble salts)

Main causes that led to the appearance of described damages are:

- Carbonation
- Poor quality of concrete
- Insufficient depth of cover
- Wind
- Inadequate water drainage system

Figures VI-23 – VI-27 show characteristic damages of RC elements.

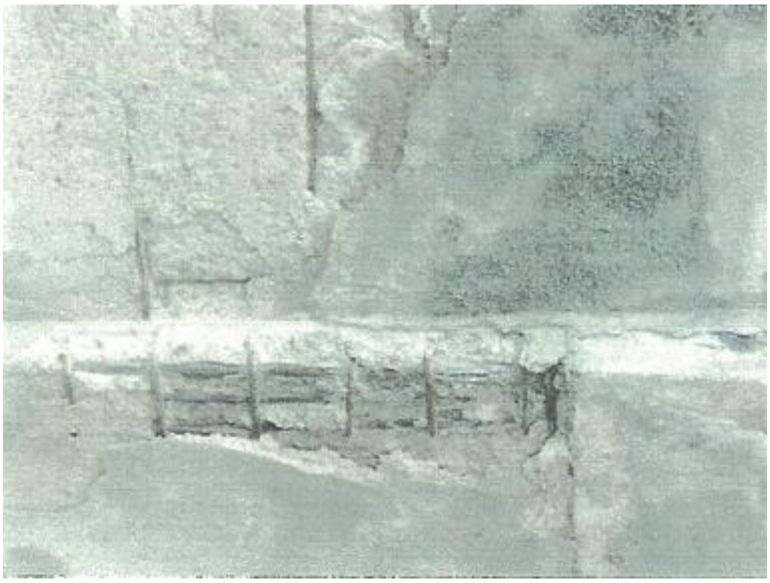


Figure VI-23. Exposed reinforced bars in lateral beam of arch slab, and in cantilever slab. Corrosion of bars, falling down of cover and plaster layer

Figure VI-24. Leakage of water through cantilever slab and through joint between arch slab and simple supported beam slab, damaged edge of cantilever slab (edge beam)

Figure VI-25. View of deck ceiling

Figure VI-26. Exposed reinforced in beam slab next to the cantilever, corrosion of bars, falling down of cover and plaster coating

Figure VI-27. Deck ceiling slab: corrosion of rebars, delamination and falling down of deck ceiling cover, wet stains

The most damaged elements are cantilever slabs with edge beams. The characteristic damage is falling down of concrete cover and ordinary plaster layer (Fig.VI-23, VI-26). Some reinforced bars were bared and affected by surface corrosion. The edges of slabs are rough and stain of water- and water-soluble salts can be seen.

The next most damage element is deck ceiling slab. The most damaged parts of slab are lateral beams and center of simple beam slab (Fig. VI-26, VI-27). The characteristic damage on both places is reinforced corrosion. Due to the expansion of corroded bars, the concrete cover is cracked and fallen off. The thin layer of plaster separated from the deck ceiling and fell down. Due the corrosion expansion of bars the whole concrete layer of down part of box cross section has been cracked and almost falling to the road (Fig. VI-27).

Other concrete structural elements (exterior walls, Abutments and underpass ceilings are also damaged due to corrosion of reinforced bars.

Visual inspection encompassed other bridge elements, like sidewalks, curb stones, catch pits and fences. All mentioned elements have been seriously damaged.

1.3. General conclusion for bridge Souk Athulatha1

The bridge Souk Athulatha1 has been old about 50 years when it was inspected for the first time. The main conclusion of the inspection was that the bridge is damaged.

The characteristic defect of arch slabs and lateral beams have been insufficient concrete cover. This defect was unsuccessfully solved by plastering with ordinary cement mortar.

The main cause of damage appearance is carbonation. Almost all inspected elements had the problem with carbonation especially ceiling. In some cases, the front of carbonation even passed behind the bars.

The second cause of damage appearance is inadequate drainage of water from the deck. This problem caused leakage of water through joints and overflow of water over the edge of cantilever slabs. Consequently, the corrosion of reinforced bars in deck ceiling and cantilever slabs were caused.

Analyzing concrete compressive strength obtained by cores it can be seen that the difference between minimum and maximum value for each tested element is large and vary from 12 to 22MPa. This led to the conclusion that built-in concrete has very unequal quality and compressive strength differ from one to another location

The results of concrete compressive strength obtained by Schmidt hammer test show a very large dispersion of compressive strength for each analyzed element of structure.

Comparing compressive strengths obtained by cores and by Schmidt hammer test it is concluded that there is no significant difference, when carbonation reduction is taken account. So, the general conclusion can be established that built in concrete has large dispersion in quality from very high (~45MPa) up to very low (~15MPa).

According EN 206-1 the compressive strength classes of concrete given in next table can be used for the control calculation.

Bridge Element	Compressive strength class
Abutment	~C20/25
Exterior wall	C25/30
Ceiling slab	C25/30
Top slab	~C12/15

On the bases of results obtained by pull-off method it can be concluded that concrete tensile strength is very low and smaller than minimum require value.

Chloride content in concrete in the bridge structure is not hazardous to imbedded reinforced bars.

Finally, the main conclusion can be drawn:

- Durability of all structural elements is decreased, because of numerous damages that occurred in elapsed time.
- Bearing capacity of structural elements is not jeopardized because there are no serious cracking or deformations of RC elements.
- Global stability and stability of each structural element are not threatened and
- Functionality of bridge is partly reduced, because of damages of surface asphalt layers and local instability of delaminated concrete pieces that occurred on the bottom sides of ceiling slab, lateral beams cantilever slabs and edge beams.

2. SOUK ATHULATHA 2 BRIDGE

Technical description, assessment, rating and repair of bridges in Tripoli (2009).

2.1. Technical description

In this part, the location and type of the bridge will be identified.

Location of bridge: Souk Athulatha 2

Bridge Souk Athulatha 2 is located in the west part of the capital Tripoli, about 360 meters from the sea to the north. It is considered as one of major bridge to the capital Tripoli. It connects several main roads leading to the center of the capital. In Figure VI-28, VI-29, VI-30 and VI-31 are shown situation plan and views of bridge. The coordinates for this bridge are320 52'45.2" N 130 09'26.2"E.

Figure VI-28. Souk Athulatha 2 Bridge location on google maps

Figure VI-29. Souk Athulatha 2 bridge, south side

Figure VI-30. Cantilever slab

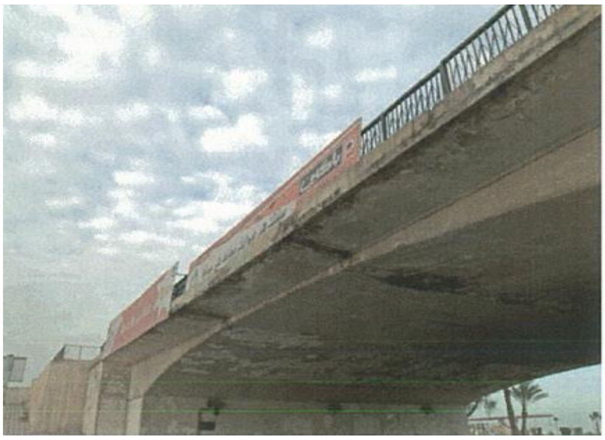


Figure VI-31. Cantilever and deck ceiling slab

Type of bridge

Bridge Souk Athulatha2 is designed as Simple Arch Bridge made of reinforced concrete. This bridge was built in the middle of XX centuries. In Figure VI-32 and VI-33 north and south sides of the bridge are shown. The plan of the bridge is given in Figure VI-34.

The characteristic dimensional data of the Bridge are:

- Length: 39.00m
- Width: 25m
- Height: 5.60m
- Main span: 27.57m
- Sidewalk (right side): 4.38m
- Sidewalk (left side): 5.56m

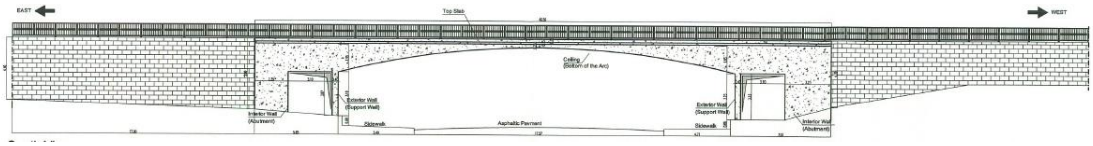


Figure VI-32. Longitudinal cross section of bridge (north side)

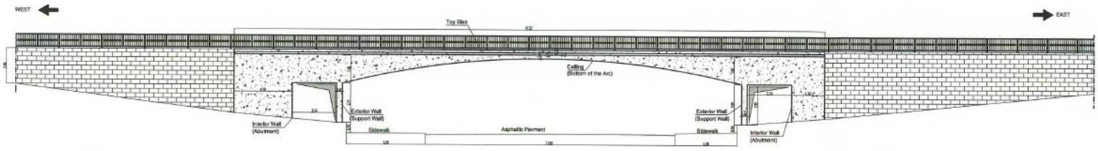


Figure VI-33. Longitudinal cross section of bridge (south side)

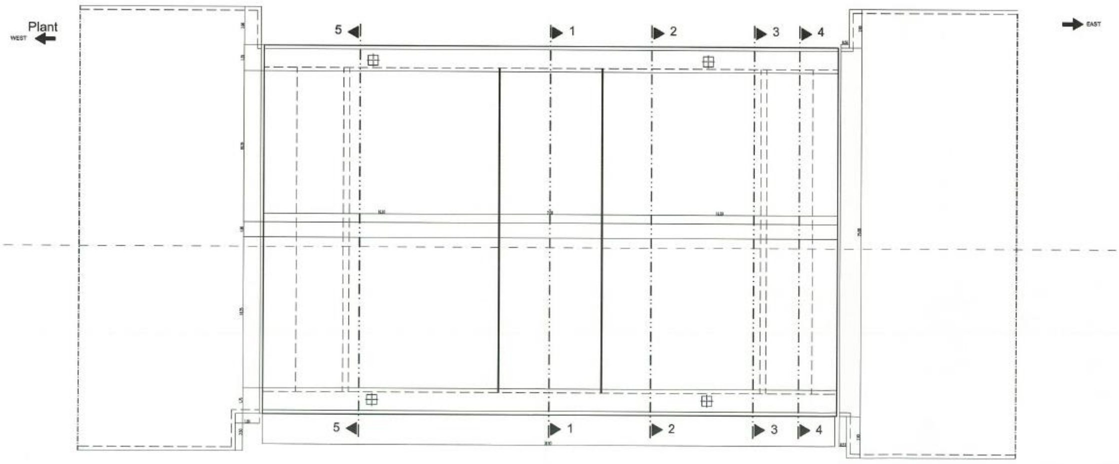


Figure VI-34. Plan of the bridge – upper side

Basic elements of bridge are:

- Interior wall
- Exterior wall
- Arc cantilever slab
- Simple Beam slab
- Cantilever side slab

Arc cantilever slabs and simple beam slab ware continuous during the construction.

Disposition of basic bridge elements are signed in Figures VI-35 and VI-36.

Figure VI-35. Disposition of arch cantilever slabs (green), simple beam slab (yellow) and cantilever side slab (blue) in plane of the bridge (bottom side)

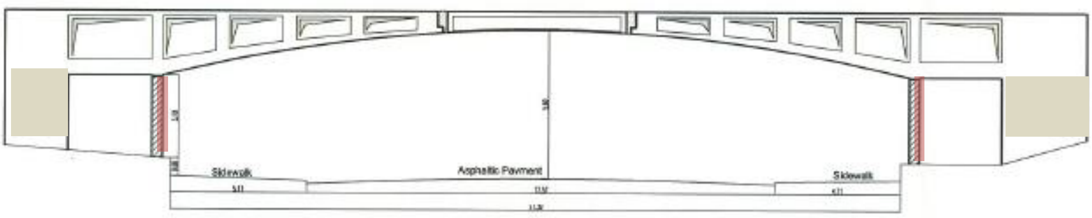


Figure VI-36. Disposition of exterior (brown) and interior walls of bridge (gray) (section 6-6)

The following text provides a brief description of basic elements of the bridge.

Bridge Souk Athulatha2 has two interior walls. Both walls have no openings. The basic dimensions of each interior wall are:

Interior wall on east side:

- Length: 22.23m
- Height: 3.60m (visible part of total height)
- Depth: 2.0 m

Interior wall on west side

- Length: 22.23m
- High: 3.60m(visible part of total height)
- Depth: 3.80m

Figure VI-37 shows the general view of walls. Longitudinal view of interior walls given in Figure VI-38.

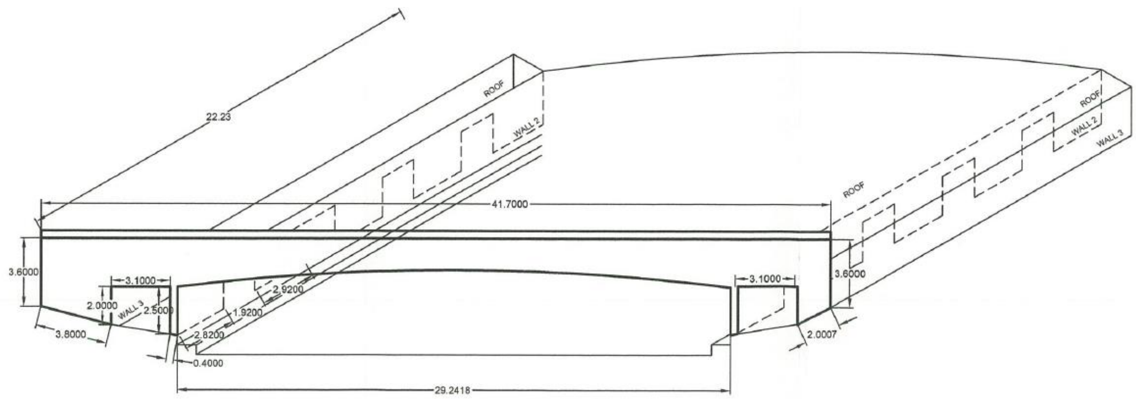


Figure VI-37. General view of walls

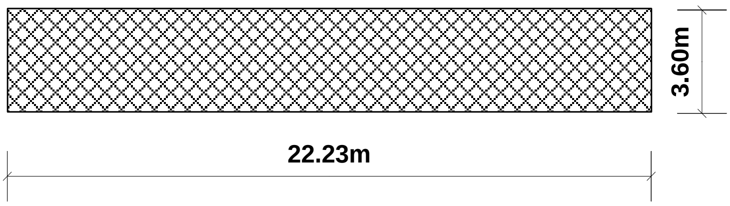


Figure VI-38. Longitudinal view of interior wall

Bridge Souk Athulatha 2 has two exterior walls. Both walls have four openings. The basic dimensions of each exterior wall are:

- Exterior wall on west side:
- Length: total 22.23m (with four openings)
- Height: variable 2.5-3.23m(visible part of total height)
- Depth: 0.4m

Exterior wall on east side

- Length: total 22.23m (with four openings)
- Height: 2.5-3.10m (visible part of total height)
- Depth: 0.4m

Figure VI-37 shows the general view of walls. Longitudinal view of exterior walls given in Figure VI-39.

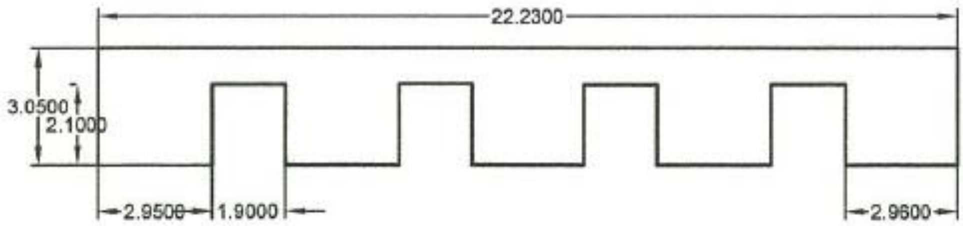


Figure VI-39. Longitudinal view of exterior wall

Upper (horizontal) part of the bridge is designed as arc slab. This slab consists of two arc cantilever slabs (Figure VI-40) and simple beam slab (Figure VI-44). Cantilever slabs have box cross section. The basic data of arc cantilever slabs are:

- Length :14.97m
- Width: 22.23m
- Depth: variable from 50cm (hinge) up to the 205cm (fixed end)

In Figures VI-40, VI-41 and VI-42 the views of arch slab from bottom side and cross sections near hinge and fixed ends are shown.

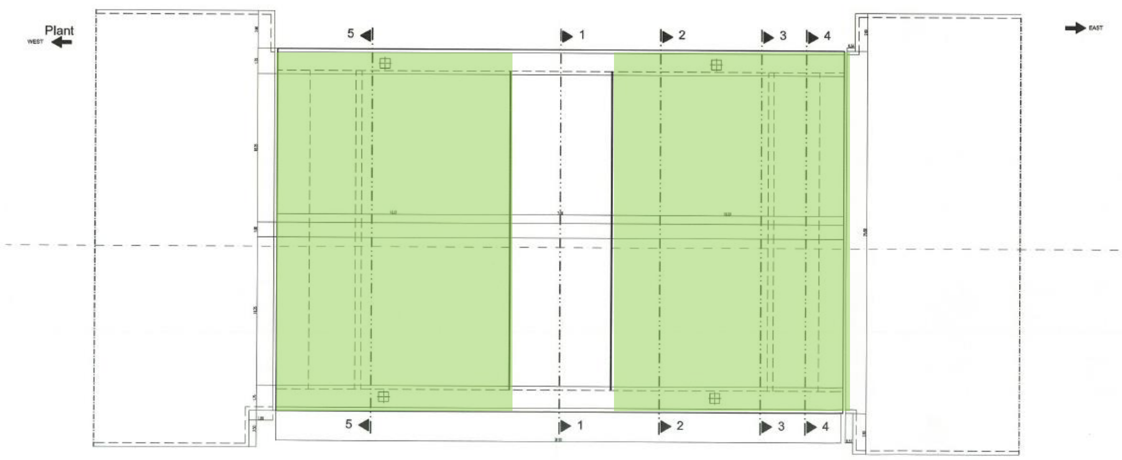


Figure VI-40. The location of cantilever arch slabs in plan of bridge

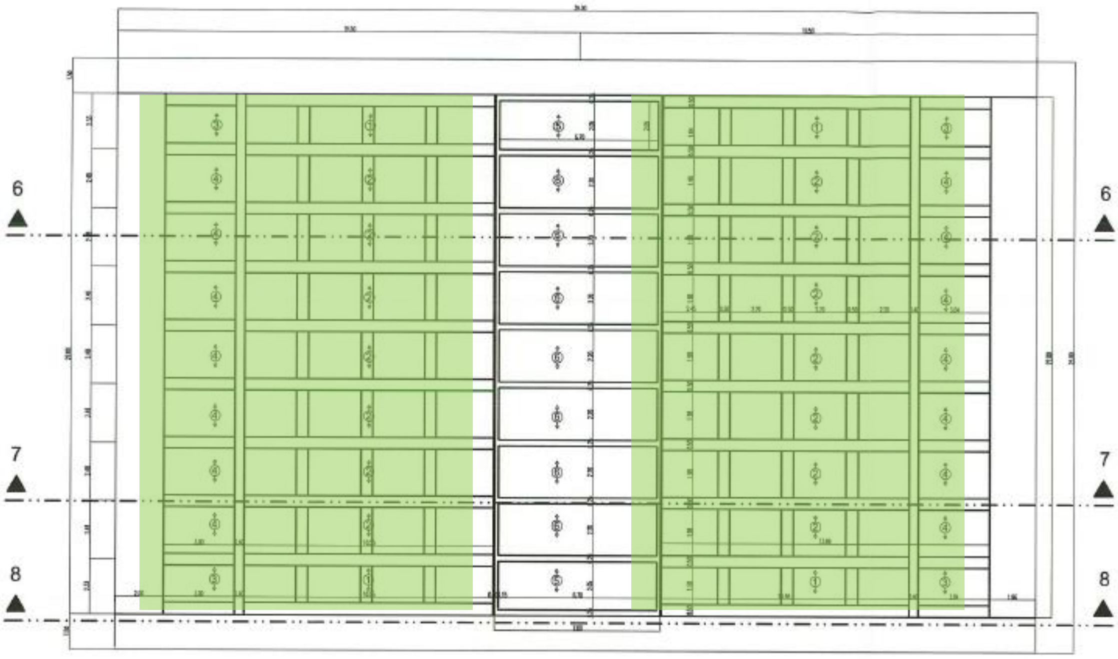


Figure VI-41. Arch slab view from bottom side and location of cantilever arch slabs

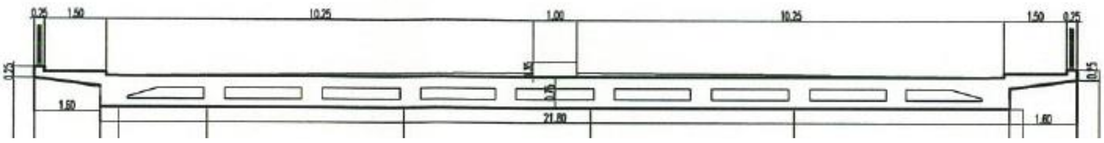


Figure VI-42. Cross section of arch slab near the hinge (section 2-2)

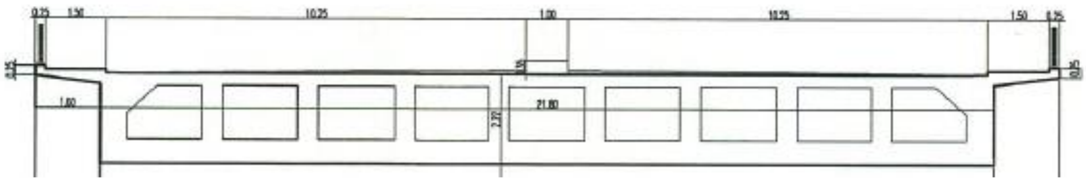


Figure VI-43. Cross section of arch slab near the fixed ends (section 4-4)

Simple beam slabs located in the middle of span. It has box cross section. Characteristic dimensions are:

- Length :3.49m
- Width:22m
- Depth :50cm

Disposition of simple beam slabs and characteristic cross sections are given in Figures (VI-44, VI-45, VI-46, VI-47, VI-48) below.

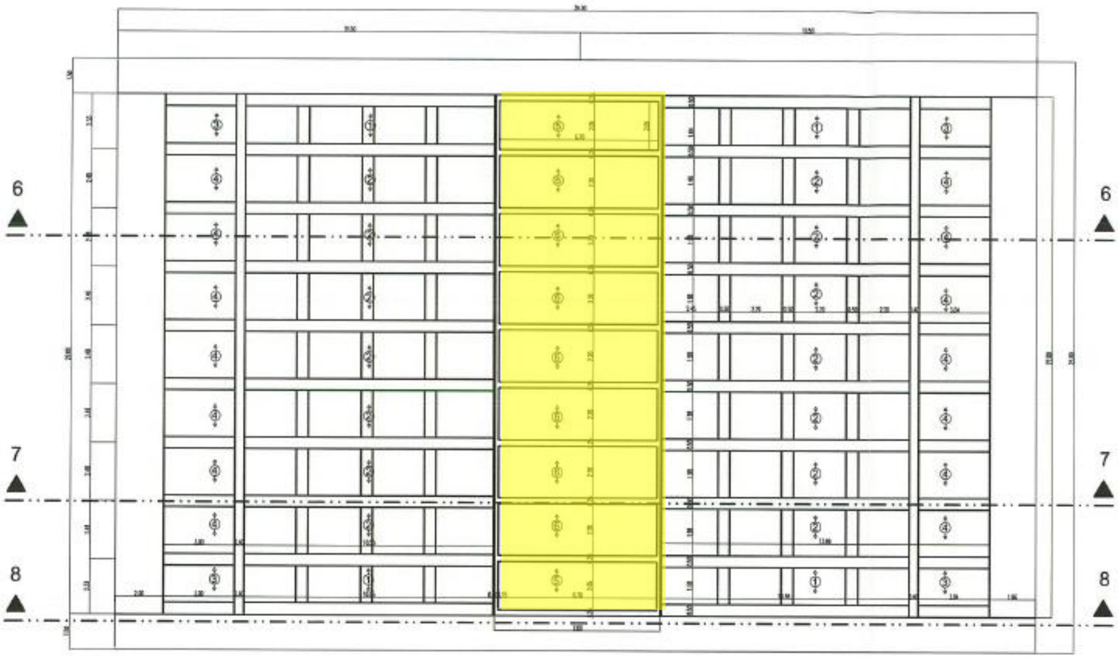


Figure VI-44. Simple beam slab view from bottom side

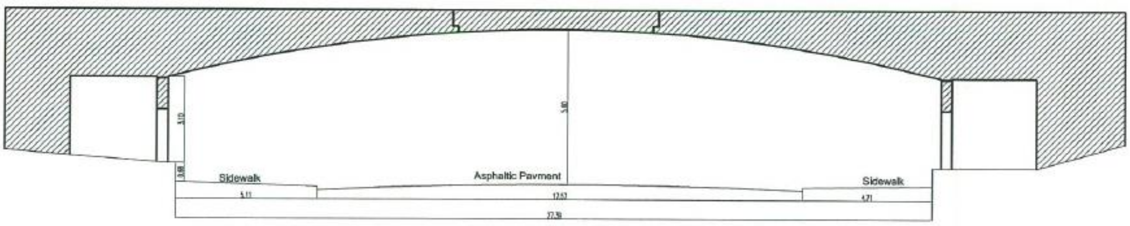


Figure VI-45. Disposition of simple beam slab in the span of bridge

(Section 7-7)

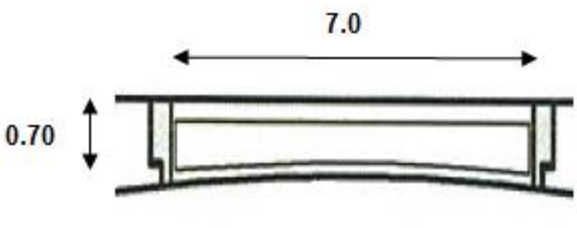


Figure VI-46. Dimensions of simple beam slab in cross section

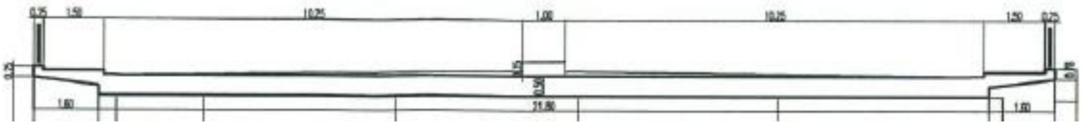


Figure VI-47. Longitudinal view of simple beam slab

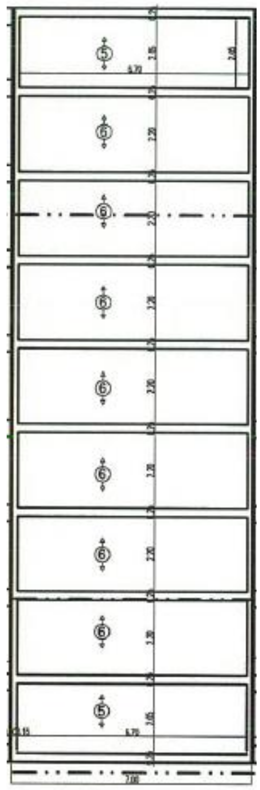


Figure VI-48. Plane of simple beam slab with characteristic dimensions

Bridge Souk Athulatha 2 has two pedestrian paths that are designed as Cantilever side slabs with edge beam. The characteristic dimensions of side slabs (Figure VI- 49) are:

- Length:41.57m
- Width: 1.60m
- Depth: variable from 18cm (free end) up to the 40cm (fixed end)

and of end beam (Figure VI-22) are:

- Length: 41.57m
- Cross section: 25cmx25cm

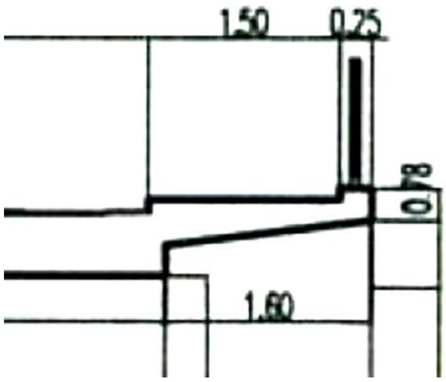


Figure VI-49. Cantilevers

2.2. Assessment of Bridge Souk Athulatha 2

In the aim of choose repair materials and technics for this bridge, next activities were planned:

- In-situ testing of concrete quality and
- Visual inspection of visible parts of bearing elements.

Numbered activities were done in 2009.

2.2.1. Testing of concrete in bearing elements of bridge

The testing of concrete quality encompassed the next activities:

- Measurement of carbonation depth,
- Chloride ion content,
- In-situ testing of concrete by taking of cores,
- In-situ testing of concrete by Schmidt Hammer test and
- In-situ testing of concrete by Pull-off method.

Carbonation depth
The extent of carbonation was assessed by treating with phenolphthalein indicator the fresh exposed surfaces of drilled cores, which were extracted from structure elements for testing concrete compressive strength or for testing carbonation depth.

All data of testing elements, measured depth of carbonation and rebar location are given in Table (VI-.10).

Table VI-10. Data of testing elements, measured depth of carbonation and rebar location

Reference 1	Carbonization 2		Rebar Depth 3	Depth of carbonization 4	In Rebar plan 5	Element 6
	Yes	No	mm	mm		
06.01	x		-	20	-	Exterior wall south side
06.02	x		20	40	Y	Exterior wall south side
06.03	x		-	40	-	Exterior wall south side
06.04	x		-	20	-	Exterior wall south side
06.05	x		-	50	-	Exterior wall in the middle
06.06	x		50	50	Y	Exterior wall in the middle
06.07	x		-	30	-	Exterior wall in the middle
06.08	x		-	20	-	Exterior wall in the middle
06.09	x		30	10	N	Exterior wall south side
06.10	x		-	20	-	Exteriorwall south side
06.11	x		40	10	-	Interior wall south side
06.12	x		30	30	-	Exterior wall south side
06.13		x	40	-	-	Interior wall south side
06.14	x		30	10	N	Exteriorwall south side
06.15	x		20	20	-	Interior wall in the middle
06.16	x		10	20	Y	Ceiling- center
06.17	x		20	20	Y	Ceiling- center
06.18	x		10	40	Y	Ceiling- center

For better analyzing of obtained results the Figure VI-50 is formed.

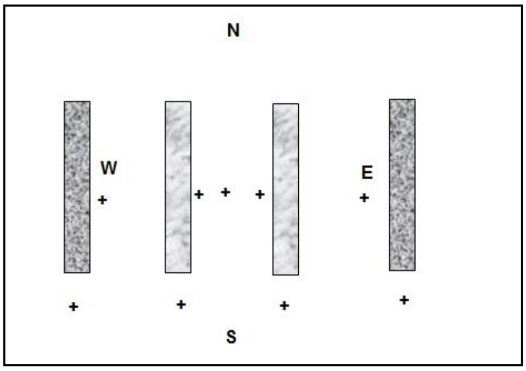


Figure VI-50. Carbonation test results for interior walls and ceiling

After analyzing carbonation results, the next conclusions can be derivate:

- The carbonization is expressed on ceilingand walls.
- Both interior walls and exterior walls have the same influence of concrete carbonation. (see Figure VI-50).
- Other concrete elements were not tested.
- The front of carbonization came up to reinforced bars, even passed behind the bars.

Chloride test

The content of ion chloride in concrete is checked by using small pieces of drilled cores which were pulverized and dissolved in acid liquid. The chloride ions react with acid in an electrochemical reaction. An electrode was inserted into the liquid and the change in voltage was measured. On the basis of measured voltage, the instruments showed the chloride content in concrete in %. The obtained results are given in Table VI-11.

Table VI-11. Chloride test result

Elements 1	Reference 2	Rebar Presence 3		Rebar 4		Pouder quantity 5			Lab test result Chloride (%) 6		
		Yes	NO	Cov(cm)	Ø(mm)	0-2cm	2-6cm	6-8cm	0-2cm	2-6cm	6-8cm
Exterior wall(south)	06.01		X	2	12	18	26	20	0.0122	0.0034	0.0043
Interior (south)	06.02		X	2	12	26	23	17	0.0077	0.0035	0.0020
Ceiling -center	06.03		X	3	25	20	17	17	0.0175	0.0031	0.0023
Ceiling-center 13/04	06.04		X	2	12	16	16	13	0.0705	0.0629	0.0697
Ceiling-center 13/04	06.05		X	1	12	14	15	12	0.0661	0.0153	0.0090
Ceiling-north 13/04	06.06		X	2	12	12	14	13	0.1377	0.0417	0.0417

For analyzing given results next criterion was used: The maximum of chloride ion content by mass of cement for reinforced concrete with ordinary carbon steel is 0.40% (class Cl 0.40) (BS 8500).

After comparing obtained results with specified criterion, the next conclusion was made:

- All testing results are smaller than criteria value.
- Chloride content in concrete in the bridge structure is not hazardous to imbedded reinforced bars.

Core test

For testing concrete compressive strength, the core tests were done. Cores were extracted from three different locations. In order to determine differences between surface and inner concrete quality, cores were taken out from whole depth of elements. The chosen locations for taking out cores were:

- Exterior (Support) walls -three cores,
- Lateral beam – two cores and
- Deck ceiling – two cores.

In the laboratory extracted cores were splitting in the next way:

- In three parts from exterior walls and
- In two parts for lateral beam.

Then, all obtained cores were visually inspected and prepared for testing compressive strength by cupping. Testing procedure for compressive strength is described in standard BS 1881: Part 120:1983. All obtained results of estimate in-situ compressive strength are given in table VI-12, and they represent cube compressive strength. For changing cylinder compressive strength to cube compressive strength, the factor of correction was used. This factor depends of dimensions of specimens and of direction of drilling. On the basis of visual inspection, it was concluded that all specimens did not have reinforced bars and that all specimens were homogenous.

Table VI-12. Core test result

	Specimens 1	Direction of drilling [V/H] 2	Element 3	Density (kg/m3) 4	Cylindar strength (MPa) 5	Estimated In-situ cube Strength (MPa) 6
	N0.1.1	H	Exterior wall	2266.83	34.82	36
	N0.1.2	H	Exterior wall	2219.54	15.17	16
	N0.1.3	H	Exterior wall	2284.94	35.13	37
	N0.2.1	H	Exterior wall	2263.81	17.50	18
	N0.2.2	H	Exterior wall	2245.55	31.69	33
	N0.2.3	H	Exterior wall	2234.65	24.40	26
	N0.3.1	H	Exterior wall	2331.24	24.14	25
	N0.3.2	H	Exterior wall	2378.11	29.48	31
	N0.3.3	H	Exterior wall	2367.67	20.60	21
	N0.4.2	H	North lateral beam	2249.50	41.51	44
	N0.4.1	H	North lateral beam	2251.87	31.25	33
	N0.5.1	H	North lateral beam	2303.76	30.31	31
	N0.5.2	H	North lateral beam	2279.27	44.91	47
	N0.6	V	Deck ceiling	2215.52	22.64	27
	N0.7	V	Deck ceiling	2263.35	42.71	41

In aim to make conclusion of concrete quality, the average value and the range of estimated in-situ cube compressive strength are calculated and shone in table VI-13.

Analyzing that results it can be seen that the difference between minimum and maximum value for each tested element is large and vary from 14 to 21MPa.

This led to the conclusion that built-in concrete has very unequal quality and compressive strength differ from one to another location. Even at the same location the obtained compressive strengths are varied each other and depth of wall and beam influenced the quality of concrete.

Table VI-13. Compressive strength test result

Compressive strength of concrete cores in souk athulatha2 bridge (MPa)- cube values				
	Element 1	Cube result 2	fck, average 3	Range of fckn 4
	Exterior walls	36	27.14	16-37
		16		
		37		
		18		
		33		
		26		
		25		
		31		
		21		
	Slab ceiling	27	34.00	27-41
		41		
	Beams	44	38.50	31-47
		33		
		31		
		47		

Schmidt hammer test

For getting more information of built-in concrete quality the Schmidt hammer test, as a nondestructive- surface hardness method is chosen.

Data about tested elements and number of measure places are given in table VI-14.

Table VI-14. Tasted elements and number of measuring points

<table>
<tr><th>Element</th><th>Part of element</th><th>Number of measuring point</th><th>Total number of measuring point per element</th><th>Total number of measuring point</th></tr>
<tr><td rowspan="4">Arch slab</td><td>Ceiling-South side</td><td>3</td><td rowspan="4">14</td><td rowspan="7">30</td></tr>
<tr><td>Ceiling – north side</td><td>4</td></tr>
<tr><td>Ceiling - center</td><td>3</td></tr>
<tr><td>Ceiling-South side tunnel</td><td>4</td></tr>
<tr><td rowspan="2">Exterior wall</td><td>South side</td><td>2</td><td rowspan="2">9</td></tr>
<tr><td>North side</td><td>7</td></tr>
<tr><td>Interior wall</td><td>South side</td><td>7</td><td>7</td></tr>
</table>

On each test location 10 rebound readings were done. Prior to test the surface of concrete was smoothed by carbonudum stone and cleaned. Rebound number was calculated by using next rule: Each result from one test location is valid if it is in range of ±7 points of average value. For each reading the single compressive strength was calculated by using calibration curves and finally the average and standard deviation are calculated too. The calculate values of compressive strengths and standard deviations are given in table VI-15.

Table VI-15. Schmidt hammer test result

Element 1	Part of element 2	Wmed (MPa) 3	σ 4
Arch slab	Ceiling- south side	76.14	2.35
	Ceiling- south side	65.22	2.24
	Ceiling- south side	63.08	0.45
	Ceiling-north side	61.34	1.63
	Ceiling-north side	59.91	3.07
	Ceiling-north side	69.01	1.65
	Ceiling-north side	63.92	3.13
	Ceiling-center	80.99	0.96
	Ceiling-center	84.14	0.99
	Ceiling-center	73.79	0.94
	Ceiling- south side(tunnel)	71.19	1.52
	Ceiling- south side(tunnel)	67.39	0.66
	Ceiling- south side(tunnel)	60.67	1.50

	Ceiling- south side(tunnel)	60.67	1.50
Exterior wall	Exterior wall- south side	47.70	2.09
	Exterior wall-south side	39.70	1.71
	Exterior wall-north side	50.71	2.30
	Exterior wall-north side	44.42	1.06
	Exterior wall-north side	37.98	1.91
	Exterior wall-north side	47.16	1.79
	Exterior wall-north side	46.95	1.45
	Exterior wall-north side	50.71	2.30
	Exterior wall-north side	50.71	2.30
Interior wall	Interior wall-south side	26.62	2.15
	Interior wall-south side	38.23	0.93
	Interior wall-south side	58.05	2.39
	Interior wall-south side	31.31	2.47
	Interior wall-south side	20.77	1.32
	Interior wall-south side	29.31	1.64
	Interior wall-south side	23.11	1.39

Discussion and Conclusion

In order to make some conclusion of concrete compressive strength, obtained by Schmidt hammer test, the individual results were grouped and mean value of compressive strength and standard deviation were calculated. The obtained data are shown in Table VI-16.

Table VI-16. Schmidt hammer test result analyse

Element 1	Part of element 2	Wmed (MPa) 3	σ 4	Carbonization test
Arch slab	Ceiling- south side	68,15	±7,00	Not controlled
	Ceiling-north side	63,55	±4,00	Not controlled
	Ceiling-center	79,64	±5,30	Y
	Ceiling- south side(tunnel)	64,98	±5,21	Not controlled
Exterior wall	Exterior wall- south side	43,70	±5,66	Y
	Exterior wall-north side	46,95	±4,64	Not controlled
Interior wall	Interior wall-south side	32,49	±12,63	Y

The results given in previous table show a moderate dispersion of compressive strength for each analyzed element of structure except for Interior wall which has large dispersion. Obtained results of Schmidt hammer compressive strength for Arch slab are very high and uniform, for Exterior wall moderate and also satisfactory uniform and moderate too low for Interior wall, but unequal and with large dispersion. According to this analyze the next conclusion can be derived: the built-in concrete in most tested elements has satisfactory uniformity.

Some results given in Table VI-16 can be compared with results of compressive strength obtained by testing cores, Table VI-13 but only if carbonization takes into account.

It is well known that the carbonation makes concrete surface layer to be harder. In extreme cases the overestimate of compressive strength from this cause may be up to 50%. On the base of carbonation test results (Table VI-10) the majority of tested elements are affected with process of carbonation and, according to previous comment, show the higher values of compressive strength from the real value. Therefore, the real estimated compressive strengths of tested elements are smaller from values given in table (6.15) for all tested elements and have to be corrected.

The coefficient of correction is calculated by average of compressive strength obtained by core and average of compressive strength obtained by Schmidt hammer:

$f_{ck,av}$=31.07MPa (average compressive strength obtained by core)

$f_{ch,av}$=53.36MPa(average compressive strength obtained by Schmidt hammer)

$F_{c, comp}$=31.07/53.36=0.5823

In table VI-16a the average results of Schmidt hammer compressive strength before and after correction has been shown.

Table VI-16a. Schmidt hammer test compressive strength before and after correction

Element	compressive strength before correction (MPa)	compressive strength after correction (MPa)
Interior wall	32.49	18.92
Exterior wall (south side)	43.70	25.44
Exterior wall (north side)	46.95	27.34
Ceiling slab (south)	68.15	39.68
Ceiling slab (north)	63.54	37.00
Ceiling slab (centre)	79.64	46.37
Tunnel ceiling (south)	64.98	37.87
Top slab	Not verified	Not verified

After correction of results the next conclusion is made:

The highest value of concrete compressive strength in center ceiling slab is 46.37MPa

The smallest value of concrete compressive strength in interior wall is18.92MPa.

Comparing compressive strengths for exterior wall it is concluded that there is no significant difference, when carbonation reduction is taken account, but the different conclusion can be made for ceiling slab (center). The higher value has been obtained by Schmidt hammer, even when the carbonization is taken in the analysis.

Pull off test

For measuring in-situ concrete tensile strength the pull-off method is used. The procedure is described in BS 1881: Part 207. The test were conducted in three places. Obtained results are given in table VI-17.

Table VI-17. Pull off test result

<table>
<tr><td rowspan="2">Reference
1</td><td rowspan="2">Element
2</td><td rowspan="2">W med (MPa)
3</td><td rowspan="2">σ
4</td><td colspan="3">Failure mode (%)
5</td></tr>
<tr><td>concrete</td><td>Surface concrete</td><td>Epoxy glue</td></tr>
<tr><td>06.01/06.05</td><td>Interior wall</td><td>0.520</td><td>0.23</td><td>40%</td><td>38%</td><td>22%</td></tr>
<tr><td>06.06/06.10</td><td>Exterior wall</td><td>0.870</td><td>0.63</td><td>100%</td><td>0%</td><td>0%</td></tr>
<tr><td>06.11/06.15</td><td>Deck ceiling</td><td>0.741</td><td>0.24</td><td>91%</td><td>2%</td><td>7%</td></tr>
</table>

On the bases of given results, it can be seen that all values of tensile strength are small and smaller than minimum require value and that build-in concrete has very bad quality.

Density

Calculation of density of hardened concrete is very good method for checking the quality of built-in concrete. For calculation of the concrete density, the mass of extracted cores is usually used. Obtained results are given in table VI-18.

On the bases of given results, it can be seen that all values of densities are close each other. It can be concluded that concrete built in all tested elements has good uniformity of density (~2260kg/m^3).

Table VI-18. Density test result – analyze

<table>
<tr><td>Element of structure</td><td>Density, kg/m^3</td><td>Average for each measuring place kg/m3</td><td>Average for element of structure, kg/m^3</td></tr>
<tr><td rowspan="9">Exterior wall</td><td>2331.24</td><td rowspan="3">2359</td><td rowspan="9">2288</td></tr>
<tr><td>2378.11</td></tr>
<tr><td>2367.67</td></tr>
<tr><td>2266.83</td><td rowspan="3">2257</td></tr>
<tr><td>2219.54</td></tr>
<tr><td>2284.94</td></tr>
<tr><td>2263.81</td><td rowspan="3">2248</td></tr>
<tr><td>2245.55</td></tr>
<tr><td>2234.65</td></tr>
<tr><td></td><td>2215.52</td><td>2239</td><td>2239</td></tr>
</table>

<table>
<tr><td>Slab ceiling</td><td>2263.35</td><td></td><td></td></tr>
<tr><td rowspan="4">Beams</td><td>2249.50</td><td rowspan="2">2251</td><td rowspan="4">2272</td></tr>
<tr><td>2251.87</td></tr>
<tr><td>2303.76</td><td rowspan="2">2292</td></tr>
<tr><td>2279.27</td></tr>
</table>

2.2.2. Visual inspection of Bridge Souk Athulatha2

First visual inspection of all visible bearing elements or part of bearing elements of bridge were done in 2009.

During the visual inspection it has been noticed that horizontal elements of structure elements were plastered by thin layer of ordinary cement mortar and covered by paint.

During the visual inspection a lot of damages were registered.

Characteristic damages are:

- Corrosion of reinforced
- Damage of concrete due to reinforced corrosion.
  - Falling down of cover.
  - Cracking of cover
  - Separation and falling down of plaster layer
- White stains on concrete surface (water soluble salts)
- Dark stains on concrete surface (water overflow)

Main reasons that caused described damages are:

- Carbonation
- Poor quality of concrete
- Very small tensile strength (adhesion)
- Insufficient depth of cover
- Wind
- No adequate water drainage system.

Figures VI-51, VI-52, VI-53, VI-54, VI-55 and VI-56 show characteristic damages of RC elements.

Figure VI-51. General view of the bridge

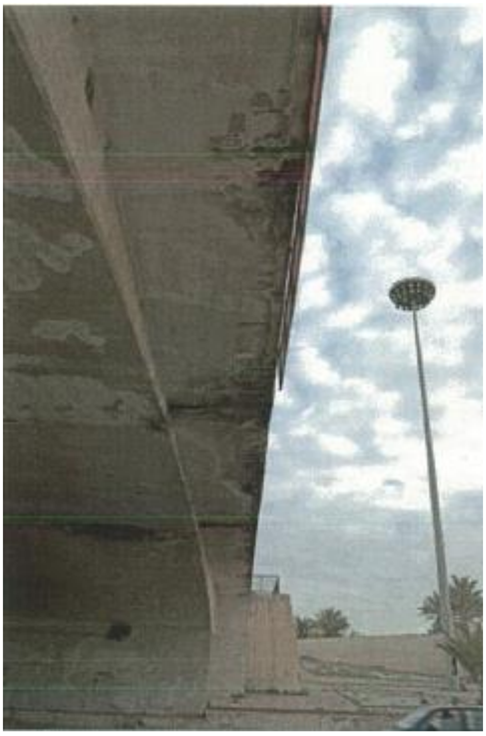


Figure VI-52. Cantilever slab and part of the ceiling deck; falling off painting and plaster layer, white and dark stains along the edge of slab, damage of concrete due to corrosion of reinforced bars

Figure VI-53. Deck ceiling, falling off painting and plaster layer, damage of concrete due to corrosion of reinforced bars

Figure VI-54. Cantilever slab and deck ceiling slab; the largest damage in the centre of these elements

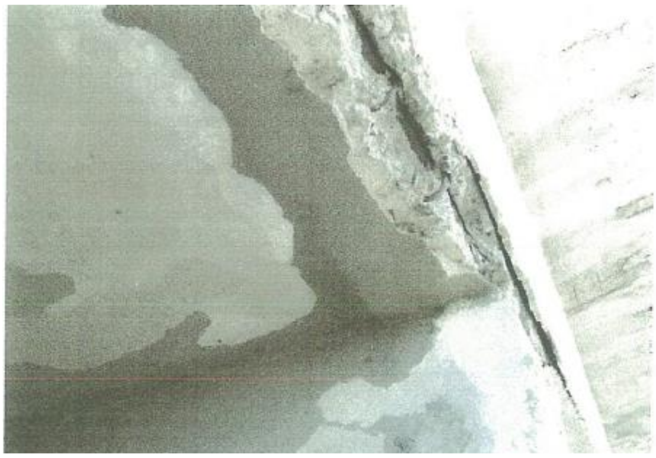


Figure VI-55. Detail of ceiling slab: falling off painting and plaster layer due to bad adhesion and concrete cover caused by corrosion of bars

Figure VI-56. Interior wall; longitudinal cracks near edges due to corrosion of reinforced bars

The most damaged elements are cantilever slabs with edge beams. The characteristic damage is falling down of concrete cover and ordinary plaster layer (Fig. VI-52, VI-54). Some reinforced bars were bared and affected by surface corrosion. The edges of slabs are rough and stain of water- and water-soluble salts can be seen.

The next most damage element is deck ceiling slab. The most damaged parts of slab are lateral beams and center of simple beam slab (Fig. VI-53, VI-55). The characteristic damage on both places is reinforced corrosion. Due to the expansion of corroded bars, the concrete cover is cracked and fallen off. Apart from the center of ceiling slab, described damage has been noticed at several locations on the down side of ceiling slab (Fig. VI-53).

Exterior walls have been damaged due to reinforced corrosion. Characteristic damage is vertical cracks, along bars near the edges of wall (Fig. VI-56).

Other concrete structural elements (interior walls and underpass (tunnel) ceilings) have been also damaged due to corrosion of reinforced bars.

Visual inspection encompassed other bridge elements, like sidewalks, curb stones, catch pits and fences. All mentioned elements have been seriously damaged.

2.3. General conclusion for bridge Souk Athulatha2

The bridge Souk Athulatha 2 has been old about 50 years when it was inspected for the first time. The main conclusion of the inspection was that the bridge is damaged.

The characteristic defect of arch slabs and lateral beams have been insufficient concrete caver and bad quality of concreting works. These defects were unsuccessfully solved by plastering with ordinary cement mortar.

The main cause of damage appearance is carbonation. Almost all inspected elements had the problem with carbonation especially ceiling. In some cases, the front of carbonation even passed behind the bars.

The second cause of damage appearance is inadequate drainage of water from the deck. This problem caused leakage of water through joints and overflow of water over the edge of cantilever slabs. Consequently, the corrosion of reinforced bars in deck ceiling and cantilever slabs were caused.

Analyzing concrete compressive strength obtained by cores it can be seen that the difference between minimum and maximum value for each tested element is large and vary from 14 to 21MPa. This led to the conclusion that built-in concrete has very unequal quality and compressive strength differ from one to another location

The results of concrete compressive strength obtained by Schmidt hammer test show a moderate dispersion of compressive strength for each analyzed element of structure except Interior wall which shows a large dispersion.

Comparing compressive strengths obtained by cores and by Schmidt hammer test it is concluded:

- The Schmidt hammer test and core test were not performed on the same element of bridge, so there are not enough results for comparing.
- There is no significant difference, when carbonation reduction is taken account for compressive strength of concrete built in Exterior walls, but for ceiling slab (center) the difference is significant.
- The higher value has been obtained by Schmidt hammer, even when the carbonization is taken in the analysis.

So, the general conclusion can be established that built in concrete has large dispersion in quality from very high (~47MPa) up to very low (~16MPa).

According EN 206-1 the compressive strength classes of concrete given in next table can be used for the control calculation.

Bridge Element	Compressive strength class
Interior wall	C20/25
Exterior wall	C20/25
Ceiling slab	C25/30
Top slab	-

On the bases of results obtained by pull-off method it can be concluded that concrete tensile strength is very low and smaller than minimum require value.

Chloride content in concrete in the bridge structure is not hazardous to imbedded reinforced bars.

Finally, the main conclusion can be drawn:

- Durability of all structural elements is decreased, because of numerous damages that occurred in elapsed time.
- Bearing capacity of structural elements is not jeopardized because there are no serious cracking or deformations of RC elements.
- Global stability and stability of each structural element are not threatened and
- Functionality of bridge is partly reduced, because of damages of surface asphalt layers and local instability of delaminated concrete pieces, that occurred on the bottom sides of ceiling slab, lateral beams cantilever slabs and edge beams.

3. ALSSEKA ROAD BRIDGE

Technical description, assessment, rating and repair of bridges in Tripoli (2009)

3.1. Technical description

Bridge Alsseka Road is located in the east part of the capital Tripoli, about 2.66km from the sea to the north. It is considered as a major bridge to the capital Tripoli. It connects the city center and the university and connects the roads leading to the collection of state institutions buildings. In Figure VI-58, VI-59 and VI-60 are shown situation plan and views of bridge. The coordinates for this bridge are 32052'22'' N 130 11'55''E.

Figure VI-58. Alsseka Road Bridge location on Google Maps

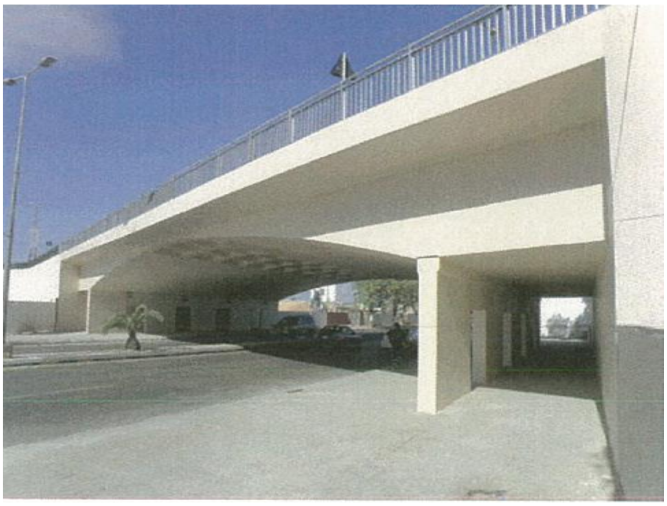


Figure VI-59. Alsseka bridge interior wall and arch cantilever

Figure VI-60. Alsseka bridge ceiling and cantilever

Type of bridge

Bridge Alsseka Road is designed as Simple Arch Bridge made of reinforced concrete. This bridge was built in the middle of XX centuries. In Figure VI-61 and VI-62 north and south sides of the bridge are shown. The plan of the bridge is given in Figure VI-63.

The characteristic dimensional data of the bridge are:

- Length: 40.12m
- width: 27.90m
- Height: 4.85m
- Main span: 28.50m
- Sidewalk (right side): 4.0m
- Sidewalk (left side): 3.90m

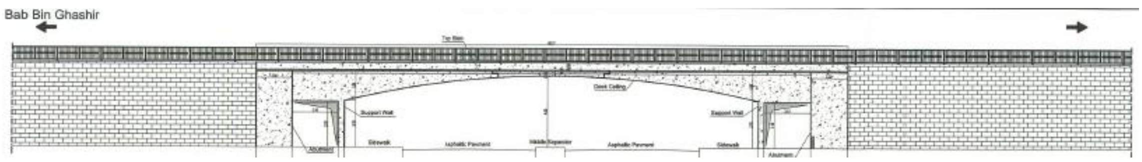


Figure VI-61. Longitudinal cross section of bridge (north side)

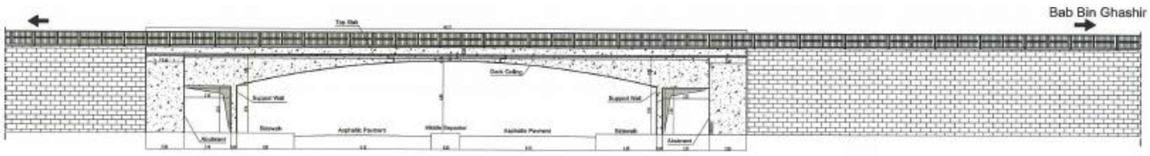


Figure VI-62. Longitudinal cross section of bridge (south side)

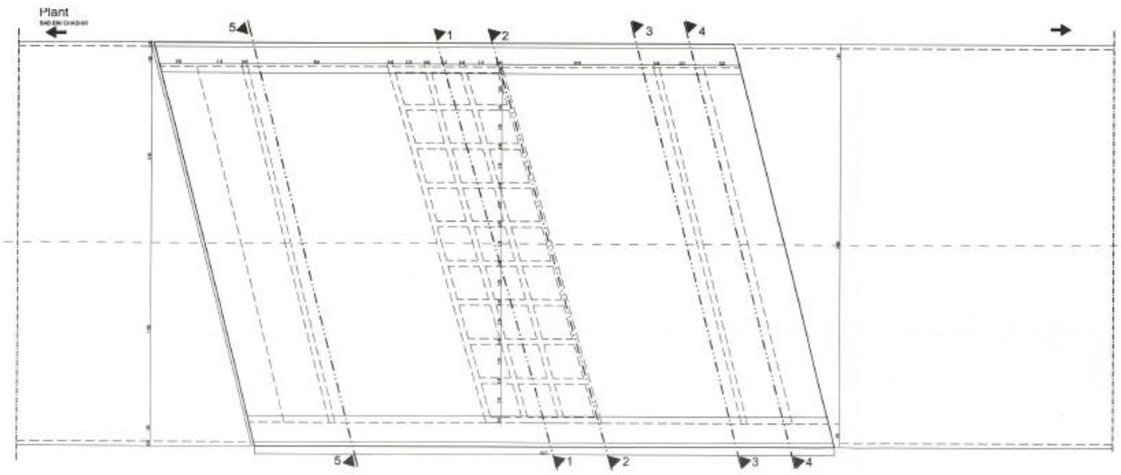


Figure VI-63. Plan of the bridge – upper side

Basic elements of bridge are:

- Interior wall (support wall)
- Exterior wall (abutment)
- Arc cantilever slab
- Simple Beam slab

Arc cantilever slabs and simple beam slab were continuous during the construction.

Disposition of basic bridge elements are signed in Figures VI-64 and VI-65.

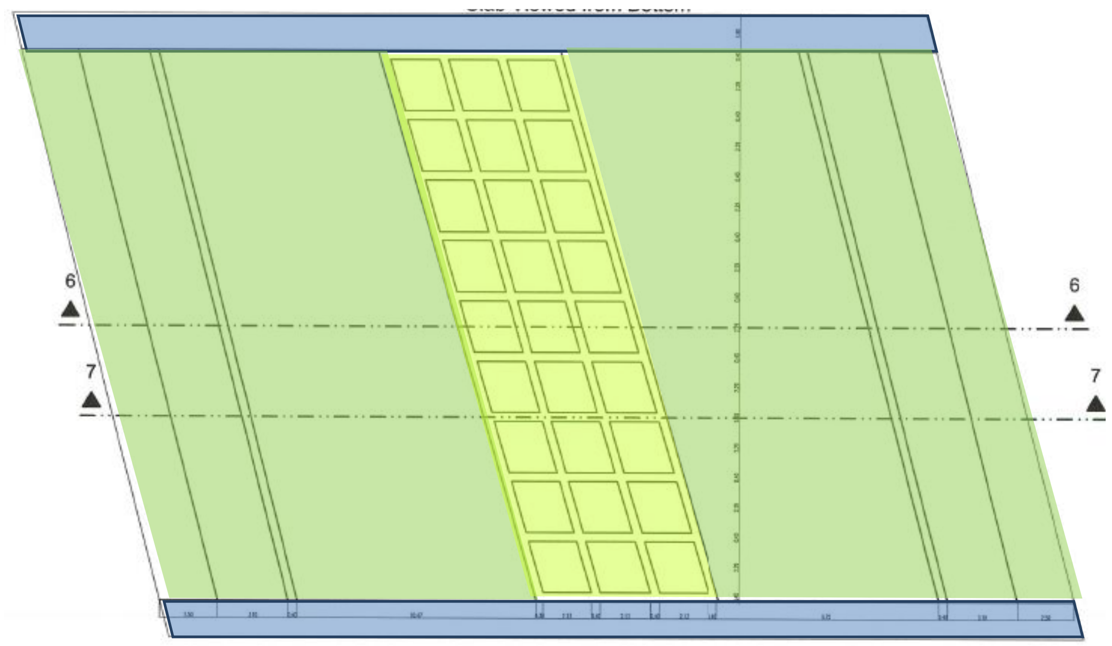


Figure VI-64. Disposition of arch cantilever slabs (green), simple beam slab (yellow)and cantilever side slab (blue) in plane of the bridge (bottom side)

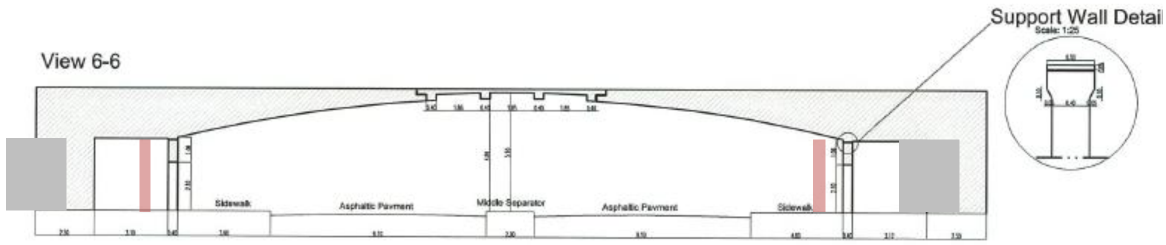


Figure VI-65. Disposition of exterior (brown) and interior walls of bridge (gray) (section 6-6)

The following text provides a brief description of basic elements of the bridge. Bridge Alsseka Road has two interior walls. Both walls have no openings. The basic dimensions of each interior wall are:

Interior wall on east side:

- Length: 24.50m
- Height: 5.08m (visible part of total height)
- Depth: 2.30 m

Interior wall on west side

- Length: 24.50m
- High: 5.08m (visible part of total height)
- Depth: 2.40m

Bridge Alsseka Road has two exterior walls. Both walls have four openings. The basic dimensions of each exterior wall are:

Exterior wall on west side and east side:

- Length: total 24.50m (with four openings)
- Height: variable 3.13m (visible part of total height)
- Depth: 0.4m

Figure VI-66 shows the general view of exterior walls.

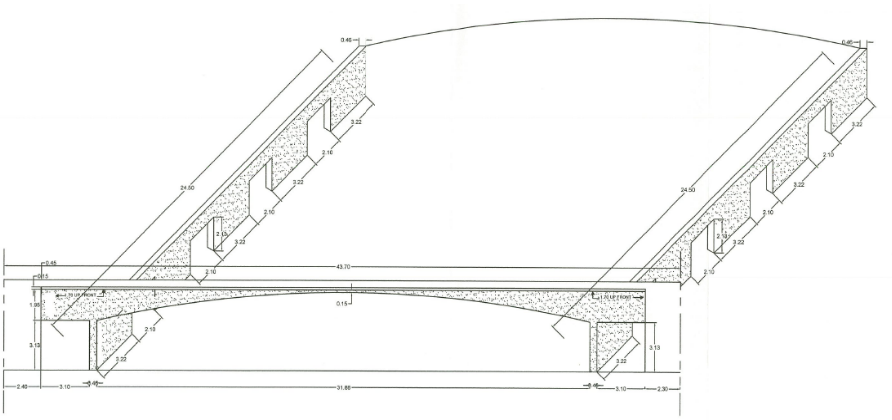


Figure VI-66. General view of exterior walls

Upper (horizontal) part of the bridge is designed as arc slab. This slab consists of two arc cantilever slabs and simple beam slab (Figure 7.64). Arc cantilever slabs have solid cross section. The basic data of arc cantilever slabs are:

- Length: 14.97m
- Width: 22.18m
- Depth: variable from 45cm (hinge) up to the 220cm (fixed end)

In Figures VI-67, VI-68 and VI-69 the view of arch slab from bottom side and cross sections near hinge and fixed ends are shown.

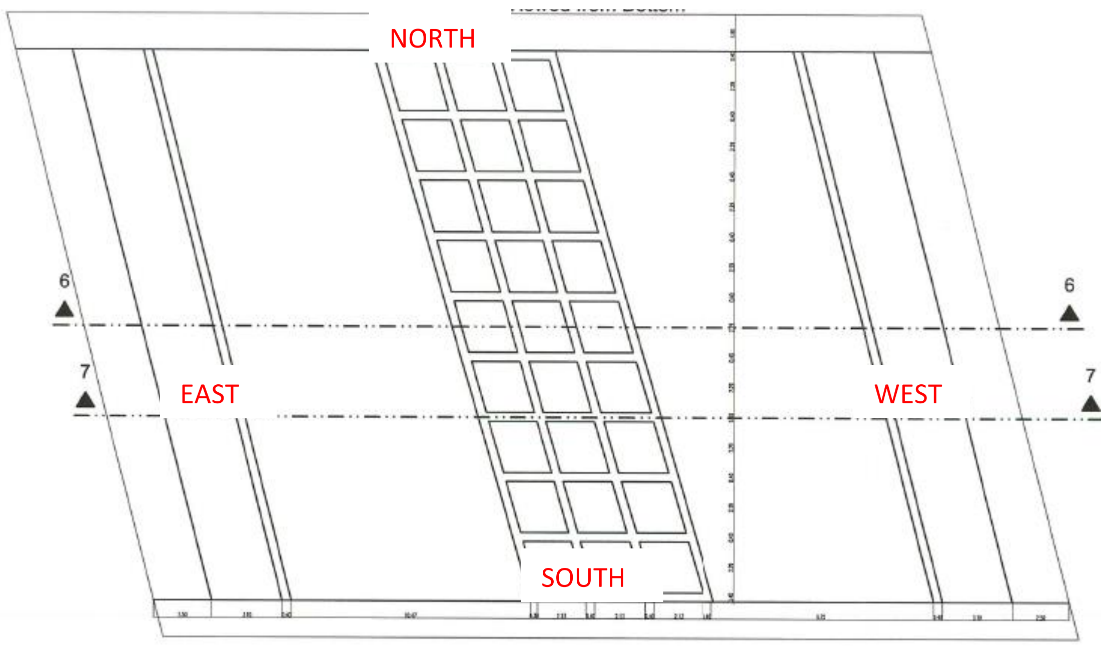


Figure VI-67. Arch slab view from bottom side and location of cantilever arch slabs

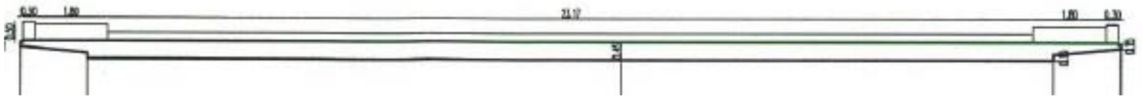


Figure VI-68. Cross section of arch slab in the hinge (section 2-2)

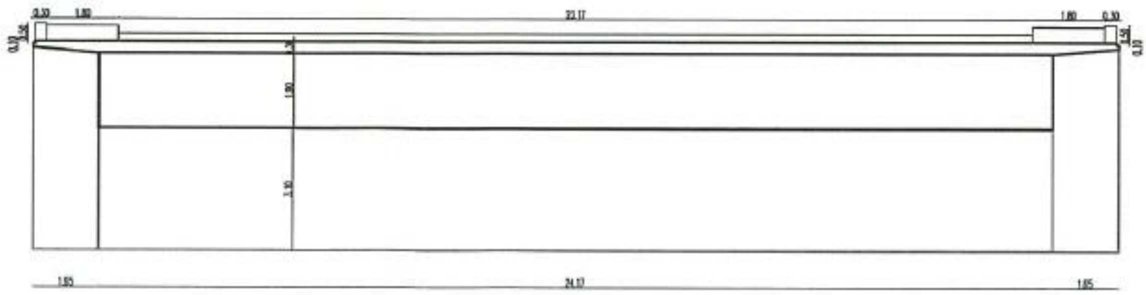


Figure VI-69. Cross section of arch slab near the fixed ends (section 4-4)

Simple beam slabs located in the middle of span. It has two-way ribbed cross section. Characteristic dimensions are:

- Length: 7.16m
- Width: 24.17m
- Depth: 45cm

Disposition of simple beam slab and characteristic cross sections are given in Figures (VI-65, VI-70, VI-71, VI-72, VI-73, VI-74) below.

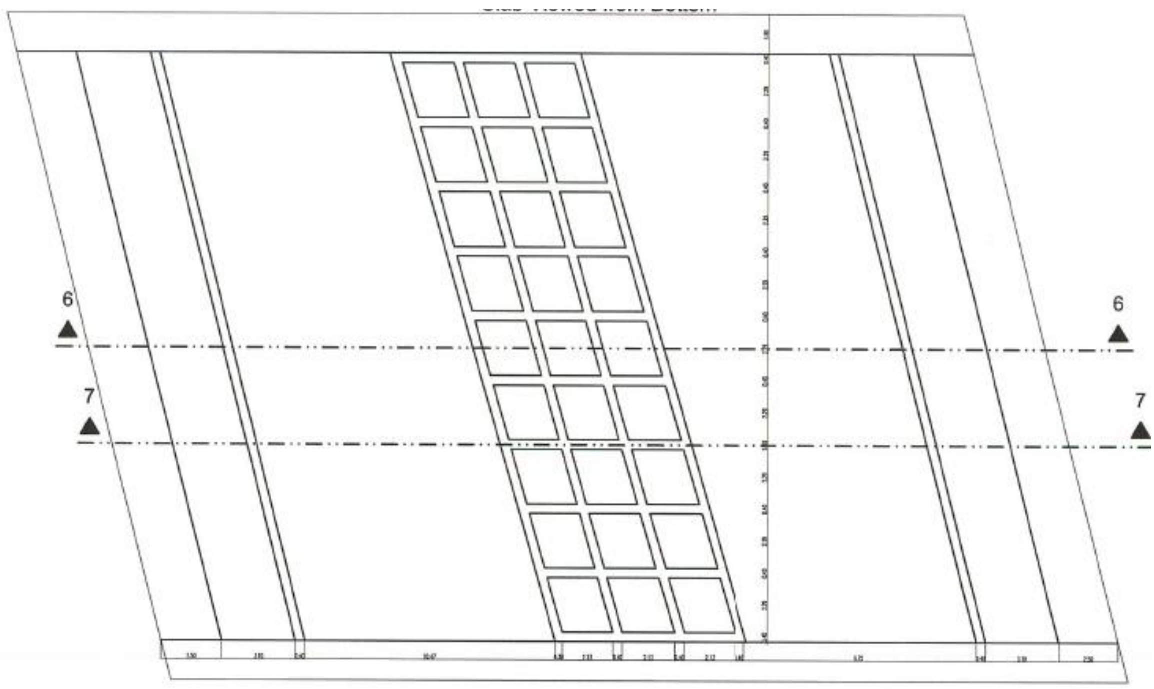


Figure VI-.70 Simple beam slab view from bottom side

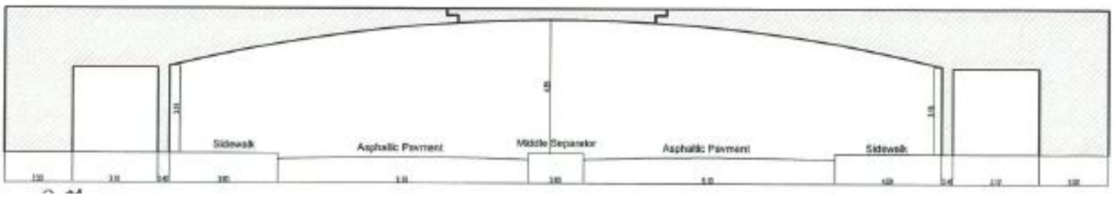


Figure VI-71. Disposition of simple beam slab in the span of bridge (Section 7-7)

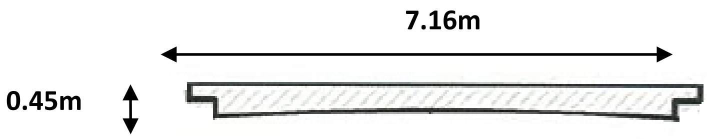


Figure VI-72. Dimensions of simple beam slab in cross section

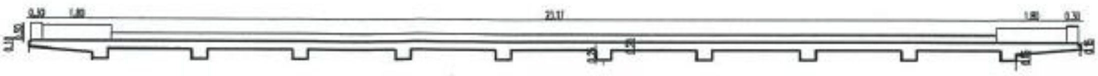


Figure VI-73. Longitudinal view of simple beam slab

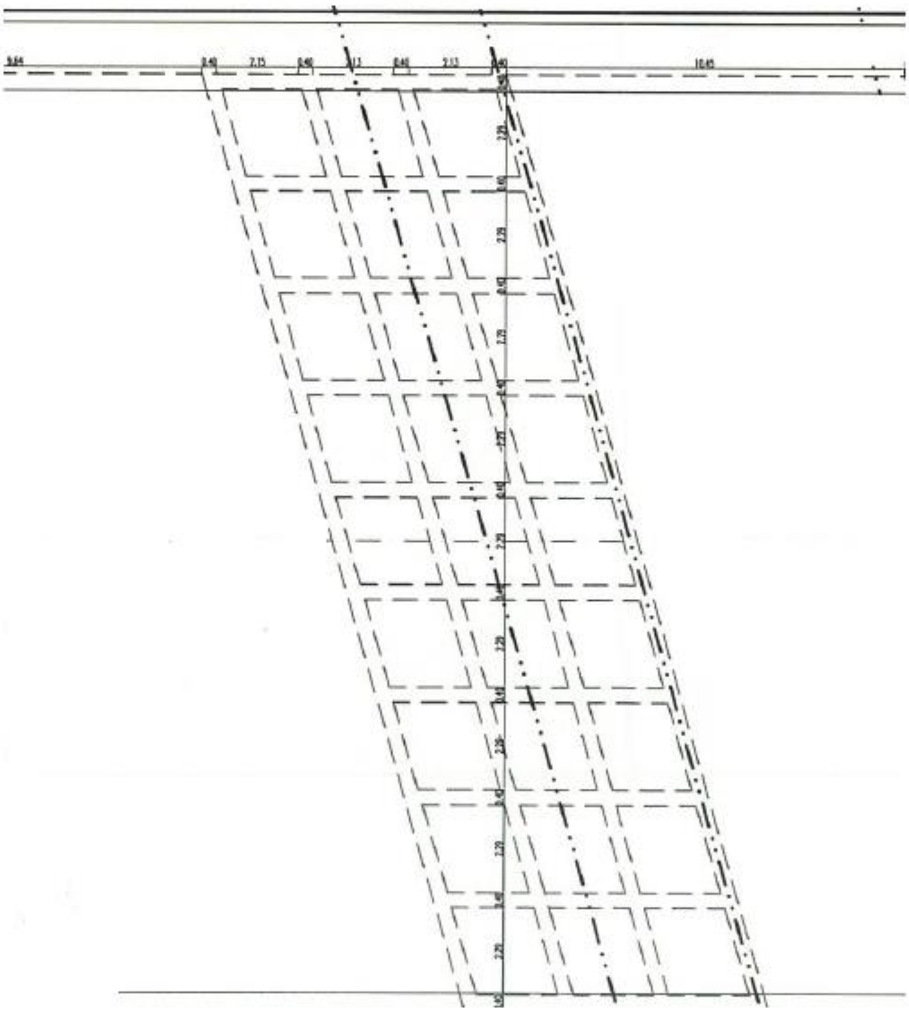


Figure VI-74. Plane of simple beam slab with characteristic dimensions

Bridge Alsseka Road has two pedestrian paths that are designed as Cantilever side slabs with edge beam. The characteristic dimensions of side slabs (Figure 7. 78) are:

- Length: 40.12m
- Width: 1.65m
- Depth:variable from 18cm (free end) up to the 40cm (fixed end)

and of end beam (Figure VI-. 78) are:

- Length: 40.12m
- Cross section: 0.50mx0.30m

1.80m **0.30m**

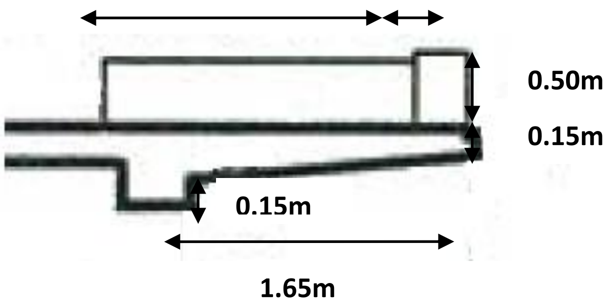


Figure VI-78. Cantilevers

3.2. Assessment of Bridge Alsseka Road

In the aim of choose repair materials and technics for this bridge, next activities were planned:

- In-situ testing of concrete quality and
- Visual inspection of visible parts of bearing elements.

Numbered activities were done in 2009.

3.2.1 Testing of concrete in bearing elements of bridge

The testing of concrete quality encompassed the next activities:

- Measurement of carbonation depth,
- Chloride ion content,
- In-situ testing of concrete by taking of cores,
- In-situ testing of concrete by Schmidt Hammer test and
- In-situ testing of concrete by Pull-off method.

Carbonation depth

The extent of carbonation was assessed by treating with phenolphthalein indicator the fresh exposed surfaces of drilled cores, which were extracted from structure elements for testing concrete compressive strength or for testing carbonation depth.

All data of testing elements, measured depth of carbonation and rebar location are given in Table VI-19.

Table VI-19. Data of testing elements, measured depth of carbonation and rebar location

Reference 1	Carbonization 2		Cover Depth 3	Depth of carbonization 4	In rebar plan 5	Element 6
	Yes	No	mm	mm		
03.01	x		-	20	-	Exterior wall
03.02	x		-	20	-	Exterior wall
03.03	x		-	20	-	Exterior wall
03.04	x		-	20	-	Exterior wall
03.05	x		-	10	-	Exterior wall
03.06	x		-	10	-	Exterior wall
03.07	x		-	10	-	Exterior wall
03.08	x		-	80	-	Interior wall
03.09	x		-	90	-	Interior wall
03.10	x		-	80	-	Interior wall
03.11	x		-	80	-	Interior wall
03.12	x		30	60	Y	Interior wall
03.13	x		90	80	Y	Interior wall
03.14	x		90	80	Y	Interior wall
03.15	x		10	50	Y	Ceiling
03.16	x		10	70	Y	Ceiling
03.17	x		20	60	Y	Ceiling
03.18	x		20	60	Y	Ceiling

After analyzing carbonization results, the next conclusions can be derivate:

- The minimum depth of carbonization is 10mm.
- The maximum depth of carbonization is 90mm
- The mean valuesare:15.7mm for exterior walls, 78.6mm for interior wall and 60mm for ceiling
- The front of carbonization come up to reinforced bars, even passed behind the bars (especially in interior wall and ceiling).
- The carbonization is most expressed of abutment (interior wall) and ceiling.

Chloride test

The content of ion chloride in concrete is checked by using small pieces of drilled cores which were pulverized and dissolved in acid liquid. The chloride ions react with acid in an electrochemical reaction. An electrode was inserted into the liquid and the change in voltage was measured. On the basis of measured voltage, the instruments showed the chloride content in concrete in %. The obtained results are given in Table VI-20.

Table VI-20. Chloride test result

Elements 1	Reference 2	% Chloride in concrete (Equipment reading) 3			% Chloride ion content by mass of cement 4		
		0-2cm	2-6cm	6-8cm	0-2cm	2-6cm	6-8cm
Exterior wall	03.01	0.0206	0.0036	0.0019	0.0026	0.0005	0.0003
Exterior wall	03.02	0.0040	0.0020	0.0014	0.0005	0.0003	0.0002
Interior wall	03.03	0.0064	0.0036	0.0028	0.0008	0.0005	0.0004
Ceiling 16/4/09	03.04	0.0117	0.0086	0.0043	0.0015	0.0011	0.0005
Ceiling 16/4/09	03.05	0.0218	0.0063	0.0047	0.0027	0.0008	0.0006
Ceiling 16/4/09	03.06	0.0035	0.0035	0.0044	0.0004	0.0004	0.0006

In analyzing given results next criterion was used: The maximum of chloride ion content by mass of cement for reinforced concrete with ordinary carbon steel is 0.40% (class Cl 0.40) (BS 8500)

After comparing obtained results with specified criterion, the next conclusion was made:

- All testing results are smaller than criteria value.
- Chloride content in concrete in the bridge structure is not hazardous to imbedded reinforced bars.

Core test

For testing concrete compressive strength, the core tests were done. Cores were extracted from two different locations. In order to determine differences between surface and inner concrete quality, cores were taken out from whole depth of elements. The chosen locations for taking out cores were:

- Exterior walls -three cores,
- Deck ceiling – beams - two cores.

In the laboratory extracted cores were splitting in the next way:

- In three parts from exterior walls and
- In one part for ceiling beam.

Then, all obtained cores were visually inspected and prepared for testing compressive strength by cupping. Testing procedure for compressive strength is described in standard BS 1881: Part 120:1983. All obtained results of estimate in-situ compressive strength are given in table7.21, and they represent cube compressive strength. For changing cylinder compressive strength to cube compressive strength, the factor of correction was used. This factor depends of dimensions of specimens and of direction of drilling.

In aim to make conclusion of concrete quality, the average value and the range of estimated in-situ cube compressive strength are calculated and shone in the same table VI-21.

Table VI-21 Compressive strength test result

Compressive strength of concrete cores in ALSSEKA ROAD bridge (MPa)- cube values				
	Element 1	Cube result 2	fck, average 3	Range of fckn 4
	Exterior walls	16	21.86	11-28
		11		
		23		
		23		
		28		
		24		
		20		
		24		
		23		
	Beams	20	23.00	20-33
		25		
		23		
		21		
		33		

Analyzing those results it can be seen that the difference between minimum and maximum value for exterior wall is large and amounts to 17MPa. This led to the conclusion that built-in concrete has very unequal quality and compressive strength differ from one to another location. The compressive strength varies by depth for the same location for two of three extraction places.

But for the ceiling it has been concluded that only one result has large deviation. When we ignore this result, the difference between minimum and maximum value becomes small and built-in concrete has good uniformity.

The obtained value of concrete compressive strength for both tested elements is small (~22MPa).

Schmidt hammer test

For getting more information of built-in concrete quality the Schmidt hammer test, as a nondestructive- surface hardness method is chosen. Data about tested elements and number of measure places are given in table VI-22.

Table VI-22. Tasted elements and number of measuring points

Element	Number of measuring point	Total number of measuring point per element	Total number of measuring point
Interior wall	11	11	30
Exterior wall	10	10	
Ceiling	9	9	

On each test location 10 rebound readings were done. Prior to test the surface of concrete was smoothed by carbonudum stone and cleaned. Rebound number was calculated by using next rule: Each result from one test location is valid if it is in range of ±7 points of average value. For each reading the single compressive strength was calculated by using calibration curves and finally the average and standard deviation are calculated too. The calculate values of compressive strengths and standard deviations are given in table VI-23.

Table VI-23. Schmidt hammer test result

Element 1	Wmed (MPa) 2	σ 3
Interior wall	12.30	0.87
	12.55	0.54
	10.07	1.58
	10.50	0.65
	10.64	0.45
	10.09	0.62
	10.09	0.62
	10.14	1.11
	11.58	0.98
	11.69	0.73
	12.70	0.59
Exterior wall	40.00	1.42
	40.00	1.42
	28.80	2.30
	28.43	2.18
	28.94	2.22
	34.49	1.01
	33.14	2.11
	36.87	0.85

	32.61	1.79
	35.08	1.45
Ceiling	33.13	1.58
	36.85	1.03
	29.54	1.44
	29.50	0.83
	32.31	0.86
	30.24	0.84
	35.17	1.15
	33.58	1.28
	35.10	1.57

Discussion and Conclusion

In order to make some conclusion of concrete compressive strength, obtained by Schmidt hammer test, the individual results were grouped and mean value of compressive strength and standard deviation were calculated. The obtained data are shown in Table VI-24.

Table VI-24. Schmidt hammer test result analyse

Element 1	W_{med} (MPa) 2	σ (MPa) 3	Carbonization test
Interior wall	11,12	±1,061585	Y
Exterior wall	33,70	±4,316823	Y
Ceiling	32,82	±2,653597	Y

The results given in previous table show a very small dispersion of compressive strength for each analyzed element of structure, but very large dispersion between interior wall and other tested elements (exterior wall and ceiling). The results obtained by Schmidt hammer for interior wall are too small for reinforced concrete. According to this analyze the next conclusion can be derived: the built-in concrete has very bad uniformity.

Some results given in Table VI-24 can be compared with results of compressive strength obtained by testing cores, Table VI-21. Comparing compressive strengths for exterior wall it is concluded that there is no significant difference, when carbonation reduction is taken account. The similar conclusion can be made for ceiling beams.

It is well known that the carbonation makes concrete surface layer to be harder. In extreme cases the overestimate of compressive strength from this cause may be up to 50%. On the base of carbonation test results the majority of tested elements are affected with process of carbonation and, according to previous comment, show the higher values of compressive strength from the real value. Therefore, the real estimated compressive strengths of tested elements are smaller from values given in table VI-24 for ceiling and exterior wall.

The coefficient of correction is calculated by average of compressive strength obtained by core for exterior wall and ceiling and average of compressive strength obtained by Schmidt hammer for the same elements:

fck,av=22.43MPa (average compressive strength obtained by core)

fch,av=33.26MPa (average compressive strength obtained by Schmidt hammer)

Fc, comp=22.43/33.26=0.6744

In table VI-24a the average results of compressive strength before and after correction has been shown.

Table VI-24a. Correction of Schmidt hammer compressive strength

Element	compressive strength before correction (MPa)	compressive strength after correction (MPa)
Interior wall	11,12273	-
Exterior wall	33,70	22.70
Ceiling	32,82	22.12

After comparison of given results, the next conclusions could be made:

The compressive strength of concrete built in exterior wall and ceiling corresponds to the class of concrete C16/20.

The results obtained by Schmidt hammer for interior wall are too small for reinforced concrete.

Pull off test

For measuring in-situ concrete tensile strength the pull-off method is used. The procedure is described in BS 1881: Part 207. The steel disks and epoxy resin glue were used. The test was conducted in three places. Obtained results are given in table VI-25.

Table VI-25. Pull off test result

Reference 1	Element 2	W med (MPa) 3	σ 4	Failure mode (%) 5		
				concrete	Surface concrete	Epoxy glue
03.01/03.05	Exterior wall	0.880	0.29	84%	16%	0%
03.06/03.10	Interior wall	0.180	0.05	20%	38%	42%
03.11/03.15	Deck ceiling	0.469	0.23	98%	0%	2%

On the bases of given results, it can be seen that all values of tensile strength are smaller than minimum require value and that build-in concrete has very bad quality.

Density

Calculation of density of hardened concrete is very good method for checking the quality of built-in concrete. For calculation of the concrete density, the mass of extracted cores is usually used. Obtained results are given in table VI-26.

On the bases of given results, it can be seen that all values of densities are smaller than expected value (~2300kg/m^3). It is supposed that the concrete was not enough compacted and because of that has small density.

Table VI-26. Density test result analyse

<table>
<tr><th>Element of structure</th><th>Density, kg/m^3</th><th>Average for each measuring place kg/m^3</th><th>Average for element of structure, kg/m3</th></tr>
<tr><td rowspan="9">Exterior wall</td><td>2028.29</td><td rowspan="3">2078</td><td rowspan="9">2185</td></tr>
<tr><td>2064.84</td></tr>
<tr><td>2141.82</td></tr>
<tr><td>2278.12</td><td rowspan="3">2266</td></tr>
<tr><td>2274.78</td></tr>
<tr><td>2245.84</td></tr>
<tr><td>2201.54</td><td rowspan="3">2211</td></tr>
<tr><td>2206.66</td></tr>
<tr><td>2225.85</td></tr>
<tr><td rowspan="5">Beams</td><td>2145.33</td><td rowspan="3">2194</td><td rowspan="5">2181</td></tr>
<tr><td>2226.93</td></tr>
<tr><td>2208.82</td></tr>
<tr><td>2138.02</td><td rowspan="2">2167</td></tr>
<tr><td>2196.69</td></tr>
</table>

3.2.2. Visual inspection of Bridge ALSSEKA ROAD

First visual inspection of all visible bearing elements or part of bearing elements of bridge were done in 2009.

All visible surfaces have been painted in the past and paint covered some faulty spots (honeycombs, segregations, bared reinforced bars…).

All elements have very rough surface because the wooden frameworks were used during concreting.

During the visual inspection o lot of damages were registered.

The characteristic damages are:

- Corrosion of reinforced
- Damage of concrete due to reinforced corrosion.

  - Falling down of cover.
- White stains on concrete surface (water soluble salts)
- Pilling off protecting coating
- Deformed and twisted reinforced bars in ceiling beam.

Main reason that caused described damages are:

  - Carbonation
  - Insufficient depth of concrete cover
  - Poor quality of concrete
  - Wind
  - No adequate water drainage system
  - Impact by vehicles

Figures VI-79-VI-85 show characteristic damages of RC elements.

Figure VI-79. Desk ceiling, pilling off painting cover

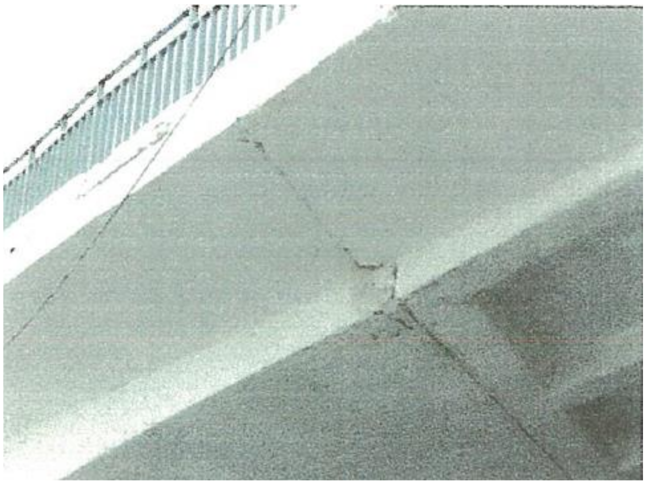


Figure VI-80. Desk ceiling, damaged joint between simple beam slab and arch slab, local corrosion of bars

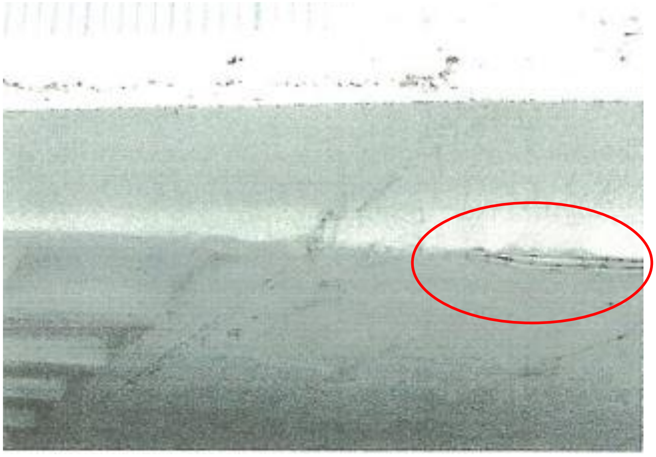


Figure VI-81. Bared and deformed reinforced bars

Figure Vi-82. Bared, deformed and twisted reinforced bars in ceiling beam

Figure VI-83 Ceiling, Local honeycomb, visible corroded bars

Figure VI-84. View of cantilever slabs with edge beams.

Figure VI-85. Tunnel between the interior wall and exterior wall, uneven concrete surface

The characteristic defect of ceiling is small cover depth. The average depth is ~1cm, but in some places not exists.

The characteristic damage of ceiling is falling down of thin concrete cover and pilling off painting cover (Fig. VI-79). Some reinforced bars were borne and affected by surface corrosion. The edges of slabs are rough and stain of water- and water-soluble salts can be seen.

The most damage element is lateral beam. This beam was hit by truck. (Fig .VI-81 and IV-82). Because of very strong impact, down longitudinal bars were deformed and twisted and lost adhesion with concrete, several stirrups were broken and thin concrete cover was cracked and fallen down.

The depth of concrete cover in exterior walls was between 2cm and 3cm and visible RC bars were not seen. Interior walls were built with very small quantity of reinforced. On these concrete structural elements only, poor-quality cover was noticed during visual inspection (Fig. VI-85).

Visual inspection encompassed other bridge elements, like sidewalks, curb stones, catch pits and fences. All mentioned elements have been seriously damaged.

Views of old fence and side walk are illustrated in Figures VI-86 and VI-87.

Figure VI-86. Damaged and corroded old fence and damaged part of side walk

Figure VI-87. View of old fence in detail

3.3. General conclusion for bridge Alsseka

The bridge Alsseka has been old about 50 years when it was inspected for the first time. The main conclusion of the inspection was that the bridge is damaged.

The characteristic defect of arch slabs and lateral beams have been insufficient concrete caver.

The main cause of damage appearance is carbonation. All inspected elements had the problem with carbonation. The depth of carbonization varied from 10mm up to 90mm. The worst results have been obtained in interior walls and ceiling where front of carbonization passed behind the bars.

The second cause of damage appearance is inadequate drainage of water from the deck. This problem caused leakage of water through joints and overflow of water over the edge of cantilever slabs. Consequently, the local corrosion of reinforced bars in deck ceiling and cantilever slabs were caused.

The most damage element is south lateral beam. This beam was hit by truck. Longitudinal bars were deformed and twisted and lost adhesion with concrete, several stirrups were broken and thin concrete cover was cracked and fallen down.

Analyzing results of core compressive strength, it can be seen that the difference between minimum and maximum value for exterior wall is large. This led to the conclusion that built-in concrete has very unequal quality and compressive strength differ from one to another location. But for the ceiling it has been concluded that built-in concrete has good uniformity. The obtained value of concrete compressive strength for both tested elements is small (~22MPa).

The results of concrete compressive strength obtained by Schmidt hammer test show very small dispersion of compressive strength for each analyzed element of structure, but very large dispersion between interior wall and other tested elements (exterior wall and ceiling). The results obtained by Schmidt hammer for interior wall are too small for reinforced concrete.

Comparing compressive strengths of exterior wall and ceiling, obtained by cores and by Schmidt hammer test, it is concluded that there is no significant difference, when carbonation reduction is taken account.

According EN 206-1 the compressive strength classes of concrete given in next table can be used for the control calculation.

Bridge Element	Compressive strength class
Exterior wall	C16/20
Ceiling slab	C16/20

On the bases of results obtained by pull-off method it can be concluded that concrete tensile strength is very low and smaller than minimum require value.

Chloride content in concrete in the bridge structure is not hazardous to imbedded reinforced bars.

Average value of Density of hardened concrete is 2180kg/m^3. This value is smaller than expected value (~2300kg/m^3) and not enough compacted.

Finally, the main conclusion can be drawn:

- Durability of all structural elements is decreased, because of numerous defects that occurred during the construction of this bridge.
- Ceiling was built with very thin concrete cover (~1cm)
- Built in concrete has low compressive strength (C16/20) and low density (~2180kg/m^3)
- The carbonization exists in all concrete elements.
- Bearing capacity of south lateral beam is jeopardized because of damaged of main reinforced bars.
- Bearing capacity of other structural elements is not jeopardized because there are no serious cracking or deformations of RC elements.
- Global stability and stability of each structural element are not threatened and
- Functionality of bridge is partly reduced, because of damages of surface asphalt layers and local instability of delaminated concrete pieces that occurred on the bottom sides of ceiling slab, lateral beams cantilever slabs and edge beams.

4. BAB BIN GHESHIR ROAD BRIDGE

Technical description, assessment, rating and repair of bridges in Tripoli (2009)

4.1. Technical description

Location of bridge: BAB BIN GHESHIR ROAD BRIDGE

Bridge Bab Bin Ghashir is located in the east part of the capital Tripoli, about 2.66km from the sea to the north. It is considered as a major bridge to the capital Tripoli. It connects several main roads leading to the center of the capital. In Figure VI-88, VI-89 and VI-90 are shown situation plan and views of bridge. The coordinates for this bridge are320 52'22.2" N 130 11'45.1"E

Figure VI-88. Bab Bin Gheshir road bridge location on google maps

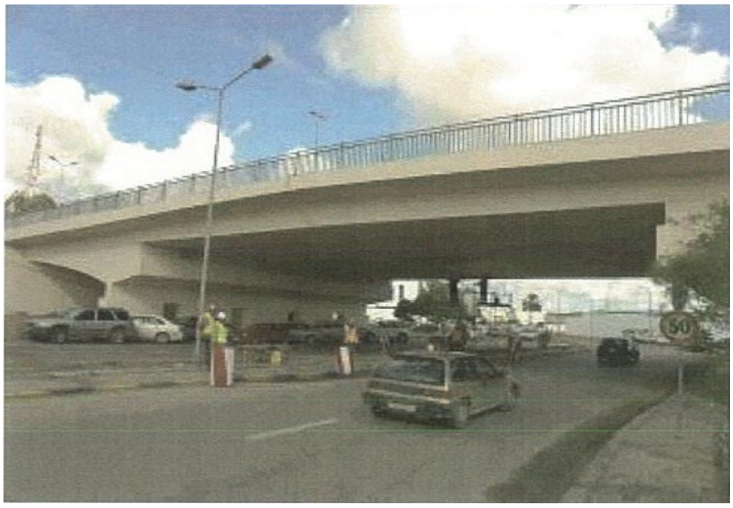


Figure VI-89. General view of Bab Bin Gheshir road bridge

Figure VI-90. Bab Bin Gheshir road bridge view

Type of bridge

Bridge Bab Bin Ghashir is an overpass with three spans supported by reinforced concrete support walls and abutments. This bridge was built in the middle of XX centuries. In Figure VI-91 and VI-92 north and south sides of the bridge are shown.

The characteristic dimensional data of the Bridge are:

- Length: 54.30m
- Width: 25.71m
- Height: 5.50m
- Main span: 21.70m
- Sidewalk: (right side and left side): 1.50m

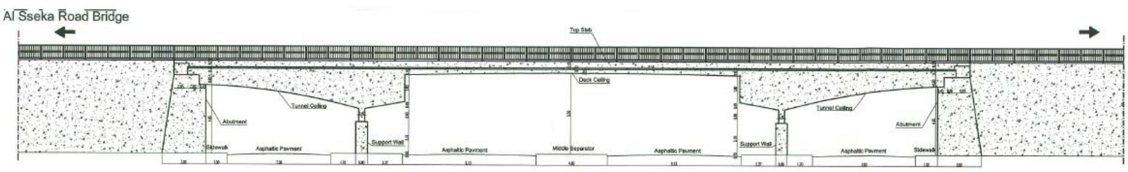


Figure VI-91. Longitudinal view of bridge (North side)

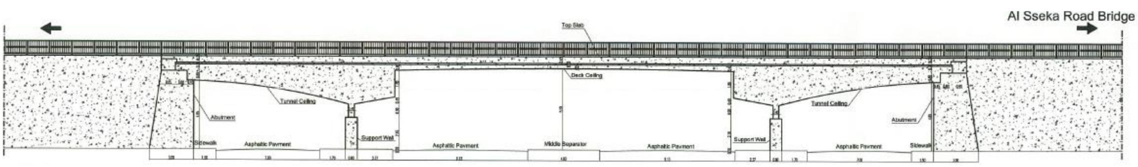


Figure VI-92. Longitudinal view of bridge (South side)

Basic elements of bridge are:

- Interior (abutment) wall
- Exterior (Supporting) wall
- Deck ceiling
- Top slab
- Tunnel ceiling
- Cantilever slab

Disposition of basic bridge elements are signed in Figures VI-93 and VI-94.

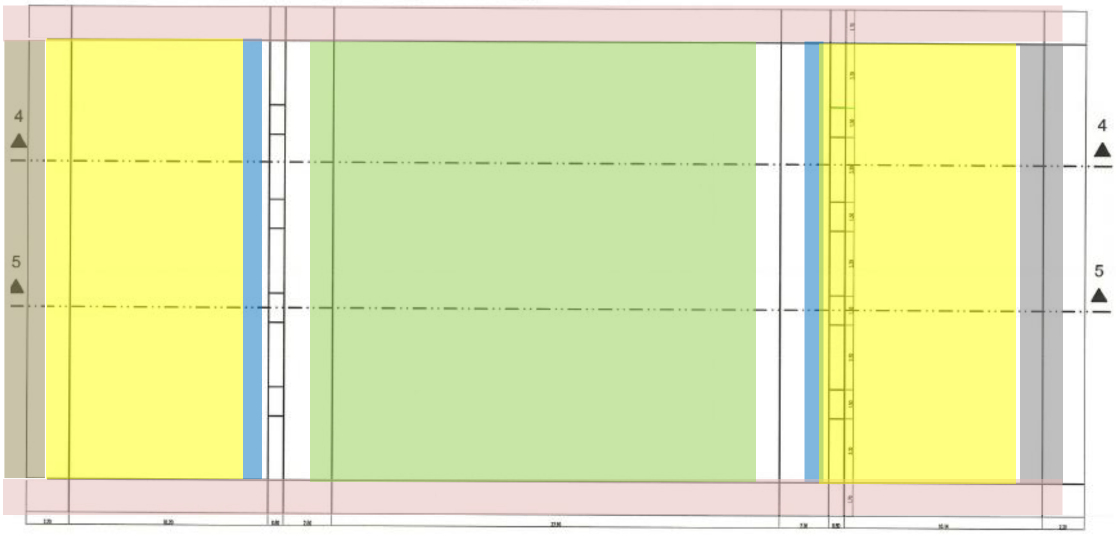


Figure VI-93. Disposition of exterior (blue), interior walls of bridge (grey) deck ceiling slab (green), supporting elements for deck ceiling (white), tunnel ceiling (yellow) and cantilever slabs (brown)

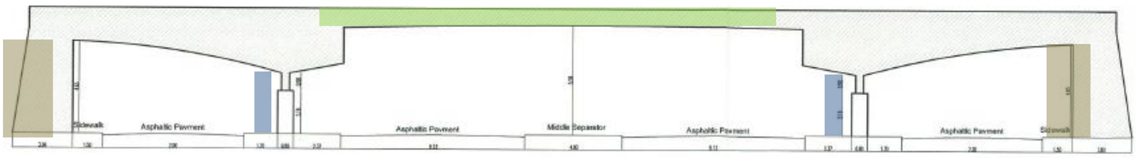


Figure VI-94. Disposition of exterior (blue), interior walls of bridge (grey) and deck ceiling slab

The following text provides a brief description of basic elements of the bridge.

Bridge Bab Bin Ghashir has two interior walls. The basic dimensions of each interior wall are:

Interior wall on east side:

- Length: 23.00m
- Height:4.65m (visible part of total height)

- Depth: 3.0 m

Interior wall on west side

- Length: 23.00m
- High: 4.65m (visible part of total height)
- Depth: 3.0m

Bridge Bab Bin Ghashir has two exterior walls. Both walls have four openings. The basic dimensions of each exterior wall are:

Exterior wall on west side:

- Length: total 23.00m (with four openings)
- Height: variable 3.16m(visible part of total height)
- Depth: 0.8m

Exterior wall on east side

- Length: total 23.00m (with four openings)
- Height: 3.16m (visible part of total height)
- Depth: 0.8 up to the height of 2,16m and 0,40m above part of this wall with height of 0,90m

Figure VI-95 shows the general view of supporting/external walls. Longitudinal view of exterior wall is given in Figure VI-96.

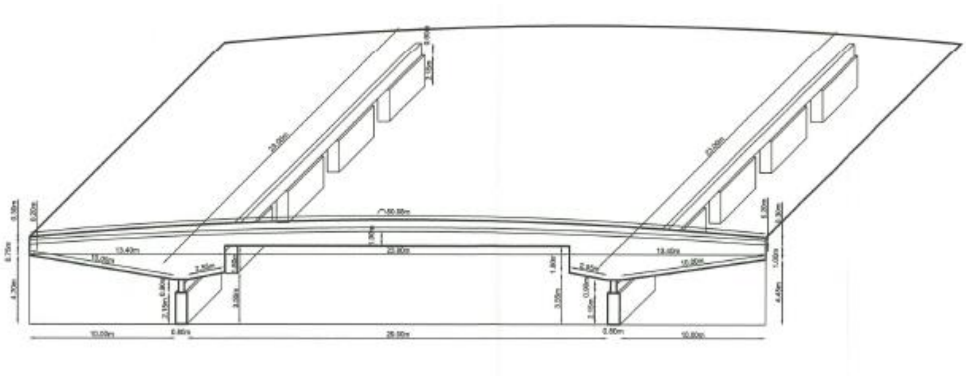


Figure VI-95. General view of supporting walls

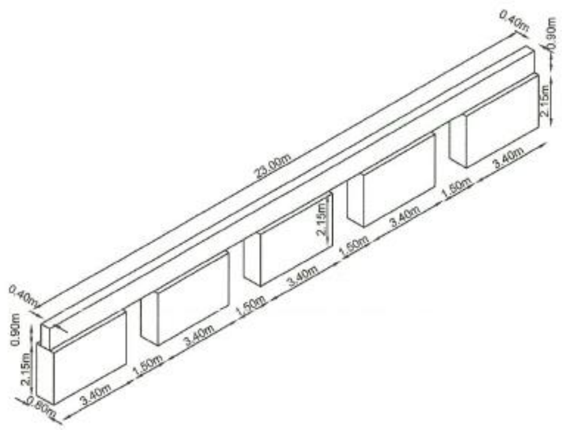


Figure VI-96. Longitudinal view of exterior wall

Upper (horizontal) part of the bridge (deck ceiling slab in the main span) is designed as slab with constant depth which is supported on exterior walls through deep supporting concrete elements. Type of joint between this slab and deep supporting element is unknown. The basic data of this slab are:

- Length: 23.16m
- Width: 21.71m
- Depth: 0.90m

In Figures VI-97 and VI-98 the view of deck ceiling from bottom side are highlighted and characteristic cross sections are labelled.

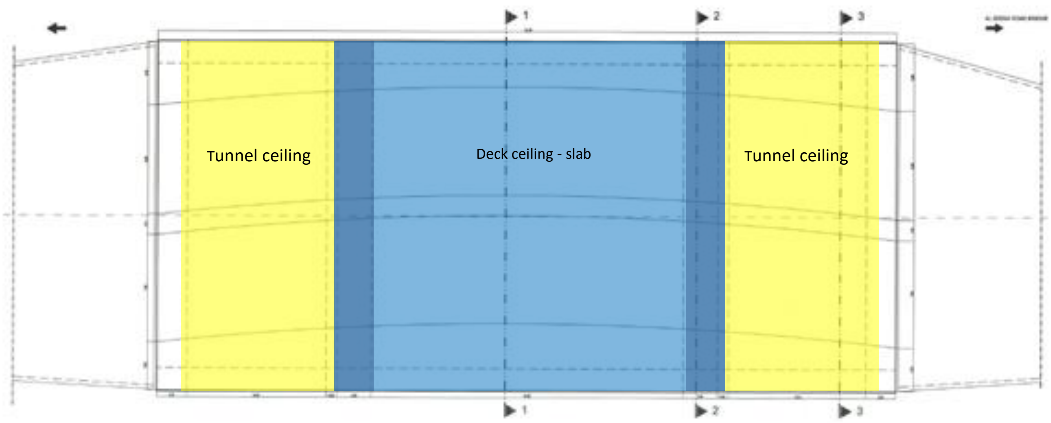


Figure VI-97. The position of characteristic transversal cross sections of the bridge

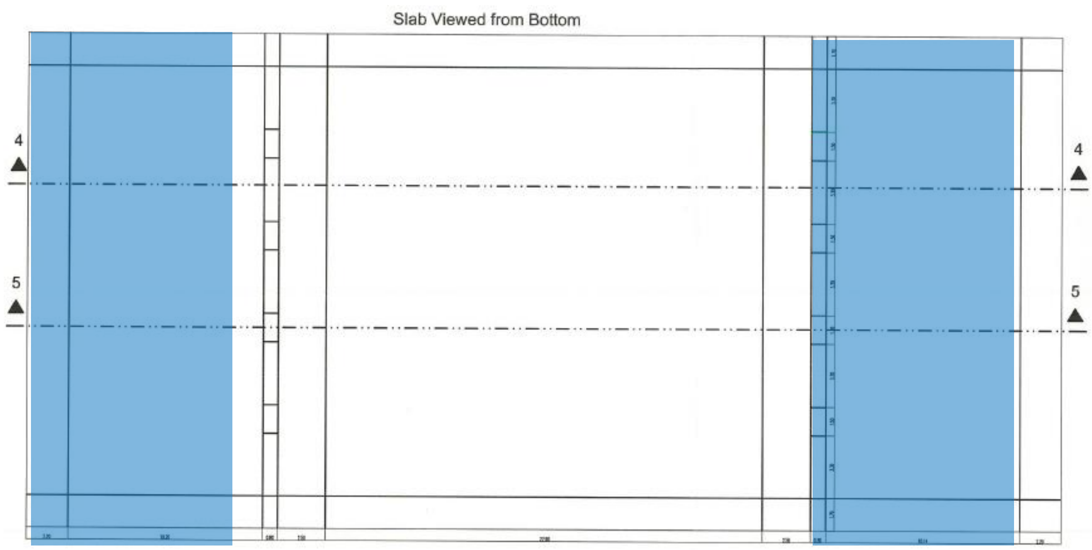


Figure VI-98. Arch slab view from bottom side and location of cantilever arch slabs

The characteristic cross sections are shown in Figures VI-99 – VI-103.

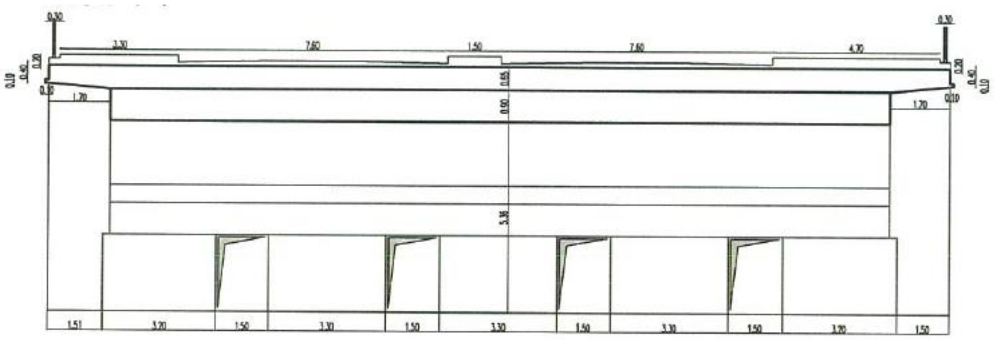


Figure VI-99. Cross-section (1-1) through supporting slab

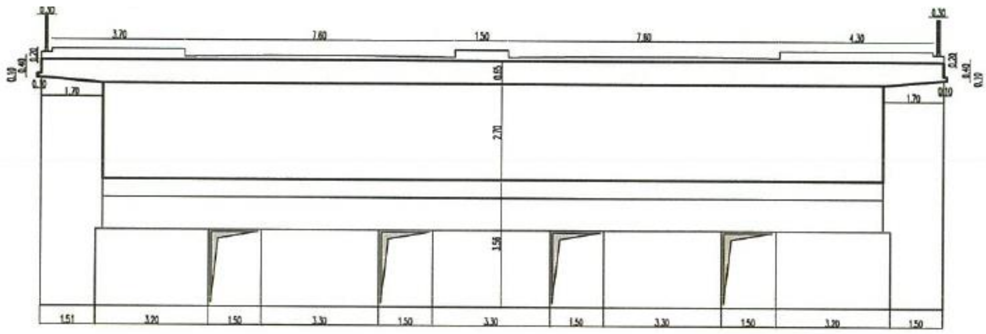


Figure VI-100. Cross-section (2-2) through deep supporting element

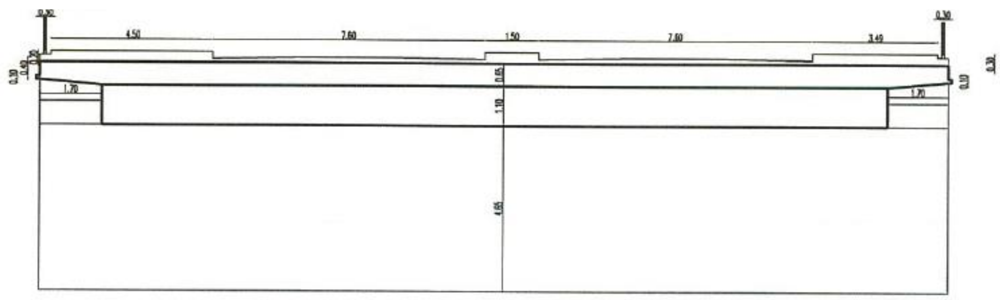


Figure VI-101. Cross-section (3-3) through tunnel ceiling

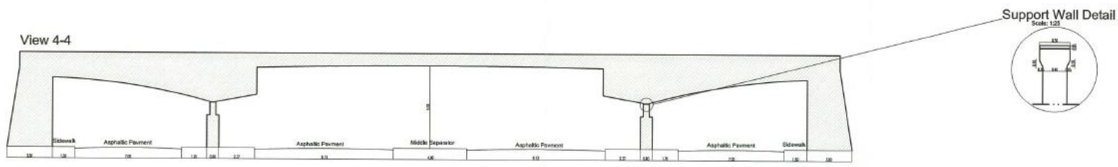


Figure VI-102. Longitudinal cross section of the bridge (section 4-4)

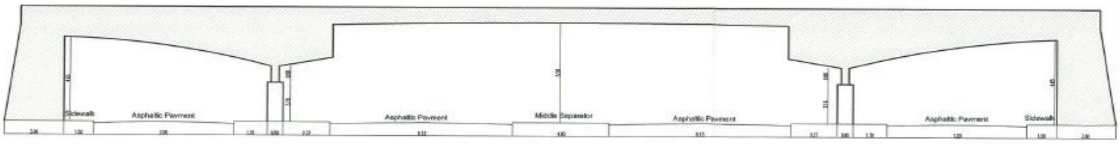


Figure VI-103. Longitudinal cross section of the bridge (section 5-5)

Deep supporting concrete elements have box cross section. Characteristic dimensions of deep supporting concrete elements are:

- Length: 21,71m
- Width: 2,77m
- Height: from 3,34m to 1,78m

Bridge Bab Bin Gheshir bridge has two pedestrian paths that are designed as Cantilever side slabs with edge beam. The characteristic dimensions of side slabs (Figure VI-104) are:

- Length:54.30m
- Width: 1.70m
- Depth:variable from 18cm (free end) up to the 40cm (fixed end)

Dimensions of end beam are:

- Length:54,30m
- Cross section: 0.50mx0.30m

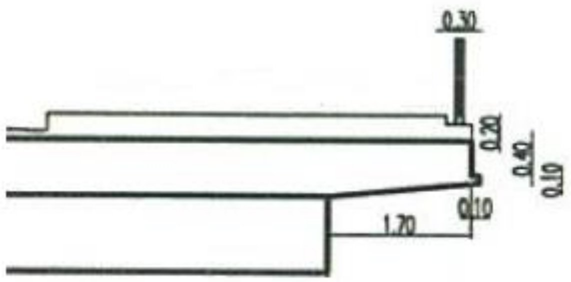


Figure VI-104. Cross section and dimensions of cantilever slab

4.2 Assessment of Bridge Bab Bin Ghashir

In the aim of choose repair materials and techniques for this bridge, next activities were planned:

- In-situ testing of concrete quality and
- Visual inspection of visible parts of bearing elements.

Numbered activities were done in 2009.

4.2.1. Testing of concrete in bearing elements of bridge

The testing of concrete quality encompassed the next activities:

- Measurement of carbonation depth,
- Chloride ion content,
- In-situ testing of concrete by Schmidt Hammer test and.
- In-situ testing of concrete by Pull-off method.

Carbonation depth

The extent of carbonation was assessed by treating with phenolphthalein indicator the fresh exposed surfaces of drilled cores, which were extracted from structure elements for testing concrete compressive strength or for testing carbonation depth.

All data of testing elements, measured depth of carbonation and rebar location are given in Table VI-27.

Table VI-27. Data of testing elements, measured depth of carbonation and rebar location

Reference 1	Carbonization 2		Rebar Depth 3	Depth of carbonization 4	In rebar plan 5	Element 6
	Yes	No	mm	mm		
04.01	x		60	40	N	Exterior wall
04.02	x		60	40	N	Exterior wall
04.03	x		-	40	-	Exterior wall
04.04	x		30	60	Y	Exterior wall
04.05	x		-	50	-	Exterior wall
04.06	x		-	50	-	Interior wall
04.07	x		20	40	Y	Interior wall
04.08	x		30	30	Y	Interior wall
04.09	x		-	30	-	Interior wall
04.10	x		-	40	-	Interior wall
04.11	x		0	50	Y	Deck Ceiling
04.12	x		0	35	Y	Deck Ceiling
04.13	x		0	45	Y	Deck Ceiling
04.14	x		0	60	Y	Deck Ceiling
04.15	x		0	20	Y	Deck Ceiling
04.16	x		40	60	Y	Deep supporting element
04.17	x		10	20	Y	Deep supporting element

04.18	x		40	50	Y	Deep supporting element

After analyzing carbonation results, the next conclusions can be derivate:

- The front of carbonization come up to reinforced bars and in most cases even passed behind the plan of bars.
- The carbonization is most expressed ondeck ceiling slab and deep supporting elements. In the case of these elements front of carbonization always passed behind the bars.
- Exterior and interior walls also have problem with carbonation, but the front of carbonization does not always passed the plan of bars.

Chloride test

The content of ion chloride in concrete is checked by using small pieces of drilled cores which were pulverized and dissolved in acid liquid. The chloride ions react with acid in an electrochemical reaction. An electrode was inserted into the liquid and the change in voltage was measured. On the basis of measured voltage, the instruments showed the chloride content in concrete in %. The obtained results are given in Table VI-28.

Table VI-28. Chloride test result

Elements 1	Reference 2	% Chloride in concrete (Equipment reading) 3			% Chloride ion content by mass of cement 4		
		0-2cm	2-6cm	6-8cm	0-2cm	2-6cm	6-8cm
Exterior wall	03.01	0.0016	0.0011	0.0008	0.0002	0.0001	0.0001
Deck ceiling	03.02	0.0005	0.0002	0.0003	0.00006	0.00003	0.00004
Deck ceiling	03.03	0.0002	0.0003	0.0003	0.00003	0.00004	0.00004

For analyzing given results next criterion was used: The maximum of chloride ion content by mass of cement for reinforced concrete with ordinary carbon steel is 0.40% (class Cl 0.40) (BS 8500).

After comparing obtained results with specified criterion, the next conclusion was made:

- All testing results are smaller than criteria value.

Chloride content in concrete in the bridge structure is not hazardous to imbedded reinforced bars.

Core test

For testing concrete compressive strength, the core tests were done. Cores were extracted from two different locations. In order to determine differences between surface and inner concrete quality, cores were taken out from whole depth of elements. The chosen locations for taking out cores were:

- Exterior walls –three or four cores,
- Beams - two cores.

In the laboratory extracted cores were splitting in the next way:

- In three parts from exterior walls and
- In one part for beams.

Then, all obtained cores were visually inspected and prepared for testing compressive strength by cupping. Testing procedure for compressive strength is described in standard BS 1881: Part 120:1983. All obtained results of estimate in-situ compressive strength are given in table 6.29, and they represent cube compressive strength. For changing cylinder compressive strength to cube compressive strength, the factor of correction was used. This factor depends of dimensions of specimens and of direction of drilling.

In aim to make conclusion of concrete quality, the average value and the range of estimated in-situ cube compressive strength are calculated and shone in the same table VI-29.

Table VI-29. Compressive strength test result

Compressive strength of concrete cores in Bab Bin Gheshir bridge (MPa)- cube values				
	Element 1	Cube result 2	fck, average 3	Range of fckn 4
	Exterior walls	19	21.86	11-24
		22		
		12		
		15		
		14		
		11		
		18		
		24		
		10		
		23		
	Lateral Beams (Part of deck ceiling slab)	19	23.00	11-48
		21		
		11		
		48		
		33		

Analyzing those results it can be seen that the difference between minimum and maximum value for both tested element is large and amounts up to 37MPa. The compressive strength also varies by depth for the same location for four of six extraction places. These led to the conclusion that built-in concrete has very unequal quality and compressive strength differ from one to another location.

The obtained value of concrete compressive strength for both tested elements is small 20MPa).

Schmidt hammer test

For getting more information of built-in concrete quality the Schmidt hammer test, as a nondestructive- surface hardness method, is chosen. Data about tested elements and number of measure places are given in table VI-30.

Table VI-30. Tested elements and number of measuring points

Element	Number of measuring point	Total number of measuring point per element	Total number of measuring point
Exterior wall	10	10	30
Interior wall	10	10	
Deck ceiling	10	10	

On each test location 10 rebound readings were done. Prior to test the surface of concrete was smoothed by carbonudum stone and cleaned. Rebound number was calculated by using next rule: Each result from one test location is valid if it is in range of ±7 points of average value. For each reading the single compressive strength was calculated by using calibration curves and finally the average and standard deviation are calculated too. The calculate values of compressive strengths and standard deviations are given in table VI-31.

Table VI-31. Schmidt hammer test result

Element 1	Wmed (MPa) 2	σ 3
Exterior wall	24.23	1.88
	18.69	1.16
	20.40	1.33
	17.17	1.99
	15.89	1.38
	15.89	1.38
	18.77	0.81
	18.87	1.80
	14.46	1.29
	16.61	1.58

Interior wall	17.95	1.43
	17.45	1.43
	20.26	1.35
	18.93	0.93
	23.22	1.06
	19.23	0.82
	20.35	1.57
	24.32	2.05
	39.04	1.47
	27.23	2.25
Deck ceiling	23.08	1.58
	21.85	2.11
	25.30	1.30
	19.74	1.38
	27.19	0.91
	22.36	1.58
	35.40	1.49
	29.66	2.49
	32.62	1.91
	32.22	2.28

Discussion and Conclusion

In order to make some conclusion of concrete compressive strength, obtained by Schmidt hammer test, the individual results were grouped and mean value of compressive strength and standard deviation were calculated. The obtained data are shown in Table VI-32.

Table VI-32. Schmidt hammer test result – analyze

Element 1	Wmed (MPa) 2	σ (MPa) 3	Carbonization test
Exterior wall	18.098	±1,46	Y
Interior wall	22.798	±1,436	Y
Deck Ceiling	26.942	±1,703	Y

The results given in previous table show a very small dispersion of compressive strength for each analyzed element of structure, medium dispersion between exterior and interior walls and large dispersion between exterior wall and deck ceiling.

Some results given in Table VI-32 can be compared with results of compressive strength obtained by testing cores, Table VI-29. Comparing compressive strengths for exterior wall it is concluded that there is no significant difference, when carbonation reduction is not taken account. The similar conclusion can be made for deck ceiling.

It is well known that the carbonation makes concrete surface layer to be harder. In extreme cases the overestimate of compressive strength from this cause may be up to 50%. On the base of carbonation test results the majority of tested elements are affected with process of carbonation and, according to previous comment, show the higher values of compressive strength from the real value. Therefore, the real estimated compressive strengths of tested elements are smaller from values given in table VI-32 for deck ceiling.

The coefficient of correction is calculated by average of compressive strength obtained by core for exterior wall and deck ceiling and average of compressive strength obtained by Schmidt hammer for the same elements:

$f_{ck,av}$=20MPa (average compressive strength obtained by core)

$f_{ch,av}$=22.61MPa(average compressive strength obtained by Schmidt hammer)

$F_{c,\ comp}$=20/22.61=0.8846

In table VI-32a the average results of compressive strength before and after correction has been shown.

Table VI-32a. Correction of Schmidt hammer compressive strength

Element	compressive strength before correction (MPa)	compressive strength after correction (MPa)
Exterior wall	18.098	16.01
Interior wall	22.798	20.18
Deck Ceiling	26.942	23.83

After comparison of given results, the next conclusions could be made:

- The difference between results of compressive strength of concrete built in deck ceiling obtained by core test and by Schmidt hammer test is not significant. The compressive strength of concrete built in deck ceiling slab corresponds to the class of concrete C16/20.
- The results obtained by Schmidt hammer for exterior wall are too small for reinforced concrete but referent values is compressive strength obtained by core test, thus it can be concluded that concrete built in exterior wall corresponds to the class of concrete C16/20.

Pull off test

For measuring in-situ concrete tensile strength the pull-off method is used. The procedure is described in BS 1881: Part 207. The steel disks and epoxy resin glue were used. The tests were conducted in three places. Obtained results are given in table VI-33.

Table VI-33. Pull off test result

Reference 1	Element 2	W med (MPa) 3	σt (MPa) 4	Failure mode (%) 5		
				concrete	Surface concrete	Epoxy glue
03.01/03.05	Exterior wall	0.292	0.22	7%	57%	36%
03.06/03.10	Interior wall	0.846	0.25	0%	76%	24%
03.11/03.15	Deck ceiling	1.305	0.11	10%	80%	10%

On the bases of given results, it can be seen that all values of tensile strength are smaller than minimum require value and that build-in concrete has very bad quality.

Density

Calculation of density of hardened concrete is very good method for checking the quality of built-in concrete. For calculation of the concrete density, the mass of extracted cores is usually used. Obtained results are given in table VI-34.

On the bases of given results, it can be seen that all values of densities are close to expected value (~2300kg/m^3). It is supposed that the concrete was enough compacted.

Table VI-34. Density test result analyse

Element of structure	Density, kg/m^3	Average for each measuring place kg/m^3	Average for element of structure, kg/m^3
Exterior wall	2287.25	2294	2225
	2319.16		
	2276.62		
	2199.65	2213	
	2214.17		
	2223.92		
	2141.82	2168	
	2189.78		
	2173.58		
Beams	2222.98	2242	2274
	2245.28		
	2258.24		
	2316.53	2305	
	2292.85		

4.2.2. Visual inspection of Bridge Bab Bin Ghashir

First visual inspection of all visible bearing elements or part of bearing elements of bridge were done in 2009.

Damages

During the visual inspection it has been noticed that all visible surfaces of structural elements were plastered by thin layer of ordinary cement mortar and covered by paint.

During the visual inspection several damages were registered.

Characteristic damages are:

- Corrosion of reinforced
- Damage of concrete due to reinforced corrosion.
    - Falling down of cover.
    - Cracking of cover
    - Separation and falling down of plaster layer (spalling)
- White stains on concrete surface (water soluble salts)
- Dark stains on concrete surface (water overflow)

Main causes of damage appearance are:

- Non adequate water drainage system
- Poor quality of material
- Carbonization
- Weathering

Figures VI-105- VI-109 show characteristic damages of RC elements.

Figure VI-105. Damaged plaster and concrete next to the interior wall

Figure VI-106. View of damaged cantilever from top the bridge

Figure VI-107. Dark traces of leakage water on lateral beam

Figure VI-108. View of deck ceiling and exterior wall

Figure VI-109. Spalling of plaster on deep supporting element

Figure VI-110. Traces of leakage water on lateral beam and cantilever slab, corrosion of rebars, spalling of plaster, pilling off of paint

After visual inspection of visible parts of elements of this bridge it was concluded that no serious damages were seen. Most noticed damages had local appearances (spalling of plaster (Fig. VI-105, VI-106, VI-108 and VI-109), surface spalling of concrete (Fig. VI-105), corrosion of rebars (Fig. VI-109) except of traces of leakage water on lateral beam and cantilever slab. They were visible on almost whole lateral surfaces of these elements (Fig. VI-107 and VI-109). The most damaged elements are cantilever slabs with edge beams. The characteristic damage is falling down of ordinary plaster layer (Fig. VI-106). The edges of slabs are rough and stain of water- and water-soluble salts can be seen.

The next most damage elements are lateral beams. These beams had problem with leakage of water through horizontal cold joints (Fig. VI-107 and VI-109). Described leakage of water caused local surface corrosion of reinforced bars and also local spalling of plaster layer.

Other concrete structural elements (exterior walls, interior walls and underpass (tunnel) ceilings) have minor local damage.

Visual inspection encompassed other bridge elements, like sidewalks, curb stones, catch pits and fences. All mentioned elements have been seriously damaged.

During removal of old plaster layer and carbonated concrete cover it was noticed, that concrete had very bad adhesion with rebars (VI-110), so they decided to remove all “weak” concrete. They also discovered the almost all RC elements had a small concrete cover.

Figure VI-111. Bad adhesion between reinforced bars and concrete (cantilever slab)

4.3. General conclusion for bridge Bab Bin Ghashir

Finally, the main conclusion can be drawn:

- Durability of all structural elements is decreased, because of numerous defects that occurred during the construction of this bridge.
- Ceiling was built with very thin concrete cover (~1cm)
- Built in concrete has low compressive strength (C16/20) and wicked tensile strength (~0,20MPa)
- Densityof hardened concrete is regular (~2250kg/m^3)
- The carbonization exists in all concrete elements and is accented on deck ceiling slab and deep supporting elements
- Bearing capacity of all structural elements is not jeopardized because there are no serious cracking or deformations of RC elements.
- Global stability and stability of each structural element are not threatened and
- Functionality of bridge is partly reduced, because of damages of surface asphalt layers and local instability of delaminated concrete pieces that occurred on the bottom sides of ceiling slab, lateral beams cantilever slabs and edge beams.

5. AL SREEM ROAD BRIDGE

Technical description, assessment, rating and repair of bridges in Tripoli (2009)

5.1 Technical description

Location of bridge: AL SREEM ROAD BRIDGE

Bridge Al Sreem Road is located in the south part of the capital Tripoli, about 2.21km from the sea to the north. It connects several main roads leading to the center of the capital and road to the airport. In Figure VI-112, VI-113 and VI-114 the situation plans and views of bridge are shown. The coordinates for this bridge are:

320 53'03.3" N 130 10'29.5"E

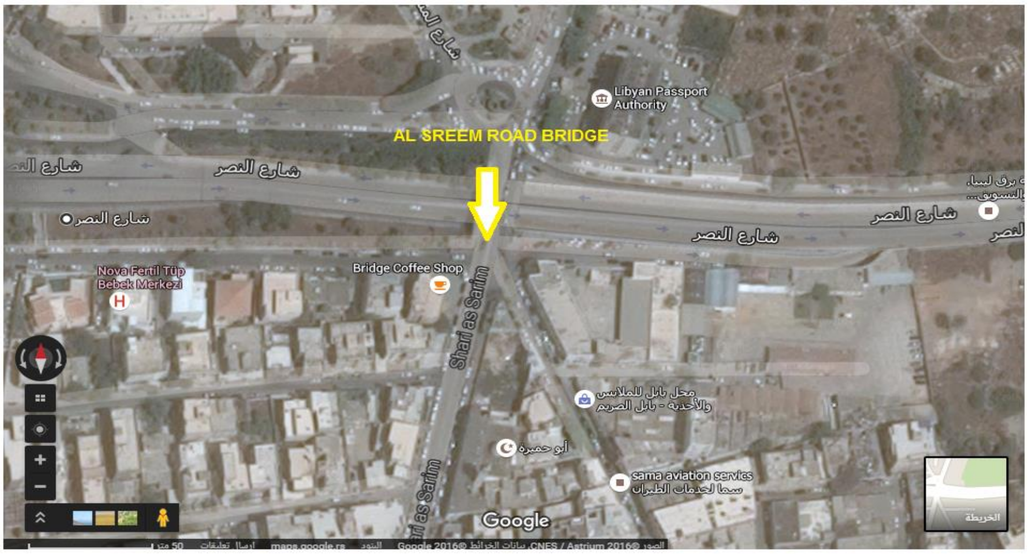


Figure VI-112. ALSreem Road bridge location on google maps

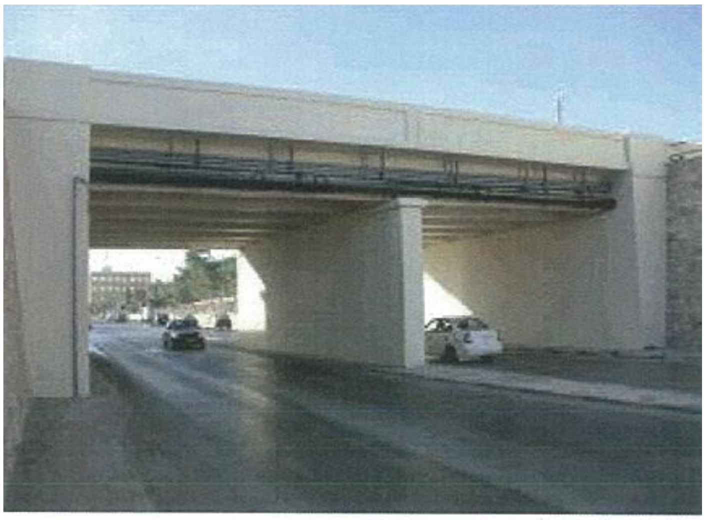


Figure VI-113. ALSreem Road bridge, south side

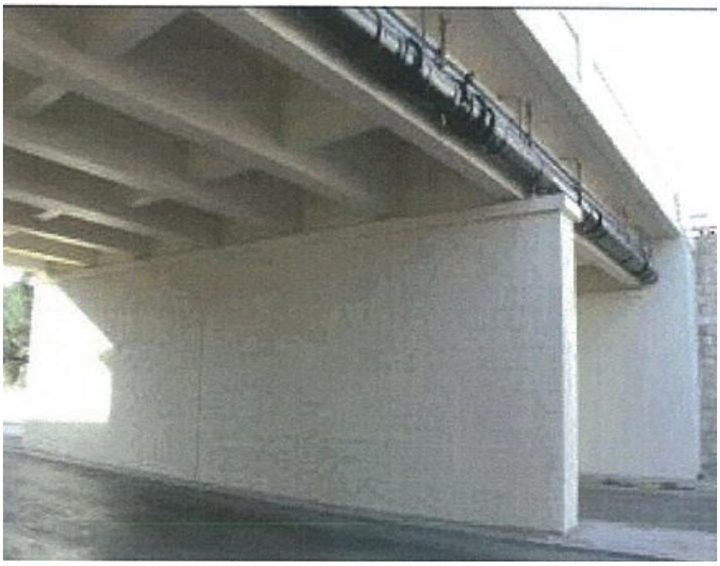


Figure VI-114. AL Sreem Road bridge: view of ceiling and exterior wall

Type of bridge

Bridge AlSreem Road is two span beam bridges. It is also overpass with RC bridge superstructure which is supported by masonry bridge substructure. Masonry bridge substructure consists of one stone support wall and two stone abutments. All masonry elements were covered by plastering. RC bridge superstructure consists of two decks which are supporting on longitudinal and transversal beams.

This bridge was built in the middle of XX centuries.

In Figure VI-115 and VI-116 north and south sides of the bridge are shown. The plan of the bridge is given in Figure VI-117.

The characteristic dimensional data of the bridge are:

- Total Length: 21.10m
- Span length: 2x9.675m (two spans)
- Total Width: 16.80m
- Height: 6.81m
- Sidewalk (right side and left side): 1.00m

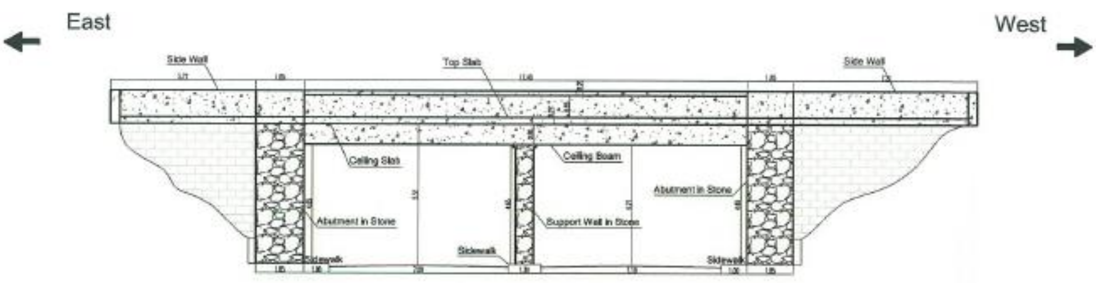


Figure VI-115. Longitudinal cross section of bridge (north side)

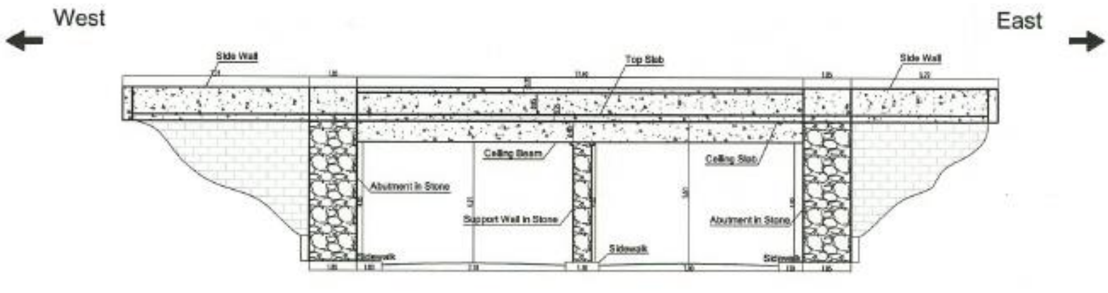


Figure VI-116. Longitudinal cross section of bridge (south side)

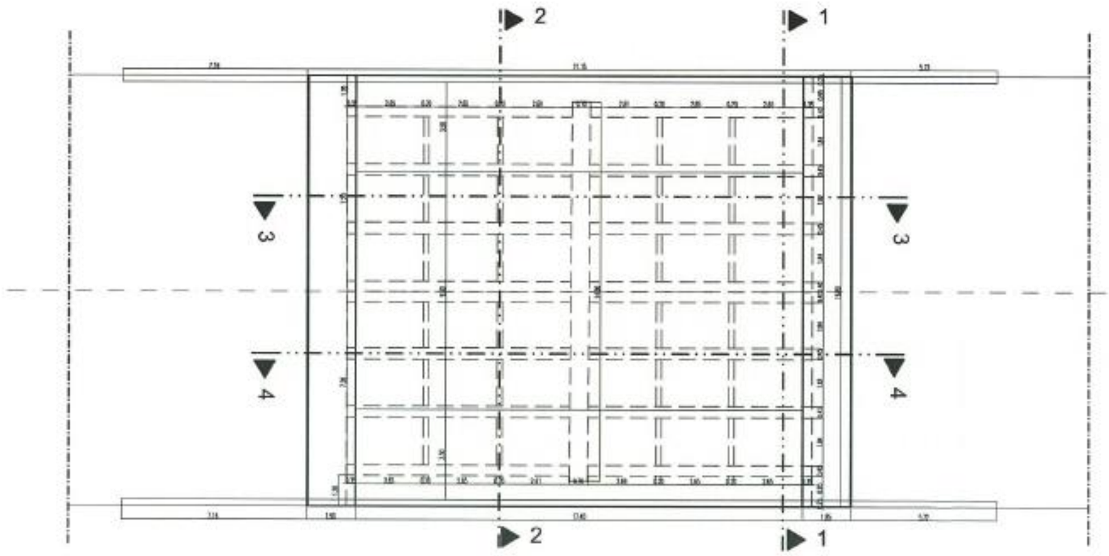


Figure VI-117. Plan of the bridge – upper side

Basic elements of bridge are:

- Interior wall (Masonry abutmentsmade of stone)
- Exterior wall (Masonry support wallsmade of stone)
- Deck slab (reinforced concrete)
- cantilever slab (reinforced concrete)
- Longitudinal and transversal supporting (ceiling) beams.(reinforced concrete)

Disposition of basic bridge elements are signed in Figures VI-118 and VI-119.

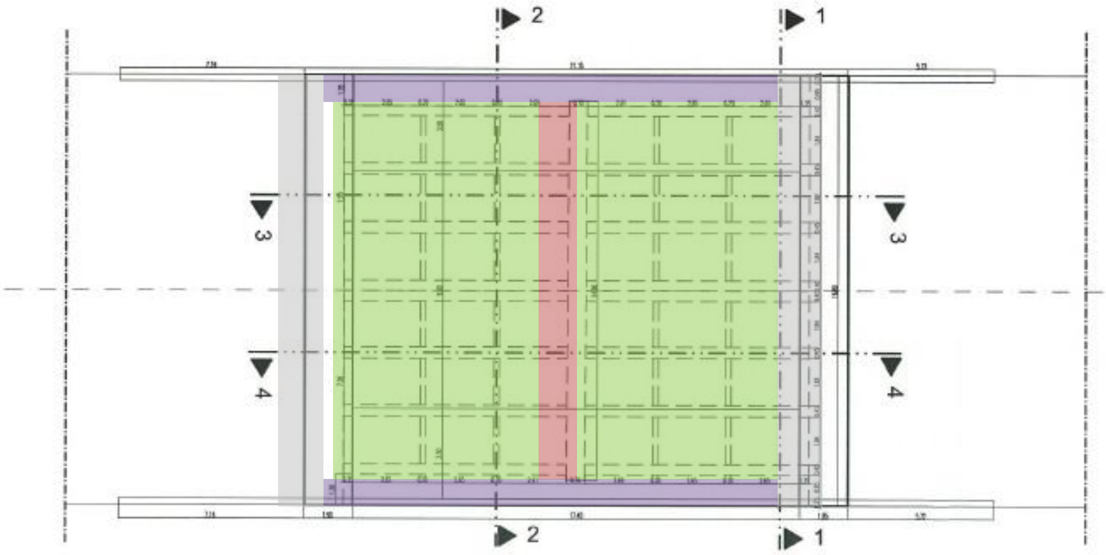


Figure VI-118. Disposition of deck slabs (green), cantilever side slabs (yellow), interior walls (grey) and exterior walls (brawn) in plane of the bridge (bottom and plan side)

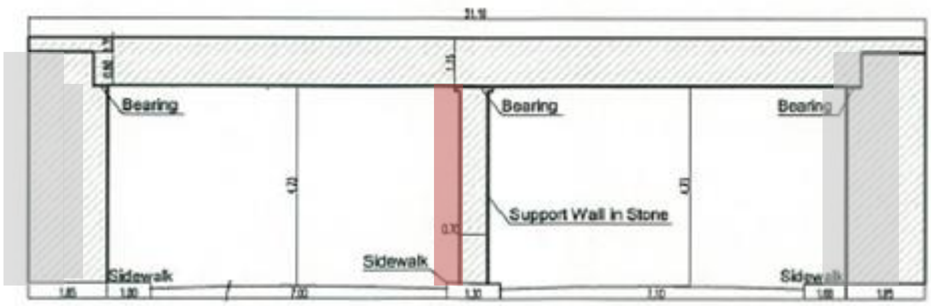


Figure VI-119. Disposition of exterior (brown) and interior walls of bridge (gray) (section 4-4)

The following text provides a brief description of basic elements of the bridge.

Bridge Al Sreem Road has two interior (abutment) walls which were built as stone masonry structures. The surfaces of these walls were plastered with ordinary cement mortar and painted. The basic dimensions of each interior wall are:

Interior wall on east and west side:

- Length: 16.80m
- Height: 5.45m (visible part of total height)
- Depth: 1.85m

Longitudinal view of interior wall given in Figure VI-120.

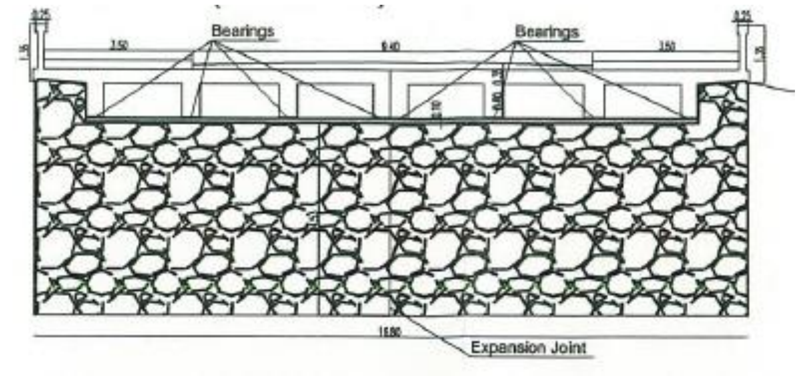


Figure VI-120. Longitudinal view of interior wall and cross section of deck slabs (Section1-1)

Bridge Alsreem Road has two exterior (supporting) walls which were built as stone masonry structures. The surfaces of these walls were painted with ordinary colour. The basic dimensions of exterior wall are:

Exterior wall (one exterior wall in middle)

- Length: total 8.40m
- High: 4.65m (visible part of total height)
- Depth: 0.70m

Longitudinal view of exterior wall given in Figure VI-121.

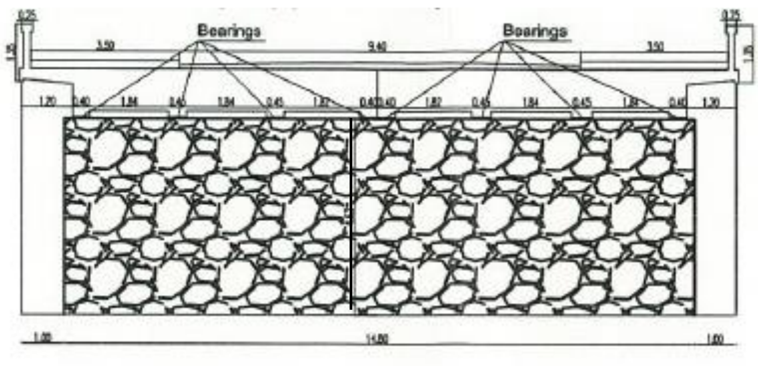


Figure VI-121. Longitudinal view of exterior wall sand cross section of deck slabs (section 2-2)

Superstructure (horizontal part) of the bridge consists of two deck slabs and ceiling (supporting) beams in longitudinal and transverse direction (Figure VI-122). The basic data of deck slabs are:

- Length :17,80m

- Width: 7,20m
- Thickness: 0,35m

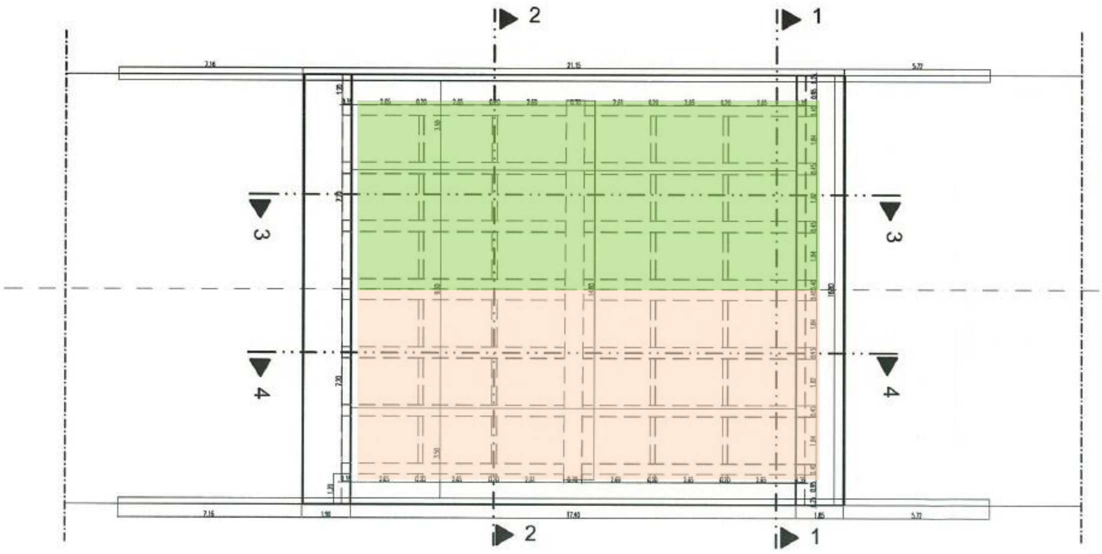


Figure VI-122 Position of deck slabs, longitudinal and transverse ceiling beams and position of characteristic cross sections

Longitudinal beams have T cross section (Fig. VI-123 and VI-126). The basic data of longitudinal beam slabs are:

- Length: 17,80m
- Width of rib: 0,45m
- Height: 1,15m

There are two types of transverse beams primary and secondary (Fig. VI-124 and VI-125). Both types of beams have T cross section. Every part of superstructure has only one primary beam, which located in the middle of bridge span. The basic data of primary transverse beam are:

- Length: 7,20m
- Width of rib: 0,70m
- Height: 1,15m

Every part of superstructure has four secondary beams, which located in the sixth of bridge span. The basic data of secondary transverse beam are:

- Length: 7,20m
- Width of rib: 0,20m
- Height: 1,00m

This bridge has two pedestrian paths that are designed as Cantilever side slabs (Figure VI-126). The characteristic dimensions of side slabs are:

- Length: 21,10m
- Width: 1.20m
- Thickness: variable from 20cm (free end) up to the 35cm (fixed end)

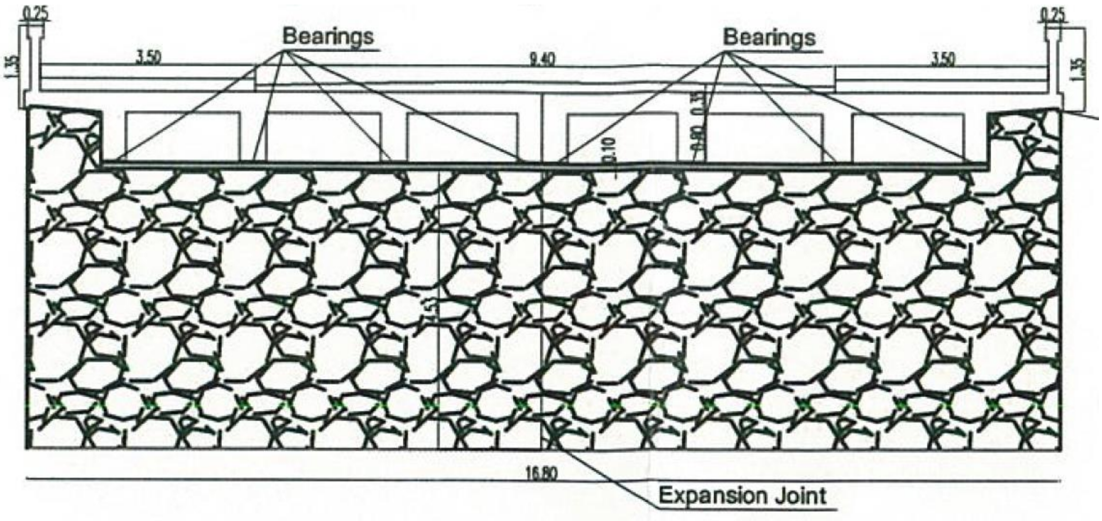


Figure VI-123. Cross section 1-1

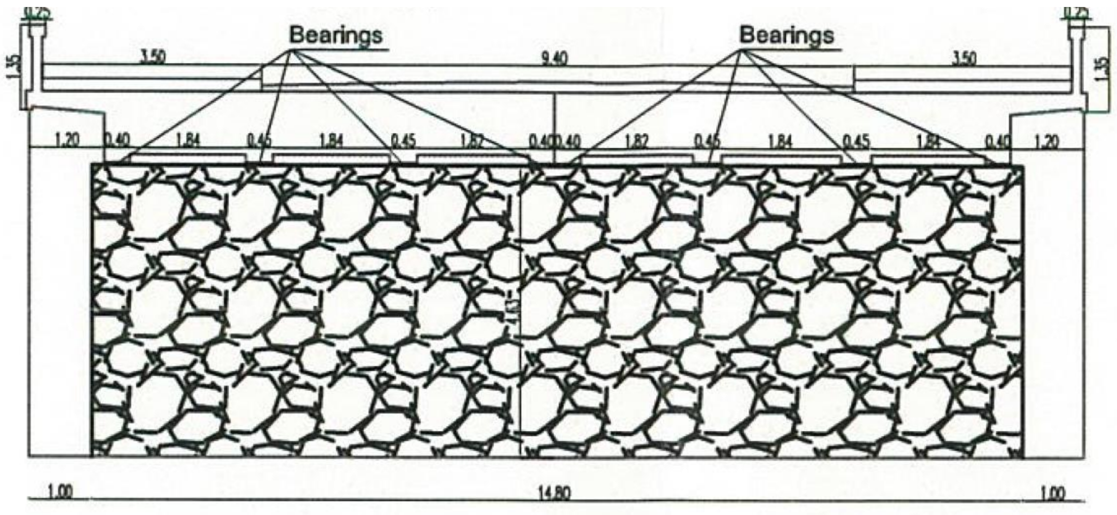


Figure VI-124. Cross section 2-2

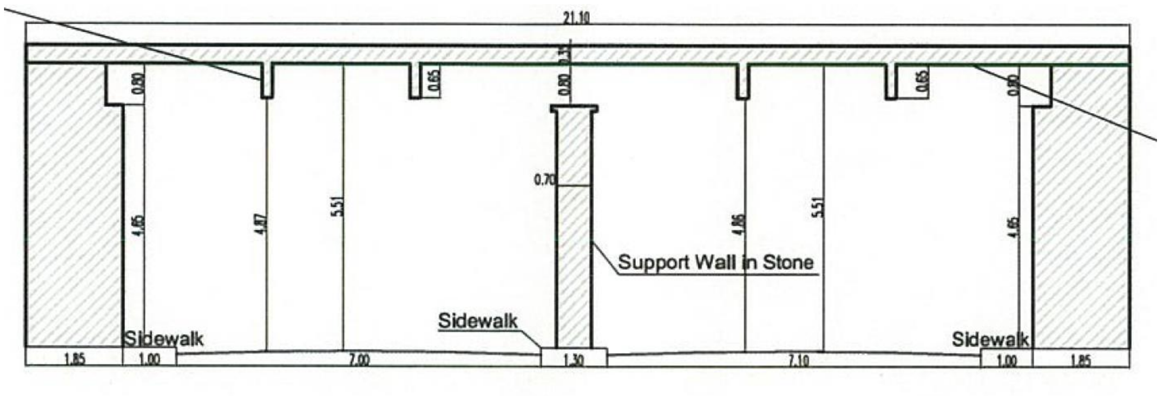


Figure VI-125. Cross section 3-3

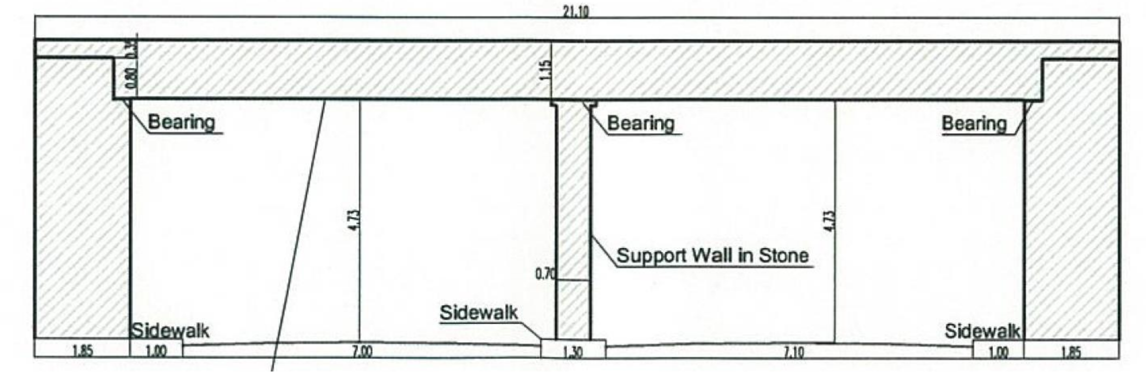


Figure VI-126. Cross section 4-4

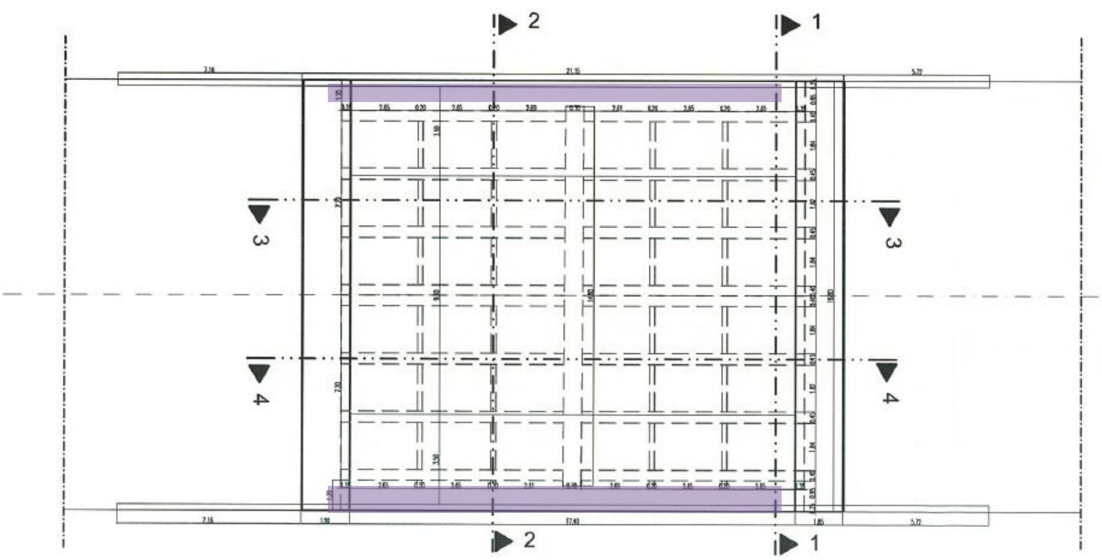


Figure VI-127. The location of cantilever slabs in plan of bridge

5.2. Assessment of Bridge Al Sreem Road

In the aim of choose repair materials and techniques for this bridge, next activities were planned:

- In-situtesting of concrete quality and
- Visual inspection of visible parts of bearing elements.

Numbered activities were done in 2009.

5.2.1 Testing of concrete in bearing elements of bridge

The testing of concrete quality encompassed the next activities:

- Measurement of carbonation depth,
- Chloride ion content,
- In-situ testing of concrete by taking of cores,
- In-situ testing of concrete by Pull-off method.
- In-situ testing of concrete by Schmidt Hammer test and

Carbonation depth

The extent of carbonation was assessed by treating with phenolphthalein indicator the fresh exposed surfaces of drilled cores, which were extracted from structure elements for testing concrete compressive strength or for testing carbonation depth.

All data of testing elements, measured depth of carbonation and rebar location are given in Table VI-35.

Table VI-35. Data of testing elements, measured depth of carbonation and rebar location

Reference 1	Carbonization 2		Rebar Depth 3	Depth of carbonization 4	In rebar plan 5	Element 6
	Yes	No	mm	mm		
08.01	x		30	10	N	Ceiling Beams
08.02	x		30	20	N	Ceiling Beams
08.03	x		20	20	Y	Ceiling Beams
08.04	x		30	30	Y	Ceiling Beams
08.05	x		20	30	Y	Ceiling Beams
08.06	x		20	20	Y	Ceiling Beams
08.07	x		20	20	Y	Ceiling Beams
08.08	x		30	10	N	Ceiling Beams
08.09	x		20	10	N	Ceiling Beams
08.10	x		30	10	N	Ceiling Beams
08.11	x		20	10	N	Ceiling Beams
08.12	x		10	30	Y	Ceiling Beams

08.13	x		10	10	Y	Ceiling Beams
08.14	x		10	10	Y	Ceiling Beams
08.15	x		20	20	Y	Ceiling Beams
08.16	x		20	20	Y	Ceiling Beams
08.17	x		20	10	N	Slab ceiling
08.18	x		20	10	N	Slab ceiling

After analyzing carbonation results, the next conclusions can be derivate:

- The minimum depth of carbonization is 10mm.
- The maximum depth of carbonization is 30mm
- The mean values are:17.5mm for ceiling beam, 10mm for slab ceiling
- In case of ceiling beams the front of carbonization came up to reinforced bars and even passed behind the bars at 10 of 16 measurement points.
- The front of carbonation did not come up to reinforced bars in a case of ceiling slab
- The carbonization is most expressed in ceiling beams.

Chloride test

The content of ion chloride in concrete is checked by using small pieces of drilled cores which were pulverized and dissolved in acid liquid. The chloride ions react with acid in an electrochemical reaction. An electrode was inserted into the liquid and the change in voltage was measured. On the basis of measured voltage, the instruments showed the chloride content in concrete in %. The obtained results are given in Table VI-36.

Table VI-36. Chloride test result

Elements 1	Reference 2	% Chloride in concrete (Equipment reading) 3			% Chloride ion content by mass of cement 4		
		0-2cm	2-6cm	6-8cm	0-2cm	2-6cm	6-8cm
Ceiling beam	08.01	0.0004	0.0004	0.0002	0.0001	0.0001	0.0001
Ceiling beam	08.02	0.0048	0.0006	0.0009	0.0006	0.0001	0.0001
Ceiling beam	08.03	0.0018	0.0012	0.0006	0.0002	0.0002	0.0001

For analyzing given results next criterion was used: The maximum of chloride ion content by mass of cement for reinforced concrete with ordinary carbon steel is 0.40% (class Cl 0.40) (BS 8500).

After comparing obtained results with specified criterion, the next conclusion was made:

- All testing results are smaller than criteria value.
- Chloride content in concrete in the bridge structure is not hazardous to imbedded reinforced bars.

Core test

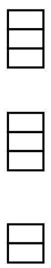
For testing concrete compressive strength, the core tests were done. Cores were extracted from one location. In order to determine differences between surface and inner concrete quality, cores were taken out from whole depth of element. Three cores were taken out only from ceiling beams. In the laboratory extracted cores were cut in the next way:

- Two cores were cut in three parts and
- One core was cut in two parts.

Then, all obtained cores were visually inspected and prepared for testing compressive strength by cupping. Testing procedure for compressive strength is described in standard BS 1881: Part 120:1983. All obtained results of estimate in-situ compressive strength are given in table VI-37, and they represent cube compressive strength. For changing cylinder compressive strength to cube compressive strength, the factor of correction was used. This factor depends of dimensions of specimens and of direction of drilling.

In aim to make conclusion of concrete quality, the average value and the range of estimated in-situ cube compressive strength are calculated and shone in the same table VI-37.

Table VI-37. Compressive strength test result

Compressive strength of concrete cores in AL sreem road bridge (MPa)- cube values				
	Element 1	Cube result 2	fck, average 3	Range of fckn 4
	Beams	21	25	15-39
		20		
		19		
		15		
		28		
		24		
		39		
		34		

Analyzing those results it can be seen that the difference between minimum and maximum value for beams is large and amounts to 24MPa. This led to the conclusion that built-in concrete has very unequal quality and compressive strength differ from one to another location. The compressive strength varies by depth for the same location for one of three extraction places.

The obtained value of concrete compressive strength of tested element is relatively small (~25MPa).

Pull off test

For measuring in-situ concrete tensile strength the pull-off method is used. The procedure is described in BS 1881: Part 207. The steel disks and epoxy resin glue were used. The test was conducted in three places. Obtained results are given in table VI-38.

Table VI-38. Pull off test result

Reference 1	Element 2	W med (MPa) 3	σ 4	Failure mode (%) 5		
				concrete	Surface concrete	Epoxy glue
08.01	Slab ceiling	0,29		100%	0%	0%
		0,27		100%	0%	0%
		0,42		100%	0%	0%
		0,43		100%	0%	0%
		0,39		50%	50%	0%
Mean values		0,364	0.07	90%	10%	0%
08.06	Ceiling beams	0,46		100%	0%	0%
		0,47		50%	50%	0%
		0,38		50%	50%	0%
		0,28		100%	0%	0%
		0,42		100%	0%	0%
Mean values		0,408	0.06	80%	20%	0%

On the bases of given results, it can be seen that all values of tensile strength are smaller than minimum require value and that build-in concrete has very bad quality.

Schmidt hammer test

For getting more information of built-in concrete quality the Schmidt hammer test, as a nondestructive surface hardness method, is chosen. Data about tested elements and number of measure places are given in table VI-39.

Table VI-39. Tasted elements and number of measuring points

Element	Part of element	Number of measuring point	Total number of measuring point per element	Total number of measuring point
Ceiling beams	Ceiling beams	15	15	30
Slab ceiling	Slab ceiling	15	15	

On each test location 10 rebound reading was done. Prior to test the surface of concrete was smoothed by carbonudum stone and cleaned. Rebound number was calculated by using next rule: Each result from one test location is valid if it is in range of ±7 points of average value. For each reading the single compressive strength was calculated by using calibration curves and finally the average and standard deviation are calculated too. The calculate values of compressive strengths and standard deviations are given in table VI-40.

Table VI-40. Schmidt hammer test result

Element 1	Wmed (MPa) 2	σ 3
Ceiling beams	14.51	0.97
	27.10	3.03
	34.20	1.34
	32.65	1.91
	41.00	4.91
	32.63	1.53
	37.70	1.70
	27.17	1.71
	32.52	3.82
	41.95	1.56
	36.30	2.51
	38.44	2.07
	34.66	1.70
	35.37	3.90
	29.22	1.11
Ceiling slab	30.34	1.70
	29.22	1.26
	25.95	1.94
	23.77	1.48

Discussion and Conclusion

In order to make some conclusion of concrete compressive strength, obtained by Schmidt hammer test, the individual results were grouped and mean value of compressive strength and standard deviation were calculated. The obtained data are shown in Table VI-41.

Table VI-41. Schmidt hammer test result analyse

Element 1	Wmed (MPa) 2	σ (MPa) 3	Carbonization test
Ceiling beams	33.028	±2.251	Y
Slab ceiling	27,32	±3.012	Y

The results given in previous table show a small dispersion of compressive strength for each analyzed element of structure and medium difference between ceiling beams and slab ceiling.

According to this analyze the next conclusion can be derived: the built-in concrete has satisfactory surface uniformity.

Some results given in Table VI-41 can be compared with results of compressive strength obtained by testing cores, Table VI-37.

The coefficient of correction is calculated by average of compressive strength obtained by core for ceiling beams and average of compressive strength obtained by Schmidt hammer for the same elements:

$f_{ck,av}$=25,0 MPa (average compressive strength obtained by core)

$f_{ch,av}$=30,174MPa(average compressive strength obtained by Schmidt hammer)

$f_{c,\ comp}$=25/30,174=0.829

In table VI-41a the average results of compressive strength before and after correction has been shown.

Table VI-41a. Correction of Schmidt hammer compressive strength

Element	compressive strength before correction (MPa)	compressive strength after correction (MPa)
Ceiling beams	33.028	27,36
Slab ceiling	27,32	22,63

After those corrected values, the next conclusion is made:

The compressive strength of concrete built in ceiling beam corresponds to the class of concrete C20/25.

The compressive strength of concrete built in ceiling slab corresponds to the class of concrete C16/20.

Density

Calculation of density of hardened concrete is very good method for checking the quality of built-in concrete. For calculation of the concrete density, the mass of extracted cores is usually used. Obtained results are given in table VI-42.

On the bases of given results, it can be seen that all values of densities are close to expected value (~2300kg/m^3). It is supposed that the concrete was enough compacted.

Table VI-42. Density test result – analyze

Element of structure	Density, kg/m^3	Average for each measuring place kg/m^3	Average for element of structure, kg/m^3
Beams	2247	2242	2273
	2229		
	2250		
	2252	2271	
	2284		
	2276		
	2296	2306	
	2315		

5.2.2. Visual inspection of Bridge ALSREEM ROAD

First visual inspection of all visible bearing elements or part of bearing elements of bridge were done in 2009. Visual inspection covered elements of superstructure that are made of reinforced concrete and elements of substructure which are made of stone.

During the visual inspection it has been noticed that horizontal elements of superstructure were plastered by thin layer of ordinary cement mortar and covered by paint.

Visual inspection of elements of superstructures

Characteristic damages of slab beams are:

- Visible, deformed or broken reinforcing bars
- Corrosion of reinforced
- Cracked and crushed concrete
- Separation and falling down of plaster layer (spalling)
- White stains on concrete surface (water soluble salts)
- Dark stains on concrete surface (water overflow)

- Main causes:
  - Truck hitting
  - Non adequate water drainage system
  - Poor quality of material
  - Exposure to atmospheric conditions

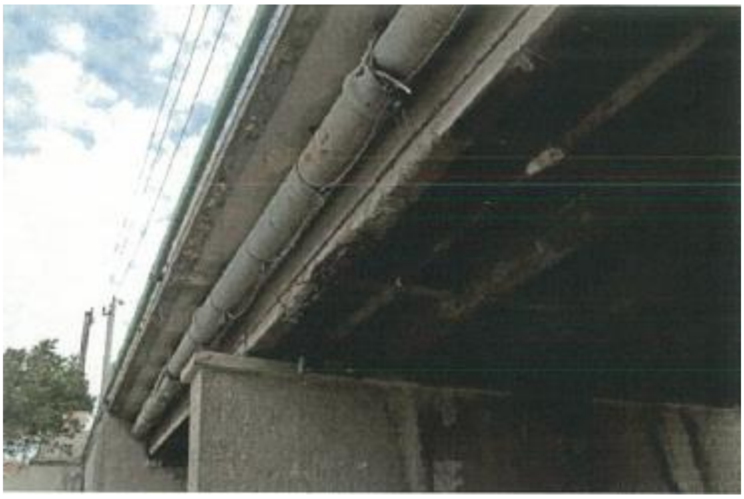


Figure VI-128. View of superstructure of bridge

Figure VI-129. Damaged external longitudinal beam: bared, deformed and twisted rebars, crashed and cracked concrete reinforced in beam: spalling of mortar from cantilever slab

Figure VI-130. Joint between two deck slabs: water leakage, spalling of mortar layer, white and dark stains

The most damaged elements are longitudinal slab beams. The characteristic damages are crashed concrete and deformed, twisted and even broken reinforced rebars (Fig.VI-129, VI-130). Some reinforced bars were bared and then affected by surface corrosion.

The main cause of described damages was hitting by truck. External longitudinal beams of both slab decks are significantly damaged because they were hit by truck several times.

The next characteristic damage is local spalling of mortar layer from down surface of cantilever slabs and slab beams and from masonry elements (exterior and interior walls).

Traces of water leakage and of overflow water could be seen in gap between deck slabs and on down surfaces of cantilever slabs.

Visual inspection encompassed other bridge elements, like sidewalks, curb stones, catch pits and fences. All mentioned elements have been seriously damaged.

General conclusion

The bridge Al Sreem Bridge has been old about 50 years when it was inspected for the first time. The main conclusion of the inspection was that the bridge is damaged.

The main cause of damage is hitting by trucks.

Thus, the superstructure of this bridge is made of reinforced concrete; the carbonization was performed on ceiling beams and ceiling slab only. It was concluded carbonation is characteristic for ceiling beams not for ceiling slab.

In case of ceiling beams the front of carbonization came up to reinforced bars and even passed behind the bars at 10 of 16 measurement points.

In some cases, the front of carbonation even passed behind the bars.

The second cause of damage appearance is inadequate drainage of water from the deck. This problem caused leakage of water through joints and overflow of water over the edge of cantilever slabs. Consequently, the dark and white stains occurred on surfaces of these elements and spalling of mortar also.

Masonry structural elements (abutments and supporting walls) do not have cracks or other types of serious damages. Only a surface damages are registered in the form of mortar spalling and cracking on interior walls (abutments).

Analyzing concrete compressive strength obtained by cores it can be seen that the difference between minimum and maximum value for ceiling beams is large and vary from 15 to 39MPa. This led to the conclusion that built-in concrete has very unequal quality and compressive strength differ from one to another ceiling beam.

The results of concrete compressive strength obtained by Schmidt hammer test show a small dispersion of compressive strength for both analyzed element of superstructure.

Comparing compressive strengths obtained by cores and by Schmidt hammer test it is concluded:

- The Schmidt hammer test and core test were not performed on the same element of bridge, so there are not enough results for comparing.
- There is no significant difference, when carbonation reduction is taken account for compressive strength of concrete built in ceiling beams.
- Negligible higher value has been obtained by Schmidt hammer, when the carbonization is taken in the analysis.

So, the general conclusion can be established that built in concrete has large dispersion in quality from very high (~39MPa) up to very low (~15MPa).

According EN 206-1 the compressive strength classes of concrete given in next table can be used for the control calculation.

Bridge Element	Compressive strength class
Ceiling beams	C20/25
Ceiling slab	C16/20

On the bases of results obtained by pull-off method it can be concluded that concrete tensile strength is very low and smaller than minimum require value.

Chloride content in concrete in the bridge structure is not hazardous to imbedded reinforced bars.

The density of hardened concrete is close to 2300kg/m^3. It is supposed that the concrete built in superstructure was enough compacted.

5.3. General conclusion for bridge ALSREEM ROAD

Finally, the main conclusion can be drawn:

- Durability of all structural elements is decreased, because of carbonation of concrete.
- Bearing capacity of several longitudinal slab beams are jeopardized because the main reinforced rebars are deformed and twisted. Also, the large part of cross section was crushed in the same locations.
- Global stability of bridge is not threatened and
- Functionality of bridge is partly reduced, because of damages of surface asphalt layers and local instability of crushed concrete pieces that occurred on the bottom sides of slab beams.

6. ALSHAAB PORT BRIDGE

Technical description, assessment, rating and repair of bridges in Tripoli (2009)

6.1. Technical description

Location of bridge: Alshaab Port

Alshaab Port Bridge is located in the north part of the capital Tripoli, about 125.77 meters from the sea to the north. It connects several main roads leading to the center of the capital. In Figure VI-131, VI-132 and VI-133 are shown situation plan and views of bridge. The coordinates for this bridge are 320 53' 48.7'' N 130 12' 02.3'' E.

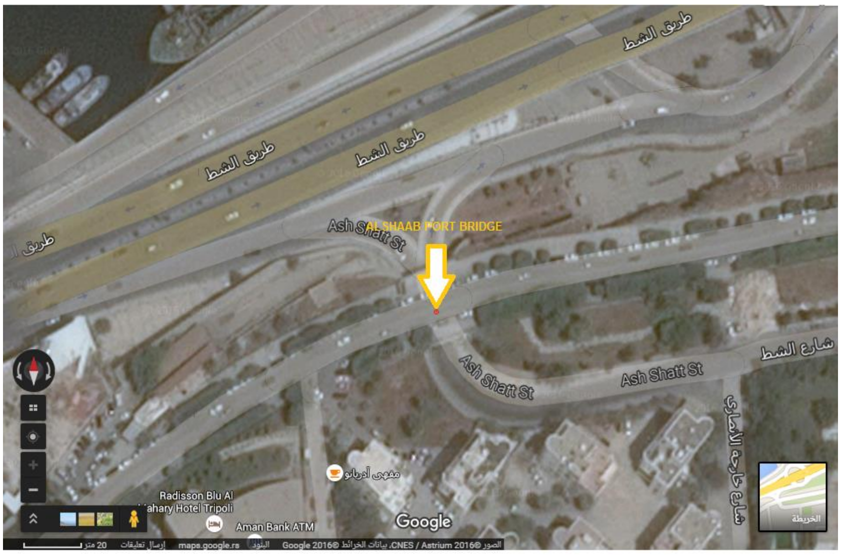


Figure VI-131. Alshaab Port Bridge location on google maps

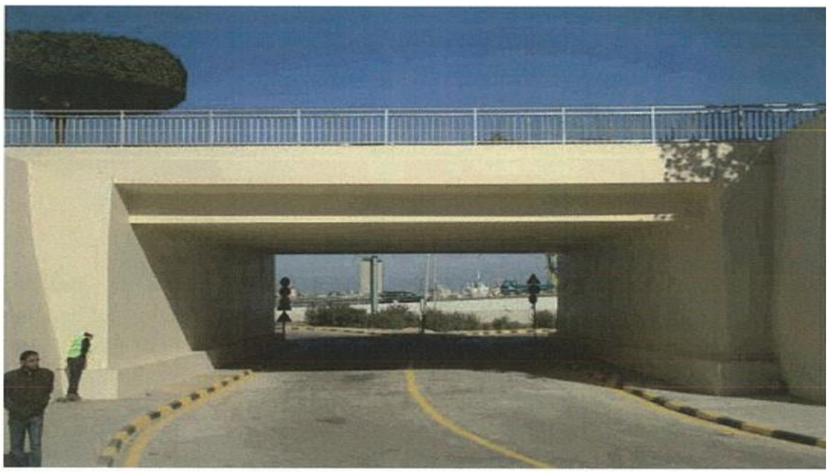


Figure VI-132. View of Alshaab Port Bridge

Figure VI-133. Alshaab Port bridge view

Type of bridge

Alshaab Port Bridge is designed as Simple Beam Bridge made of reinforced concrete. This bridge was built in the middle of XX centuries. In Figure VI-134 and VI-135 north and south sides of the bridge are shown. The plan of the bridge is given in Figure 7.136.

The characteristic dimensional data of the Bridge are:

- Length: 17,40m
- Width: 24.70m
- Height: 5.60m
- Main span: 13.40m
- Sidewalk (right side): 1.20m
- Sidewalk (left side): 1.20m

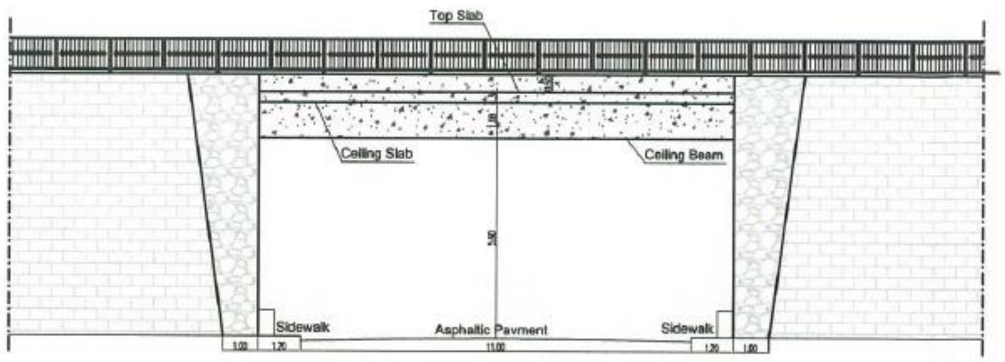


Figure VI-134. Longitudinal cross section of bridge (north side)

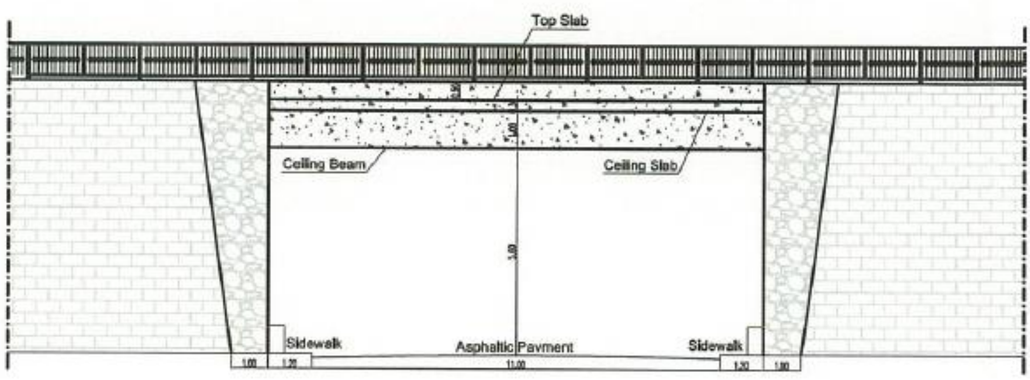


Figure VI-135. Longitudinal cross section of bridge (south side)

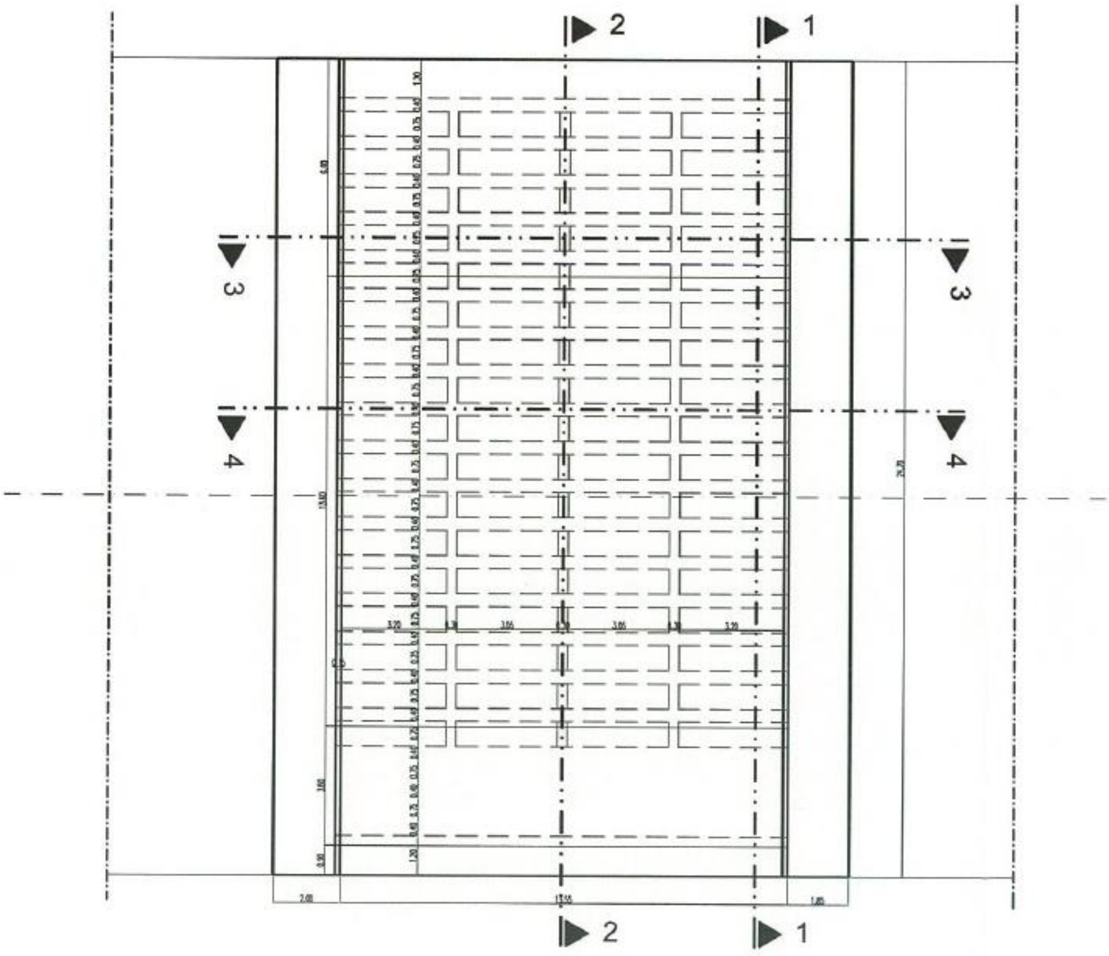


Figure VI-136. Plan of the bridge – upper side

Basic elements of bridge are:

- Abutment walls
- Ribbed deck slab and
- Cantilever slabs

Disposition of basic bridge elements are signed in Figures VI-137 and VI-138.

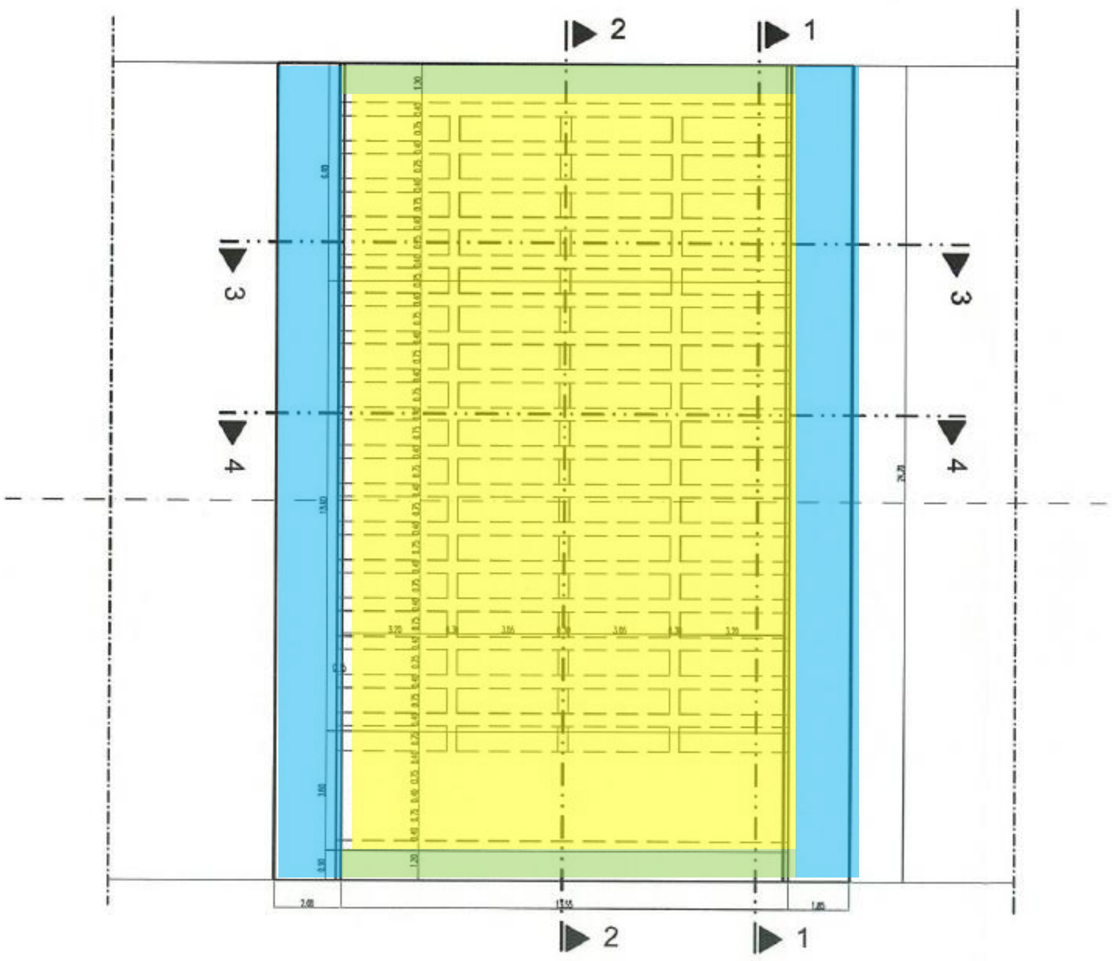


Figure VI-137. Disposition of cantilever slabs (green), ribbed deck slab (yellow) and abutment walls (blue) in plane of the bridge (upper side)

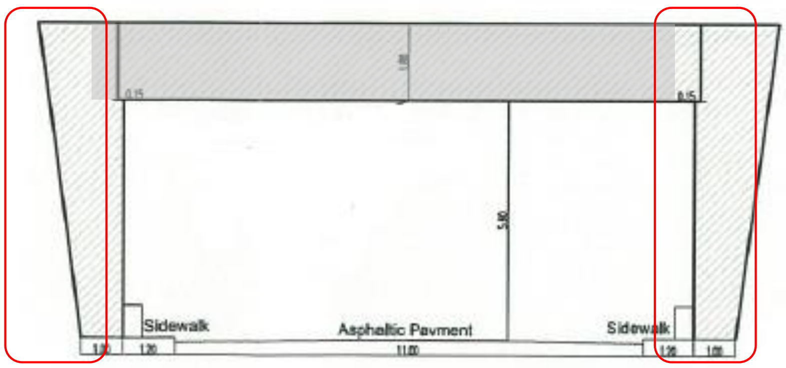


Figure VI-138. Disposition of abutment walls of bridge (section 4-4)

The following text provides a brief description of basic elements of the bridge.

Bridge Alshaab Port has two abutment stone walls. Their basic dimensions of these walls are:

- Length: 24.70m
- Height: 7.40m (visible part of total height)
- Width: from 1, 00 to 2,00m (continual change of width)

The abutments walls are made as masonry structures of local stone. Visible surfaces of walls are plastered with ordinary mortar.

Figure VI-139 shows the longitudinal view of abutment wall.

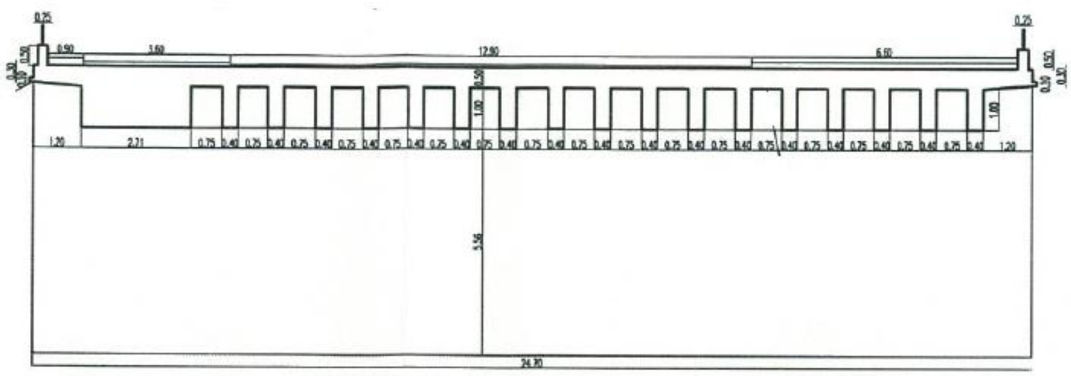


Figure Vi-139. Longitudinal view of abutment wall and cross-section (1-1) of superstructure (location and dimensions of main beams).

Superstructure is designed as one-way ribbed deck slab made of reinforced concrete. This slab consists of: main longitudinal beams/ribs, secondary transverse beams/ribs and ceiling slab.

Superstructure has 17 main beams/ribs, which are located at distance of 0,75m (each-other). The basic data of main beams/ribs are:

- Length:14,00m
- Width: 0,40m
- Height :1,80m

Secondary beams/ribs are located in quarters of span at the distance of 3,35m. The basic data of secondary beams/ribs are:

- Length:1,15m
- Width: 0,30m
- Height :1,50m

The basic data of ceiling slabs:

- Span :1,15m
- Thickness: 0,80m

Disposition of main and secondary beams/ribs and characteristic cross sections of superstructure are given in Figures Vi-140, VI-141, VI-142 below.

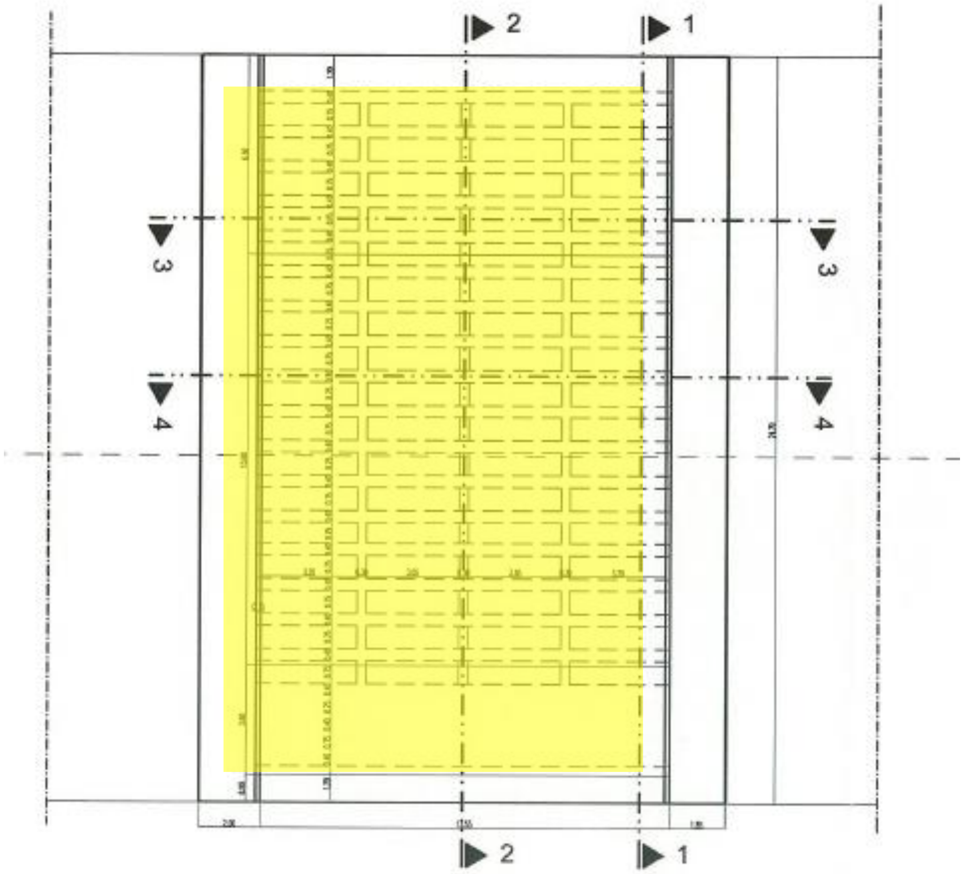


Figure VI-140. Position of main and secondary beams/ribs in superstructure, view from upper side

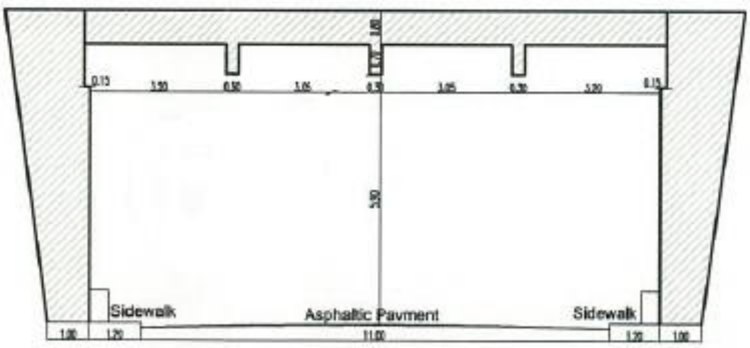


Figure VI-141. Dimensions of secondary beams, cross section (3-3)

The basic data of cantilever slabs are:

- Length :24.70m
- Width: 1,20m
- Depth: 0.30m

In Figures VI-142 and VI-143 the view of cantilever slab from upper side and characteristic dimension of cross section are shown.

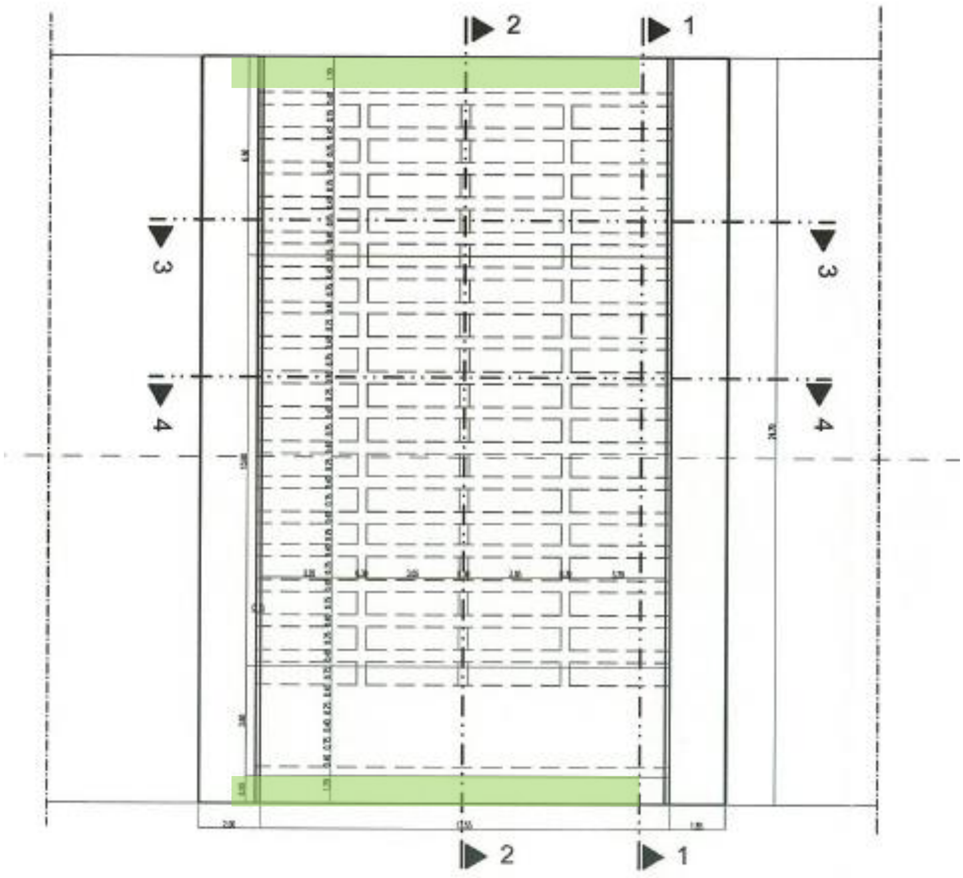


Figure VI-142. Location of cantilever slabs in plan of bridge

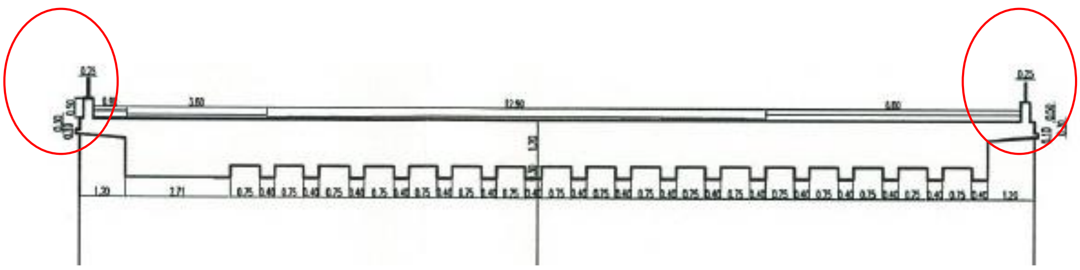


Figure Vi-143. Characteristic dimensions of cantilever slabs, cross section 2-2

6.2. Assessment of ALSHAAB PORT BRIDGE

In the aim of choose repair materials and techniques for this bridge, next activities were planned:

- In-situ testing of concrete quality and
- Visual inspection of visible parts of bearing elements.

Numbered activities were done in 2009.

6.2.1 Testing of concrete in bearing elements of bridge

The testing of concrete quality encompassed the next activities:

- Measurement of carbonation depth,
- Chloride ion content,
- In-situ testing of concrete by Schmidt Hammer test and
- In-situ testing of concrete by Pull-off method.

Carbonation depth

The extent of carbonation was assessed by treating with phenolphthalein indicator the fresh exposed surfaces of drilled cores, which were extracted from structure elements for testing concrete compressive strength or for testing carbonation depth.

All data of testing elements, measured depth of carbonation and rebar location are given in Table VI-43.

Table VI-43. Data of testing elements, measured depth of carbonation and rebar location

Reference 1	Carbonization 2		Rebar Depth 3	Depth of carbonization 4	In rebar plan 5	Element 6
	Yes	No	mm	mm		
01.01	x		5	60	Y	Ceiling beams
01.02	x		5	60	Y	Ceiling beams
01.03	x		5	50	Y	Ceiling beams
01.04	x		5	80	Y	Ceiling beams
01.05	x		5	80	Y	Ceiling beams
01.06	x		5	80	Y	Ceiling beams
01.07	x		5	80	Y	Ceiling beams
01.08	x		5	60	Y	Ceiling beams
01.09	x		5	20	Y	Ceiling beams
01.10	x		5	30	Y	Ceiling beams
01.11	x		5	20	Y	Ceiling beams
01.12	x		5	20	Y	Ceiling beams
01.13	x		5	20	Y	Ceiling beams

01.14	x		5	20	Y	Ceiling beams
01.15	x		5	20	Y	Ceiling beams
01.16	x		5	30	Y	Ceiling beams
01.17	x		5	60	Y	Slab ceiling
01.18	x		5	60	Y	Slab ceiling

After analyzing carbonation results, the next conclusions can be derivate:

- The front of carbonization passed behind the reinforced bars at all testing location.
- The depth of carbonization varied from 20mm to 80mm in ceiling beams, but in ceiling slabs had the constant depth of 60mm.
- Concrete cover has insufficient depth in all tested places (only 5mm).

Chloride test

The content of ion chloride in concrete is checked by using small pieces of drilled cores which were pulverized and dissolved in acid liquid. The chloride ions react with acid in an electrochemical reaction. An electrode was inserted into the liquid and the change in voltage was measured. On the basis of measured voltage, the instruments showed the chloride content in concrete in %. The obtained results are given in Table VI-44.

Table VI-44. Chloride test result

Elements 1	Reference 2	% Chloride in concrete (Equipment reading) 3			% Chloride ion content by mass of cement 4		
		0-2cm	2-6cm	6-8cm	0-2cm	2-6cm	6-8cm
Ceiling beam 1	01.01	0.0092	0.0019	0.0028	0.0012	0.0002	0.0004
Ceiling beam 2	01.02	0.0022	0.0018	0.0011	0.0003	0.002	0.0001
Slab ceiling	01.03	0.0039	0.0033	0.0033	0.0005	0.0004	0.0004

For analyzing given results, the next criterion was used: The maximum of chloride ion content by mass of cement for reinforced concrete with ordinary carbon steel is 0.40% (class Cl 0.40) (BS 8500).

After comparing obtained results with specified criterion, the next conclusion was made:

- All testing results are smaller than criteria value.
- Chloride content in concrete in the bridge structure is not hazardous to imbedded reinforced bars.

Core test

For testing concrete compressive strength, the core tests were done. Cores were extracted from ceiling beams at three different locations. In order to determine differences between surface and inner concrete quality, one core were taken out from whole depth of chosen beam.

In the laboratory one extracted core was split in two parts.

Then, all obtained cores were visually inspected and prepared for testing compressive strength by cupping. Testing procedure for compressive strength is described in standard BS 1881: Part 120:1983.

In aim to make conclusion of concrete quality, the average value and the range of estimated in-situ cube compressive strength are calculated and shone in the same table VI-45.

Table VI-45. Compressive strength test result

Compressive strength of concrete cores in Alshaab port bridge (MPa)- cube values				
	Element 1	Cube result 2	fck, average 3	Range of fckn 4
	Beams	19	28,33	19 - 44
		25		
		38		
		44		

Analyzing those results it can be seen that the difference between minimum and maximum value for tested elements is large and amounts up to 25MPa. These led to the conclusion that built-in concrete has very unequal quality and compressive strength differ from one to another location. The compressive strength also varies by depth for the same location but both obtained results are high, so this variation may be ignored.

Schmidt hammer test

For getting more information of built-in concrete quality the Schmidt hammer test, as a nondestructive - surface hardness method is chosen. Data about tested elements and number of measure places are given in table VI-46.

Table VI-46. Tested elements and number of measuring points

Element	Number of measuring point	Total number of measuring point per element	Total number of measuring point
Ceiling beams	10	10	20
Slab ceiling	10	10	

On each test location 10 rebound reading was done. Prior to test the surface of concrete was smoothed by carbonudum stone and cleaned. Rebound number was calculated by using next rule: Each result from one test location is valid if it is in range of ±7 points of average value. For each reading the single compressive strength was calculated by using calibration curves and finally the average and standard deviation are calculated too. The calculate values of compressive strengths and standard deviations are given in table VI-47.

Table VI-47. Schmidt hammer test result

Element 1	Wmed (MPa) 2	σ 3
Ceiling beams	18.45	0.81
	15.99	0.71
	15.88	1.24
	18.52	1.29
	23.42	1.06
	18.69	1.14
	13.11	1.15
	13.76	0.92
	12.42	0.86
	12.41	0.80
Slab ceiling	5.66	1.15
	5.12	0.54
	1.26	0.47
	5.27	0.96
	13.53	0.94
	4.79	0.80
	4.09	0.68
	10.46	0.70
	5.43	0.90
	7.12	1.17

Discussion and Conclusion

In order to make some conclusion of concrete compressive strength, obtained by Schmidt hammer test, the individual results were grouped and mean value of compressive strength and standard deviation were calculated. The obtained data are shown in Table VI-48.

Table VI-48. Schmidt hammer test result analyse

Element 1	Wmed (MPa) 2	σ (MPa) 3	Carbonization test
Ceiling beams	16,26	±3,544	Y
Slab ceiling	6.273	±3,439	Y

The results given in previous table show a very small dispersion of compressive strength for each analyzed element of superstructure, but high difference between ceiling beams and slab ceiling.

Results obtained by Schmidt hammer test are too small for reinforced concrete requirement, especially when carbonization is taken into account. It is well known that the carbonation makes concrete surface layer to be harder, but in cases of down surfaces of ceiling beams and deck slabs concrete cover is very thin and weak. As a consequence of that results obtained by Schmidt hammer test will not compare with compressive strength obtained by core test.

The compressive strength of concrete built in elements of superstructure corresponds to the class of concrete C20/25.

Pull off test

For measuring in-situ concrete tensile strength the pull-off method is used. The procedure is described in BS 1881: Part 207. The steel disks and epoxy resin glue were used. The tests were conducted in three places. Obtained results are given in table VI-49.

Table VI-49. Pull off test result

Reference 1	Element 2	W med (MPa) 3	σ 4	Failure mode (%) 5		
				concrete	Surface concrete	Epoxy glue
01.01/01.05	Slab ceiling	0.381	0.1	100%	0%	0%
01.06/01.10	Ceiling beams	0.401	0.08	100%	0%	0%

On the bases of given results, it can be seen that all values of tensile strength are smaller than minimum require value and that build-in concrete has very bad quality.

Density

Calculation of density of hardened concrete is very good method for checking the quality of built-in concrete. For calculation of the concrete density, the mass of extracted cores is usually used. Obtained results are given in table VI-50.

On the bases of given results, it can be seen that all values of densities are close to expected value (~2300kg/m^3). It is supposed that the concrete was enough compacted.

Table VI-50. Density test result analyse

Element of structure	Density, kg/m^3	Average for each measuring place kg/m^3	Average for element of structure, kg/m^3
Beams	2270	2308	2290
	2346		
	2270	2272	
	2274		

6.2.2 Visual inspection of Bridge Alshaab Port

First visual inspection of all visible bearing elements or part of bearing elements of bridge were done in 2009.

During the visual inspection an insufficient concrete cover on visible part of superstructure elements is noticed. Also, a lot of damages were registered.

Characteristic damages are:

- Corrosion of reinforced bars
- Damage of concrete due to reinforcement corrosion.
    - Falling down of cover.
    - Cracking of cover especially in corners
- White and dark stains on concrete surface.

Main causes of described damages are:

- Carbonation
- Poor quality of concrete surface
- Insufficient depth of concrete cover
- Wind
- No adequate water drainage system

Figures VI-144 – VI-148 show characteristic damages of RC elements.

Figure VI-144. General view of the bridge after 50 years of utilization

Figure VI-145. Damaged main beams: dark and white steins, reinforcement corrosion, spalling of concrete cover

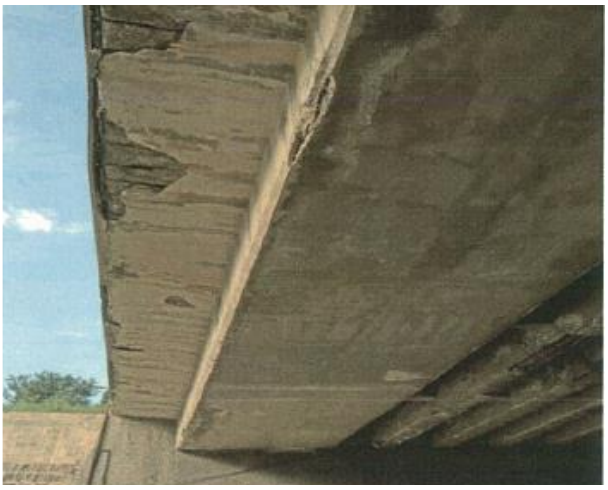


Figure VI-146. Cantilever slabs with damaged concrete (south side): delamination of concrete due to corrosion of rebars and running down of water

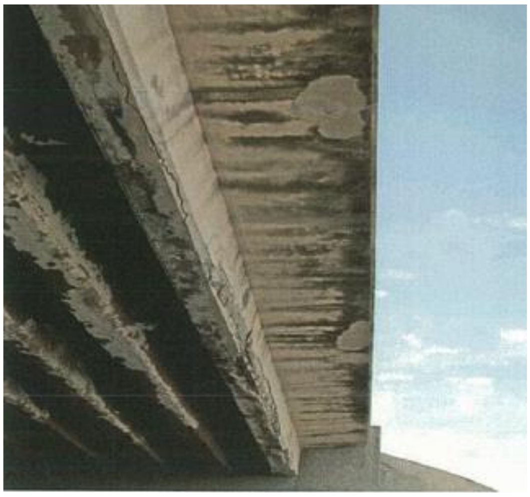


Figure VI-147. View of cantilever slab and main beams (north side): Longitudinal crack along the corner rebar, dark and white steins on beams and cantilever slab

FigureVI-148. Exposed reinforced bars in slabs beams

The most damaged elements are main-longitudinal beams and cantilever slabs. The characteristic damage is reinforcement corrosion and, as result of that, falling down of concrete cover (Fig. VI-146, Vi-148). Some reinforced bars were bared and affected by surface corrosion. The edges of slabs and beams are rough and stain of water- and water-soluble salts can be seen. Due to the corrosion expansion of bars some parts of concrete cover on down side of cantilever slabs have been delaminated and falling to the road (Fig. VI-146).

Plaster Layer of abutment walls has been damaged.

Visual inspection encompassed other bridge elements, like sidewalks, curb stones, catch pits and fences. All mentioned elements have been seriously damaged.

6.3. General conclusion for bridge Alshaab Port

The bridge Alshaab has been old about 50 years when it was inspected for the first time. The main conclusion of the inspection was that the bridge is damaged.

The characteristic defect of reinforced elements has been insufficient concrete caver.

The main cause of damage appearance is insufficient concrete cover. Measured value of concrete cover in elements of superstructure (ceiling beams and slabs) is only 5mm.

The second cause of damage appearance is concrete carbonization. Depth of carbonization varied from 20mm up to 80mm and in all tested locations front of carbonation passed behind the reinforced bars.

The next cause of damage appearance is inadequate drainage of water from the deck. This problem caused leakage of water over the edge of cantilever slabs. Consequently, the corrosion of reinforced bars in deck ceiling and cantilever slabs were caused.

Analyzing concrete compressive strength obtained by cores it can be seen that the difference between minimum and maximum value is large and vary from 19 to 44MPa. This led to the conclusion that built-in concrete has very unequal quality and compressive strength differ from one to another location

The results of concrete compressive strength obtained by Schmidt hammer test are too small for reinforced concrete and differ from values obtained by core test. Consequently, they did not taken into account for estimation of concrete compressive strength.

According EN 206-1 the compressive strength classes of concrete given in next table can be used for the control calculation.

Bridge Element	Compressive strength class
Ceiling beams	~C20/25

On the bases of results obtained by pull-off method it can be concluded that concrete tensile strength is very low and smaller than minimum require value.

Chloride content in concrete in the bridge structure is not hazardous to imbedded reinforced bars.

Finally, the main conclusion can be drawn:

- Durability of all elements of superstructure is decreased, because of numerous damages that occurred in elapsed time.
- Bearing capacity of structural elements is not jeopardized because there are no serious cracking or deformations of RC elements.
- Global stability and stability of each structural element are not threatened and
- Functionality of Bridge is partly reduced, because of damages of surface asphalt layers and local instability of delaminated concrete pieces, that occurred on the bottom sides of main ceiling beams and cantilever slabs.

7. ABDUL SALAM AREF BRIDGE

7.1. Technical description

Location of bridge: Abdul Salam Aref

Bridge Abdul Salam Aref is located in the west part of the capital Tripoli, about 567.43 meters from the sea to the north. It is considered as a major bridge to the capital Tripoli. It connects several main roads leading to the center of the capital. Close to the rapid transit station. In Figures 7.149, 7.150 and 7.151 are shown situation plan and views of bridge. The coordinates for this bridge are

320 53' 56.4" N 130 12' 47.0" E

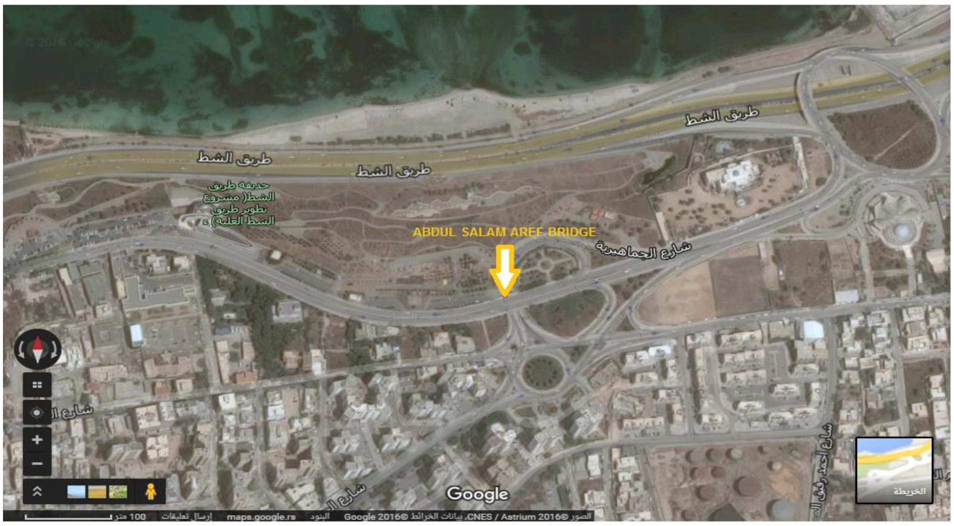


Figure VI-149. Abdul Salam Aref bridge location on Google maps

Figure VI-150. Abdul Salam Aref bridge

Figure VI-151. Abdul Salam Aref bridge aspect

Type of bridge

Bridge Abdul Salam Aref is designed as two spans; I beam bridge with a row of columns supporting the deck in the center. In longitudinal way the bridge is split in two independent parts, so it consists of two parallel twins' bridges. They were made of reinforced concrete. This bridge was built in the middle of XX centuries. In Figure VI-152 and VI-153 north and south sides of the bridge are shown. The plan of the bridge is given in Figure VI-154.

The characteristic dimensional data of the bridge are:

- Length: 19.69m
- Width: 2x12.5m +0.20m (expansion joint) =25.20m
- Height (the distance between sidewalk and down side of main girders): 4.85m
- Main span: 8,545m
- Sidewalk: (right and left side): 1.50m

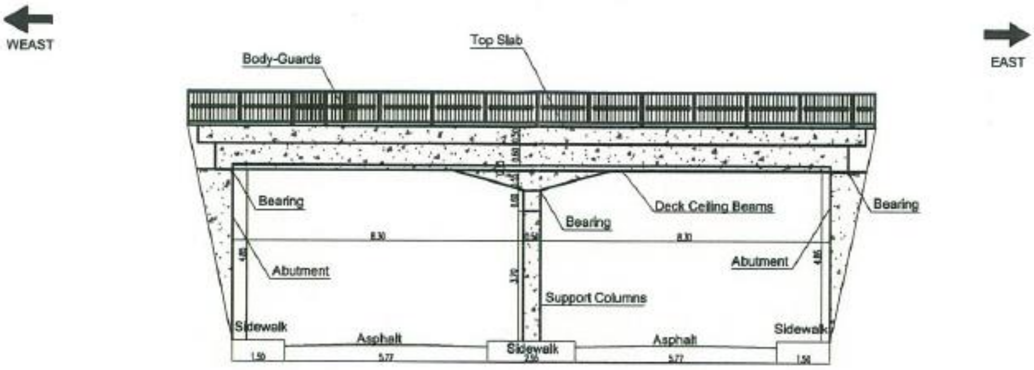


Figure VI-152. Longitudinal cross section of bridge (north side)

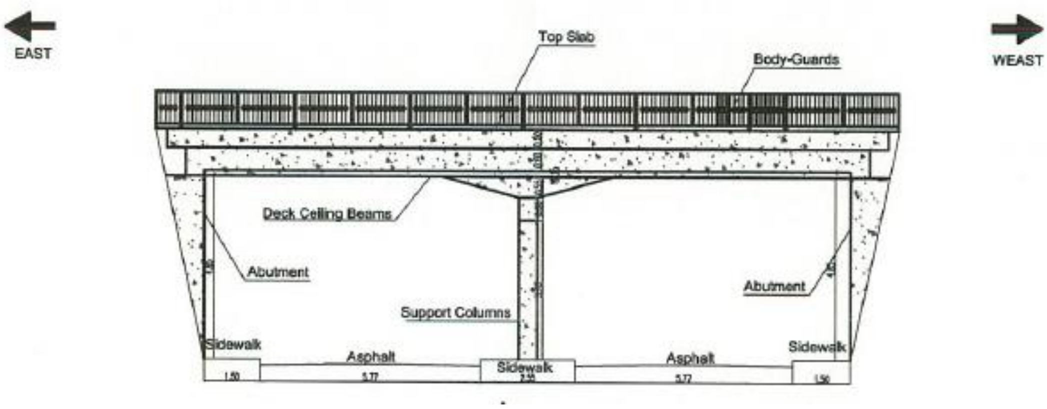


Figure VI-153. Longitudinal cross section of bridge (south side)

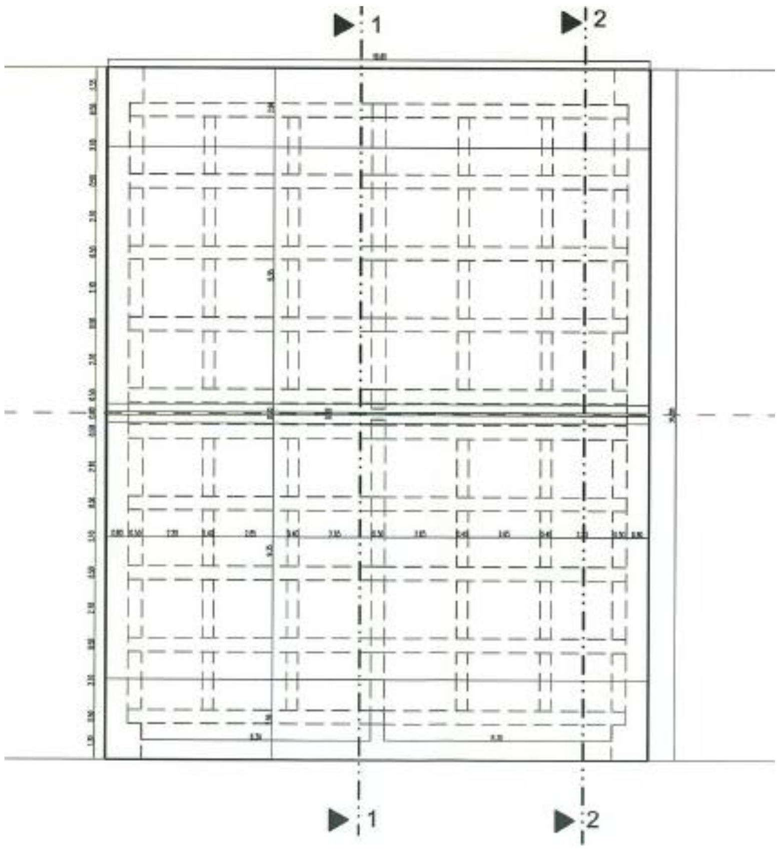


Figure VI-154. Plan of the bridge – upper side

Basic elements of bridge are:

- Abutment
- Support columns
- Transverse beam
- Top slab
- Deck ceiling beams
- Cantilever slab

Disposition of basic bridge elements are signed in Figures VI-155 and VI-156.

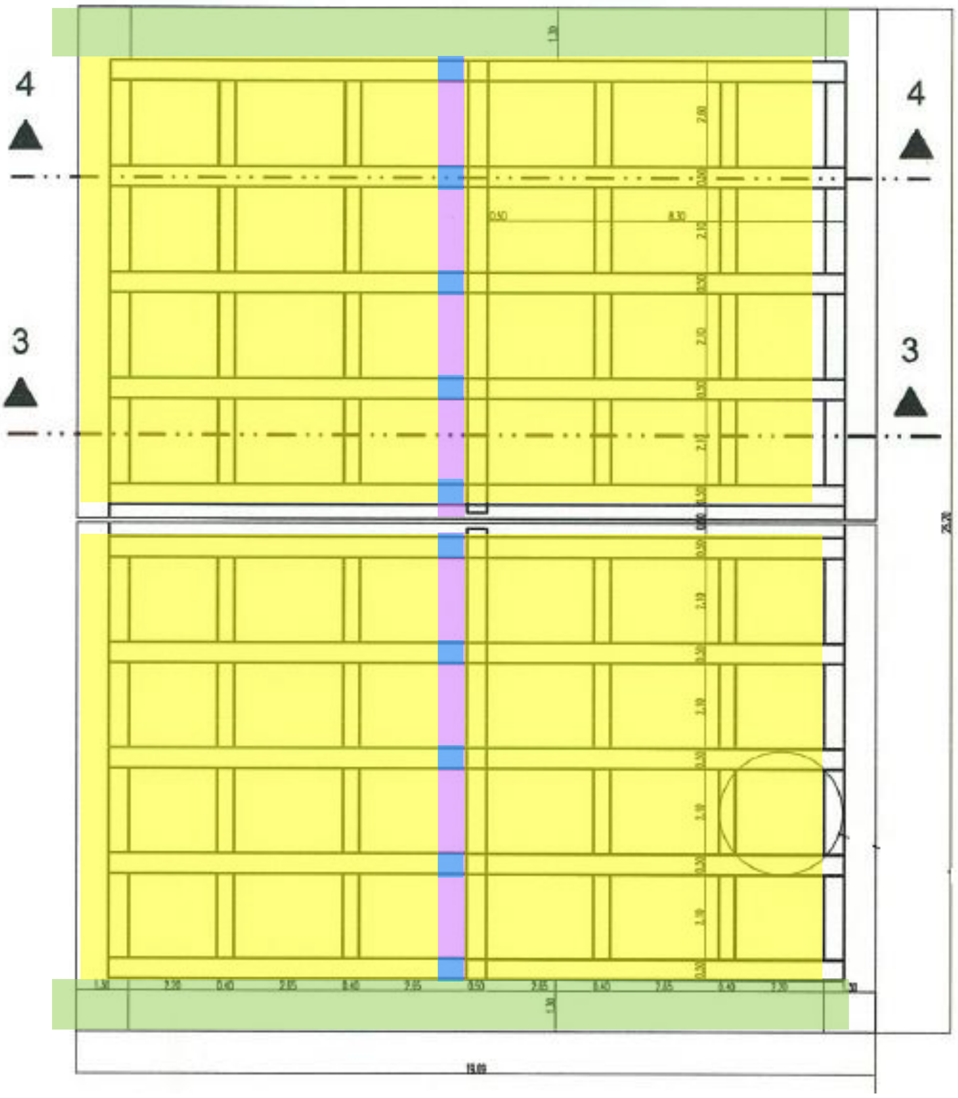


Figure VI-155. Disposition of cantilever slabs (green), top slab and deck ceiling beams (yellow), support columns (blue) and abutments () in plane of the bridge (bottom side)

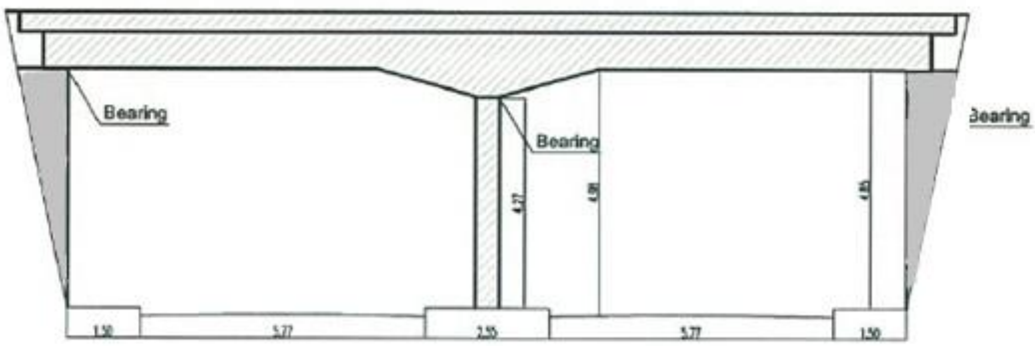


Figure VI-156. Disposition of abutments of bridge (gray) (section 4-4)

The following text provides a brief description of basic elements of the bridge.

Bridge Abdul Salam Aref has four abutments. The basic dimensions of each abutment are:

Abutment on east side:

- Length: 12.5m
- Height: 4.85m (visible part of total height)
- Depth: 1.3 m

Abutment on west side

- Length: 12.5m
- High: 4.85m (visible part of total height)
- Depth: 1.3m

Figure VI-157 shows the longitudinal view of abutment.

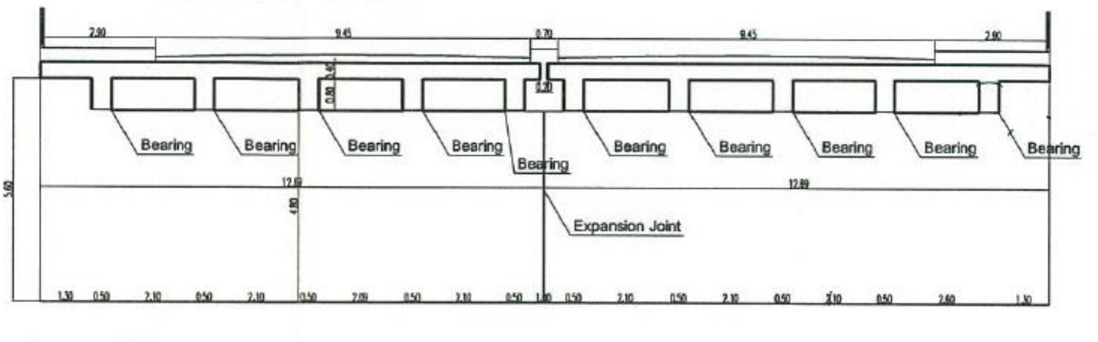


Figure VI-157. Longitudinal view of interior wall

Support columns located in the middle of span. Characteristic dimensions are:

- Number of column: 10
- Height: 4.27m
- Cross section: squared 0.50x0,50m

Disposition of support columns and characteristic cross sections are given in Figures VI-158, VI-159, VI-160 below.

Transverse beam is located above the columns. Its role is to form continuous frame with columns and, in that way, improve stability of columns. Characteristic dimensions are:

- Span :4x2.6m
- Cross section: rectangular, 0.50x0,60m

Disposition of transverse beam and characteristic cross sections are given in Figures VI-158, VI-159, VI-160.

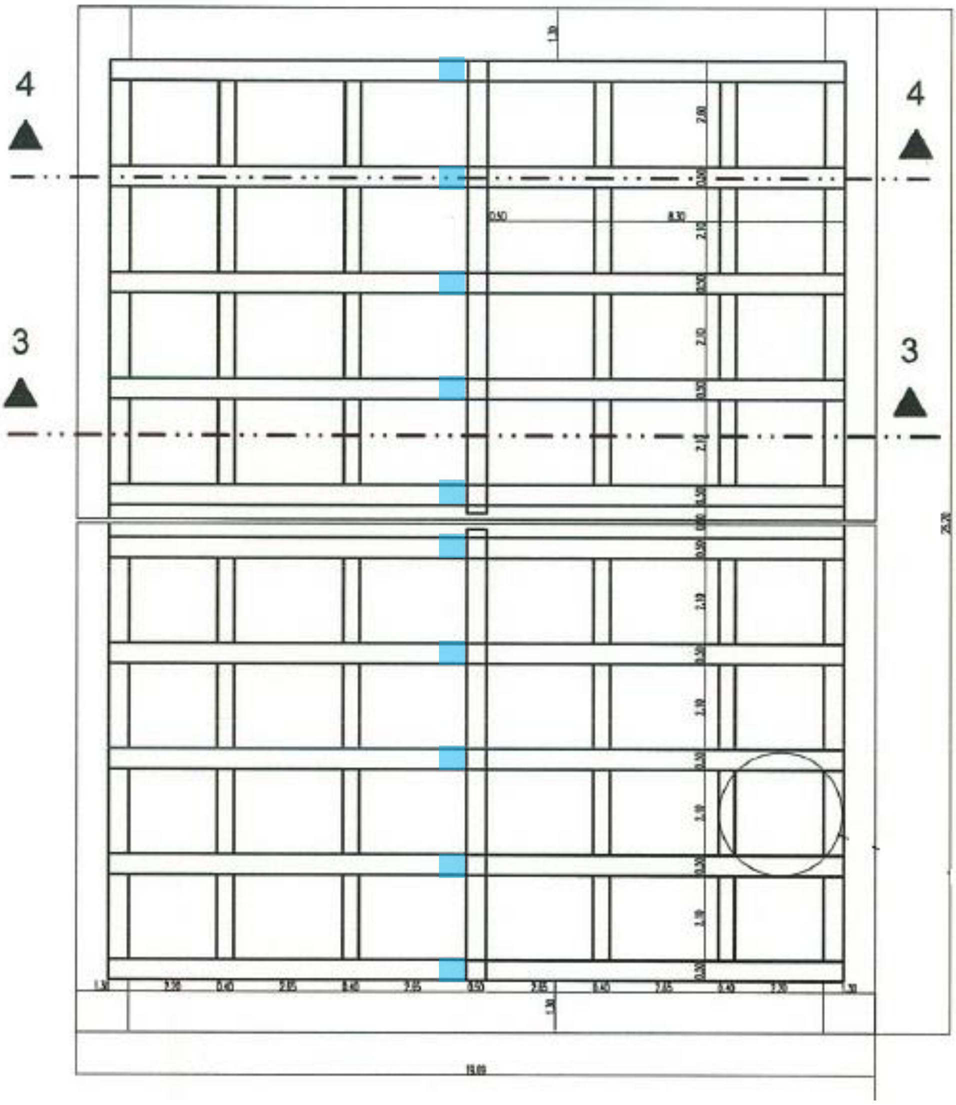


Figure VI-158. View of support columns from bottom side

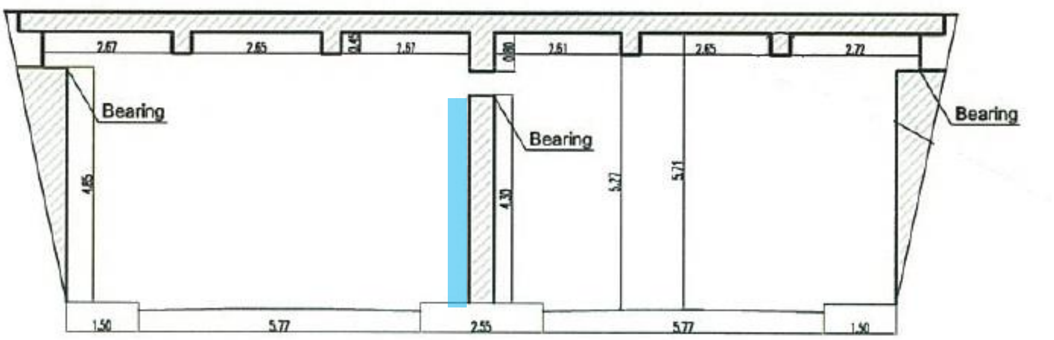


Figure VI-159. Disposition of Support columns in the span of bridge (section 3-3)

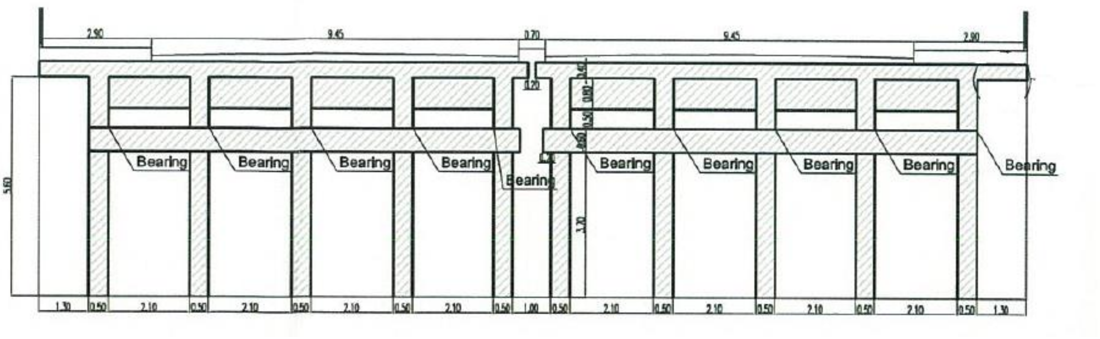


Figure VI-160. Longitudinal view of Support columns, disposition of transverse beam

The upper part of Abdul Salam Aref bridge consists of main (longitudinal) and secondary (transversal) girders (Deck ceiling beams) and top slab.

Characteristic dimensions of mentioned elements are:

Main girder:

- Length: 8,545m
- Cross section: I beam, height:1.2m with haunch of 0.50m in height, near column, width:0.50m

Secondary girder (middle and external)

- Length: 2.60m
- Cross section: rectangular, height:0.80m, width:0.50m

Secondary girder (other)

- Length: 2.60m
- Cross section: rectangular, height:0.45m, width:0.40m

Top slab:

- Cross section: full, with constant depth of 0.40m

7.2. Assessment of Bridge Abdul Salam Aref

For the purpose of choosing repair materials and techniques for this bridge, next activities were planned:

- In-situ testing of concrete quality and
- Visual inspection of visible parts of bearing elements.

Numbered activities were done in 2009.

7.2.1 Testing of concrete in bearing elements of bridge

The testing of concrete quality encompassed the next activities:

- Measurement of carbonation depth,
- Chloride ion content,
- In-situ testing of concrete by taking of cores,
- In-situ testing of concrete by Schmidt Hammer test and
- In-situ testing of concrete by Pull-off method.

Carbonation depth

The extent of carbonation was assessed by treating with phenolphthalein indicator the fresh exposed surfaces of drilled cores, which were extracted from structure elements for testing concrete compressive strength or for testing carbonation depth.

All data of testing elements, measured depth of carbonation and rebar location are given in Table VI-51.

Table VI-51. Data of testing elements, measured depth of carbonation and rebar location

Reference 1	Carbonization 2		Rebar Depth 3	Depth of carbonization 4	In rebar plan 5	Element 6
	Yes	No	mm	mm		
02.01	x		100	70	N	Abutment
02.02	x		100	70	N	Abutment
02.03	x		100	100	Y	Abutment
02.04	x		100	70	N	Abutment
02.05	x		100	70	N	Abutment
02.06	x		100	70	N	Abutment
02.07	x		50	20	N	Support column
02.08	x		50	20	N	Support column
02.09	x		50	20	N	Support column
02.10	x		50	20	N	Support column
02.11	x		50	40	Y	Support column
02.12	x		50	50	Y	Support column
02.13	x		-	30	-	Ceiling
02.14	x		-	30	-	Ceiling
02.15	x		-	30	-	Ceiling
02.16	x		-	40	-	Ceiling
02.17	x		-	40	-	Ceiling
02.18	x		-	50	-	Ceiling

After analyzing carbonation results, the next conclusions can be derivate:

- The minimum depth of carbonization is 20mm.

- The maximum depth of carbonization is 100mm
- The mean values are: 75mm for abutment, 28mm for support column, and 36mm for ceiling.
- The carbonization is most expressed in abutment.

Chloride test

The content of ion chloride in concrete is checked by using small pieces of drilled cores which were pulverized and dissolved in acid liquid. The chloride ions react with acid in an electrochemical reaction. An electrode was inserted into the liquid and the change in voltage was measured. On the basis of measured voltage, the instruments showed the chloride content in concrete in %. The obtained results are given in Table VI-52.

Table VI-52. Chloride test result

Elements 1	Reference 2	% Chloride in concrete (Equipment reading) 3			% Chloride ion content by mass of cement 4		
		0-2cm	2-6cm	6-8cm	0-2cm	2-6cm	6-8cm
Support column1	02.01	0.0162	0.0148	0.0058	0.0020	0.0019	0.0007
Interior wall	02.02	0.0013	0.0013	0.0008	0.0002	0.0002	0.0001
Support column 2	02.03	0.0128	0.0040	0.0038	0.0016	0.0005	0.0005
Ceiling 1	02.04	0.0368	0.0150	0.0041	0.0046	0.0019	0.0005
Ceiling 2	02.05	0.0070	0.0021	0.0020	0.0009	0.0003	0.0003
Ceiling 3	02.06	0.0048	0.0021	0.0020	0.0006	0.0003	0.0003

In analyzing given results next criterion was used: The maximum of chloride ion content by mass of cement for reinforced concrete with ordinary carbon steel is 0.40% (class Cl 0.40) (BS 8500).

After comparing obtained results with specified criterion, the next conclusion was made:

- All testing results are smaller than criteria value.
- Chloride content in concrete in the bridge structure is not hazardous to imbedded reinforced bars.

Core test

For testing concrete compressive strength, the core tests were done. Cores were extracted from three different locations. In order to determine differences between surface and inner concrete quality, cores were taken out from whole depth of elements. The chosen locations for taking out cores were:

- Abutment - three cores,
- Beam - three cores

In the laboratory extracted cores were splitting in the next way:

- In two parts from interior walls and
- In three parts for beam.

Then, all obtained cores were visually inspected and prepared for testing compressive strength by cupping. Testing procedure for compressive strength is described in standard BS 1881: Part 120:1983. All obtained results of estimate in-situ compressive strength are given in table VI-53, and they represent cube compressive strength. For changing cylinder compressive strength to cube compressive strength, the factor of correction was used. This factor depends of dimensions of specimens and of direction of drilling.

In aim to make conclusion of concrete quality, the average value and the range of estimated in-situ cube compressive strength are calculated and shone in the same table VI.53.

Table VI-53. Compressive strength test result

	Compressive strength of concrete cores in Abdul salam aref bridge (MPa)-cube values			
	Element 1	Cube result 2	fck, average 3	Range of fckn 4
	Abutment	25	26.50	18-35
		28		
		35		
		18		
	Beams	34	34.33	24-46
		24		
		34		
		46		
		32		
		36		

Analyzing those results it can be seen that the difference between minimum and maximum value for abutment is large and amounts to 17MPa. This led to the conclusion that built-in concrete has very unequal quality and compressive strength differ from one to another location. The compressive strength varies by depth for the same location for one of three extraction places.

The obtained value of concrete compressive strength of tested element is relatively small (~26.50MPa).

Difference between minimum and maximum value for beams is large and amounts to 22MPa. This led to the conclusion that built-in concrete has very unequal quality and compressive strength differ from one to another location. The compressive strength varies by depth for the same location for one of three extraction places.

The obtained value of concrete compressive strength of tested element is ~34.33MPa.

Schmidt hammer test

For getting more information of built-in concrete quality the Schmidt hammer test, as a nondestructive- surface hardness method, is chosen. Data about tested elements and number of measure places are given in table VI-54.

Table VI-54. Tasted elements and number of measuring points

Element	Number of measuring point	Total number of measuring point per element	Total number of measuring point
Abutment	10	10	30
Support columns	10	10	
Deck ceiling	10	10	

On each test location 10 rebound reading was done. Prior to test the surface of concrete was smoothed by carbonudum stone and cleaned. Rebound number was calculated by using next rule: Each result from one test location is valid if it is in range of ±7 points of average value. For each reading the single compressive strength was calculated by using calibration curves and finally the average and standard deviation are calculated too. The calculate values of compressive strengths and standard deviations are given in table VI-55.

Table VI-55. Schmidt hammer test result

Element 1	Wmed(MPa) 2	σ 3
Abutment	21.85	1.76
	13.22	1.64
	10.86	1.18
	9.25	1.05
	12.65	1.57
	17.57	1.12
	22.28	1.35
	9.64	1.39
	13.93	1.21
	17.39	1.20
	7.10	2.08
	4.44	1.57
	30.69	1.44

Support columns	16.09	2.44
	15.31	0.82
	18.69	2.78
	22.08	1.13
	22.13	5.71
	20.78	1.58
	19.09	1.52
Deck ceiling	33.77	1.86
	29.82	1.49
	20.08	1.32
	34.90	2.18
	20.83	1.12
	19.93	1.71
	35.08	3.44
	33.99	0.71
	29.34	1.01
	30.93	2.58

Discussion and Conclusion

In order to make some conclusion of concrete compressive strength, obtained by Schmidt hammer test, the individual results were grouped and mean value of compressive strength and standard deviation were calculated. The obtained data are shown in Table VI-56.

Table VI-56. Schmidt hammer test result – analyze

Element 1	Wmed (MPa) 2	σ (MPa) 3	Carbonization test
Abutment	14.864	±1.347	Y
Support columns	17.64	±2.107	Y
Deck ceiling	28.867	±1.742	Y

Some results given in Table VI-56 can be compared with results of compressive strength obtained by testing cores, Table VI-532.

The coefficient of correction is calculated by average of compressive strength obtained by core for ceiling beams and average of compressive strength obtained by Schmidt hammer for the same elements:

$f_{ck,av}$=30,415 MPa (average compressive strength obtained by core)

$f_{ch,av}$=20.457MPa(average compressive strength obtained by Schmidt hammer) fc, comp=30.415/20.457=1.487

In table VI-56a the average results of compressive strength before and after correction has been shown.

Table VI-56a. Correction of Schmidt hammer compressive strength

Element	compressive strength before correction (MPa)	compressive strength after correction (MPa)
Abutment	14.864	22.10
Support columns	17.64	26.23
Deck ceiling	28.867	42.92

Pull off test

For measuring in-situ concrete tensile strength the pull-off method is used. The procedure is described in BS 1881: Part 207. The steel disks and epoxy resin glue were used. The test was conducted in three places. Obtained results are given in table VI-57.

Table VI-57. Pull off test result

Reference 1	Element 2	W med (MPa) 3	σ 4	Failure mode (%) 5		
				concrete	Surface concrete	Epoxy glue
02.01/02.05	Ceiling	1.420	0.40	0%	80%	20%
02.06/02.10	Abutment	0.928	0.10	80%	20%	0%
02.11/02.15	Support columns	0.435	0.22	84%	16%	0%

On the bases of given results, it can be seen that all values of tensile strengths don't satisfied required criterion (>1.5MPa).

Density

Calculation of density of hardened concrete is very good method for checking the quality of built-in concrete. For calculation of the concrete density, the mass of extracted cores is usually used. Obtained results are given in table VI-58.

On the bases of given results, it can be seen that all values of densities are close to expected value (~2300kg/m^3). It is supposed that the concrete was enough compacted.

Table VI-58. Density test result analyse

Element of structure	Density, kg/m^3	Average for each measuring place kg/m^3	Average for element of structure, kg/m^3

Abutment	2364,819	2364,819	2356
	2368,391	2385,779	
	2403,167		
	2317,707	2317,707	
Beams	2266,365	2279,691	2290
	2293,018		
	2324,991	2271,14	
	2217,396		
	2309,369	2317,684	
	2326		

7.2.2 Visual inspection of Bridge Abdul Salam Aref

First visual inspection of all visible bearing elements or part of bearing elements of bridge were done in 2009.

During the visual inspection o lot of damages were registered, especially on columns and abutments.

Characteristic damages are:

- Corrosion of reinforcing bars
- Damage of concrete due to corrosion of still.
    - Cracking of cover
    - Falling off (delamination) of cover
- Water stains on concrete surface
-

Main reasons that caused described damages are:

- Carbonation
- Poor quality of built in concrete
- Insufficient depth of cover
- Wind
- No adequate water drainage system
- Non adequate maintenance of expansion joints

Figures VI-161-VI-166 show characteristic damages of RC elements.

Figure VI-161. Damaged column: Exposed reinforcement bars, corrosion of rebars, cracking of concrete along the edge rebars, spalling off corner concrete

Figure VI-162. Damaged column: A large delamination and spalling off of cover, Exposed corroded reinforcing bars

Figure Vi-163. Cracking of cover in support column, spalling off corner concrete

Figure Vi-164. Damaged upper part of abutment: deep spalling off of caver

Figure Vi-165. Beam for fence with damaged concrete

Figure VI-166. Beam for fence: Corrosion of reinforcing bars, insufficient depth of cover, cracking and falling off of concrete cover, water steins due to overflow of water over the edge of the beam

The drainage system of traffic lanes below the bridge is completely blocked with sand Fig VI-167. Because of that, storm water is collected below the bridge and formed accumulation of water up to 1,5m high. Those accumulations need long period of time for drying. The wetting and drying cycles during service life of bridge lead to heavy corrosion of the reinforcement in columns and abutments.

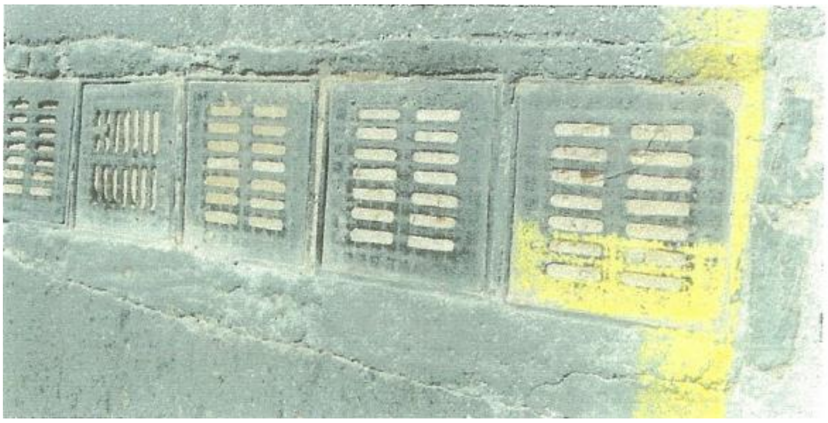


Figure VI-167. Blockage sewerage with sand

The most damaged elements are supporting columns and abutments. The characteristic damage is deep delamination and falling down of concrete cover (Fig. VI-159 – VI-162) and corrosion of rebars. The delamination and spalling off of caver affected a large area of columns, especially in corners, and a large area of abutments, also. Exposed reinforcing bars lost adhesion with concrete core (other part of concrete). The existing reinforcing bars in abutments are located very deep, almost in the middle of cross section and do not have any role in caring and transfer of load.

The next most damage element is beam for fence in the bridge deck. These elements have damages in the form of surface corrosion of rebars followed by cracking and falling off of cover (Fig. VI-161 and VI-168). The edges of beams are rough and stain of water- and water-soluble salts can be seen (Fig. VI-166). The main cause of described damage appearance is insufficient concrete cover and overflow of water over the edge of those beams.

Other concrete structural elements (transversal beam, deck ceiling beams with top slab are also damaged due to corrosion of reinforced bars in the form of cracking and spalling of concrete cover, but described damages aren't deep and catch only concrete cover. The main cause of described damage appearance is insufficient concrete cover and leakage of water over through expansions and cold joints. The concrete cover is thin and, in some cases, doesn't exist. Characteristic view of down surface of damaged deck ceiling beams is illustrated in Fig. VI-168.

Figure VI-168. Damaged deck ceiling beams, exposed rebar, thin cover, spalling off of cover

Visual inspection encompassed other bridge elements, like sidewalks, curb stones, catch pits, fences and expansion joints. All mentioned elements have been seriously damaged.

7.3. General conclusion for bridge Abdul Salam Aref

The bridge Abdul Salam Aref has been old about 50 years when it was inspected for the first time. The main conclusion of the inspection was that the bridge is damaged.

All inspected elements had the problem with carbonation. The depth of carbonization varied from 20mm up to 100mm. The carbonization was most expressed in abutments and the average value of depth of carbonization for those elements is 75mm. On the basis of measured results of carbonization depth and location of rebars in abutments and support columns, it can be concluded that front of carbonization hasn't reached rebars but in some cases got close to the bars.

Insufficient cover is characteristic for side surfaces of beams for fence and for down part of deck ceiling beams and top slab.

The main cause of damage appearance especially in columns and abutments is inadequate drainage of water from the deck and under bridge traffic lanes. This problem caused leakage of water through joints and overflow of water over the edge of slab. Consequently, the local corrosion of reinforced bars in cantilever slabs and beams for fence were caused. Also, a local flood usually appears during heavy rain, and cause wetting of columns and abutments.

The most damage elements are supporting columns and abutments. The main cause of damages is corrosion of reinforcing bars. The delamination and spalling off of caver affected a large area of columns, especially in corners, and a large area of abutments, also. Exposed reinforcing bars lost adhesion with concrete core. During visual inspection an inadequate arrangement of stirrups has been noticed in columns and the inadequate arrangement of horizontal and vertical rebars has been spotted in abutments. The distance between stirrups and reinforcing bars is too large. Other concrete elements also have damages caused by corrosion of still, but the degree of registered damages is lower than those in columns and abutments.

Analyzing results of core compressive strength, it can be seen that the difference between minimum and maximum value is large for both tested elements (abutment and beam). This led to the conclusion that built-in concrete has very unequal quality and compressive strength is changed not only from one to other location, but through the depth of the same element, also. The obtained value of concrete compressive strength for both tested elements fulfilled criterion for reinforcement concrete.

The results of concrete compressive strength obtained by Schmidt hammer test show moderate dispersion of compressive strength for each analyzed element of structure, but very large dispersion between abutment and columns in comparison with deck ceiling. Also, obtained results for columns and abutments, pointed out very bad quality of concrete in surface layer (cover).

According EN 206-1 the compressive strength classes of concrete given in next table can be used for the control calculation.

Bridge Element	Compressive strength class
Abutment	C20/25
Deck Ceiling Beams	C25/30
Deck slab	C25/30
Support columns	C20/25

On the bases of results obtained by pull-off method it can be concluded that concrete tensile strength is smaller than require value.

Chloride content in concrete in the bridge structure is not hazardous to imbedded reinforced bars.

Average value of Density of hardened concrete is 2323kg/m^3. This value match expected value (~2300kg/m^3) and it can be concluded that built in concrete is well compacted.

Finally, the main conclusion can be drawn:

- Durability of all structural elements is decreased, because of numerous defects that occurred during the construction of this bridge.
- Built in concrete has compressive strength that varied from C20/25 by C25/30) and satisfactory density (~2320kg/m^3)
- The carbonization exists in all concrete elements.
- Bearing capacity of supporting columns and abutments is jeopardized because of concrete cross section decreasing and losing of adhesion between rebars and surrounding concrete.
- Bearing capacity of other structural elements is not jeopardized.
- Global stability and stability of each structural element are not threatened and
- Functionality of bridge is partly reduced, because of damages of surface asphalt layers and local floods of under bridge traffic lanes during heavy rain.

CHAPTER VII

Rating and ranking of bridges before repair

CHAPTER VII

Rating and ranking of bridges before repair

INTRODUCTION

Major inspections involve visual inspection and testing (material investigations) of all parts of a structure.

Damage and condition assessment are performed according to Germany methodology. Directive for Uniform Determination, Assessment, Recording, and Analysis of the Results of the Inspection of the Structures (German methodology is described in chapter IV).

In this chapter, seven bridges in Libya were evaluated according to the German methodology, and all the damages in each bridge were counted. And knowing which bridge has a lot of damage and needs maintenance first.

This assessment of the condition of the bridges was in 2009.

1. SOUK ATHULATHA 1 BRIDGE

1.1. LEVEL 1: REGULAR BRIDGE INSPECTION

Group 1: Superstructure

Description of damage and defects	**S**	**V**	**D**	
Bridges, concrete / reinforced concrete / pre-stressed construction structures				
Dependencies: type of construction = bridge, basic building element = structure construction, material of the spanning structure = concrete				
Graffiti on visible surfaces	0	0	0	$Z_{1.1}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{1.2}$
Less wear of the protective layer	0	0	1	$Z_{1.3}$
The rust on the lower sides of the construction	0	0	1	$Z_{1.4}$
Pollution of internal passages of the building (remains of the formwork or other)	0	0	0	$Z_{1.5}$
Pollution of internal passages of building (bird feces or other)	X	X	X	
Coarse granularity of concrete of the spanning structure	0	0	1	$Z_{1.6}$
The protective layer above the auxiliary rebar for the installation of the main rebar is too small	0	0	1	$Z_{1.7}$
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) Good quality of concrete	X	X	X	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) poor quality of concrete	X	X	X	
The protective layer of the main rebar on the underside of the structure is too small (1.0 to 2.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1,0 to 2,9cm) Poor concrete quality	0	0	3	$Z_{1.8}$
The protective layer of the main rebar on the underside of the structure is too small (below 1.0cm)	0	0	3	$Z_{1.9}$
The carbonate front reached the main rebar	0	0	3	$Z_{1.10}$
Visible main rebar on the underside of the structure, the reinforcement is lightly corroded (without significant reduction of the cross section)	1	0	3	$Z_{1.11}$
The main rebar of the spanning structure lies in the area of carbonization and is slightly corroded (it does not apply to the prefabrication reinforcement)	1	0	3	$Z_{1.12}$
Visible main rebar on the underside of the spanning structure, corroded rebar (there is a decrease in the cross-section)	2	0	3	$Z_{1.13}$
Blooming (water traces) in the area of heavily corroded main reinforcement on the underside of the spanning structure (advanced reductions in cross-section)	x	x	x	
Blooming in the area of heavily corroded main reinforcement on the underside of the spanning structure (the main reinforcement is partly excluded from the load)	x	x	x	
Partial moisture penetration	0	0	2	$Z_{1.14}$
Penetration of moisture on large surfaces	0	0	3	$Z_{1.15}$
Description of damage / defect	**S**	**V**	**D**	
Bridges, cracks in concrete / reinforced concrete / pre-stressed structure				
Dependencies: type of construction = bridge, basic building element = structure , material of the structure = concrete, damage = cracks				
Surface cracks outside the humidification area (widths) of ≤ 0.1mm in reinforced concrete or prestressed structure (eg cracks from shrinkage)	x	x	x	
Cracks outside the area of humidification (shrinkage) width of 0.1 - <0.2mm in reinforced concrete- or prestressed structure	x	x	x	
Cracks width 0.1 - <0.2mm in the area of humidification (shrinkage) for RC structure	X	X	X	

Surface cracks in the humidification area (widths) 0.2 -≤ 0.4 mm in the RC structure	0	0	2	$Z_{1.16}$
Parallel cracks with prestressing of a width of 0.2 -≤ 0.4mm in the area of humidification (squeezing) in the prestressed structure	x	x	x	
Shrinkage widths> 0.4mm in the area of humidification (shrinkage) for RC structure	X	X	X	
Cracks with a width of> 0.4mm with a prestressed structure (not in the cable extension area)	X	X	X	
Cracks with a width of <0.2mm with prestressed structure (in cable extension)	X	X	X	
Cracks with a width of 0.2 - ≤ 0.4mm at the front of the structure (in the area of cable extension)	X	X	X	
Cracks with a width of> 0.4mm with a prestressed structure (in the cable extension area)	X	X	X	
Cracks> 0.4mm under load	x	x	x	

Summary Group 1

$Z_{1.1}$ = 1.0	$\Delta Z_{1.1}$ = 0	$Z_{1.1}$ = 1.0+0=1.0
$Z_{1.2}$ = 1.0	$\Delta Z_{1.2}$ = +0.1	$Z_{1.2}$ = 1.0+0.1=1.1
$Z_{1.3}$ = 1.1	$\Delta Z_{1.3}$ = +0.1	$Z_{1.3}$ = 1.1+0.1= 1.2
$Z_{1.4}$ = 1.1	$\Delta Z_{1.4}$ = -0.1	$Z_{1.4}$ = 1.1-0.1= 1
$Z_{1.5}$ = 1.0	$\Delta Z_{1.5}$ = 0	$Z_{1.5}$ = 1.0+0= 1.0
$Z_{1.6}$ = 1.1	$\Delta Z_{1.6}$ = 0	$Z_{1.6}$ = 1.1+0= 1.1
$Z_{1.7}$ = 1.1	$\Delta Z_{1.7}$ = -0.1	$Z_{1.7}$ = 1.1-0.1= 1.0
$Z_{1.8}$ = 2.5	$\Delta Z_{1.8}$ = 0	$Z_{1.8}$ = 2.5+0= 2.5
$Z_{1.9}$ = 2.5	$\Delta Z_{1.9}$ = -0.1	$Z_{1.9}$ = 2.5-0.1= 2.4
$Z_{1.10}$ = 2.5	$\Delta Z_{1.10}$ = 0	$Z_{1.10}$ = 2.5+0= 2.5
$Z_{1.11}$ = 2.7	$\Delta Z_{1.11}$ = +0.1	$Z_{1.11}$ = 2.7+0.1= 2.8
$Z_{1.12}$ = 2.7	$\Delta Z_{1.12}$ = 0	$Z_{1.12}$ = 2.7+0= 2.7
$Z_{1.13}$ = 2.8	$\Delta Z_{1.13}$ = 0	$Z_{1.13}$ = 2.8+0=2.8
$Z_{1.14}$ = 2.0	$\Delta Z_{1.14}$ = 0	$Z_{1.14}$ = 2.0+0= 2.0
$Z_{1.15}$ = 2.5	$\Delta Z_{1.15}$ = +0.1	$Z_{1.15}$ = 2.5+0.1= 2.6
$Z_{1.16}$ = 2.0	$\Delta Z_{1.16}$ = 0	$Z_{1.16}$ =2.0+0= 2.0
Sum group 1		**29.7**

Group 2 Substructure

Bridges, substructure	S	V	D	
Dependencies: type of construction = bridge, basic building element = substructure				
Graffiti on visible surfaces	0	0	0	$Z_{1.17}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{1.18}$
Less wear of the protective layer	0	0	1	$Z_{1.19}$
Less rinses in the area of water flows	0	0	1	$Z_{1.20}$
Cleaning of the bearing bench (moldings or other)	x	x	x	
Invalidation of the benches (bird droppings or other)	x	x	x	
Remains of the formwork that press the construction	0	0	2	$Z_{1.21}$
Cleaning the bearing bench with accumulated moisture	x	x	x	

Formwork material (polystyrene) on the connection with the structure has not been removed	1	0	2	$Z_{1.22}$
Less dropping of stone linings	x	x	x	
The installation cover is not correct / damaged	x	x	x	
Cracking stone wall	x	x	x	
Partial moisture on stone wall / reinforced concrete	0	0	2	$Z_{1.23}$
Moisture on large surfaces of stone wall / reinforced concrete	0	0	3	$Z_{1.24}$
Bridges, cracks in concrete- / RC substructure				
Dependencies: type of construction = bridge, basic building element = substructure, damage = cracks				
Dry cracks outside the humidification (spinning) area <0.2mm (no reaction sulfuric acid - RSK)	x	x	x	
Surface cracks in the area of humidification (squeezing), cracks can run water, width <0.2mm (without RSK) REACTION SULFURIC ACID	x	x	x	
Dry cracks outside the humidification area (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, 0.2 -≤ 0.4mm wide (without RSK)	x	x	x	
Dry cracks outside the area of humidification (shrinkage) of width> 0,4mm (without RSK)	0	0	2	$Z_{1.25}$
Surface cracks in the area of humidification (shrinkage) of width> 0.4mm (without RSK)	x	x	x	
Description of damage / defect	S	V	D	
Cracks in the area of humidification (cracks), cracks can run water, width> 0,4mm, unarmed concrete (without RSK)	x	x	x	
Cracks in the area of humidification (sprinkling), cracks can run water, width> 0.4mm, RC bottom structure (without RSK)	0	0	3	$Z_{1.26}$

Summary Group 2

$Z_{1.17} = 1.0$	$\Delta Z_{1.17} = 0$	$Z_{1.17} = 1.0+0= 1.0$
$Z_{1.18} = 1.0$	$\Delta Z_{1.18} = +0.1$	$Z_{1.10} = 1.0+0.1= 1.1$
$Z_{1.19} = 1.1$	$\Delta Z_{1.19} = +0.1$	$Z_{119} = 1.1+0.1= 1.2$
$Z_{1.20} = 1.1$	$\Delta Z_{1.20} = 0$	$Z_{1.20} = 1.1+0= 1.1$
$Z_{1.21} = 2.0$	$\Delta Z_{1.21} = 0$	$Z_{1.21} = 2.0+0= 2.0$
$Z_{1.22} = 2.2$	$\Delta Z_{1.22} = 0$	$Z_{1.22} = 2.2+0= 2.2$
$Z_{1.23} = 2.0$	$\Delta Z_{1.23} = 0$	$Z_{1.23} = 2.0+0= 2.0$
$Z_{1.24} = 2.5$	$\Delta Z_{1.24} = 0$	$Z_{1.24} = 2.5+0= 2.5$
$Z_{1.25} =2.0$	$\Delta Z_{1.25} = -0.1$	$Z_{1.25} = 2.0-0.1 = 1.9$
$Z_{1.26} = 2.5$	$\Delta Z_{1.26} = 0$	$Z_{1.26} = 2.5+0= 2.5$
Sum group 2		**17.5**

Group 9: Transition devices

Transition devices (joints)	S	V	D	
Dependencies: structural element = transitional device				
Contaminated transitional device (moving still possible)	x	x	x	

A highly Contaminated transient device (scrolling limited)	x	x	x	
Loosening the fixing of the profiles in the carpet of the structures, the profile is still held	x	x	x	
Loosening the fixing of the profiles in the carpet construction, the profile loosened	x	x	x	
Rubber profile dropped or multiple damaged	x	x	x	
Asphalt crossings, thin asphalt mass (rough, small open cracks)	0	0	0	$Z_{1.27}$
The asphalt crossing cracked and depressed	0	1	2	$Z_{1.28}$
The transient device is missing, the spanning structure is cracked at the ends	x	x	x	
The transient device is missing, the spanning structure at the ends is cracked and shrugged	x	x	x	

Summary Group 9

$Z_{1.27} = 1.0$	$\Delta Z_{1.27} = +0.1$	$Z_{1.27} = 1.0+0.1= 1.1$
$Z_{1.28} = 2.1$	$\Delta Z_{1.28} = +0.1$	$Z_{1.28} = 2.1+0.1= 2.2$
Sum group 9		**3.3**

Group: 13 Fence

Protective means	S	V	D	
Fence				
Dependencies: element of construction = protective means, fence				
The compound fence - construction of the defect	0	0	1	$Z_{1.29}$
There is no fence, there are bumpers at> 50km / h, no pedestrian traffic is planned	X	X	X	
There are no fences or segments of the fence outside the end of the bridge, small building, no pedestrian traffic is foreseen (temporary hazard sign exists)	X	X	X	
There are no fences or segments of the fence outside the end of the bridge, pedestrian traffic planned	X	X	X	
Missing wire in the handrail of the fence, the length of the building is <20m	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, there is a bumper	X	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, the bumper is missing	x	x	x	
Fence height is not in accordance with regulations (difference ≤ 5cm)	x	x	x	
Fence height is not in accordance with regulations (difference 5 - 10cm)	x	x	x	
Fence height is not in accordance with regulations (difference > 10cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference ≤ 2cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference > 2cm)	0	2	0	$Z_{1.30}$
Missing individual fence filling rods	x	x	x	
Missing more consecutive fence filling rods	x	x	x	
Guardrail				
Dependencies: structural element = protection agent, bumper				
Bumper height is not in accordance with regulations (difference ≤ 3cm)	x	x	x	
Bumper height is not in accordance with regulations (difference > 3cm)	x	x	x	
The bumper is partially deformed	x	x	x	
a fence and a bumper are missing	x	x	x	

Weak anchor fasteners of the bumper / fence	x	x	x	
There is a lack of anchoring of protective devices on the length of the move	0	3	0	$Z_{1.31}$
Corrosion of protective agents				
Dependencies: structural element = protective agents, damage = surface, metal				
Insufficient thickness of the protective layer	0	0	2	$Z_{1.32}$
Local scattering (breaking) of the protective layer	x	x	x	
Scattering (breaking) of the protective layer on larger surfaces	x	x	x	
Local start of corrosion	0	0	1	$Z_{1.33}$
Corrosion of large surfaces	0	0	2	$Z_{1.34}$
The corrosion of individual support elements of the protecting agents	1	1	2	$Z_{1.35}$
The corrosion of more consecutive support elements of the protecting agents	x	x	x	

Summary Group 13

$Z_{1.29}$ = 1.1	$\Delta Z_{1.29}$ = +0.1	$Z_{1.29}$ = 1.1+0.1= 1.2
$Z_{1.30}$ =2.1	$\Delta Z_{1.30}$ = +0.1	$Z_{1.30}$ = 2.1+0.1= 2.2
$Z_{1.31}$= 2.5	$\Delta Z_{1.31}$= -0.1	$Z_{1.31}$= 2.5-0.1= 2.4
$Z_{1.32}$= 2.0	$\Delta Z_{1.32}$= +0	$Z_{1.32}$= 2.0+0= 2.0
$Z_{1.33}$ = 1.1	$\Delta Z_{1.33}$= -0.1	$Z_{1.33}$ = 1.1-0.1= 1
$Z_{1.34}$ = 2.0	$\Delta Z_{1.34}$= 0	$Z_{1.34}$ = 2.0+0= 2.0
$Z_{1.35}$ = 2.3	$\Delta Z_{1.35}$= +0.1	$Z_{1.35}$ = 2.3+0.1= 2.4
Sum group 13		**13.2**

Group 11: Road surface

Road surface	S	V	D	
Dependencies: structural element = road surface				
pavement				
Subsidence of the pavement behind the abutments (≤ 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm)	0	2	1	$Z_{1.36}$
Subsidence of the pavement behind the abutments (> 2cm), there are warning signs	x	x	x	
Drainage does not work, the risk of drifting	0	2	0	$Z_{1.37}$
Paving grooves / indentations, depth <1cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm	0	2	0	$Z_{1.38}$
Pulley grooves / indentations, depth 1 - 3cm, there are warning signs	x	x	X	
Pulley grooves / indentations, depth> 3cm	X	x	x	
Paving grooves / indentations, depth> 3cm, there are warning signs	x	x	X	
Bubbles, heights of ≤ 2cm	X	x	x	
Bubbles, height 2 - 5cm	x	x	X	
Bubbles, height 2 - 5cm, there are warning signs	X	x	x	
Bubbles, height > 5cm	X	x	X	
Bubbles, height > 5cm, there are warning signs	x	x	x	
Impact hole, depth ≤ 2cm	x	x	X	
Impact hole, depth 2 - 5cm	0	2	0	$Z_{1.39}$
Impact hole, depth 2 - 5cm, there are warning signs	X	x	x	
Impact hole, depth > 5cm	X	x	X	
Description of damages / defections	S	V	D	

Impact hole, depth> 5cm, there are warning signs	x	x	x	
A pedestrian hallway				
Erosion of surface layer <2cm	x	x	x	
Erosion of surface layer ≥2cm	x	x	x	
Erosion of surface layer ≥2cm, there are warning signs	0	1	2	$Z_{1.40}$
The layers break and fall in pieces	x	x	X	
Slipping risk	x	x	x	

Summary Group 11: Road Surface

$Z_{1.36}$ =2.1	$\Delta Z_{1.36}$= 0	$Z_{1.36}$ = 2.1+0= 2.1
$Z_{1.37}$ =2.0	$\Delta Z_{1.37}$= 0	$Z_{1.37}$ = 2.0+0= 2.0
$Z_{1.38}$ =2.0	$\Delta Z_{1.38}$= 0	$Z_{1.38}$ = 2.0+0= 2.0
$Z_{1.39}$ = 2.0	$\Delta Z_{1.39}$= 0	$Z_{1.39}$ = 2.0+0= 2.0
$Z_{1.40}$ = 2.1	$\Delta Z_{1.40}$= +0.1	$Z_{1.40}$ = 2.1+0.1= 2.1
Sum group 11		**10.3**

Group 14: Other

Signs	S	V	D	
Dependencies: structural element = Signs				
Missing building designation number	X	X	X	
The load limit sign is missing / incorrect S = 2 - 4, V = 2 - 4	X	X	X	
Drainage of the bridge				
Dependencies: element of construction = equipment, drainage of the bridge				
The fastening of the drainage pipe was corroded	1	0	2	$Z_{1.41}$
The fastening parts are missing, outside the traffic area	x	x	x	
The fastening parts are missing, above the traffic surface, V = 1 - 3	x	x	x	
Water leaks from the pipe (above the field)	x	x	x	
Water leaks from the pipe (above the parts of the structure)	0	0	2	$Z_{1.42}$
Water leaks from the pipe (above the traffic surfaces)	0	1	2	$Z_{1.43}$
Less corrosion damage on drainage pipes	0	0	1	$Z_{1.44}$
Significant corrosion damage on drainage pipes	x	x	X	
Missing dilatation of the drainage pipes at the transition of the structure / field	x	x	x	
Rain grid / clogged pipe	0	2	1	$Z_{1.45}$
In the raining grid there is a missing catcher of a garbage (pot)	X	x	x	
The drainage grid/ cleaning hole in the hinge plate is not secured	x	x	x	
The drainage grid is broken	x	x	x	
Inspection agents (inspection tools)				
Dependencies: element of construction = equipment, inspection agents				
Cracks on the rail of the inspected vehicle (risk of falling from height)	X	X	X	
Padlock is missing, third parties have unobstructed access to the building site	X	X	X	
Ladders, The distance between rungs is too large (> 280mm)	X	X	X	
Ladders, rungs too close to the building (<150mm)	X	X	X	
Ladder, The distance between the end rungs and the working surface too large (> 100mm)	X	X	X	

Ladders, according to the regulations, the necessary back protection is missing	X	X	X	
Corrosion of inspection agents				
Dependencies: element of construction = equipment, inspection means, damage = surface, metal				
Thickness of anti-corrosion protection too small	x	x	x	
Locally blown (cracked) anti-corrosion protection	x	X	X	
Anti-corrosive protection on large surfaces is blooming	X	X	X	
The beginning of corrosion - locally	x	x	x	
The beginning of corrosion - a large surface	x	x	x	
Corrosion of individual carriers	x	x	x	
Corrosion of several carriers in a row	x	x	x	
(Tools for protection) Means of protection from invading birds				
Dependencies: structural element = protective agents, protection from birds				
protection agents of invading birds are missing / damaged	X	X	X	
Overview of the building site				
Dependencies: damage = review				
The view of the whole building is not possible? a fictitious assessment of the situation is issued	x	x	x	
The building is very overgrown, only partial inspection possible	0	0	2	$Z_{1.46}$

Summary Group 14 Other

$Z_{1.41}$ = 2.2	$\Delta Z_{1.41}$= 0	$Z_{1.41}$ = 2.2+0= 2.2
$Z_{1.42}$ = 2.0	$\Delta Z_{1.42}$= 0	$Z_{1.42}$ = 2.0+0= 2.0
$Z_{1.43}$ = 2.1	$\Delta Z_{1.43}$= -0.1	$Z_{1.43}$ = 2.1-0.1= 2
$Z_{1.44}$ = 1.1	$\Delta Z_{1.44}$= -0.1	$Z_{1.44}$ = 1.1-0.1= 1
$Z_{1.45}$ =2.1	$\Delta Z_{1.45}$= +0.1	$Z_{1.45}$ = 2.1+0.1= 2.2
$Z_{1.46}$ =2.0	$\Delta Z_{1.46}$=0	$Z_{1.46}$ = 2.0+0= 2.0
Sum group 14		**11.4**

1.2. LEVEL 2: MAXIMUM DAMAGE

NO.	Group1	Group2	Group9	Group13	Group11	Group14
	29.7	17.5	3.3	13.2	10.3	11.4

1.3. LEVEL 3:

Group	Z	ΔZ	Z_{BG}
Group 1	$Z_{1.11}$ = 2.8	ΔZ_2 = 0	Z_{BG}= 2.8 + 0= 2.8
	$Z_{1.13}$ = 2.8	ΔZ_2 = -0.1	

Group 2	$Z_{1.24} = 2.5$	$\Delta Z_2 = 0$	$Z_{BG}= 2.5 + 0= 2.5$
	$Z_{1.26} = 2.5$	$\Delta Z_2 = -0.1$	
Group 9	$Z_{1.28} = 2.2$	$\Delta Z_2 = +0.1$	$Z_{BG}= 2.2 + 0.1= 2.3$
Group 11	$Z_{1.36} = 2.1$	$\Delta Z_2= -0.1$	$Z_{BG}= 2.1 + 0= 2.1$
	$Z_{1.40}= 2.1$	$\Delta Z_2= 0$	
Group 13	$Z_{1.31}= 2.4$	$\Delta Z_2 = -0.1$	$Z_{BG}= 2.4 + 0= 2.4$
	$Z_{1.35}= 2.4$	$\Delta Z_2 = 0$	
Group 14	$Z_{1.41} = 2.2$	$\Delta Z_2 = -0.1$	$Z_{BG}= 2.2 + 0.1= 2.3$
	$Z_{1.45} = 2.2$	$\Delta Z_2 = +0.1$	

$Z_{ges}= 2.8$ $\Delta Z_3 = 0$ (GROUP 1 THE MAXIMUM Z_{BG})

Sufficient condition of the bridge structure

The stability of the structure is ensured.

The traffic safety of the structure may be impaired.

The stability and/or durability of at least one component group can be impaired.

The durability of the structure may be affected. A spread of damage or consequential damage to the structure, which in the medium term leads to significant impairments to stability and/or traffic safety or increased wear, is then to be expected.

Ongoing maintenance required.

Short-term repair required.

Measures to eliminate damage or warnings to maintain road safety may be necessary at short term.

2. SOUK ATHULATHA 2 BRIDGE

2.1. LEVEL 1: REGULAR BRIDGE INSPECTION

Group 1: Superstructure

Description of damage and defects	S	V	D	
Bridges, concrete / reinforced concrete / pre-stressed construction structures				
Dependencies: type of construction = bridge, basic building element = structure construction, material of the spanning structure = concrete				
Graffiti on visible surfaces	0	0	0	$Z_{2.1}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{2.2}$
Less wear of the protective layer	0	0	1	$Z_{2.3}$
The rust on the lower sides of the construction	0	0	1	$Z_{2.4}$
Pollution of internal passages of the building (remains of the formwork or other)	0	0	0	$Z_{2.5}$
Pollution of internal passages of building (bird feces or other)	x	x	x	x
Coarse granularity of concrete of the spanning structure	0	0	1	$Z_{2.6}$
The protective layer above the auxiliary rebar for the installation of the main rebar is too small	0	0	1	$Z_{2.7}$
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) Good quality of concrete	x	x	x	x
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) poor quality of concrete	x	x	x	x
The protective layer of the main rebar on the underside of the structure is too small (1.0 to 2.9cm) Good quality of concrete	x	x	x	x
The protective layer of the main rebar on the underside of the structure is too small (1,0 to 2,9cm) Poor concrete quality	0	0	3	$Z_{2.8}$
The protective layer of the main rebar on the underside of the structure is too small (below 1.0cm)	0	0	3	$Z_{2.9}$
The carbonate front reached the main rebar	0	0	3	$Z_{2.10}$
Visible main rebar on the underside of the structure, the reinforcement is lightly corroded (without significant reduction of the cross section)	1	0	3	$Z_{2.11}$
The main rebar of the spanning structure lies in the area of carbonization and is slightly corroded (it does not apply to the prefabrication reinforcement)	1	0	3	$Z_{2.12}$
Visible main rebar on the underside of the spanning structure, corroded rebar (there is a decrease in the cross-section)	2	0	3	$Z_{2.13}$
Blooming (water traces) in the area of heavily corroded main reinforcement on the underside of the spanning structure (advanced reductions in cross-section)	x	x	x	x
Blooming in the area of heavily corroded main reinforcement on the underside of the spanning structure (the main reinforcement is partly excluded from the load)	x	x	x	x
Partial moisture penetration	0	0	2	$Z_{2.14}$
Penetration of moisture on large surfaces	0	0	3	$Z_{2.15}$
Description of damage / defect	S	V	D	
Bridges, cracks in concrete / reinforced concrete / pre-stressed structure				
Dependencies: type of construction = bridge, basic building element = structure , material of the structure = concrete, damage = cracks				
Surface cracks outside the humidification area (widths) of ≤ 0.1mm in reinforced concrete or prestressed structure (eg cracks from shrinkage)	x	x	x	x
Cracks outside the area of humidification (shrinkage) width of 0.1 - <0.2mm in reinforced concrete- or prestressed structure	X	x	x	x
Cracks width 0.1 - <0.2mm in the area of humidification (shrinkage) for RC structure	x	x	x	x
Surface cracks in the humidification area (widths) 0.2 -≤ 0.4 mm in the RC structure	0	0	2	$Z_{2.16}$

Parallel cracks with prestressing of a width of 0.2 -≤ 0.4mm in the area of humidification (squeezing) in the prestressed structure	x	x	x	x
Shrinkage widths> 0.4mm in the area of humidification (shrinkage) for RC structure	X	x	x	x
Cracks with a width of> 0.4mm with a prestressed structure (not in the cable extension area)	x	x	x	x
Cracks with a width of <0.2mm with prestressed structure (in cable extension)	X	x	x	x
Cracks with a width of 0.2 - ≤ 0.4mm at the front of the structure (in the area of cable extension)	x	x	x	x
Cracks with a width of> 0.4mm with a prestressed structure (in the cable extension area)	X	x	x	x
Cracks> 0.4mm under load	x	x	x	x

Summary Group 1

$Z_{2.1}$ = 1.0	$\Delta Z_{2.1}$ = +0.1	$Z_{2.1}$ = 1.0+0.1=1.1
$Z_{2.2}$ = 1.0	$\Delta Z_{2.2}$ = +0.1	$Z_{2.2}$ = 1.0+0.1=1.1
$Z_{2.3}$ = 1.1	$\Delta Z_{2.3}$ = 0	$Z_{2.3}$ = 1.1+0= 1.1
$Z_{2.4}$ = 1.1	$\Delta Z_{2.4}$ = 0	$Z_{2.4}$ = 1.1+0= 1.1
$Z_{2.5}$ = 1.0	$\Delta Z_{2.5}$ = 0	$Z_{2.5}$ = 1.0+0= 1.0
$Z_{2.6}$ = 1.1	$\Delta Z_{2.6}$ = +0.1	$Z_{2.6}$ = 1.1+0.1= 1.2
$Z_{2.7}$ = 1.1	$\Delta Z_{2.7}$ = 0	$Z_{2.7}$ = 1.1+0= 1.1
$Z_{2.8}$ = 2.5	$\Delta Z_{2.8}$ = 0	$Z_{2.8}$ = 2.5+0= 2.5
$Z_{2.9}$ = 2.5	$\Delta Z_{2.9}$ = -0.1	$Z_{2.9}$ = 2.5-0.1= 2.4
$Z_{2.10}$ = 2.5	$\Delta Z_{2.10}$ = 0	$Z_{2.10}$ = 2.5+0= 2.5
$Z_{2.11}$ = 2.7	$\Delta Z_{2.11}$ = +0.1	$Z_{2.11}$ = 2.7+0.1= 2.8
$Z_{2.12}$ = 2.7	$\Delta Z_{2.12}$ = 0	$Z_{2.12}$ = 2.7+0= 2.7
$Z_{2.13}$ = 2.8	$\Delta Z_{2.13}$ = 0	$Z_{2.13}$ = 2.8+0=2.8
$Z_{2.14}$ = 2.0	$\Delta Z_{2.14}$ = 0	$Z_{2.14}$ = 2.0+0= 2.0
$Z_{2.15}$ = 2.5	$\Delta Z_{2.15}$ = +0.1	$Z_{2.15}$ = 2.5+0.1= 2.6
$Z_{2.16}$ = 2.0	$\Delta Z_{2.16}$ = 0	$Z_{2.16}$ =2.0+0= 2.0
Sum group 1		**30**

Group 2: Substructure

Bridges, substructure	S	V	D	
Dependencies: type of construction = bridge, basic building element = substructure				
Graffiti on visible surfaces	0	0	0	$Z_{2.17}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{2.18}$
Less wear of the protective layer	0	0	1	$Z_{2.19}$
Less rinses in the area of water flows	0	0	1	$Z_{2.20}$
Cleaning of the bearing bench (moldings or other)	x	x	x	x
Invalidation of the benches (bird droppings or other)	x	x	x	x
Remains of the formwork that press the construction	0	0	2	$Z_{2.21}$
Cleaning the bearing bench with accumulated moisture	x	x	x	x
Formwork material (polystyrene) on the connection with the structure has not been removed	1	0	2	$Z_{2.22}$
Less dropping of stone linings	x	x	x	x

The installation cover is not correct / damaged	x	x	x	
Cracking stone wall	x	x	x	
Partial moisture on stone wall / reinforced concrete	0	0	2	$Z_{2.23}$
Moisture on large surfaces of stone wall / reinforced concrete	0	0	3	$Z_{2.24}$
Bridges, cracks in concrete- / RC substructure				
Dependencies: type of construction = bridge, basic building element = substructure, damage = cracks				
Dry cracks outside the humidification (spinning) area <0.2mm (no reaction sulfuric acid - RSK)	x	x	x	
Surface cracks in the area of humidification (squeezing), cracks can run water, width <0.2mm (without RSK) REACTION SULFURIC ACID	x	x	x	
Dry cracks outside the humidification area (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, 0.2 -≤ 0.4mm wide (without RSK)	x	x	x	
Dry cracks outside the area of humidification (shrinkage) of width> 0,4mm (without RSK)	0	0	2	$Z_{2.25}$
Surface cracks in the area of humidification (shrinkage) of width> 0.4mm (without RSK)	x	x	x	
Description of damage / defect	S	V	D	
Cracks in the area of humidification (cracks), cracks can run water, width> 0,4mm, unarmed concrete (without RSK)	x	x	x	
Cracks in the area of humidification (sprinkling), cracks can run water, width> 0.4mm, RC bottom structure (without RSK)	0	0	3	$Z_{2.26}$

Summary Group 2

$Z_{2.17} = 1.0$	$\Delta Z_{2.17} = 0$	$Z_{2.17} = 1.0+0= 1.0$
$Z_{2.18} = 1.0$	$\Delta Z_{2.18} = +0.1$	$Z_{2.18} = 1.0+0.1= 1.1$
$Z_{2.19} = 1.1$	$\Delta Z_{2.19} = +0.1$	$Z_{2.19} = 1.1+0.1= 1.2$
$Z_{2.20} = 1.1$	$\Delta Z_{2.20} = 0$	$Z_{2.20} = 1.1+0= 1.1$
$Z_{2.21} = 2.0$	$\Delta Z_{2.21} = 0$	$Z_{2.21} = 2.0+0= 2.0$
$Z_{2.22} = 2.2$	$\Delta Z_{2.22} = 0$	$Z_{2.22} = 2.2+0= 2.2$
$Z_{2.23} = 2.0$	$\Delta Z_{2.23} = 0$	$Z_{2.23} = 2.0+0= 2.0$
$Z_{2.24} = 2.5$	$\Delta Z_{2.24} = 0$	$Z_{2.24} = 2.5+0= 2.5$
$Z_{2.25} =2.0$	$\Delta Z_{2.25} = -0.1$	$Z_{2.25} = 2.0-0.1 = 1.9$
$Z_{2.26} = 2.5$	$\Delta Z_{2.26} = 0$	$Z_{2.26} = 2.5+0= 2.5$
Sum group 2		**17.5**

Group 9: Transition devices

	S	V	D	
Transition devices (joints)				
Dependencies: structural element = transitional device				
Contaminated transitional device (moving still possible)				
A highly Contaminated transient device (scrolling limited)				
Loosening the fixing of the profiles in the carpet of the structures, the profile is still held				
Loosening the fixing of the profiles in the carpet construction, the profile loosened				
Rubber profile dropped or multiple damaged				

Asphalt crossings, thin asphalt mass (rough, small open cracks)	0	0	0	$Z_{2.27}$
The asphalt crossing cracked and depressed	0	1	2	$Z_{2.28}$
The transient device is missing, the spanning structure is cracked at the ends	x	x	x	
The transient device is missing, the spanning structure at the ends is cracked and shrugged	x	x	x	

Summary Group 9

$Z_{2.27}$ = 1.0	$\Delta Z_{2.27}$ = +0.1	$Z_{2.27}$ = 1.0+0.1= 1.1
$Z_{2.28}$ = 2.1	$\Delta Z_{2.28}$ = +0.1	$Z_{2.28}$ = 2.1+0.1= 2.2
Sum group 9		**3.3**

Group 13: Fence

Protective means	S	V	D	
Fence				
Dependencies: element of construction = protective means, fence				
The compound fence - construction of the defect	0	0	1	$Z_{2.29}$
There is no fence, there are bumpers at> 50km / h, no pedestrian traffic is planned	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, small building, no pedestrian traffic is foreseen (temporary hazard sign exists)	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, pedestrian traffic planned	x	x	x	
Missing wire in the handrail of the fence, the length of the building is <20m	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, there is a bumper	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, the bumper is missing	x	x	x	
Fence height is not in accordance with regulations (difference ≤ 5cm)	x	x	x	
Fence height is not in accordance with regulations (difference 5 - 10cm)	x	x	x	
Fence height is not in accordance with regulations (difference > 10cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference ≤ 2cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference > 2cm)	0	2	0	$Z_{2.30}$
Missing individual fence filling rods	x	x	x	
Missing more consecutive fence filling rods	x	x	x	
Guardrail	x	x	x	
Dependencies: structural element = protection agent, bumper	x	x	x	
Bumper height is not in accordance with regulations (difference ≤ 3cm)	x	x	x	
Bumper height is not in accordance with regulations (difference > 3cm)	x	x	x	
The bumper is partially deformed	x	x	x	
a fence and a bumper are missing	x	x	x	
Weak anchor fasteners of the bumper / fence	x	x	x	
There is a lack of anchoring of protective devices on the length of the move	0	3	0	$Z_{2.31}$
Corrosion of protective agents				
Dependencies: structural element = protective agents, damage = surface, metal				
Insufficient thickness of the protective layer	0	0	2	$Z_{2.32}$

Local scattering (breaking) of the protective layer	x	x	x	
Scattering (breaking) of the protective layer on larger surfaces	x	x	x	
Local start of corrosion	0	0	1	$Z_{2.33}$
Corrosion of large surfaces	0	0	2	$Z_{2.34}$
The corrosion of individual support elements of the protecting agents	1	1	2	$Z_{2.35}$
The corrosion of more consecutive support elements of the protecting agents				

Summary Group 13

$Z_{2.29}$ = 1.1	$\Delta Z_{2.29}$ = +0.1	$Z_{2.29}$ = 1.1+0.1= 1.2
$Z_{2.30}$ =2.1	$\Delta Z_{2.30}$ = +0.1	$Z_{2.30}$ = 2.1+0.1= 2.2
$Z_{2.31}$= 2.5	$\Delta Z_{2.31}$= -0.1	$Z_{2.31}$= 2.5-0.1= 2.4
$Z_{2.32}$= 2.0	$\Delta Z_{2.32}$= +0	$Z_{2.32}$= 2.0+0= 2.0
$Z_{2.33}$ = 1.1	$\Delta Z_{2.33}$= -0.1	$Z_{2.33}$ = 1.1-0.1= 1
$Z_{2.34}$ = 2.0	$\Delta Z_{2.34}$= 0	$Z_{2.34}$ = 2.0+0= 2.0
$Z_{2.35}$ = 2.3	$\Delta Z_{2.35}$= +0.1	$Z_{2.35}$ = 2.3+0.1= 2.4
Sum group 13		**13.2**

Group 11: Road surface

Road surface	S	V	D	
Dependencies: structural element = useful surface				
pavement				
Subsidence of the pavement behind the abutments (≤ 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm)	0	2	1	$Z_{2.36}$
Subsidence of the pavement behind the abutments (> 2cm), there are warning signs	x	x	x	
Drainage does not work, the risk of drifting	0	2	0	$Z_{2.37}$
Paving grooves / indentations, depth <1cm	x	x	X	
Pulley grooves / indentations, depth 1 - 3cm	0	2	0	$Z_{2.38}$
Pulley grooves / indentations, depth 1 - 3cm, there are warning signs	x	x	x	
Pulley grooves / indentations, depth> 3cm	x	x	x	
Paving grooves / indentations, depth> 3cm, there are warning signs	x	x	x	
Bubbles, heights of ≤ 2cm	x	x	x	
Bubbles, height 2 - 5cm	x	x	x	
Bubbles, height 2 - 5cm, there are warning signs	x	x	x	
Bubbles, height > 5cm	x	x	x	
Bubbles, height > 5cm, there are warning signs	x	x	x	
Impact hole, depth ≤ 2cm	x	x	x	
Impact hole, depth 2 - 5cm	0	2	0	$Z_{2.39}$
Impact hole, depth 2 - 5cm, there are warning signs	x	x	x	
Impact hole, depth > 5cm	x	x	x	
Description of damages / defections	S	V	D	
Impact hole, depth> 5cm, there are warning signs	x	x	x	
A pedestrian hallway				
Erosion of surface layer <2cm				
Erosion of surface layer ≥2cm				
Erosion of surface layer ≥2cm, there are warning signs	0	1	2	$Z_{2.40}$
The layers break and fall in pieces	x	x	x	

Slipping risk	x	x	x	

Group 11: Road surface

$Z_{2.36}$ =2.1	$\Delta Z_{2.36}$= 0	$Z_{2.36}$ = 2.1+0= 2.1
$Z_{2.37}$ =2.0	$\Delta Z_{2.37}$= 0	$Z_{2.37}$ = 2.0+0= 2.0
$Z_{2.38}$ =2.0	$\Delta Z_{2.38}$= 0	$Z_{2.38}$ = 2.0+0= 2.0
$Z_{2.39}$ = 2.0	$\Delta Z_{2.39}$= 0	$Z_{2.39}$ = 2.0+0= 2.0
$Z_{2.40}$ = 2.1	$\Delta Z_{2.40}$= +0.1	$Z_{2.40}$ = 2.1+0.1= 2.1
Sum group 11		**10.3**

Group 14: Other

Signs	S	V	D	
Dependencies: structural element = Signs				
Missing building designation number	x	x	x	
The load limit sign is missing / incorrect S = 2 - 4, V = 2 - 4	x	x	x	
Drainage of the bridge				
Dependencies: element of construction = equipment, drainage of the bridge				
The fastening of the drainage pipe was corroded	1	0	2	$Z_{2.41}$
The fastening parts are missing, outside the traffic area	x	x	x	
The fastening parts are missing, above the traffic surface, V = 1 - 3	x	x	x	
Water leaks from the pipe (above the field)	x	x	x	
Water leaks from the pipe (above the parts of the structure)	0	0	2	$Z_{2.42}$
Water leaks from the pipe (above the traffic surfaces)	0	1	2	$Z_{2.43}$
Less corrosion damage on drainage pipes	0	0	1	$Z_{2.44}$
Significant corrosion damage on drainage pipes	x	x	x	
Missing dilatation of the drainage pipes at the transition of the structure / field	x	x	x	
Rain grid / clogged pipe	0	2	1	$Z_{2.45}$
In the raining grid there is a missing catcher of a garbage (pot)	x	x	x	
The drainage grid/ cleaning hole in the hinge plate is not secured	x	x	x	
The drainage grid is broken	x	x	x	
Inspection agents (inspection tools)				
Dependencies: element of construction = equipment, inspection agents				
Cracks on the rail of the inspected vehicle (risk of falling from height)	x	x	x	
Padlock is missing, third parties have unobstructed access to the building site	x	x	x	
Ladders, The distance between rungs is too large (> 280mm)	x	x	x	
Ladders, rungs too close to the building (<150mm)	x	x	x	
Ladder, The distance between the end rungs and the working surface too large (> 100mm)	x	x	x	
Ladders, according to the regulations, the necessary back protection is missing	x	x	x	
Corrosion of inspection agents				
Dependencies: element of construction = equipment, inspection means, damage = surface, metal				

Thickness of anti-corrosion protection too small	x	x	x	
Locally blown (cracked) anti-corrosion protection	x	x	x	
Anti-corrosive protection on large surfaces is blooming	x	x	x	
The beginning of corrosion - locally	x	x	x	
The beginning of corrosion - a large surface	x	x	x	
Corrosion of individual carriers	x	x	x	
Corrosion of several carriers in a row	x	x	x	
(Tools for protection) Means of protection from invading birds				
Dependencies: structural element = protective agents, protection from birds				
protection agents of invading birds are missing / damaged	x	x	x	
Overview of the building site				
Dependencies: damage = review				
The view of the whole building is not possible? a fictitious assessment of the situation is issued	x	x	x	
The building is very overgrown, only partial inspection possible	0	0	2	$Z_{2.46}$

Group 14 Other

$Z_{2.41} = 2.2$	$\Delta Z_{2.41} = 0$	$Z_{2.41} = 2.2+0 = 2.2$
$Z_{2.42} = 2.0$	$\Delta Z_{2.42} = 0$	$Z_{2.42} = 2.0+0 = 2.0$
$Z_{2.43} = 2.1$	$\Delta Z_{2.43} = -0.1$	$Z_{2.43} = 2.1-0.1 = 2$
$Z_{2.44} = 1.1$	$\Delta Z_{2.44} = -0.1$	$Z_{2.44} = 1.1-0.1 = 1$
$Z_{2.45} = 2.1$	$\Delta Z_{2.45} = +0.1$	$Z_{2.45} = 2.1+0.1 = 2.2$
$Z_{2.46} = 2.0$	$\Delta Z_{2.46} = 0$	$Z_{2.46} = 2.0+0 = 2.0$
Sum group 14		**11.4**

2.2. LEVEL 2: MAXIMUM DAMAGE

NO.	Group1	Group2	Group9	Group13	Group11	Group14
	30	17.5	3.3	13.2	10.3	11.4

2.3. LEVEL 3:

Group	Z	ΔZ	**Z_{BG}**

Group 1	$Z_{2.11}$ = 2.8	ΔZ_2 = 0	Z_{BG}= 2.8 + 0= 2.8
	$Z_{2.13}$ = 2.8	ΔZ_2 = -0.1	
Group 2	$Z_{2.24}$ = 2.5	ΔZ_2 = +0.1	Z_{BG}= 2.5 + 0.1= 2.6
	$Z_{2.26}$ = 2.5	ΔZ_2 = 0	
Group 9	$Z_{2.28}$ = 2.2	ΔZ_2 = +0.1	Z_{BG}= 2.2 + 0.1= 2.3
Group 11	$Z_{2.36}$ = 2.1	ΔZ_2= -0.1	Z_{BG}= 2.1 + 0= 2.1
	$Z_{2.40}$= 2.1	ΔZ_2= 0	
Group 13	$Z_{2.31}$= 2.4	ΔZ_2 = -0.1	Z_{BG}= 2.4 + 0= 2.4
	$Z_{2.35}$= 2.4	ΔZ_2 = 0	
Group 14	$Z_{2.41}$ = 2.2	ΔZ_2 = -0.1	Z_{BG}= 2.2 + 0.1= 2.3
	$Z_{2.45}$ = 2.2	ΔZ_2 = +0.1	

Z_{ges}= 2.8 ΔZ_3 = 0 (GROUP 1 THE MAXIMUM Z_{BG})

Sufficient condition of the bridge structure

The stability of the structure is ensured.

The traffic safety of the structure may be impaired.

The stability and/or durability of at least one component group can be impaired.

The durability of the structure may be affected. A spread of damage or consequential damage to the structure, which in the medium term leads to significant impairments to stability and/or traffic safety or increased wear, is then to be expected.

Ongoing maintenance required.

Short-term repair required.

Measures to eliminate damage or warnings to maintain road safety may be necessary at short term.

3. AL SEEKA ROAD BRIDGE

3.1. LEVEL 1: REGULAR BRIDGE INSPECTION

Group 1: Superstructure

Description of damage and defects	S	V	D	
Bridges, concrete / reinforced concrete / pre-stressed construction structures				
Dependencies: type of construction = bridge, basic building element = structure construction, material of the spanning structure = concrete				
Graffiti on visible surfaces	0	0	0	$Z_{3.1}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{3.2}$
Less wear of the protective layer	0	0	1	$Z_{3.3}$
The rust on the lower sides of the construction	0	0	1	$Z_{3.4}$
Pollution of internal passages of the building (remains of the formwork or other)	0	0	0	$Z_{3.5}$
Pollution of internal passages of building (bird feces or other)	x	x	x	x
Coarse granularity of concrete of the spanning structure	0	0	1	$Z_{3.6}$
The protective layer above the auxiliary rebar for the installation of the main rebar is too small	0	0	1	$Z_{3.7}$
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) poor quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1.0 to 2.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1,0 to 2,9cm) Poor concrete quality	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (below 1.0cm)	x	x	x	
The carbonate front reached the main rebar	0	0	3	$Z_{3.8}$
Visible main rebar on the underside of the structure, the reinforcement is lightly corroded (without significant reduction of the cross section)	1	0	3	$Z_{3.9}$
The main rebar of the spanning structure lies in the area of carbonization and is slightly corroded (it does not apply to the prefabrication reinforcement)	1	0	3	$Z_{3.10}$
Visible main rebar on the underside of the spanning structure, corroded rebar (there is a decrease in the cross-section)	2	0	3	$Z_{3.11}$
Blooming (water traces) in the area of heavily corroded main reinforcement on the underside of the spanning structure (advanced reductions in cross-section)	x	x	x	
Blooming in the area of heavily corroded main reinforcement on the underside of the spanning structure (the main reinforcement is partly excluded from the load)	x	x	x	
Partial moisture penetration	0	0	2	$Z_{3.12}$
Penetration of moisture on large surfaces	0	0	3	$Z_{3.13}$
Description of damage / defect	S	V	D	
Bridges, cracks in concrete / reinforced concrete / pre-stressed structure				
Dependencies: type of construction = bridge, basic building element = structure , material of the structure = concrete, damage = cracks				
Surface cracks outside the humidification area (widths) of ≤ 0.1mm in reinforced concrete or prestressed structure (eg cracks from shrinkage)	x	x	x	
Cracks outside the area of humidification (shrinkage) width of 0.1 - <0.2mm in reinforced concrete- or prestressed structure	x	x	x	
Cracks width 0.1 - <0.2mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Surface cracks in the humidification area (widths) 0.2 -≤ 0.4 mm in the RC structure	x	x	x	

Parallel cracks with prestressing of a width of 0.2 -≤ 0.4mm in the area of humidification (squeezing) in the prestressed structure	x	x	x	
Shrinkage widths> 0.4mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (not in the cable extension area)	x	x	x	
Cracks with a width of <0.2mm with prestressed structure (in cable extension)	x	x	x	
Cracks with a width of 0.2 - ≤ 0.4mm at the front of the structure (in the area of cable extension)	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (in the cable extension area)	x	x	x	
Cracks> 0.4mm under load	x	x	x	

Summary Group 1

$Z_{3.1} = 1.0$	$\Delta Z_{3.1} = 0$	$Z_{3.1} = 1.0+0=1.0$
$Z_{3.2} = 1.0$	$\Delta Z_{3.2} = +0.1$	$Z_{3.2} = 1.0+0.1=1.1$
$Z_{3.3} = 1.1$	$\Delta Z_{3.3} = +0.1$	$Z_{3.3} = 1.1+0.1= 1.2$
$Z_{3.4} = 1.1$	$\Delta Z_{3.4} = -0.1$	$Z_{3.4} = 1.1-0.1= 1$
$Z_{3.5} = 1.0$	$\Delta Z_{3.5} = 0$	$Z_{3.5} = 1.0+0= 1.0$
$Z_{3.6} = 1.1$	$\Delta Z_{3.6} = +0.1$	$Z_{3.6} = 1.1+0.1= 1.2$
$Z_{3.7} = 1.1$	$\Delta Z_{3.7} = -0.1$	$Z_{3.7} = 1.1-0.1= 1.0$
$Z_{3.8} = 2.5$	$\Delta Z_{3.8} = 0$	$Z_{3.8} = 2.5+0= 2.5$
$Z_{3.9} = 2.7$	$\Delta Z_{3.9} = +0.1$	$Z_{3.9} = 2.7+0.1= 2.8$
$Z_{3.10} = 2.7$	$\Delta Z_{3.10} = 0$	$Z_{3.10} = 2.7+0= 2.7$
$Z_{3.11} = 2.8$	$\Delta Z_{3.11} = 0$	$Z_{3.11} = 2.8+0=2.8$
$Z_{3.12} = 2.0$	$\Delta Z_{3.12} = 0$	$Z_{3.12} = 2.0+0= 2.0$
$Z_{3.13} = 2.5$	$\Delta Z_{3.13} = +0.1$	$Z_{3.13} = 2.5+0.1= 2.6$
Sum group 1		**22.9**

Group 2: Substructure

Bridges, substructure	S	V	D	
Dependencies: type of construction = bridge, basic building element = substructure				
Graffiti on visible surfaces	0	0	0	$Z_{3.14}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{3.15}$
Less wear of the protective layer	0	0	1	$Z_{3.16}$
Less rinses in the area of water flows	0	0	1	$Z_{3.17}$
Cleaning of the bearing bench (moldings or other)	x	x	x	
Invalidation of the benches (bird droppings or other)	x	x	x	
Remains of the formwork that press the construction	0	0	2	$Z_{3.18}$
Cleaning the bearing bench with accumulated moisture	x	x	x	
Formwork material (polystyrene) on the connection with the structure has not been removed	1	0	2	$Z_{3.19}$
Less dropping of stone linings	x	x	x	
The installation cover is not correct / damaged	x	x	x	
Cracking stone wall	x	x	x	
Partial moisture on stone wall / reinforced concrete	x	x	x	
Moisture on large surfaces of stone wall / reinforced concrete	x	x	x	
Bridges, cracks in concrete- / RC substructure				

Dependencies: type of construction = bridge, basic building element = substructure, damage = cracks				
Dry cracks outside the humidification (spinning) area <0.2mm (no reaction sulfuric acid - RSK)	x	x	x	
Surface cracks in the area of humidification (squeezing), cracks can run water, width <0.2mm (without RSK) REACTION SULFURIC ACID	x	x	x	
Dry cracks outside the humidification area (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, 0.2 -≤ 0.4mm wide (without RSK)	x	x	x	
Dry cracks outside the area of humidification (shrinkage) of width> 0,4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) of width> 0.4mm (without RSK)	x	x	x	
Description of damage / defect	S	V	D	
Cracks in the area of humidification (cracks), cracks can run water, width> 0,4mm, unarmed concrete (without RSK)	x	x	x	
Cracks in the area of humidification (sprinkling), cracks can run water, width> 0.4mm, RC bottom structure (without RSK)	x	x	x	

Summary Group 2

$Z_{3.14}$= 1.0	$\Delta Z_{3.14}$ = 0	$Z_{3.14}$ = 1.0+0= 1.0
$Z_{3.15}$ = 1.0	$\Delta Z_{3.15}$ = +0.1	$Z_{3.15}$ = 1.0+0.1= 1.1
$Z_{3.16}$ = 1.1	$\Delta Z_{3.16}$ = +0.1	Z_{316} = 1.1+0.1= 1.2
$Z_{3.17}$ = 1.1	$\Delta Z_{3.17}$ = 0	$Z_{3.17}$ = 1.1+0= 1.1
$Z_{3.18}$ = 2.0	$\Delta Z_{3.18}$ = 0	$Z_{3.18}$ = 2.0+0= 2.0
$Z_{3.19}$ = 2.2	$\Delta Z_{3.19}$ = 0	$Z_{3.19}$ = 2.2+0= 2.2
Sum group 2		**8.6**

Group 9: Transition devices

Transition devices (joints)	S	V	D	
Dependencies: structural element = transition device				
Contaminated transitional device (moving still possible)	x	x	x	
A highly Contaminated transient device (scrolling limited)	x	x	x	
Loosening the fixing of the profiles in the carpet of the structures, the profile is still held	x	x	x	
Loosening the fixing of the profiles in the carpet construction, the profile loosened	x	x	x	
Rubber profile dropped or multiple damaged	x	x	x	
Asphalt crossings, thin asphalt mass (rough, small open cracks)	0	0	0	$Z_{3.20}$
The asphalt crossing cracked and depressed	0	1	2	$Z_{3.21}$
The transient device is missing, the spanning structure is cracked at the ends	x	x	X	
The transient device is missing, the spanning structure at the ends is cracked and shrugged	x	x	X	

Summary Group 9

$Z_{3.20}$ = 1.0	$\Delta Z_{3.20}$ = +0.1	$Z_{3.20}$ = 1.0+0.1= 1.1

$Z_{3.21}$ = 2.1	$\Delta Z_{3.21}$ = +0.1	$Z_{3.21}$ = 2.1+0.1= 2.2
Sum group 9		**3.3**

Group 13: Fence

Protective means	S	V	D	
Fence				
Dependencies: element of construction = protective means, fence				
The compound fence - construction of the defect	0	0	1	$Z_{3.22}$
There is no fence, there are bumpers at> 50km / h, no pedestrian traffic is planned	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, small building, no pedestrian traffic is foreseen (temporary hazard sign exists)	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, pedestrian traffic planned	x	x	x	
Missing wire in the handrail of the fence, the length of the building is <20m	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, there is a bumper	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, the bumper is missing	x	x	x	
Fence height is not in accordance with regulations (difference ≤ 5cm)	x	x	x	
Fence height is not in accordance with regulations (difference 5 - 10cm)	x	x	x	
Fence height is not in accordance with regulations (difference > 10cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference ≤ 2cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference > 2cm)	x	x	x	
Missing individual fence filling rods	x	x	x	
Missing more consecutive fence filling rods	x	x	x	
Guardrail				
Dependencies: structural element = protection agent, bumper				
Bumper height is not in accordance with regulations (difference ≤ 3cm)	x	x	x	
Bumper height is not in accordance with regulations (difference > 3cm)	x	x	x	
The bumper is partially deformed	x	x	x	
a fence and a bumper are missing	x	x	x	
Weak anchor fasteners of the bumper / fence	x	x	x	
There is a lack of anchoring of protective devices on the length of the move	x	x	x	
Corrosion of protective agents				
Dependencies: structural element = protective agents, damage = surface, metal				
Insufficient thickness of the protective layer	0	0	2	$Z_{3.23}$
Local scattering (breaking) of the protective layer	x	x	x	
Scattering (breaking) of the protective layer on larger surfaces	x	x	x	
Local start of corrosion	0	0	1	$Z_{3.24}$
Corrosion of large surfaces	0	0	2	$Z_{3.25}$
The corrosion of individual support elements of the protecting agents	1	1	2	$Z_{3.26}$
The corrosion of more consecutive support elements of the protecting agents	x	x	x	

Summary Group 13

$Z_{3.22}$ = 1.1	$\Delta Z_{3.22}$ = +0.1	$Z_{3.22}$ = 1.1+0.1= 1.2
$Z_{3.23}$ =2.0	$\Delta Z_{3.23}$ = +0.1	$Z_{3.23}$ = 2.0+0.1= 2.1
$Z_{3.24}$= 1.1	$\Delta Z_{3.24}$= +0.1	$Z_{3.24}$= 2.5+0.1= 2.6
$Z_{3.25}$= 2.0	$\Delta Z_{3.25}$= +0	$Z_{3.25}$= 2.0+0= 2.0
$Z_{3.26}$ = 2.3	$\Delta Z_{3.26}$= +0.1	$Z_{3.26}$ = 2.3+0.1= 2.4
Sum group 13		**10.3**

Group 11: Road surface

Road surface	S	V	D	
Dependencies: structural element = useful surface				
pavement				
Subsidence of the pavement behind the abutments (≤ 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm), there are warning signs	x	x	x	
Drainage does not work, the risk of drifting	0	2	0	$Z_{3.27}$
Paving grooves / indentations, depth <1cm	X	x	x	
Pulley grooves / indentations, depth 1 - 3cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm, there are warning signs	x	x	x	
Pulley grooves / indentations, depth> 3cm	x	x	x	
Paving grooves / indentations, depth> 3cm, there are warning signs	x	x	x	
Bubbles, heights of ≤ 2cm	x	x	x	
Bubbles, height 2 - 5cm	x	x	x	
Bubbles, height 2 - 5cm, there are warning signs	x	x	x	
Bubbles, height > 5cm	x	x	x	
Bubbles, height > 5cm, there are warning signs	x	x	x	
Impact hole, depth ≤ 2cm	x	x	x	
Impact hole, depth 2 - 5cm	x	x	x	
Impact hole, depth 2 - 5cm, there are warning signs	x	x	x	
Impact hole, depth > 5cm	x	x	x	
Description of damages / defections	S	V	D	
Impact hole, depth> 5cm, there are warning signs	x	x	x	
A pedestrian hallway	x	x	x	
Erosion of surface layer <2cm	x	x	x	
Erosion of surface layer ≥2cm	x	x	x	
Erosion of surface layer ≥2cm, there are warning signs	x	x	x	
The layers break and fall in pieces	x	x	x	
Slipping risk	x	x	x	

Group 11: Road surface

$Z_{3.27}$ =2.0	$\Delta Z_{3.27}$= +0.1	$Z_{3.27}$ = 2.0+0.1= 2.1
Sum group 11		**2.1**

Group 14: Other

Signs	S	V	D	

Dependencies: structural element = Signs				
Missing building designation number	x	x	X	
The load limit sign is missing / incorrect S = 2 - 4, V = 2 - 4	X	x	x	
Drainage of the bridge				
Dependencies: element of construction = equipment, drainage of the bridge				
The fastening of the drainage pipe was corroded	1	0	2	$Z_{3.28}$
The fastening parts are missing, outside the traffic area	x	x	X	
The fastening parts are missing, above the traffic surface, V = 1 - 3	x	x	x	
Water leaks from the pipe (above the field)	x	x	x	
Water leaks from the pipe (above the parts of the structure)	0	0	2	$Z_{3.29}$
Water leaks from the pipe (above the traffic surfaces)	0	1	2	$Z_{3.30}$
Less corrosion damage on drainage pipes	0	0	1	$Z_{3.31}$
Significant corrosion damage on drainage pipes	x	x	x	
Missing dilatation of the drainage pipes at the transition of the structure / field	x	x	x	
Rain grid / clogged pipe	0	2	1	$Z_{3.32}$
In the raining grid there is a missing catcher of a garbage (pot)	x	x	x	
The drainage grid/ cleaning hole in the hinge plate is not secured	x	x	x	
The drainage grid is broken	x	x	x	
Inspection agents (inspection tools)				
Dependencies: element of construction = equipment, inspection agents				
Cracks on the rail of the inspected vehicle (risk of falling from height)	x	x	x	
Padlock is missing, third parties have unobstructed access to the building site	x	x	x	
Ladders, The distance between rungs is too large (> 280mm)	x	x	x	
Ladders, rungs too close to the building (<150mm)	x	x	x	
Ladder, The distance between the end rungs and the working surface too large (> 100mm)	x	x	x	
Ladders, according to the regulations, the necessary back protection is missing	x	x	x	
Corrosion of inspection agents				
Dependencies: element of construction = equipment, inspection means, damage = surface, metal				
Thickness of anti-corrosion protection too small	x	x	x	
Locally blown (cracked) anti-corrosion protection	x	x	x	
Anti-corrosive protection on large surfaces is blooming	x	x	x	
The beginning of corrosion - locally	x	x	x	
The beginning of corrosion - a large surface	x	x	x	
Corrosion of individual carriers	x	x	x	
Corrosion of several carriers in a row	x	x	x	
(Tools for protection) Means of protection from invading birds				
Dependencies: structural element = protective agents, protection from birds				
protection agents of invading birds are missing / damaged	x	x	x	
Overview of the building site				
Dependencies: damage = review				

The view of the whole building is not possible? a fictitious assessment of the situation is issued	x	x	x	
The building is very overgrown, only partial inspection possible	0	0	2	$Z_{3.33}$

Group 14: Other

$Z_{3.28}$ = 2.2	$\Delta Z_{3.28}$= 0	$Z_{3.28}$ = 2.2+0= 2.2
$Z_{3.29}$ = 2.0	$\Delta Z_{3.29}$= 0	$Z_{3.29}$ = 2.0+0= 2.0
$Z_{3.30}$ = 2.1	$\Delta Z_{3.30}$= -0.1	$Z_{3.30}$ = 2.1-0.1= 2
$Z_{3.31}$ = 1.1	$\Delta Z_{3.31}$= -0.1	$Z_{3.31}$ = 1.1-0.1= 1
$Z_{3.32}$ =2.1	$\Delta Z_{3.32}$= +0.1	$Z_{3.32}$ = 2.1+0.1= 2.2
$Z_{3.33}$ =2.0	$\Delta Z_{3.33}$=0	$Z_{3.33}$ = 2.0+0= 2.0
Sum group 14		**11.4**

3.2. LEVEL 2: MAXIMUM DAMAGE

NO.	Group1	Group2	Group9	Group13	Group11	Group14
	22.9	8.6	3.3	10.3	2.1	11.4

3.3. LEVEL 3:

Group	Z	ΔZ	Z_{BG}
Group 1	$Z_{3.9}$ = 2.8	ΔZ_2 = -0.1	Z_{BG}= 2.8 + 0.1= 2.9
	$Z_{3.11}$ = 2.8	ΔZ_2 = +0.1	
Group 2	$Z_{3.19}$ = 2.2	ΔZ_2 = -0.1	Z_{BG}= 2.2 - 0.1= 2.1
Group 9	$Z_{3.21}$ = 2.2	ΔZ_2 = 0	Z_{BG}= 2.2 + 0= 2.2
Group 11	$Z_{3.27}$ = 2.1	ΔZ_2= +0.1	Z_{BG}= 2.1 + 0.1= 2.2
Group 13	$Z_{3.24}$= 2.6	ΔZ_2 = +0.1	Z_{BG}= 2.6 + 0.1= 2.7
Group 14	$Z_{3.28}$ = 2.2	ΔZ_2 = 0	Z_{BG}= 2.2 + 0.1= 2.3
	$Z_{3.32}$ = 2.2	ΔZ_2 = +0.1	

Z_{ges}= 2.9 ΔZ_3 = +0.1 (GROUP 1 THE MAXIMUM Z_{BG})

Sufficient condition of the bridge structure

The stability of the structure is ensured.

The traffic safety of the structure may be impaired.

The stability and/or durability of at least one component group can be impaired.

The durability of the structure may be affected. A spread of damage or consequential damage to the structure, which in the medium term leads to significant impairments to stability and/or traffic safety or increased wear, is then to be expected.

Ongoing maintenance required.

Short-term repair required.

Measures to eliminate damage or warnings to maintain road safety may be necessary at short term.

4. BAB BIN GHESHIR ROAD BRIDGE

4.1. LEVEL 1: REGULAR BRIDGE INSPECTION

Group 1: Superstructure

Description of damage and defects	S	V	D	
Bridges, concrete / reinforced concrete / pre-stressed construction structures				
Dependencies: type of construction = bridge, basic building element = structure construction, material of the spanning structure = concrete				
Graffiti on visible surfaces	0	0	0	$Z_{4.1}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{4.2}$
Less wear of the protective layer	0	0	1	$Z_{4.3}$
The rust on the lower sides of the construction	0	0	1	$Z_{4.4}$
Pollution of internal passages of the building (remains of the formwork or other)	0	0	0	$Z_{4.5}$
Pollution of internal passages of building (bird feces or other)	x	x	x	x
Coarse granularity of concrete of the spanning structure	0	0	1	$Z_{4.6}$
The protective layer above the auxiliary rebar for the installation of the main rebar is too small	0	0	1	$Z_{4.7}$
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) poor quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1.0 to 2.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1,0 to 2,9cm) Poor concrete quality	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (below 1.0cm)	x	x	x	
The carbonate front reached the main rebar	0	0	3	$Z_{4.8}$
Visible main rebar on the underside of the structure, the reinforcement is lightly corroded (without significant reduction of the cross section)	1	0	3	$Z_{4.9}$
The main rebar of the spanning structure lies in the area of carbonization and is slightly corroded (it does not apply to the prefabrication reinforcement)	1	0	3	$Z_{4.10}$
Visible main rebar on the underside of the spanning structure, corroded rebar (there is a decrease in the cross-section)	2	0	3	$Z_{4.11}$
Blooming (water traces) in the area of heavily corroded main reinforcement on the underside of the spanning structure (advanced reductions in cross-section)	x	x	x	
Blooming in the area of heavily corroded main reinforcement on the underside of the spanning structure (the main reinforcement is partly excluded from the load)	x	x	x	
Partial moisture penetration	0	0	2	$Z_{4.12}$
Penetration of moisture on large surfaces	0	0	3	$Z_{4.13}$
Description of damage / defect	S	V	D	
Bridges, cracks in concrete / reinforced concrete / pre-stressed structure				
Dependencies: type of construction = bridge, basic building element = structure , material of the structure = concrete, damage = cracks				
Surface cracks outside the humidification area (widths) of ≤ 0.1mm in reinforced concrete or prestressed structure (eg cracks from shrinkage)	x	x	x	
Cracks outside the area of humidification (shrinkage) width of 0.1 - <0.2mm in reinforced concrete- or prestressed structure	x	x	x	
Cracks width 0.1 - <0.2mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Surface cracks in the humidification area (widths) 0.2 -≤ 0.4 mm in the RC structure	x	x	x	

Parallel cracks with prestressing of a width of 0.2 -≤ 0.4mm in the area of humidification (squeezing) in the prestressed structure	x	x	x	
Shrinkage widths> 0.4mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (not in the cable extension area)	x	x	x	
Cracks with a width of <0.2mm with prestressed structure (in cable extension)	x	x	x	
Cracks with a width of 0.2 - ≤ 0.4mm at the front of the structure (in the area of cable extension)	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (in the cable extension area)	x	x	x	
Cracks> 0.4mm under load	x	x	x	

Summary Group 1

$Z_{4.1} = 1.0$	$\Delta Z_{4.1} = 0$	$Z_{4.1} = 1.0+0=1.0$
$Z_{4.2} = 1.0$	$\Delta Z_{4.2} = +0.1$	$Z_{4.2} = 1.0+0.1=1.1$
$Z_{4.3} = 1.1$	$\Delta Z_{4.3} = +0.1$	$Z_{4.3} = 1.1+0.1= 1.2$
$Z_{4.4} = 1.1$	$\Delta Z_{4.4} = -0.1$	$Z_{4.4} = 1.1-0.1= 1$
$Z_{4.5} = 1.0$	$\Delta Z_{4.5} = 0$	$Z_{4.5} = 1.0+0= 1.0$
$Z_{4.6} = 1.1$	$\Delta Z_{4.6} = +0.1$	$Z_{4.6} = 1.1+0.1= 1.2$
$Z_{4.7} = 1.1$	$\Delta Z_{4.7} = -0.1$	$Z_{4.7} = 1.1-0.1= 1.0$
$Z_{4.8} = 2.5$	$\Delta Z_{4.8} = 0$	$Z_{4.8} = 2.5+0= 2.5$
$Z_{4.9} = 2.7$	$\Delta Z_{4.9} = +0.1$	$Z_{4.9} = 2.7+0.1= 2.8$
$Z_{4.10} = 2.7$	$\Delta Z_{4.10} = 0$	$Z_{4.10} = 2.7+0= 2.7$
$Z_{4.11} = 2.8$	$\Delta Z_{4.11} = 0$	$Z_{4.11} = 2.8+0=2.8$
$Z_{4.12} = 2.0$	$\Delta Z_{4.12} = 0$	$Z_{4.12} = 2.0+0= 2.0$
$Z_{4.13} = 2.5$	$\Delta Z_{4.13} = +0.1$	$Z_{4.13} = 2.5+0.1= 2.6$
Sum group 1		**22.9**

Group 2: Substructure

Bridges, substructure	S	V	D	
Dependencies: type of construction = bridge, basic building element = substructure				
Graffiti on visible surfaces	0	0	0	$Z_{4.14}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{4.15}$
Less wear of the protective layer	0	0	1	$Z_{4.16}$
Less rinses in the area of water flows	0	0	1	$Z_{4.17}$
Cleaning of the bearing bench (moldings or other)	x	x	x	
Invalidation of the benches (bird droppings or other)	x	x	x	
Remains of the formwork that press the construction	0	0	2	$Z_{4.18}$
Cleaning the bearing bench with accumulated moisture	x	x	x	
Formwork material (polystyrene) on the connection with the structure has not been removed	1	0	2	$Z_{4.19}$
Less dropping of stone linings	x	x	x	
The installation cover is not correct / damaged	x	x	x	
Cracking stone wall	x	x	x	
Partial moisture stone wall / reinforced concrete	x	x	x	
Moisture on large surfaces of stone wall / reinforced concrete	x	x	x	
Bridges, cracks in concrete- / RC substructure				

Dependencies: type of construction = bridge, basic building element = substructure, damage = cracks				
Dry cracks outside the humidification (spinning) area <0.2mm (no reaction sulfuric acid - RSK)	x	x	x	
Surface cracks in the area of humidification (squeezing), cracks can run water, width <0.2mm (without RSK) REACTION SULFURIC ACID	x	x	x	
Dry cracks outside the humidification area (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, 0.2 -≤ 0.4mm wide (without RSK)	x	x	x	
Dry cracks outside the area of humidification (shrinkage) of width> 0,4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) of width> 0.4mm (without RSK)	x	x	x	
Description of damage / defect	S	V	D	
Cracks in the area of humidification (cracks), cracks can run water, width> 0,4mm, unarmed concrete (without RSK)	x	x	x	
Cracks in the area of humidification (sprinkling), cracks can run water, width> 0.4mm, RC bottom structure (without RSK)	x	x	x	

Summary Group 2

$Z_{4.14}$= 1.0	$\Delta Z_{4.14}$ = 0	$Z_{4.14}$ = 1.0+0= 1.0
$Z_{4.15}$ = 1.0	$\Delta Z_{4.15}$ = +0.1	$Z_{4.15}$ = 1.0+0.1= 1.1
$Z_{4.16}$ = 1.1	$\Delta Z_{4.16}$ = +0.1	$Z_{4.16}$ = 1.1+0.1= 1.2
$Z_{4.17}$ = 1.1	$\Delta Z_{4.17}$ = 0	$Z_{4.17}$ = 1.1+0= 1.1
$Z_{4.18}$ = 2.0	$\Delta Z_{4.18}$ = 0	$Z_{4.18}$ = 2.0+0= 2.0
$Z_{4.19}$ = 2.2	$\Delta Z_{4.19}$ = 0	$Z_{4.19}$ = 2.2+0= 2.2
Sum group 2		**8.6**

Group 9: Transition devices

Transition devices (joints)	S	V	D	
Dependencies: structural element = transitional device				
Contaminated transitional device (moving still possible)	x	x	x	
A highly Contaminated transient device (scrolling limited)	x	x	x	
Loosening the fixing of the profiles in the carpet of the structures, the profile is still held	x	x	x	
Loosening the fixing of the profiles in the carpet construction, the profile loosened	x	x	x	
Rubber profile dropped or multiple damaged	x	x	x	
Asphalt crossings, thin asphalt mass (rough, small open cracks)	0	0	0	$Z_{4.20}$
The asphalt crossing cracked and depressed	0	1	2	$Z_{4.21}$
The transient device is missing, the spanning structure is cracked at the ends	x	x	X	
The transient device is missing, the spanning structure at the ends is cracked and shrugged	x	x	X	

Summary Group 9

$Z_{4.20}$ = 1.0	$\Delta Z_{4.20}$ = +0.1	$Z_{4.20}$ = 1.0+0.1= 1.1

$Z_{4.21}$ = 2.1	$\Delta Z_{4.21}$ = +0.1	$Z_{4.21}$ = 2.1+0.1= 2.2
Sum group 9		**3.3**

Group 13: Fence

Protective means	S	V	D	
Fence				
Dependencies: element of construction = protective means, fence				
The compound fence - construction of the defect	0	0	1	$Z_{4.22}$
There is no fence, there are bumpers at> 50km / h, no pedestrian traffic is planned	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, small building, no pedestrian traffic is foreseen (temporary hazard sign exists)	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, pedestrian traffic planned	x	x	x	
Missing wire in the handrail of the fence, the length of the building is <20m	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, there is a bumper	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, the bumper is missing	x	x	x	
Fence height is not in accordance with regulations (difference ≤ 5cm)	x	x	x	
Fence height is not in accordance with regulations (difference 5 - 10cm)	x	x	x	
Fence height is not in accordance with regulations (difference > 10cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference ≤ 2cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference > 2cm)	x	x	x	
Missing individual fence filling rods	x	x	x	
Missing more consecutive fence filling rods	x	x	x	
Guardrail				
Dependencies: structural element = protection agent, bumper				
Bumper height is not in accordance with regulations (difference ≤ 3cm)	x	x	x	
Bumper height is not in accordance with regulations (difference > 3cm)	x	x	x	
The bumper is partially deformed	x	x	x	
a fence and a bumper are missing	x	x	x	
Weak anchor fasteners of the bumper / fence	x	x	x	
There is a lack of anchoring of protective devices on the length of the move	x	x	x	
Corrosion of protective agents				
Dependencies: structural element = protective agents, damage = surface, metal				
Insufficient thickness of the protective layer	0	0	2	$Z_{4.23}$
Local scattering (breaking) of the protective layer	0	0	1	$Z_{4.24}$
Scattering (breaking) of the protective layer on larger surfaces	x	x	x	
Local start of corrosion	0	0	1	$Z_{4.25}$
Corrosion of large surfaces	0	0	2	$Z_{4.26}$
The corrosion of individual support elements of the protecting agents	1	1	2	$Z_{4.27}$
The corrosion of more consecutive support elements of the protecting agents	x	x	x	

Summary Group 13

$Z_{4.22}$ = 1.1	$\Delta Z_{4.22}$ = +0.1	$Z_{4.22}$ = 1.1+0.1= 1.2

$Z_{4.23}$ =2.0	$\Delta Z_{4.23}$ = +0.1	$Z_{4.23}$ = 2.0+0.1= 2.1
$Z_{4.24}$=1.1	$\Delta Z_{4.24}$ = 0	$Z_{4.24}$ = 1.1+0= 1.1
$Z_{4.25}$= 1.1	$\Delta Z_{4.25}$= +0.1	$Z_{4.26}$= 2.5+0.1= 2.6
$Z_{4.26}$= 2.0	$\Delta Z_{4.26}$= +0	$Z_{4.26}$= 2.0+0= 2.0
$Z_{4.27}$ = 2.3	$\Delta Z_{4.27}$= +0.1	$Z_{4.27}$ = 2.3+0.1= 2.4
Sum group 13		**11.4**

Group 11: Road surface

Road surface	S	V	D	
Dependencies: structural element = useful surface				
pavement				
Subsidence of the pavement behind the abutments (≤ 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm), there are warning signs	x	x	x	
Drainage does not work, the risk of drifting	0	2	0	$Z_{4.28}$
Paving grooves / indentations, depth <1cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm, there are warning signs	x	x	x	
Pulley grooves / indentations, depth> 3cm	x	x	x	
Paving grooves / indentations, depth> 3cm, there are warning signs	x	x	x	
Bubbles, heights of ≤ 2cm	x	x	x	
Bubbles, height 2 - 5cm	x	x	x	
Bubbles, height 2 - 5cm, there are warning signs	x	x	x	
Bubbles, height > 5cm	x	x	x	
Bubbles, height > 5cm, there are warning signs	x	x	x	
Impact hole, depth ≤ 2cm	x	x	x	
Impact hole, depth 2 - 5cm	x	x	x	
Impact hole, depth 2 - 5cm, there are warning signs	x	x	x	
Impact hole, depth > 5cm	x	x	x	
Description of damages / defections	S	V	D	
Impact hole, depth> 5cm, there are warning signs	x	x	x	
A pedestrian hallway	x	x	x	
Erosion of surface layer <2cm	x	x	x	
Erosion of surface layer ≥2cm	x	x	x	
Erosion of surface layer ≥2cm, there are warning signs	x	x	x	
The layers break and fall in pieces	x	x	x	
Slipping risk	x	x	x	

Summary Group 11: Useful Surface

$Z_{4.28}$ =2.0	$\Delta Z_{4.28}$= +0.1	$Z_{4.28}$ = 2.0+0.1= 2.1
Sum group 11		**2.1**

Group 14: Other

Signs	S	V	D	
Dependencies: structural element = Signs				

Missing building designation number	x	x	x	
The load limit sign is missing / incorrect S = 2 - 4, V = 2 - 4	x	x	x	
Drainage of the bridge				
Dependencies: element of construction = equipment, drainage of the bridge				
The fastening of the drainage pipe was corroded	1	0	2	$Z_{4.29}$
The fastening parts are missing, outside the traffic area	x	x	x	
The fastening parts are missing, above the traffic surface, V = 1 - 3	x	x	x	
Water leaks from the pipe (above the field)	x	x	x	
Water leaks from the pipe (above the parts of the structure)	0	0	2	$Z_{4.30}$
Water leaks from the pipe (above the traffic surfaces)	0	1	2	$Z_{4.31}$
Less corrosion damage on drainage pipes	0	0	1	$Z_{4.32}$
Significant corrosion damage on drainage pipes	x	x	x	
Missing dilatation of the drainage pipes at the transition of the structure / field	x	x	x	
Rain grid / clogged pipe	0	2	1	$Z_{4.33}$
In the raining grid there is a missing catcher of a garbage (pot)	x	x	x	
The drainage grid/ cleaning hole in the hinge plate is not secured	x	x	x	
The drainage grid is broken	x	x	x	
Inspection agents (inspection tools)				
Dependencies: element of construction = equipment, inspection agents				
Cracks on the rail of the inspected vehicle (risk of falling from height)	x	x	x	
Padlock is missing, third parties have unobstructed access to the building site	x	x	x	
Ladders, The distance between rungs is too large (> 280mm)	x	x	x	
Ladders, rungs too close to the building (<150mm)	x	x	x	
Ladder, The distance between the end rungs and the working surface too large (> 100mm)	x	x	x	
Ladders, according to the regulations, the necessary back protection is missing	x	x	x	
Corrosion of inspection agents				
Dependencies: element of construction = equipment, inspection means, damage = surface, metal				
Thickness of anti-corrosion protection too small	x	x	x	
Locally blown (cracked) anti-corrosion protection	x	x	x	
Anti-corrosive protection on large surfaces is blooming	x	x	x	
The beginning of corrosion - locally	x	x	x	
The beginning of corrosion - a large surface	x	x	x	
Corrosion of individual carriers	x	x	x	
Corrosion of several carriers in a row	x	x	x	
(Tools for protection) Means of protection from invading birds				
Dependencies: structural element = protective agents, protection from birds				
protection agents of invading birds are missing / damaged	x	x	x	
Overview of the building site				
Dependencies: damage = review				
The view of the whole building is not possible? a fictitious assessment of the situation is issued	x	x	x	

The building is very overgrown, only partial inspection possible	0	0	2	$Z_{4.34}$

Summary Group 14: Signs

$Z_{4.29}$ = 2.2	$\Delta Z_{4.29}$= 0	$Z_{4.29}$ = 2.2+0= 2.2
$Z_{4.30}$ = 2.0	$\Delta Z_{4.30}$= 0	$Z_{4.30}$ = 2.0+0= 2.0
$Z_{4.31}$ = 2.1	$\Delta Z_{4.31}$= -0.1	$Z_{4.31}$ = 2.1-0.1= 2
$Z_{4.32}$ = 1.1	$\Delta Z_{4.32}$= -0.1	$Z_{4.32}$ = 1.1-0.1= 1
$Z_{4.33}$ =2.1	$\Delta Z_{4.33}$= +0.1	$Z_{4.33}$ = 2.1+0.1= 2.2
$Z_{4.34}$ =2.0	$\Delta Z_{4.34}$=0	$Z_{4.34}$ = 2.0+0= 2.0
Sum group 14		**11.4**

4.2. LEVEL 2: MAXIMUM DAMAGE

NO.	Group1	Group2	Group9	Group13	Group11	Group14
	22.9	8.6	3.3	11.4	2.1	11.4

4.3. LEVEL 3:

Group	Z	ΔZ	**Z_{BG}**
Group 1	$Z_{4.9}$ = 2.8	ΔZ_2 = -0.1	Z_{BG}= 2.8 + 0.1= 2.9
	$Z_{4.11}$ = 2.8	ΔZ_2 = +0.1	
Group 2	$Z_{4.19}$ = 2.2	ΔZ_2 = -0.1	Z_{BG}= 2.2 - 0.1= 2.1
Group 9	$Z_{4.21}$ = 2.2	ΔZ_2 = 0	Z_{BG}= 2.2 + 0= 2.2
Group 11	$Z_{4.28}$ = 2.1	ΔZ_2= +0.1	Z_{BG}= 2.1 + 0.1= 2.2
Group 13	$Z_{4.25}$= 2.6	ΔZ_2 = +0.1	Z_{BG}= 2.6 + 0.1= 2.7
Group 14	$Z_{4.29}$ = 2.2	ΔZ_2 = 0	Z_{BG}= 2.2 + 0.1= 2.3
	$Z_{4.33}$ = 2.2	ΔZ_2 = +0.1	

Z_{ges}= 2.9 ΔZ_3 = +0.1 (GROUP 1 THE MAXIMUM Z_{BG})

Sufficient condition of the bridge structure

The stability of the structure is ensured.

The traffic safety of the structure may be impaired.

The stability and/or durability of at least one component group can be impaired.

The durability of the structure may be affected. A spread of damage or consequential damage to the structure, which in the medium term leads to significant impairments to stability and/or traffic safety or increased wear, is then to be expected.

Ongoing maintenance required.

Short-term repair required.

Measures to eliminate damage or warnings to maintain road safety may be necessary at short term.

5. AL SREEM ROAD BRIDGE

5.1. LEVEL 1: REGULAR BRIDGE INSPECTION

Group 1: Superstructure

Description of damage and defects	S	V	D	
Bridges, concrete / reinforced concrete / pre-stressed construction structures				
Dependencies: type of construction = bridge, basic building element = structure construction, material of the spanning structure = concrete				
Graffiti on visible surfaces	0	0	0	$Z_{5.1}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{5.2}$
Less wear of the protective layer	0	0	1	$Z_{5.3}$
The rust on the lower sides of the construction	0	0	1	$Z_{5.4}$
Pollution of internal passages of the building (remains of the formwork or other)	0	0	0	$Z_{5.5}$
Pollution of internal passages of building (bird feces or other)	X	X	X	X
Coarse granularity of concrete of the spanning structure	0	0	1	$Z_{5.6}$
The protective layer above the auxiliary rebar for the installation of the main rebar is too small	0	0	1	$Z_{5.7}$
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) poor quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1.0 to 2.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1,0 to 2,9cm) Poor concrete quality	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (below 1.0cm)	x	x	x	
The carbonate front reached the main rebar	0	0	3	$Z_{5.8}$
Visible main rebar on the underside of the structure, the reinforcement is lightly corroded (without significant reduction of the cross section)	1	0	3	$Z_{5.9}$
The main rebar of the spanning structure lies in the area of carbonization and is slightly corroded (it does not apply to the prefabrication reinforcement)	1	0	3	$Z_{5.10}$
Visible main rebar on the underside of the spanning structure, corroded rebar (there is a decrease in the cross-section)	x	x	x	
Blooming (water traces) in the area of heavily corroded main reinforcement on the underside of the spanning structure (advanced reductions in cross-section)	x	x	x	
Blooming in the area of heavily corroded main reinforcement on the underside of the spanning structure (the main reinforcement is partly excluded from the load)	x	x	x	
Partial moisture penetration	0	0	2	$Z_{5.11}$
Penetration of moisture on large surfaces	0	0	3	$Z_{5.12}$
Description of damage / defect	S	V	D	
Bridges, cracks in concrete / reinforced concrete / pre-stressed structure				
Dependencies: type of construction = bridge, basic building element = structure , material of the structure = concrete, damage = cracks				
Surface cracks outside the humidification area (widths) of ≤ 0.1mm in reinforced concrete or prestressed structure (eg cracks from shrinkage)	x	x	x	
Cracks outside the area of humidification (shrinkage) width of 0.1 - <0.2mm in reinforced concrete- or prestressed structure	x	x	x	
Cracks width 0.1 - <0.2mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Surface cracks in the humidification area (widths) 0.2 -≤ 0.4 mm in the RC structure	x	x	x	

Parallel cracks with prestressing of a width of 0.2 -≤ 0.4mm in the area of humidification (squeezing) in the prestressed structure	x	x	x	
Shrinkage widths> 0.4mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (not in the cable extension area)	x	x	x	
Cracks with a width of <0.2mm with prestressed structure (in cable extension)	x	x	x	
Cracks with a width of 0.2 - ≤ 0.4mm at the front of the structure (in the area of cable extension)	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (in the cable extension area)	x	x	x	
Cracks> 0.4mm under load	x	x	x	

Summary Group 1

$Z_{5.1} = 1.0$	$\Delta Z_{5.1} = 0$	$Z_{5.1} = 1.0+0=1.0$
$Z_{5.2} = 1.0$	$\Delta Z_{5.2} = +0.1$	$Z_{5.2} = 1.0+0.1=1.1$
$Z_{5.3} = 1.1$	$\Delta Z_{5.3} = +0.1$	$Z_{5.3} = 1.1+0.1= 1.2$
$Z_{5.4} = 1.1$	$\Delta Z_{5.4} = -0.1$	$Z_{5.4} = 1.1-0.1= 1$
$Z_{5.5} = 1.0$	$\Delta Z_{5.5} = 0$	$Z_{5.5} = 1.0+0= 1.0$
$Z_{5.6} = 1.1$	$\Delta Z_{5.6} = +0.1$	$Z_{5.6} = 1.1+0.1= 1.2$
$Z_{5.7} = 1.1$	$\Delta Z_{5.7} = +0.1$	$Z_{5.7} = 1.1+0.1= 1.2$
$Z_{5.8} = 2.5$	$\Delta Z_{5.8} = +0.1$	$Z_{5.8} = 2.5+0.1= 2.6$
$Z_{5.9} = 2.7$	$\Delta Z_{5.9} = -0.1$	$Z_{5.9} = 2.7-0.1= 2.6$
$Z_{5.10} = 2.7$	$\Delta Z_{5.10} = 0$	$Z_{5.10} = 2.7+0= 2.7$
$Z_{5.11} = 2.0$	$\Delta Z_{5.11} = 0$	$Z_{5.11} = 2.0+0= 2.0$
$Z_{5.12} = 2.5$	$\Delta Z_{5.12} = +0.1$	$Z_{5.12} = 2.5+0.1= 2.6$
Sum group 1		**20.2**

Group 2: Substructure

Bridges, substructure	S	V	D	
Dependencies: type of construction = bridge, basic building element = substructure				
Graffiti on visible surfaces	0	0	0	$Z_{5.13}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{5.14}$
Less wear of the protective layer	0	0	1	$Z_{5.15}$
Less rinses in the area of water flows	0	0	1	$Z_{5.16}$
Cleaning of the bearing bench (moldings or other)	x	x	x	
Invalidation of the benches (bird droppings or other)	x	x	x	
Remains of the formwork that press the construction	0	0	2	$Z_{5.17}$
Cleaning the bearing bench with accumulated moisture	x	x	x	
Formwork material (polystyrene) on the connection with the structure has not been removed	x	x	x	
Less dropping of stone linings	x	x	x	
The installation cover is not correct / damaged	x	x	x	
Cracking stone wall	x	x	x	
Partial moisture stone wall / reinforced concrete	x	x	x	
Moisture on large surfaces of stone wall / reinforced concrete	x	x	x	
Bridges, cracks in concrete- / RC substructure				

Dependencies: type of construction = bridge, basic building element = substructure, damage = cracks				
Dry cracks outside the humidification (spinning) area <0.2mm (no reaction sulfuric acid - RSK)	x	x	x	
Surface cracks in the area of humidification (squeezing), cracks can run water, width <0.2mm (without RSK) REACTION SULFURIC ACID	x	x	x	
Dry cracks outside the humidification area (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, 0.2 -≤ 0.4mm wide (without RSK)	x	x	x	
Dry cracks outside the area of humidification (shrinkage) of width> 0,4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) of width> 0.4mm (without RSK)	x	x	x	
Description of damage / defect	S	V	D	
Cracks in the area of humidification (cracks), cracks can run water, width> 0,4mm, unarmed concrete (without RSK)	x	x	x	
Cracks in the area of humidification (sprinkling), cracks can run water, width> 0.4mm, RC bottom structure (without RSK)	x	x	x	

Summary Group 2

$Z_{5.13}$= 1.0	$\Delta Z_{5.13}$ = 0	$Z_{5.13}$ = 1.0+0= 1.0
$Z_{5.14}$ = 1.0	$\Delta Z_{5.14}$ = +0.1	$Z_{5.14}$ = 1.0+0.1= 1.1
$Z_{5.15}$ = 1.1	$\Delta Z_{5.15}$ = +0.1	$Z_{5.15}$ = 1.1+0.1= 1.2
$Z_{5.16}$ = 1.1	$\Delta Z_{5.16}$ = 0	$Z_{5.16}$ = 1.1+0= 1.1
$Z_{5.17}$ = 2.0	$\Delta Z_{5.17}$ = 0	$Z_{5.17}$ = 2.0+0= 2.0
Sum group 2		**6.4**

Group 9: Transition devices

Transition devices (joints)	S	V	D	
Dependencies: structural element = transitional device				
Contaminated transitional device (moving still possible)	x	x	x	
A highly Contaminated transient device (scrolling limited)	x	x	x	
Loosening the fixing of the profiles in the carpet of the structures, the profile is still held	x	x	x	
Loosening the fixing of the profiles in the carpet construction, the profile loosened	x	x	x	
Rubber profile dropped or multiple damaged	x	x	x	
Asphalt crossings, thin asphalt mass (rough, small open cracks)	0	0	0	$Z_{5.18}$
The asphalt crossing cracked and depressed	0	1	2	$Z_{5.19}$
The transient device is missing, the spanning structure is cracked at the ends	x	x	x	
The transient device is missing, the spanning structure at the ends is cracked and shrugged	x	x	x	

Summary Group 9

$Z_{5.18}$ = 1.0	$\Delta Z_{5.18}$ = +0.1	$Z_{5.18}$ = 1.0+0.1= 1.1
$Z_{5.19}$ = 2.1	$\Delta Z_{5.19}$ = +0.1	$Z_{5.19}$ = 2.1+0.1= 2.2

Sum group 9	3.3

Group 13: Fence

Protective means	S	V	D	
Fence				
Dependencies: element of construction = protective means, fence				
The compound fence - construction of the defect	0	0	1	$Z_{5.20}$
There is no fence, there are bumpers at> 50km / h, no pedestrian traffic is planned	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, small building, no pedestrian traffic is foreseen (temporary hazard sign exists)	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, pedestrian traffic planned	x	x	x	
Missing wire in the handrail of the fence, the length of the building is <20m	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, there is a bumper	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, the bumper is missing	x	x	x	
Fence height is not in accordance with regulations (difference ≤ 5cm)	x	x	x	
Fence height is not in accordance with regulations (difference 5 - 10cm)	x	x	x	
Fence height is not in accordance with regulations (difference > 10cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference ≤ 2cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference > 2cm)	x	x	x	
Missing individual fence filling rods	x	x	x	
Missing more consecutive fence filling rods	x	x	x	
Guardrail				
Dependencies: structural element = protection agent, bumper				
Bumper height is not in accordance with regulations (difference ≤ 3cm)	x	x	x	
Bumper height is not in accordance with regulations (difference > 3cm)	x	x	x	
The bumper is partially deformed	x	x	x	
a fence and a bumper are missing	x	x	x	
Weak anchor fasteners of the bumper / fence	x	x	x	
There is a lack of anchoring of protective devices on the length of the move	x	x	x	
Corrosion of protective agents				
Dependencies: structural element = protective agents, damage = surface, metal				
Insufficient thickness of the protective layer	0	0	2	$Z_{5.21}$
Local scattering (breaking) of the protective layer	0	0	1	$Z_{5.22}$
Scattering (breaking) of the protective layer on larger surfaces	x	x	x	
Local start of corrosion	0	0	1	$Z_{5.23}$
Corrosion of large surfaces	0	0	2	$Z_{5.24}$
The corrosion of individual support elements of the protecting agents	1	1	2	$Z_{5.25}$
The corrosion of more consecutive support elements of the protecting agents	x	x	x	

Summary Group 13

$Z_{5.20}$ = 1.1	$\Delta Z_{5.20}$ = +0.1	$Z_{5.20}$ = 1.1+0.1= 1.2
$Z_{5.21}$ =2.0	$\Delta Z_{5.21}$ = +0.1	$Z_{5.21}$ = 2.0+0.1= 2.1
$Z_{5.22}$=1.1	$\Delta Z_{5.22}$ = 0	$Z_{5.22}$ = 1.1+0= 1.1
$Z_{5.23}$= 1.1	$\Delta Z_{5.23}$= 0	$Z_{5.23}$= 1.1+0= 1.1
$Z_{5.24}$= 2.0	$\Delta Z_{5.24}$= +0	$Z_{5.24}$= 2.0+0= 2.0
$Z_{5.25}$ = 2.3	$\Delta Z_{5.25}$= +0.1	$Z_{5.25}$ = 2.3+0.1= 2.4
Sum group 13		**9.9**

Group 11: Road surface

Road surface	S	V	D	
Dependencies: structural element = useful surface				
pavement				
Subsidence of the pavement behind the abutments (≤ 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm), there are warning signs	x	x	x	
Drainage does not work, the risk of drifting	0	2	0	$Z_{5.26}$
Paving grooves / indentations, depth <1cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm, there are warning signs	x	x	x	
Pulley grooves / indentations, depth> 3cm	x	x	x	
Paving grooves / indentations, depth> 3cm, there are warning signs	x	x	x	
Bubbles, heights of ≤ 2cm	x	x	x	
Bubbles, height 2 - 5cm	x	x	x	
Bubbles, height 2 - 5cm, there are warning signs	x	x	x	
Bubbles, height > 5cm	x	x	x	
Bubbles, height > 5cm, there are warning signs	x	x	x	
Impact hole, depth ≤ 2cm	x	x	x	
Impact hole, depth 2 - 5cm	x	x	x	
Impact hole, depth 2 - 5cm, there are warning signs	x	x	x	
Impact hole, depth > 5cm	x	x	x	
Description of damages / defections	S	V	D	
Impact hole, depth> 5cm, there are warning signs	x	x	x	
A pedestrian hallway	x	x	x	
Erosion of surface layer <2cm	x	x	x	
Erosion of surface layer ≥2cm	x	x	x	
Erosion of surface layer ≥2cm, there are warning signs	x	x	x	
The layers break and fall in pieces	x	x	x	
Slipping risk	x	x	x	

Summary Group 11

$Z_{5.26}$ =2.0	$\Delta Z_{5.26}$= +0.1	$Z_{5.26}$ = 2.0+0.1= 2.1
Sum group 11		**2.1**

Group 14: Other

Signs	S	V	D	
Dependencies: structural element = Signs				
Missing building designation number	x	x	x	
The load limit sign is missing / incorrect S = 2 - 4, V = 2 - 4	x	x	x	
Drainage of the bridge				
Dependencies: element of construction = equipment, drainage of the bridge				
The fastening of the drainage pipe was corroded	1	0	2	$Z_{5.27}$
The fastening parts are missing, outside the traffic area	x	x	x	
The fastening parts are missing, above the traffic surface, V = 1 - 3	x	x	x	
Water leaks from the pipe (above the field)	x	x	x	
Water leaks from the pipe (above the parts of the structure)	0	0	2	$Z_{5.28}$
Water leaks from the pipe (above the traffic surfaces)	0	1	2	$Z_{5.29}$
Less corrosion damage on drainage pipes	0	0	1	$Z_{5.30}$
Significant corrosion damage on drainage pipes	x	x	x	
Missing dilatation of the drainage pipes at the transition of the structure / field	x	x	x	
Rain grid / clogged pipe	0	2	1	$Z_{5.31}$
In the raining grid there is a missing catcher of a garbage (pot)	x	x	x	
The drainage grid/ cleaning hole in the hinge plate is not secured	x	x	x	
The drainage grid is broken	x	x	x	
Inspection agents (inspection tools)				
Dependencies: element of construction = equipment, inspection agents				
Cracks on the rail of the inspected vehicle (risk of falling from height)	x	x	x	
Padlock is missing, third parties have unobstructed access to the building site	x	x	x	
Ladders, the distance between rungs is too large (> 280mm)	x	x	x	
Ladders, rungs too close to the building (<150mm)	x	x	x	
Ladder, The distance between the end rungs and the working surface too large (> 100mm)	x	x	x	
Ladders, according to the regulations, the necessary back protection is missing	x	x	x	
Corrosion of inspection agents				
Dependencies: element of construction = equipment, inspection means, damage = surface, metal				
Thickness of anti-corrosion protection too small	x	x	x	
Locally blown (cracked) anti-corrosion protection	x	x	x	
Anti-corrosive protection on large surfaces is blooming	x	x	x	
The beginning of corrosion - locally	x	x	x	
The beginning of corrosion - a large surface	x	x	x	
Corrosion of individual carriers	x	x	x	
Corrosion of several carriers in a row	x	x	x	
(Tools for protection) Means of protection from invading birds				
Dependencies: structural element = protective agents, protection from birds				
protection agents of invading birds are missing / damaged	x	x	x	

Overview of the building site				
Dependencies: damage = review				
The view of the whole building is not possible? a fictitious assessment of the situation is issued	x	x	x	
The building is very overgrown, only partial inspection possible	0	0	2	$Z_{5.32}$

Summary Group 14

$Z_{5.27}$ = 2.2	$\Delta Z_{5.27}$=- 0 .1	$Z_{5.27}$ = 2.2-0.1= 2.1
$Z_{5.28}$ = 2.0	$\Delta Z_{5.28}$= 0	$Z_{5.28}$ = 2.0+0= 2.0
$Z_{5.29}$ = 2.1	$\Delta Z_{5.29}$= -0.1	$Z_{5.29}$ = 2.1-0.1= 2
$Z_{5.30}$ = 1.1	$\Delta Z_{5.30}$= -0.1	$Z_{5.30}$ = 1.1-0.1= 1
$Z_{5.31}$ =2.1	$\Delta Z_{5.31}$= +0.1	$Z_{5.31}$ = 2.1+0.1= 2.2
$Z_{5.32}$ =2.0	$\Delta Z_{5.32}$=0	$Z_{5.32}$ = 2.0+0= 2.0
Sum group 14		**11.3**

5.2. LEVEL 2: MAXIMUM DAMAGE

NO.	Group1	Group2	Group9	Group13	Group11	Group14
	20.2	6.4	3.3	9.9	2.1	11.3

5.3. LEVEL 3:

Group	Z	ΔZ	**Z_{BG}**
Group 1	$Z_{5.10}$ = 2.7	ΔZ_2 = -0.1	Z_{BG}= 2.7 - 0.1= 2.6
Group 2	$Z_{5.17}$ = 2.0	ΔZ_2 = -0.1	Z_{BG}= 2.0 - 0.1= 1.9
Group 9	$Z_{5.19}$ = 2.2	ΔZ_2 = 0	Z_{BG}= 2.2 + 0= 2.2
Group 11	$Z_{5.26}$ = 2.1	ΔZ_2= +0.1	Z_{BG}= 2.1 + 0.1= 2.2
Group 13	$Z_{5.25}$= 2.4	ΔZ_2 = 0	Z_{BG}= 2.4 + 0= 2.4
Group 14	$Z_{5.31}$ = 2.2	ΔZ_2 = 0	Z_{BG}= 2.2 + 0= 2.2

Z_{ges}= 2.6 ΔZ_3 = - 0.1 (GROUP 1 THE MAXIMUM Z_{BG})

Sufficient condition of the bridge structure

The stability of the structure is ensured.

The traffic safety of the structure may be impaired.

The stability and/or durability of at least one component group can be impaired.

The durability of the structure may be affected. A spread of damage or consequential damage to the structure, which in the medium term leads to significant impairments to stability and/or traffic safety or increased wear, is then to be expected.

Ongoing maintenance required.

Short-term repair required.

Measures to eliminate damage or warnings to maintain road safety may be necessary at short term.

6. ALSHAAB PORT BRIDGE

6.1. LEVEL 1: REGULAR BRIDGE INSPECTION

Group 1: Superstructure

Description of damage and defects	S	V	D	
Bridges, concrete / reinforced concrete / pre-stressed construction structures				
Dependencies: type of construction = bridge, basic building element = structure construction, material of the spanning structure = concrete				
Graffiti on visible surfaces	0	0	0	$Z_{6.1}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{6.2}$
Less wear of the protective layer	0	0	1	$Z_{6.3}$
The rust on the lower sides of the construction	0	0	1	$Z_{6.4}$
Pollution of internal passages of the building (remains of the formwork or other)	0	0	0	$Z_{6.5}$
Pollution of internal passages of building (bird feces or other)	x	x	x	
Coarse granularity of concrete of the spanning structure	0	0	1	$Z_{6.6}$
The protective layer above the auxiliary rebar for the installation of the main reinforcement is too small	0	0	1	$Z_{6.7}$
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) Good quality of concrete	X	X	X	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) poor quality of concrete	X	X	X	
The protective layer of the main rebar on the underside of the structure is too small (1.0 to 2.9cm) Good quality of concrete	X	X	X	
The protective layer of the main rebar on the underside of the structure is too small (1,0 to 2,9cm) Poor concrete quality	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (below 1.0cm)	x	x	x	
The carbonate front reached the main rebar	0	0	3	$Z_{6.8}$
Visible main rebar on the underside of the structure, the reinforcement is lightly corroded (without significant reduction of the cross section)	X	X	X	
The main rebar of the spanning structure lies in the area of carbonization and is slightly corroded (it does not apply to the prefabrication reinforcement)	1	0	3	$Z_{6.9}$
Visible main rebar on the underside of the spanning structure, corroded rebar (there is a decrease in the cross-section)	x	x	x	
Blooming (water traces) in the area of heavily corroded main reinforcement on the underside of the spanning structure (advanced reductions in cross-section)	X	X	X	
Blooming in the area of heavily corroded main reinforcement on the underside of the spanning structure (the main reinforcement is partly excluded from the load)	X	X	X	
Partial moisture penetration	0	0	2	$Z_{6.10}$
Penetration of moisture on large surfaces	0	0	3	$Z_{6.11}$
Description of damage / defect	S	V	D	
Bridges, cracks in concrete / reinforced concrete / pre-stressed structure				
Dependencies: type of construction = bridge, basic building element = structure , material of the structure = concrete, damage = cracks				
Surface cracks outside the humidification area (widths) of ≤ 0.1mm in reinforced concrete or prestressed structure (eg cracks from shrinkage)	X	X	X	
Cracks outside the area of humidification (shrinkage) width of 0.1 - <0.2mm in reinforced concrete- or prestressed structure	X	X	X	
Cracks width 0.1 - <0.2mm in the area of humidification (shrinkage) for RC structure	X	X	X	
Surface cracks in the humidification area (widths) 0.2 -≤ 0.4 mm in the RC structure	x	x	X	

Parallel cracks with prestressing of a width of 0.2 -≤ 0.4mm in the area of humidification (squeezing) in the prestressed structure	X	x	x	
Shrinkage widths> 0.4mm in the area of humidification (shrinkage) for RC structure	x	x	X	
Cracks with a width of> 0.4mm with a prestressed structure (not in the cable extension area)	x	x	X	
Cracks with a width of <0.2mm with prestressed structure (in cable extension)	X	x	x	
Cracks with a width of 0.2 - ≤ 0.4mm at the front of the structure (in the area of cable extension)	x	x	X	
Cracks with a width of> 0.4mm with a prestressed structure (in the cable extension area)	x	x	x	
Cracks> 0.4mm under load	x	x	x	

Summary Group 1

$Z_{6.1} = 1.0$	$\Delta Z_{6.1} = +0.1$	$Z_{6.1} = 1.0+0.1=1.1$
$Z_{6.2} = 1.0$	$\Delta Z_{6.2} = +0.1$	$Z_{6.2} = 1.0+0.1=1.1$
$Z_{6.3} = 1.1$	$\Delta Z_{6.3} = +0.1$	$Z_{6.3} = 1.1+0.1= 1.2$
$Z_{6.4} = 1.1$	$\Delta Z_{6.4} = -0.1$	$Z_{6.4} = 1.1-0.1= 1$
$Z_{6.5} = 1.0$	$\Delta Z_{6.5} = 0$	$Z_{6.5} = 1.0+0= 1.0$
$Z_{6.6} = 1.1$	$\Delta Z_{6.6} = +0.1$	$Z_{6.6} = 1.1+0.1= 1.2$
$Z_{6.7} = 1.1$	$\Delta Z_{6.7} = +0.1$	$Z_{6.7} = 1.1+0.1= 1.2$
$Z_{6.8} = 2.5$	$\Delta Z_{6.8} = +0.1$	$Z_{6.8} = 2.5+0.1= 2.6$
$Z_{6.9} = 2.7$	$\Delta Z_{6.9} = 0$	$Z_{6.9} = 2.7+0= 2.7$
$Z_{6.10} = 2.0$	$\Delta Z_{6.10} = 0$	$Z_{6.10} = 2.0+0= 2.0$
$Z_{6.11} = 2.5$	$\Delta Z_{6.11} = +0.1$	$Z_{6.11} = 2.5+0.1= 2.6$
Sum group 1		**17.7**

Group 2: Substructure

Bridges, substructure	S	V	D	
Dependencies: type of construction = bridge, basic building element = substructure				
Graffiti on visible surfaces	0	0	0	$Z_{6.12}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{6.13}$
Less wear of the protective layer	0	0	1	$Z_{6.14}$
Less rinses in the area of water flows	0	0	1	$Z_{6.15}$
Cleaning of the bearing bench (moldings or other)	x	x	x	
Invalidation of the benches (bird droppings or other)	x	x	x	
Remains of the formwork that press the construction	0	0	2	$Z_{6.16}$
Cleaning the bearing bench with accumulated moisture	x	x	x	
Formwork material (polystyrene) on the connection with the structure has not been removed	x	x	x	
Less dropping of stone linings	x	x	x	
The installation cover is not correct / damaged	x	x	x	
Cracking stone wall	x	x	x	
Partial moisture stone wall / reinforced concrete	0	0	2	$Z_{6.17}$
Moisture on large surfaces of stone wall / reinforced concrete	0	0	3	$Z_{6.18}$
Bridges, cracks in concrete- / RC substructure				
Dependencies: type of construction = bridge, basic building element = substructure, damage = cracks				
Dry cracks outside the humidification (spinning) area <0.2mm (no reaction	x	x	x	

sulfuric acid - RSK)				
Surface cracks in the area of humidification (squeezing), cracks can run water, width <0.2mm (without RSK) REACTION SULFURIC ACID	x	x	x	
Dry cracks outside the humidification area (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, 0.2 -≤ 0.4mm wide (without RSK)	x	x	x	
Dry cracks outside the area of humidification (shrinkage) of width> 0,4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) of width> 0.4mm (without RSK)	x	x	x	
Description of damage / defect	S	V	D	
Cracks in the area of humidification (cracks), cracks can run water, width> 0,4mm, unarmed concrete (without RSK)	x	x	x	
Cracks in the area of humidification (sprinkling), cracks can run water, width> 0.4mm, RC bottom structure (without RSK)	x	x	x	

Summary Group 2

$Z_{6.12}$= 1.0	$\Delta Z_{6.12}$ = 0	$Z_{6.12}$ = 1.0+0= 1.0
$Z_{6.13}$ = 1.0	$\Delta Z_{6.13}$ = +0.1	$Z_{6.13}$ = 1.0+0.1= 1.1
$Z_{6.14}$ = 1.1	$\Delta Z_{6.14}$ = +0.1	$Z_{6.14}$ = 1.1+0.1= 1.2
$Z_{6.15}$ = 1.1	$\Delta Z_{6.15}$ = 0	$Z_{6.15}$ = 1.1+0= 1.1
$Z_{6.16}$ = 2.0	$\Delta Z_{6.16}$ = 0	$Z_{6.16}$ = 2.0+0= 2.0
$Z_{6.17}$ = 2.0	$\Delta Z_{6.17}$ = 0	$Z_{6.17}$= 2.0+0= 2.0
$Z_{6.18}$ = 2.5	$\Delta Z_{6.18}$ = +0.1	$Z_{6.18}$= 2.0+0.1= 2.1
Sum group 2		**10.5**

Group 9: Transition devices

Transition devices (joints)	S	V	D	
Dependencies: structural element = transitional device				
Contaminated transitional device (moving still possible)	x	x	x	
A highly Contaminated transient device (scrolling limited)	x	x	x	
Loosening the fixing of the profiles in the carpet of the structures, the profile is still held	x	x	x	
Loosening the fixing of the profiles in the carpet construction, the profile loosened	x	x	x	
Rubber profile dropped or multiple damaged	x	x	x	
Asphalt crossings, thin asphalt mass (rough, small open cracks)	0	0	0	$Z_{6.19}$
The asphalt crossing cracked and depressed	0	1	2	$Z_{6.20}$
The transient device is missing, the spanning structure is cracked at the ends	x	x	X	
The transient device is missing, the spanning structure at the ends is cracked and shrugged	x	x	X	

Summary Group 9

$Z_{6.19}$ = 1.0	$\Delta Z_{6.19}$ = +0.1	$Z_{6.19}$ = 1.0+0.1= 1.1
$Z_{6.20}$= 2.1	$\Delta Z_{6.20}$ = +0.1	$Z_{6.20}$ = 2.1+0.1= 2.2
Sum group 9		**3.3**

Group 13: Fence

Protective means	S	V	D	
Fence				
Dependencies: element of construction = protective means, fence				
The compound fence - construction of the defect	0	0	1	$Z_{6.21}$
There is no fence, there are bumpers at> 50km / h, no pedestrian traffic is planned	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, small building, no pedestrian traffic is foreseen (temporary hazard sign exists)	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, pedestrian traffic planned	x	x	x	
Missing wire in the handrail of the fence, the length of the building is <20m	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, there is a bumper	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, the bumper is missing	x	x	x	
Fence height is not in accordance with regulations (difference ≤ 5cm)	x	x	x	
Fence height is not in accordance with regulations (difference 5 - 10cm)	x	x	x	
Fence height is not in accordance with regulations (difference > 10cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference ≤ 2cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference > 2cm)	x	x	x	
Missing individual fence filling rods	x	x	x	
Missing more consecutive fence filling rods	x	x	x	
Guardrail				
Dependencies: structural element = protection agent, bumper				
Bumper height is not in accordance with regulations (difference ≤ 3cm)	x	x	x	
Bumper height is not in accordance with regulations (difference > 3cm)	x	x	x	
The bumper is partially deformed	x	x	x	
a fence and a bumper are missing	x	x	x	
Weak anchor fasteners of the bumper / fence	x	x	x	
There is a lack of anchoring of protective devices on the length of the move	x	x	x	
Corrosion of protective agents				
Dependencies: structural element = protective agents, damage = surface, metal				
Insufficient thickness of the protective layer	0	0	2	$Z_{6.22}$
Local scattering (breaking) of the protective layer	0	0	1	$Z_{6.23}$
Scattering (breaking) of the protective layer on larger surfaces	x	x	x	
Local start of corrosion	0	0	1	$Z_{6.24}$
Corrosion of large surfaces	0	0	2	$Z_{6.25}$
The corrosion of individual support elements of the protecting agents	1	1	2	$Z_{6.26}$
The corrosion of more consecutive support elements of the protecting agents	x	x	x	

Summary Group 13

$Z_{6.21} = 1.1$	$\Delta Z_{6.21} = +0.1$	$Z_{6.21} = 1.1+0.1= 1.2$

$Z_{6.22}$ =2.0	$\Delta Z_{6.22}$ = +0.1	$Z_{6.22}$ = 2.0+0.1= 2.1
$Z_{6.23}$=1.1	$\Delta Z_{6.23}$ = 0	$Z_{6.23}$ = 1.1+0= 1.1
$Z_{6.24}$= 1.1	$\Delta Z_{6.24}$= 0	$Z_{6.24}$= 1.1+0= 1.1
$Z_{6.25}$= 2.0	$\Delta Z_{6.25}$= +0	$Z_{6.25}$= 2.0+0= 2.0
$Z_{6.26}$ = 2.3	$\Delta Z_{6.26}$= 0	$Z_{6.26}$ = 2.3+0= 2.3
Sum group 13		**9.8**

Group 11: Road surface

Road surface	S	V	D	
Dependencies: structural element = useful surface				
pavement				
Subsidence of the pavement behind the abutments (≤ 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm)	x	x	X	
Subsidence of the pavement behind the abutments (> 2cm), there are warning signs	x	x	X	
Drainage does not work, the risk of drifting	0	2	0	$Z_{6.27}$
Paving grooves / indentations, depth <1cm	X	x	x	
Pulley grooves / indentations, depth 1 - 3cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm, there are warning signs	x	x	x	
Pulley grooves / indentations, depth> 3cm	x	x	x	
Paving grooves / indentations, depth> 3cm, there are warning signs	x	x	x	
Bubbles, heights of ≤ 2cm	x	x	x	
Bubbles, height 2 - 5cm	x	x	x	
Bubbles, height 2 - 5cm, there are warning signs	x	x	x	
Bubbles, height > 5cm	x	x	x	
Bubbles, height > 5cm, there are warning signs	x	x	x	
Impact hole, depth ≤ 2cm	x	x	x	
Impact hole, depth 2 - 5cm	x	x	x	
Impact hole, depth 2 - 5cm, there are warning signs	x	x	x	
Impact hole, depth > 5cm	x	x	x	
Description of damages / defections	S	V	D	
Impact hole, depth> 5cm, there are warning signs	x	x	x	
A pedestrian hallway	x	x	x	
Erosion of surface layer <2cm	x	x	x	
Erosion of surface layer ≥2cm	x	x	x	
Erosion of surface layer ≥2cm, there are warning signs	x	x	x	
The layers break and fall in pieces	x	x	x	
Slipping risk	x	x	x	

Summary Group 11

$Z_{6.27}$ =2.0	$\Delta Z_{6.27}$= +0.1	$Z_{6.27}$ = 2.0+0.1= 2.1
Sum group 11		**2.1**

Group 14: Other

Signs	S	V	D	
Dependencies: structural element = Signs				

Missing building designation number	x	x	x	
The load limit sign is missing / incorrect S = 2 - 4, V = 2 - 4	x	x	x	
Drainage of the bridge				
Dependencies: element of construction = equipment, drainage of the bridge				
The fastening of the drainage pipe was corroded	x	x	x	
The fastening parts are missing, outside the traffic area	x	x	X	
The fastening parts are missing, above the traffic surface, V = 1 - 3	x	x	x	
Water leaks from the pipe (above the field)	x	x	x	
Water leaks from the pipe (above the parts of the structure)	x	x	x	
Water leaks from the pipe (above the traffic surfaces)	x	x	x	
Less corrosion damage on drainage pipes	x	x	x	
Significant corrosion damage on drainage pipes	x	x	X	
Missing dilatation of the drainage pipes at the transition of the structure / field	X	x	x	
Rain grid / clogged pipe	0	2	1	$Z_{6.28}$
In the raining grid there is a missing catcher of a garbage (pot)	X	x	x	
The drainage grid/ cleaning hole in the hinge plate is not secured	x	x	x	
The drainage grid is broken	x	x	x	
Inspection agents (inspection tools)				
Dependencies: element of construction = equipment, inspection agents				
Cracks on the rail of the inspected vehicle (risk of falling from height)	x	x	x	
Padlock is missing, third parties have unobstructed access to the building site	x	x	x	
Ladders, The distance between rungs is too large (> 280mm)	x	x	x	
Ladders, rungs too close to the building (<150mm)	x	x	x	
Ladder, The distance between the end rungs and the working surface too large (> 100mm)	x	x	x	
Ladders, according to the regulations, the necessary back protection is missing	x	x	x	
Corrosion of inspection agents				
Dependencies: element of construction = equipment, inspection means, damage = surface, metal				
Thickness of anti-corrosion protection too small	x	x	x	
Locally blown (cracked) anti-corrosion protection	x	x	x	
Anti-corrosive protection on large surfaces is blooming	x	x	x	
The beginning of corrosion - locally	x	x	x	
The beginning of corrosion - a large surface	x	x	x	
Corrosion of individual carriers	x	x	x	
Corrosion of several carriers in a row	x	x	x	
(Tools for protection) Means of protection from invading birds				
Dependencies: structural element = protective agents, protection from birds				
protection agents of invading birds are missing / damaged	x	x	x	
Overview of the building site				
Dependencies: damage = review				

The view of the whole building is not possible? a fictitious assessment of the situation is issued	x	x	x	
The building is very overgrown, only partial inspection possible	0	0	2	$Z_{6.29}$

Summary Group 14

$Z_{6.28}$ =2.1	$\Delta Z_{6.28}$= +0.1	$Z_{6.28}$ = 2.1+0.1= 2.2
$Z_{6.29}$ =2.0	$\Delta Z_{6.29}$=0	$Z_{6.29}$ = 2.0+0= 2.0
Sum group 14		**4.2**

6.2. LEVEL 2: MAXIMUM DAMAGE

NO.	Group1	Group2	Group9	Group13	Group11	Group14
	17.7	10.5	3.3	9.8	2.1	4.2

6.3. LEVEL 3:

Group	Z	ΔZ	**Z_{BG}**
Group 1	$Z_{6.9}$ = 2.7	ΔZ_2 = -0.1	Z_{BG}= 2.7 - 0.1= 2.6
Group 2	$Z_{6.18}$ = 2.1	ΔZ_2 = 0	Z_{BG}= 2.1 +0 = 2.1
Group 9	$Z_{6.20}$ = 2.2	ΔZ_2 = 0	Z_{BG}= 2.2 + 0= 2.2
Group 11	$Z_{6.27}$ = 2.1	ΔZ_2= +0.1	Z_{BG}= 2.1 + 0.1= 2.2
Group 13	$Z_{6.26}$= 2.3	ΔZ_2 = 0	Z_{BG}= 2.3 + 0 = 2.3
Group 14	$Z_{6.28}$ = 2.2	ΔZ_2 = 0	Z_{BG}= 2.2 + 0 = 2.2

Z_{ges}= 2.6 ΔZ_3 = - 0.1 (GROUP 1 THE MAXIMUM Z_{BG})

Sufficient condition of the bridge structure

The stability of the structure is ensured.

The traffic safety of the structure may be impaired.

The stability and/or durability of at least one component group can be impaired.

The durability of the structure may be affected. A spread of damage or consequential damage to the structure, which in the medium term leads to significant impairments to stability and/or traffic safety or increased wear, is then to be expected.

Ongoing maintenance required.

Short-term repair required.

Measures to eliminate damage or warnings to maintain road safety may be necessary at short term.

7. ABDUL SALAM AREF BRIDGE

7.1. LEVEL 1: REGULAR BRIDGE INSPECTION

Group 1: Superstructure

Description of damage and defects	S	V	D	
Bridges, concrete / reinforced concrete / pre-stressed construction structures				
Dependencies: type of construction = bridge, basic building element = structure construction, material of the spanning structure = concrete				
Graffiti on visible surfaces	0	0	0	$Z_{7.1}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{7.2}$
Less wear of the protective layer	0	0	1	$Z_{7.3}$
The rust on the lower sides of the construction	0	0	1	$Z_{7.4}$
Pollution of internal passages of the building (remains of the formwork or other)	0	0	0	$Z_{7.5}$
Pollution of internal passages of building (bird feces or other)	x	x	x	x
Coarse granularity of concrete of the spanning structure	0	0	1	$Z_{7.6}$
The protective layer above the auxiliary rebar for the installation of the main reinforcement is too small	0	0	1	$Z_{7.7}$
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) Good quality of concrete	X	X	X	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) poor quality of concrete	X	X	X	
The protective layer of the main rebar on the underside of the structure is too small (1.0 to 2.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1,0 to 2,9cm) Poor concrete quality	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (below 1.0cm)	x	x	x	
The carbonate front reached the main rebar	0	0	3	$Z_{7.8}$
Visible main rebar on the underside of the structure, the reinforcement is lightly corroded (without significant reduction of the cross section)	x	x	x	
The main rebar of the spanning structure lies in the area of carbonization and is slightly corroded (it does not apply to the prefabrication reinforcement)	x	x	x	
Visible main rebar on the underside of the spanning structure, corroded rebar (there is a decrease in the cross-section)	1	0	3	$Z_{7.9}$
Blooming (water traces) in the area of heavily corroded main reinforcement on the underside of the spanning structure (advanced reductions in cross-section)	x	x	x	
Blooming in the area of heavily corroded main reinforcement on the underside of the spanning structure (the main reinforcement is partly excluded from the load)	x	x	x	
Partial moisture penetration	0	0	2	$Z_{7.10}$
Penetration of moisture on large surfaces	0	0	3	$Z_{7.11}$
Description of damage / defect	S	V	D	
Bridges, cracks in concrete / reinforced concrete / pre-stressed structure				
Dependencies: type of construction = bridge, basic building element = structure , material of the structure = concrete, damage = cracks				
Surface cracks outside the humidification area (widths) of ≤ 0.1mm in reinforced concrete or prestressed structure (eg cracks from shrinkage)	x	x	x	
Cracks outside the area of humidification (shrinkage) width of 0.1 - <0.2mm in reinforced concrete- or prestressed structure	x	x	x	
Cracks width 0.1 - <0.2mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Surface cracks in the humidification area (widths) 0.2 -≤ 0.4 mm in the RC structure	x	x	x	

Parallel cracks with prestressing of a width of 0.2 -≤ 0.4mm in the area of humidification (squeezing) in the prestressed structure	x	x	x	
Shrinkage widths> 0.4mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (not in the cable extension area)	x	x	x	
Cracks with a width of <0.2mm with prestressed structure (in cable extension)	x	x	x	
Cracks with a width of 0.2 - ≤ 0.4mm at the front of the structure (in the area of cable extension)	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (in the cable extension area)	x	x	x	
Cracks> 0.4mm under load	x	x	x	

Summary Group 1

$Z_{7.1} = 1.0$	$\Delta Z_{7.1} = +0.1$	$Z_{7.1} = 1.0+0.1=1.1$
$Z_{7.2} = 1.0$	$\Delta Z_{7.2} = +0.1$	$Z_{7.2} = 1.0+0.1=1.1$
$Z_{7.3} = 1.1$	$\Delta Z_{7.3} = +0.1$	$Z_{7.3} = 1.1+0.1= 1.2$
$Z_{7.4} = 1.1$	$\Delta Z_{7.4} = -0.1$	$Z_{7.4} = 1.1-0.1= 1$
$Z_{7.5} = 1.0$	$\Delta Z_{7.5} = 0$	$Z_{7.5} = 1.0+0= 1.0$
$Z_{7.6} = 1.1$	$\Delta Z_{7.6} = +0.1$	$Z_{7.6} = 1.1+0.1= 1.2$
$Z_{7.7} = 1.1$	$\Delta Z_{7.7} = +0.1$	$Z_{7.7} = 1.1+0.1= 1.2$
$Z_{7.8} = 2.5$	$\Delta Z_{7.8} = +0.1$	$Z_{7.8} = 2.5+0.1= 2.6$
$Z_{7.9} = 2.7$	$\Delta Z_{7.9} = +0.1$	$Z_{7.9} = 2.7+0.1= 2.8$
$Z_{7.10} = 2.0$	$\Delta Z_{7.10} = 0$	$Z_{7.10} = 2.0+0= 2.0$
$Z_{7.11} = 2.5$	$\Delta Z_{7.11} = +0.1$	$Z_{7.11} = 2.5+0.1= 2.6$
Sum group 1		**17.8**

Group 2: Substructure

Bridges, substructure	S	V	D	
Dependencies: type of construction = bridge, basic building element = substructure				
Graffiti on visible surfaces	0	0	0	$Z_{7.12}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{7.13}$
Less wear of the protective layer	0	0	1	$Z_{7.14}$
Less rinses in the area of water flows	0	0	1	$Z_{7.15}$
Cleaning of the bearing bench (moldings or other)	x	x	x	
Invalidation of the benches (bird droppings or other)	x	x	x	
Remains of the formwork that press the construction	0	0	2	$Z_{7.16}$
Cleaning the bearing bench with accumulated moisture	x	x	x	
Formwork material (polystyrene) on the connection with the structure has not been removed	x	x	x	
Less dropping of stone linings	x	x	x	
The installation cover is not correct / damaged	x	x	x	
Cracking stone wall	x	x	x	
Partial moisture stone wall / reinforced concrete	x	x	x	
Moisture on large surfaces of stone wall / reinforced concrete	x	x	X	
Bridges, cracks in concrete- / RC substructure				

Dependencies: type of construction = bridge, basic building element = substructure, damage = cracks				
Dry cracks outside the humidification (spinning) area <0.2mm (no reaction sulfuric acid - RSK)	x	x	x	
Surface cracks in the area of humidification (squeezing), cracks can run water, width <0.2mm (without RSK) REACTION SULFURIC ACID	x	x	x	
Dry cracks outside the humidification area (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, 0.2 -≤ 0.4mm wide (without RSK)	x	x	x	
Dry cracks outside the area of humidification (shrinkage) of width> 0,4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) of width> 0.4mm (without RSK)	x	x	x	
Description of damage / defect	S	V	D	
Cracks in the area of humidification (cracks), cracks can run water, width> 0,4mm, unarmed concrete (without RSK)	x	x	x	
Cracks in the area of humidification (sprinkling), cracks can run water, width> 0.4mm, RC bottom structure (without RSK)	x	x	x	

Summary Group 2

$Z_{7.12}$= 1.0	$\Delta Z_{7.12}$ = 0	$Z_{7.12}$ = 1.0+0= 1.0
$Z_{7.13}$ = 1.0	$\Delta Z_{7.13}$ = +0.1	$Z_{7.13}$ = 1.0+0.1= 1.1
$Z_{7.14}$ = 1.1	$\Delta Z_{7.14}$ = +0.1	$Z_{7.14}$ = 1.1+0.1= 1.2
$Z_{7.15}$ = 1.1	$\Delta Z_{7.15}$ = 0	$Z_{7.15}$ = 1.1+0= 1.1
$Z_{7.16}$ = 2.0	$\Delta Z_{7.16}$ =- 0.1	$Z_{7.16}$ = 2.0-0.1= 1.9
Sum group 2		**6.3**

Group 9: Transition devices

Transition devices (joints)	S	V	D	
Dependencies: structural element = transitional device				
Contaminated transitional device (moving still possible)	x	x	x	
A highly Contaminated transient device (scrolling limited)	x	x	x	
Loosening the fixing of the profiles in the carpet of the structures, the profile is still held	x	x	x	
Loosening the fixing of the profiles in the carpet construction, the profile loosened	x	x	x	
Rubber profile dropped or multiple damaged	x	x	x	
Asphalt crossings, thin asphalt mass (rough, small open cracks)	0	0	0	$Z_{7.17}$
The asphalt crossing cracked and depressed	0	1	2	$Z_{7.18}$
The transient device is missing, the spanning structure is cracked at the ends	x	x	x	
The transient device is missing, the spanning structure at the ends is cracked and shrugged	x	x	x	

Summary Group 9

$Z_{7.17}$ = 1.0	$\Delta Z_{7.17}$ = +0.1	$Z_{7.17}$ = 1.0+0.1= 1.1
$Z_{7.18}$= 2.1	$\Delta Z_{7.18}$ = +0.1	$Z_{7.18}$ = 2.1+0.1= 2.2

Sum group 9	3.3

Group 13: Fence

Protective means	S	V	D	
Fence				
Dependencies: element of construction = protective means, fence				
The compound fence - construction of the defect	0	0	1	$Z_{7.19}$
There is no fence, there are bumpers at> 50km / h, no pedestrian traffic is planned	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, small building, no pedestrian traffic is foreseen (temporary hazard sign exists)	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, pedestrian traffic planned	x	x	x	
Missing wire in the handrail of the fence, the length of the building is <20m	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, there is a bumper	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, the bumper is missing	x	x	x	
Fence height is not in accordance with regulations (difference ≤ 5cm)	x	x	x	
Fence height is not in accordance with regulations (difference 5 - 10cm)	x	x	x	
Fence height is not in accordance with regulations (difference > 10cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference ≤ 2cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference > 2cm)	x	x	x	
Missing individual fence filling rods	x	x	x	
Missing more consecutive fence filling rods	x	x	x	
Guardrail				
Dependencies: structural element = protection agent, bumper				
Bumper height is not in accordance with regulations (difference ≤ 3cm)	x	x	x	
Bumper height is not in accordance with regulations (difference > 3cm)	x	x	x	
The bumper is partially deformed	x	x	x	
a fence and a bumper are missing	x	x	x	
Weak anchor fasteners of the bumper / fence	x	x	x	
There is a lack of anchoring of protective devices on the length of the move	x	x	x	
Corrosion of protective agents				
Dependencies: structural element = protective agents, damage = surface, metal				
Insufficient thickness of the protective layer	0	0	2	$Z_{7.20}$
Local scattering (breaking) of the protective layer	0	0	1	$Z_{7.21}$
Scattering (breaking) of the protective layer on larger surfaces	x	x	x	
Local start of corrosion	0	0	1	$Z_{7.22}$
Corrosion of large surfaces	0	0	2	$Z_{7.23}$
The corrosion of individual support elements of the protecting agents	1	1	2	$Z_{7.24}$
The corrosion of more consecutive support elements of the protecting agents	x	x	x	

Summary Group 13

$Z_{7.19}$ = 1.1	$\Delta Z_{7.19}$ = +0.1	$Z_{7.19}$ = 1.1+0.1= 1.2
$Z_{7.20}$ =2.0	$\Delta Z_{7.20}$ = +0.1	$Z_{7.20}$ = 2.0+0.1= 2.1
$Z_{7.21}$=1.1	$\Delta Z_{7.21}$ =+ 0.1	$Z_{7.21}$ = 1.1+0.1= 1.2
$Z_{7.22}$= 1.1	$\Delta Z_{7.22}$= 0	$Z_{7.22}$= 1.1+0= 1.1
$Z_{7.23}$= 2.0	$\Delta Z_{7.23}$= +0	$Z_{7.23}$= 2.0+0= 2.0
$Z_{7.24}$ = 2.3	$\Delta Z_{7.24}$= 0	$Z_{7.24}$ = 2.3+0= 2.3
Sum group 13		**9.9**

Group 11: Road surface

Road surface	S	V	D	
Dependencies: structural element = useful surface				
pavement				
Subsidence of the pavement behind the abutments (≤ 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm)	x	x	X	
Subsidence of the pavement behind the abutments (> 2cm), there are warning signs	x	x	X	
Drainage does not work, the risk of drifting	0	2	0	$Z_{7.25}$
Paving grooves / indentations, depth <1cm	X	x	x	
Pulley grooves / indentations, depth 1 - 3cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm, there are warning signs	x	x	x	
Pulley grooves / indentations, depth> 3cm	x	x	x	
Paving grooves / indentations, depth> 3cm, there are warning signs	x	x	x	
Bubbles, heights of ≤ 2cm	x	x	x	
Bubbles, height 2 - 5cm	x	x	x	
Bubbles, height 2 - 5cm, there are warning signs	x	x	x	
Bubbles, height > 5cm	x	x	x	
Bubbles, height > 5cm, there are warning signs	x	x	x	
Impact hole, depth ≤ 2cm	x	x	x	
Impact hole, depth 2 - 5cm	x	x	x	
Impact hole, depth 2 - 5cm, there are warning signs	x	x	x	
Impact hole, depth > 5cm	x	x	x	
Description of damages / defections	S	V	D	
Impact hole, depth> 5cm, there are warning signs	x	x	x	
A pedestrian hallway	x	x	x	
Erosion of surface layer <2cm	x	x	x	
Erosion of surface layer ≥2cm	x	x	x	
Erosion of surface layer ≥2cm, there are warning signs	x	x	x	
The layers break and fall in pieces	x	x	x	
Slipping risk	x	x	x	

Summary Group 11

$Z_{7.25}$ =2.0	$\Delta Z_{7.25}$= +0.1	$Z_{7.25}$ = 2.0+0.1= 2.1
Sum group 11		**2.1**

Group 14: Other

Signs	S	V	D	
Dependencies: structural element = Signs				
Missing building designation number	x	x	x	
The load limit sign is missing / incorrect S = 2 - 4, V = 2 - 4	x	x	x	
Drainage of the bridge				
Dependencies: element of construction = equipment, drainage of the bridge				
The fastening of the drainage pipe was corroded	x	x	x	
The fastening parts are missing, outside the traffic area	x	x	X	
The fastening parts are missing, above the traffic surface, V = 1 - 3	x	x	x	
Water leaks from the pipe (above the field)	x	x	x	
Water leaks from the pipe (above the parts of the structure)	0	0	2	$Z_{7.26}$
Water leaks from the pipe (above the traffic surfaces)	x	x	x	
Less corrosion damage on drainage pipes	x	x	x	
Significant corrosion damage on drainage pipes	x	x	X	
Missing dilatation of the drainage pipes at the transition of the structure / field	X	x	x	
Rain grid / clogged pipe	0	2	1	$Z_{7.27}$
In the raining grid there is a missing catcher of a garbage (pot)	X	x	x	
The drainage grid/ cleaning hole in the hinge plate is not secured	x	x	x	
The drainage grid is broken	x	x	x	
Inspection agents (inspection tools)				
Dependencies: element of construction = equipment, inspection agents				
Cracks on the rail of the inspected vehicle (risk of falling from height)	x	x	x	
Padlock is missing, third parties have unobstructed access to the building site	x	x	x	
Ladders, The distance between rungs is too large (> 280mm)	x	x	x	
Ladders, rungs too close to the building (<150mm)	x	x	x	
Ladder, The distance between the end rungs and the working surface too large (> 100mm)	x	x	x	
Ladders, according to the regulations, the necessary back protection is missing	x	x	x	
Corrosion of inspection agents				
Dependencies: element of construction = equipment, inspection means, damage = surface, metal				
Thickness of anti-corrosion protection too small	x	x	x	
Locally blown (cracked) anti-corrosion protection	x	x	x	
Anti-corrosive protection on large surfaces is blooming	x	x	x	
The beginning of corrosion - locally	x	x	x	
The beginning of corrosion - a large surface	x	x	x	
Corrosion of individual carriers	x	x	x	
Corrosion of several carriers in a row	x	x	x	
(Tools for protection) Means of protection from invading birds				
Dependencies: structural element = protective agents, protection from birds				
protection agents of invading birds are missing / damaged	x	x	x	

Overview of the building site				
Dependencies: damage = review				
The view of the whole building is not possible? a fictitious assessment of the situation is issued	x	x	x	
The building is very overgrown, only partial inspection possible	0	0	2	$Z_{7.28}$

Summary Group 14

$Z_{7.26}$ =2.0	$\Delta Z_{7.26}$= +0.1	$Z_{7.26}$ = 2.0+0.1= 2.1
$Z_{7.27}$ =2.1	$\Delta Z_{7.27}$= +0.1	$Z_{7.27}$ = 2.1+0.1= 2.2
$Z_{7.28}$ =2.0	$\Delta Z_{7.28}$=0	$Z_{7.28}$ = 2.0+0= 2.0
Sum group 14		**6.3**

7.2. LEVEL 2: MAXIMUM DAMAGE

NO.	Group1	Group2	Group9	Group13	Group11	Group14
	17.8	6.3	3.3	9.9	2.1	6.3

7.3. LEVEL 3:

Group	Z	ΔZ	**Z_{BG}**
Group 1	$Z_{7.9}$ = 2.8	ΔZ_2 = +0.1	Z_{BG}= 2.8 + 0.1= 2.9
Group 2	$Z_{7.16}$ = 1.9	ΔZ_2 = -0.1	Z_{BG}= 1.9 - 0.1= 1.8
Group 9	$Z_{7.18}$ = 2.2	ΔZ_2 = 0	Z_{BG}= 2.2 + 0= 2.2
Group 11	$Z_{7.25}$ = 2.1	ΔZ_2= +0.1	Z_{BG}= 2.1 + 0.1= 2.2
Group 13	$Z_{7.24}$= 2.3	ΔZ_2 = 0	Z_{BG}= 2.3 + 0= 2.3
Group 14	$Z_{7.27}$ = 2.2	ΔZ_2 = +0.1	Z_{BG}= 2.2 + 0.1= 2.3

Z_{ges}= 2.9 ΔZ_3 = +0.1 (GROUP 1 THE MAXIMUM Z_{BG})

Sufficient condition of the bridge structure

The stability of the structure is ensured.

The traffic safety of the structure may be impaired.

The stability and/or durability of at least one component group can be impaired.

The durability of the structure may be affected. A spread of damage or consequential damage to the structure, which in the medium term leads to significant impairments to stability and/or traffic safety or increased wear, is then to be expected.

Ongoing maintenance required.

Short-term repair required.

Measures to eliminate damage or warnings to maintain road safety may be necessary at short term.

8. RANKING OF ANALYSED BRIDGES BEFORE REPAIR

Through the evaluation of the seven bridges before the repair, it was found that the most damaged bridges are:

- Al Seeka Road Bridge Damage Rate Was 2.9
- Bab Bin Gheshir Road Bridge Damage Rate Was 2.9
- Abdul Salam Aref Bridge Damage Rate Was 2.9

As these bridges need the priority of repair because of the great damage in them compared to the rest of the bridges that have less damage.

Because the presence of this damage affects the traffic safety of the structure, as well as weakens the bladder of the structure.

Then the bridges are arranged in terms of percentage of damage and repairs are carried out after prioritizing maintenance to the previous bridges:

- Souk Athultha 1 Bridge damage rate was 2.8
- Souk Athultha 2 Bridge damage rate was 2.8
- Al sreem road bridge damage rate was 2.6
- Al shaab port bridge damage rate was 2.6

According to the calculated rating all bridges have same damage category and belong to the group of structures with „sufficient condition“ (2,5-2,9), for which the following description is given in german BMS:

- The stability of the structure is ensured.
- The traffic safety of the structure may be impaired.
- The stability and/or durability of at least one component group can be impaired.
- The durability of the structure may be affected. A spread of damage or consequential damage to the structure, which in the medium term leads to significant impairments to stability and/or traffic safety or increased wear, is then to be expected.
- Ongoing maintenance required.
- Short-term repair required.
- Measures to eliminate damage or warnings to maintain road safety may be necessary at short term.

CHAPTER VIII

REPAIR MEASURES OF 7 BRIDGES IN TRIPOLI

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REPAIR MEASURES OF 7 BRIDGES IN TRIPOLI

1. SOUK ATHULATHA 1 BRIDGE

1.1. Introduction

Through the assessment of the condition of structural elements of Souk Athulatha 1 Bridge the next conclusions have been derived:

- The characteristic defect is insufficient concrete cover depth, mostly within 10mm of depth.
- The characteristic damages of RC elements are reinforcement corrosion and loss of concrete cover.
- The main cause of described damages is carbonization of concrete and insufficient concrete cover depth.

In some cases, the bad quality of works during the bridge construction and local penetration of water also contributed to the development of bars corrosion.

The degree of observed damages is not dangerous for stability and bearing capacity of structures, but reduces durability and functionality. The most damaged elements have been edge beams, and they have to be removed.

Whereas the stability and bearing capacity of bridge have not been jeopardized the most suggested repair measures belong to the group of non-structural repair and surface protection.

As the carbonization and insufficient concrete cover present general problem, all elements which have been affected by carbonization or have insufficient concrete cover depth, have to be repaired. These elements are:

- Cantilever slabs with edge beams.
- Lateral beams.
- Arch slab (ceiling).
- Interior walls.
- Exterior walls.
- Underpass ceiling.

Repair measures include removing the old cement plaster and the old carbonized concrete cover and execution of new cover with increased thickness. Repair measures also include cleaning, protection or even replacement of corroded bars.

For better bonding between old, but healthy concrete and new cover the special agents is proposed. Also, the special coatings are suggested for concrete surface protection.

The most damaged elements are Cantilever slabs and edge beams that belong to cantilever slabs.

Since edge beams have small cross section and described damages cover most part of cross section, it has been decided to remove whole these elements.

During removing the old ordinary cement mortar, it was noticed that surface of concrete was very porous and full of voids and pockets. Because of such bad condition of protecting cover, it was decided to remove complete concrete cover from all investigated elements (cantilever slabs, arch ceiling, interior and exterior walls and underpass ceiling).

Repair measure of bridges in Tripoli in 2009.

For increasing of longevity of this bridge the next repair measures were recommended:

- Complete removal and execution of new RC edge beams,
- Complete concrete cover removal,
- Complete rebar treatment (even including rebar replacement)
- Rebar protection
- Bonding slurry application
- Execution of new cover with minimum depth of 20mm.
- Corrosion inhibition impregnation of all bridge exposed surfaces with 0.5 kg/m^2 impregnation material.
- Surface protection with coating of all repaired concrete elements.
- Hereinafter all recommended measures will be described in detail.

1.2. Damaged concrete and plaster removal

As it was recommended, all plaster layer and poor-quality concrete cover or/and carbonated part of concrete have to be removed. For that operation the electric hammers with max weight of 6kg have been chosen.

According with the testing results the major problem was carbonization within depths of 40 to 60 mm, frequently overpassing the rebar plan.

In some cases, the removal of concrete with 70mm depth has been suggested to be ensure that all damaged concrete is removed and all rebars are treated (10mm covering + 10mm strips+ 32mm rebars+ 20mm over rebar plan).

During the removal of concrete cover, a special attention has to be pay to imbedded reinforcement bars, as the impact method is chosen for this operation. The removal of concrete close to bars has to be done very carefully to avoid the damage of bars.

After finishing concrete removal all surfaces must be clean with water jet with pressure of 200bar.The application of water jet has to be with angle of 450 and with 5 cm distance.

The removal of edge beam is illustrated on Fig. (VIII-1)

Figure VIII-1. Removal of “old” edge beam

The removal of old carbonated concrete cover from exterior wall and ceiling beam is shown on the next two pictures.

Figure (VIII-2) Damaged concrete removal from exterior walls

Figure (VIII-3): Damaged concrete removal from ceiling beams

Figure (VIII-4): Damaged concrete removal from tunnel ceiling

Figure (VIII-5): Cleaning the surface of concrete after removing concrete cover by water jet machine

Apart from removing concrete cover from upper mentioned elements, removal works were carried out on next elements:

- Removed all old side walk over the bridge (Figure VIII-6)
- Removed all old concrete under old fences

Figure (VIII-6): Sidewalk on bridge

1.3. Cleaning of reinforcement bars

After removing of “old” damaged concrete cover, the next operation has been cleaning of reinforcement bars from remains of hardened cement paste and rust as products of steel corrosion. For this operation the water sand blasting technic is selected with pressure of 250 bar. This technic represents innovated version that is not harmful for workers. The application of mix water-send jet has to be with angle of 450 and with 5 cm distance. The requested steel surface quality according ISO 8501 was Sa2 grade (surface preparation grade) as it is shown on Figure (VIII-7)

Figure (VIII-7): Requested visual cleanliness of bars – preparation grades Sa 2

Figure (VIII-8) show the operation of cleaning bars with mixed water-sand blasting method.

Figure (VIII-8): Rebars cleaning with water-sand blasting in deck ceiling slab

Figure (VIII-9): Rebars cleaning with water-sand blasting on the interior walls

Figure (VIII-10): The view of reinforcement rods after cleaning by mixed water- sand blasting method

1.4. Reinforcement rebar replacement

If rebar loses more than 28% of the area it must be complemented, if it loses more than 50 % it must be replaced.

All complementation rebars must have sufficient anchoring length. In situation when there is not enough space for anchoring rebars, they have to be welded to existing bars.

Rods can also be replaced when there is not enough space to complement.

The criteria to define the complement of rebar are:

- When analyzed the bending rebar reinforcement area of a concrete element is reduced more than 20%.
- If any individual steel reinforcement rebar have aloss in area section for more than 28%,or as the same the individual diameter is less than 85% of the original diameter.

- If the loss in section is greater than 50% is advisable to remove the corroded rebar and replace by a new one.

The calculation of lacking reinforcement area can be done according with detail given in Fig (VIII-11) where is: As – original section/area, As,corr – the section of rebar with corrosion, As, ref – new, complemented section/area

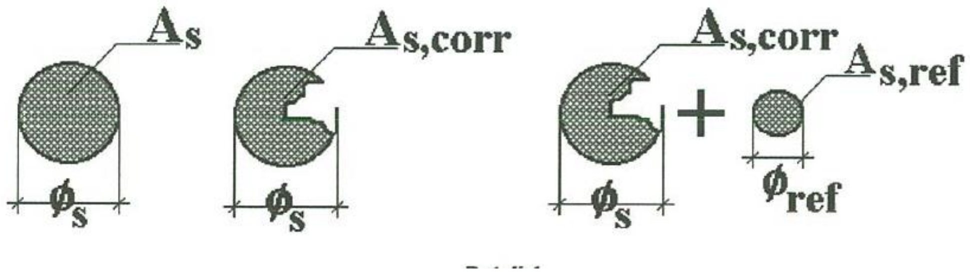


Figure (VIII-11): Calculation of lacking reinforcement area

The data of lose cross section of corroded rebars and about complemented or replacement rebar are given in table VIII-1.

Table VIII-1: rebar complementation selection Lose of section ≤28%

Φ Rebar mm	A section cm^2	A with section loss cm^2	Φrebars with corrosion mm	Δ complementation section cm^2	Φof complemented/replacement rebars mm
6	0.28	0.20	5.05	0.08	6
8	0.50	0.36	6.77	0.14	6
10	0.79	0.57	8.52	0.22	6
12	1.13	0.81	10.16	0.32	8
16	2.01	1.45	13.59	0.56	10
20	3.14	2.26	16.96	0.88	12
25	4.91	3.54	21.23	1.37	16
32	8.04	5.79	27.15	2.25	20
28%<Lose of section ≤ 50%					
6	0.28	0.14	4.22	0.14	6
8	0.50	0.25	5.64	0.25	6
10	0.79	0.40	7.14	0.39	8
12	1.13	0.57	8.52	0.56	10
16	2.01	1.01	11.34	1.00	12
20	3.14	1.57	14.14	1.57	16
25	4.91	2.46	17.70	2.45	20
32	8.04	4.02	22.62	4.02	25
Lose section>50%					
6	0.28	0.07	2.99	0.21	6
8	0.50	0.13	4.07	0.37	8

10	0.79	0.20	5.05	0.59	10
12	1.13	0.28	5.97	0.85	12
16	2.01	0.50	7.98	1.51	16
20	3.14	0.79	10.03	2.35	20
25	4.91	1.23	12.51	3.68	25
32	8.04	2.01	16.00	6.03	32

1.5. Rebar protection

After cleaning the rebars must be treated with cementitious base protection material. The consumption of protection material to ensure the needed preservation of the rebars is 3.0kg/m^2 (2 layers each 1mm depth) and will be applied in all rebars, existing and new.

Photos (VIII-12, VIII-13, and VIII-14) illustrate the process of treatment rebars with cementitious base protection material and view of protected rebars.

Figure (VIII-12) Application of cementitious base material for protection of rebars

Figure (VIII-13) Covering the rebars on ceiling beams with cementitious base protection

Figure (VIII-14) Rebars protective coating and leveling rips of wood ready for application of cementitious base special mortar

1.6. Bonding agent application

Prior to the restoring the original concrete it is recommendable to apply a bonding agent on prepared old concrete surface. As the bonding agent contains polymer binder it is strongly recommended to apply this material just before repair mortar application, to avoid bonding agent drying and hardening. Estimated consumption of bonding agent is 3kg/m^2.

1.7. Concrete restoration

For concrete restoration and for compensation of missing concrete cover the spatial type of repair mortar on cement base is chosen. Selected repair mortar should has next properties: strength class ≥45MPa, low shrinkage, good adhesion ≥2MPa, lower modulus of elasticity (≥20GPa) and good carbonation resistance.

For applying repair mortar, the method of spraying of repair mortar is suggested. This method has been selected as large surfaces have to be restored. Hence the new mortar cover sometimes overpasses 70mm it strongly recommended to apply mortar in min 2 layers to avoid drop of the fresh mortar, especially from celling.

The curing procedure has to be start just after finishing the surface of the new mortar layer, but not later than 1 hour from the mortar application. The spatial agent for curing concrete or mortar surfaces is selected therefore ceilings and walls are not suitable for water curing. The selected agent will be applied by spray technic and will form a thin film which will prevent water vapor from repair mortar.

Photos (VIII-15, VIII-16, VIII-17, and VIII-18) illustrate procedures of applying and finishing of new mortar layer on ceiling and on walls.

Figure (VIII-15) cementitious base special mortar on deck ceiling beams

Figure (VIII-16) Deck ceiling slab with special mortar

Figure (VIII-17) cementitious base special mortar on exterior walls

Figure (VIII-18) Levelling on the interior walls

1.8. Corrosion inhibition by impregnation

For protection of all imbedded rebars, especially those which were not protected by cementious slurry, the impregnation by corrosion inhibition agent is proposed. This agent will be applied by low pressure spraying up to the saturation of concrete or mortar. Estimated consumption of impregnation material is about 0.5 kg/m^2.

1.9. Surface protection

For prolong the durability of repaired bridge it is advised to protect surface of concrete/repair mortar by applying surface coating. This coating will make continuous protective layer against carbonation and capillary water absorption. The acrylate resin base material is chosen for this purpose. Selected coating should has following properties: permeability to CO2 (Sd>50m), permeability to water vapor Class II, capillary water absorption (w<0,1kg/m^{2}h0,5), adhesion by pull-off test (≥1,5 MPa). Selected protecting coating should be applied after finishing curing of concrete/repair mortar. For applying coating, the roller or brush can be used.

Application of protecting coating is shown in Figure (VIII-19)

Figure (VIII-19) Application of protecting coating on ceiling by roller

1.10. Materials recommended for repair of the bridge

After analyzing specifications of materials for structural and non-structural repair, principles and methods of repair and available materials in Libya, the next products have been selected:

Cementitious base material for rebar protection:	EmacoNanocreteAP(BASF)
Bond material:	EmacoNanocrete AP(BASF)
Cementitious base pre-mixed mortar:	EmacoNanocreteR4(BASF)
Currying protection material:	Masterkure 181(BASF)
Corrosion inhibition impregnation:	SikaFerroguard 903(SIKA)
Surface protective and paint layer:	Sikaguard 680 ES Betoncolor
Reinforcement rebar steel:	Grade A-615

The estimated quantities of listed products after visual inspection, after plaster removal and additional testing of materials and their real consumption are given in Table VIII-2.

Table VIII-2 - Calculated quantities of used products and their real consumption

Material	Estimated	Calculated after additional testing	Real quantities
EmacoNanocrete AP	570kg	4.530kg	1.800kg
EmacoNanocrete R4	19.225kg	142.800kg	117.713kg
Master flow 928/980	-	9.828kg	14.000kg
Mastercure 181	200Lt	600Lt	400Lt
SikaFerroguard 903	-	875Lt	650Lt
Sikaguard 680 ES Betoncolor	525Lt	525Lt	615Lt
Sikadur 31CF	-	5kg	-
Sikadur 52	-	42kg	-

In table (VIII-3) the real consumption of used products per measuring unit is given.

Table VIII-3 - The real consumption of used products per measuring unit (ratio)

Quantity ratios					
Work	Material	Unit	Qty	Ratio	Average thickness
Concrete repair	EmacoNanocrete R4	1.461,76m^2	117.713kg	80.53kg/m^2	42mm
	EmacoNanocreteAP	1.461,76m^2	1.800kg	1.23kg/m^2	-
	Master flow 928/980	78ml	14.000kg	179.5kg/ml	-

Another repair works:

Other repair works with aim to provide functionality involve procedures such as:

- Re-plaster and paint the surrounding walls of the bridge,
- Repair or replacement of sidewalks and curbstone,
- Removal of old fences and assemblage of new one,
- Installation of new guard rails and catch pits,
- Execution of new asphalt layers and
- Assemblage a new traffic signs, electrical lights…etc.

Also, it has been decided to add the protective concrete layers, called “shoulder”, on the side of exterior wall towards the traffic lanes.

As the bridges Souk Athulatha 1 and 2 are near each other, it was decided to repair both bridges and the road between them at the same time (Fig VI-20). Since the other repair measures are similar for both bridges and road between them, they will not be described separately. Hereinafter the other repair works are described and illustrated.

Figure (VIII-20) Site plan of bridges Souk Athulatha 1 and 2

During the repair of these bridges, it was decided to remove all sidewalks and part of curbstone and old concrete under the fences over the bridges Souk Athulatha1 and 2 and between these bridges (Fig VIII-21). After repairing the edge beams and other structural elements of bridge and preparing sub base between the bridges, it has been planned to cast new sidewalks and curbstone (FigVIII-22, VIII-23).

Figure (VIII-21) Removing of old sidewalks on bridge and between them

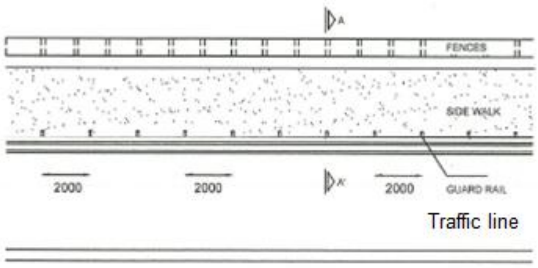


Figure (VIII-22) Sidewalk elements arrangement

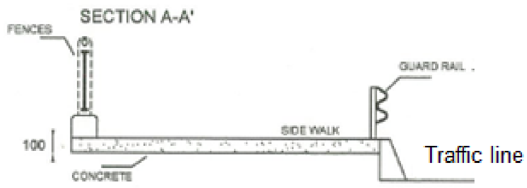


Figure (VIII-23) Cross section of sidewalk elements

This operation includes following steps:

- Concreting of beam under side walk and lining of sub base layer,
- Concreting of the sidewalk slabs with dimensions 1.20m×2.50mx0.1m on top of concrete beam, with 2cm expansion joint filled with elastic material.
- Concreting of the layer under the fences with the height 10cm above the sidewalk level.
- Castingthe new lean concrete under the new curbstone with a 30cm width.
- Concreting of new curbstone with expansion joint every 12m.

- Concreting and installing ofthe new catch pit.

The cross section of new sidewalks between the bridges with all necessary elements is shown in Fig (VIII-24)

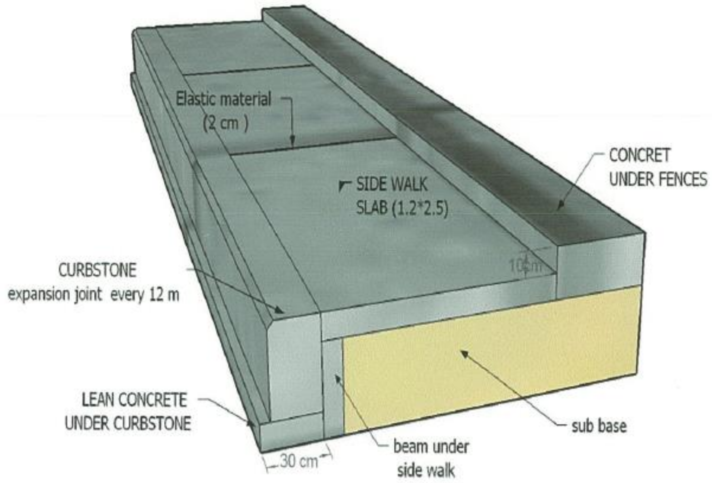


Figure (VIII-24). New sidewalk between the bridges Souk Athulatha1 and 2

Figures (VIII-25, VIII-26) show the detail of reinforcement of curbstone, the plan of installing the new guard rail and the view and plan of installing of new fence.

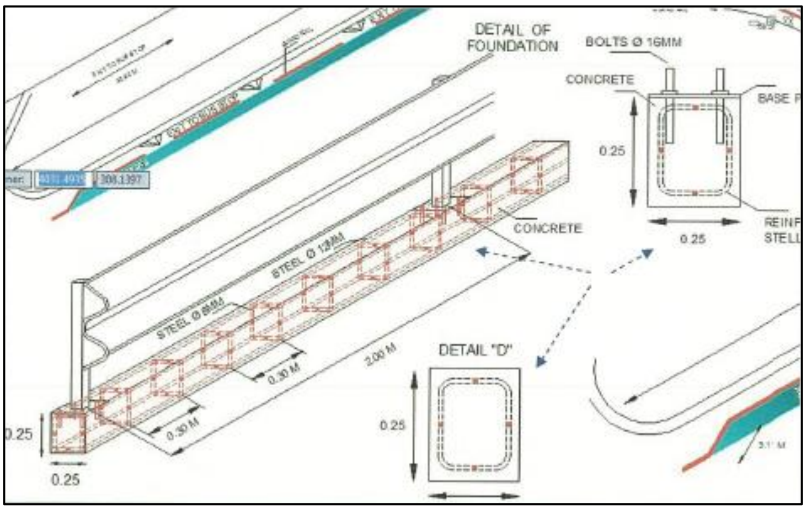


Figure (VIII-25) Cross section and reinforcement plan of curb stone and the plan of assemblage of guard rail

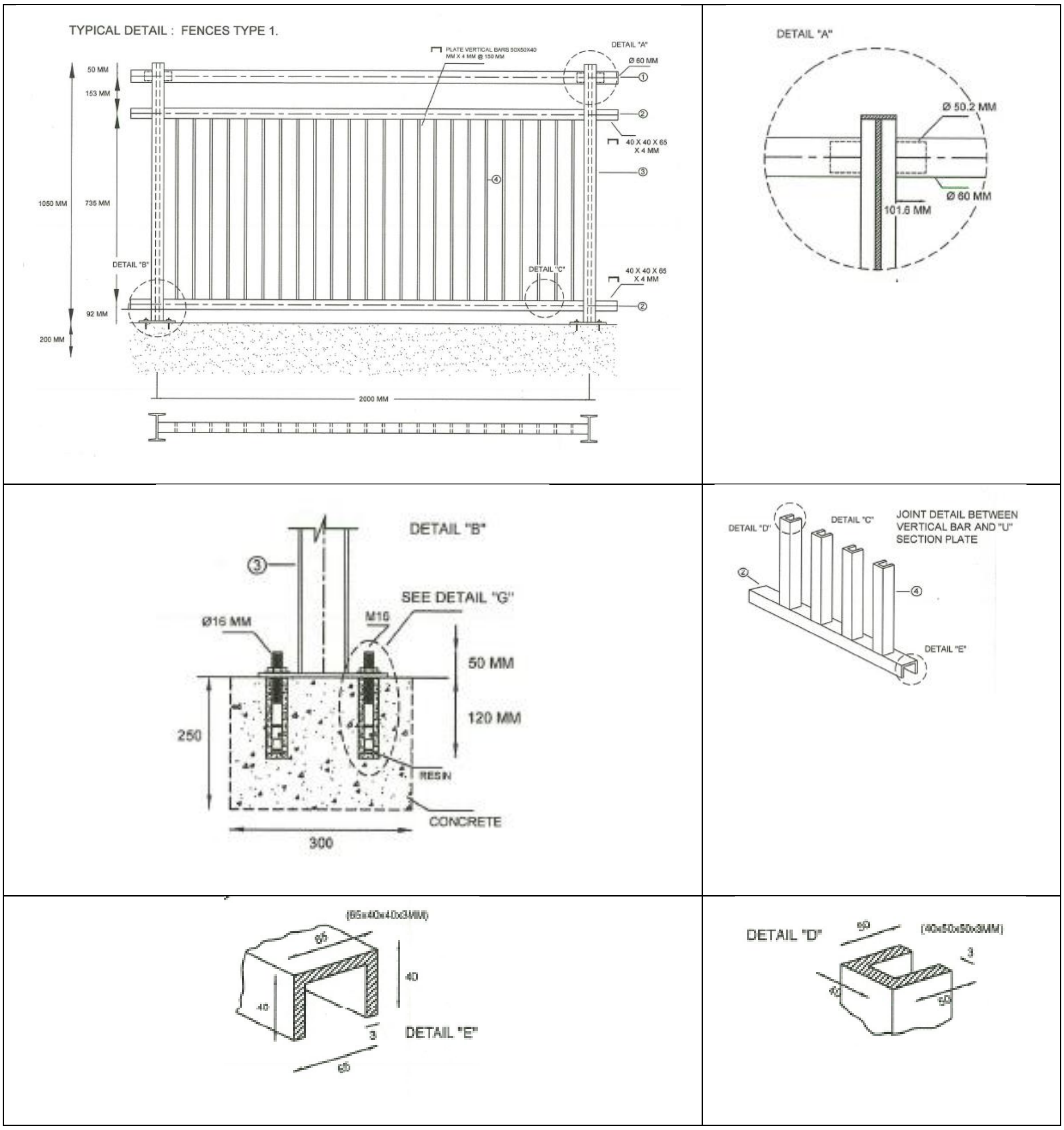


Figure (VIII-26) Fences view and installing plan

The plan for execution of new catch pits near the bridges is illustrated in Fig (VIII-27).

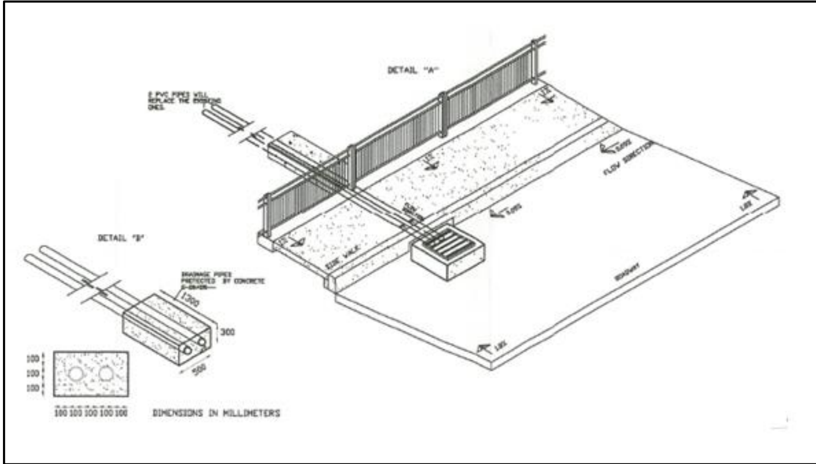


Figure (VIII-27) Catch pits on bridges

Some details from casting of described elements are presented in Fig (VI-28-VI-33).

Figure (VIII-28) Reinforcement for new beam

Figure (VIII-29) Casted edge beam

Figure (VIII-30) New side walk on area between two bridges and new curb stone

Figure (VIII-31) Columns for fences over beam and fence and guard rail after assemblage

Figure (VIII-32) fence and guard rail after assemblage

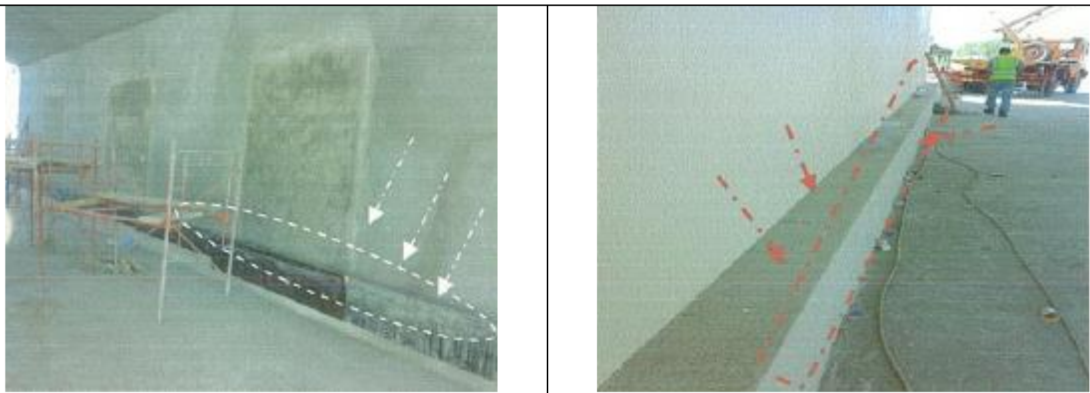


Figure (VIII-33) Plywood form for shoulder and shoulder after finish

Figure (VIII-34). Detail of cover for box of valve and catch pit after finish

1.11. Asphalt Works

The following activities have been planned for repairing the traffic lines:

- Removing the surface layer over the bridge (t=4cm) using milling machine.
- Opening the cracks
- Cleaning cracks and whole surfaces
- Injecting of all cracks in binder course
- Laying of new wearing layer

In Fig (VIII-35 and VIII-36) the condition of upper layer of asphalt and view after its removing are shown.

Figure (VIII-35) A view of old asphalt layer on Souk Athulatha Bridge 1

Figure (VIII-36) Joint after removed asphalt

Since a lot of cracks have been noticed in down layer, it was decided to fulfill them by injection. Before injection, the cracks have to be opened and cleaned. All these operations are shown in fig (VIII-37-VIII-38).

Figure (VIII-37) Cleaning the road by air compressor after removing of upper asphalt layer

Figure (VIII-38) Cracks in down layer, after cleaning

Figure (VIII-39) Special machine for open and clean cracks

Figure (VIII-40) Opening the crack by special machine

Figure (VIII-41) The view of cracks after use machine

Figure (VIII-41) The view of cracks after use machine

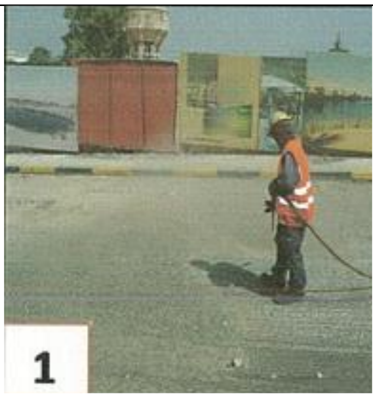
Figure (VIII-42) Injection of cracks by machine

Figure (VIII-43) View of cracks after injection

After finishing of all preparing works, the new wearing asphalt layer has to be placed.

Placing of new bituminous wearing layer encompasses:

1. Cleaning the surface by using air compressor.
2. Spraying bitumen MC-250 by using MC tank according to required spray rate.
3. Checking of the temperature of asphalt mixture when the trucks arrive.
4. Controlling of thickness of asphalt by elevation of the steel wires.
5. Spreading wearing course by automatic controlled pavers.
6. Compacting asphalt by using steel roller and tire roller.
7. Taking asphalt cores for checking the thickness and compaction.



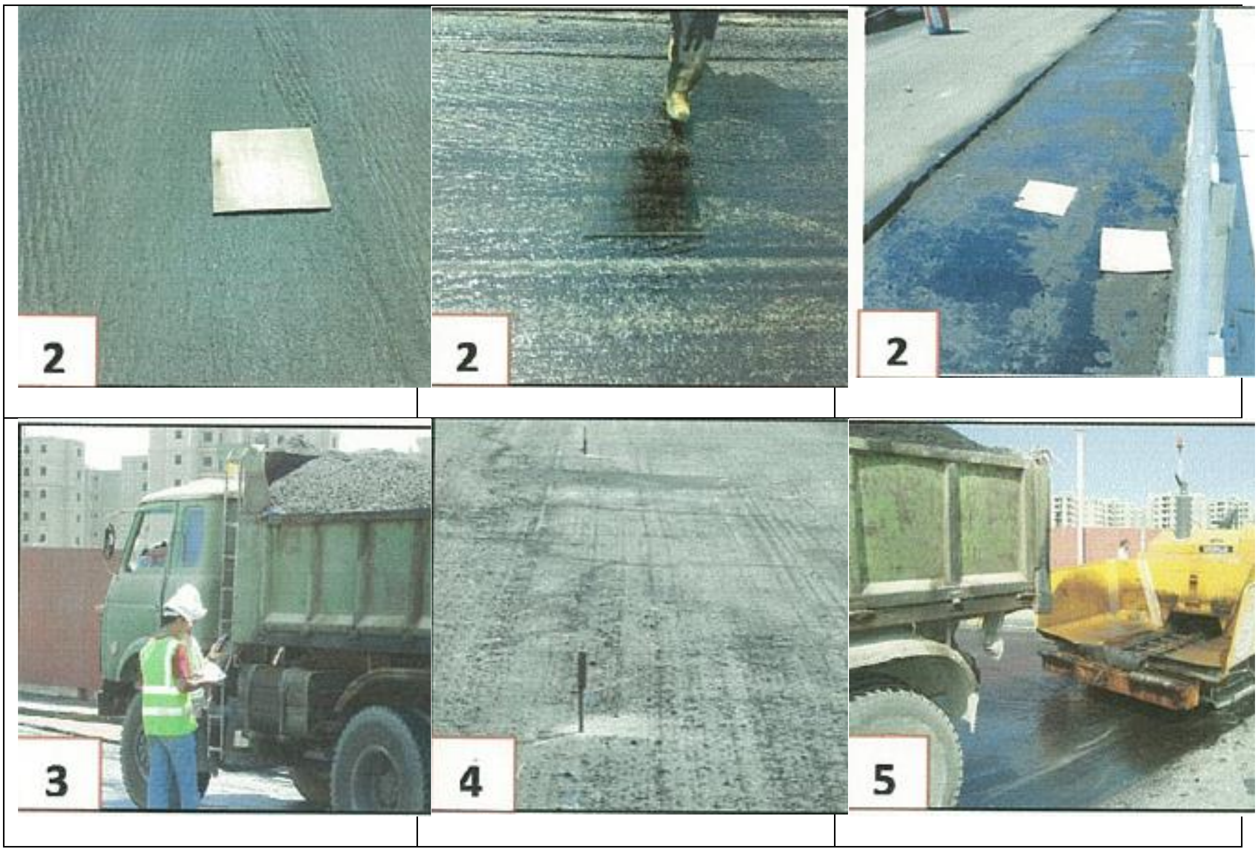


Figure (VIII-44) Phases of placing of bituminous materials

Conti. ….Figure (VIII-44) Phases of placing of bituminous materials

Figure (VIII-45) the road after finish asphalting

To construct a new lane for buses, old sidewalk has to be removed. New lane for buses consists of:

- Lining of 40cm sub base in 2 layers (20cm sub-base and 20cm sub-base with 4% cement).
- Spraying bitumen MC-250 by using MC tank according to required spray rate.
- Checking of the temperature of asphalt mixture when the trucks arrive.
- Controlling of thickness of asphalt by elevation of the steel wires.
- Spreading wearing course by automatic controlled pavers.
- Compacting asphalt by using steel roller and tire roller.
- Taking asphalt cores for checking the thickness and compaction.
-

Fig (VIII-46, VIII-47, and VIII-48) shows some details from execution of a new lane for buses.

	
Figure (VIII-46) Compacted sub base, 1stlayer	Figure (VIII-47) Compacted sub base+4% cement, 2st layer

Figure (VIII-48) Sub base after spray MC

The typical cross section of road is presented in Fig (VIII-49)

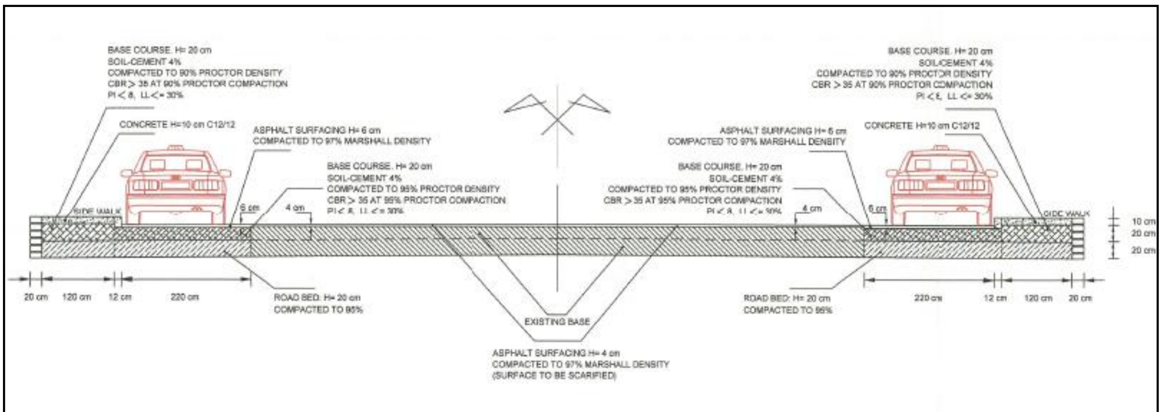


Figure (VIII-49) Typical cross section of asphalt road between bridges

1.12. OTHER NONSTRUCTURAL REPAIR

Other nonstructural repair measures are:

- Painting of curbstones, protective walls and traffic lines.
- Electrical works,
- Removal of old fences and fixed new fence and guardrail and
- Setting of reflective sign in guardrail.

Described works have been illustrated in next Fig (VIII-50-VIII-51).

	
Figure (VIII-50) Curb stone before paint	Figure (VIII-51) Curb stone after paint

	
Figure (VIII-52) Painting of protective walls	Figure (VIII-53) Painting of protective walls

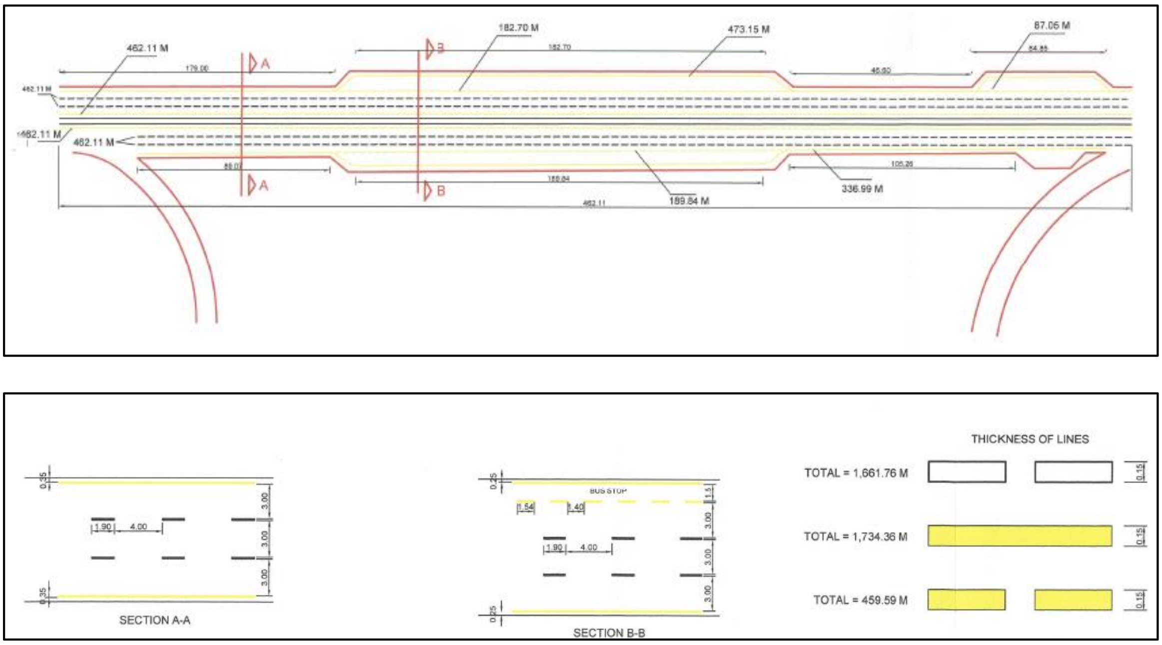


Figure (VIII-54) Plan of traffic lines paintworks

After finishing all painting on the bridges, the electrical reflectors were installed (Fig VIII-55-VIII-56).

	
Figure (VIII-55) Fence and guardrail after installation	Figure (VIII-56) Fence on bridge after fixing

	
Figure (VIII-57) Reflective sign in guardrail	Figure (VIII-58) Reflective sign in guardrail, detail

1.13. CONCRETE WORKS

Concrete works include concreting of following elements:

- Edge beams,
- Shoulders,
- Curbstones,
- Sidewalks and
- Different types of beams (under sidewalks and curbstone and for fixing the fences)

For concreting of all mentioned elements, the two types of concrete have been selected:

- C25/30 for execution of edge beams, shoulders, curbstones, sidewalks and curbstones for fixing the fences, and
- 12/15 for execution lean concrete.

1.14. Important information

The assessment of the condition of bridge has been done through 2 phases. In the first phase the VSL done only visual inspection of concrete structural elements. On the bases of results obtained by visual inspection they wrote the program of structural elements repair.

During the removal of old and damaged plaster and concrete cover, they decided to expand the assessment program by testing of quality built in material using semi destructive and nondestructive methods (second phase). The results of field testing were much worse than predicted as most concrete surface showed high level of carbonation and they decided to remove all carbonated concrete. In this phase, they also decided to replace some rebars or just add missing area of rebar and to protect all imbedded reinforcement bars by corrosion inhibition impregnation.

2. SOUK ATHULATHA 2 BRIDGE

2.1. Introduction

Through assessment of the condition of structural elements of bridge the next conclusions have been derived:

The characteristic defects are insufficient concrete cover depth and bad quality of concreting works.

The characteristic damages of RC elements are reinforcement corrosion and loss of concrete cover.

The main causes of described damages are carbonization of concrete, insufficient concrete cover depth and bad quality of concrete.

In some cases, the bad quality of works during the bridge construction and local penetration of water also contributed to the development of bars corrosion.

The degree of observed damages is not dangerous for stability and bearing capacity of structures, but reduces durability and functionality. The most damaged elements have been edge beams, and they have to be removed.

Whereas the stability and bearing capacity of bridge have not been jeopardized the most suggested repair measures belong to the group of non-structural repair and surface protection.

As the carbonization and insufficient concrete cover present general problem, all elements which have been affected by carbonization or have insufficient concrete cover depth, have to be repaired. These elements are:

- Cantilever slabs
- Lateral beams
- Arc slab (ceiling)
- Interior walls
- Exterior walls
- Underpass (tunnel) ceiling

The suggested repair measures include removing the old painting layer, old cement plaster and the old carbonized concrete cover and execution of new cover with increased depth. Additional reason for suggested measures has been bad results of pull-off test, which were smaller than required minimum value.

Repair measures also include cleaning, protection or even replacement of corroded bars.

For better bonding between old, but healthy, concrete and new cover the special agents is proposed.

Also, the special coatings are suggested for concrete surface protection.

The most damaged elements are Cantilever slabs.

During removing the old ordinary cement mortar, it was noticed that surface of concrete was very porous and full of voids and pockets. Because of such bad condition of protecting cover, it was decided to remove complete concrete cover from all investigated elements (cantilever slabs, arch ceiling, interior and exterior walls and underpass ceiling).

2.2. Repair measure of bridges in Tripoli in 2009

-

For increasing of longevity of this bridge the next repair measures were recommended:

- Complete removal and execution of new RC edge beams,
- Complete concrete cover removal,
- Complete rebar treatment (even including rebar replacement)
- Rebar protection
- Bonding slurry application
- Execution of new concrete coverwith minimum depth of 20mm.
- Corrosion inhibition impregnation of all bridge exposed surfaces.
- Surface protection with coating of all repaired concrete elements.

All recommended measures were described in detail in chapter XXX, in which the detail program for repairing Bridge Souk Athulatha1 was given.

Follows, through several photos, above mentioned repair measures will be illustrated.

The removal of old carbonated concrete cover from interior and exterior walls, and tunnel ceiling is shown on the next three pictures.

Figure (VIII-59) Damaged concrete removal from interior and exterior walls

Figure (VIII-60) Damaged concrete removal from tunnel ceiling

Fig (VIII-61, VIII-62) show the operation of cleaning bars with mixed water-sand blastering method.

Figure (VIII-61) Rebars cleaning with water-sand blastering in deck ceiling slab

Figure (VIII-62) Rebars cleaning with water-sand blastering on lateral beam

Photos (VIII-63, VIII-64) lustrate the process of treatment rebars with cementitious base protection material and view of protected rebars.

Figure (VIII-63) Covering the rebars on ceiling beams with cementitious base protection

Figure (VIII-64) Rebars protective coating on cantilever

Photos (VIII-65 – VIII-66) illustrate procedures of applying and finishing of new mortar layer on ceiling and on walls.

Figure (VIII-65) Cementitious base special mortar on deck ceiling beams

Figure (VIII-66) Deck ceiling slab with special mortar

Figure (VIII-67) Cementitious base special mortar on exterior walls

Figure (VIII-68) Levelling on the interior walls

Application of protecting coating is shown in fig (VIII-69).

Figure (VIII-69) Application of protecting coating on ceiling by roller

2.3. Reinforcement rebar replacement

If rebar loses more than 28% of the area it must be complemented, if it loses more than 50 % it must be replaced.

All complementation rebars must have sufficient anchoring length. In situation when there is not enough space for anchoring rebars, they have to be welded to existing bars.

Rods can also be replaced when there is not enough space to complement.

The data of lose cross section of corroded rebars and about complemented or replacement rebar are given in table VIII-4.

Table VIII-4 - Rebar complementation selection Lose of section ≤28%

Φ Rebar mm	A section cm^2	A with section loss cm^2	Φrebars with corrosion mm	Δ complementation section cm^2	Φof complemented/replacement rebars mm
6	0.28	0.20	5.05	0.08	6
8	0.50	0.36	6.77	0.14	6
10	0.79	0.57	8.52	0.22	6
12	1.13	0.81	10.16	0.32	8
16	2.01	1.45	13.59	0.56	10
20	3.14	2.26	16.96	0.88	12
25	4.91	3.54	21.23	1.37	16
32	8.04	5.79	27.15	2.25	20
28%<Lose of section ≤ 50%					
6	0.28	0.14	4.22	0.14	6
8	0.50	0.25	5.64	0.25	6
10	0.79	0.40	7.14	0.39	8
12	1.13	0.57	8.52	0.56	10
16	2.01	1.01	11.34	1.00	12
20	3.14	1.57	14.14	1.57	16
25	4.91	2.46	17.70	2.45	20
32	8.04	4.02	22.62	4.02	25
Lose section>50%					
6	0.28	0.07	2.99	0.21	6
8	0.50	0.13	4.07	0.37	8
10	0.79	0.20	5.05	0.59	10
12	1.13	0.28	5.97	0.85	12
16	2.01	0.50	7.98	1.51	16
20	3.14	0.79	10.03	2.35	20
25	4.91	1.23	12.51	3.68	25
32	8.04	2.01	16.00	6.03	32

2.4. Materials recommended for repair of the bridge

After analyzing specifications of materials for structural and non-structural repair, principles and methods of repair and available materials in Libya, the next products have been selected:

Cementitious base material for rebar protection:	EmacoNanocreteAP(BASF)
Bond material:	EmacoNanocrete AP(BASF)
Cementitious base pre-mixed mortar:	EmacoNanocreteR4(BASF)
Currying protection material:	Masterkure 181(BASF)
Corrosion inhibition impregnation:	SikaFerroguard 903(SIKA)
Surface protective and paint layer:	Sikaguard 680 ES Betoncolor
Reinforcement rebar steel:	Grade A-615

The estimated quantities of listed products after visual inspection, after plaster removal and additional testing of materials and their real consumption are given in Table VIII-5.

Table VIII-5 - Calculated quantities of used products and their real consumption

Material	Estimated	Calculated after additional testing	Real quantities
EmacoNanocrete AP	653kg	4.110kg	2.000kg
EmacoNanocrete R4	22.334kg	130.200kg	117.713kg
Master flow 928/980	-	8.825kg	14.000kg
Mastercure 181	200Lt	600Lt	400Lt
SikaFerroguard 903	-	875Lt	695Lt
Sikaguard 680 ES Betoncolor	500Lt	500Lt	700Lt
Sikadur 31CF	-	5kg	-
Sikadur 52	-	42kg	-

In table VIII-6 the real consumption of used products per measuring unit is given.

Table VIII-6 - The real consumption of used products per measuring unit (ratio)

Quantity ratios					
Work	Material	Unit	Qty	Ratio	Average thickness
Concrete repair	EmacoNanocrete R4	1.516,20m^2	117.713kg	77.64kg/m^2	41mm
	EmacoNanocreteAP	1.516,20m^2	2.200kg	1.32kg/m^2	-
	Master flow 928/980	78ml	14.000kg	179.5kg/ml	-

2.5. OTHER REPAIR WORKS

Other repair works with aim to provide functionality involve procedures such as:

- Re-plaster and paint the surrounding walls of the bridge,
- Repair or replacement of sidewalksand curbstone,
- Removal of old fences and assemblage of new one,
- Installation of new guard rails and catch pits,
- Execution of new asphalt layers and
- Assemblage a new traffic signs, electrical lights…etc.

Also, it has been decided to add the protective concrete layers, called “shoulder”, on the side of exterior wall towards the traffic lanes.

As the bridges Souk Athulatha 2 and 1 are near each other, it was decided to repair both bridges and the road between them at the same time (Fig 8.70). Since the other repair measures are similar for both bridges and road between them, they will not be described separately. The other repair works were described and illustrated in Chapter XX (Souk Athulatha1).

Figure (VIII-70) Site plan of bridges Souk Athulatha 2 and 1

Final bridge view is shown in Fig (VIII-71)

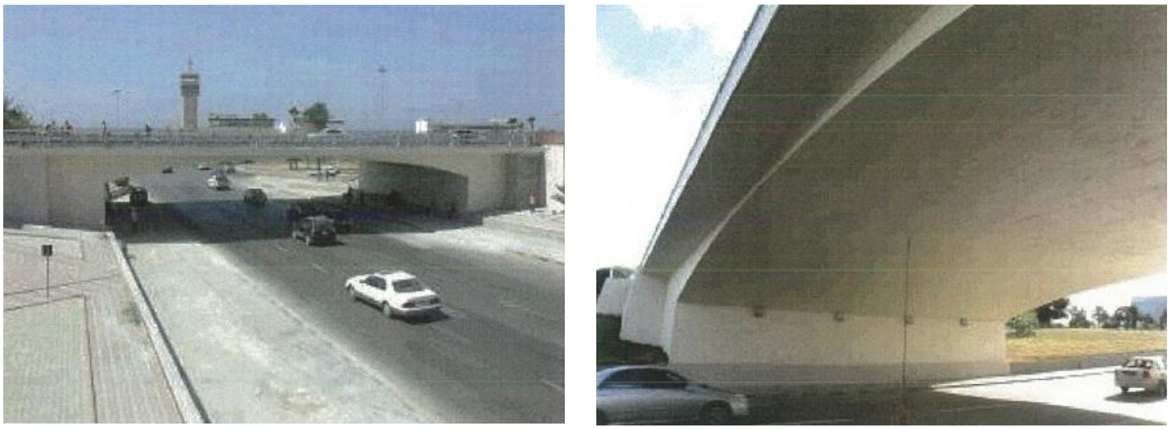


Figure (VIII-71) Final bridge

3. AL SEEKA ROAD BRIDGE

3.1. Introduction

Through assessment of the condition of structural elements of bridge the next conclusions have been derived:

The characteristic defect of arch slabs and lateral beams have been insufficient concrete caver (~1cm).

Built in concrete has low compressive strength (C16/20) and low density (~2180kg/m^3)

The characteristic damages of RC elements are reinforcement corrosion which is pronounced in transverse deck joints.

Main causes of described damage are carbonization of concrete (up to 70mmin ceiling and up to 90mm in abutment), insufficient concrete cover depth and porous concrete.

In some cases, the bad quality of works during the bridge construction and local penetration of water also contributed to the development of bars corrosion.

The most damaged element is south lateral beam. Bearing capacity of south lateral beam is jeopardized because of damage of main reinforcement bars. This girder was damaged due to hitting by truck.

The degree of other observed damages is not dangerous for stability and bearing capacity of structures, but reduces durability and functionality.

Whereas the global stability and bearing capacity of bridge has not been jeopardized the most suggested repair measures belong to the group of non-structural repair and surface protection.

As the carbonization and bad concrete quality cover present general problem, all elements which have been affected by carbonization or have insufficient concrete cover depth, have to be repaired. These elements are:

- Cantilever slabs
- Arc slab beams (ceiling).
- Lateral beams
- Arch slab (ceiling)
- Abutment walls
- Supporting walls
- Underpass ceiling

Repair measures include removing the old carbonized concrete cover and execution of new cover with increased depth on arc slab ceiling and lateral beams.

Repair measures also include cleaning, protection or even replacement of corroded bars.

For better bonding between old, but healthy, concrete and new cover the special agents is proposed.

Also, the special coatings are suggested for concrete surface protection.

The most damaged element is south lateral beam.

During removing the old cover, it was noticed that surface of concrete was very porous and full of voids and pockets. Because of such bad condition of protecting cover, it was decided to remove complete concrete cover from all investigated elements (cantilever slabs; arch ceiling, interior and exterior walls and underpass ceiling).

Apart from removing concrete cover from upper mentioned elements, removal works were carried out on next elements:

- Removed all old side walk over the bridge
- Removed all old concrete under old fences
- Removed all beam on the bridge edge it was damaged (width of 50cm, height of 25cm).

3.2. Repair measure

For increasing of longevity of this bridge the next repair measures were recommended:

- Complete damaged/ carbonated concrete cover removal,
- Complete rebar treatment (even including rebar replacement) at ST2 grade with mixed water-sand blastering,
- Rebar replacement if it is necessary,
- Rebar protection,
- Execution of new concrete cover with minimum depth of 40mm (increasing the depth),
- Corrosion inhibition impregnation of all bridge exposed surfaces,
- Surface protection with coating of all repaired concrete elements.

Hereinafter all recommended measures will be described in detail.

3.3. Damaged concrete removal

As it was recommended, all poor-quality concrete cover or/and carbonated part of concrete have to be removed. For that operation the electric hammers with max weight of 6kg have been chosen.

According with the testing results the major problem was carbonization within depths of 10 to 90 mm, frequently over passing the rebar plan.

The removal of concrete close to bars has to be done very carefully to avoid the damage of bars.

After finishing concrete removal all surfaces must be clean with water jet with pressure of 200bar.The application of water jet has to be with angle of 45^0 and with 5 cm distance.

The removal of old porous and carbonated concrete cover from abutment wall and ceiling beam is shown in figures Fig (VIII-72- VIII-75).

Figure (VIII-72) Damaged concrete cover removal from interior walls

Figure (VIII-73) Damaged concrete cover removal from ceiling beams

Figure (VIII-74) Local removal work on ceiling

Figure (VIII-75) Local removal work on ceiling of simple beam

3.4. Cleaning of reinforcement bars

After removing of "old" damaged concrete cover, the next operation has been cleaning of reinforcing bars from remains of hardened cement paste and rust as the products of steel corrosion. For this operation the water sand blasting technic is selected with pressure of 250 bar.

3.5. Repairing works

All recommended measures for repairing damaged concrete elements were: rebar protection, reinforcement rebar replacement (deformed and broken ones), bonding agent application, concrete restoration and execution of new cover, corrosion inhibition protection and surface protection.

Some of the above-mentioned works on Al Seeka road bridge are illustrated in figures (VIII-76-VIII-81).

Figure (VIII-76) Rebars protection with cementious base material

Figure (VIII-77) Preparing for plastering ceiling

Figure (VIII-78) Execution of new protecting mortar cover on ceiling

Figure (VIII-79) Finalizing of protecting mortar cover on ceiling

Figure (VIII-80) Plastering of exterior wall

Figure (VIII-81) Painting of repaired surfaces of bridge elements

3.6. Materials recommended for repair of the bridge

After analyzing specifications of materials for structural and non-structural repair, principles and methods of repair and available materials in Libya, the next products has been selected:

Cementitious base material for rebar protection:	EmacoNanocreteAP(BASF)
Bond material:	EmacoNanocrete AP(BASF)
Cementitious base pre-mixed mortar:	EmacoNanocreteR4(BASF)
Currying protection material:	Masterkure 181(BASF)
Corrosion inhibition impregnation:	SikaFerroguard 903(SIKA)
Surface protective and paint layer:	Sikaguard 680 ES Betoncolor
Reinforcement rebar steel:	Grade A-615

The estimated quantities of listed products after visual inspection, after plaster removal and additional testing of materials and their real consumption are given in Table VIII-7.

Table VIII-7 - Calculated quantities of used products and their real consumption

Material	Estimated	Calculated after additional testing	Real quantities
EmacoNanocrete R4	23.450kg	52.225kg	70.650kg
EmacoNanocrete AP	700kg	1.695kg	1.300kg
Master flow 928/980	-	14.975kg	9.870kg
Mastercure 181	100Lt	400Lt	400Lt
Sikadur 31CF	-	-	-
Sikadur 52	-	-	-
SikaFerroguard 903	-	850 Lt	400Lt

Sikaguuard 680 ES Betoncolor	500Lt	500 Lt	500Lt

In table VIII-8 the difference between calculated and real quantities of material is given.

Table VIII-8 - Difference between calculated and real quantities of material

Quantity ratios					
Work	Material	Unit	Qty	Ratio	Average thickness
Concrete repair	EmacoNanocrete R4	1.516,20m^2	117.713kg	77.64kg/m^2	41mm
	EmacoNanocreteAP	1.516,20m^2	2.200kg	1.32kg/m^2	-
	Master flow 928/980	78ml	14.000kg	179.5kg/ml	-

In Al Seeka bridge maintenance project documentation, we did not found detail description for repairing part of lateral beam which was damaged by truck hitting nor any in-situ photos. We found only following description: “some rebars in the ceiling had to be substituted because a truck hit one beam and twisted it so much that was impossible to replace them, as you can see in the pictures below”.

All recommended measures for repairing damaged concrete elements, such as rebar protection, reinforcement rebar replacement (if necessary), bonding agent application, concrete restoration, corrosion inhibition protection and surface protection, were described in detail in chapter 1, in which the detail program for repairing Bridge Souk Athulatha 1 was given.

3.7. OTHER REPAIR WORKS

Other repair works with aim to provide functionality involve procedures such as:

- Re-plaster and paint the surrounding walls of the bridge,
- Repair or replacement of sidewalks and curbstone,
- Removal of old fences and assemblage of new one,
- Installation of new catch pits,
- Execution of new asphalt layers and
- Assemblage a new traffic signs, electrical lights…etc.

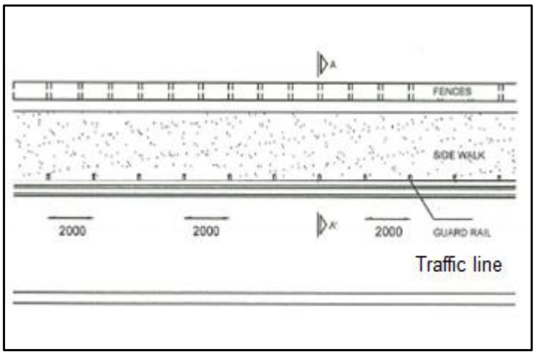
Hereinafter the other repair works are described and illustrated.

During the repair of these bridges, it was decided to remove all old sidewalks on Center Island over bridges (AL Sseka and Bab Bin Gheshir road) and part of curbstone and old concrete under the fences over the bridges and between these bridges Fig (VIII-84). The detail of removal of damaged sidewalk is shown in figure (VIII-85).

Figure (VIII-82) Site plan of bridges ALseeka and Bab Bin Ghasir

Figure (VIII-83). Removing of old sidewalks on bridge and between them

After repairing the edge beams and other structural elements of bridge and preparing sub base between the bridges, it has been planned to cast new sidewalks and curbstone (Fig VIII-84 and VIII-85).



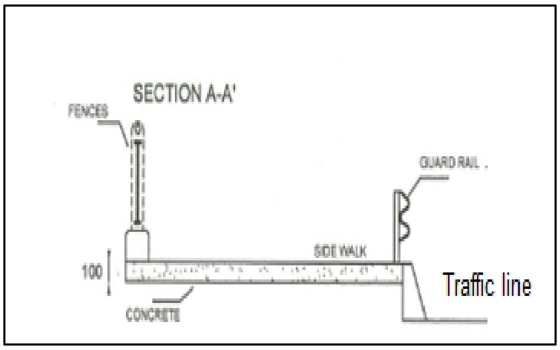


Figure (VIII-84). Sidewalk elements arrangement

Figure (VIII-85). Cross section of sidewalk elements

This operation includes following steps:

- Concreting of beam under side walk and lining of sub base layer,
- Concreting of the sidewalk slabs with dimensions 1.20m×2.50mx0.1m on top of concrete beam, with 2cm expansion joint filled with elastic material.
- Concreting of the layer under the fences with the height 10cm above the sidewalk level.
- Casting the new lean concrete under the new curbstone with a 30cm width.
- Concreting of new curbstone with expansion joint every 12m.
- Concreting and installing of the new catch pit.

The cross section of new sidewalks between the bridges with all necessary elements is shown in Fig (VIII-86).

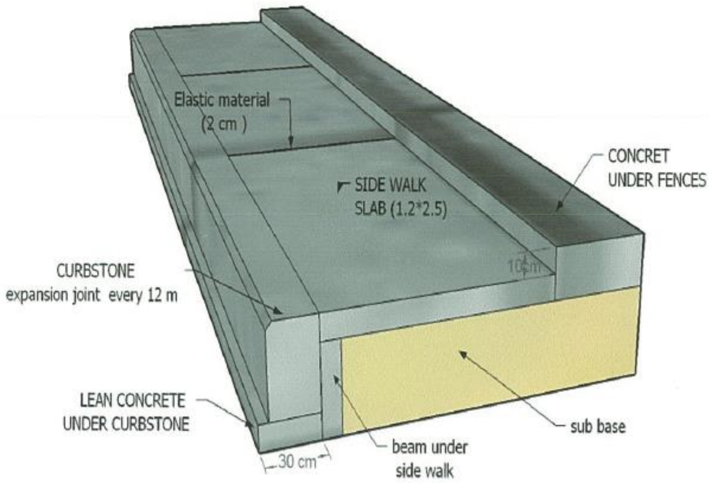


Figure (VIII-86). New sidewalk between the bridges Bab Bin Gheshir road and AL Sseka

Figure (VIII-87) shows the details of new fence and the plan of installing the new fence.

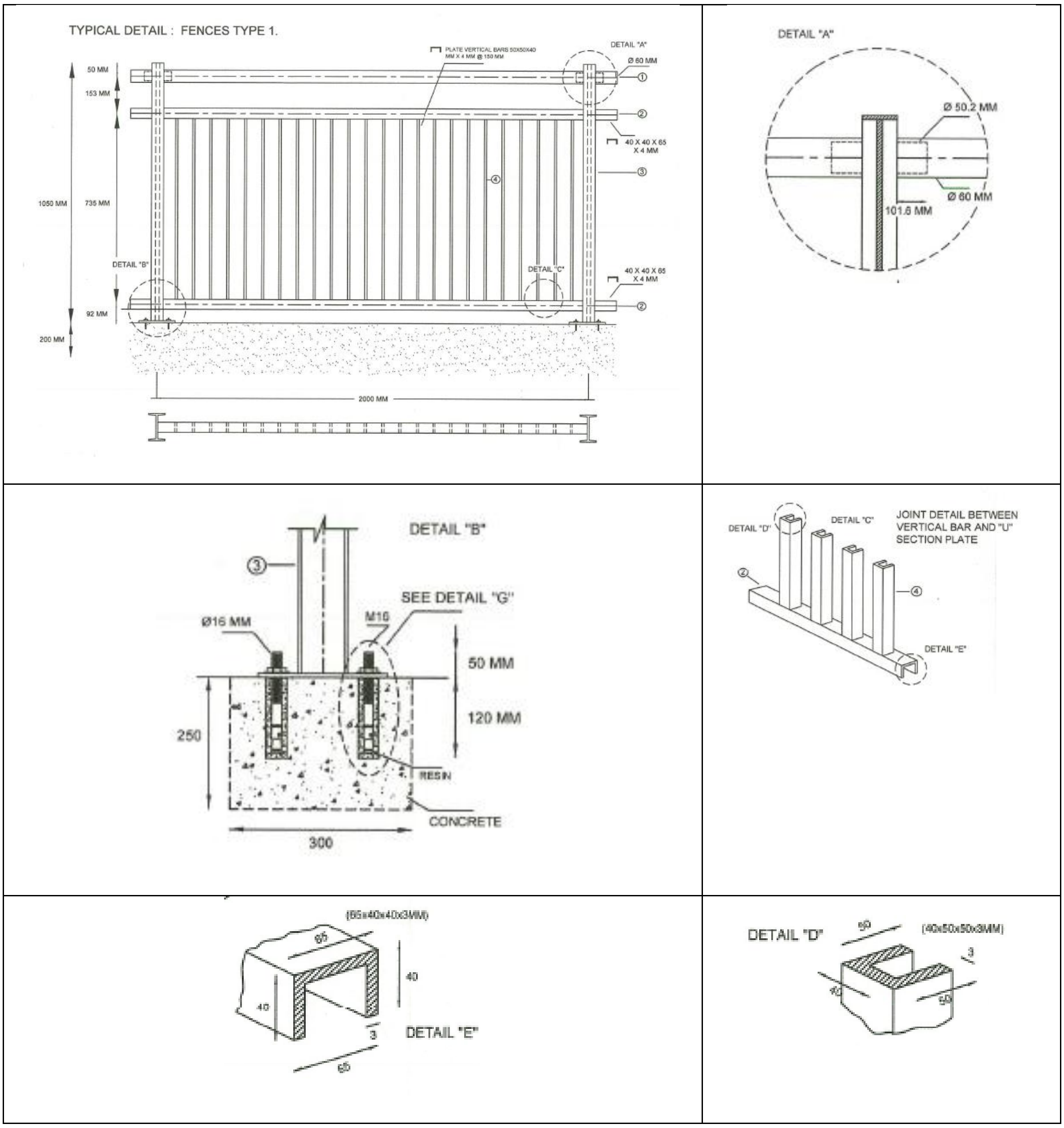


Figure (VIII-87) Fences view and installing plan

Some details from casting of described elements are presented in Figures VIII-88 – VIII-89.

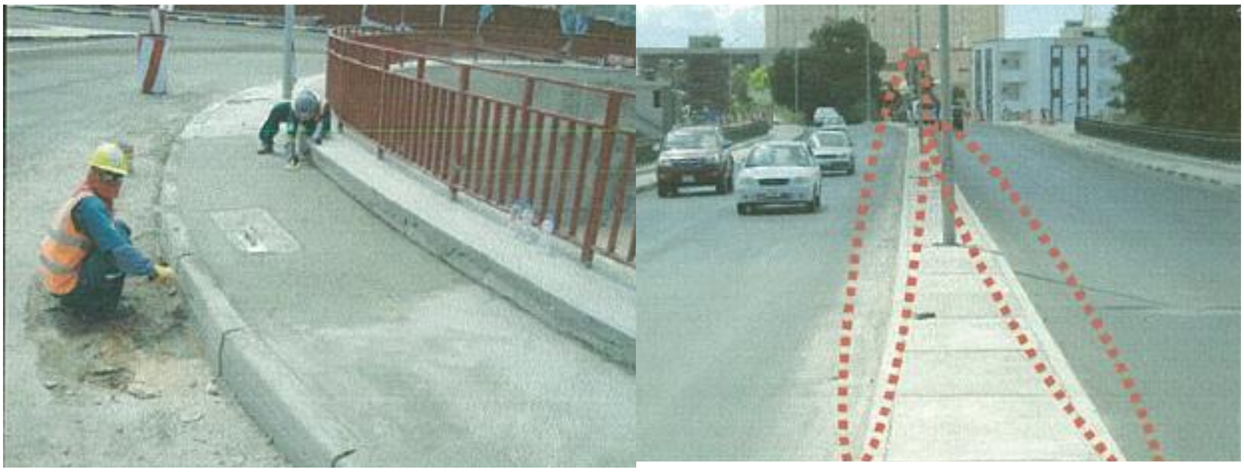


Figure (VIII-88) New side walk with new curbstone and casting of new catch pit

Figure (VIII-89). Removal of old fence

3.8. Asphalt Works

The following activities have been planned for repairing the traffic lines:

- Removing the surface layer over the bridge (t=4cm) using milling machine.
- Opening the cracks
- Cleaning cracks and whole surfaces
- Injecting of all cracks in binder course
- Laying of new wearing layer

In fig (VIII-90 and VIII-91) the condition of upper layer of asphalt before removing of surface layer and the view after its removal are shown.

Figure (VIII-90). A view of old asphalt layer on the bridge before removing of surface layer

Figure (VIII-91). The view of surface after asphalt was removed

Since a lot of cracks have been noticed in down layer, it was decided to fulfill them by injection. Before injection, the cracks have to be opened and cleaned. All these operations are shown in fig (VIII-92 and VIII-93).

Figure (VIII-92). cracks before injection

Figure (VIII-93). Cracks after injection

After finishing of all preparing works, the new wearing asphalt layer has to be placed.

Placing of new bituminous wearing layer encompasses:

1- Cleaning the surface by using air compressor.
2- Spraying bitumen MC-250 by using MC tank according to required spray rate.
3- Checking of the temperature of asphalt mixture when the trucks arrive.
4- Controlling of thickness of asphalt by elevation of the steel wires.
5- Spreading wearing course by automatic controlled pavers.
6- Compacting asphalt by using steel roller and tire roller.
7- Taking asphalt cores for checking the thickness and compaction.

Phases of placing bituminous wearing layer are shown in figures (VIII-94). , and the final view of bituminous wearing layer is shown in figures (VIII-95).

Figure (VIII-94). Phases of placing of bituminous materials

Conti. … Figure (VIII-94). Phases of placing of bituminous materials

Figure (VIII-95). The road after finish asphalting

Other nonstructural repair measure is painting of protective walls and traffic lines (fig VIII-96 and VIII-97).

Figure (VIII-96). Painting of protective walls

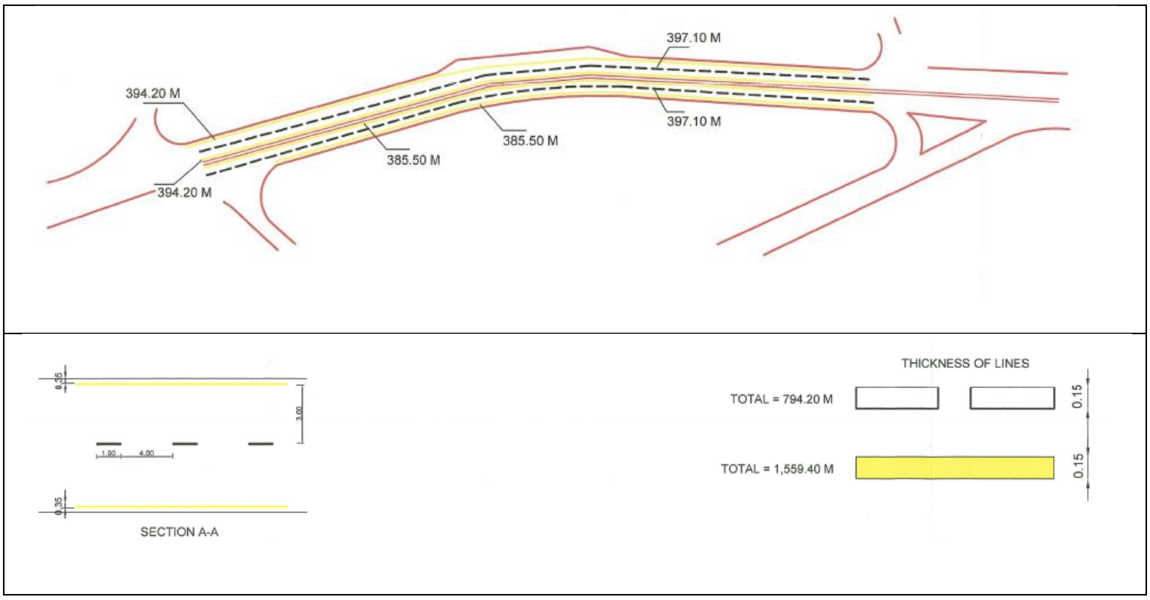


Figure (VIII-97). Plan of traffic lines paintworks

The quantities of works, such as works on sidewalks, fences, plastering of approaching structure and etc., are shown in table VIII-9.

Table VIII-9. The quantities of other works on the bridge and approaching structure

Number	description	unit	quantity
1	Removal of sidewalk	m^2	122.75
2	Replacement of sidewalk	m^2	122.75
3	Removal of curbstone	m^1	288.84
4	Replacement of curbstone	m^1	288.84
5	Removal of fences	m^1	111.30
6	New fences	m^1	111.30
7	Repair of fences	m^1	164.45
8	Removal plaster	m^2	2,692.09
9	New plaster (first layer)	m^2	1,188.05
9.1	New plaster (second layer)	m^2	1,188.05
10	Nanocrete	m^2	1,504.04
11	Removal of asphalt	m^2	1,468.31
12	New asphalt	m^2	1,468.31
13	paint	m^2	1504.04
14	Graffiti	m^2	1,188.05

3.9. Plan of Expansion Joint

In origin design, the expansion joints were planned in four places. The layout of expansion joints is given in Figure (VIII-98). The description of suggested repair measure for them is also given in figure VIII- 26, and specific detail could be seen in Figure (VIII-99).

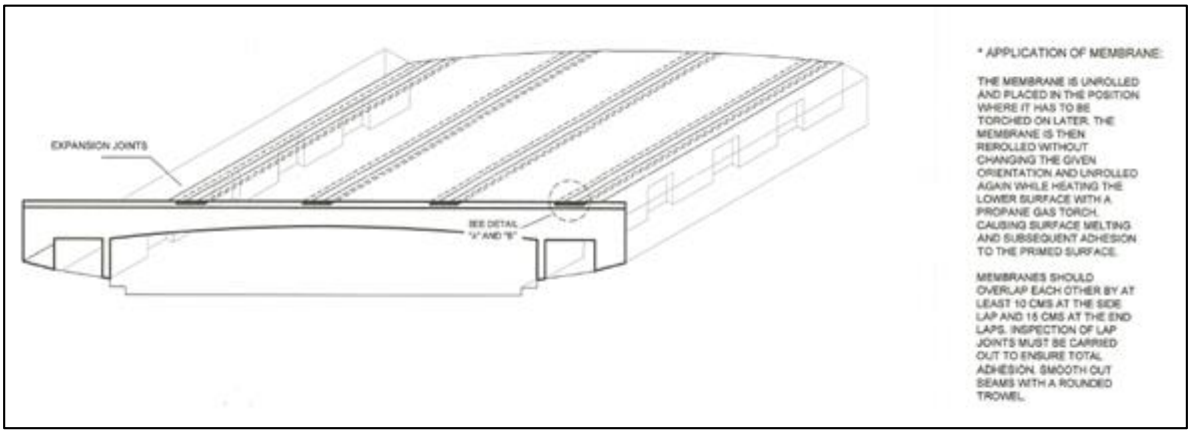


Figure (VIII-98) Plan of expansion joints in bridge structure

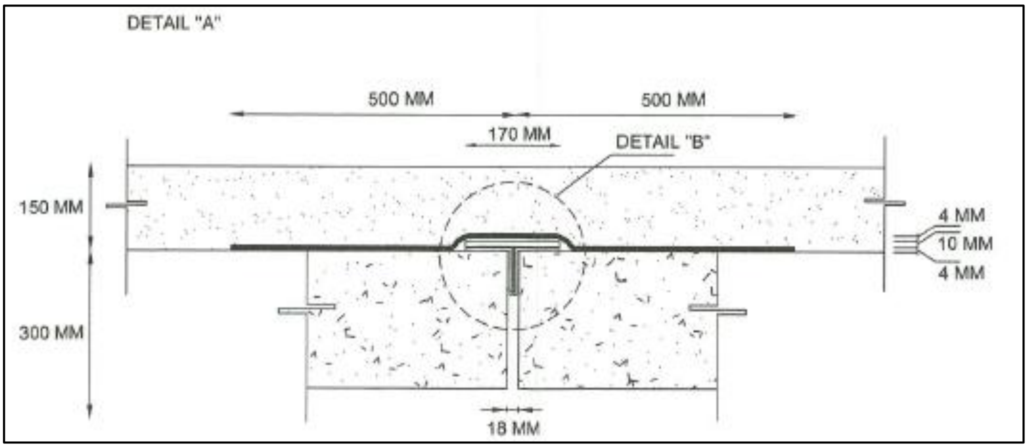


Figure (VIII-99) A detail of setting up of membrane over the expansion joints in bridge structure

The view of Al Seeka bridge after all repair measures is shown in Fig (VIII-100).

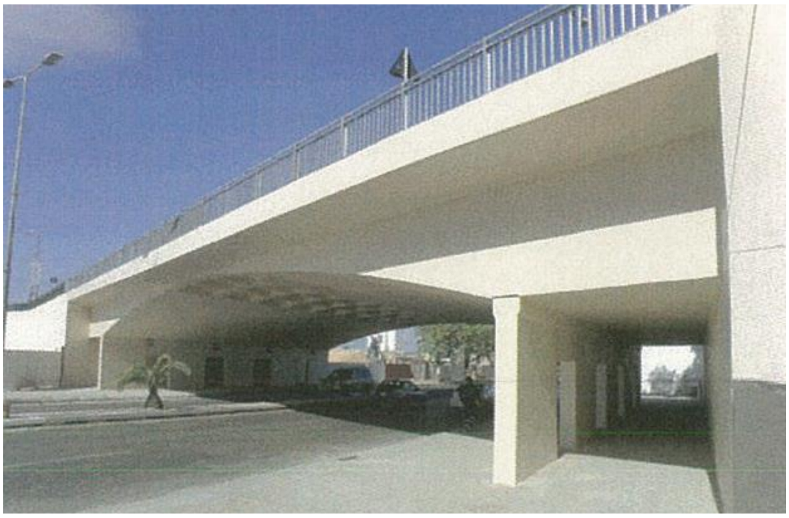


Figure (VIII-100) Al Seeka Bridge after all repair measures.

Recapitulation of all designed repair works on RC elements of Al Seeka road bridge

	Arch slab (ceiling)	Lateral beams	Cantilever slabs	Underpass ceiling	Supporting walls	Abutment walls	Edge beam
Removal of concrete cover	+	+	+	+	+	+	The whole
Replacement of deformed and broken of rebars	+	+					
Rebar treatment	+	+	+	+	+	+	New reinforcement
Rebar protection	+	+	+	+	+	+	+
Execution of new concrete cover	+ With increased depth, by plastering	+ With increased depth, by plastering	+ by plastering	+ by plastering	+ by plastering	+ by plastering	New concrete
Corrosion inhibition impregnation	+	+	+	+	+	+	+
Surface protection	+ paint	+	+	+	+	+	
Execution of new element							+

4. BAB BIN GHESHIR ROAD BRIDGE

4.1. Introduction

Through assessment of the condition of structural elements of bridge the next conclusions have been derived:

The characteristic damages of RC elements are reinforcement corrosion and loss of concrete cover.

The main cause of described damages is carbonization of concrete (up to 60mm) and insufficient concrete cover depth (mostly 10mm depth), especially in deck ceiling slab.

In some cases, the bad quality of works during the bridge construction and local penetration of water also contributed to the development of bars corrosion.

The degree of observed damages is not dangerous for stability and bearing capacity of structures, but reduces durability and functionality. The most damaged elements have been cantilever slabs, edge beams and expanded joints (due to leakage)

Whereas the stability and bearing capacity of bridge have not been jeopardized the most suggested repair measures belong to the group of non-structural repair and surface protection.

As the carbonization and insufficient concrete cover present general problem, all elements which have been affected by carbonization or have insufficient concrete cover depth, have to be repaired. These elements are:

- Cantilever slabs
- Lateral beams
- Arch slab (ceiling)
- Abutment walls
- Supporting walls
- Underpass ceiling

Repair measures include removing the old cement plaster and the old carbonized concrete cover and execution of new cover with increased depth.

Repair measures also include cleaning, protection or even replacement of corroded reinforcing bars.

For better bonding between old, but healthy, concrete and new cover the special agents is proposed.

Also, the special coatings are suggested for concrete surface protection.

The most damaged elements are Cantilever slabs and interior wall near the cantilever.

During removing the old ordinary cement mortar it was noticed that surface of concrete was very porous and full of voids and pockets. Because of such bad condition of protecting cover it was decided to remove complete concrete cover from all investigated elements (cantilever slabs, arch ceiling, abutments and supporting walls and underpass ceiling).

Apart from removing concrete cover from upper mentioned elements, removal works were carried out on next elements:

– Removed all old side walk on center island over bridge
– Removed all old concrete under old fences
– Removed all beam on the bridge edge it was damaged (width of 50cm, height of 25cm).

4.2. Repair measures

For increasing of longevity of this bridge the next repair measures were recommended:

- Complete damaged/ carbonated concrete cover removal,
- Complete rebar treatment (even including rebar replacement) at ST2 grade with mixed water-sand blastering,
- Rebar protection,
- Execution of new concrete cover with minimum depth of 40mm,
- Crack injection,
- Corrosion inhibition impregnation of all bridge exposed surfaces,
- Surface protection with coating of all repaired concrete elements,

Hereinafter all recommended measures will be described in detail.

4.3. Damaged concrete and plaster removal

As it was recommended, all plaster layer and poor quality concrete cover or/and carbonated part of concrete have to be removed. For that operation the electric hammers with max weight of 6kg have been chosen.

According with the testing results the major problem was carbonization within depths of 20 to 60 mm, frequently overpassing the rebar plan.

In some cases the removal of concrete with 70mm depth has been suggested to be ensure that all damaged concrete is removed and all rebars are treated (10mm existing cover + 10mm stirrups+ 32mm rebars+ 20mm over rebar plan).

After finishing concrete removal all surfaces must be clean with water jet with pressure of 200bar.

The removal of old carbonated concrete cover by electric hammers from deck ceiling slab is shown in figures (VIII-101- VIII-103). The removal of whole edge beams is shown in figure VIII-104.

Figure (VIII-101) Damaged concrete removal from deck ceiling slab

Figure (VIII-102) Supporting wall with damaged concrete removed and exposed reinforcing bars

Figure (VIII-103) General view of the supporting wall with damaged concrete removed and exposed rebars

Figure (VIII-104) Removal of old edge beam

4.4. Cleaning of reinforcement bars

After removing of “old” damaged concrete cover, the next operation has been cleaning of reinforcing bars from remains of hardened cement paste and rust as the products of steel corrosion. For this operation the water sand blasting technic is selected with pressure of 250 bar. The procedure of cleaning reinforcing bars is shown in figure (VIII-105).

Figure (VIII-105) Rebars cleaning with water-sand blastering

4.5. Repaire works

The following recommended measures for repairing damaged concrete elements were: rebar protection, bonding agent application, injection of cracks, concrete restoration and execution of new cover, corrosion inhibition protection and surface protection.

In this case, the repair works on the damaged abutment walls was included crack injection and/or crack sealing. The cracks with wideness w≤0.2mm should be only sealed, but cracks with wideness w≥0.2mm should be sealed and injected. The crack w≥0.2mm have to be sealed with epoxy mortar prior to injection. Then, drilled packers have to be installed at the distance of 20cm, from each other. The holes for packers have to be cleaned by compressed air to remove dust. Injection of epoxy resin should be done with low pressure pump (≤1bar). After 24hours of injection, all surface protection and packers have to be removed. The view of noticed cruck in abutment and its preparation for injection are shown in Figures VIII-113 and VIII-114.

Some of the above-mentioned works on Bab Gheshir road bridge are illustrated in figures VIII-106 – VIII-112.

Figure (VIII-106) Corroded steel on the cantilevers

	
Figure (VIII-107) Finalizing underpass ceiling with cementitious base special mortar	Figure (VIII-108) Finalizing underpass ceiling with special mortar

	
Figure (VIII-109) Finalizing cantilevers and lateral beam with special mortar	Figure (VIII-110) Finalized lateral beam with special mortar

	
Figure (VIII-111) Finalizing deck ceiling with special mortar	Figure (VIII-112) plastering for ceiling

	
Figure (VIII-113) A very large crack in abutment wall	Figure (VIII-114) The crack in abutment prepared for injection

	
Figure (VIII-115) Abutment wall repair	Figure (VIII-116) Plastering of supporting wall

Figure (VIII-117) & (VIII-118) Painting the bridge

4.6. Materials recommended for repair of the bridge

After analyzing specifications of materials for structural and non-structural repair, principles and methods of repair and available materials in Libya, the next products has been selected:

Cementitious base material for rebar protection:	EmacoNanocrete AP(BASF)
Bond material:	EmacoNanocrete AP(BASF)
Cementitious base pre-mixed mortar:	EmacoNanocrete R4(BASF)
Currying protection material:	Masterkure 181(BASF)
Adhesive and repair mortar for crack sealing	Sikadur 31CF
Low Viscosity Injection Resin	Sikadur 52
Corrosion inhibition impregnation:	SikaFerroguard 903(SIKA)
Reinforcement rebar steel:	Grade A-615

The estimated quantities of listed products after visual inspection, after plaster removal and additional testing of materials and their real consumption are given in Table VIII-10.

Table VIII-10 - Calculated quantities of used products and their real consumption

Material	Estimated	Calculated after additional testing	Real quantities
EmacoNanocrete R4	3.775kg	65.250 kg	136.642kg
EmacoNanocrete AP	175kg	2.250 kg	1.500kg
Master flow 928/980	-	11.650 kg	19.250kg
Mastercure 181	100Lt	400Lt	400Lt
Sikadur 31CF	-	10 kg	-
Sikadur 52	-	123 kg	-
SikaFerroguard 903	-	550 Lt	525Lt
Sikaguuard 680 ES Betoncolor	550Lt	550 Lt	575Lt

In table VIII-11 the real consumption of used products per measuring unit is given.

Table VIII-11 - The real consumption of used products per measuring unit (ratio)

Quantity ratios					
Work	Material	Unit	Qty	Ratio	Average thickness
Concrete repair	Emaco Nanocrete R4	1.885,40m^2	136.642kg	72.47kg/m^2	38mm
	Emaco NanocreteAP	1.885,40m^2	1.500kg	0.80kg/m^2	-
	Master flow 928/980	108.60ml	19.250kg	177.3kg/ml	-

All recommended measures for repairing damaged concrete elements, such as rebar protection, reinforcement rebar replacement (if necessary), bonding agent application, concrete restoration, corrosion inhibition protection and surface protection, were described in detail in chapter 1, in which the detail program for repairing Bridge Souk Athulatha 1 was given.

4.7. Other Repair Works

Other repair works with aim to provide functionality involve procedures such as:

- Re-plaster and paint the surrounding walls of the bridge,
- Repair or replacement of sidewalks and curbstone,
- Removal of old fences and assemblage of new one,
- Installation of new catch pits,
- Execution of new asphalt layers and
- Assemblage a new traffic signs, electrical lights…etc.

Hereinafter the other repair works are described and illustrated.

During the repair of these bridges, it was decided to remove all old sidewalks on Center Island over bridges (Bab Bin Gheshir road and AL Sseka) and part of curbstone and old concrete under the fences over the bridges and between these bridges (Fig VIII-119). The detail of removal of damaged sidewalk is shown in figure (VIII-120).

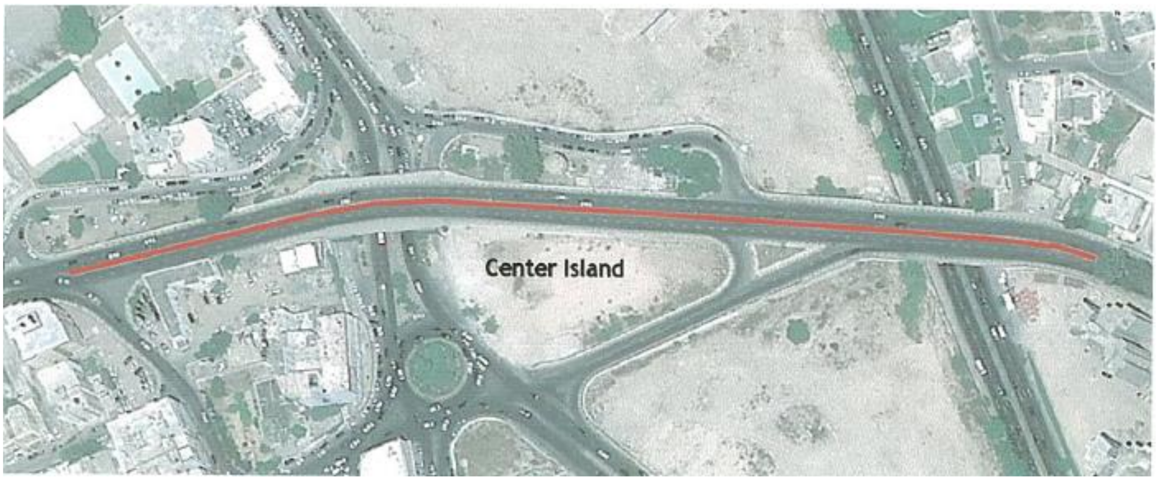
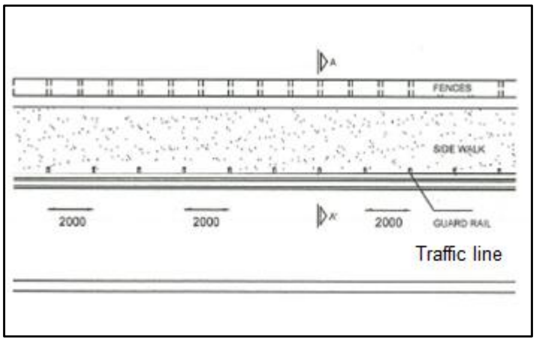


Figure (VIII-119) Site plan of bridges Bab Bin Gheshir road and AL Sseka


Figure (VIII-120) Removing of old sidewalks on bridge and between them

After repairing the edge beams and other structural elements of bridge and preparing sub base between the bridges, it has been planned to cast new sidewalks and curbstone (Fig VIII-121 and VIII-122).



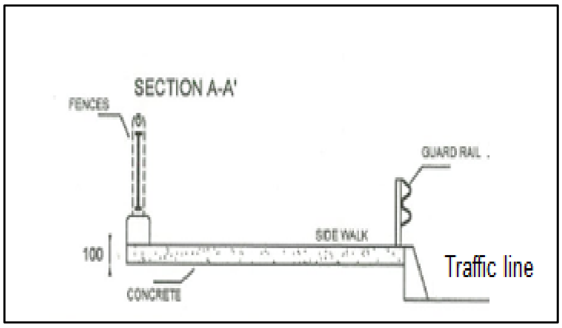


Figure (VIII-121) Sidewalk elements arrangement

Figure (VIII-122) Cross section of sidewalk elements

This operation includes following steps:

- Concreting of beam under side walk and lining of sub base layer,
- Concreting of the sidewalk slabs with dimensions 1.20m×2.50mx0.1m on top of concrete beam, with 2cm expansion joint filled with elastic material.
- Concreting of the layer under the fences with the height 10cm above the sidewalk level.
- Casting the new lean concrete under the new curbstone with a 30cm width.
- Concreting of new curbstone with expansion joint every 12m.
- Concreting and installing of the new catch pit.

The cross section of new sidewalks between the bridges with all necessary elements is shown in Fig VIII-123.

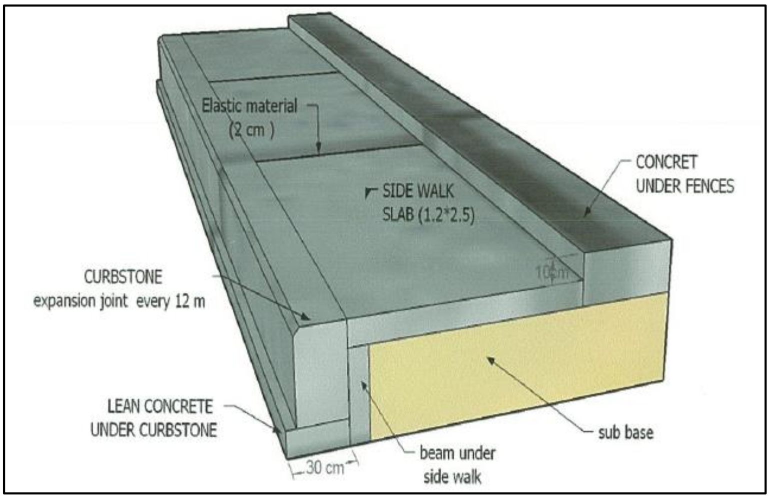
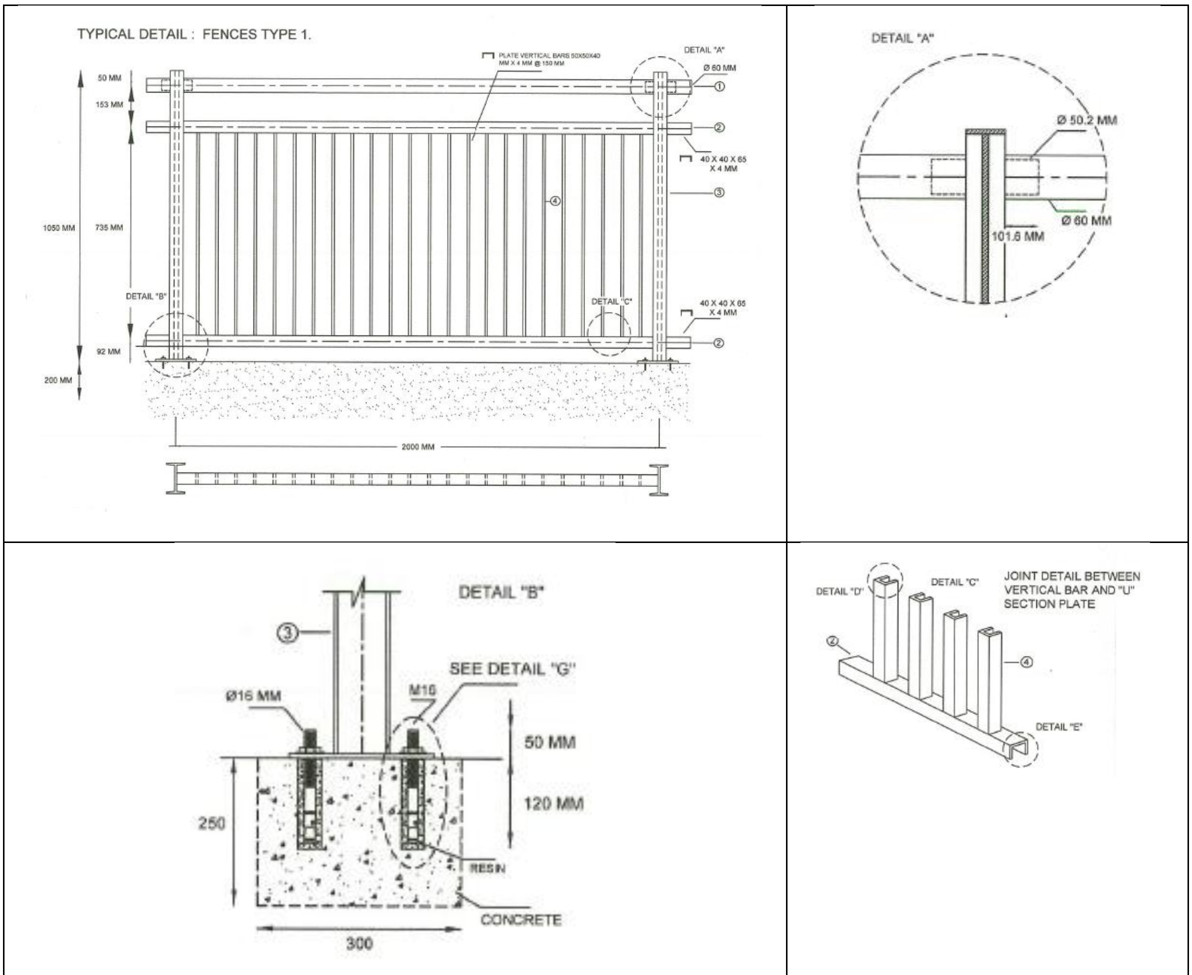


Figure (VIII-123) new sidewalk between the bridges Bab Bin Gheshir road and AL Seeka

Figure (VIII-124) shows the details of new fence and the plan of installing the new fence.



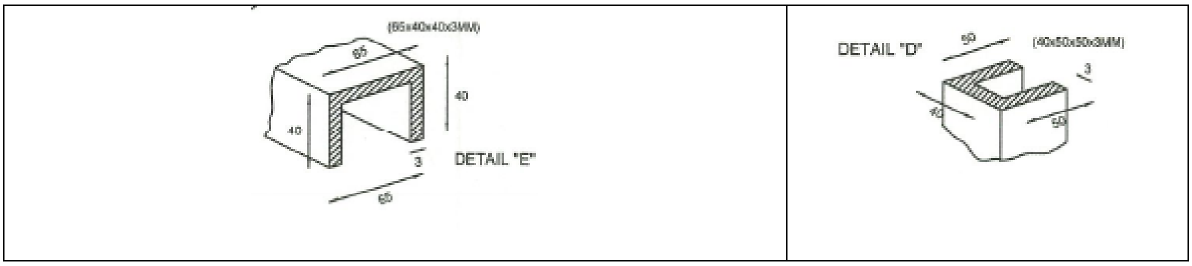


Figure (VIII-124) Fences view and installing plan

Some details from casting of described elements are presented in Figures VIII-125 – VIII-127.

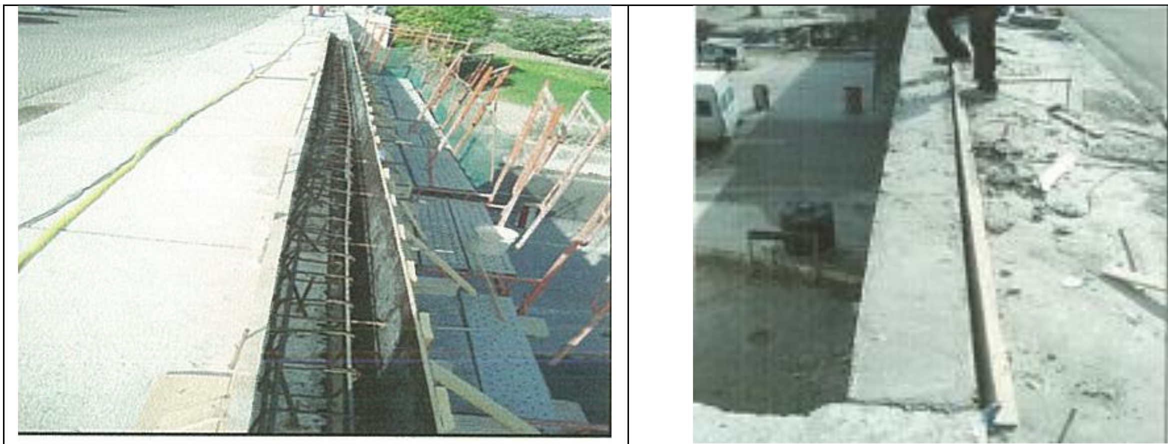


Figure (VIII-125) Preparing for casting of and casted edge beam

Figure (VIII-126) New sidewalks, new curb stone and new catch pit

Figure (VIII-127) Protecting and fixed new fence

4.8. Asphalt Works

The following activities have been planned for repairing the traffic lines:

- Removing the surface layer over the bridge (t=4cm) using milling machine.
- Opening the cracks
- Cleaning cracks and whole surfaces
- Injecting of all cracks in binder course
- Laying of new wearing layer

In fig (VIII-128) and (VIII-129) the condition of upper layer of asphalt before removing of surface layer and the view after its removal are shown.

Figure (VIII-128) A view of old asphalt layer on the bridge before removing of surface layer

Figure (VIII-129) The view of surface after asphalt was removed

Since a lot of cracks have been noticed in down layer, it was decided to fulfill them by injection. Before injection, the cracks have to be opened and cleaned. All these operations are shown in fig VIII-130 and VIII-131.

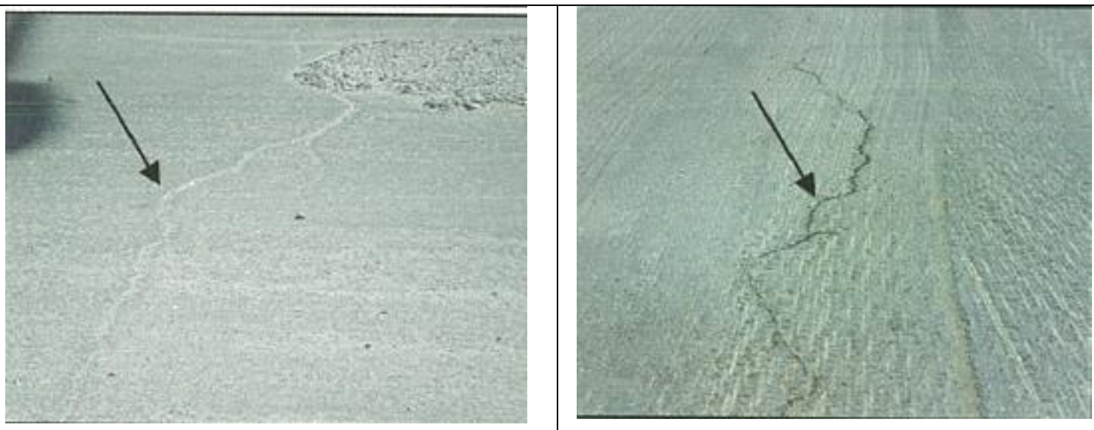


Figure (VIII-130) cracks before injection

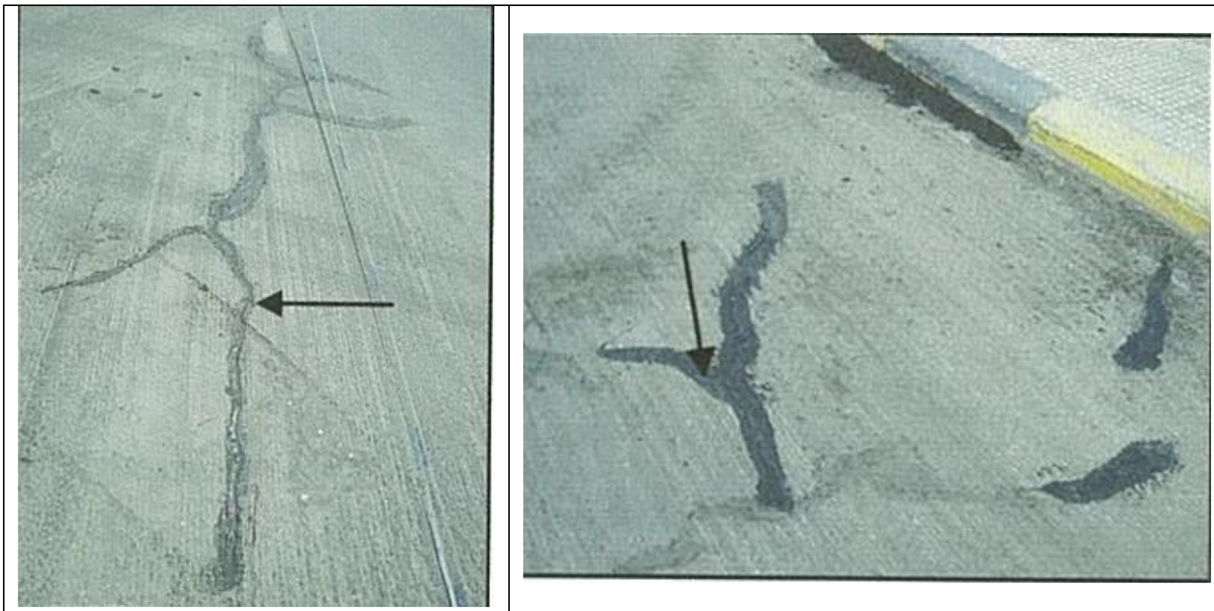


Figure (VIII-131) Cracks after injection

After finishing of all preparing works, the new wearing asphalt layer has to be placed.

Placing of new bituminous wearing layer encompasses:

1. Cleaning the surface by using air compressor.
2. Spraying bitumen MC-250 by using MC tank according to required spray rate.
3. Checking of the temperature of asphalt mixture when the trucks arrive.
4. Controlling of thickness of asphalt by elevation of the steel wires.
5. Spreading wearing course by automatic controlled pavers.
6. Compacting asphalt by using steel roller and tire roller.
7. Taking asphalt cores for checking the thickness and compaction.

Phases of placing bituminous wearing layer are shown in figures (VIII-132), and the final view of bituminous wearing layer is shown in figures (VIII-133).

Figure (VIII-132) Phases of placing of bituminous materials

Conti. ….Figure (VIII-132) Phases of placing of bituminous materials

Figure (VIII-133) The road after finish asphalting

Other nonstructural repair measure is painting of protective walls and traffic lines (fig VIII-134 and VIII-135).

Figure (VIII-134) Painting of protective walls

The quantities of works, such as works on sidewalks, fences, plastering of approaching structure and etc., are shown in table VIII-12.

Table VIII-12 The quantities of other works on the bridge and approaching structure.

Number	description	unit	quantity
1	Removal of sidewalk	m^2	246.16
2	Replacement of sidewalk	m^2	246.16
3	Removal of curbstone	m^1	579.20
4	Replacement of curbstone	m^1	579.20
5	Removal of fences	m^1	108.00
6	New fences	m^1	108.00
7	Repair of fences	m^1	404.5
8	Removal plaster	m^2	4,356.99
9	New plaster (first layer)	m^2	2,471.59
9.1	New plaster (second layer)	m^2	2,471.59
10	Nanocrete	m^2	1,885.40
11	Removal of asphalt	m^2	4,458.00
12	New asphalt	m^2	4,458.00
13	Paint	m^2	1,885.40
14	Graffiti	m^2	2,471.59

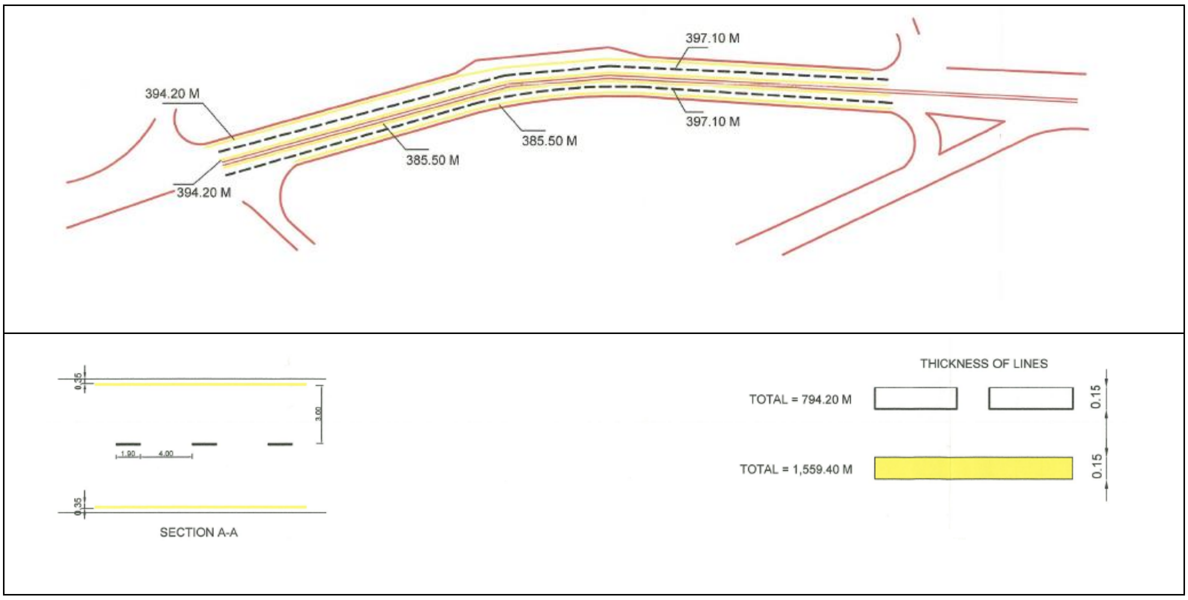


Figure (VIII-135) Plan of traffic lines paintworks

4.9. Plan of expansion Joint

In origin design, the expansion joints were planned in four places. The layout of expansion joints is given in Figure 6.303. The description of suggested repair measure for them is also given in figure6.303, and specific detail could be seen in Figure VIII-136.

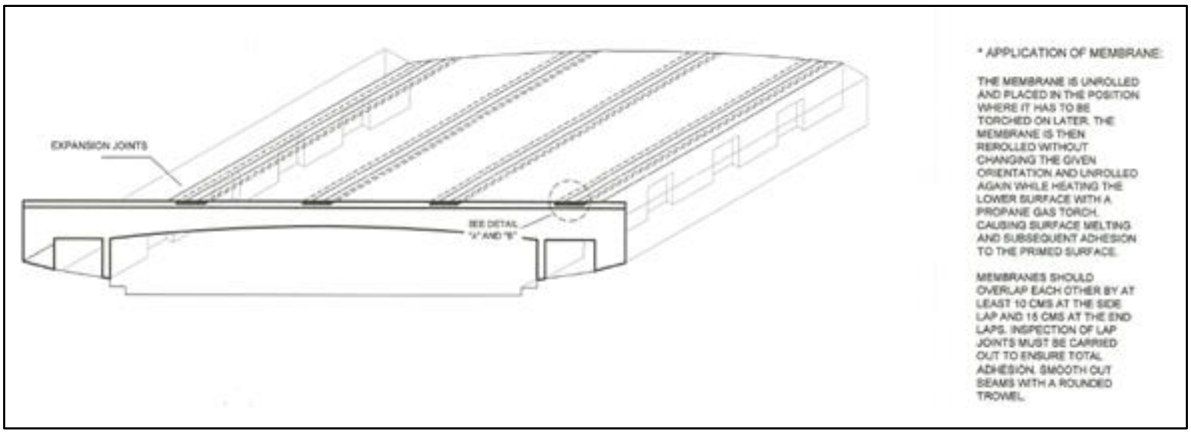


Figure (VIII-136) Plan of expansion joints in bridge structure

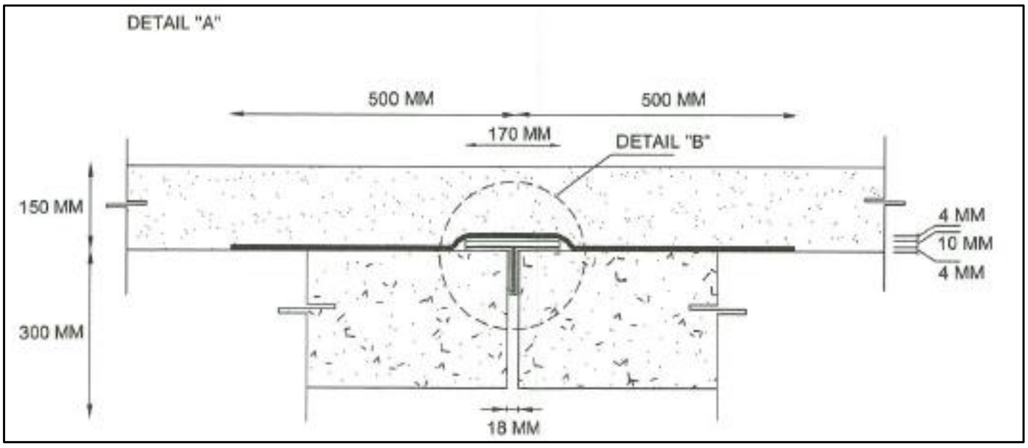


Figure (VIII-137) A detail of setting up of membrane over the expansion joints in bridge structure

Recapitulation of all designed repair works on RC elements of Bab Bin Gheshir Road Bridge

	Arch slab (ceiling)	Lateral beams	Cantilever slabs	Underpass ceiling	Supporting walls	Abutment walls	Edge beam
Removal of concrete cover	+	+	+	+	+	+	The whole
Rebar treatment	+	+	+	+	+	+	New reinforcement
Rebar protection	+	+	+	+	+	+	+
Execution of new concrete cover	+ With increased depth, by plastering	+ With increased depth, by plastering	+ by plastering	+ by plastering	+ by plastering	+ by plastering	New concrete
Corrosion inhibition impregnation	+	+	+	+	+	+	+
Surface protection	+ paint	+	+	+	+	+	
Execution of new element							+
Crack injection						+	

5. AL SREEM ROAD BRIDGE

5.1. Introduction

Through assessment of the condition of structural elements of this bridge the next conclusions have been derived:

The most damaged elements are longitudinal slab beams. The characteristic damages are crashed concrete and deformed, twisted and even broken reinforced rebars. Some reinforced bars were bared and then affected by surface corrosion.

The main cause of described damages was hitting by truck. External longitudinal beams of both slab decks are significantly damaged because they were hit by truck several times.

The next characteristic damage is local spalling of mortar layer from down surface of cantilever slabs and slab beams and from masonry elements (exterior and interior walls).

Traces of water leakage and of over flow water could be seen in gap between deck slabs and on down surfaces of cantilever slabs.

The carbonization process has started and it is more pronounced in longitudinal slab beams. The depth of carbonization varies from 10-30mm.

Visual inspection encompassed other bridge elements, like sidewalks, curb stones, catch pits and fences. All mentioned elements have been seriously damaged.

As the carbonization and bad concrete quality cover present general problem, all elements which have been affected by carbonization or have insufficient concrete cover depth, have to be repaired. These elements are:

- Deck slab
- Cantilever slab
- Abutments wall
- Supporting wall
- Longitudinal and transversal supporting (ceiling) beams

Repair measures include removing the old carbonized concrete cover and execution of new cover with increased depth on slab ceiling and ceiling beams.

Repair measures also include cleaning, protection or even replacement of corroded bars.

For better bonding between old, but healthy, concrete and new cover the special agents is proposed.

Also, the special coatings are suggested for concrete surface protection.

Apart from removing concrete cover from upper mentioned elements, removal works were carried out on concrete part of fences all old concrete sidewalks.

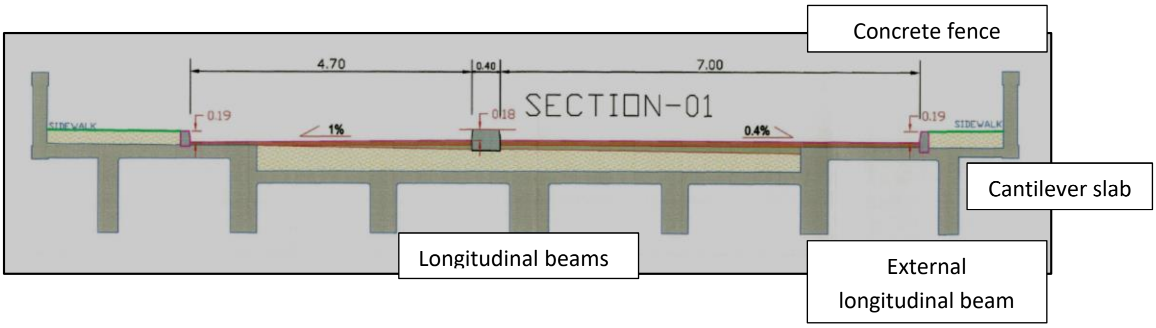


Figure (VIII-138) The cross section of superstructure

5.2. Repair measures

For increasing of longevity of this bridge the next repair measures were recommended for superstructure:

Hereinafter all recommended measures will be described in detail.

For repairing of masonry abutment walls the following measures were suggested:

Complete removal of mortar/plaster layer,

Adding of new wire mash

Rebar protection,

Execution of new mortar cover by plastering

Surface protection with coating of all repaired concrete elements.

5.3. Damaged concrete removal

As it was recommended, all poor-quality concrete cover or/and carbonated part of concrete have to be removed. For that operation the electric hammers with max weight of 6kg have been chosen.

According with the testing results the major problem was carbonization with in depths of 10 to 30 mm, frequently over passing the rebar plan.

The removal of concrete close to bars has to be done very carefully to avoid the damage of bars.

After finishing concrete removal all surfaces must be clean with water jet with pressure of 200bar.The application of water jet has to be with angle of 45^0 and with 5 cm distance.

The removal of old porous and carbonated concrete cover from abutment wall and ceiling beam is shown in figures VIII-139 – VIII-141.

Figure (VIII-139) Damaged cover removal from concrete part of fence, top of the bridge

Figure (VIII-140) Damaged concrete cover removal from longitudinal slab beam

Figure (VIII-141) Damaged plastering and concrete removal from the abutments and sidewalk

5.4. Cleaning of reinforcement bars

Figure (VIII-142) Cleaning of rebars with water-sand blastering, deck ceiling beams

Figure (VIII-143) Cleaning of rebars with water-sand blastering, top beam of the support wall

5.5. Replacement/ adding of reinforcement bars

Twisted, deformed and broken reinforcing bars in longitudinal slab beams were replaced with new rebars (Fig VIII-144 & VIII-145).

Figure.VIII-144 & VIII-145. Rebar replacement on the damaged longitudinal slab beams

5.6. Repairing works

The following recommended measures for repairing damaged concrete elements were: rebar protection, reinforcement rebar replacement, bonding agent application, concrete restoration and execution of new cover, corrosion inhibition protection and surface protection.

For reprofilizing the removed part of longitudinal beams, two technics were used:

- Grouting/pouring of self-compacting mortar into prepared wooden ply formwork for bottom part of beam (Fig. VIII-146), and
- Plastering with mortar for vertical (side) surfaces.

Some of the above-mentioned works on Al Sreem road bridge are illustrated in figures VIII-146 – VIII-151.

Figure (VIII-146) Reprofilation of lateral slab beams, bottom part

Figure (VIII-147) Execution of new cover on ceiling of bridge superstructure by plastering

Figure (VIII-148) Repaired top beam of stone supporting wall

Figure (VIII-149) plastering of abutment wall

Figure (VIII-150) Repaired and painted concrete part of fence

Figure (VIII-151) View of bridge after repairing and painting measures

5.7. Materials recommended for repair of the bridge

After analyzing specifications of materials for structural and non-structural repair, principles and methods of repair and available materials in Libya, the next products has been selected:

Cementitious base material for rebar protection:	EmacoNanocreteAP(BASF)
Bond material:	EmacoNanocrete AP(BASF)
Cementitious base pre-mixed mortar:	EmacoNanocreteR4(BASF)
Currying protection material:	Masterkure 181(BASF)
Corrosion inhibition impregnation:	SikaFerroguard 903(SIKA)
Surface protective and paint layer:	Sikaguard 680 ES Betoncolor
Reinforcement rebar steel:	Grade A-615

The estimated quantities of listed products after visual inspection, after plaster removal and additional testing of materials and their real consumption are given in Table VIII-13.

Table VIII-13 - Calculated quantities of used products and their real consumption

Material	Estimated	Calculated after additional testing	Real quantities
EmacoNanocrete R4	18.350kg	40.000kg	42.800kg
EmacoNanocrete AP	540kg	1.468kg	1.100kg
Master flow 928/980	-	18.950kg	3.600kg
Mastercure 181	200Lt	245Lt	400Lt
Sikadur 31CF	-	-	-
Sikadur 52	-	-	-
SikaFerroguard 903	-	950 Lt	900Lt
Sikaguuard 680 ES Betoncolor	400Lt	950 Lt	1000Lt

In table VIII-14, the difference between calculated and real quantities of material is given.

Table VIII-14

Quantity ratios					
Work	Material	Unit	Qty	Ratio	Average thickness
Concrete repair	EmacoNanocrete R4	942.65m^2	42.800kg	45.4kg/m^2	24mm
	EmacoNanocreteAP	942.65m^2	1.100kg	1.16kg/m^2	-
	Master flow 928/980	24.90ml	3.600kg	144.6kg/ml	-

All recommended measures for repairing damaged concrete elements, such as rebar protection, reinforcement rebar replacement (if necessary), bonding agent application, concrete restoration, corrosion inhibition protection and surface protection, were described in detail in chapter 1, in which the detail program for repairing Bridge Souk Athulatha 1 was given.

5.8. Other Repair Works

Other repair works with aim to provide functionality involve procedures such as:

- Re-plaster and paint the surrounding walls of the bridge,
- Repair or replacement of sidewalks and curbstone,
- Removal of old metal fences and assemblage of new one,
- Execution of new asphalt layers and
- Assemblage a new traffic signs, electrical lights…etc.

Hereinafter the other repair works are described and illustrated.

The detail of execution of new sidewalks is shown in figure VIII-152 and VIII-153.

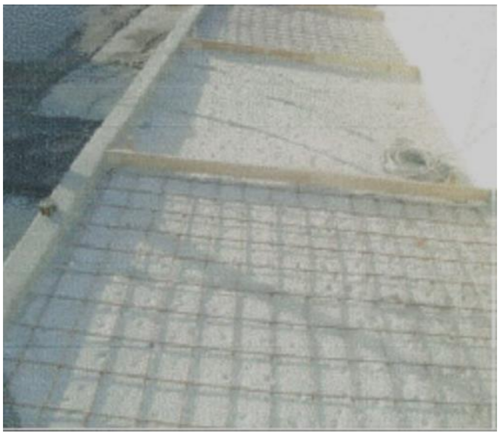


Figure (VIII-152) New wire mesh for new layer of concrete sidewalk

Figure (VIII-153) New sidewalk

5.9 Asphalt Works

After removal of old wearing asphalt layer the new one was executed. The reclaimed asphalt was used.

The installing of new asphalt wearing course layer covers next activities:

- Cleaning was performed by brushes directly after milling & by air compressor prior to spreading tack coat,
- Tack coat was spread within specification limits (0.1-0.6 lt/m^2),
- Laboratory team checks the temp. Of asphalt mixture for each asphalt truck to maintain the temp. of asphalt to be more than 135c^0,
- Asphalt mix sample was taken for performing bitumen extraction marshall stability & sieve analysis tests,
- Wearing course was laid by automatic controlled pavers,
- Compacting was performed by using steel roller and pneumatic tire roller,

- Required cores were taken to check the thickness & compaction.

The whole procedure is shortly shown in following figures VIII-154 - VIII-156.

Figure (VIII-154) Removing existing asphalt wearing layer, reclaimed asphalt

Figure (VIII-155) Spreading tack coat (RC2)

Figure (VIII-156) Placing the new wearing course

5.10. Expansion joint repairing:

Due to leakage problem, the expansion joints needed to be repaired. The procedure covers:

- Removing asphalt road layers,
- Cleaning of concrete surface,
- Placing of steel sheet over the joint,
- Properly fixing of waterproofing membrane to avoid water leakage and
- Placing new asphalt layer

The procedure is shown in figures VIII-157-VIII-159.

		
Figure (VIII-157) View of expansion joint before repair, after removal of old asphalt cover	Figure (VIII-158) Joint after fixing waterproofing membrane	Figure (VIII-159) Placing of new asphalt layer

Recapitulation of all designed repair works on RC elements of Al Sreem Road Bridge

	Deck topping slab	Longitudinal and transversal supporting (ceiling) beams	Cantilever slabs	Supporting walls	Abutment walls	RC part of fence
Removal of concrete cover	+	+	+		+ (mortar)	+
Replacement of deformed and broken of rebars		+ (longitudinal)				
Rebar treatment		+		+ (top beam)		+
Rebar protection		+		+		+
Execution of new concrete cover	+	+ (by grouting & by plastering)	+ (mortar)	+ (top beam)	+ (mortar layer)	+ (mortar)
Corrosion inhibition impregnation	+	+	+	+ (top beam)		+
Surface protection	+	+	+	+ (top beam)	+	+

6. ALSHAAB PORT BRIDGE

6.1. Introduction

The bridge Alshaab has been old about 50 years when it was inspected for the first time. This bridge is single span structure with RC superstructure and masonry abutments made of local stone. The main conclusion of the inspection was that the bridge is damaged.

The characteristic defect of reinforced elements has been insufficient concrete caver.

The main cause of damage appearance is insufficient concrete cover. Measured value of concrete cover in elements of superstructure (ceiling beams and slabs) is only 5mm.

The second cause of damage appearance is concrete carbonization. Depth of carbonization varied from 20mm up to 80mm and in all tested locations front of carbonation passed behind the reinforced bars.

The next cause of damage appearance is inadequate drainage of water from the deck. This problem caused leakage of water over the edge of cantilever slabs. Consequently, the corrosion of reinforced bars in deck ceiling and cantilever slabs were caused.

Analyzing concrete compressive strength obtained by cores it can be concluded that built-in concrete has very unequal quality and compressive strength differ from one to another location.

As the carbonization, insufficient concrete cover and reinforcement corrosion present general problem, all RC elements which have one or more mentioned problem have to be repaired. These elements are:

Longitudinal and transversal slab beams,

- Ceiling of topping slab
- Cantilever slab

Repair measures include removing the old cement plaster and the old carbonized concrete cover and execution of new cover with increased depth.

Repair measures also include cleaning, protection or even replacement of corroded bars.

For better bonding between old, but healthy, concrete and new cover the special agents is proposed.

Also, the special coatings are suggested for concrete surface protection.

Plaster layer of masonry abutment walls was damaged.

Apart from removing concrete cover from upper mentioned elements, removal works were carried out on concrete part of fences all old concrete sidewalks.

Repair measure of bridges in Tripoli in 2009

For increasing of longevity of this bridge the next repair measures were recommended:

- Complete concrete cover removal,
- Complete rebar treatment (even including rebar replacement)
- Crack injection,
- Rebar replacement,
- Rebar protection
- Adding a new reinforcing bars
- Execution of new concrete cover with minimum depth of 20mm.
- Corrosion inhibition impregnation of all bridge exposed surfaces Surface protection with coating of all repaired concrete elements.
- Hereinafter all recommended measures will be described in detail.
- For repairing of masonry abutment walls the following measures were suggested:

- Complete removal of mortar/plaster layer,
- Adding of new wire mash
- Rebar protection,
- Execution of new mortar cover by plastering
- Surface protection with coating of all repaired concrete elements.

6.2. Damaged concrete and plaster removal

As it was recommended, all poor quality concrete cover or/and carbonated part of concrete have to be removed. For that operation the electric hammers with max weight of 6kg have been chosen.

According with the testing results the major problem was carbonization within depths of 40 to 60 mm, frequently over passing the rebar plan.

The removal of concrete close to bars has to be done very carefully to avoid the damage of bars.

After finishing concrete removal all surfaces must be clean with water jet with pressure of 200bar.The application of water jet has to be with angle of 45^0 and with 5 cm distance.

6.3. Cleaning of reinforcement bars

After removing of “old” damaged concrete cover, the next operation has been cleaning of reinforcing bars from remains of hardened cement paste and rust as the products of steel corrosion. For this operation the water sand blasting technic is selected with pressure of 250 bar.

6.4. Crack Injection

During the removing damaged concrete from ceiling of topping slab, several cracks were spotted. In this case, the repair works on the damaged topping slab was included crack injection. The cracks with wideness $w \geq 0.2$mm should be sealed and injected. The crack $w \geq 0.2$mm have to be sealed with epoxy mortar prior to injection. Then, drilled packers have to be installed at the distance of 20cm, from each other. The holes for packers have to be cleaned by compressed air to remove dust. Injection of epoxy resin should be done with low pressure pump ($\leq$1bar). After 24hours of injection, all surface protection and packers have to be removed. The view of noticed cruck in topping slab is shown in Figures VIII-160 and VIII-161.

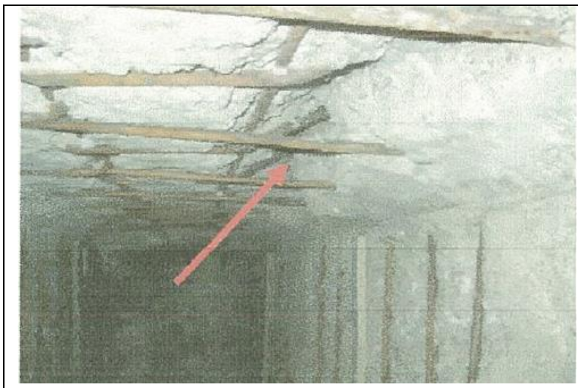
 

Figure (VIII-160 & VI-161) Crack in topping slab

6.5. Reinforcement rebars replacement

If rebar loses more than 28% of the area due to corrosion, it must be complemented, if it loses more than 50 % it must be replaced.

All complementation rebars must have sufficient anchoring length. In situation when there is not enough space for anchoring rebars, they have to be welded to existing bars.

Rods can also be replaced when there is not enough space to complement.

6.6. Repairing works

The following recommended measures for repairing damaged concrete elements were: rebar protection, reinforcement rebar replacement, bonding agent application, concrete restoration and execution of new cover, corrosion inhibition protection and surface protection.

For reprofilizing the removed part of longitudinal beams, two technics were used:

- Grouting/pouring of self-compacting mortar into prepared wooden ply formwork for bottom part of beam (Fig. VIII-165 & VIII-166), and
- Plastering with mortar for vertical (side) surfaces.

Some of the above mentioned works on repar of Al Shaab port bridge are illustrated in figures VIII-162 – VIII-169.

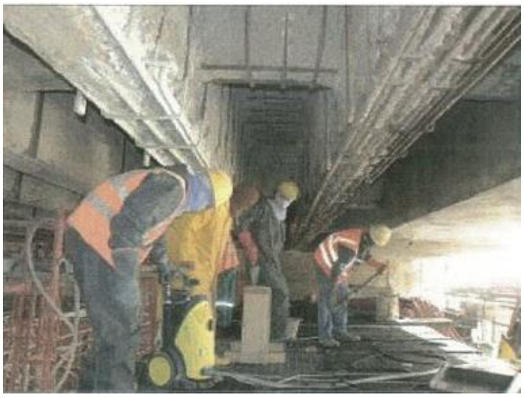
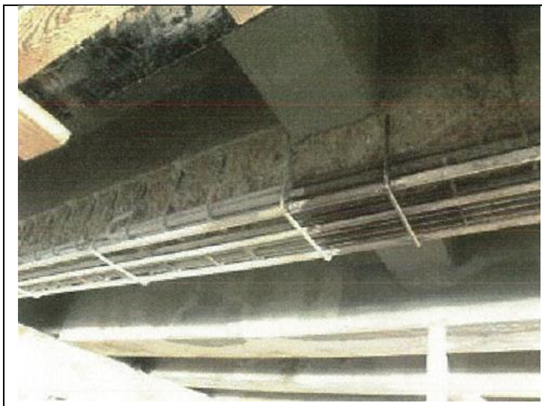


Figure (VIII-162) Removal of damaged concrete and rebars cleaning with water-sand blastering, longitudinal slab beam

Figure (VIII-163) Reinforcement rods after cleaning, longitudinal slab beam



Figure(VIII-164) Rebar protective coating, longitudinal slab beam

Figure (VIII-165) Longitudinal slab beam bottom grouting

Figure (VIII-166) Longitudinal slab beam bottom repaired

Figure (VIII-167) View of superstructure of the bridge, after painting

Figure (VIII-168) Abutment wall wire mash application

Figure (VIII-169) View of abutment wall after plastering and painting

Cementitious base material for rebar protection:	EmacoNanocreteAP(BASF)
Bond material:	EmacoNanocrete AP(BASF)
Cementitious base pre-mixed mortar:	EmacoNanocreteR4(BASF)
Currying protection material:	Masterkure 181(BASF)
Corrosion inhibition impregnation:	SikaFerroguard 903(SIKA)
Surface protective and paint layer:	Sikaguard 680 ES Betoncolor
Reinforcement rebar steel:	Grade A-615

6.7. Materials recommended for repair of the bridge

After analyzing specifications of materials for structural and non-structural repair, principles and methods of repair and available materials in Libya, the next products has been selected:

The estimated quantities of listed products after visual inspection, after plaster removal and additional testing of materials and their real consumption are given in Table VIII-15.

Table VIII-15 - Calculated quantities of used products and their real consumption

Material	Estimated	Calculated after additional testing	Real quantities
EmacoNanocrete R4	18.200kg	54.150kg	66.500kg
EmacoNanocrete AP	550kg	2.775kg	750kg
Master flow 928/980	-	4.800kg	10.255kg
Mastercure 181	200Lt	600Lt	400Lt
Sikadur 31CF	-	-	-
Sikadur 52	-	-	20Lt
SikaFerroguard 903	-	475 Lt	400Lt
Sikaguuard 680 ES Betoncolor	550Lt	475 Lt	400Lt

In table VIII-16 the difference between calculated and real quantities of material is given.

Table VIII-16

Quantity ratios					
Work	Material	Unit	Qty	Ratio	Average thickness
Concrete repair	EmacoNanocrete R4	953.75m^2	66.500 kg	69.72 kg/m^2	37mm
	EmacoNanocreteAP	953.75m^2	750 kg	0.79 kg/m^2	-
	Master flow 928/980	94.50ml	10.255 kg	108 kg/ml	-

All recommended measures for repairing damaged concrete elements, such as rebar protection, reinforcement rebar replacement (if necessary), bonding agent application, concrete restoration, corrosion inhibition protection and surface protection, were described in detail in chapter 1, in which the detail program for repairing Bridge Souk Athulatha 1 was given.

6.8. Other Repair Works

Other repair works with aim to provide functionality involve procedures such as:

- Removal and execution of new RC beam for fence with concrete C25/30,
- Removal of damaged part of sidewalks and execution the new one by installing new wire mash and casting new concrete C25/30, with expansion joints every 2m in length. Joints have to be filled with bituminous material.
- Removal of old metal fences and assemblage of new one,
- Assemblage a new traffic signs, electrical lights…etc.

- Re-plaster and paint the surrounding walls of the bridge,

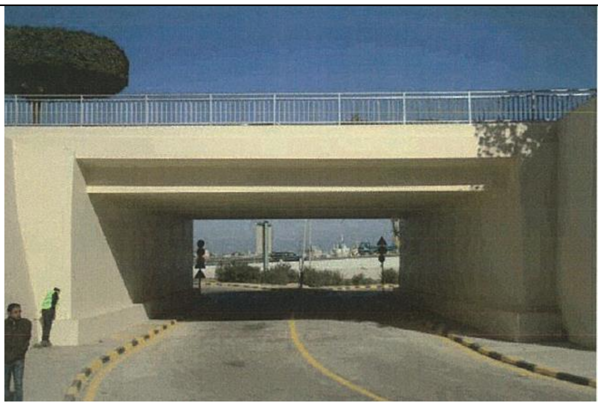
Hereinafter the other repair works are described and illustrated (Fig VIII-170-VIII-172).

Figure (VIII-170) Removal of “old” beam for fence

Figure (VIII-171)The new RC beam for fence: casting of concrete in framework

Figure (VIII-172) Execution of a new part of sidewalk under bridge

The view Al Shaab port bridge after all measures for its recovering were applied is given in Fig VIII-173 &VIII-174.

	
Figure VIII-173 &VI-174. The view of bridge after reconstruction	

Recapitulation of all designed repair works on RC elements of Al Shaab Port Bridge

	Longitudinal and transversal slab beams	Ceiling of topping slab	Cantilever slabs	masonry Abutment walls
Removal of concrete cover	+ longitudinal	+	+	+ (plaster)
Replacement of deformed and broken of rebars	+	+		+(new wire mash)
Rebar treatment	+	+		
Rebar protection	+ longitudinal	+		
Execution of new concrete cover	+ (by gouting and by plastering)	+ (mortar)	+ (mortar)	+ (mortar)
Corrosion inhibition impregnation	+	+		
Surface protection	+	+	+	+
Crack injection		+		

7. ABDUL SALAM AREF BRIDGE

7.1 Introduction

Through the assessment of the condition of structural elements of Abdul Salam Aref bridge the next conclusions have been derived:

The most damage elements are supporting columns and abutments. The main cause of damages is corrosion of reinforcing bars. The delamination and spalling off of caver affected a large area of columns, especially in corners, and a large area of abutments, also. Exposed reinforcing bars lost adhesion with concrete core. During visual inspection an inadequate arrangement of stirrups has been noticed in columns and an inadequate arrangement of horizontal and vertical rebars has been spotted in abutments, also. The distance between stirrups and reinforcing bars is too large. Bearing capacity of supporting columns and abutments is jeopardized because of significant decrease of concrete cross section, corrosion of steel and losing of adhesion between rebars and surrounding concrete.

Other concrete elements also have damages caused by corrosion of still, but the degree of registered damages is lower than those in columns and abutments.

The carbonization of concrete presents general problem of all RC elements. In addition, on the basis of the results obtained by Pull-off and Schmidt hammer tests, it is concluded that concrete cover of columns and abutments is porous, with low hardness and tensile strength.

Since the stability and the bearing capacity of vital bridge elements have been jeopardized a part of suggested repair measures belongs to the group of structural repair. These measures are related to supporting columns and abutments. For other RC bridge elements non-structural repair measures in a form of execution of new cover and surface protection are suggested. The exception is made for RC beam for fence, where its upgrading is also suggested because of fixing new fence.

7.2. Structural repair

Structural repair is recommended for all supporting columns and both abutments. It includes strengthening of both elements by adding a new reinforcing bars and a new concrete layer. The dimensions of cross section of columns after repair will be increased from 50x50cm by 70x70cm. Since existing concrete cover of abutments is too large (~10cm), the strengthening by adding new reinforcing bars and cover will not cause an increasing of cross section dimensions.

Repair measures for supporting columns and abutments include removal of all porous, cracked, delaminated and carbonized concrete cover, up to the “healthy” concrete, cleaning of reinforcing bars, adding a new reinforcing bars and execution of new concrete layer.

Before executing of strengthening repair of columns the additional temporary supports have to be use. The view and location of support towers is given in Fig (VIII-175)

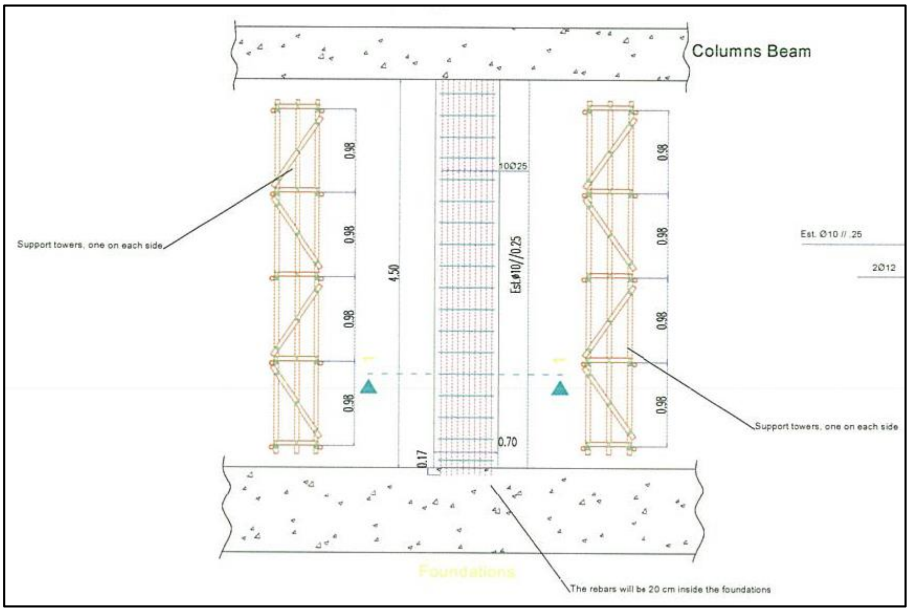


Figure (VIII-175) Support towers for temporary supporting upper part of bridge during column repair

Strengthening of columns includes: enlargement of cross section by addition of new reinforcing bars 4cm far from existing bars and a new cover with thickness of 5cm. The characteristic cross section of strengthened columns is shown in Fig (VIII-176)

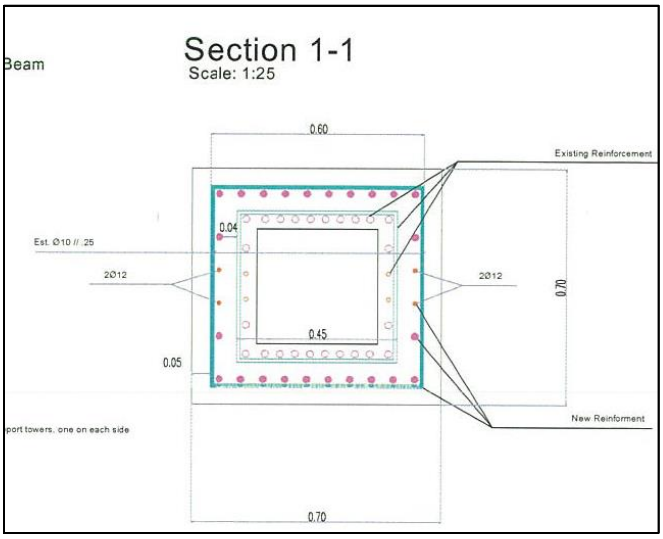


Figure (VIII-176) Cross section of strengthened columns with arrangement of new reinforcing bars

The thickness of concrete layer, which should be removed from abutments, is quite large and varies from 10 to 30cm. Because of that it is recommended to use concrete as a material for reprofilation of damage abutments. The abutments are massive concrete elements and they need steel mash just to avoid damage of concrete due to shrinkage. The plan of reprofilation of damaged abutment is given in Figure (VIII-177)

As it can be seen, it is suggested to split reparation in two phases: The 1st phase that includes down part of abutment up to the height of 2,4m and the 2nd phase that includes rest (upper) part of abutment.

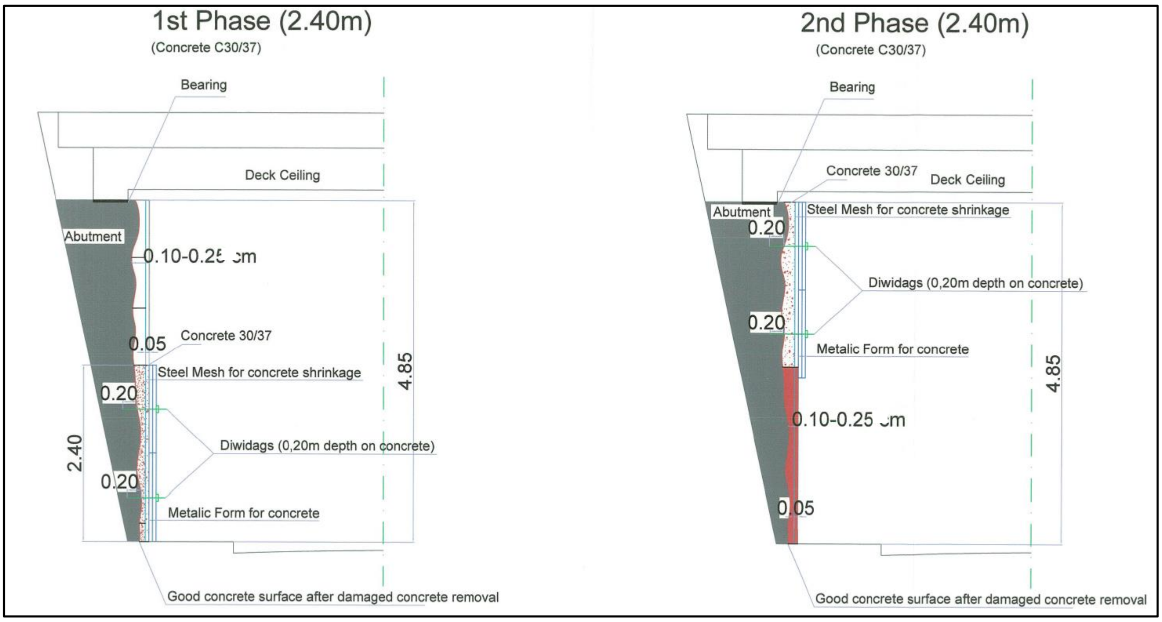


Figure (VIII-177) Plan of two stage repair of damaged abutments; adding of new steel mash for shrinkage control and new concrete layer

The third elements for structural repair are RC beams for fence. According to visual inspection, those beams were not seriously damaged, but it is supposed that the upper part of those beams will be damaged during the removal of the fence. Also, installation of new fence requires upgrading of the beams. The way of upgrading beams for fence can be seen in Figure (VIII-178).

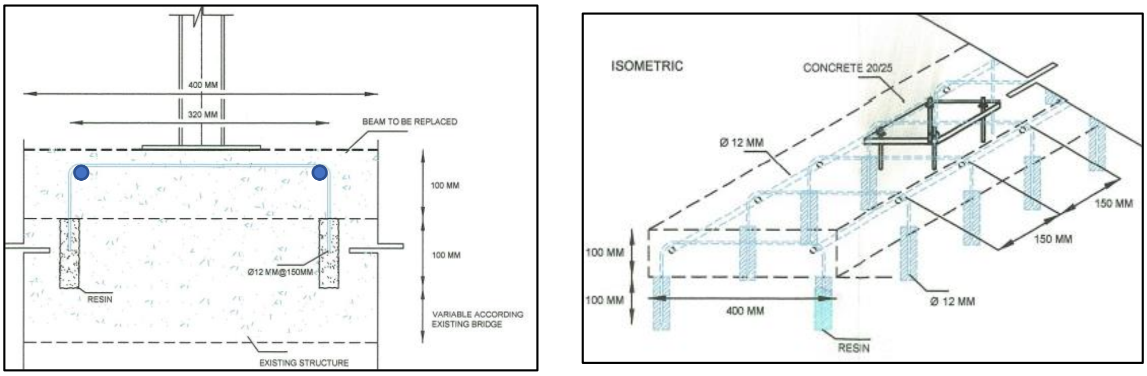


Figure (VIII-178) Upgrading of beam for fence: arrangement of reinforcing bars and the way of fixing Π bars in “old” concrete

7.3. Nonstructural repair

Nonstructural repair measures are planned for side surfaces of beams for fence and down parts of deck ceiling beams and top slab. Repair measures include removal of old carbonized and/or damaged concrete cover, execution of new cover with increased thickness. The special type of factory made mortar or grout is chosen for new cover.

Repair measures also include cleaning, protection or even replacement of corroded reinforcing bars.

For better bonding between old, but healthy concrete and new cover the special agents is proposed.

Also, the special coating is suggested for concrete surface protection.

7.4. Repair measure of bridges in Tripoli in 2009

For increasing of bearing capacity and service life of Abdul Salam Aref bridge next structural repair measures are recommended:

Temporary supporting of transversal beam in the vicinity of two columns that are chosen for repair by support towers,

Complete removal of damaged concrete and cover from all columns, abutments and beams for fence

Complete treatment of bared rebars,

Addition of new reinforcing bars with adequate anchoring and fixing, according to the plan (figs VIII-175-VIII-178 columns, abutment, and beam for fence). For columns it is suggested to add 100% of existing reinforcement. For abutments it is recommended to add steel mash to prevent concrete damage caused by shrinkage.

Rebar protection with cementitious base protection material,

Formwork installation,

Pouring of concrete C30/37 with cover of 5cm,

Surface protection with protecting coating.

To prolong of service life of rest of bridge RC elements next non-structural repair measures are recommended:

Complete concrete cover removal,

Complete treatment of bared rebar at ST2 grade with mixed water-sand blasting,

Rebar protection with cementitious base protection material,

Bonding slurry application for improving adhesion of repair mortar,

Execution of new cover with minimum depth of 20mm by repair mortar application

Application of curing agents

Surface protection with coating.

All above mentioned operations for non-structural repair are described in detail in Souk Athlatha 1 bridge repair measures chapter (7).

Hereinafter the short description of recommended measures for structural repair is given.

7.5. The short description of structural repair measures

As it was recommended, all poor quality concrete cover or/and carbonized part of concrete have to be removed. For that operation the electric hammers with max weight of 6kg have been chosen.

The estimated thickness of concrete that should be removed varies from 5 up to 30cm.

The removal of concrete close to bars has to be done very carefully to avoid the damage of bars.

After finishing concrete removal all surfaces must be clean with water jet with pressure of 200bar (Fig VIII-182).The application of water jet has to be with angle of 45^0 and with 5 cm distance.

Bared reinforcing bars should be cleaned with mixed water-sand blasting (250bar pressure) until ST2 grade is obtained.

Vertical rebars of new reinforcement should be fixed in foundation concrete by anchoring in drilled holes, at least 17cm depth, previously filled with appropriate adhesive. All rebars (existing and new) have to be protected with cementitious materials.

After preparing new reinforcement a wooden framework is assembled.

The concrete mixture, C30/37, with maximum aggregate grain size of 20mm and with appropriate workability should be used for reprofilation.

The plan of chosen repair structural and nonstructural measure are illustrated in next drawings (Fig VIII-179, Fig VIII-180)

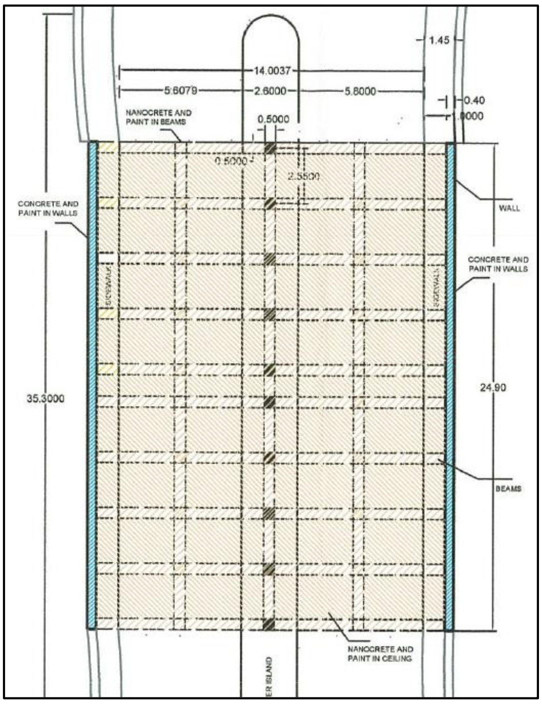


Figure (VIII-179) Layout of elements that have to be repaired with chosen repair materials

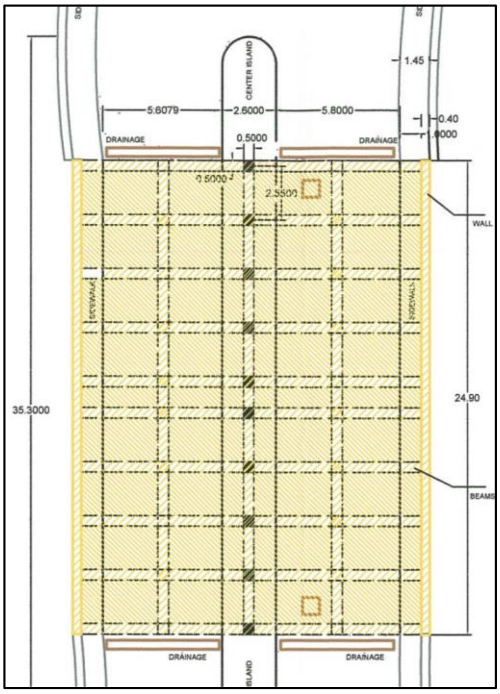


Figure (VIII-180) Layout of elements that have to be repaired (drainage system)

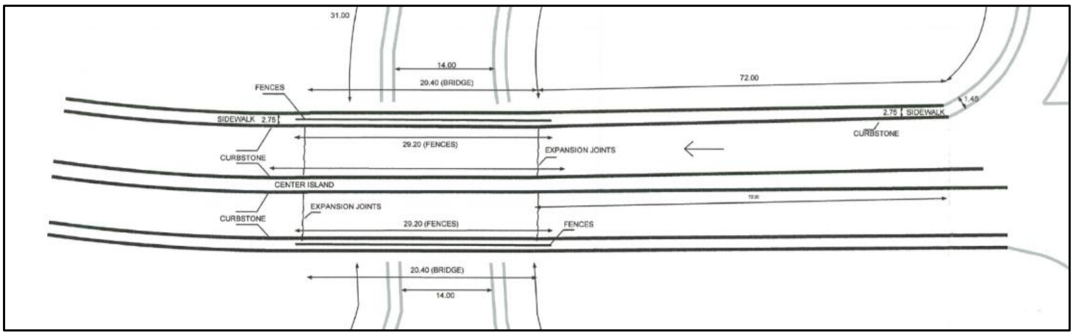


Figure (VIII-181) Layout of elements that have to be repair (expansion joints, curbs, and fence)

Through several photos, above mentioned repair measures are illustrated.

Figure (VIII-182) Column: Blasting of rebars with mix of sand and water

Figure (VIII-183) Column: Protection of rebars with cementitious materials

Figure (VIII-184) Prepared column with formwork ready to cast

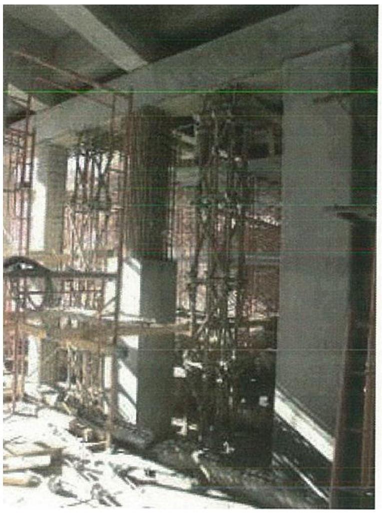


Figure (VIII-185) View of repaired column and column in 2nd phase of repair

Figure (VIII-186) The view of abutment after removal of all damaged concrete

Figure (VIII-187) Reinforcing rebars covered with cementitious protecting material

Figure (VIII-188) Repairing of abutment: steel mash for shrinkage control

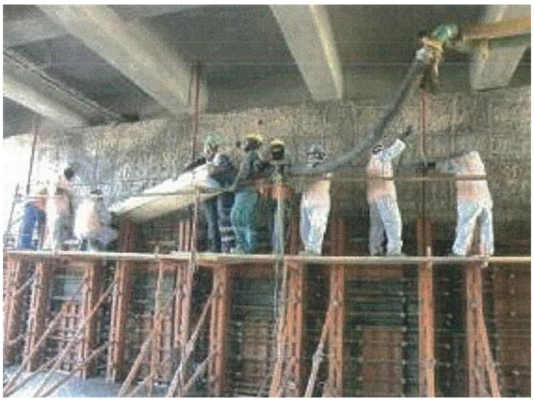


Figure (VIII-189) Repairing of abutment: placing of new concrete, 1st phase

After finishing of preparation of concrete surfaces on beams for fence, down surfaces of deck ceiling beams and top slab and application of bond agent, the chosen pre-mixed repair mortar is applied by spray technique. Then, on all repaired surfaces the curing agent is applied. After curing period, final protecting coating is applied in two layers.

Figure (VIII-190) The view of the concrete bridge elements after application of all recommended measures and materials

7.6. Materials recommended for repair of concrete bridge elements

After analyzing specifications of materials for structural and non-structural repair, principles and methods of repair and available materials in Libya, the next products has been selected:

Cementitious base material for rebar protection and for better adhesion of repair mortar:	EmacoNanocreteAP(BASF)
Cementitious base pre-mixed mortar:	EmacoNanocreteR4(BASF)
Currying protection material:	Masterkure 181(BASF)
Corrosion inhibition impregnation:	SikaFerroguard 903(SIKA)
Surface protective and paint layer:	Sikaguard 680 ES Betoncolor
Reinforcement rebar steel:	Grade A-615

The estimated quantities of listed products after visual inspection, after concrete removal and additional testing of materials and their real consumption are given in Table VIII-17.

Table VIII-17 - Calculated quantities of used products and their real consumption

Material	Estimated	Calculated after additional testing	Real quantities
EmacoNanocrete R4	19.125kg	62.150kg	58.800kg
EmacoNanocrete AP	555kg	2.760kg	1.250kg
Master flow 928/980	-	4.800kg	-
Mastercure 181	200Lt	600Lt	400Lt
SikaFerroguard 903	-	475 Lt	400Lt
Sikaguuard 680 ES Betoncolor	575Lt	475 Lt	400Lt

In table (VIII-18) the real consumption of used products per measuring unit is given.

Table VIII-18 - The real consumption of used products per measuring unit (ratio)

Quantity ratios					
Work	Material	Unit	Qty	Ratio	Average thickness
Concrete repair (with mortar)	EmacoNanocrete R4	1495.65m^2	58.800kg	39.31kg/m^2	21mm
	EmacoNanocreteAP	1495.65m^2	1.250kg	0.84kg/m^2	-
Concrete repair (with concrete)	Concrete C30/37 (columns)	10 columns	18m^3	1.80m^3/columns	-
	Concrete C30/37 (abutment)	244.44m^2	50m^3	-	201mm

7.7. Repair of expansion joints

During visual inspection it was noticed that expansion joints ware seriously damaged. In the text below a short description of measures recommended for their rehabilitation is given.

The processes of rehabilitation have to start with removal of old damaged asphalt layer by cutting with saw cutter and removing old asphalt by pick hammers. Then, the next operations should be done in next order:

- Cleaning and chipping of surface
- Fixing of shutters
- Casting of cementitious base grout for leveling down surface
- Fixing of steel shit over the joint
- Lining of waterproofing membrane to prevent leakage
- Cleaning and spreading of tack cout
- Placement and compaction of wearing asphalt course

Figures, given below, show phases of repair of expansion joints.

Figure (VIII-191) Expansion joint before repair

Figure (VIII-192) View of joint after removal of old asphalt and chipping of concrete substrate

Figure (VIII-193) Joint after grouting of cementitios based material

Figure (VIII-194) fixing of water proofing membrane

Figure (VIII-195) Placement and compaction of wearing asphalt course

Number and disposition of expansion joints that have to be repaired is given in Fig (VIII-196)

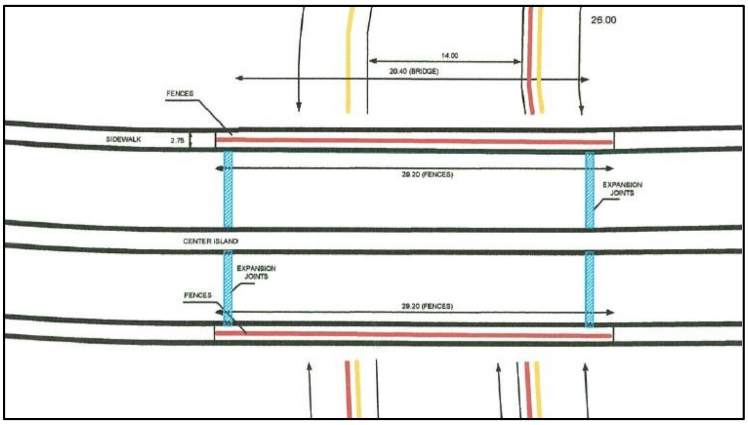


Figure (VIII-196) Disposition of the expansion joints

7.8. Other works

Other works include concreting of new curbs and assembling of new fence.

For execution of concrete curbs, the concrete C25/30 has been selected.

7.9. Important information

The assessment of the condition of this bridge has been done through 2 phases. In the first phase the VSL done only visual inspection of concrete structural elements. On the bases of results obtained by visual inspection they wrote the program of structural elements repair.

During the removal of old and damaged concrete cover, they decided to expand the assessment program by testing of quality built in material using semi destructive and nondestructive methods (second phase). The results of field testing were much worse than predicted as most concrete surface showed high level of carbonation and they decided to remove all carbonated concrete. In this phase, they also decided to repair abutments by adding new steel mash and new concrete layer.

CHAPTER IX

ROUTINE INSPECTION OF 7 BRIDGES IN TRIPOLI 6 YEARS AFTER REPAIR

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ROUTINE INSPECTION OF 7 BRIDGES IN TRIPOLI 6 YEARS AFTER REPAIR

1. SOUK ATHULATHA 1 BRIDGE

The works on repair of Souk Athulatha 1 Bridge started in March 2009 and ended in August 2009.

The first routine inspection was done in August 2016.

This routine inspection has included:

- Visual inspection of all available elements of RC structure of the bridge, as well as otrrr elements, such as fences, curbs pedestrian paths, guardrails and asphalt wearing layers and
- Checking of depth of concrete/mortar cover carbonization.

The results of routine inspection are given below.

1.1. Visual inspection of available RC elements

The visual inspection has encompassed the next RC elements:

- Lateral beam,
- Deck slab (arc cantilever slabs and simple beam slab)
- Cantilever slabs, and
- Supporting walls.

A general appearance of the bridge is shown in Fig IX-1 and Fig IX-2.

Figure (IX-1) A general view of Souk Athulatha 1

Figure (IX-2) A general view of Souk Athulatha 1

Lateral beam

The condition of lateral beam is illustrated in Fig IX-3 – Fig IX-5.

Figure (IX-3) A View of supporting part of lateral beam of arch slab, east side.

Figure (IX-4) A View of lateral beam of arch slab and cantilever slab, east side.

Figure (IX-5) A View of lateral beam of arch slab and cantilever slab, east side.

Figure (IX-6) A View of supporting part of lateral beam of arch slab, west side

By the visual inspection of lateral beams, the following damages are noticed:

- Traces and stains of water (Fig IX-5 and IX-6)
- Spalling off of repair mortar (Fig IX-5 and, IX-6) and
- Net like cracks (Fig IX-6)

The traces and stains of water are noticed in the middle part of lateral beam, as well as several local spalling off of repair mortar which were caused by hitting by bullets.

The characteristic damage of the down side of lateral beams is net like cracks caused by drying shrinkage of repair mortar.

In the middle part of span on down side of lateral beam several transversal and very straight local zones of spalling off of repair mortar were appeared (Fig IX-6).

The noticed local spalling off of repair mortars are shallow and no reinforcing bars have been visible, yet.

The net like cracks are the most serious damage due to the possibility of corrosion of reinforcement. This damage might cause the reduction of durability.

Deck slab (arch cantilever slabs and simple beam slab)

The condition of down side of deck slab is illustrated in Fig IX-7 – Fig IX-8.

	
Figure (IX-7) View of deck slab with abutment	Figure (IX-8) Net like cracks in repair mortar, down side of arch cantilever slab

By the visual inspection the following damages are noticed:

- Traces and stains of water (Fig IX-8)
- Spalling off of repair mortar (Fig IX-7 and IX-10) and
- Net like cracks (Fig IX-7-IX-8)

The traces and stains of water are noticed in the middle part of deck slab near the lateral beams.

The characteristic damage of the down side of deck slab is net like cracks caused by drying shrinkage of repair mortar. Described damage is noticed on the whole down side of deck slab (Fig IX-7-IX-10).

Spalling off of repair mortar (cover) was appeared in the middle of the bridge deck span – on simple beam slab. This damage is shallow, the reinforcement is not visible and is caused by bad bonding between concrete and repair mortar (Fig IX-8).

The deeper spalling off of cover was noticed near the lateral beam (Fig IX-10).

On the place of connection between cantilever arch slabs and simple beam slab the “channel” was formed by cutting of repair mortar due to release of surface stresses and because of different moving of these elements, also (Fig IX-8).

	
Figure (IX-9) Net like cracks, local spalling off of repair mortar (cover) and shallow cut “channels” on down side of bridge deck slab.	Figure (IX-10) Local deep spalling off of repair mortar

The noticed local spalling off of repair mortars are shallow and no reinforcing bars have been visible, yet. The deeper spalling off is very local and could be easily repaired.

The net like crucks are the most serious damage due to the possibility of corrosion of reinforcement. This damage might cause the reduction of durability.

Cantilever slab

The view of Cantilever slab is illustrated in Fig IX-4– Fig IX-6 and IX-11.

Figure (IX-11) The characteristic damages on side surface of cantilever slab

By the visual inspection the following damages are noticed on side surface of Cantilever slabs:

- Traces and stains of water (Fig IX-4 –IX-6, and IX-11)
- Peeling of protecting painting (Fig IX-4 – IX-6, and IX-11) and
- Horizontal and vertical cracks (Fig IX-11)

Traces and stains of water are characteristic for whole side surface of cantilever slabs due to bad drainage.

The horizontal crack was appeared between two layers of concrete and it stretches along the whole length of slab.

Vertical cracks are very thin and they were caused by drying shrinkage of concrete.

Only traces and stain of water were spotted on down surface of cantilever slabs.

On the basis of those descriptions, it can be concluded that registered damages are in initial phase, but to avoid appearance of new damages, the drainage of atmospheric water should be improved.

Supporting walls

The condition of supporting wall is illustrated in Fig IX-12 – Fig IX-14.

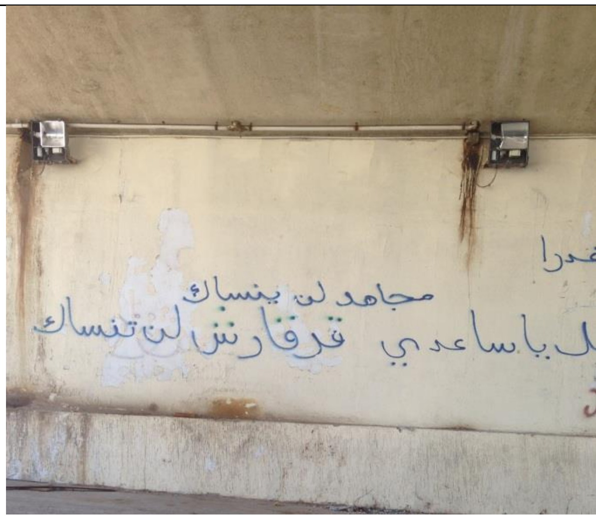


Figure (IX-12) View of supporting wall	Figure (IX-13) View of supporting wall

Figure (IX-14) Cracks in supporting wall

By the visual inspection the following defects and damages are noticed:

- Traces and stains of steel corrosion (Fig IX-12 and IX-13)
- Peeling off of protecting painting (Fig IX-12-IX-14) and
- Vertical and horizontal cracks (Fig IX-14)

Traces and stains of steel corrosion have been noticed on the places where metal parts of some installation were fixed for wall.

Peeling off of surface protecting paint is characteristic damage and it has been registered on approximately 50% of wall surface.

Vertical and horizontal cracks appeared on places where previous openings in supporting wall were closed (by reinforced concrete).

On the basis of those descriptions, it can be concluded that supporting walls are in good condition. Registered peeling off of protecting painting could be easily repaired, by changing it.

1.2. Visual inspection of other bridge elements

Pedestrian path: By the visual inspection it was noticed that cold joints are very rough and cracked. Also, longitudinal cracks and surface peeling off of thin concrete layer have been appeared in parts between cold joints as well as the appearance of some plants on the cold joints.

The condition of Pedestrian path is illustrated in Fig IX-15.

It could be concluded that initial damages have occurred on pedestrian path surface and some prevention measures for slowing down appearance of new and progress of numbered damages should be undertaken.

Figure (IX-15) A general view of Pedestrian path on bridge

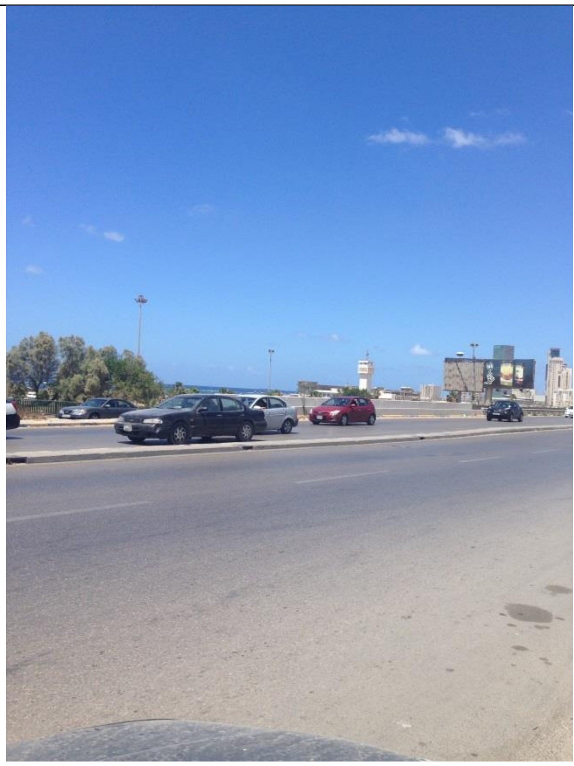


Figure (IX-16) A general view of Asphalt wearing layer

1.3. Asphalt wearing layer:

Lots of dust and sand on the bridge made the visual inspection of asphalt wearing layer very difficult. The condition of the asphalt layer is shown in Figures IX-15 and 9.16. Some minor erosion of wearing layer was noticed.

Fences

The fences are in good condition. The corrosion of deformation of elements of fences was not registered. It was spotted that one part of fences was missing (Fig IX-17). This vandal type of damages could be very dangerous for people who cross the bridge.

Curbs

Some minor cracks, corner spalling, as well as peeling off black and yellow colour were spotted. The condition of curbs is illustrated in Fig IX-18. It could be concluded that initial damages have occurred on curb surfaces and some prevention measures for slowing down appearance of new and progress of numbered damages should be undertaken.

Figure (IX-17) A general view of fences on the bridge, with missing part

Figure (IX-18) A general view of curbs on the bridge

Guardrail

It is in good condition and has no significant damage, except one missing part. The reflective signs in guardrail are good condition.

The condition of guardrail and reflective sign is illustrated in Fig IX-19 –IX-20 –IX-21.

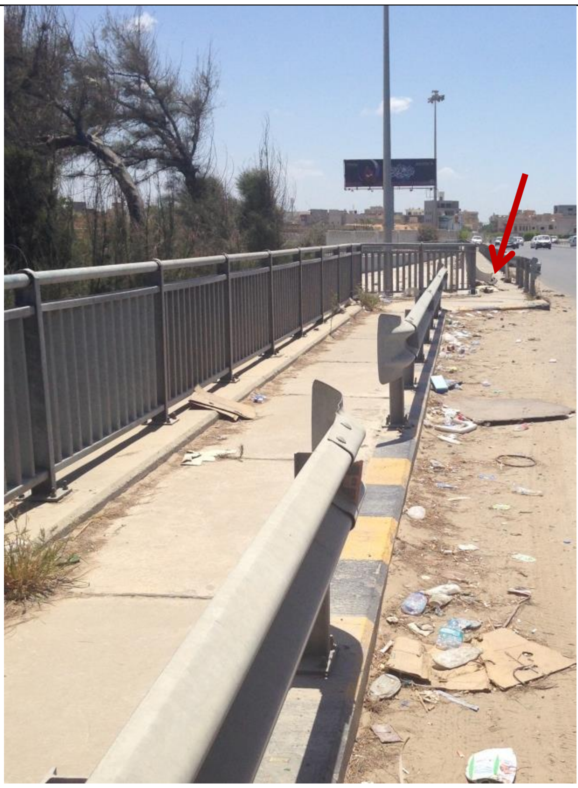


Figure (IX-19) General view of the guardrail on the bridge with the missing of part of it	Figure (IX-20) A general view of guardrail on the bridge

Figure (IX-21) A general view of reflective sign in guardrail

Catch pit

Blockage of drainage channels under the bridge with garbage, dust and sand, which causes the rainwater to not be drainage.

The condition of catch pit is illustrated in Fig IX-22.

Figure (IX-22) A general view of catch pit under the bridge

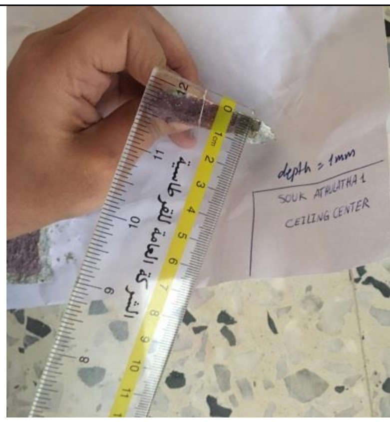
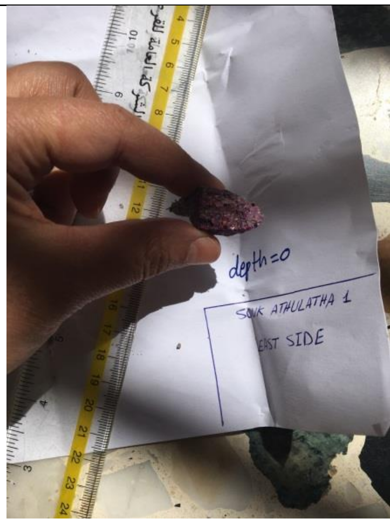
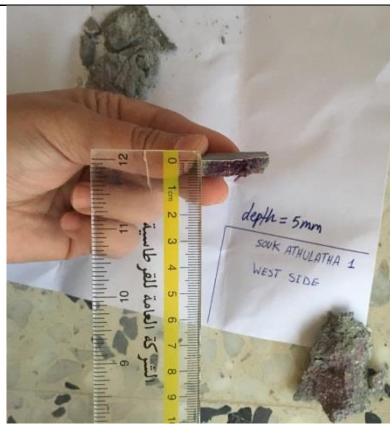
Depth of carbonation:

The extent of carbonation was assessed by treating the fresh broken part of repair material with phenolphthalein indicator. The carbonation test were done on ceiling and supporting wall (west and east side)

The obtained results are shown in the table IX-1.

By the analysing obtained results it is concluded that process of carbonation has already started in tested RC elements. The largest value of carbonation was measured on supporting wall (west side) (5mm). This value is almost double than expected. The rate of carbonation is usually 0,5mm/year and with that speed the depth of carbonation should have been about 3,5mm. Some measures to slow down the rate of carbonation should be considered.

Table (IX-1) Results of measuring the depth of carbonation on RC elements

Elements	description	Depth of carbonation (mm)	
Ceiling centre	Repair material – mortar	1	
Supporting wall (east side)	Repair material – mortar	0	
Supporting wall (west side)	Repair material – mortar	5	

1.4. General conclusion

The first routine inspection of The Souk Athulatha 1 bridge was done after 6 years of its repair. By the analysing of collected results, which were obtained by visual inspection and measuring depth of carbonation, the next conclusions are derived:

- The characteristic damage of the down side of lateral beams and deck slab are net like cracks caused by drying shrinkage of repair mortar. They cover the whole down side of deck slab. These crucks are the most serious damage due to the possibility of corrosion of reinforcement. This damage might cause the reduction of durability.
- In the middle part of span, on down side of lateral beam and on simple beam slab, the local spalling off of repair mortars is appeared. Described damages are shallow and no reinforcing bars have been spotted. The deeper spalling off is very local.
- The horizontal and vertical cracks are typical damage of cantilever slabs. The horizontal crack was appeared between two layers of concrete and it stretches along the whole length of slab, while the vertical cracks are located on side surfaces. They are very thin and caused by drying shrinkage of concrete.
- The traces of water are noticed on lateral beams, in the middle of deck slab and on vertical sides of cantilever slabs.
- The typical damage of supporting walls is peeling off of surface protecting paint. It covers of approximately 50% of wall surface. Vertical and horizontal cracks appeared on places where previous openings in supporting wall were closed.
- The initial damages have occurred on pedestrian path (cracks and pilling off of concrete surface)
- The characteristic damage of wearing layer of traffic lanes is erosion. A lot of bare aggregate grains can be seen.
- Bridge fence and guardrails are in good conditions, but some parts of them are missing due to vandalism.
- Curbs are still in good conditions.
- Catch pit are not in function, because of blocking by dust, sand and garbage.
- Due to the lack of maintenance the growth of some plants in cold joints of pedestrian paths, as well as, the presence of dust and urban rubbish on the road were noticed. Also, traffic signs, including intermittent lines, that allow the vehicle to cross, and the side lines have been disappeared.
- Carbonation has already started on ceiling and supporting walls. The largest value of carbonation was measured on supporting wall (5mm).

Finally, the stability, bearing capacity, functionality and durability have not been jeopardized, yet. As it was mentioned, damages were spotted on the surface of inspected RC elements, especially on ceiling deck and supporting walls. All damages located in cover, could be slowed down by some measures like impregnation. The same measures are suggested for RC elements caught by carbonation. The protective paint on supporting walls should be repainted.

The review of registered damages during routine visual inspection is presented in table IX-2.

Table IX-2 Review of registered damages during the first routine inspection

RC element	Cracks	Pilling off protecting paint	Spalling off	Water leakage/traces	Carbonation
Lateral beam	+ Mesh like cracks	-	+ local	+ (traces)	Not measured
Deck ceiling Arch cantilever slabs	+ Mesh like cracks	-	+ local	+ (traces)	+
Deck ceiling Simple beam slab	+ Mesh like cracks	-	+ local	+ (traces)	+
cantilever slabs	+ horizontal and vertical	+	-	+ (traces)	Not measured
Supporting walls (west side)	+ Vertical cracks	+	-	+ (traces)	+ (in progress)

2. SOUK ATHULATHA 2 BRIDGE

The works on repair of Souk Athulatha 2 Bridge started in March 2009 and ended in October 2009.

The first routine inspection was done in August 2016.

This routine inspection has included:

- Visual inspection of all available elements of RC structure of bridge, as well as others elements, such as fences, curb stones, pedestrian paths, guardrails and asphalt wearing layers and
- Checking of depth of concrete/mortar cover carbonization.

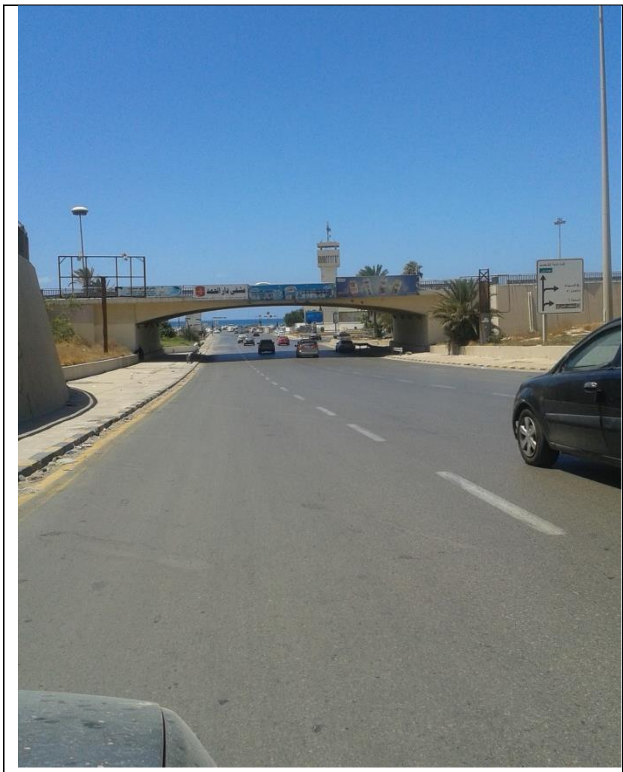
The results of routine inspection are given below.

2.1. Visual inspection of available RC elements

The visual inspection has encompassed the next RC elements:

- Lateral beam,
- Deck slab (arc cantilever slabs and simple beam slab)
- Cantilever slabs, and
- Supporting walls.

A general appearance of the bridge is shown in Fig IX-23 and Fig IX-24.



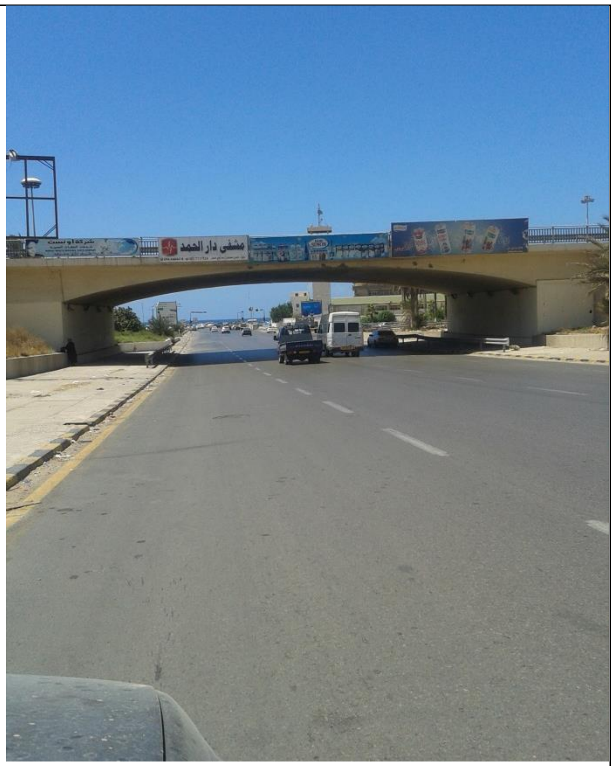


Figure (IX-23): A general view of Souk Athulatha 2

Figure (IX-23): A general view of Souk Athulatha 2

Abutment walls could not been inspected because both side passes had been closed by building new lateral walls.

Lateral beam

The condition of lateral beam is illustrated in Fig IX-24 – Fig IX-26.

Figure (IX- 24): A view of supporting part of lateral beam of arch slab, east side

Figure (IX-25): A view of supporting part of lateral beam of arch slab, west side

Figure (IX-26): A View of lateral beam of arch slab, and cantilever slab, middle part

By the visual inspection of lateral beams, the following damages are noticed:

- Traces and stains of water (Fig IX-26)
- Spalling off of repair mortar (Fig IX-25 and, IX-26) and
- Net like cracks (Fig IX-25)

The traces and stains of water are noticed on outdoor side surface in the middle part of lateral beam, as well as several local spalling off of repair mortar which were caused by hitting by bullets.

The characteristic damage of the down side of lateral beams is net like cracks caused by drying shrinkage of repair mortar.

In the middle part of span, on down edge of lateral beam two local zones of spalling off of repair mortar were registered (Fig IX-26).

These local spalling off of repair mortars was caused by hitting with bullet. They are deep up to the RC bars.

The net like cracks are the most serious damage due to the possibility of corrosion of reinforcement. This damage might cause the reduction of durability.

Deck slab (arch cantilever slabs and simple beam slab)
The condition of down side of deck slab is illustrated in Fig IX-27 – Fig IX-29.

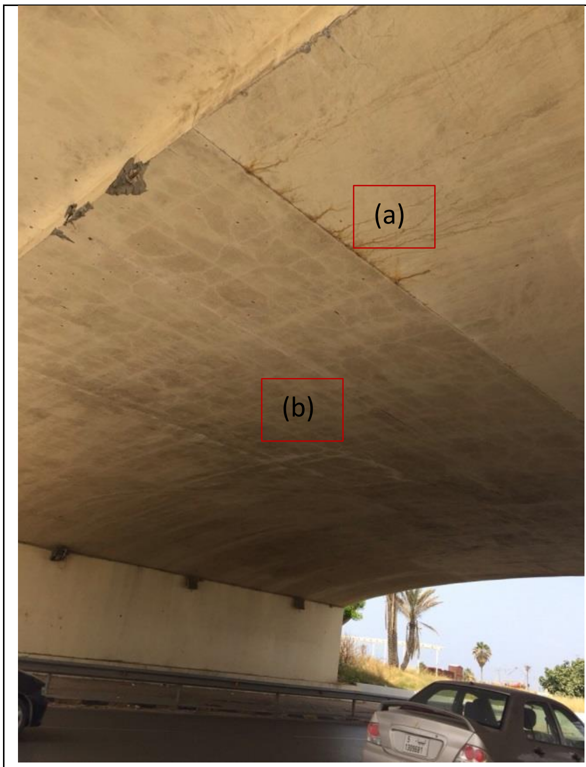


Figure (IX-27): View of deck slab with abutment, (a) rust stains, (b) white stains

Figure (IX-28): Net like cracks in repair mortar, down side of arch cantilever slab

By the visual inspection the following damages are noticed:

- Traces and stains of water (Fig IX-27 and 9.IX-28)
- Spalling off of repair mortar (Fig IX-29) and
- Net like cracks (Fig IX-27-IX-28)

Two types of traces were spotted, both due to leakage of water through RC super structure. The first type is characteristic for support place of simple beam slab on arch cantilever slab. On that place the traces of rust were noticed (Fig IX-27 (a)). The second type was noticed near the middle of span, and it has white stains caused by dissolving Ca $(OH)_2$ (Fig IX-27 (b)).

The characteristic damage of the down side of deck slab is net like cracks caused by drying shrinkage of repair mortar. Described damage is noticed on the whole down side of deck slab (Fig IX-27-IX-29).

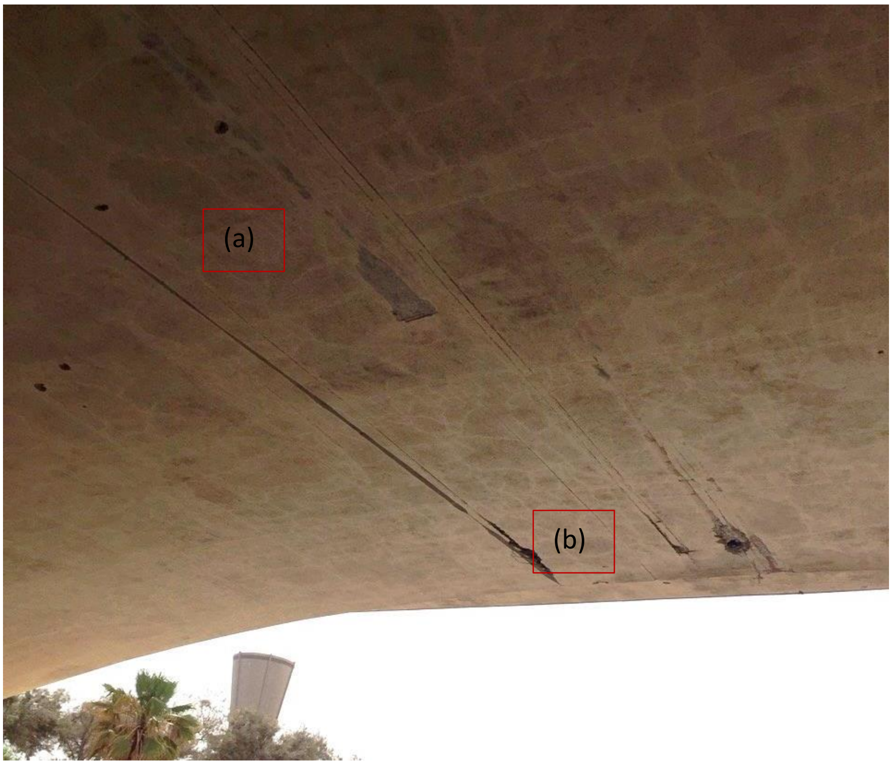


Figure (IX-29): Local shallow (a) and deep (b) spalling off of repair mortar, bridge deck ceiling slab

Spalling off of repair mortar (cover) was appeared in the middle of the bridge deck span – in the simple beam slab. This damage is shallow, the reinforcement is not visible and is caused by bad bonding between concrete and repair mortar (Fig IX-29 (a)).

The deeper spalling off of cover was noticed near the lateral beam (Fig IX-29(b)).

The noticed local spalling off of repair mortars are shallow and no reinforcing bars have been visible, yet. The deeper spalling off is very local and could be easily repaired.

The net like cracks and rust stains are the most serious damages due to the possibility of corrosion of reinforcement. This damage might cause the reduction of durability.

Cantilever slab

The view of Cantilever slab is illustrated in Fig IX-26 and IX-30.

By the visual inspection the following damages are noticed on side surface of cantilever slabs:

- Traces and stains of water
- Local spalling off concrete and
- Net like, horizontal and vertical cracks (fig IX-30)

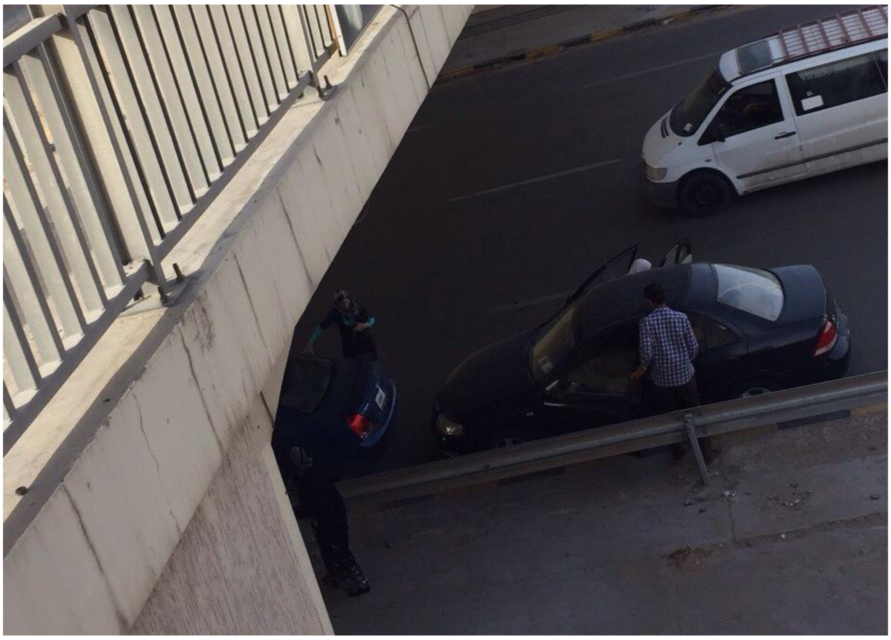


Figure (IX-30): View of side surface of cantilever slab, vertical cracks and stains.

Traces and stains of water are characteristic for whole side surface of cantilever slabs due to bad drainage.

The horizontal crack was appeared between two layers of concrete and it stretches along the whole length of slab.

Vertical cracks are very thin and they were caused by drying shrinkage of concrete and/or by corrosion of reinforcement.

Net like cracks and traces and stain of water were spotted on down surface of cantilever slabs.

In the middle of bridge span several local mechanical damages were located. They were caused by hitting with bullets

On the basis of those descriptions, it can be concluded that registered damages are in initial phase, but to avoid appearance of new damages, the drainage of atmospheric water should be improved. Local mechanical damages are shallow and could be repaired easily.

Supporting walls

The condition of supporting walls is illustrated in Fig IX-31 – Fig IX-34.

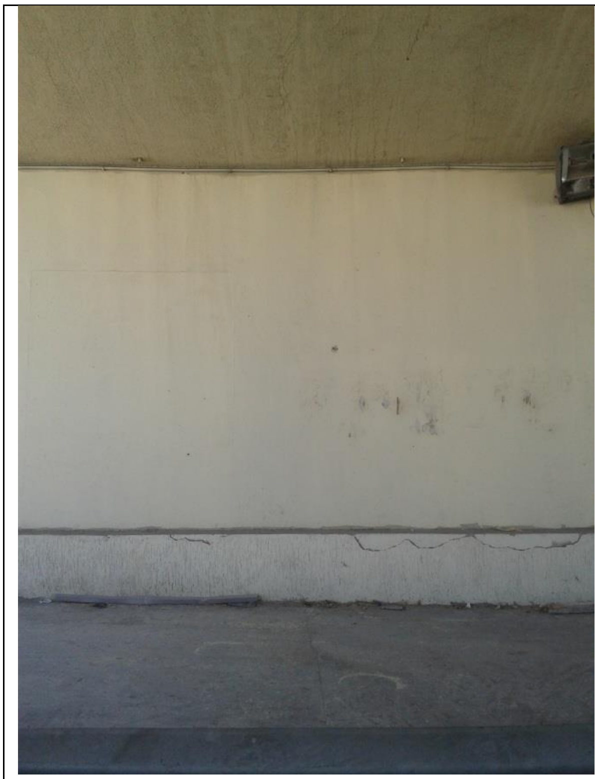


Figure (IX-31): View of supporting wall, damages of plinth

Figure (IX-32): Supporting Wall, the detail of damage of plinth

Figure (IX-33): View of supporting wall

Figure (IX-34): Look of deck celling and abutments

By the visual inspection the following damages are noticed:

- Traces and stains of steel corrosion
- Peeling off of protecting painting (Fig IX-31) and
- Separation and spalling off of protecting mortar layer (Fig IX-35)

Traces and stains of steel corrosion have been noticed on the places where metal parts of some installation were fixed for wall.

Peeling off of surface protecting paint is characteristic damage and it has been registered on approximately 40% of wall surface.

Separation and spalling off of protecting mortar layer is characteristic for plinth. This damage is caused by bad adhesion between concrete substrate and mortar protecting layer.

On the basis of those descriptions, it can be concluded that supporting walls are in good condition. Registered peeling off of protecting painting and spalling off mortar layer could be easily repaired, by changing it.

2.2. Visual inspection of other bridge elements

Pedestrian Path

Through visual inspection, several types of damages were spotted:

- Uneven cold joints
- Local surface pits and
- Biological corrosion

Figures IX-398 and IX-399 illustrate the condition of the pedestrian path.

It was observed that the cold joints are very uneven, sometimes with surplus of infilled material, some times without it and sometimes with plants which grow from joints (Fig IX-35 and IX-36).

On concrete surfaces between cold joints the shallow pits were spotted. It is supposed that they were caused by mechanical action. (Fig IX-36)

It could be concluded that initial damages have occurred on pedestrian path surface and in cold joints. Some prevention measures for slowing down appearance of new and progress of numbered damages should be undertaken.

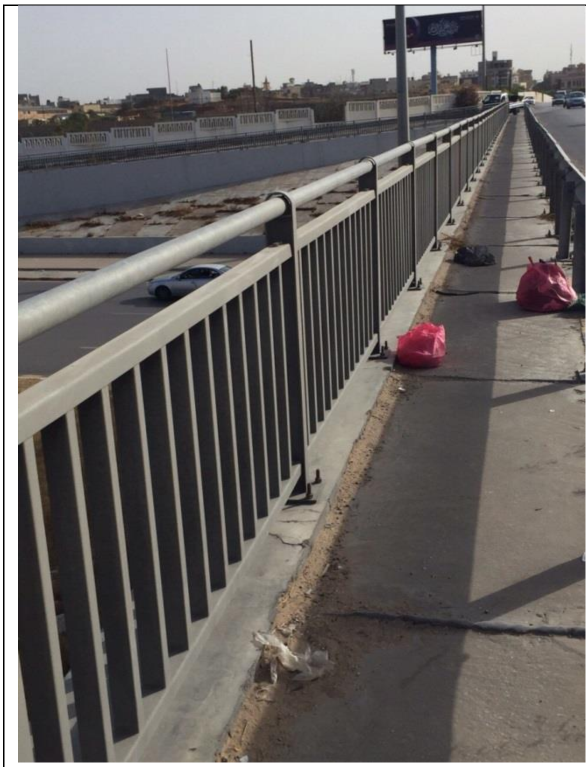


Figure (IX-35): View of pedestrian path

Figure (IX-36): View of pedestrian path

Asphalt wearing layer

Lots of dust and sand on the bridge made the visual inspection of asphalt wearing layer very difficult. The condition of the asphalt layer is shown in Figures IX-37 and IX-38.

During visual inspection two types of cracks were noticed in wearing layer of traffic lanes. They are transversal and longitudinal cracks. The cracks are dashed, with wideness of several mm. Some minor erosion of wearing layer was also noticed.

Fences

The fences are in good condition. The state of the fences is shown in Fig IX-35 and IX-39.

Curbs

The corner spalling off of concrete and peeling off black and yellow colour were spotted as typical damages. The corner spalling off is caused by hitting by car.

Figure IX-40 illustrates the state of curbs.

Figure (IX-37): A general view of asphalt wearing layer, transversal crack

Figure (IX-38): Detail of asphalt wearing layer with longitudinal crack

Figure (IX-39): A general view of fences on the bridge

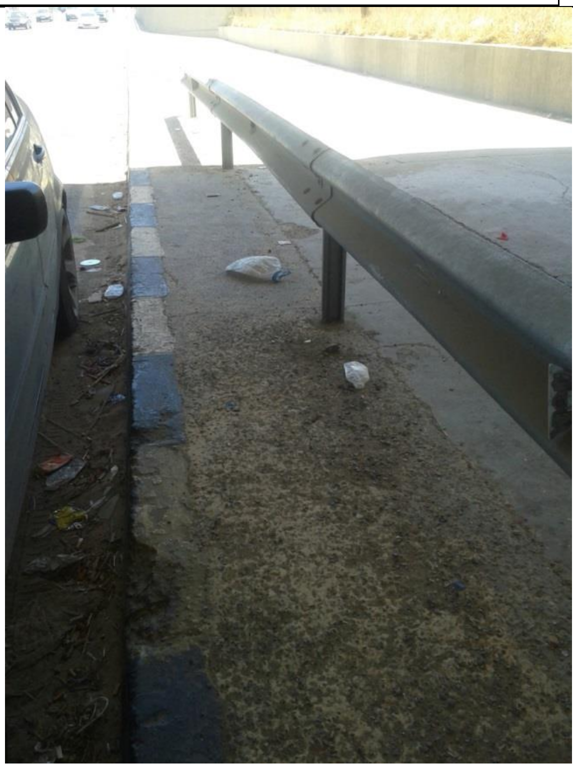


Figure (IX-40): A general view of curbs on the bridge

Guardrail

Generally, the guardrails are in good condition as they have not been caught by corrosion, but a few mechanical damages were spotted. On these places, the guardrail is missing or is deformed and broken due to car accident (Fig IX-37 and IX-41). The reflective sign in guardrail is good condition.

The condition of guardrail and reflective sign is illustrated in Fig IX-41-IX-42.

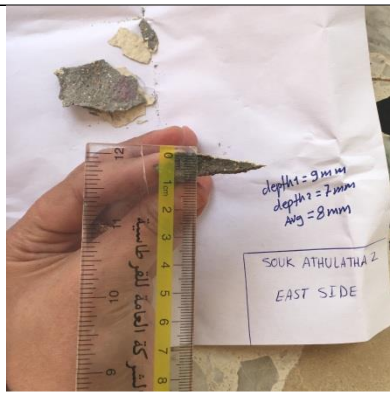
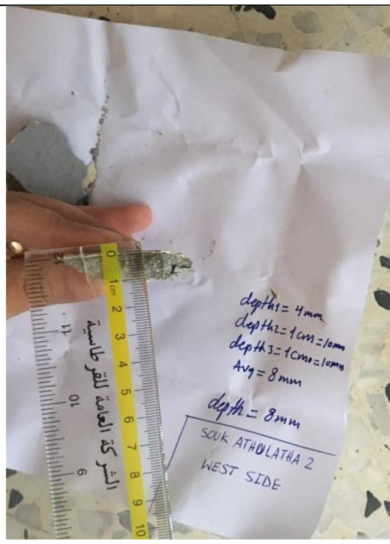
Figure (IX-41): General view of the guardrail on the bridge with damage	Figure (IX-42): A general view of guardrail under the bridge

Depth of carbonation

The extent of carbonation on was assessed by treating the fresh broken part of repair material with phenolphthalein indicator. The carbonation test were done on following RC elements: ceiling and Exterior wall (west and east side)

The obtained results are shown in the table IX-3.

Table (IX-3) Results of measuring the depth of carbonation on RC elements

Elements	description	Depth of carbonation (mm)	
Ceiling center	Repair material – mortar	12.5	
Supporting wall (east side)	Repair material – mortar	8	
Supporting wall (west side)	Repair material – mortar	8	

By the analysing obtained results it is concluded that process of carbonation has already started on all RC elements. The largest value of carbonation was measured on ceiling (12.5mm). This value is significantly larger than expected. The rate of carbonation is usually 0,5mm/year and with that speed the depth of carbonation should have been about 3,5mm. Some measures to slow down the rate of carbonation should be considered.

2.3. General conclusion

The first routine inspection of The Souk Athulatha 2 bridge was done after 6 years of its repair. By the analysing of collected results, which were obtained by visual inspection and measuring depth of carbonation, the next conclusions are derived:

- The characteristic damage of the down side of lateral beams and deck slab are net like cracks caused by drying shrinkage of repair mortar. They cover the whole down side of deck slab. Besides, water leakage was appeared through RC super structure on the places where simple beam slab is supported by arch cantilever slab (following with rust traces) and near the middle of span (white traces due to dissolving $Ca(OH)_2$). Described damages are very serious due to of the possibility of reinforcement corrosion. This damage might cause the reduction of mechanical resistance and durability.
- In the middle part of span, on down side of lateral beam and on simple beam slab, the local spalling off of repair mortars is appeared. Described damages are shallow and no reinforcing bars have been spotted. The deeper spalling off is very local.
- The horizontal and vertical cracks are typical damage of cantilever slabs. The horizontal crack was appeared between two layers of concrete and it stretches along the whole length of slab. The vertical cracks are located on side surface and they could be caused by dry shrinkage of concrete and/or by corrosion of reinforcement.
- The traces of water are noticed on lateral beams, in the middle of deck slab and on vertical sides of cantilever slabs.
- The typical damage of supporting walls is peeling off of surface protecting paint. It covers of approximately 40% of wall surface. Separation and spalling off of protecting mortar layer is characteristic for plinth, but this damage has local character.
- The initial damages have occurred on pedestrian path, such as local surface pits, biological corrosion and uneven cold joints.
- The characteristic damages of wearing layer of traffic lanes are longitudinal and transversal cracks.
- Bridge fence and guardrails are in good conditions, but some parts of guardrails are deformed and broken due to car accident.
- Curbs are locally damaged by hitting by car, but they are still in good conditions.
- Due to the lack of maintenance the growth of some plants in cold joints of pedestrian paths, as well as, the presence of dust and urban rubbish on the road were noticed. Also, traffic signs, including intermittent lines, that allow the vehicle to cross, and the side lines have been partly disappeared.

– Carbonation has already started on ceiling and supporting walls. The largest value of carbonation was measured on ceiling (12.5mm).

Finally, the stability, bearing capacity, functionality and durability have not been jeopardized, yet. As it was mentioned, damages were spotted on the surface of inspected RC elements, especially on ceiling deck. All damages located in cover (such as mesh like cracks), could be slowed down by some measures like impregnation. The same measures are suggested for RC elements caught by carbonation, but water leakage should be prevented by other methods, due to the risk of progression of reinforcement corrosion. The protective paint on supporting walls should be repainted Local separation and spalling off of repair mortar can be easily re-repaired.

The review of registered damages during routine visual inspection is presented in table IX-4.

Table IX-4 Review of registered damages during the first routine inspection

RC element	Cracks	Pilling off protecting paint	Spalling off	Water leakage/traces	Carbonation
Lateral beam	+ Mesh like cracks	-	+ local	+ (traces)	Not measured
Deck ceiling Arch cantilever slabs	+ Mesh like cracks	-	+ local	+ Leakage, rust & white traces	Not measured
Deck ceiling Simple beam slab	+ Mesh like cracks	-	+ local	+ Leakage, rust & white traces	+ (in progress)
cantilever slabs	+ horizontal and vertical	-	-	+ (traces)	Not measured
Supporting walls	-	+	+ plinth	+ (traces)	+ (in progress)

3. ALSSEKA ROAD BRIDGE

The works on repair Alsseka Road Bridge started in June 2009 and ended in November 2009.

The first routine inspection was done in August 2016.

This routine inspection has included:

- Visual inspection of all available elements of RC structure of bridge, as well as others elements, such as sidewalks, curb stones, catch pits, fences, expansion joints, guardrails and asphalt wearing layers and
- Checking of depth of concrete/mortar cover carbonization.

The results of routine inspection are given below.

3.1. Visual inspection of available RC elements

The visual inspection has encompassed the next RC elements:

- Lateral beam,
- Deck slab (arc cantilever slabs and simple beam slab),
- Cantilever slabs,
- Supporting walls and
- Abutments.

A general appearance of the bridge is shown in Fig IX-43 and Fig IX-44.

Figure (IX-43) A general view of Alsseka Road Bridge	Figure(IX-44) A general view of Alsseka Road Bridge

Lateral beam

The condition of lateral beams is illustrated in Fig IX-45 – Fig IX-47.

Figure (IX-45) A View of Lateral beams

	
Figure(IX-46) Mechanical damage of lateral beam	Figure(IX-47) Crack along the edge of lateral beam, water traces

By the visual inspection of lateral beams, the following damages are noticed:

- Traces and stains of water (Fig IX-47)
- Mechanical damage (Fig IX-46) and
- Longitudinal cracks (Fig IX-47)

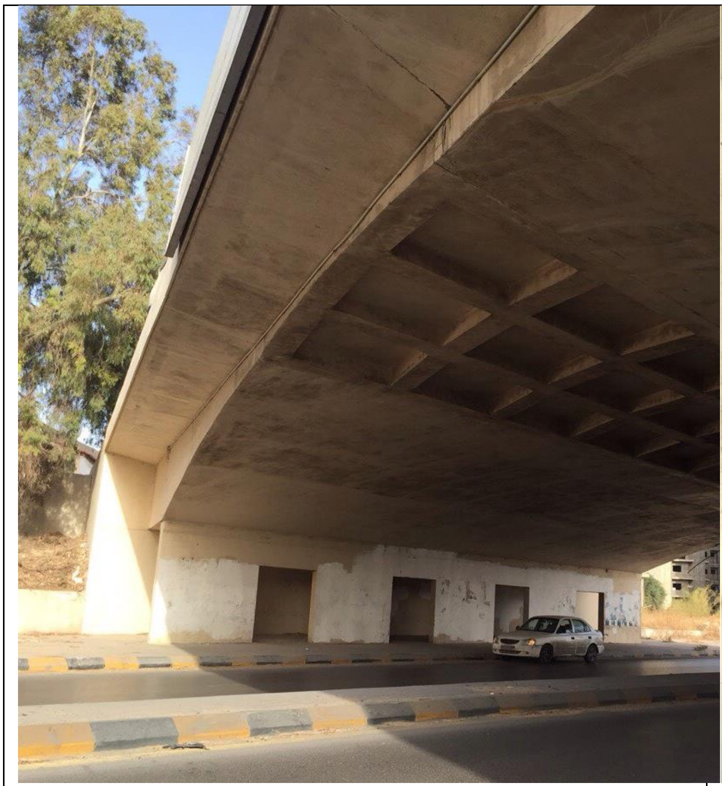
The traces and stains of water are noticed on down side lateral beam. They are caused by leakage of water through joint between arc cantilever slabs and simple beam slab.

The serious mechanical damage is registered on the down side of lateral beam (Fig IX-46). On that place concrete and repair mortar were broken and fallen down, two main reinforcing bars were pulled out and deformed and stirrups were broken and deformed. Described damage is the most serious as it reduce mechanical resistance of lateral beam.

The longitudinal crack was spotted along the external edge of lateral beam (Fig IX-47). This damage may be caused by corrosion of main reinforcing bar.

Deck slab (arc cantilever slabs and simple beam slab)

The condition of down side of deck slab is illustrated in Fig IX-48 – Fig IX-50.



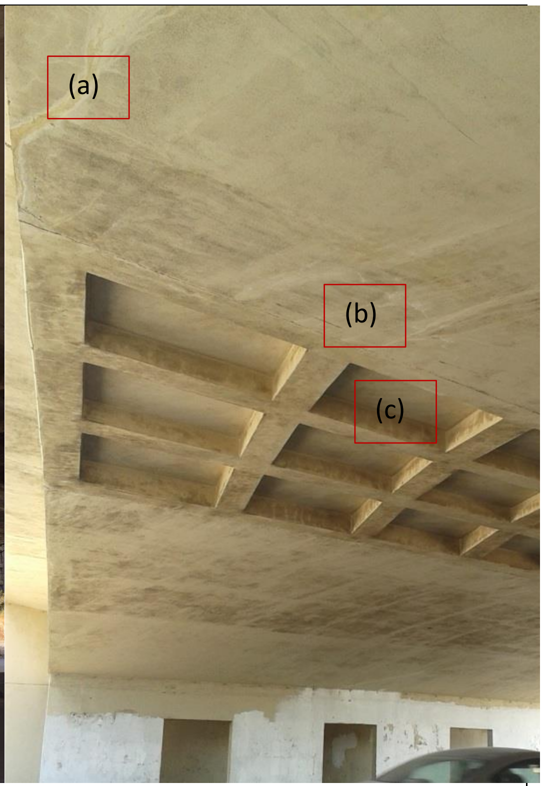


Figure (IX-48) View of deck slab, down side of arch cantilever slab with support walls	Figure (IX-49) View of deck slab, (a) water traces; (b) white stains due to water leakage through joint; (c) vertical cracks on ribs

By the visual inspection the following damages are noticed:

- Traces and stains of water (Fig IX-48 and Fig IX-49 (a) and (b))
- Vertical cracks (Fig IX-49 (c)
- Local spalling off mortar cover.

The characteristic traces are white coloured and they are caused by rain water leakage through the support place of simple beam slab on arch cantilever slab (Fig IX-49 (b)).

The vertical cracks are characteristics for side surfaces of ribs of simple beam slab (Fig IX-49 (c)). The reason for their appearance is drying shrinkage or insufficient thickness of mortar cover.

The very small places of local spalling off of mortar cover have been found during visual inspection. They are randomly arranged on surface of cantilever slabs and in support places of simple beam slab on arch cantilever slab.

The described damages have not jeopardised stability and durability of deck ceiling yet.

Cantilever slab

The view of Cantilever slab is illustrated in Fig IX-50.

By the visual inspection the following damages are noticed on cantilever slabs:

- Traces and stains of water (Fig IX-45, IX-48 and IX-50)
- Local spalling off concrete and (Fig IX-45 and IX-50)
- Net like, horizontal and vertical cracks (Fig IX-45 and IX-50).

Figure (IX-50) View of cantilever slab, vertical and horizontal cracks

Traces and stains of water are characteristic for whole side and down surface of cantilever slabs. They occurred due to bad drainage of rain water (Fig IX-45, IX-48 and IX-50). White traces of leakage are noticed in joint between two parts of superstructures of the bridge.

The horizontal crack was appeared between two layers of concrete and it stretches along the whole length of slab (Fig IX-45 and IX-50).

Vertical cracks are characteristic damage on side surface of cantilever slab. They were caused by drying shrinkage of concrete and/or by corrosion of reinforcement (Fig IX-45and IX-50).

Net like cracks are also characteristic for side surface of analysed element (Fig IX-45 and IX-50).

Spalling off of concrete is very local, shallow and it was spotted on side surfaces (Fig IX-45 and IX-50).

On the basis of those descriptions, it can be concluded that described cracks on side surface of cantilever slabs, particularly vertical cracks, could cause reinforcement corrosion. Because of that, execution of new concrete/mortar cover is recommended. Also, to avoid appearance of new damages, the drainage of atmospheric water should be improved.

Supporting walls

The condition of supporting walls is illustrated in Fig IX-51 – Fig IX-54.

Figure (IX-51) View of supporting wall with ceiling, peeling off protective painting

Figure (IX-52) View of supporting wall, net like cracks

Figure (IX-53) Net like cracks in supporting walls, the part above the openings	Figure (IX-54) A View of supporting wall, damages of plinth, mechanical damage

By the visual inspection the following damages are noticed:

- Peeling off of protecting painting (Fig IX-51) and
- Separation and spalling off of protecting mortar layer (Fig IX-54)
- Net like cracks in supporting walls (Fig IX-52 – IX-53)

Peeling off of surface protecting paint is characteristic down part of supporting walls Fig IX-51).

Separation and spalling off of protecting mortar layer is characteristic for plinth. There are two causes for this damage appearance. The first reason is bad adhesion between concrete substrate and mortar protecting layer and the second one it is mechanical impact (Fig IX-54).

Net like cracks are characteristic damage of inspected walls. They were occurred in repair mortar cover due to their drying shrinkage (Fig IX-52 and IX-53)

Abutments

The condition of abutments is illustrated in Fig IX-55 – Fig IX-56.

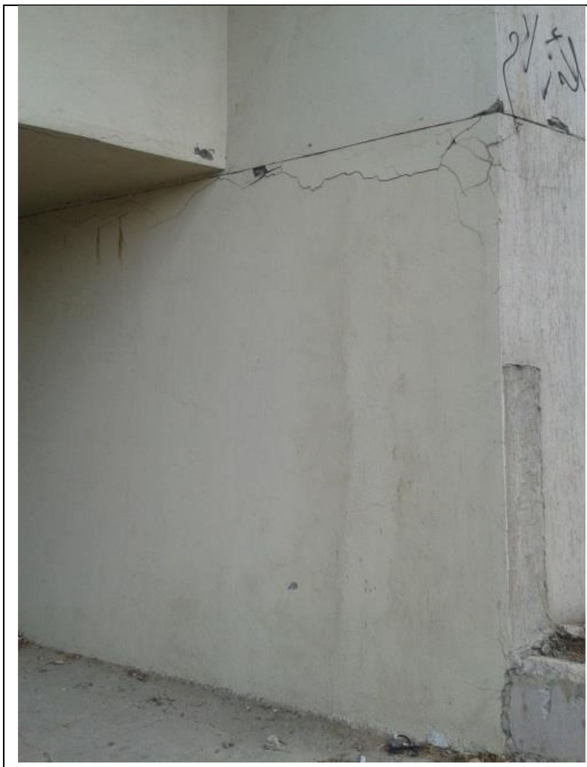


Figure (IX-55) View of external part of abutment	Figure (IX-56 View of abutment from in the tunnel

By the visual inspection the following defects and damages are noticed:

- Traces and stains of rust (Fig IX-55).
- Net like cracks (Fig IX-50 and IX-56) and
- Separations of mortar cover (Fig IX-55).

The characteristic damage of abutment walls is net like cracks. They are registered on almost all visible surface of abutment. The cracks are very thin, except on upper part of external part of abutment wall, where they are much stressed and significantly wider. These cracks are followed by separation of mortar cover and they are about to fall down (Fig IX-55).

Traces and stains of rust have been noticed on only two places near external part of abutment wall.

On the basis of those descriptions, it can be concluded that all inner parts of abutments are in good condition. They have initial damages, the net like cracks, but the cracks are still very thin, and process of deterioration has not start yet.

The most damaged part of abutments is external part in which serious cracks have been appeared, followed by separation of mortar cover. This damage could cause the corrosion of reinforced bars.

3.2. Visual inspection of other bridge elements

Pedestrian Path

Through visual inspection, several types of damages were spotted:

- Uneven cold joints
- Local surface pits and
- Biological corrosion,
- Longitudinal cracks.
- Missing part of concrete.

Figures IX-57 and IX-58 illustrate the condition of the pedestrian path.

It was observed that the cold joints are very uneven, sometimes with surplus of in filled material, some times without it and sometimes with plants which grow from joints (Fig IX-57).

On concrete surfaces between cold joints the shallow pits and erosion caused wearing were spotted. It is supposed that they were caused by mechanical action. (Fig IX-57)

The longitudinal crack was spotted in the place of connection of RC beam for fence and concrete layer of pedestrian path.

The large missing part of concrete layer is spotted in the pedestrian path under the bridge (Fig IX-58)

It could be concluded that initial damages have occurred on pedestrian path surface and in cold joints. Some prevention measures for slowing down appearance of new and progress of numbered damages should be undertaken.

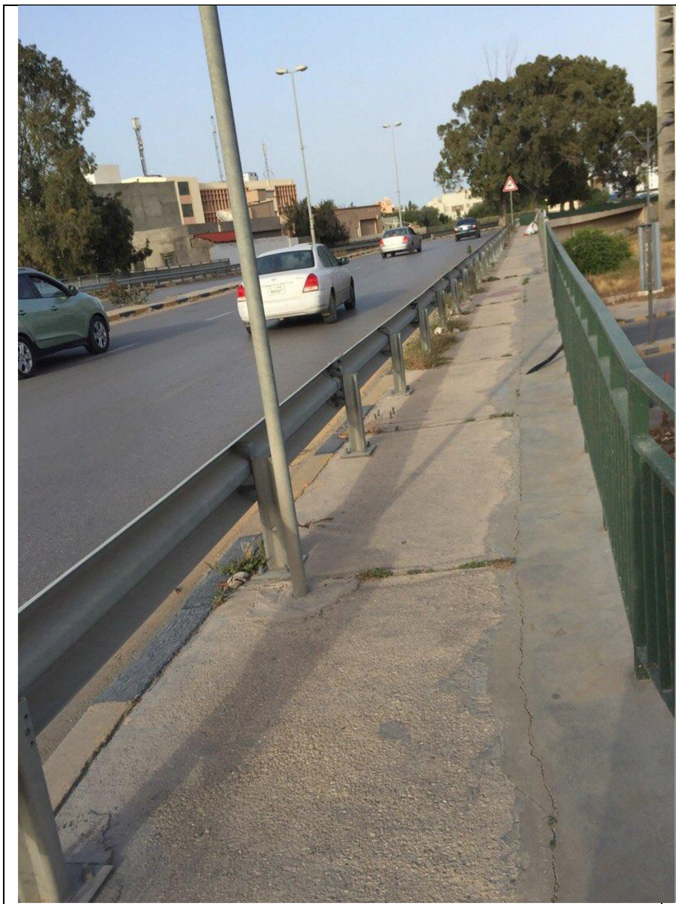


Figure (IX-57) A general view of Pedestrian path on bridge	Figure (IX-58) A general view of Pedestrian path under bridge

Asphalt wearing layer

Lots of dust and sand on the bridge made the visual inspection of asphalt wearing layer very difficult. The condition of the asphalt layer is shown in Figures IX-59 and IX-60. On visible part of asphalt wearing layer, no damages and defects were noticed.

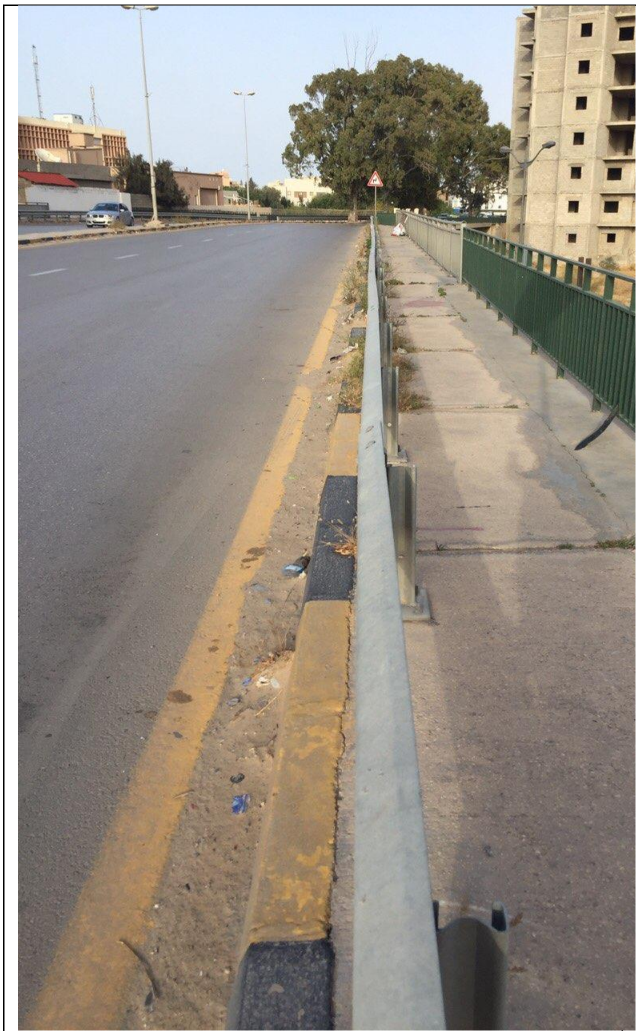



Figure (IX-59) A general view of asphalt wearing layer	Figure (IX-60) Detail of asphalt wearing layer

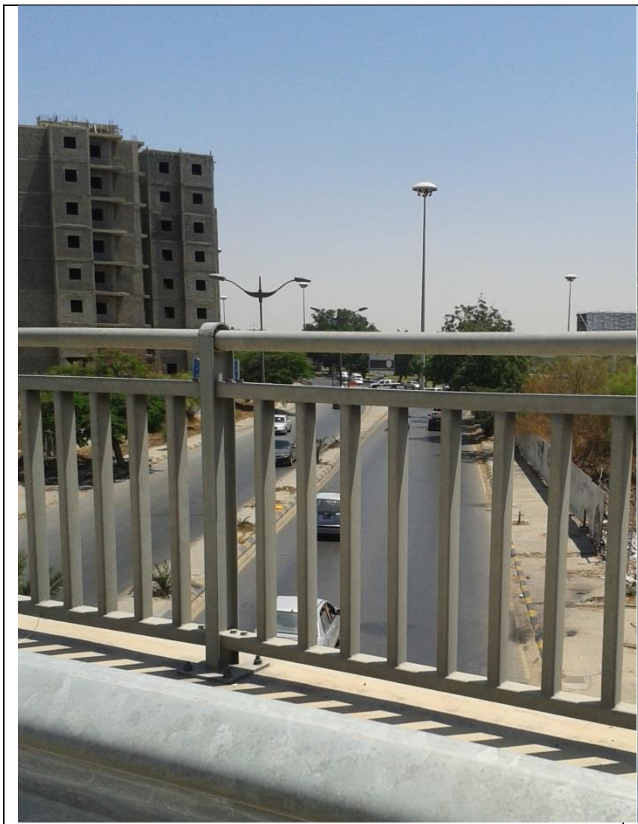
Fences

The fences are in good condition. The state of the fences is shown in Fig IX-59 and IX-61.

Curbs

The corner spalling off of concrete and peeling off black and yellow colour were spotted as typical damages. The corner spalling off is caused by hitting by car.

Figure IX-62 illustrates the state of curbs.



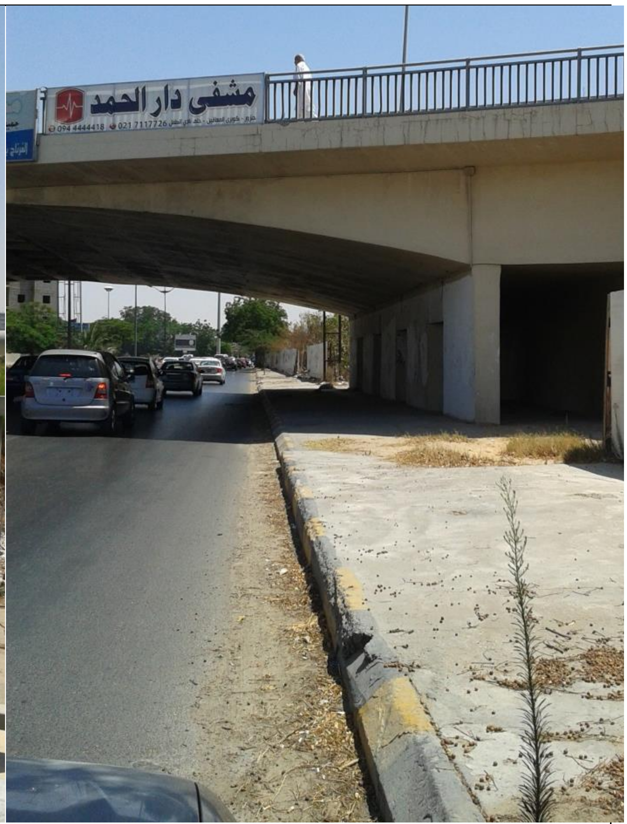


Figure (IX-61) A general view of fences on the bridge	Figure (IX-62) A general view of curbs

Guardrail

Generally, the guardrails are in good condition as they have not been caught by corrosion (Fig IX-59 and IX-63). The reflective sign in guardrail is good condition. The condition of guardrail and reflective sign is illustrated in Fig IX-63.

Figure (IX-63) A general view of guardrail on the bridge

Catch pit

Blockage of drainage channels under the bridge due to waste and soil, were not seen.

The condition of catch pit is illustrated in Fig IX-64.

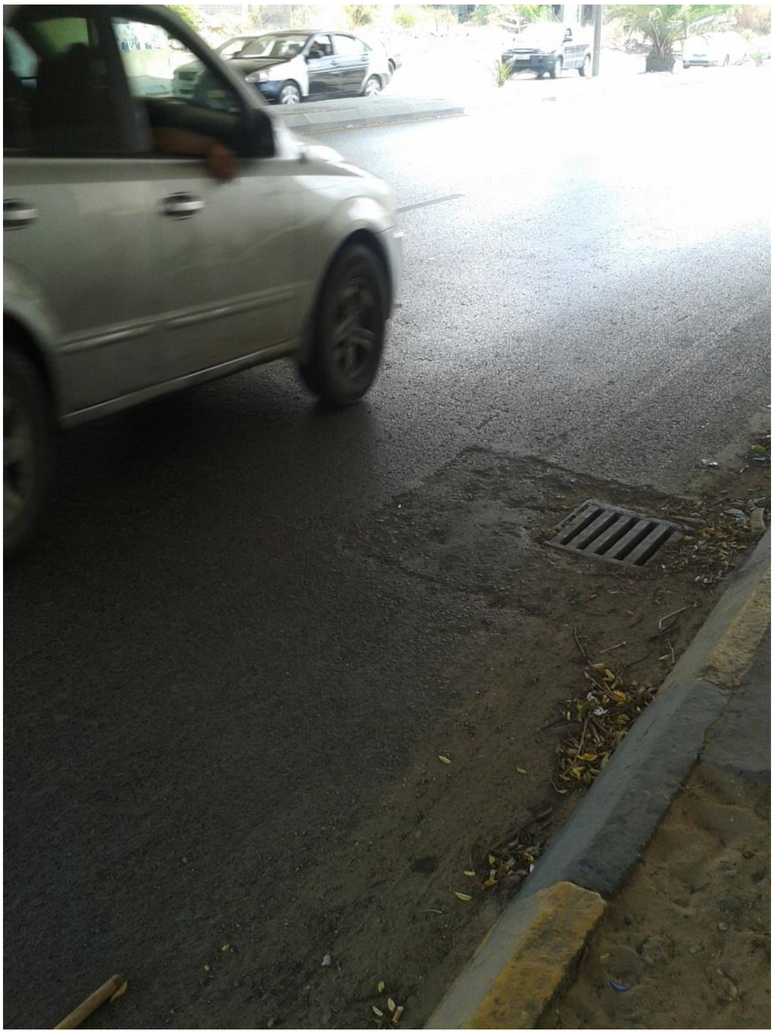


Figure (IX-64) A general view of catch pit under the bridge

Depth of carbonation

The extent of carbonation on was assessed by treating the fresh broken part of repair material with phenolphthalein indicator. The carbonation test were done on following RC elements: ceiling and abutments (west and east side)

The obtained results are shown in the table IX-5.

Table (IX-5) Results of measuring the depth of carbonation on RC elements

Elements	description	Depth of carbonation (mm)	
ceiling	Repair material – mortar	10	
Abutment (west side)	Repair material – mortar	18	
Abutment (east side)	Repair material – mortar	5	

By the analysing obtained results it is concluded that process of carbonation has already started on almost all RC elements. The largest value of carbonation was measured on Abutment (west side) (18mm). This value is almost double than expected. The rate of carbonation is usually 0,5mm/year and with that speed the depth of carbonation should have been about 3,5mm. Some measures to slow down the rate of carbonation should be considered.

3.3 General conclusion

The first routine inspection of The Alsseka Road Bridge was done after 6 years of its repair. By the analysing of collected results, which were obtained by visual inspection and measuring depth of carbonation, the next conclusions are derived:

- There are two main problems with lateral beam. The first one is serious mechanical damage and the second one is longitudinal crack of dawn surface of beam, near external edge. On the place of mechanical damage, concrete and repair mortar were broken and fallen down, the main reinforcing bars were pulled out and deformed and stirrups were broken and deformed, also. The described damages are serious as they reduce mechanical resistance and durability of lateral beam.
- The main problem of deck slab is leakage of rain water through the support place of simple beam slab on arch cantilever slab. This leakage may cause dissolving of $Ca(OH)_2$ and corrosion of reinforcing bars. Besides the water leakage, very thin vertical crack have been spotted on ribs of simple beam slab. Described damage could reduce the depth of concrete cover in following period of service life of the bridge.
- The horizontal, vertical cracks and net like cracks are typical damage of cantilever slabs. The horizontal crack was appeared between two layers of concrete and it stretches along the whole length of slab. The vertical cracks are located on side surface and they could be caused by dry shrinkage of concrete and/or by corrosion of reinforcement.
- The typical damages of supporting and abutment walls are net like cracks. They were occurred in repair mortar cover due to their drying shrinkage. Separation and spalling off of protecting mortar layer is characteristic for plinth, but this damage has local character.
- The initial damages have occurred on pedestrian path, such as local surface pits, biological corrosion and uneven cold joints.
- Neither damages nor defects were seen on wearing asphalt layer.
- Bridge fence and guardrails are in good conditions.
- Curbs are locally damaged by hitting by car, but they are still in good conditions.
- Due to the lack of maintenance the growth of some plants in cold joints of pedestrian paths, as well as, the presence of dust and urban rubbish on the road were noticed. Also, traffic signs, including intermittent lines, that allow the vehicle to cross, and the side lines have been partly disappeared.

- Carbonation has already started on ceiling and supporting walls. The largest value of carbonation was measured on abutment, west side, (18mm).

Finally, the general stability, bearing capacity, functionality and durability have not been jeopardized, yet, but local reduction of bearing capacity of lateral beam on the place of mechanical attack, is possible. Net like cracks were located in cover. They could provoke the reinforced bar corrosion. The corrosion process can be slowed down by some measures like impregnation. The same measures are suggested for RC elements caught by carbonation. Due to the risk of progression of reinforcement corrosion horizontal crack in lateral beam and vertical cracks in cantilever slabs, as well as water leakage should be prevented by other methods. Local separation and spalling off of repair mortar can be easily re-repaired.

The review of registered damages during routine visual inspection is presented in table IX-6.

Table IX-6 Review of registered damages during the first routine inspection

RC element	Cracks	Pilling off protecting paint	Spalling off	Water leakage/traces	Carbonation
Lateral beam	+ longitudinal	-	Large local damage, spalling off concrete, pulled out and deformed bars	+ Leakage on the spot of supporting Traces and stains of water through joins	Not measured
Deck ceiling Arch cantilever slabs	-	-	+ local	+ Leakage on the spot of supporting Traces and stains of water	Not measured
Deck ceiling Simple beam slab	+ Vertical, simple beam deck	-	+ local	+ Traces and stains of water	+
cantilever slabs	+ Net like, vertical	-	+ local	+ Traces and	Not measured

	and horizontal			stains of water	
Supporting walls	+ Net like	+	+	-	Not measured
Abutment	+ Net like	-	-	+ Traces and stains of rust	+ (in progress)

4. BAB BIN GHESHIR ROAD BRIDGE

The works on repair of Bab Bin Gheshir Road Bridge started in April 2009 and ended in October 2009.

The first routine inspection was done in August 2016.

This routine inspection has included:

- Visual inspection of all available elements of RC structure of bridge, as well as others elements, such as fences, curbes, pedestrian paths, guardrails and asphalt wearing layers and
- Checking of depth of concrete/mortar cover carbonization.

The results of routine inspection are given below.

9.4.1 Visual inspection of available RC elements

The visual inspection has encompassed the next RC elements:

- Deck ceiling,
- Cantilever slabs,
- Tunnel ceiling
- Supporting walls and
- Abutments.

A general appearance of the bridge is shown in Fig IX-65 – IX-68.

Figure (IX-65) A general view of Bab Bin Gheshir road bridge, middle part	Figure (IX-66) A general view of Bab Bin Gheshir road bridge, middle part

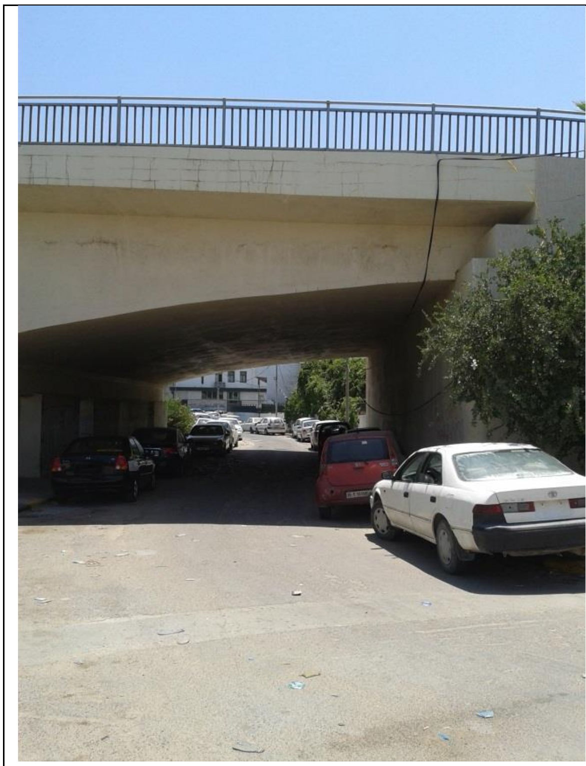


Figure (IX-67) A general view of Bab Bin Gheshir road bridge, side part, west	Figure (IX-68) A general view of Bab Bin Gheshir road bridge, side part, east

Deck ceiling

The deck ceiling slab is located in the middle part of bridge (main span). The condition of down side of deck slab is illustrated in Fig IX-69 – Fig IX-70.

By the visual inspection the following damages are noticed:

- Stains of water
- Net like cracks (Fig IX-70)

Both, traces and stains of water and net like cracks, are not characteristic damages for whole visible part of deck ceiling slab. The main part of deck ceiling slab is undamaged (Fig IX-69).

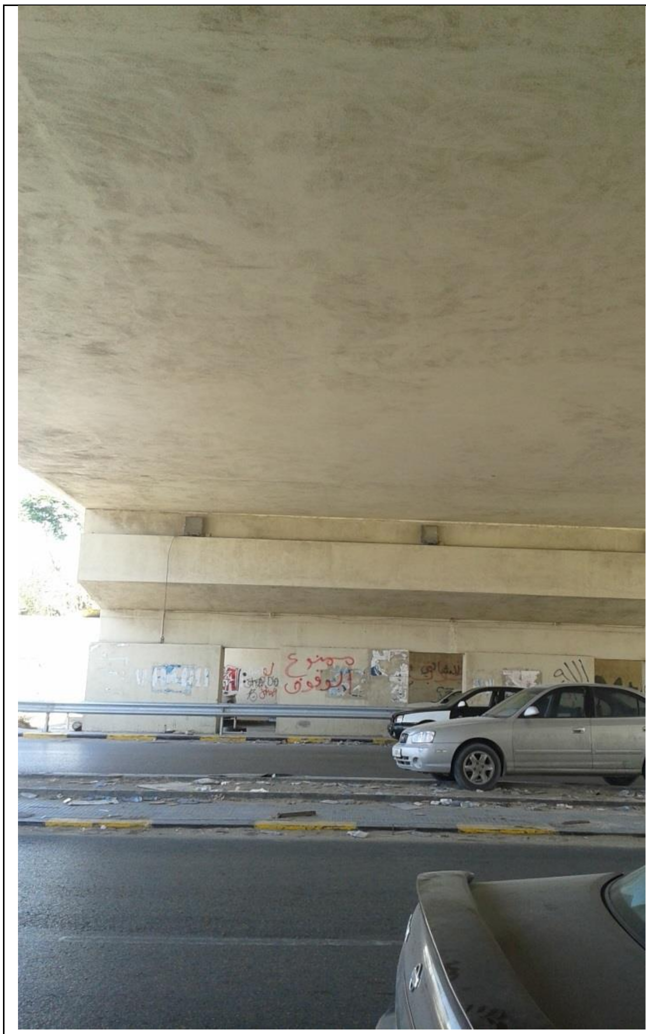


Figure (IX-69) View of deck slab with support walls, undamaged surface	Figure (IX-70) View of deck slab, down side of cantilever slab with support walls, net like crack

Cantilever slab

The view of Cantilever slab is illustrated in Fig IX-69.

By the visual inspection the following damages are noticed on cantilever slabs:

- Traces and stains of water (Fig IX-65, IX-70and IX-71)
- Local spalling off concrete and (Fig IX-71)
- Net like, horizontal and vertical cracks (Fig IX-70 and IX-71).

Figure (IX-71) View of cantilever slab, vertical and horizontal cracks

Traces and stains of water are characteristic for whole side and down surface of cantilever slabs. They occurred due to bad drainage of rain water (Fig IX-65, IX-68 and IX-69).

The upper horizontal crack was appeared between two layers of concrete and it stretches along the whole length of slab (Fig IX-65 and IX-69). Other horizontal cracks are shorter and probably caused by reinforced bar corrosion (this conclusion is driven on the base of arrangement of horizontal cracks; they are located at the same vertical distance).

Vertical cracks are, also, characteristic damage on side surface of cantilever slab. They were caused by drying shrinkage of concrete and/or by corrosion of reinforcement (Fig IX-65 and IX-71).

Vertical and horizontal cracks are followed with traces of rust.

Net like cracks are also noticed on side surface of analysed element, but they are very thin (Fig IX-65 and IX-71).

Spalling off of concrete is very local, shallow and it was spotted on side surfaces (Fig IX-65 and IX-71).

Dawn part of cantilever slab has transversal cracks which are very thin, but more noticeable near the free edge of the slab. They represent an extension of vertical cracks from the side surface of the cantilever slab.

On the basis of those descriptions, it can be concluded that described cracks on side surface of cantilever slabs, particularly vertical and horizontal cracks, could cause reinforcement corrosion. Because of that, execution of new concrete/mortar cover is recommended. Also, to avoid appearance of new damages, the drainage of atmospheric water should be improved.

Tunnel slabs

The condition of tunnel slabs is illustrated in Fig IX-72 – Fig IX-74.

Figure (IX-72) View of tunnel slab with cantilever slab, west	Figure (IX-73) View of tunnel slab with cantilever slab, east

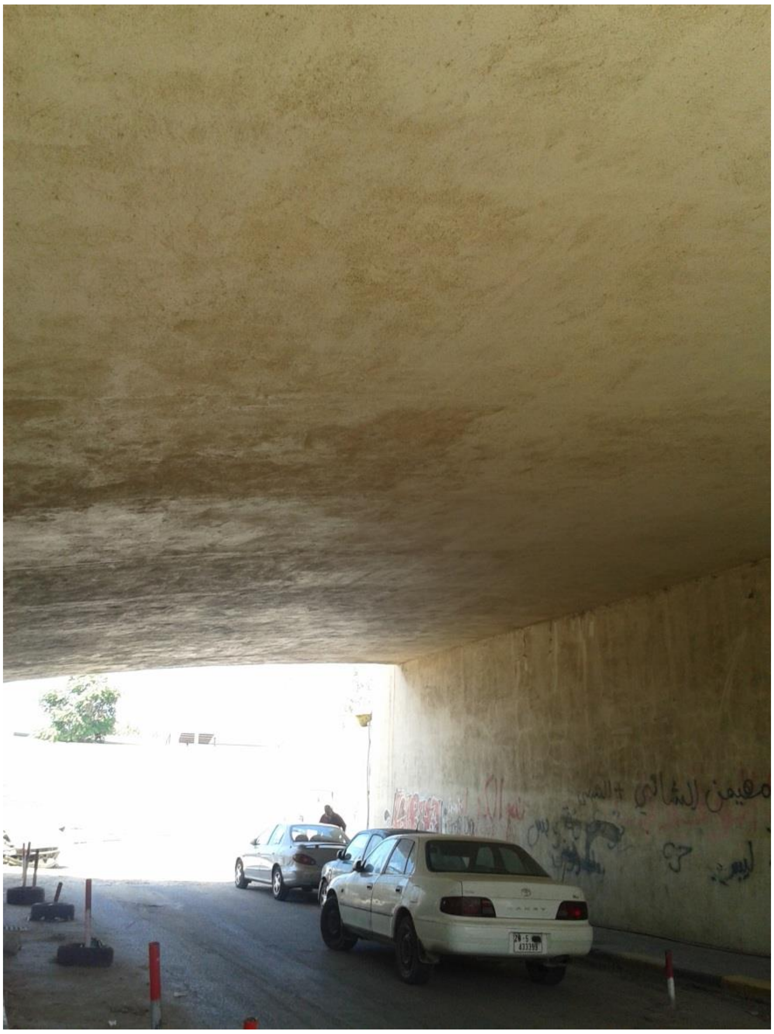


Figure (IX-74) View of tunnel ceiling with abutment

By the visual inspection the following damages are noticed on visible parts of tunnel slabs:

- Traces and stains of water (Fig IX-72, and IX-74)
- Local spalling off of concrete and (Fig IX-73)
- Net like, horizontal and vertical cracks (Fig IX-72 and IX-73).

Traces and stains of water are characteristic for whole side of tunnel slabs. They occurred due to bad drainage of rain water, and bad quality of concrete on the place of cold joins (Fig IX-72 and IX-73).

The characteristic damage of side surfaces of tunnel slabs are horizontal cracks. These cracks appeared between two layers of concrete and they stretch along the whole length of slab (Fig IX-72 and IX-73). They are followed by leakage, therefore white, rust coloured and black traces and stains are visible along the cracks. Rust coloured stains might indicate the corrosion of reinforced bars.

Vertical cracks were spotted on down part of side surface. They are located very near to edge of slabs. They are very short and uniformly arranged.

Spalling off of mortar cover is characteristic for down external edge of tunnel slabs. They have long length, but very short height.

Described damages were not seen on down part of tunnel slabs (tunnel ceiling).

The most dangerous damage is horizontal cracks. They could reduce durability and mechanical resistance due to corrosion of reinforced bars and leakage.

Supporting walls

The condition of supporting walls is illustrated in Fig IX- 75 – Fig9.78. By the visual inspection the following damages are noticed:

- Net like cracks (Fig IX-77 – IX-78)

Net like cracks are characteristic damage of inspected walls. They were occurred in repair mortar cover due to their drying shrinkage (Fig IX-77 and IX-78)

Figure (IX-75) View of supporting wall with ceiling	Figure (IX-76) View of supporting wall

	
Figure (IX-77) Net like cracks in supporting walls	Figure (IX-78) A View of supporting wall, cracks in wall

Abutment

The condition of abutment is illustrated in Fig IX-79 – Fig IX-82.

By the visual inspection the following damages are noticed on visible parts of abutments:

- Traces and stains of water (Fig IX-79 – IX-82)
- Horizontal, vertical and crack of different direction (Fig IX-81 and IX-82).

Traces and stains of water are noticed along the place of connection between abutment and tunnel slab. The horizontal crack exists at that place, also. This crack is supposed to be very deep, and is connected with expansion joint between the bridge structure and the approached structure. The conclusion was made on the basis of traces of water leakage (Fig. IX-80 & IX-82).

In the external part of abutment, the cracks of different directions were appeared (Fig. IX-81). They are very wide, and followed with dark stains. The cause of their appearance is unknown.

On the basis of those descriptions, it can be concluded that registered cracks in the external part of abutment might cause further development of damages, such as the corrosion of reinforced bars. The leakage through horizontal crack should be stopped by repairing or changing the dilatation device between the bridge and the approached structures.

Figure (IX-79): General view of abutment	Figure (IX-80): View of abutment, water stains

Figure (IX-81) View of external part of abutment, cracking of wall	Figure (IX-82) View of abutment, leakage through horizontal crack

4.2. Visual inspection of other bridge elements

Pedestrian Path: Through visual inspection, several types of damages were spotted:

- Uneven cold joints
- Local surface pits and
- Biological corrosion,
- Missing part of concrete.
- The appearance of some plants on the cold joints.

Figures IX-83 and IX-84 illustrate the condition of the pedestrian path.

It was observed that the cold joints are very uneven, sometimes with surplus of infilled material, some times without it and sometimes with plants which grow from joints (Fig IX-83-IX-84).

On concrete surfaces between cold joints the shallow pits and erosion caused wearing were spotted. It is supposed that they were caused by mechanical action.

It could be concluded that initial damages have occurred on pedestrian path surface and in cold joints. Some prevention measures for slowing down appearance of new and progress of numbered damages should be undertaken.

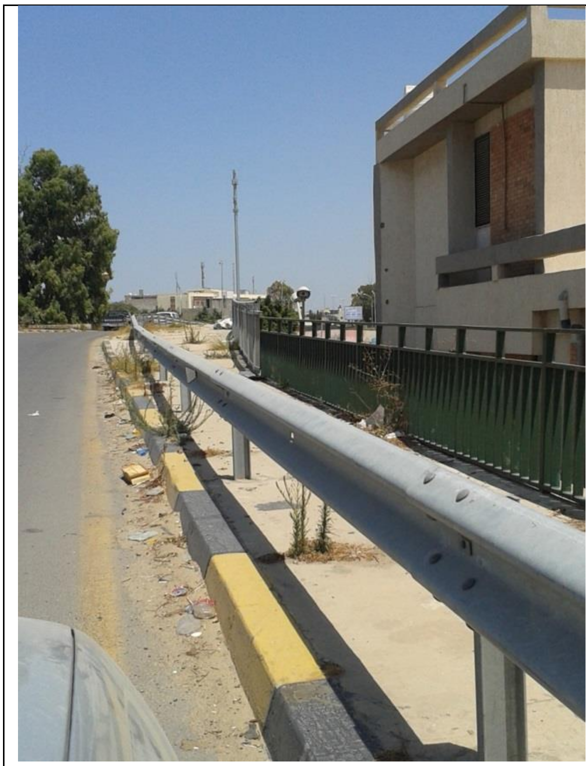


Figure (IX-83) A general view of Pedestrian path on bridge

Figure (IX-84) A general view of Pedestrian path on bridge

Asphalt wearing layer

Lots of dust and sand on the bridge made the visual inspection of asphalt wearing layer very difficult. The condition of the asphalt layer is shown in Figures IX-84 and IX-85.

On visible part of asphalt wearing layer, only one longitudinal crack and a few local surface pits were spotted.

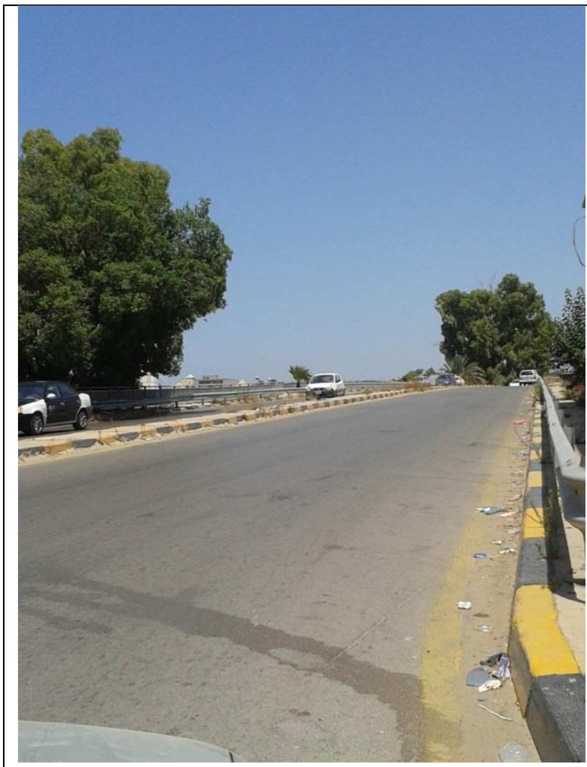


Figure (IX-85) A general view of asphalt wearing layer	Figure (IX-86) Detail of asphalt wearing layer

Fences

The fences are in good condition. The state of the fences is shown in Fig IX-87.

Curbs

The local peeling off black and yellow colour was spotted as typical damages.

The figures IX-83 to IX-88 illustrate the state of curbs.

Guardrail

Generally, the guardrails are in good condition as they have not been caught by corrosion, but a few mechanical damages were spotted. On these places, the guardrail is missing or is deformed and broken due to car accident (Fig IX-88). The reflective sign in guardrail is good condition.

The condition of guardrail and reflective sign is illustrated in Fig IX-87-IX-88.

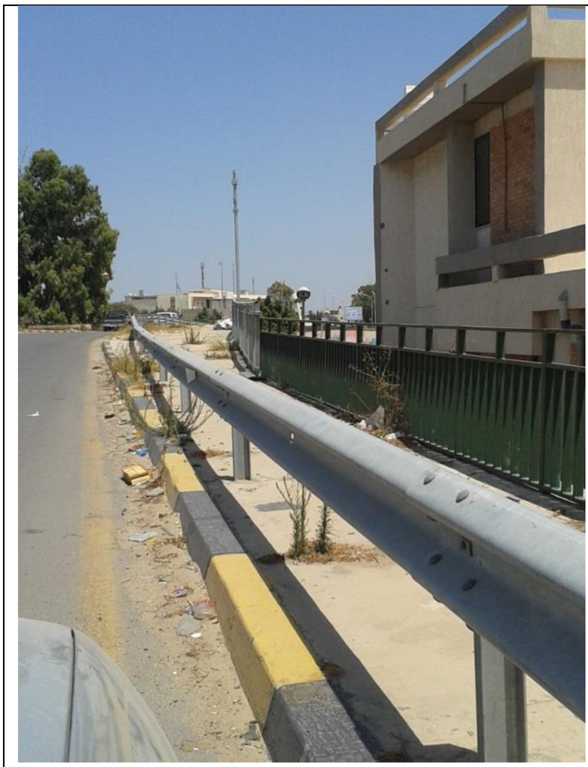


Figure (IX-87) A general view of fences on the bridge	Figure (IX-88) A general view of curbs

Catch pit

The Blockage of drainage channels under the bridge with garbage, dust and sand were noticed. Due to that the rainwater does not be drainage.

The condition of catch pit is illustrated in Fig IX-89.

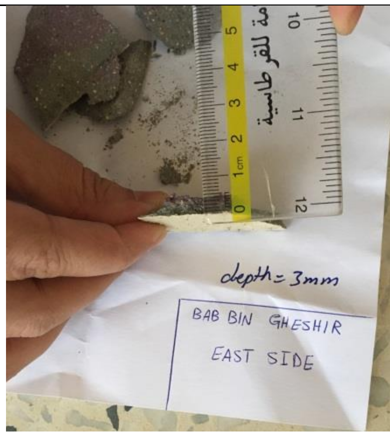
Figure (IX-89) A general view of catch pit under the bridge

Depth of carbonation

The extent of carbonation on was assessed by treating the fresh broken part of repair material with phenolphthalein indicator. The carbonation test were done on following RC elements: ceiling and abutments (west and east side)

The obtained results are shown in the table IX-7.

Table (IX-7) Results of measuring the depth of carbonation on RC elements

Elements	description	Depth of carbonation (mm)	
ceiling	Repair material – mortar	2	
Abutment (west side)	Repair material – mortar	5	
Abutment (east side)	Repair material – mortar	3	

By the analyzing obtained results it is concluded that process of carbonation has already started on almost all RC elements. The largest value of carbonation was measured on Abutment (west side) (5mm). This value is almost double than expected. The rate of carbonation is usually 0,5mm/year and with that speed the depth of carbonation should have been about 3,5mm. Some measures to slow down the rate of carbonation should be considered.

4.3. General conclusion

The first routine inspection of The Bab Bin Gheshir Road bridge was done after 6 years of its repair. By the analysing of collected results, which were obtained by visual inspection and measuring depth of carbonation, the next conclusions are derived:

- The deck ceiling slab is located in the main span of bridge. The main part of deck ceiling slab is undamaged. The traces and stains of water and net like cracks are locally visible on ceiling surface of the slab.
- Tunnel slabs are located above side spans of the bridge. The characteristic damage and at same time the most dangerous ones are the horizontal cracks. These cracks appeared between two layers of concrete. They are followed by leakage, therefore white, rust coloured and black traces and stains are visible along the cracks. Rust coloured stains might indicate the corrosion of reinforced bars. The vertical cracks were registered but, according to their description, they could be neglected. Spalling off of mortar cover is located on the down external edge of tunnel slabs. This damage is local and, has long length, but very short height.
- The horizontal and vertical cracks are typical damage of side surface of cantilever slabs. The upper horizontal crack was appeared between two layers of concrete. Other horizontal cracks are shorter and probably caused by reinforced bar corrosion while the vertical cracks appearance were caused by the same reason and/or by drying shrinkage of concrete. Vertical and horizontal cracks have traces of rust. Vertical cracks are extended on down side of the slab, but they are only noticeable next to external edge of the slabs. The main part of tunnel ceiling slab is undamaged.
- The typical damage of supporting walls is net like cracks. They are more expressed on side surfaces of walls, and they are located in concrete cover.
- Characteristic damages of abutment are cracks, horizontal and cracks in different directions. On the basis of those descriptions it can be concluded that registered cracks might cause further development of damages, such as the corrosion of reinforced bars or a decrease in bearing capacity of abutment. They are mostly followed by leakage and rust stains.
- The initial damages have occurred on pedestrian path, such as local surface pits, biological corrosion and uneven cold joints.
- Wearing layer of traffic lanes has only one longitudinal crack and a few local surface pits, thus it could be concluded that they are undamaged.
- The bridge fence is in good conditions.
- The guardrails are in good condition, as they have not been caught by corrosion, but missing or deformed parts of guardrails were spotted. On these places, the guardrail is and broken due to car accident.

- Curbs are still in good conditions.
- Catch pit are not in function, because of blocking by dust, sand and garbage.
- Due to the lack of maintenance the growth of some plants in cold joints of pedestrian paths, as well as, the presence of dust and urban rubbish on the road were noticed. Also, traffic signs, including intermittent lines, that allow the vehicle to cross, and the side lines have been disappeared.
- Carbonation has already started on ceiling and abutment walls. The largest value of carbonation was measured on abutment wall (5mm).

Finally, the stability, bearing capacity, functionality and durability have not been jeopardized, yet. The characteristic damages are cracks. They were spotted on the surface of inspected RC elements, especially on side surfaces of cantilever slabs and tunnel slabs and on supporting walls and abutments. The net like cracks could provoke the reinforced bar corrosion. This process can be slowed down by some measures like impregnation. The same measures are suggested for RC elements caught by carbonation. Due to the risk of progression of reinforcement corrosion, the horizontal, vertical cracks and cracks of different directions, as well as places of water leakage should be prevented by other methods.

Table IX-8 Review of registered damages during the first routine inspection

RC element		Cracks	Pilling off protecting paint	Spalling off	Water leakage/traces	Carbonation
Deck ceiling		+ Net like crack	-	-	+ Stains of water	+
Cantilever slabs		+ Net like, vertical and horizontal	-	+ Local	+ Traces and stains of water	Not measured
Tunnel ceiling		+ Horizontal, vertical and net like crack	-	+ Local	+ Leakage, Traces and stains of water	Not measured
Supporting walls		+ Net like crack	-	-	-	Not measured

Abutment		+ Horizontal, vertical and crack of different direction	-	-	+ Leakage, Traces and stains of water	+ (in progress)

5. AL SREEM ROAD BRIDGE

The works on repair Al Sreem Road Bridge started in October 2009 and ended in December 2009.

The first routine inspection was done in August 2016.

This routine inspection has included:

- Visual inspection of all available elements of RC structure of bridge, as well as others elements, such as sidewalks, curb stones, catch pits, fences, expansion joints, guardrails and asphalt wearing layers and
- Checking of depth of concrete/mortar cover carbonization.

The results of routine inspection are given below.

5.1. Visual inspection of available RC elements

The visual inspection has encompassed the next RC elements:

- Deck slab
- Longitudinal and transversal supporting (ceiling) RC beams
- Cantilever slab
- Supporting wall (Masonry support walls made of stone)
- Abutment wall (Masonry wall made of stone and covered by plastering)

A general appearance of the bridge is shown in Fig IX-90 and Fig IX-91.

Figure (IX-90) A general view of Al Sreem Road Bridge

Figure (IX-91) A general view of Al Sreem Road Bridge

Deck slab

The condition of down side of deck slab is illustrated in Fig IX-92 – Fig IX-93.

By the visual inspection no damages were spotted on down surface of deck slab, only traces of dust were noticed.

Figure (IX-92) View of deck slab with support walls, longitudinal and transversal supporting beams

Figure (IX-93) View of deck slab, longitudinal and transversal supporting beams

Longitudinal and transversal supporting (ceiling) RC beams

The condition of longitudinal and transverse deck ceiling beams is illustrated in Fig IX-94-IX-95.

During of the visual survey the down part of superstructure was inspected. The following defects and damages are noticed:

- Net like crack (Fig IX-94 and IX-95)
- Surface and corner spalling off of repair mortar (Fig IX-92 – IX-95)
- Leakage, traces and stains of water and dust (Fig IX-94 and IX-95)

Figure (IX-94) Longitudinal and transversal supporting beams, net like cracks, corner spalling off, traces of dust	Figure (IX-95) Net like cracks on side and down surfaces of beams, corner spalling off

The characteristic damages of all types of supporting beams are net like cracks. They are very thin and spread in repair mortar cover. They are mostly expressed on down surfaces of the beams. The main cause of their appearance is drying shrinkage of repair mortar due to inadequate curing or bad environmental conditions (hot and dry air).

The surface and corner spalling off are local, shallow and cover very small surfaces, except on external longitudinal beam (Fig IX-93), where described damage overspread the whole width of the girder. The cause of their appearance are supposed to be mechanical damage or bad adhesion between concrete surface and repair mortar.

The traces of water leakage were spotted in the place of expansion joint in superstructure of the bridge (Fig. IX-96).

Figure (IX-96) Expansion joint in superstructure of the bridge: water leakage, white stains

All described damages do not jeopardize the bearing capacity of this superstructure, but durability is reduced in a certain way.

Cantilever slab

The view of Cantilever slab is illustrated in Fig IX-93 and IX-97.

By the visual inspection the following damages are noticed on cantilever slabs:

- Local spalling off of mortar and (Fig IX-95)
- Net like cracks (Fig IX-95).

Figure (IX-97) View of cantilever slab

Both noticed types of damage are not characteristic as they cover very small surfaces.

Exterior wall (Masonry support walls made of stone)
The condition of supporting walls is illustrated in Fig IX-98 – Fig IX-99.

By the visual inspection the following damages are noticed on visible parts of supporting masonry wall:

- Traces and dark stains of water (Fig IX-98, and IX-99)

Neither cracks, nor buckling nor disintegration of stone part wall was seen.

Figure (IX-98) View of supporting wall with ceiling	Figure (IX-99) View of supporting wall

Only traces of water were spotted on the place of expansion joint in sub structure and along the RC cornice.

Masonry abutments made of stone
The condition of abutments is illustrated in Fig IX-100.

By the visual inspection the following damages are noticed on visible parts of abutments:

- Net like cracks and other cracks caused by drying shrinkage (Fig IX-100).
- Peeling of protecting painting (Fig IX-100)

Figure (IX-100) View of abutment

The cracks caused by drying shrinkage are mostly expressed on side (external) surface of abutment wall. They have horizontal direction.

5.2 Visual inspection of other bridge elements

Pedestrian Path

Through visual inspection, several types of damages were spotted:

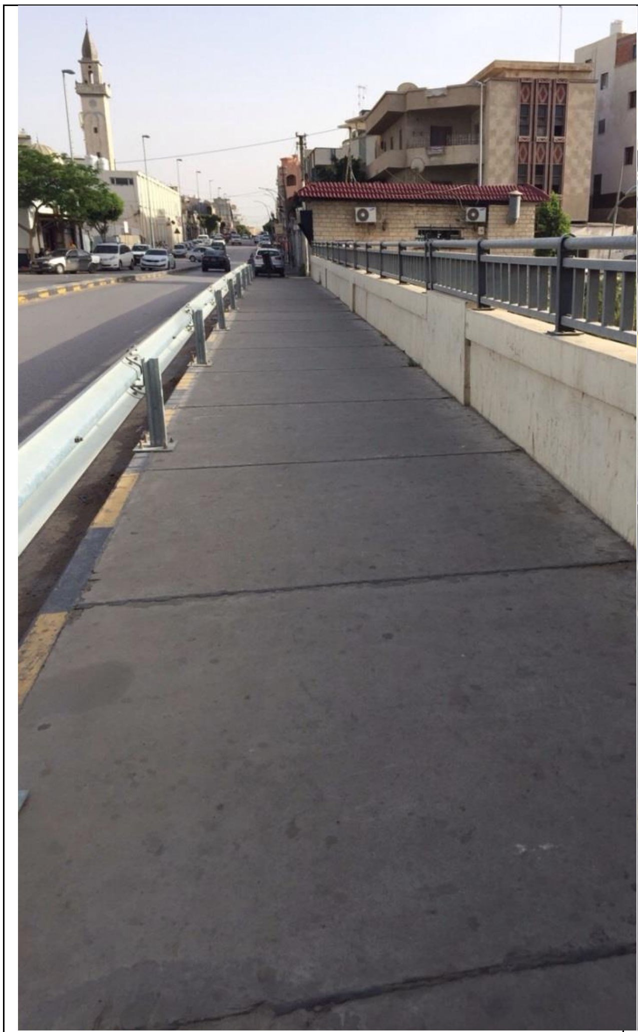
- Uneven cold joints
- Local surface pits and
- Missing part of concrete.

Figures IX-101 and IX-102 illustrate the condition of the pedestrian path.

It was observed that the cold joints are very uneven, sometimes with surplus of infilled material, some times without it (Fig IX-101-IX-102).

On concrete surfaces between cold joints the shallow pits and erosion caused wearing were spotted. It is supposed that they were caused by mechanical action.

It could be concluded that initial damages have occurred on pedestrian path surface and in cold joints. Some prevention measures for slowing down appearance of new and progress of numbered damages should be undertaken.



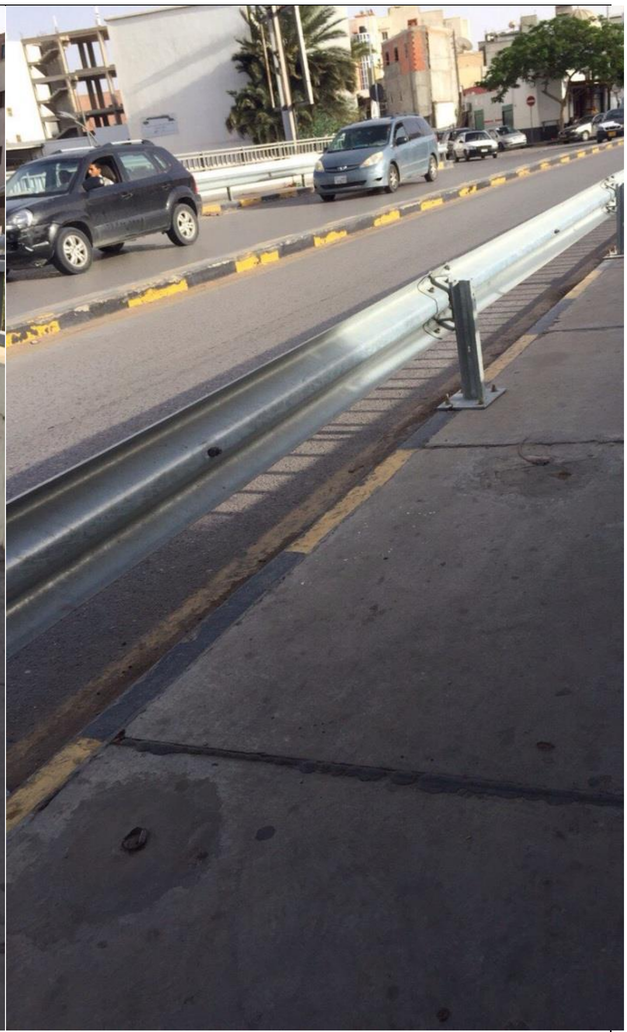


Figure (IX-101) A general view of Pedestrian path on bridge	Figure (IX-102) A general view of Pedestrian path on bridge

Asphalt wearing layer

Lots of dust and sand on the bridge made the visual inspection of asphalt wearing layer very difficult. The condition of the asphalt layer is shown in Figures IX-103 and IX-104.

Figure (IX-103) A general view of asphalt wearing layer

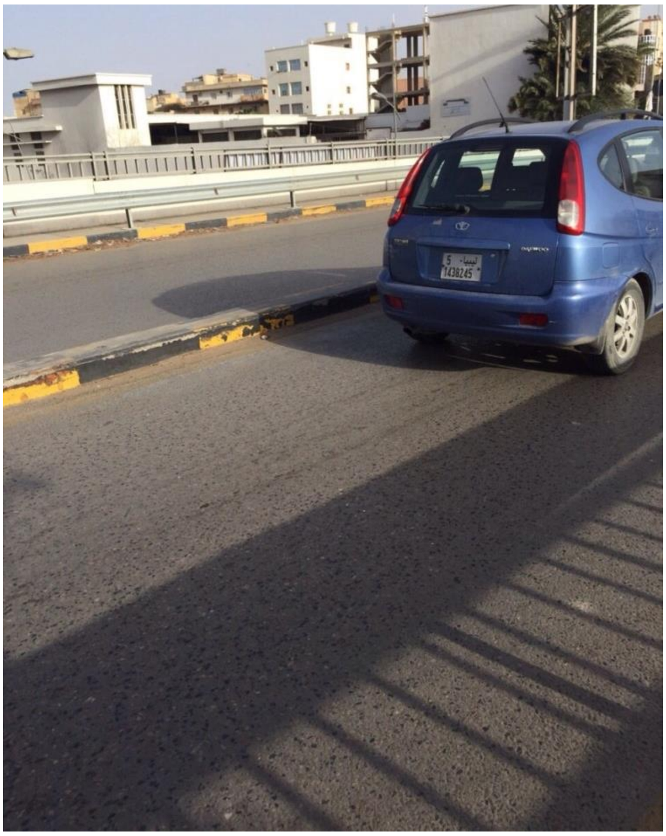


Figure (IX-104) Detail of asphalt wearing layer

The erosion of upper asphalt layer, caused by wearing, is the main type of damage that has been spotted (Fig IX-104).

Fences

The fence consists of two parts, main concrete part and upper metal part (Fig IX-105). The metal part of fences is in good condition. The state of that part of fences is shown in Fig IX-105-IX-106.

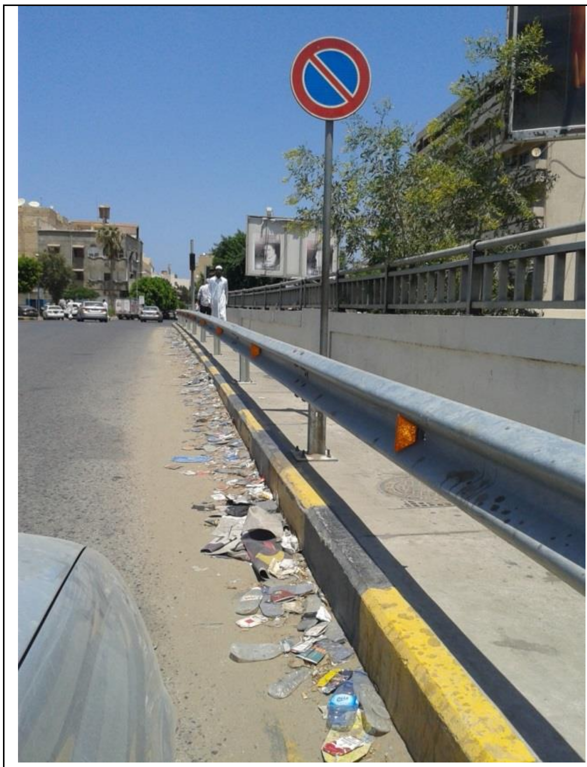


Figure (IX-105) A general view of fences, curbs and Guardrail on the bridge

Figure (IX-106) A general view of metal part of fences, curbs and Guardrail on the bridge

Figure (IX-107) A general view of concrete part of fences

The concrete part of fences can be seen in Fig IX-107. The characteristic damage is net like cracking caused by drying shrinkage. This crack does not represent any serious damage of concrete.

Curbs

The local peeling off black and yellow colour was spotted as typical damages.

The figures IX-105 to IX-106 illustrate the state of curbs.

Guardrail

Generally, the guardrails and reflective sign are good condition.

The condition of guardrail and reflective sign is illustrated in Fig IX-105-IX-106.

Catch pit

The drainage channels under the bridge have been still in function, despite a large amount of garbage, dust and sand nearby.

The condition of catch pit is illustrated in Fig IX-108.

Figure (IX-108) A general view of catch pit under the bridge

Depth of carbonation

The extent of carbonation on was assessed by treating the fresh broken part of repair material with phenolphthalein indicator. The carbonation test were done on following RC elements: ceiling and abutments (south and north)

The obtained results are shown in the table IX-9.

Table (IX-9) Results of measuring the depth of carbonation on RC elements

Elements	description	Depth of carbonation (mm)	
ceiling	Repair material – mortar	6	
Abutment (south side)	Repair material – mortar	20	
Abutment (North side)	Repair material – mortar	0	

By the analyzing obtained results it is concluded that process of carbonation has already started on almost all RC elements. The largest value of carbonation was measured on ceiling (6mm). This value is almost double than expected. The rate of carbonation is usually 0,5mm/year and with that speed the depth of carbonation should have been about 3,5mm. Some measures to slow down the rate of carbonation should be considered.

5.3. General conclusion

The first routine inspection of The Al Sreem Road Bridge was done after 7 years of its repair. By the analysing of collected results, which were obtained by visual inspection and measuring depth of carbonation, the next conclusions are derived:

- The supporting wall was built as masonry stone structure. During visual inspection no damages were spotted. Only traces of leakage through expansion joint between two supporting walls were seen.
- The abutment walls were also built of stone, but they are covered with plaster layer. These walls are in good condition, since spotted damages, such as cracks due to drying shrinkage, are only concentrated in mortar layer.
- All visible part of deck slabs of bridge superstructure are in good condition. No damages were spotted on down surface of deck slab, only traces of dust were noticed.
- The characteristic damages of all types of supporting beams are net like cracks caused by shrinkage of repair mortar. They are mostly expressed on down surfaces of the beams. The surface and corner spalling are spotted, but very locally. All described damages do not jeopardize the bearing capacity of this superstructure, but durability is reduced in a certain way.
- All visible part of cantilever slabs of bridge superstructure are in good condition. Only local spalling off of mortar and thin net like cracks were noticed.
- The expansion joints between two parts of superstructure and between supporting walls are in good condition. Only traces of leakage are noticed, but because of possible appearance of damages during time, the problem of leakage should be solved.
- The initial damages have occurred on pedestrian path (uneven cold joints and a local surface pits)
- The characteristic damages of wearing layer of traffic lanes are erosion caused by wearing.
- Due to the lack of maintenance, the presence of dust and urban rubbish on the road were noticed. Also, traffic signs, including intermittent lines, that allow the vehicle to cross, and the side lines have been disappeared, and they have to be refreshed.
- The bridge fence consists of two parts, main concrete part and upper metal part. Both parts are in good condition. Any damages of the metal part of the

bridge fence have not been spotted yet, while on the surface of the concrete part net like cracks have been appeared.

- The curbs are in good conditions. The local peeling off black and yellow colour was spotted as typical damages.
- The guardrails and reflective sign are relatively in good condition.
- The drainage channels under the bridge have been still in function, despite a large amount of garbage, dust and sand nearby.
- The carbonation has already started in deck ceiling slab. The largest value of carbonation was 6mm.

Finally, the stability, bearing capacity, functionality and durability have not been jeopardized, yet. As it was mentioned, a few damages were spotted on the surface of inspected RC elements. All damages are in the initial state and could be slowed down by some measures like impregnation. The same measures are suggested for RC elements caught by carbonation. Local spalling off of repair mortar can be repaired by re-plastering very easily.

Table IX-10 Review of registered damages during the first routine inspection

RC element	Cracks	Pilling off protecting paint	Spalling off	Water leakage/traces	Carbonation
Deck ceiling	-	-	-	+ traces of dust	+ (in progress)
Longitudinal and transversal supporting (ceiling) RC beams	+ Net like	-	+ Local	+ Leakage, traces and stains of water and dust	Not measured
Cantilever slab	+ net like, not characteristic	-	+ Local	-	Not measured
Supporting wall (Masonry support walls made of stone)	-	-	-	+ Traces and dark stains of water	Not measured

Abutment wall (Masonry wall made of stone and covered by plastering)	+ Net like	+	-	-	+ Not important
Expasion joints				+ Leakage, traces and stains	

6. ALSHAAB PORT BRIDGE

The works on repair of Al Shaab Port Bridge started in November 2009 and ended in February 2010.

The first routine inspection was done in August 2016.

This routine inspection has included:

- Visual inspection of all available elements of RC structure of bridge, as well as others elements, such as fences, curbs, pedestrian paths, expansion joints, and asphalt wearing layers and
- Checking of depth of concrete/mortar cover carbonization.

The results of routine inspection are given below.

6.1. Visual inspection of available RC elements

The visual inspection has encompassed the next RC elements:

- Ribbed deck slab and
- Cantilever slabs,
- Abutment walls.

A general appearance of the bridge is shown in Fig IX-109 and Fig IX-110. It can be seen all visible parts of RC elements.

Figure (IX-109) A general view of Al shaab port bridge

Figure (IX-110) A general view of Al shaab port bridge

Deck slab

The condition of down side of deck slab is illustrated in Fig IX-111– Fig IX-112.

By the visual inspection no damages were spotted on down surface of deck slab, only traces of dust were noticed.

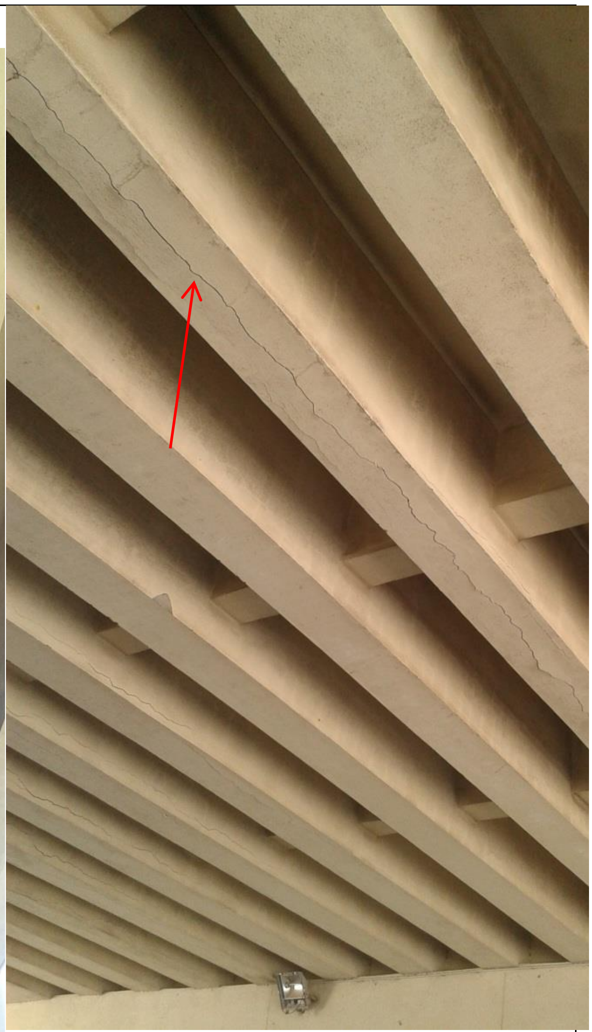


Figure (IX-111) A general view of deck slabs in superstructure of bridge	Figure (IX-112) A general view of deck slabs in superstructure of bridge

The condition of longitudinal and transverse deck ceiling beams is illustrated in Fig IX-113 –IX-114.

During of the visual survey the down part of superstructure was inspected. The following defects and damages are noticed:

- Longitudinal and transverse cracks (Fig IX-112 and IX-113)
- Spalling off of repair mortar (Fig IX-113)
- traces of dust (Fig IX-113 and IX-114)

Longitudinal cracks are characteristic damage of RC beams. They appeared on lateral and lower surfaces of longitudinal beams. Longitudinal cracks are very long and sometimes very large. They are caused by reinforcing bar corrosion.

The transversal cracks, are not characteristic damage, only few cracks of this type were spotted. They are very thin and short.

The spalling off of protecting mortar cover is also seen very rarely. They are shallow and cover small areas.

The longitudinal cracks are very serious damage due to reinforcement corrosion. For the evaluation of the degree of reinforcement corrosion the further investigation should be done. As the spalling off of protecting cover has not appeared yet, it is supposed that corrosion of reinforcement was not grabbed a large part of cross section of reinforcing bars. Other noticed damages are negligible. The changing of protecting cover and cleaning of reinforcing bars are necessary activities. The further investigation will show if an additional reinforcing bar are necessary.

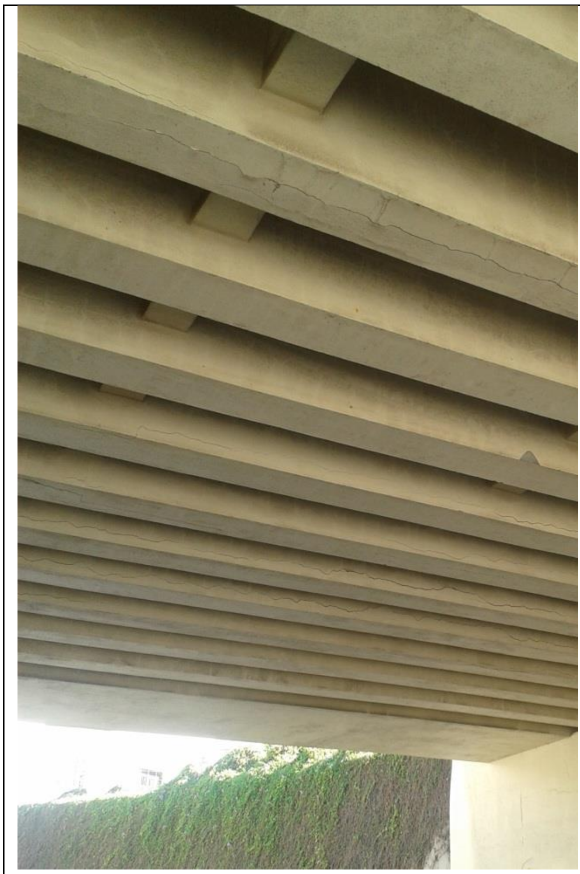


Figure (IX-113) Longitudinal and transversal supporting beams, longitudinal and transverse cracks in longitudinal beams, spalling off, traces of dust	Figure (IX-114) Longitudinal and transverse cracks in longitudinal beams

The transversal beams have a roll of secondary girders. During visual inspection no damages was spotted in them.

Cantilever slab

The view of Cantilever slab is illustrated in Fig IX- 115 and IX-116.

By the visual inspection the following damages are noticed on cantilever slabs:

- Local spalling off of mortar and (Fig IX-115) and
- Vertical and horizontal cracks (Fig IX-116),
- Traces of water (Fig IX-115).

Figure (IX-115) A view of cantilever slab lateral and lower part (north side)

Figure (IX-116) A view of cantilever slabs (south side)

Traces and stains of water are characteristic for whole side and down surface of cantilever slabs. They occurred due to bad drainage of rain water. (Fig IX-115 and IX-116).

All types of cracks were seen on side surface of cantilever slab. The upper horizontal cracks were appeared due to corrosion of longitudinal reinforcing bar and followed by traces of water and rust (Fig IX-116). Other horizontal cracks are shorter and probably caused by reinforced bar corrosion (this conclusion is driven on the base of arrangement of horizontal cracks; they are located at the same vertical distance) or by drying shrinkage.

Vertical cracks are, also, characteristic damage on side surface of cantilever slab. They were caused by drying shrinkage of concrete and/or by corrosion of reinforcement (Fig IX-115 and IX-116).

Vertical and horizontal cracks are followed with traces of rust.

Spalling off of concrete is very local, shallow and it was spotted on side surfaces (Fig IX-116).

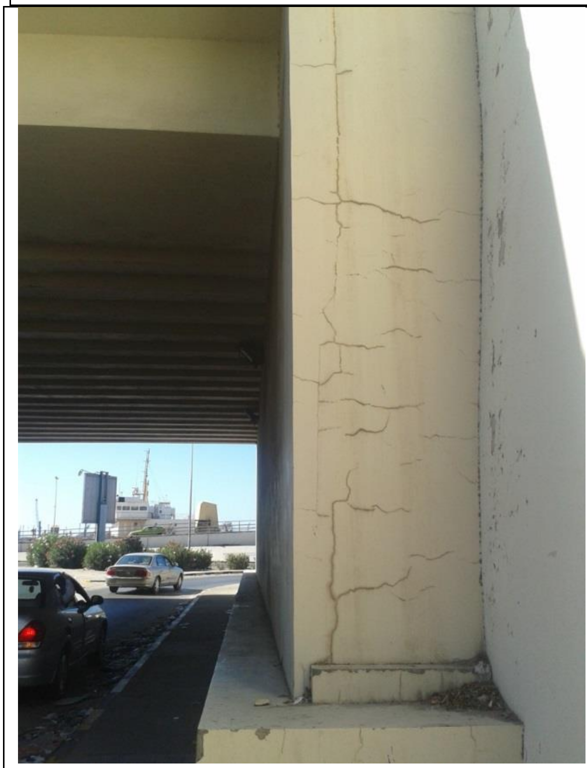
As it was mentioned, the part of horizontal and vertical cracks are caused by reinforcement corrosion, it is suggested to change the whole protective cover on lateral surface and to clean all corroded reinforcing bars.

Abutments

In the original design of bridge, the abutment had been designed as masonry stone structures, but in repair design of this bridge the designers suggested the strengthening of these walls by execution of new additional RC layer. The condition of abutments is illustrated in Fig IX-117– Fig IX-120. During visual inspection, the following damages were noticed:

- Net like, vertical and horizontal crack (Fig IX-117 – IX- 120)
- Spalling off of mortar (Fig IX-119)
- Traces and stains of water (Fig IX-118 and IX-120)

Figure (IX-117) View of abutment with ribbed deck slab	Figure (IX-118) View of abutment, west side



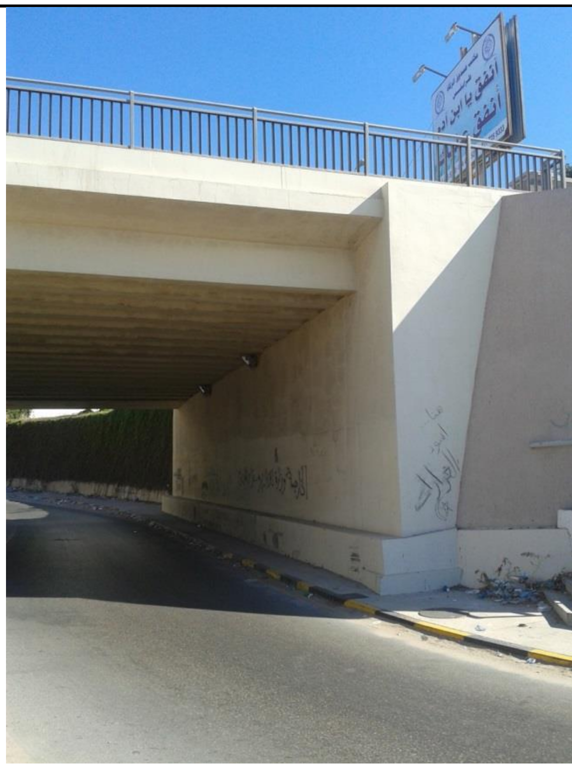


Figure (IX-119) View of lateral side of the abutment, west side	Figure (IX-120) View of abutment, east side

It is supposed that the vertical and horizontal cracks on longitudinal sides of abutments are caused by reinforcement corrosion. Vertical cracks are more pronounced, and they located on the places of vertical rebars. Some vertical and horizontal cracks are uneven or net like and they are caused by drying shrinkage of repair mortar. On lateral sides of abutments also horizontal and vertical cracks were registered. The horizontal cracks are predominant. It is supposed that they are caused by drying shrinkage of repair mortar.

On the basis of visual inspection results it can be concluded that corrosion process of reinforcement has already start and some measures like impregnation of repair mortar is suggested to delay development of steel corrosion.

6.2. Visual inspection of other bridge elements

Pedestrian Path

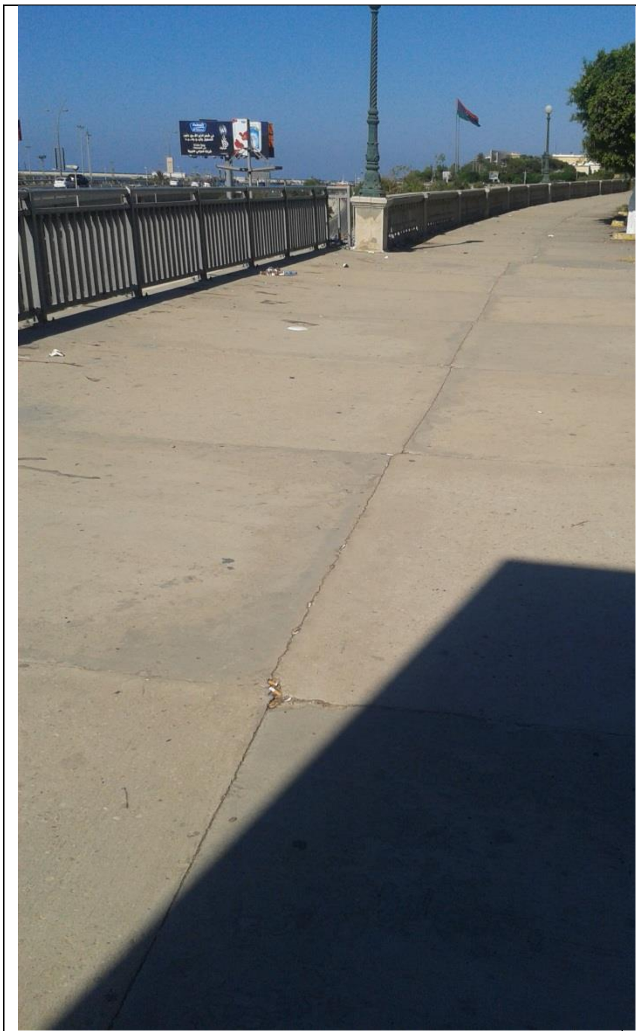
Through visual inspection, several types of damages were spotted:

- Uneven cold joints
- Local surface pits and
- Missing part of concrete.

Figures IX-121 and IX-122 illustrate the condition of the pedestrian path.

It was observed that the cold joints are very uneven, and they look like cracks (Fig IX-121-IX-122).

On concrete surfaces between cold joints the shallow pits and erosion caused wearing were spotted. It is supposed that they were caused by mechanical action.



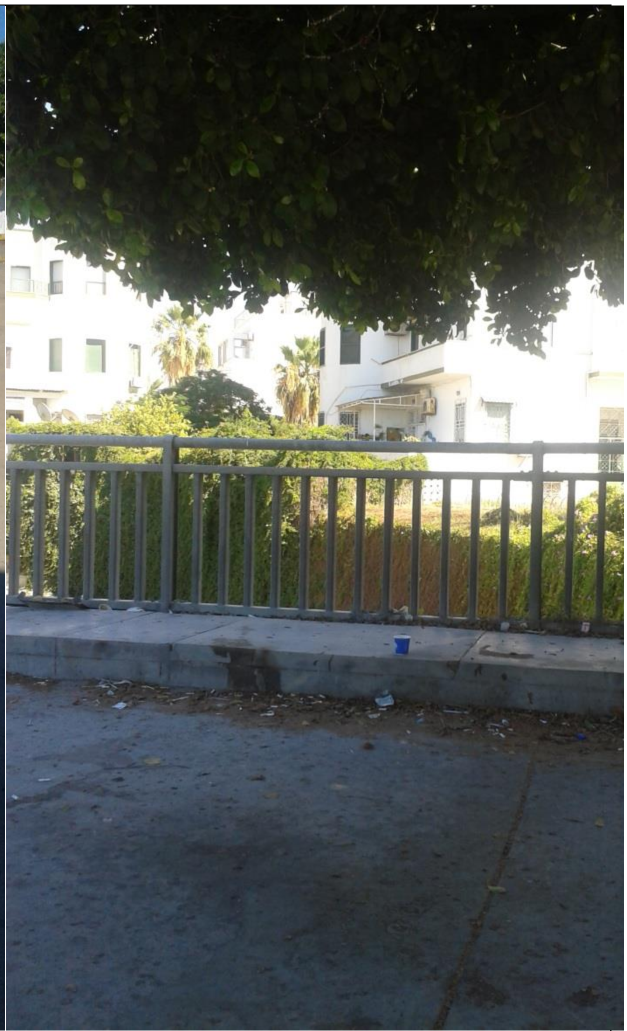


Figure (IX-121) A general view of pedestrian path on bridge (north side)	Figure (IX-122) A general view of pedestrian path on bridge (south side)

Asphalt wearing layer

The characteristic damages of wearing layer of traffic lanes are transversal cracks. The cracks are dashed with wideness of several mm.

The presence of dust and communal rubbish on the road made visual inspection more difficult and hidden a certain amount of damages.

The disappearance of traffic signs, including intermittent lines that allow the vehicle to cross, and the side lines, were noticed.

The condition of asphalt wearing layer is illustrated in Fig IX-123 and Fig IX-124.

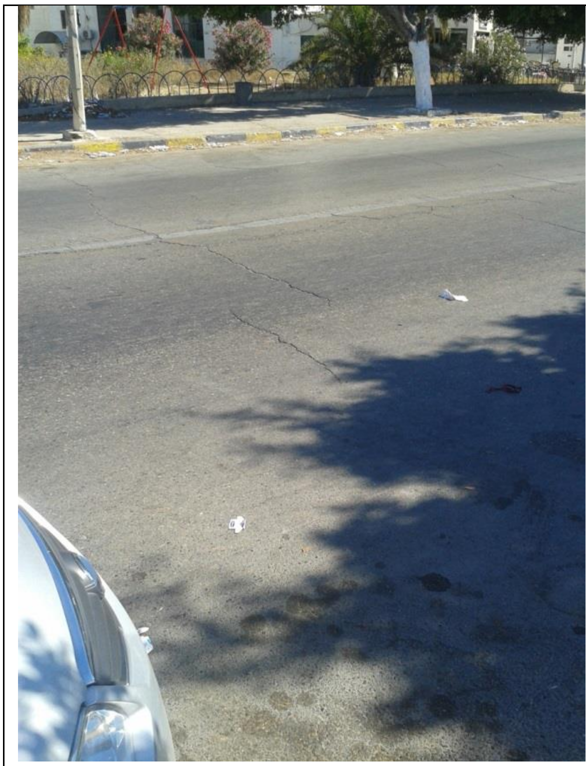


Figure (IX-123) A general view of asphalt wearing layer

Figure (IX-124) A general view of asphalt wearing layer

Fences

The fences are in good condition and there is no damage. The condition of fences is illustrated in Fig IX-125, IX-126.

Curbs

some minor cracks were spotted.

The condition of curbs is illustrated in Fig IX-127 – Fig IX-128.

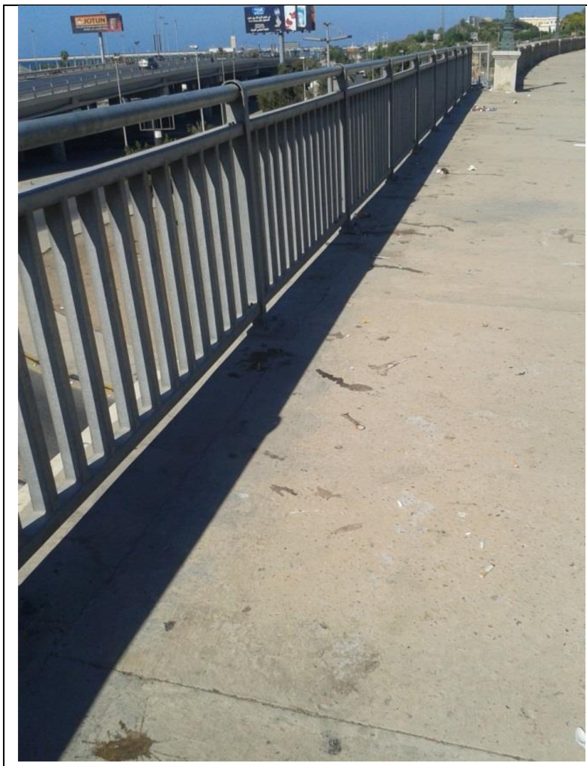


Figure (IX-125) A general view of fences on the bridge (north side)

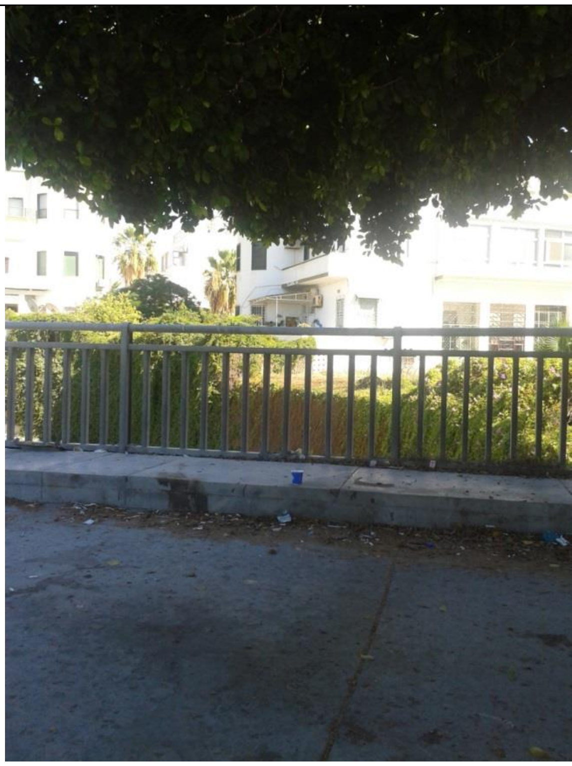


Figure (IX-126) A general view of fences on the bridge (south side)

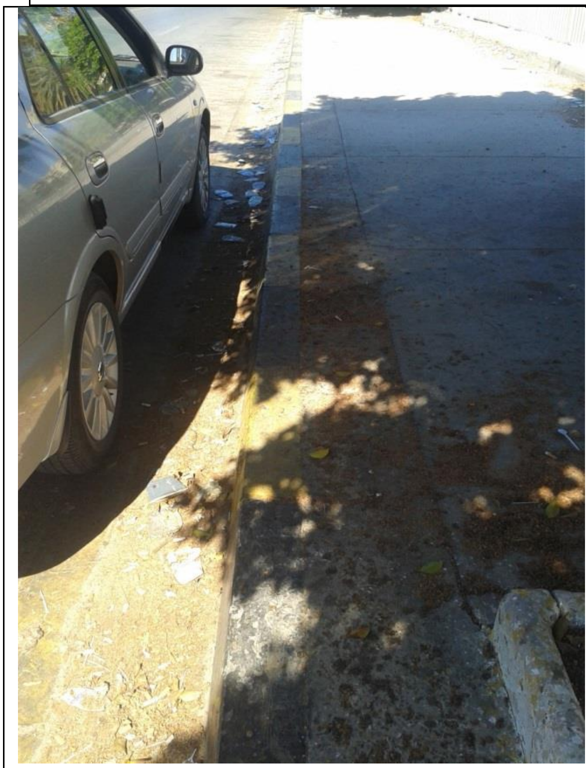


Figure (IX-127) A general view of curbs under the bridge

Figure (IX-128) A general view of curbs

Catch pit

The blockage of drainage channels under the bridge with garbage, dust and sand were noticed. Due to that the rainwater does not be drainage.

The condition of catch pit is illustrated in Fig IX-129.

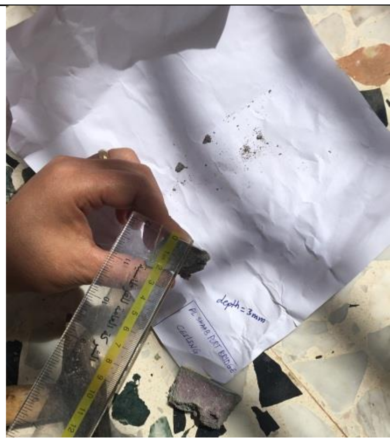
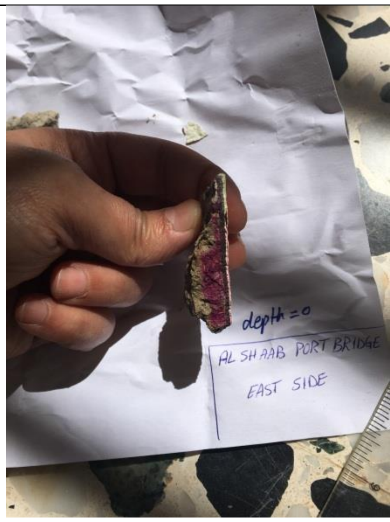
Figure (IX-129) A general view of catch pit under the bridge

Depth of carbonation

The extent of carbonation on was assessed by treating the fresh broken part of repair material with phenolphthalein indicator. The carbonation test were done on following RC elements: ceiling and abutments (west and east side)

The obtained results are shown in the table IX-11.

Table (IX-11) Results of measuring the depth of carbonation on RC elements

Elements	description	Depth of carbonation (mm)	
ceiling	Repair material – mortar	3	
Abutment (west side)	Repair material – mortar	4	
Abutment (east side)	Repair material – mortar	0	

By the analyzing obtained results it is concluded that process of carbonation has already started on almost all RC elements. Almost the same value of carbonation was measured in abutment (west side) (4mm) and in deck ceiling slab. These values are expected. The rate of carbonation is usually 0,5mm/year and with that speed the depth of carbonation would be about 3,5mm.

6.3. General conclusion

The first routine inspection of The Al Shaab Port Bridge was done after 6 years of its repair. By the analysing of collected results, which were obtained by visual inspection and measuring depth of carbonation, the next conclusions are derived:

- In the original design of bridge, the abutment had been designed as masonry stone structures, but in repair design of this bridge the designers suggested the strengthening of these walls by execution of new additional RC layer. The characteristic damage of the abutment walls are vertical and horizontal cracks. The vertical cracks are predominant on longitudinal sides of abutment walls, but on lateral surfaces the horizontal crack are pronounced. It is supposed that the main reason for vertical and horizontal cracks appearance on longitudinal surface of the walls is reinforcement corrosion. The second reason for their appearance is drying shrinkage. Characteristic horizontal cracks on lateral sides of the walls are caused by drying shrinkage. The corrosion of wire mash may reduce the bearing mechanical resistance of abutment walls.
- All visible part of deck slabs of bridge superstructure are in good condition. No damages were spotted on down surface of deck slab, only traces of dust were noticed.
- The characteristic damages of longitudinal supporting beams are longitudinal cracks. They were appeared on lateral and lower surfaces of longitudinal beams. Longitudinal cracks are very long and sometimes very large. They are caused by reinforcing bar corrosion. The longitudinal cracks are very serious damage due to reinforcement corrosion and potential reduction of bearing capacity. The other noticed damages, such as transverse cracks and spalling off of mortar are negligible. Except the potential reduction of bearing capacity, the durability is reduced in a certain way. The transversal beams are not damaged.
- The characteristic damages of cantilever slabs are horizontal and vertical cracks on their side surfaces. Both types of cracks are caused by reinforcement corrosion or by drying shrinkage. Only local spalling off of mortar was noticed. Since a certain number of horizontal and vertical cracks are generated due to the corrosion of reinforcing bars, the adequate repair measures should be undertaken. The bearing capacity of cantilever slab has not been jeopardized yet, since steel corrosion caught up subsidiary reinforcement.
- The initial damages have occurred on pedestrian path (uneven cold joints and a local surface pits)
- The characteristic damages of wearing layer of traffic lanes are transversal cracks.

- Due to the lack of maintenance, the presence of dust and urban rubbish on the road were noticed. Also, traffic signs, including intermittent lines, that allow the vehicle to cross, and the side lines have been disappeared, and they have to be refreshed.
- The metal bridge fence is in good condition.
- The curbs are in good conditions. The local peeling off black and yellow colour was spotted as well as a minor cracking.
- The blockage of drainage channels under the bridge with garbage, dust and sand were noticed. Due to that the rainwater does not be drainage
- The carbonation has already started in main supporting beams of deck ceiling slab and abutments. The largest measured value of carbonation was 4mm. This value is in the range of expected.

Finally, the stability, bearing capacity and functionality and have not been jeopardized, yet. Since the process of reinforcement corrosion was registered in all RC elements the problem of bearing capacity might be actual very soon. Since a lot of cracks have been registered, the durability of whole structure is reduced. Some radical repair measures are suggested, such as removing protecting cover, cleaning of and protection of rebars and execution of new cover on whole visible part of longitudinal supporting beams, side surfaces of cantilever slabs and longitudinal surfaces of abutment walls.

Table (IX-12) Review of registered damages during the first routine inspection

RC element	Cracks	Pilling off protecting paint	Spalling off	Water leakage/traces	Carbonation
Deck ceiling - Longitudinal supporting beams	+ Longitudinal and transverse cracks	-	+	+ traces of dust	+
Cantilever slabs Side surface	+ Vertical and horizontal cracks	-	+ Local	+ traces of water	Not measured
Abutment Additional RC layer	+ vertical and horizontal cracks	-	+ Local	+ Traces and stains of water	+

7. ABDUL SALAM AREF BRIDGE

The works on repair of Abdul Salam Aref Bridge started in November 2009 and ended in March 2010.

The first routine inspection was done in August 2016.

This routine inspection has included:

- Visual inspection of all available elements of RC structure of bridge, as well as others elements, such as fences, curbs, pedestrian paths, expansion joints, guardrails and asphalt wearing layers and
- Checking of depth of concrete/mortar cover carbonization.

The results of routine inspection are given below.

7.1 Visual inspection of available RC elements

The visual inspection has encompassed the next RC elements:

- Supporting columns,
- Abutments,
- Main longitudinal and secondary transverse deck ceiling beams,
- Deck slab (only down part) and
- Beam above columns.

A general appearance of the bridge is shown in Fig IX-130 and Fig IX-131. It can be seen that all visible parts of RC elements were paint. Thus, the repair material could not be seen during inspection.

	
Figure (IX-130) A general view of ASAB	Figure (IX-131) A general view of ASAB

Supporting columns

The condition of supporting columns is illustrated in Fig IX-132 – Fig IX-135.

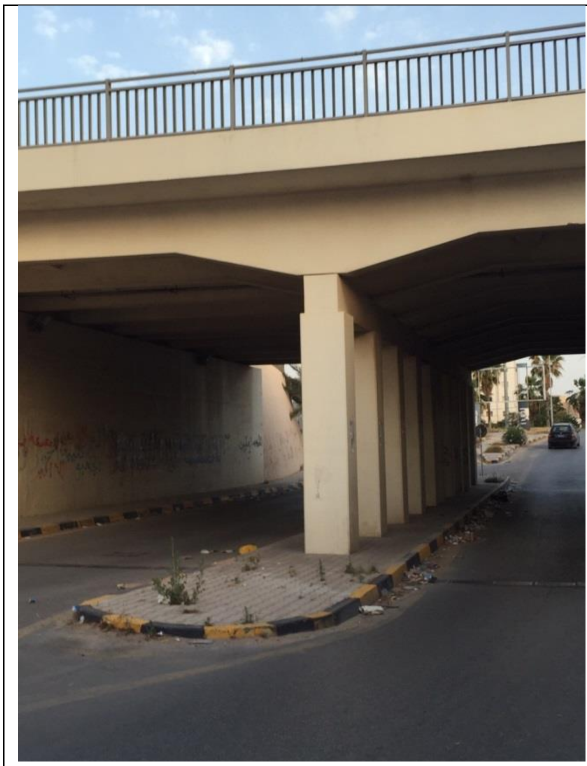


Figure (IX-132) A View of columns which were strengthened by enlarging cross sections	Figure (IX-133) Undamaged concrete surface of strengthened column

	
Figure (IX-134) A View of strengthened columns	Figure (IX-135) A View of strengthened columns

On the basis of result of visual inspections, it can be concluded that columns are in excellent conditions.

No cracks, neither crumbling of concrete nor falling off of concrete cover have been spotted.

Abutments

The condition of abutments is illustrated in Fig IX-136 – Fig IX-139.

Figure (IX-136) View of strengthened abutment

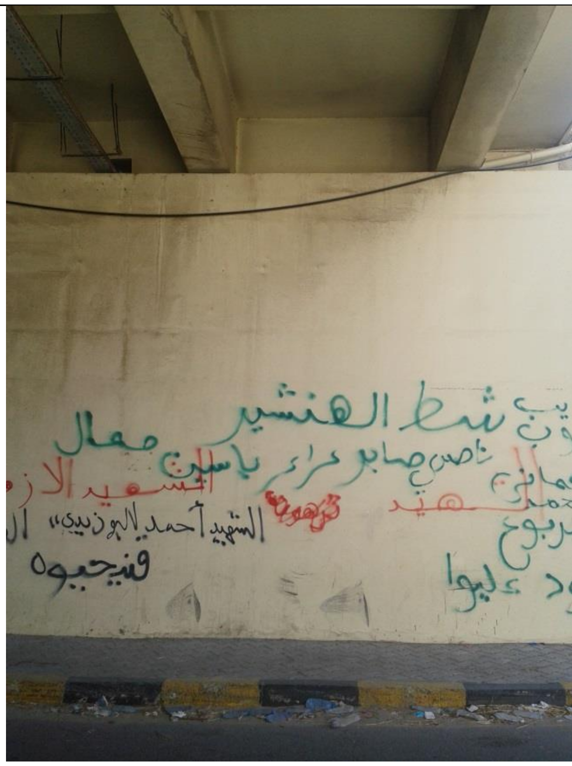


Figure (IX-137) Uneven concrete surface of abutment

	
Figure (IX-138) Look of expansion joint between two abutments	Figure (IX-139) Uneven concrete surface of an abutment, in detail

By the visual inspection the following defects and damages are noticed:

- Very uneven concrete surface (Fig IX-137 and IX-139)
- Peeling of protecting painting (Fig IX-136) and
- Cracks

Very uneven concrete surface is characteristic for all visible parts of abutments. This defect may reduce thickness of cover in some locations, but up to the period of visual inspection no damages due to reduction of cover have been noticed on any abutments.

Traces and stains of water have been noticed on the surface of down part of abutments. On the surface of the down part of abutments they have been caused by vehicles during a rain.

Peeling of protecting painting is characteristic for down part of abutments, close to pavement.

Cracks are not characteristic damage. During inspection only a few cracks have been spotted.

On the basis of those descriptions, it can be concluded that all abutments are in good condition. Registered damages are very small and have local character.

Main longitudinal and secondary transverse deck ceiling beams

The condition of main longitudinal and secondary transverse deck ceiling beams is illustrated in Fig IX-140 – Fig IX-143.

	
Figure (IX-140) A general view of main and secondary beams of superstructure of bridge	Figure (IX-141) A general view of main and secondary beams of superstructure of bridge

	
Figure (IX-142) Main beams: The transverse thin cracks on side and down surfaces of beams	Figure (IX-143) Supporting part of main beams, a longitudinal crack in haunch; Traces of water leakage on secondary beam

During of the visual survey the down part of superstructure was inspected. The following defects and damages are noticed:

- Traces and stains of water (Fig IX-143)
- Cracking of new cover (Fig IX-140, IX-142 and IX-143)

Traces and stains of water have been noticed on the side surfaces of several beams. They are caused by leakage of water through expansion joints in super structure of bridge, or by some problems in hydro-insulation layer.

Cracks are characteristic damage. During inspection two types of cracks were registered:

- Cracks along the main reinforcing bars near the edge of beams (longitudinal cracks) and
- Transversal cracks on side and down surfaces of beams

Only a few longitudinal cracks, bat o lot of transversal cracks have been noticed. Both types of cracks are very thin (dominantly 0, 1 mm, only a few up to 0,2mm).

On the basis of given descriptions, it can be concluded that all visible part of beams of bridge superstructure are in good condition. Registered cracks are very thin and have not reduced durability of those elements, yet.

Deck slab

The condition of deck slab is illustrated in Fig IX-144– Fig IX-145). Nether net like cracks nor water stains had been spotted on visible parts of down surface of deck slab. Only, longitudinal crucks were seen on the contact between main and secondary beams and slabs, because of uneven change of geometry (Fig IX-145)

Figure (IX-144) A general view of down part of deck slab in superstructure of bridge	Figure (IX-145) A general view of down part of cantilever of superstructure of bridge

Figure (IX-146) A general view of expansion joint between deck slabs in superstructure of bridge	Figure (IX-147) View of characteristic part of expansion joints

A general view of expansion joint between two deck slabs is shown in figure 9.146. Thin longitudinal cruck was spotted by the visual inspection of expansion joint between decks slabs, but without any traces of water leakage. Only, local spalling off infill materials had been seen (figure IX-147).

7.2. Visual inspection of other bridge elements

Pedestrian path

By the visual inspection it was noticed that cold joints are very rough and cracked. Also, longitudinal cracks and surface peeling off of thin concrete layer have been appeared in parts between cold joints.

The condition of Pedestrian path is illustrated in Fig IX-148.

It could be concluded that initial damages have occurred on pedestrian path surface and some prevention measures for slowing down appearance of new and progress of numbered damages should be undertaken.

Asphalt wearing layer

The characteristic damages of wearing layer of traffic lanes are transversal crucks. The crucks are dashed with wideness of several mm.

The growth of some plants between the two lanes, as well as, the presence of dust on the road and the disappearance of traffic signs, including intermittent lines that allow the vehicle to cross, and the side lines, were noticed.

The condition of asphalt wearing layer is illustrated in Fig IX-149.

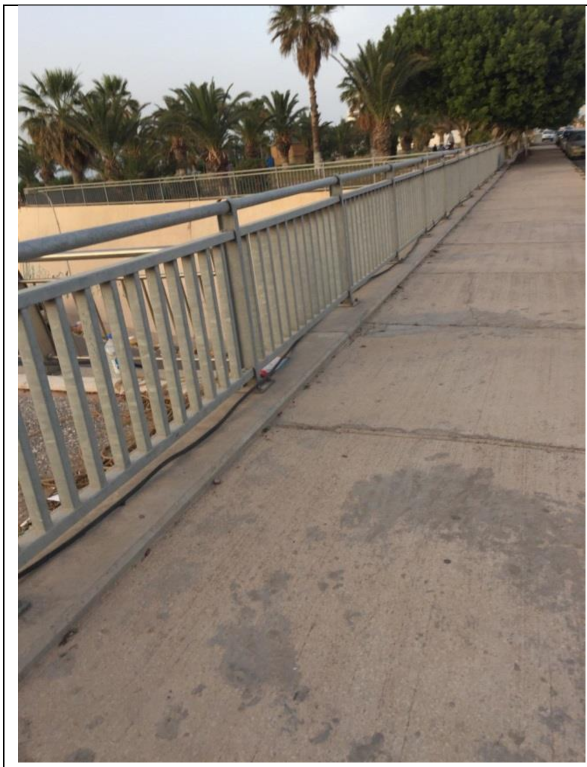


Figure (IX-148) A general view of Pedestrian path on bridge	Figure (IX-149) A general view of Asphalt wearing layer

Fences

The fences are in good condition and there is no damage. The condition of fences is illustrated in Fig IX-150, IX-151.

Curbs

Some minor cracks as well as having dirt near the curbs were spotted.

The condition of curbs is illustrated in Fig IX-152– Fig IX-153.

Figure (IX-150) A general view of fences on the bridge

Figure (IX-151) A general view of fences

Figure (IX-152) A general view of curbs under the bridge

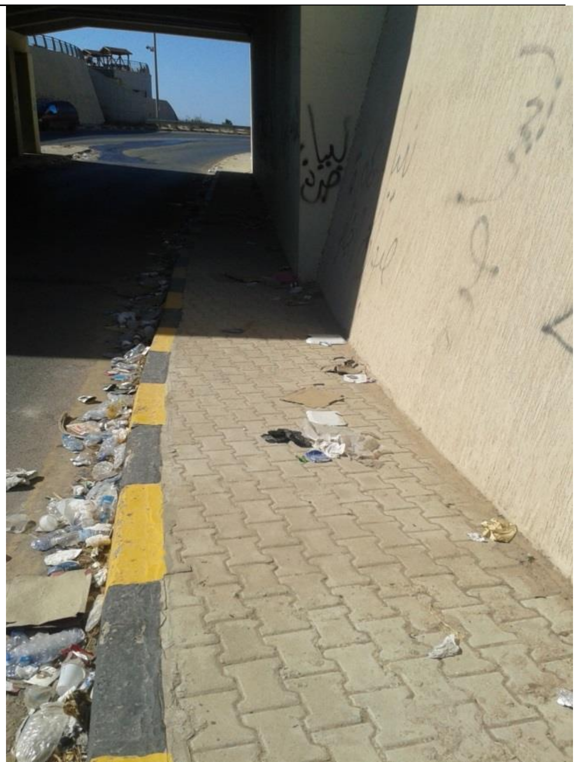


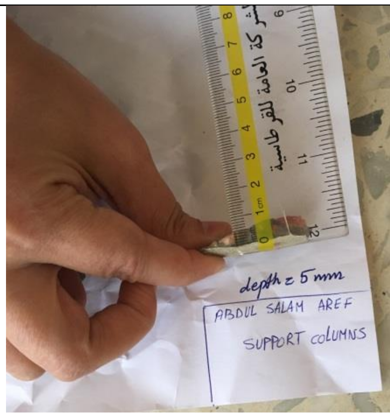
Figure (IX-153) A general view of curbs

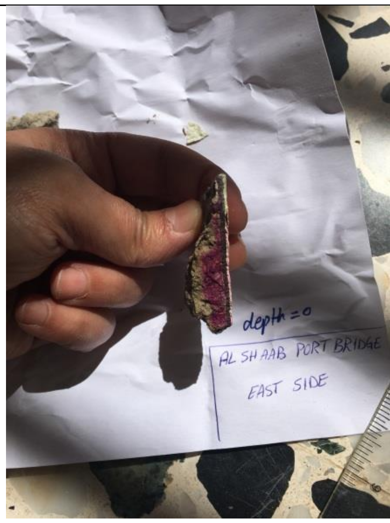
Depth of carbonation

The extent of carbonation was assessed by treating the fresh broken part of repair material with phenolphthalein indicator. The carbonation test were done on following RC elements: support column, ceiling and abutments (west and east side)

The obtained results are shown in the table IX-13.

Table (IX-13) Results of measuring the depth of carbonation on RC elements

Elements	description	Depth of carbonation (mm)	
Support column	Repair material – mortar	5	
ceiling	Repair material – mortar	9	

Abutment (west side)	Repair material – mortar	2	
Abutment (east side)	Repair material – mortar	0	

By the analysing obtained results it is concluded that process of carbonation has already started on almost all RC elements. The largest value of carbonation was measured on ceiling (9mm). This value is almost double than expected. The rate of carbonation is usually 0,5mm/year and with that speed the depth of carbonation should have been about 3,5mm. Some measures to slow down the rate of carbonation should be considered.

7.3. General conclusion

The first routine inspection of The Abdul Salam Aref Bridge was done after 6 years of its repair. By the analysing of collected results, which were obtained by visual inspection and measuring depth of carbonation, the next conclusions are derived:

- RC columns and abutments are in excellent condition. No cracks, crumbling of concrete nor falling off of concrete cover have been spotted. Only very small areas with peeling of protecting painting and few thin cracks were registered on surface of abutment.

- All visible part of beams and deck slabs of bridge superstructure are in good condition. Registered cracks are very thin and have not reduced durability of those elements, yet.
- The expansion joint between deck slabs is cracked, but does not have traces of water leakage. Only, local surface spalling off infill materials had been seen. The expansion joint between abutments is in good condition.
- The initial damages have occurred on pedestrian path (crucks and pilling off of concrete surface)
- The characteristic damages of wearing layer of traffic lanes are transversal crucks. The crucks are dashed with wideness of several mm.
- Due to the lack of maintenance the growth of some plants between the two lanes and in drainage system, as well as, the presence of dust and urban rubbish on the road were noticed. Also, traffic signs, including intermittent lines, that allow the vehicle to cross, and the side lines have been disappeared.
- Bridge fence and curbs are in good conditions, also.
- Carbonation has already started on almost all RC elements. The largest value of carbonation was measured on ceiling (9mm).

Finally, the stability, bearing capacity, functionality and durability have not been jeopardized, yet. As it was mentioned, a few damages were spotted on the surface of inspected RC elements. All damages are in the initial state and could be slowed down by some measures like impregnation. The same measures are suggested for RC elements caught by carbonation.

The review of registered damages during routine visual inspection is presented in table IX-14.

Table IX-14 Review of registered damages during the first routine inspection

RC element	Cracks	Pilling off protecting paint	Spalling off	Water leakage/traces	Carbonation
Support column	-	-	-	-	+ (In progress)
Deck ceiling	+ (On contact with beams)	-	-	+ (Cantilever part)	+ (In progress)
Main and secondary deck cilling beams	+ (Few longitudinal, a lot of trasversal), thin	-	-	+ (Traces, on side surface of external beams)	Not measured
Abutment	+	+	-	+ (traces)	+ (In initial phase)
Expansion joint	+ (longitudinal)		+ (local)	-	

CHAPTER X

Rating and ranking of bridges 6 years after repair

CHAPTER X

Rating and ranking of bridges 6 years after repair

INTRODUCTION

Major inspections involve visual inspection and testing (material investigations) of all parts of a structure.

Damage and condition assessment are performed according to Germany methodology. Directive for Uniform Determination, Assessment, Recording, and Analysis of the Results of the Inspection of the Structures.

In this chapter, seven bridges in Libya were evaluated according to the German methodology, and all the damages in each bridge were counted. And knowing which bridge has a lot of damage and needs maintenance first.

This assessment of the condition of the bridges was in 2016.

1. SOUK ATHULATHA 1 BRIDGE

1.1. LEVEL 1: REGULAR BRIDGE INSPECTION

Group 1: Superstructure

Description of damage and defects	S	V	D	
Bridges, concrete / reinforced concrete / pre-stressed construction structures				
Dependencies: type of construction = bridge, basic building element = structure construction, material of the spanning structure = concrete				
Graffiti on visible surfaces	0	0	0	$Z_{1.1.1}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{1.1.2}$
Less wear of the protective layer	0	0	1	$Z_{1.1.3}$
The rust on the lower sides of the construction	x	x	x	
Pollution of internal passages of the building (remains of the formwork or other)	x	x	x	
Pollution of internal passages of building (bird feces or other)	x	x	x	
Coarse granularity of concrete of the spanning structure	0	0	1	$Z_{1.1.4}$
The protective layer above the auxiliary rebar for the installation of the main rebar is too small	0	0	1	$Z_{1.1.5}$
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) poor quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1.0 to 2.9cm) Good quality of concrete	0	0	2	$Z_{1.1.6}$
The protective layer of the main rebar on the underside of the structure is too small (1,0 to 2,9cm) Poor concrete quality	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (below 1.0cm)	x	x	x	
The carbonate front reached the main rebar	x	x	x	
Visible main rebar on the underside of the structure, the reinforcement is lightly corroded (without significant reduction of the cross section)	x	x	x	
The main rebar of the spanning structure lies in the area of carbonization and is slightly corroded (it does not apply to the prefabrication reinforcement)	x	x	x	
Visible main rebar on the underside of the spanning structure, corroded rebar (there is a decrease in the cross-section)	x	x	x	
Blooming (water traces) in the area of heavily corroded main reinforcement on the underside of the spanning structure (advanced reductions in cross-section)	x	x	x	
Blooming in the area of heavily corroded main reinforcement on the underside of the spanning structure (the main reinforcement is partly excluded from the load)	x	x	x	
Partial moisture penetration	0	0	2	$Z_{1.1.7}$
Penetration of moisture on large surfaces	x	x	x	
Description of damage / defect	s	v	d	
Bridges, cracks in concrete / reinforced concrete / pre-stressed structure				
Dependencies: type of construction = bridge, basic building element = structure, material of the structure = concrete, damage = cracks				
Surface cracks outside the humidification area (widths) of ≤ 0.1mm in reinforced concrete or prestressed structure (eg cracks from shrinkage)	x	x	x	
Cracks outside the area of humidification (shrinkage) width of 0.1 - <0.2mm in reinforced concrete- or prestressed structure	x	x	x	
Cracks width 0.1 - <0.2mm in the area of humidification (shrinkage) for RC structure	x	x	x	

Surface cracks in the humidification area (widths) 0.2 -≤ 0.4 mm in the RC structure	x	x	x	
Parallel cracks with prestressing of a width of 0.2 -≤ 0.4mm in the area of humidification (squeezing) in the prestressed structure	x	x	x	
Shrinkage widths> 0.4mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (not in the cable extension area)	x	x	x	
Cracks with a width of <0.2mm with prestressed structure (in cable extension)	x	x	x	
Cracks with a width of 0.2 - ≤ 0.4mm at the front of the structure (in the area of cable extension)	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (in the cable extension area)	x	x	x	
Cracks> 0.4mm under load	x	x	x	

Summary Group 1

$Z_{1.1.1} = 1.0$	$\Delta Z_{1.1.1} = 0$	$Z_{1.1.1} = 1.0+0=1.0$
$Z_{1.1.2} = 1.0$	$\Delta Z_{1.1.2} = +0.1$	$Z_{1.1.2} = 1.0+0.1=1.1$
$Z_{1.1.3} = 1.1$	$\Delta Z_{1.1.3} = -0.1$	$Z_{1.1.3} = 1.1-0.1= 1$
$Z_{1.1.4} = 1.1$	$\Delta Z_{1.1.4} =- 0.1$	$Z_{1.1.4} = 1.1-0.1= 1$
$Z_{1.1.5} = 1.1$	$\Delta Z_{1.1.5} =- 0.1$	$Z_{1.1.5} = 1.1-0.1= 1$
$Z_{1.1.6} = 2.0$	$\Delta Z_{1.1.6} =- 0.1$	$Z_{1.1.6} = 2.0-0.1= 1.9$
$Z_{1.1.7} = 2.0$	$\Delta Z_{1.1.7} = 0$	$Z_{1.1.7} = 2.0+0= 2.0$
Sum group 1		**9.0**

Group 2: Substructure

Bridges, substructure	S	V	D	
Dependencies: type of construction = bridge, basic building element = substructure				
Graffiti on visible surfaces	0	0	0	$Z_{1.1.8}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{1.1.9}$
Less wear of the protective layer	0	0	1	$Z_{1.1.10}$
Less rinses in the area of water flows	0	0	1	$Z_{1.1.11}$
Cleaning of the bearing bench (moldings or other)	x	x	x	
Invalidation of the benches (bird droppings or other)	x	x	x	
Remains of the formwork that press the construction	x	x	x	
Cleaning the bearing bench with accumulated moisture	x	x	x	
Formwork material (polystyrene) on the connection with the structure has not been removed	x	x	x	
Less dropping of stone linings	x	x	x	
The installation cover is not correct / damaged	x	x	x	
Cracking stone wall	x	x	x	
Partial moister on stone wall / reinforced concrete	0	0	2	$Z_{1.1.12}$
Moisture on large surfaces of stone wall / reinforced concrete	x	x	x	
Bridges, cracks in concrete- / RC substructure				

Dependencies: type of construction = bridge, basic building element = substructure, damage = cracks				
Dry cracks outside the humidification (spinning) area <0.2mm (no reaction sulfuric acid - RSK)	x	x	x	
Surface cracks in the area of humidification (squeezing), cracks can run water, width <0.2mm (without RSK) REACTION SULFURIC ACID	x	x	x	
Dry cracks outside the humidification area (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, 0.2 -≤ 0.4mm wide (without RSK)	x	x	x	
Dry cracks outside the area of humidification (shrinkage) of width> 0,4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) of width> 0.4mm (without RSK)	x	x	x	
Description of damage / defect	s	v	d	
Cracks in the area of humidification (cracks), cracks can run water, width> 0,4mm, unarmed concrete (without RSK)	x	x	x	
Cracks in the area of humidification (sprinkling), cracks can run water, width> 0.4mm, RC bottom structure (without RSK)	x	x	x	

Summary Group 2

$Z_{1.1.8} = 1.0$	$\Delta Z_{1.1.8} = 0$	$Z_{1.1.8} = 1.0+0= 1.0$
$Z_{1.1.9} = 1.0$	$\Delta Z_{1.1.9} = +0.1$	$Z_{1.1.9} = 1.0+0.1= 1.1$
$Z_{1.1.10} = 1.1$	$\Delta Z_{1.1.10} = -0.1$	$Z_{1.1.10} = 1.1-0.1= 1$
$Z_{1.1.11} = 1.1$	$\Delta Z_{1.1.11} = 0$	$Z_{1.1.11} = 1.1+0= 1.1$
$Z_{1.1.12} = 2.0$	$\Delta Z_{1.1.12} = -0.1$	$Z_{1.1.12} = 2.0-0.1= 1.9$
Sum group 2		**6.1**

Group 9: Transition devices-joints

Transition devices (joints)	S	V	D	
Dependencies: structural element = transitional device				
Contaminated transitional device (moving still possible)	x	x	x	
A highly Contaminated transient device (scrolling limited)	x	x	x	
Loosening the fixing of the profiles in the carpet of the structures, the profile is still held	x	x	x	
Loosening the fixing of the profiles in the carpet construction, the profile loosened	x	x	x	
Rubber profile dropped or multiple damaged	x	x	x	
Asphalt crossings, thin asphalt mass (rough, small open cracks)	x	x	x	
The asphalt crossing cracked and depressed	x	x	x	
The transient device is missing, the spanning structure is cracked at the ends	x	x	x	
The transient device is missing, the spanning structure at the ends is cracked and shrugged	x	x	x	

Summary Group 9

Sum group 9	0

Group 13: Fence

Protective means	S	V	D	
Fence				
Dependencies: element of construction = protective means, fence				
The compound fence - construction of the defect	x	x	x	
There is no fence, there are bumpers at> 50km / h, no pedestrian traffic is planned	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, small building, no pedestrian traffic is foreseen (temporary hazard sign exists)	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, pedestrian traffic planned	x	x	x	
Missing wire in the handrail of the fence, the length of the building is <20m	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, there is a bumper	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, the bumper is missing	x	x	x	
Fence height is not in accordance with regulations (difference ≤ 5cm)	x	x	x	
Fence height is not in accordance with regulations (difference 5 - 10cm)	x	x	x	
Fence height is not in accordance with regulations (difference > 10cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference ≤ 2cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference > 2cm)	x	x	x	
Missing individual fence filling rods	0	2	0	$Z_{1.1.13}$
Missing more consecutive fence filling rods	x	x	x	
Guardrail				
Dependencies: structural element = protection agent, bumper				
Bumper height is not in accordance with regulations (difference ≤ 3cm)	x	x	x	
Bumper height is not in accordance with regulations (difference > 3cm)	x	x	x	
The bumper is partially deformed	x	x	x	
a fence and a bumper are missing	x	x	x	
Weak anchor fasteners of the bumper / fence	x	x	x	
There is a lack of anchoring of protective devices on the length of the move	x	x	x	
Corrosion of protective agents				
Dependencies: structural element = protective agents, damage = surface, metal				
Insufficient thickness of the protective layer	x	x	x	
Local scattering (breaking) of the protective layer	x	x	x	
Scattering (breaking) of the protective layer on larger surfaces	x	x	x	
Local start of corrosion	x	x	x	
Corrosion of large surfaces	x	x	x	
The corrosion of individual support elements of the protecting agents	x	x	x	
The corrosion of more consecutive support elements of the protecting agents	x	x	x	

Summary Group 13

$Z_{1.1.13}$ = 2.0	$\Delta Z_{1.1.13}$ = -0.1	$Z_{1.1.13}$ = 2.0-0.1= 1.9
Sum group 13		**1.9**

Group 11: Road surface

Road surface	S	V	D	
Dependencies: structural element = useful surface				
pavement				
Subsidence of the pavement behind the abutments (≤ 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm), there are warning signs	x	x	x	
Drainage does not work, the risk of drifting	0	2	0	$Z_{1.1.14}$
Paving grooves / indentations, depth <1cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm, there are warning signs	x	x	x	
Pulley grooves / indentations, depth> 3cm	x	x	x	
Paving grooves / indentations, depth> 3cm, there are warning signs	x	x	x	
Bubbles, heights of ≤ 2cm	x	x	x	
Bubbles, height 2 - 5cm	x	x	x	
Bubbles, height 2 - 5cm, there are warning signs	x	x	x	
Bubbles, height > 5cm	x	x	x	
Bubbles, height > 5cm, there are warning signs	x	x	x	
Impact hole, depth ≤ 2cm	x	x	x	
Impact hole, depth 2 - 5cm	x	x	x	
Impact hole, depth 2 - 5cm, there are warning signs	x	x	x	
Impact hole, depth > 5cm	x	x	x	
Description of damages / defections	s	v	d	
Impact hole, depth> 5cm, there are warning signs	x	x	x	
A pedestrian hallway				
Erosion of surface layer <2cm	x	x	x	
Erosion of surface layer ≥2cm	x	x	x	
Erosion of surface layer ≥2cm, there are warning signs	x	x	x	
The layers break and fall in pieces	x	x	x	
Slipping risk	x	x	x	

Summary Group 11

$Z_{1.1.14}$ =2.0	$\Delta Z_{1.1.14}$= 0	$Z_{1.1.14}$ = 2.0+0= 2.0
Sum group 11		**2.0**

Group 14: Other

Signs	S	V	D	
Dependencies: structural element = Signs				
Missing building designation number	0	0	0	$Z_{1.1.15}$
The load limit sign is missing / incorrect S = 2 - 4, V = 2 - 4	2	2	0	$Z_{1.1.16}$
Drainage of the bridge				
Dependencies: element of construction = equipment, drainage of the bridge				
The fastening of the drainage pipe was corroded	1	0	2	$Z_{1.1.17}$
The fastening parts are missing, outside the traffic area	x	x	x	
The fastening parts are missing, above the traffic surface, V = 1 - 3	x	x	x	
Water leaks from the pipe (above the field)	x	x	x	
Water leaks from the pipe (above the parts of the structure)	0	0	2	$Z_{1.1.18}$
Water leaks from the pipe (above the traffic surfaces)	x	x	x	
Less corrosion damage on drainage pipes	0	0	1	$Z_{1.1.19}$
Significant corrosion damage on drainage pipes	x	x	x	
Missing dilatation of the drainage pipes at the transition of the structure / field	x	x	x	
Rain grid / clogged pipe	0	2	1	$Z_{1.1.20}$
In the raining grid there is a missing catcher of a garbage (pot)	x	x	x	
The drainage grid/ cleaning hole in the hinge plate is not secured	x	x	x	
The drainage grid is broken	x	x	x	
Inspection agents (inspection tools)				
Dependencies: element of construction = equipment, inspection agents				
Cracks on the rail of the inspected vehicle (risk of falling from height)	x	x	x	
Padlock is missing, third parties have unobstructed access to the building site	x	x	x	
Ladders, The distance between rungs is too large (> 280mm)	x	x	x	
Ladders, rungs too close to the building (<150mm)	x	x	x	
Ladder, The distance between the end rungs and the working surface too large (> 100mm)	x	x	x	
Ladders, according to the regulations, the necessary back protection is missing	x	x	x	
Corrosion of inspection agents (tools)				
Dependencies: element of construction = equipment, inspection means, damage = surface, metal				
Thickness of anti-corrosion protection too small	x	x	x	
Locally blown (cracked) anti-corrosion protection	x	x	x	
Anti-corrosive protection on large surfaces is blooming	x	x	x	
The beginning of corrosion - locally	x	x	x	
The beginning of corrosion - a large surface	x	x	x	
Corrosion of individual carriers	x	x	x	
Corrosion of several carriers in a row	x	x	x	
(Tools for protection) Means of protection from invading birds				
Dependencies: structural element = protective agents, protection from birds				
protection agents of invading birds are missing / damaged	x	x	x	

Overview of the building site				
Dependencies: damage = review				
The view of the whole building is not possible? a fictitious assessment of the situation is issued	x	x	x	
The building is very overgrown, only partial inspection possible	0	0	2	$Z_{1.1.21}$

Summary Group 14

$Z_{1.1.15}$ = 1.0	$\Delta Z_{1.1.15}$= -0.1	$Z_{1.1.15}$ = 1.0-0.1= 0.9
$Z_{1.1.16}$ = 2.3	$\Delta Z_{1.1.16}$= -0.1	$Z_{1.1.16}$ = 2.3-0.1= 2.2
$Z_{1.1.17}$ = 2.2	$\Delta Z_{1.1.17}$= 0	$Z_{1.1.17}$ = 2.2+0= 2.2
$Z_{1.1.18}$ = 2.0	$\Delta Z_{1.1.18}$= 0	$Z_{1.1.18}$ = 2.0+0= 2.0
$Z_{1.1.19}$ = 1.1	$\Delta Z_{1.1.19}$= -0.1	$Z_{1.1.19}$ = 1.1-0.1= 1
$Z_{1.1.20}$ =2.1	$\Delta Z_{1.1.20}$= +0.1	$Z_{1.1.20}$ = 2.1+0.1= 2.2
$Z_{1.1.21}$ =2.0	$\Delta Z_{1.1.21}$=0	$Z_{1.1.21}$ = 2.0+0= 2.0
Sum group 14		**12.5**

1.2. LEVEL 2: MAXIMUM DAMAGE

NO.	Group1	Group2	Group9	Group13	Group11	Group14
	9.0	6.1	0	1.9	2	12.5

1.3. LEVEL 3

Group	Z	ΔZ	**Z_{BG}**
Group 1	$Z_{1.1.6}$ = 2.0	ΔZ_2 = 0	Z_{BG}= 2.0+0=2.0
Group 2	$Z_{1.1.12}$ = 2.0	ΔZ_2 = -0.1	Z_{BG}= 2.0-0.1= 1.9
Group 9			Z_{BG}= 0
Group 11	$Z_{1.1.14}$ = 2.0	ΔZ_2= 0	Z_{BG}= 2.0 + 0= 2.0
Group 13	$Z_{1.1.13}$= 2.0	ΔZ_2 = -0.1	Z_{BG}= 2.0-0.1=1.9
Group 14	$Z_{1.1.16}$ = 2.3	ΔZ_2 = -0.1	Z_{BG}= 2.3 - 0.1= 2.2

Z_{ges}= 2.2 ΔZ_3 = 0 (GROUP 14 THE MAXIMUM Z_{BG})

Satisfactory condition

The stability and traffic safety of the structure are given.

The stability and/or durability of at least one component group can be impaired.

The durability of the structure can be affected in the long term. A spread of damage or consequential damage to the structure, which in the long term leads to significant impairment of stability and/or traffic safety or increased wear and tear, is possible.

Ongoing maintenance required.

Medium-term repair required.

Measures to eliminate damage or warnings to maintain road safety may become necessary at short term.

2. SOUK ATHULATHA 2 BRIDGE

2.1. LEVEL 1: REGULAR BRIDGE INSPECTION

Group 1: Superstructure

Description of damage and defects	S	V	D	
Bridges, concrete / reinforced concrete / pre-stressed construction structures				
Dependencies: type of construction = bridge, basic building element = structure construction, material of the spanning structure = concrete				
Graffiti on visible surfaces	0	0	0	$Z_{2.2.1}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{2.2.2}$
Less wear of the protective layer	0	0	1	$Z_{2.2.3}$
The rust on the lower sides of the construction	x	x	x	
Pollution of internal passages of the building (remains of the formwork or other)	x	x	x	
Pollution of internal passages of building (bird feces or other)	x	x	x	
Coarse granularity of concrete of the spanning structure	x	x	x	
The protective layer above the auxiliary rebar for the installation of the main reinforcement is too small	0	0	1	$Z_{2.2.4}$
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) poor quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1.0 to 2.9cm) Good quality of concrete	0	0	3	$Z_{2.2.5}$
The protective layer of the main rebar on the underside of the structure is too small (1,0 to 2,9cm) Poor concrete quality	x	x	x	
The protective layer of the main rebar the underside of the structure is too small (below 1.0cm)	x	x	x	
The carbonate front reached the main rebar	x	x	x	
Visible main rebar on the underside of the structure, the rebar is lightly corroded (without significant reduction of the cross section)	x	x	x	
The main rebar of the spanning structure lies in the area of carbonization and is slightly corroded (it does not apply to the prefabrication reinforcement)	x	x	x	

Visible main rebar on the underside of the spanning structure, corroded rebar (there is a decrease in the cross-section)	x	x	x	
Blooming (water traces) in the area of heavily corroded main reinforcement on the underside of the spanning structure (advanced reductions in cross-section)	x	x	x	
Blooming in the area of heavily corroded main reinforcement on the underside of the spanning structure (the main reinforcement is partly excluded from the load)	x	x	x	
Partial moisture penetration	0	0	2	$Z_{2.2.6}$
Penetration of moisture on large surfaces	x	x	x	
Description of damage / defect	s	v	d	
Bridges, cracks in concrete / reinforced concrete / pre-stressed structure				
Dependencies: type of construction = bridge, basic building element = structure , material of the structure = concrete, damage = cracks				
Surface cracks outside the humidification area (widths) of ≤ 0.1mm in reinforced concrete or prestressed structure (eg cracks from shrinkage)	x	x	x	
Cracks outside the area of humidification (shrinkage) width of 0.1 - <0.2mm in reinforced concrete- or prestressed structure	0	0	1	$Z_{2.2.7}$
Cracks width 0.1 - <0.2mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Surface cracks in the humidification area (widths) 0.2 -≤ 0.4 mm in the RC structure	x	x	x	
Parallel cracks with prestressing of a width of 0.2 -≤ 0.4mm in the area of humidification (squeezing) in the prestressed structure	x	x	x	
Shrinkage widths> 0.4mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (not in the cable extension area)	x	x	x	
Cracks with a width of <0.2mm with prestressed structure (in cable extension)	x	x	x	
Cracks with a width of 0.2 - ≤ 0.4mm at the front of the structure (in the area of cable extension)	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (in the cable extension area)	x	x	x	
Cracks> 0.4mm under load	x	x	x	

Summary Group 1

$Z_{2.2.1}$ = 1.0	$\Delta Z_{2.2.1}$ = -0.1	$Z_{2.2.1}$ = 1.0-0.1=0.9
$Z_{2.2.2}$ = 1.0	$\Delta Z_{2.2.2}$ = +0.1	$Z_{2.2.2}$ = 1.0+0.1=1.1
$Z_{2.2.3}$ = 1.1	$\Delta Z_{2.2.3}$ = - 0.1	$Z_{2.2.3}$ = 1.1-0.1= 1
$Z_{2.2.4}$ = 1.1	$\Delta Z_{2.2.4}$ = -0.1	$Z_{2.2.4}$ = 1.1-0.1= 1
$Z_{2.2.5}$ = 1.1	$\Delta Z_{2.2.5}$ = -0.1	$Z_{2.2.5}$ = 1.1-0.1= 1
$Z_{2.2.6}$ = 2	$\Delta Z_{2.2.6}$ = -0.1	$Z_{2.2.6}$= 2-0.1= 1.9
$Z_{2.2.7}$ = 1.1	$\Delta Z_{2.2.7}$ = 0	$Z_{2.2.7}$ = 1.1+0= 1.1
Sum group 1		**8.0**

Group 2: Substructure

Bridges, substructure	S	V	D	

Dependencies: type of construction = bridge, basic building element = substructure				
Graffiti on visible surfaces	0	0	0	$Z_{2.2.8}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{2.2.9}$
Less wear of the protective layer	0	0	1	$Z_{2.2.10}$
Less rinses in the area of water flows	0	0	1	$Z_{2.2.11}$
Cleaning of the bearing bench (moldings or other)	x	x	x	
Invalidation of the benches (bird droppings or other)	x	x	x	
Remains of the formwork that press the construction	x	x	x	
Cleaning the bearing bench with accumulated moisture	x	x	x	
Formwork material (polystyrene) on the connection with the structure has not been removed	x	x	x	
Less dropping of stone linings	x	x	x	
The installation cover is not correct / damaged	x	x	x	
Cracking stone wall	x	x	x	
Partial moisture zone on stone wall / reinforced concrete	x	x	x	
Moisture on large surfaces of stone wall / reinforced concrete	x	x	x	
Bridges, cracks in concrete- / RC substructure				
Dependencies: type of construction = bridge, basic building element = substructure, damage = cracks				
Dry cracks outside the humidification (spinning) area <0.2mm (no reaction sulfuric acid - RSK)	x	x	x	
Surface cracks in the area of humidification (squeezing), cracks can run water, width <0.2mm (without RSK) REACTION SULFURIC ACID	x	x	x	
Dry cracks outside the humidification area (shrinkage) 0.2 -≤ 0.4mm (without RSK)	0	0	1	$Z_{2.2.12}$
Surface cracks in the area of humidification (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, 0.2 -≤ 0.4mm wide (without RSK)	x	x	x	
Dry cracks outside the area of humidification (shrinkage) of width> 0,4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) of width> 0.4mm (without RSK)	x	x	x	
Description of damage / defect	s	v	d	
Cracks in the area of humidification (cracks), cracks can run water, width> 0,4mm, unarmed concrete (without RSK)	x	x	x	
Cracks in the area of humidification (sprinkling), cracks can run water, width> 0.4mm, RC bottom structure (without RSK)	x	x	x	

Summary Group 2

$Z_{2.2.8} = 1.0$	$\Delta Z_{2.2.8}$ =- 0.1	$Z_{2.2.8}$ = 1.0-0.1= 0.9
$Z_{2.2.9} = 1.0$	$\Delta Z_{2.2.9}$ = +0.1	$Z_{2.2.9}$ = 1.0+0.1= 1.1
$Z_{2.2.10} = 1.1$	$\Delta Z_{2.2.10}$ = -0.1	$Z_{2.2.10}$ = 1.1-0.1= 1
$Z_{2.2.11} = 1.1$	$\Delta Z_{2.2.11}$ = 0	$Z_{2.2.11}$ = 1.1+0= 1.1
$Z_{2.2.12} = 1.1$	$\Delta Z_{2.2.12}$ = -0.1	$Z_{2.2.12}$ = 1.1-0.1= 1.0
Sum group 2		**5.1**

Group 9: Transition devices

Transition devices (joints)	S	V	D	
Dependencies: structural element = transitional device				
Contaminated transitional device (moving still possible)	x	x	x	
A highly Contaminated transient device (scrolling limited)	x	x	x	
Loosening the fixing of the profiles in the carpet of the structures, the profile is still held	x	x	x	
Loosening the fixing of the profiles in the carpet construction, the profile loosened	x	x	x	
Rubber profile dropped or multiple damaged	x	x	x	
Asphalt crossings, thin asphalt mass (rough, small open cracks)	0	0	0	$Z_{2.2.13}$
The asphalt crossing cracked and depressed	x	x	x	
The transient device is missing, the spanning structure is cracked at the ends	x	x	x	
The transient device is missing, the spanning structure at the ends is cracked and shrugged	x	x	x	

Summary Group 9

$Z_{2.2.13} = 1.0$	$\Delta Z_{2.2.13} = -0.1$	$Z_{2.2.13} = 1.0\text{-}0.1 = 0.9$
Sum group 9		**0.9**

Group 13: Fence

Protective means (protective parts of construction)	S	V	D	
Fence				
Dependencies: element of construction = protective means, fence				
The compound fence - construction of the defect	x	x	x	
There is no fence, there are bumpers at> 50km / h, no pedestrian traffic is planned	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, small building, no pedestrian traffic is foreseen (temporary hazard sign exists)	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, pedestrian traffic planned	x	x	x	
Missing wire in the handrail of the fence, the length of the building is <20m	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, there is a bumper	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, the bumper is missing	x	x	x	
Fence height is not in accordance with regulations (difference ≤ 5cm)	x	x	x	
Fence height is not in accordance with regulations (difference 5 - 10cm)	x	x	x	
Fence height is not in accordance with regulations (difference > 10cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference ≤ 2cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference > 2cm)	x	x	x	
Missing individual fence filling rods	x	x	x	
Missing more consecutive fence filling rods	x	x	x	
Guardrail	x	x	x	

Dependencies: structural element = protection agent, bumper	x	x	x	
Bumper height is not in accordance with regulations (difference ≤ 3cm)	x	x	x	
Bumper height is not in accordance with regulations (difference > 3cm)	x	x	x	
The guardrail is partially deformed	0	1	1	$Z_{2.2.14}$
a fence and a bumper are missing	x	x	x	
Weak anchor fasteners of the bumper / fence	x	x	x	
There is a lack of anchoring of protective devices on the length of the move	x	x	x	
Corrosion of protective agents (protective tools)				
Dependencies: structural element = protective agents, damage = surface, metal				
Insufficient thickness of the protective layer	x	x	x	
Local scattering (breaking) of the protective layer	x	x	x	
Scattering (breaking) of the protective layer on larger surfaces	x	x	x	
Local start of corrosion	x	x	x	
Corrosion of large surfaces	x	x	x	
The corrosion of individual support elements of the protecting agents	x	x	x	
The corrosion of more consecutive support elements of the protecting agents				

Summary Group 13

$Z_{2.2.14}$ =1.3	$\Delta Z_{2.2.14}$= 0	$Z_{2.2.14}$ = 1.3+0= 1.3
Sum group 13		**1.3**

Group 11: Road surface

Road surface	S	V	D	
Dependencies: structural element = useful surface				
pavement				
Subsidence of the pavement behind the abutments (≤ 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm), there are warning signs	x	x	x	
Drainage does not work, the risk of drifting	0	2	0	$Z_{2.2.15}$
Paving grooves / deflection , depth <1cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm, there are warning signs	x	x	x	
Pulley grooves / indentations, depth> 3cm	x	x	x	
Paving grooves / indentations, depth> 3cm, there are warning signs	x	x	x	
Bubbles, heights of ≤ 2cm	x	x	x	
Bubbles, height 2 - 5cm	x	x	x	
Bubbles, height 2 - 5cm, there are warning signs	x	x	x	
Bubbles, height > 5cm	x	x	x	
Bubbles, height > 5cm, there are warning signs	x	x	x	
Impact hole, depth ≤ 2cm	x	x	x	
Impact hole, depth 2 - 5cm	x	x	x	
Impact hole, depth 2 - 5cm, there are warning signs	x	x	x	

Impact hole, depth > 5cm	x	x	x	
Description of damages / defections	s	v	d	
Impact hole, depth> 5cm, there are warning signs	x	x	x	
A pedestrian hallway				
Erosion of surface layer <2cm	0	1	0	$Z_{2.2.16}$
Erosion of surface layer ≥2cm				
Erosion of surface layer ≥2cm, there are warning signs	x	x	x	
The layers break and fall in pieces	x	x	x	
Slipping risk	x	x	x	

Summary Group11

$Z_{2.2.15}$ =2.0	$\Delta Z_{2.2.15}$= 0	$Z_{2.2.15}$ = 2.0+0= 2.0
$Z_{2.2.16}$ =1.1	$\Delta Z_{2.2.16}$= 0	$Z_{2.2.16}$ = 1.1+0= 1.1
Sum group 11		**3.1**

Group 14: Other

Signs	S	V	D	
Dependencies: structural element = Signs				
Missing building designation number	0	0	0	$Z_{2.2.17}$
The load limit sign is missing / incorrect S = 2 - 4, V = 2 - 4	x	x	x	
Drainage of the bridge				
Dependencies: element of construction = equipment, drainage of the bridge				
The fastening of the drainage pipe was corroded	x	x	x	
The fastening parts are missing, outside the traffic area	x	x	x	
The fastening parts are missing, above the traffic surface, V = 1 - 3	x	x	x	
Water leaks from the pipe (above the field)	x	x	x	
Water leaks from the pipe (above the parts of the structure)	x	x	x	
Water leaks from the pipe (above the traffic surfaces)	x	x	x	
Less corrosion damage on drainage pipes	x	x	x	
Significant corrosion damage on drainage pipes	x	x	x	
Missing dilatation of the drainage pipes at the transition of the structure / field	x	x	x	
Rain grid / clogged pipe	0	2	1	$Z_{2.2.18}$
In the raining grid there is a missing catcher of a garbage (pot)	x	x	x	
The drainage grid/ cleaning hole in the hinge plate is not secured	x	x	x	
The drainage grid is broken	x	x	x	
Inspection agents (inspection devices or inspection tools)				
Dependencies: element of construction = equipment, inspection agents				
Cracks on the rail of the inspected vehicle (risk of falling from height)	x	x	x	
Padlock is missing, third parties have unobstructed access to the building site	x	x	x	
Ladders, The distance between rungs is too large (> 280mm)	x	x	x	
Ladders, rungs too close to the building (<150mm)	x	x	x	
Ladder, The distance between the end rungs and the working surface too large (> 100mm)	x	x	x	

Ladders, according to the regulations, the necessary back protection is missing	x	x	x	
Corrosion of inspection agents				
Dependencies: element of construction = equipment, inspection means, damage = surface, metal				
Thickness of anti-corrosion protection too small	x	x	x	
Locally blown (cracked) anti-corrosion protection	x	x	x	
Anti-corrosive protection on large surfaces is blooming	x	x	x	
The beginning of corrosion - locally	x	x	x	
The beginning of corrosion - a large surface	x	x	x	
Corrosion of individual carriers	x	x	x	
Corrosion of several carriers in a row	x	x	x	
(Tools for protection) Means of protection from invading birds				
Dependencies: structural element = protective agents, protection from birds				
protection agents of invading birds are missing / damaged	x	x	x	
Overview of the building site				
Dependencies: damage = review				
The view of the whole building is not possible? a fictitious assessment of the situation is issued	x	x	x	
The building is very overgrown, only partial inspection possible	0	0	2	$Z_{2.2.19}$

Summary Group14

$Z_{2.2.17}$ =1.0	$\Delta Z_{2.2.17}$= -0.1	$Z_{2.2.17}$ = 1-0.1=0.9
$Z_{2.2.18}$ =2.1	$\Delta Z_{2.2.18}$= 0	$Z_{2.2.18}$ = 2.1+0= 2.1
$Z_{2.2.19}$ =2.0	$\Delta Z_{2.2.19}$=0	$Z_{2.2.19}$ = 2.0+0= 2.0
Sum group 14		**5.0**

2.2. LEVEL 2: Maximum Damage

NO.	Group1	Group2	Group9	Group13	Group11	Group14
	8.0	5.1	0.9	1.3	3.1	5.0

2.3. LEVEL 3

Group	Z	ΔZ	Z_{BG}
Group 1	$Z_{2.2.6}$ = 2.0	ΔZ_2 = -0.1	Z_{BG}= 2.0-0.1=1.9

Group 2	$Z_{2.2.10} = 1.1$	$\Delta Z_2 = 0$	$Z_{BG} = 1.1+0 = 1.1$
	$Z_{2.2.11} = 1.1$	$\Delta Z_2 = -0.1$	
	$Z_{2.2.12} = 1.1$	$\Delta Z_2 = -0.1$	
Group 9	$Z_{2.2.13} = 1.0$	$\Delta Z_2 = -0.1$	$Z_{BG} = 1.0-0.1=0.9$
Group 11	$Z_{2.2.15} = 2.0$	$\Delta Z_2 = 0$	$Z_{BG} = 2.0 + 0 = 2.0$
Group 13	$Z_{2.2.14} = 1.3$	$\Delta Z_2 = 0$	$Z_{BG} = 1.3+0=1.3$
Group 14	$Z_{2.2.18} = 2.1$	$\Delta Z_2 = 0$	$Z_{BG} = 2.1 +0 = 2.1$

$Z_{ges} = 2.1$ $\Delta Z_3 = 0$ (GROUP 14 THE MAXIMUM Z_{BG})

Satisfactory condition

The stability and traffic safety of the structure are given.

The stability and/or durability of at least one component group can be impaired.

The durability of the structure can be affected in the long term. A spread of damage or consequential damage to the structure, which in the long term leads to significant impairment of stability and/or traffic safety or increased wear and tear, is possible.

Ongoing maintenance required.

Medium-term repair required.

Measures to eliminate damage or warnings to maintain road safety may become necessary at short term.

3. AL SEEKA ROAD BRIDGE

3.1. Level 1: Regular bridge inspection

Group 1: Superstructure

Description of damage and defects	S	V	D	
Bridges, concrete / reinforced concrete / pre-stressed construction structures				
Dependencies: type of construction = bridge, basic building element = structure construction, material of the spanning structure = concrete				
Graffiti on visible surfaces	0	0	0	$Z_{3.3.1}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{3.3.2}$
Less wear of the protective layer	0	0	1	$Z_{3.3.3}$
The rust on the lower sides of the construction	x	x	x	

Pollution of internal passages of the building (remains of the formwork or other)	x	x	x	
Pollution of internal passages of building (bird feces or other)	x	x	x	
Coarse granularity of concrete of the spanning structure	x	x	x	
The protective layer above the auxiliary rebar for the installation of the main rebar is too small	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) poor quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1.0 to 2.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1,0 to 2,9cm) Poor concrete quality	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (below 1.0cm)	x	x	x	
The carbonate front reached the main rebar	x	x	x	
Visible main rebar on the underside of the structure, the reinforcement is lightly corroded (without significant reduction of the cross section)	x	x	x	
The main rebar of the spanning structure lies in the area of carbonization and is slightly corroded (it does not apply to the prefabrication reinforcement)	x	x	x	
Visible main rebar on the underside of the spanning structure, corroded rebar (there is a decrease in the cross-section)	x	x	x	
Blooming (water traces) in the area of heavily corroded main reinforcement on the underside of the spanning structure (advanced reductions in cross-section)	x	x	x	
Blooming in the area of heavily corroded main reinforcement on the underside of the spanning structure (the main reinforcement is partly excluded from the load)	x	x	x	
Partial moisture penetration	0	0	2	$Z_{3.3.4}$
Penetration of moisture on large surfaces	x	x	x	
Surface cracks outside the humidification area (widths) of ≤ 0.1mm in reinforced concrete or prestressed structure (eg cracks from shrinkage)	x	x	x	
Cracks outside the area of humidification (shrinkage) width of 0.1 - <0.2mm in reinforced concrete- or prestressed structure	0	0	1	$Z_{3.3.5}$
Cracks width 0.1 - <0.2mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Surface cracks in the humidification area (widths) 0.2 -≤ 0.4 mm in the RC structure	x	x	x	
Parallel cracks with prestressing of a width of 0.2 -≤ 0.4mm in the area of humidification (squeezing) in the prestressed structure	x	x	x	
Shrinkage widths> 0.4mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (not in the cable extension area)	x	x	x	
Cracks with a width of <0.2mm with prestressed structure (in cable extension)	x	x	x	
Cracks with a width of 0.2 - ≤ 0.4mm at the front of the structure (in the area of cable extension)	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (in the cable extension area)	x	x	x	
Cracks> 0.4mm under load	x	x	x	

Summary Group1

$Z_{3.3.1}$ = 1.0	$\Delta Z_{3.3.1}$ = 0	$Z_{3.3.1}$ = 1.0+0=1.0
$Z_{3.3.2}$ = 1.0	$\Delta Z_{3.3.2}$ = +0.1	$Z_{3.3.2}$ = 1.0+0.1=1.1
$Z_{3.3.3}$ = 1.1	$\Delta Z_{3.3.3}$ = 0	$Z_{3.3.3}$ = 1.1+0= 1.1
$Z_{3.3.4}$ = 2.0	$\Delta Z_{3.3.4}$ = 0	$Z_{3.3.4}$ = 2.0+0= 2.0

$Z_{3.3.5}$ = 1.1	$\Delta Z_{3.3.6}$ = -0.1	$Z_{3.3.5}$= 1.1-0.1= 1.0
Sum group 1		**6.2**

Group 2: Substructure

Bridges, substructure	S	V	D	
Dependencies: type of construction = bridge, basic building element = substructure				
Graffiti on visible surfaces	0	0	0	$Z_{3.3.6}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{3.3.7}$
Less wear of the protective layer	0	0	1	$Z_{3.3.8}$
Less rinses in the area of water flows	0	0	1	$Z_{3.3.9}$
Cleaning of the bearing bench (moldings or other)	x	x	x	
Invalidation of the benches (bird droppings or other)	x	x	x	
Remains of the formwork that press the construction	x	x	x	
Cleaning the bearing bench with accumulated moisture	x	x	x	
Formwork material (polystyrene) on the connection with the structure has not been removed	x	x	x	
Less dropping of stone linings	x	x	x	
The installation cover is not correct / damaged	x	x	x	
Cracking stone wall	x	x	x	
Partial moisture stone wall / reinforced concrete	x	x	x	
Moisture on large surfaces of stone wall / reinforced concrete	x	x	x	
Bridges, cracks in concrete- / RC substructure				
Dependencies: type of construction = bridge, basic building element = substructure, damage = cracks				
Dry cracks outside the humidification (spinning) area <0.2mm (no reaction sulfuric acid - RSK)	x	x	x	
Surface cracks in the area of humidification (squeezing), cracks can run water, width <0.2mm (without RSK) REACTION SULFURIC ACID	x	x	x	
Dry cracks outside the humidification area (shrinkage) 0.2 -≤ 0.4mm (without RSK)	0	0	1	$Z_{3.3.10}$
Surface cracks in the area of humidification (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, 0.2 -≤ 0.4mm wide (without RSK)	x	x	x	
Dry cracks outside the area of humidification (shrinkage) of width> 0,4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) of width> 0.4mm (without RSK)	x	x	x	
Description of damage / defect	S	V	D	
Cracks in the area of humidification (cracks), cracks can run water, width> 0,4mm, unarmed concrete (without RSK)	x	x	x	
Cracks in the area of humidification (sprinkling), cracks can run water, width> 0.4mm, RC bottom structure (without RSK)	x	x	x	

Summary Group2

$Z_{3.3.6}$= 1.0	$\Delta Z_{3.3.6}$ = 0	$Z_{3.3.6}$ = 1.0+0= 1.0
$Z_{3.3.7}$ = 1.0	$\Delta Z_{3.3.7}$ = +0.1	$Z_{3.3.7}$ = 1.0+0.1= 1.1
$Z_{3.3.8}$ = 1.1	$\Delta Z_{3.3.8}$ = -0.1	$Z_{3.3.8}$ = 1.1-0.1= 1

$Z_{3.3.9} = 1.1$	$\Delta Z_{3.3.9} = 0$	$Z_{3.3.9} = 1.1+0= 1.1$
$Z_{3.3.10}= 1.1$	$\Delta Z_{3.3.10} = -0.1$	$Z_{3.3.10} = 1.1-0.1= 1.0$
Sum group 2		**5.2**

Group 9: Transition devices

Transition devices (joints)	S	V	D	
Dependencies: structural element = transitional device				
Contaminated transitional device (moving still possible)	x	x	x	
A highly Contaminated transient device (scrolling limited)	x	x	x	
Loosening the fixing of the profiles in the carpet of the structures, the profile is still held	x	x	x	
Loosening the fixing of the profiles in the carpet construction, the profile loosened	x	x	x	
	x	x	x	
Rubber profile dropped or multiple damaged	x	x	x	
Asphalt crossings, thin asphalt mass (rough, small open cracks)	x	x	x	
The asphalt crossing cracked and depressed	x	x	x	
The transient device is missing, the spanning structure is cracked at the ends	x	x	x	
The transient device is missing, the spanning structure at the ends is cracked and shrugged	x	x	x	

Summary Group9

Sum group 9	Nothing (good condition)

Group 13: Fence

Protective means	S	V	D	
Fence				
Dependencies: element of construction = protective means, fence				
The compound fence - construction of the defect	x	x	x	
There is no fence, there are bumpers at> 50km / h, no pedestrian traffic is planned	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, small building, no pedestrian traffic is foreseen (temporary hazard sign exists)	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, pedestrian traffic planned	x	x	x	
Missing wire in the handrail of the fence, the length of the building is <20m	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, there is a bumper	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, the bumper is missing	x	x	x	
Fence height is not in accordance with regulations (difference ≤ 5cm)	x	x	x	
Fence height is not in accordance with regulations (difference 5 - 10cm)	x	x	x	
Fence height is not in accordance with regulations (difference > 10cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference ≤ 2cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference > 2cm)	x	x	x	

Missing individual fence filling rods	x	x	x	
Missing more consecutive fence filling rods	x	x	x	
Guardrail				
Dependencies: structural element = protection agent, guardrail				
Bumper height is not in accordance with regulations (difference ≤ 3cm)	x	x	x	
Bumper height is not in accordance with regulations (difference > 3cm)	x	x	x	
The bumper is partially deformed	x	x	x	
a fence and a bumper are missing	x	x	x	
Weak anchor fasteners of the bumper / fence	x	x	x	
There is a lack of anchoring of protective devices on the length of the move	x	x	x	
Corrosion of protective agents (devices, tools)				
Dependencies: structural element = protective agents, damage = surface, metal				
Insufficient thickness of the protective layer	x	x	x	
Local scattering (breaking) of the protective layer	x	x	x	
Scattering (breaking) of the protective layer on larger surfaces	x	x	x	
Local start of corrosion	x	x	x	
Corrosion of large surfaces	x	x	x	
The corrosion of individual support elements of the protecting agents	x	x	x	
The corrosion of more consecutive support elements of the protecting agents	x	x	x	

Summary Group13

Sum group 13	Nothing

Group 11: Road surface

Road surface	S	V	D	
Dependencies: structural element = useful surface				
pavement				
Subsidence of the pavement behind the abutments (≤ 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm), there are warning signs	x	x	x	
Drainage does not work, the risk of drifting	0	2	0	$Z_{3.3.11}$
Paving grooves / indentations, depth <1cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm, there are warning signs	x	x	x	
Pulley grooves / indentations, depth> 3cm	x	x	x	
Paving grooves / indentations, depth> 3cm, there are warning signs	x	x	x	
Bubbles, heights of ≤ 2cm	x	x	x	
Bubbles, height 2 - 5cm	x	x	x	
Bubbles, height 2 - 5cm, there are warning signs	x	x	x	
Bubbles, height > 5cm	x	x	x	
Bubbles, height > 5cm, there are warning signs	x	x	x	
Impact hole, depth ≤ 2cm	x	x	x	
Impact hole, depth 2 - 5cm	x	x	x	
Impact hole, depth 2 - 5cm, there are warning signs	x	x	x	

Impact hole, depth > 5cm	x	x	x	
Impact hole, depth> 5cm, there are warning signs	x	x	x	
A pedestrian path	x	x	x	
Erosion of surface layer <2cm	x	x	x	
Erosion of surface layer ≥2cm	x	x	x	
Erosion of surface layer ≥2cm, there are warning signs	x	x	x	
The layers break and fall in pieces	x	x	x	
Slipping risk	x	x	x	

Summary Group11

$Z_{3.3.11}$ =2.0	$\Delta Z_{3.3.11}$= 0	$Z_{3.3.11}$ = 2.0+0= 2.0
Sum group 11		2

Group 14: Other

Signs	S	V	D	
Dependencies: structural element = Signs				
Missing building designation number	0	0	0	$Z_{3.3.12}$
The load limit sign is missing / incorrect S = 2 - 4, V = 2 - 4	x	x	x	
Drainage of the bridge				
Dependencies: element of construction = equipment, drainage of the bridge				
The fastening of the drainage pipe was corroded	x	x	x	
The fastening parts are missing, outside the traffic area	x	x	x	
The fastening parts are missing, above the traffic surface, V = 1 - 3	x	x	x	
Water leaks from the pipe (above the field)	x	x	x	
Water leaks from the pipe (above the parts of the structure)	x	x	x	
Water leaks from the pipe (above the traffic surfaces)	x	x	x	
Less corrosion damage on drainage pipes	x	x	x	
Significant corrosion damage on drainage pipes	x	x	x	
Missing dilatation of the drainage pipes at the transition of the structure / field	x	x	x	
Rain grid / clogged pipe	0	2	1	$Z_{3.3.13}$
In the raining grid there is a missing catcher of a garbage (pot)	x	x	x	
The drainage grid/ cleaning hole in the hinge plate is not secured	x	x	x	
The drainage grid is broken	x	x	x	
Inspection agents (inspection tools)				
Dependencies: element of construction = equipment, inspection agents				
Cracks on the rail of the inspected vehicle (risk of falling from height)	x	x	x	
Padlock is missing, third parties have unobstructed access to the building site	x	x	x	
Ladders, The distance between rungs is too large (> 280mm)	x	x	x	
Ladders, rungs too close to the building (<150mm)	x	x	x	
Ladder, The distance between the end rungs and the working surface too large (> 100mm)	x	x	x	
Ladders, according to the regulations, the necessary back protection is missing	x	x	x	

Corrosion of inspection agents (devices, tools)				
Dependencies: element of construction = equipment, inspection means, damage = surface, metal				
Thickness of anti-corrosion protection too small	x	x	x	
Locally blown (cracked) anti-corrosion protection	x	x	x	
Anti-corrosive protection on large surfaces is blooming	x	x	x	
The beginning of corrosion - locally	x	x	x	
The beginning of corrosion - a large surface	x	x	x	
Corrosion of individual carriers	x	x	x	
Corrosion of several carriers in a row	x	x	x	
(Tools for protection) Means of protection from invading birds				
Dependencies: structural element = protective agents, protection from birds				
protection agents of invading birds are missing / damaged	x	x	x	
Overview of the building site				
Dependencies: damage = review				
The view of the whole building is not possible? a fictitious assessment of the situation is issued	x	x	x	
The building is very overgrown, only partial inspection possible	0	0	2	$Z_{3.3.14}$

Summary Group14

$Z_{3.3.12}$ =1.0	$\Delta Z_{3.3.12}$= -0.1	$Z_{3.3.12}$ = 1.0-0.1=0.9
$Z_{3.3.13}$ =2.1	$\Delta Z_{3.3.13}$=-0.1	$Z_{3.3.13}$ = 2.1-0.1= 2.0
$Z_{3.3.14}$ =2.0	$\Delta Z_{3.3.14}$=0	$Z_{3.3.14}$ = 2.0-0= 2.0
Sum group 14		**4,9**

3.2. Level 2: Maximum Damage

NO.	Group1	Group2	Group9	Group13	Group11	Group14
	6,2	5.2	nothing	nothing	2	4,9

3.3. Level 3:

Group	Z	ΔZ	**Z_{BG}**
Group 1	$Z_{3.3.4}$ = 2.0	ΔZ_2 = 0	Z_{BG}= 2.0+0=2.0
Group 2	$Z_{3.3.8}$ = 1.1	ΔZ_2 = -0.1	Z_{BG}= 1.1+0 = 1.1
	$Z_{3.3.9}$ = 1.1	ΔZ_2 = -0.1	
	$Z_{3.3.10}$ = 1.1	ΔZ_2 = 0	
Group 9			Z_{BG}= 0

Group 11	$Z_{3.3.11} = 2.0$	$\Delta Z_2 = -0.1$	$Z_{BG} = 2.0 - 0.1 = 1.9$
Group 13			$Z_{BG} = 0$
Group 14	$Z_{3.3.14} = 2.0$	$\Delta Z_2 = 0$	$Z_{BG} = 2.0 - 0 = 2.0$

$Z_{ges} = 2.0$ $\Delta Z_3 = 0$ (GROUP 1 and 14 THE MAXIMUM Z_{BG})

Satisfactory condition

The stability and traffic safety of the structure are given.

The stability and/or durability of at least one component group can be impaired.

The durability of the structure can be affected in the long term. A spread of damage or consequential damage to the structure, which in the long term leads to significant impairment of stability and/or traffic safety or increased wear and tear, is possible.

Ongoing maintenance required.

Medium-term repair required.

Measures to eliminate damage or warnings to maintain road safety may become necessary at short term.

4. BAB BIN GHESHIR ROAD BRIDGE

4.1. Level 1: Regular Bridge Inspection

Group 1: Superstructure

Description of damage and defects	S	V	D	
Bridges, concrete / reinforced concrete / pre-stressed construction structures				
Dependencies: type of construction = bridge, basic building element = structure construction, material of the spanning structure = concrete				
Graffiti on visible surfaces	0	0	0	$Z_{4.4.1}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{4.4.2}$
Less wear of the protective layer	0	0	1	$Z_{4.4.3}$
The rust on the lower sides of the construction	x	x	x	
Pollution of internal passages of the building (remains of the formwork or other)	x	x	x	
Pollution of internal passages of building (bird feces or other)	x	x	x	
Coarse granularity of concrete of the spanning structure	x	x	x	
The protective layer above the auxiliary rebar for the installation of the main reinforcement is too small	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm)	x	x	x	

Good quality of concrete				
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) poor quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1.0 to 2.9cm) Good quality of concrete	0	0	2	$Z_{4.4.4}$
The protective layer of the main reinforcement on the underside of the structure is too small (1,0 to 2,9cm) Poor concrete quality	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (below 1.0cm)	x	x	x	
The carbonate front reached the main rebar	x	x	x	
Visible main rebar on the underside of the structure, the rebar is lightly corroded (without significant reduction of the cross section)	x	x	x	
The main rebar of the spanning structure lies in the area of carbonization and is slightly corroded (it does not apply to the prefabrication reinforcement)	x	x	x	
Visible main rebar on the underside of the spanning structure, corroded rebar (there is a decrease in the cross-section)	x	x	x	
Blooming (water traces) in the area of heavily corroded main reinforcement on the underside of the spanning structure (advanced reductions in cross-section)	x	x	x	
Blooming in the area of heavily corroded main reinforcement on the underside of the spanning structure (the main reinforcement is partly excluded from the load)	x	x	x	
Partial moisture penetration	0	0	2	$Z_{4.4.5}$
Penetration of moisture on large surfaces	x	x	x	
Bridges, cracks in concrete / reinforced concrete / pre-stressed structure				
Dependencies: type of construction = bridge, basic building element = structure , material of the structure = concrete, damage = cracks				
Surface cracks outside the humidification area (widths) of ≤ 0.1mm in reinforced concrete or prestressed structure (eg cracks from shrinkage)	0	0	0	$Z_{4.4.6}$
Cracks outside the area of humidification (shrinkage) width of 0.1 - <0.2mm in reinforced concrete- or prestressed structure	x	x	x	
Cracks width 0.1 - <0.2mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Surface cracks in the humidification area (widths) 0.2 -≤ 0.4 mm in the RC structure	x	x	x	
Parallel cracks with prestressing of a width of 0.2 -≤ 0.4mm in the area of humidification (squeezing) in the prestressed structure	x	x	x	
Shrinkage widths> 0.4mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (not in the cable extension area)	x	x	x	
Cracks with a width of <0.2mm with prestressed structure (in cable extension)	x	x	x	
Cracks with a width of 0.2 - ≤ 0.4mm at the front of the structure (in the area of cable extension)	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (in the cable extension area)	x	x	x	
Cracks> 0.4mm under load	x	x	x	

Summary Group1

$Z_{4.4.1}$ = 1.0	$\Delta Z_{4.4.1}$ = 0	$Z_{4.4.1}$ = 1.0+0=1.0
$Z_{4.4.2}$ = 1.0	$\Delta Z_{4.4.2}$ = +0.1	$Z_{4.4.2}$ = 1.0+0.1=1.1
$Z_{4.4.3}$ = 1.1	$\Delta Z_{4.4.3}$ = -0.1	$Z_{4.4.3}$ = 1.1-0.1= 1
$Z_{4.4.4}$ = 2.0	$\Delta Z_{4.4.4}$ = -0.1	$Z_{4.4.4}$= 2.0-0.1= 1,9
$Z_{4.4.5}$ = 2.0	$\Delta Z_{4.4.5}$ = 0	$Z_{4.4.5}$ = 2.0+0= 2.0

$Z_{4.4.6} = 1.0$	$\Delta Z_{4.4.6} = 0$	$Z_{4..6} = 1.0+0 = 1$
Sum group 1		**8**

Group 2: Substructure

Bridges, substructure	S	V	D	
Dependencies: type of construction = bridge, basic building element = substructure				
Graffiti on visible surfaces	0	0	0	$Z_{4.4.7}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{4.4.8}$
Less wear of the protective layer	0	0	1	$Z_{4.4.9}$
Less rinses in the area of water flows	0	0	1	$Z_{4.4.10}$
Cleaning of the bearing bench (moldings or other)	x	x	x	
Invalidation of the benches (bird droppings or other)	x	x	x	
Remains of the formwork that press the construction	x	x	x	
Cleaning the bearing bench with accumulated moisture	x	x	x	
Formwork material (polystyrene) on the connection with the structure has not been removed	x	x	x	
Less dropping of stone linings	x	x	x	
The installation cover is not correct / damaged	x	x	x	
Cracking stone wall	x	x	x	
Partial moisture on stone wall / reinforced concrete	x	x	x	
Moisture on large surfaces of stone wall / reinforced concrete	x	x	x	
Dry cracks outside the humidification (spinning) area <0.2mm (no reaction sulfuric acid - RSK)	x	x	x	
Surface cracks in the area of humidification (squeezing), cracks can run water, width <0.2mm (without RSK) REACTION SULFURIC ACID	x	x	x	
Dry cracks outside the humidification area (shrinkage) 0.2 -≤ 0.4mm (without RSK)	0	0	1	$Z_{4.4.11}$
Surface cracks in the area of humidification (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, 0.2 -≤ 0.4mm wide (without RSK)	x	x	x	
Dry cracks outside the area of humidification (shrinkage) of width> 0,4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) of width> 0.4mm (without RSK)	x	x	x	
Description of damage / defect	S	V	D	
Cracks in the area of humidification (cracks), cracks can run water, width> 0,4mm, unarmed concrete (without RSK)	x	x	x	
Cracks in the area of humidification (sprinkling), cracks can run water, width> 0.4mm, RC bottom structure (without RSK)	x	x	x	

Summary Group2

$Z_{4.4.7} = 1.0$	$\Delta Z_{4.4.7} = 0$	$Z_{4.4.7} = 1.0+0 = 1.0$
$Z_{4.4.8} = 1.0$	$\Delta Z_{4.4.8} = +0.1$	$Z_{4.4.8} = 1.0+0.1 = 1.1$
$Z_{4.4.9} = 1.1$	$\Delta Z_{4.4.9} = -0.1$	$Z_{4.4.9} = 1.1-0.1 = 1$
$Z_{4.4.10} = 1.1$	$\Delta Z_{4.4.10} = 0$	$Z_{4.4.10} = 1.1+0 = 1.1$

$Z_{4.4.11} = 1.1$	$\Delta Z_{4.4.11} = -0.1$	$Z_{4.4.11} = 1.1\text{-}0.1 = 1$
Sum group 2		**5.2**

Group 9: Transition devices

Transition devices (joints)	S	V	D	
Dependencies: structural element = transitional device				
Contaminated transitional device (moving still possible)	x	x	x	
A highly Contaminated transient device (scrolling limited)	x	x	x	
Loosening the fixing of the profiles in the carpet of the structures, the profile is still held	x	x	x	
Loosening the fixing of the profiles in the carpet construction, the profile loosened	x	x	x	
	x	x	x	
Rubber profile dropped or multiple damaged	x	x	x	
Asphalt crossings, thin asphalt mass (rough, small open cracks)	0	0	0	$Z_{4.4.12}$
The asphalt crossing cracked and depressed	x	x	x	
The transient device is missing, the spanning structure is cracked at the ends	x	x	x	
The transient device is missing, the spanning structure at the ends is cracked and shrugged	x	x	x	

Summary Group9

$Z_{4.4.12} = 1.0$	$\Delta Z_{4.4.12} = 0$	$Z_{4.4.12} = 1.0+0 = 1.0$
Sum group 9		**1.0**

Group 13: Fence

Protective means	S	V	D	
Fence				
Dependencies: element of construction = protective means, fence				
The compound fence - construction of the defect	x	x	x	
There is no fence, there are bumpers at> 50km / h, no pedestrian traffic is planned	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, small building, no pedestrian traffic is foreseen (temporary hazard sign exists)	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, pedestrian traffic planned	x	x	x	
Missing wire in the handrail of the fence, the length of the building is <20m	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, there is a bumper	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, the bumper is missing	x	x	x	
Fence height is not in accordance with regulations (difference ≤ 5cm)	x	x	x	
Fence height is not in accordance with regulations (difference 5 - 10cm)	x	x	x	
Fence height is not in accordance with regulations (difference > 10cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference ≤ 2cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference	x	x	x	

> 2cm)				
Missing individual fence filling rods	x	x	x	
Missing more consecutive fence filling rods	x	x	x	
Bumper				
Dependencies: structural element = protection agent, bumper				
Bumper height is not in accordance with regulations (difference ≤ 3cm)	x	x	x	
Bumper height is not in accordance with regulations (difference > 3cm)	x	x	x	
The bumper is partially deformed	x	x	x	
a fence and a bumper are missing	x	x	x	
Weak anchor fasteners of the bumper / fence	x	x	x	
There is a lack of anchoring of protective devices on the length of the move	x	x	x	
Corrosion of protective agents				
Dependencies: structural element = protective agents, damage = surface, metal				
Insufficient thickness of the protective layer	x	x	x	
Local scattering (breaking) of the protective layer	x	x	x	
Scattering (breaking) of the protective layer on larger surfaces	x	x	x	
Local start of corrosion	x	x	x	
Corrosion of large surfaces	x	x	x	
The corrosion of individual support elements of the protecting agents	x	x	x	
The corrosion of more consecutive support elements of the protecting agents	x	x	x	

Summary Group13

Sum group 13	nothing

Group 11: Road Surface

Road surface	S	V	D	
Dependencies: structural element = useful surface				
pavement				
Subsidence of the pavement behind the abutments (≤ 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm), there are warning signs	x	x	x	
Drainage does not work, the risk of drifting	0	2	0	$Z_{4.4.13}$
Paving grooves / indentations, depth <1cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm, there are warning signs	x	x	x	
Pulley grooves / indentations, depth> 3cm	x	x	x	
Paving grooves / indentations, depth> 3cm, there are warning signs	x	x	x	
Bubbles, heights of ≤ 2cm	x	x	x	
Bubbles, height 2 - 5cm	x	x	x	
Bubbles, height 2 - 5cm, there are warning signs	x	x	x	
Bubbles, height > 5cm	x	x	x	
Bubbles, height > 5cm, there are warning signs	x	x	x	
Impact hole, depth ≤ 2cm	x	x	x	
Impact hole, depth 2 - 5cm	x	x	x	
Impact hole, depth 2 - 5cm, there are warning signs	x	x	x	

Impact hole, depth > 5cm	x	x	x	
Description of damages / defections	s	v	d	
Impact hole, depth> 5cm, there are warning signs	x	x	x	
A pedestrian hallway	x	x	x	
Erosion of surface layer <2cm (layer worn out)	0	1	0	$Z_{4.4.14}$
Erosion of surface layer ≥2cm	x	x	x	
Erosion of surface layer ≥2cm, there are warning signs	x	x	x	
The layers break and fall in pieces	x	x	x	
Slipping risk	x	x	x	

Summary Group11

$Z_{4.4.13}$ =2.0	$\Delta Z_{4.4.13}$= +0.1	$Z_{4.4.13}$ = 2.0+0.1= 2.1
$Z_{4.4.14}$ =1.1	$\Delta Z_{4.4.14}$= 0	$Z_{4.4.14}$ = 1,1+0= 1.1
Sum group 11		**3.2**

Group 14: Other

Signs	S	V	D	
Dependencies: structural element = Signs				
Missing building designation number	0	0	0	$Z_{4.4.15}$
The load limit sign is missing / incorrect S = 2 - 4, V = 2 - 4	x	x	x	
Drainage of the bridge				
Dependencies: element of construction = equipment, drainage of the bridge				
The fastening of the drainage pipe was corroded	x	x	x	
The fastening parts are missing, outside the traffic area	x	x	x	
The fastening parts are missing, above the traffic surface, V = 1 - 3	x	x	x	
Water leaks from the pipe (above the field)	x	x	x	
Water leaks from the pipe (above the parts of the structure)	0	0	2	$Z_{4.4.16}$
Water leaks from the pipe (above the traffic surfaces)	0	1	2	$Z_{4.4.17}$
Less corrosion damage on drainage pipes	x	x	x	
Significant corrosion damage on drainage pipes	x	x	x	
Missing dilatation of the drainage pipes at the transition of the structure / field	x	x	x	
Rain grid / clogged pipe	0	2	1	$Z_{4.4.18}$
In the raining grid there is a missing catcher of a garbage (pot)	x	x	x	
The drainage grid/ cleaning hole in the hinge plate is not secured	x	x	x	
The drainage grid is broken	x	x	x	
Inspection agents (inspection tools)				
Dependencies: element of construction = equipment, inspection agents				
Cracks on the rail of the inspected vehicle (risk of falling from height)	x	x	x	
Padlock is missing, third parties have unobstructed access to the building site	x	x	x	
Ladders, The distance between rungs is too large (> 280mm)	x	x	x	

Ladders, rungs too close to the building (<150mm)	x	x	x	
Ladder, The distance between the end rungs and the working surface too large (> 100mm)	x	x	x	
Ladders, according to the regulations, the necessary back protection is missing	x	x	x	
Corrosion of inspection agents				
Dependencies: element of construction = equipment, inspection means, damage = surface, metal				
Thickness of anti-corrosion protection too small	x	x	x	
Locally blown (cracked) anti-corrosion protection	x	x	x	
Anti-corrosive protection on large surfaces is blooming	x	x	x	
The beginning of corrosion - locally	x	x	x	
The beginning of corrosion - a large surface	x	x	x	
Corrosion of individual carriers	x	x	x	
Corrosion of several carriers in a row	x	x	x	
(Tools for protection)Means of protection from invading birds				
Dependencies: structural element = protective agents, protection from birds				
protection agents of invading birds are missing / damaged	x	x	x	
Overview of the building site				
Dependencies: damage = review				
The view of the whole building is not possible? a fictitious assessment of the situation is issued	x	x	x	
The building is very overgrown, only partial inspection possible	0	0	2	$Z_{4.4.19}$

Summary Group14

$Z_{4.4.15}$ = 1.0	$\Delta Z_{4.4.15}$= - 0.1	$Z_{4.4.15}$ = 1.0-0.1= 0.9
$Z_{4.4.16}$ = 2.0	$\Delta Z_{4.4.16}$=+ 0.1	$Z_{4.4.16}$ = 2.0+0.1= 2.1
$Z_{4.4.17}$ = 2.1	$\Delta Z_{4.4.17}$= 0	$Z_{4.4.17}$ = 2.1+0= 2.1
$Z_{4.4.18}$ =2.1	$\Delta Z_{4.4.18}$= +0.1	$Z_{4.4.18}$ = 2.1+0.1= 2.2
$Z_{4.4.19}$=2.0	$\Delta Z_{4.4.19}$=0	$Z_{4.4.19}$ = 2.0+0= 2.0
Sum group 14		9,3

4.2. LEVEL 2: MAXIMUM DAMAGE

NO.	Group1	Group2	Group9	Group13	Group11	Group14
	8	5.2	1.0	0	3.2	9,3

4.3. LEVEL 3:

Group	Z	ΔZ	**Z_{BG}**
Group 1	$Z_{4.4.5}$ = 2.0	ΔZ_2 = 0	Z_{BG}= 2.0+0=2.0
Group 2	$Z_{4.4.9}$ = 1.1	ΔZ_2 = -0.1	Z_{BG}= 1.1+0 = 1.1

	$Z_{4.4.10} = 1.1$	$\Delta Z_2 = 0$	
	$Z_{4.4.11} = 1.1$	$\Delta Z_2 = -0.1$	
Group 9	$Z_{4.4.12} = 1.0$	$\Delta Z_2 = 0$	$Z_{BG} = 1.0+0=1.0$
Group 11	$Z_{4.4.14} = 2.0$	$\Delta Z_2 = + 0.1$	$Z_{BG} = 2.0 + 0.1 = 2.1$
Group 13			$Z_{BG} = 0$
Group 14	$Z_{4.4.18} = 2.1$	$\Delta Z_2 = +0.1$	$Z_{BG} = 2.1+0.1 = 2.2$

Level 3 (Group 14 is Maximum)

$Z_{ges} = 2.2$ $\Delta Z_3 = 0$ (GROUP 14 THE MAXIMUM Z_{BG})

Satisfactory condition

The stability and traffic safety of the structure are given.

The stability and/or durability of at least one component group can be impaired.

The durability of the structure can be affected in the long term. A spread of damage or consequential damage to the structure, which in the long term leads to significant impairment of stability and/or traffic safety or increased wear and tear, is possible.

Ongoing maintenance required.

Medium-term repair required.

Measures to eliminate damage or warnings to maintain road safety may become necessary at short term.

5. AL SREEM ROAD BRIDGE

5.1. LEVEL 1: Regular bridge inspection

Group 1: Superstructure

Description of damage and defects	S	V	D	
Bridges, concrete / reinforced concrete / pre-stressed construction structures				
Dependencies: type of construction = bridge, basic building element = structure construction, material of the spanning structure = concrete				
Graffiti on visible surfaces	0	0	0	$Z_{5.5.1}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{5.5.2}$
Less wear of the protective layer	0	0	1	$Z_{5.5.3}$
The rust on the lower sides of the construction	x	x	x	
Pollution of internal passages of the building (remains of the formwork or other)	x	x	x	
Pollution of internal passages of building (bird feces or other)	x	x	x	
Coarse granularity of concrete of the spanning structure	x	x	x	
The protective layer above the auxiliary rebar for the installation of the main rebar is too small	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) poor quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1.0 to 2.9cm) Good quality of concrete	0	0	2	$Z_{5.5.4}$
The protective layer of the main rebar on the underside of the structure is too small (1,0 to 2,9cm) Poor concrete quality	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (below 1.0cm)	x	x	x	
The carbonate front reached the main rebar	x	x	x	
Visible main rebar on the underside of the structure, the reinforcement is lightly corroded (without significant reduction of the cross section)	x	x	x	
The main rebar of the spanning structure lies in the area of carbonization and is slightly corroded (it does not apply to the prefabrication reinforcement)	x	x	x	
Visible main rebar on the underside of the spanning structure, corroded rebar (there is a decrease in the cross-section)	x	x	x	
Blooming (water traces) in the area of heavily corroded main reinforcement on the underside of the spanning structure (advanced reductions in cross-section)	x	x	x	
Blooming in the area of heavily corroded main reinforcement on the underside of the spanning structure (the main reinforcement is partly excluded from the load)	x	x	x	
Partial moisture penetration	0	0	2	$Z_{5.5.5}$
Penetration of moisture on large surfaces	x	x	x	
Bridges, cracks in concrete / reinforced concrete / pre-stressed structure				
Dependencies: type of construction = bridge, basic building element = structure , material of the structure = concrete, damage = cracks				
Surface cracks outside the humidification area (widths) of ≤ 0.1mm in reinforced concrete or prestressed structure (eg cracks from shrinkage)	x	x	x	
Cracks outside the area of humidification (shrinkage) width of 0.1 - <0.2mm in reinforced concrete- or prestressed structure	0	0	1	$Z_{5.5.6}$
Cracks width 0.1 - <0.2mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Surface cracks in the humidification area (widths) 0.2 -≤ 0.4 mm in the RC structure	x	x	x	

Parallel cracks with prestressing of a width of 0.2 -≤ 0.4mm in the area of humidification (squeezing) in the prestressed structure	x	x	x	
Shrinkage widths> 0.4mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (not in the cable extension area)	x	x	x	
Cracks with a width of <0.2mm with prestressed structure (in cable extension)	x	x	x	
Cracks with a width of 0.2 - ≤ 0.4mm at the front of the structure (in the area of cable extension)	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (in the cable extension area)	x	x	x	
Cracks> 0.4mm under load	x	x	x	

Summary Group 1

$Z_{5.5.1}$ = 1.0	$\Delta Z_{5.5.1}$ = 0	$Z_{5.5.1}$ = 1.0+0=1.0
$Z_{5.5.2}$ = 1.0	$\Delta Z_{5.2}$ = +0.1	$Z_{5.5.2}$ = 1.0+0.1=1.1
$Z_{5.5.3}$ = 1.1	$\Delta Z_{5.5.3}$ = 0	$Z_{5.5.3}$ = 1.1+0= 1.1
$Z_{5.5.4}$ = 2.0	$\Delta Z_{5.5.4}$ = -0.1	$Z_{5.5.4}$ = 2.5-0.1= 1,9
$Z_{5.5.5}$ = 2.0	$\Delta Z_{5.5.5}$ = 0	$Z_{5.5.5}$= 2.0+0= 2.0
$Z_{5.5.6}$ = 1.1	$\Delta Z_{5.5.6}$ = -0.1	$Z_{5.5.6}$= 1.1-0.1 = 1.0
Sum group 1		**8.1**

Group 2: Substructure

Bridges, substructure	S	V	D	
Dependencies: type of construction = bridge, basic building element = substructure				
Graffiti on visible surfaces	0	0	0	$Z_{5.5.6}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{5.5.7}$
Less wear of the protective layer	0	0	1	$Z_{5.5.8}$
Less rinses in the area of water flows	0	0	1	$Z_{5.5.9}$
Cleaning of the bearing bench (moldings or other)	x	x	x	
Invalidation of the benches (bird droppings or other)	x	x	x	
Remains of the formwork that press the construction	x	x	x	
Cleaning the bearing bench with accumulated moisture	x	x	x	
Formwork material (polystyrene) on the connection with the structure has not been removed	x	x	x	
Less dropping of stone linings	x	x	x	
The installation cover is not correct / damaged	x	x	x	
Cracking stone wall	1	0	2	$Z_{5.5.10}$
Partial moisture in stone wall / reinforced concrete	0	0	2	$Z_{5.5.11}$
Moisture on large surfaces of stone wall / reinforced concrete	x	x	x	
Dry cracks outside the humidification (spinning) area <0.2mm (no reaction sulfuric acid - RSK)	x	x	x	
Surface cracks in the area of humidification (squeezing), cracks can run water, width <0.2mm (without RSK) REACTION SULFURIC ACID	x	x	x	
Dry cracks outside the humidification area (shrinkage) 0.2 -≤ 0.4mm (without RSK)	0	0	1	$Z_{5.5.12}$

Surface cracks in the area of humidification (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, 0.2 -≤ 0.4mm wide (without RSK)	x	x	x	
Dry cracks outside the area of humidification (shrinkage) of width> 0,4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) of width> 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, width> 0,4mm, unarmed concrete (without RSK)	x	x	x	
Cracks in the area of humidification (sprinkling), cracks can run water, width> 0.4mm, RC bottom structure (without RSK)	x	x	x	

Summary Group 2

$Z_{5.5.6}$= 1.0	$\Delta Z_{5.5.6}$ = 0	$Z_{5.5.6}$ = 1.0+0= 1.0
$Z_{5.5.7}$ = 1.0	$\Delta Z_{5.5.7}$ = +0.1	$Z_{5.5.7}$ = 1.0+0.1= 1.1
$Z_{5.5.8}$ = 1.1	$\Delta Z_{5.5.8}$ = 0	$Z_{5.5.8}$ = 1.1+0= 1.1
$Z_{5.5.9}$ = 1.1	$\Delta Z_{5.5.9}$ = 0	$Z_{5.5.9}$ = 1.1+0= 1.1
$Z_{5.5.10}$ = 2.2	$\Delta Z_{5.5.10}$ = 0	$Z_{5.5.10}$ = 2.2+0= 2.2
$Z_{5.5.11}$ = 2.0	$\Delta Z_{5.5.11}$ =- 0.1	$Z_{5.5.11}$ = 2.0 -0.1= 1.9
$Z_{5.5.12}$ = 1.1	$\Delta Z_{5.5.10}$ = - 0.1	$Z_{5.5.12}$ = 1.1-0.1= 1.0
Sum group 2		**9.4**

Group 9: Transition devices (joint)

Transition devices (joints)	S	V	D	
Dependencies: structural element = transitional device				
Contaminated transitional device (moving still possible)	0	0	1	$Z_{5.5.13}$
A highly Contaminated transient device (scrolling limited)	x	x	x	
Loosening the fixing of the profiles in the carpet of the structures, the profile is still held	x	x	x	
Loosening the fixing of the profiles in the carpet construction, the profile loosened	x	x	x	
	x	x	x	
Rubber profile dropped or multiple damaged	x	x	x	
Asphalt crossings, thin asphalt mass (rough, small open cracks)	0	0	0	$Z_{5.5.14}$
The asphalt crossing cracked and depressed	x	x	x	
The transient device is missing, the spanning structure is cracked at the ends	x	x	x	
The transient device is missing, the spanning structure at the ends is cracked and shrugged	x	x	x	

Summary Group 9

$Z_{5.5.13}$ = 1.1	$\Delta Z_{5.5.13}$ = -0.1	$Z_{5.5.13}$ = 1.1-0.1= 1

$Z_{5.5.14}$ = 1.0	$\Delta Z_{5.5.14}$ = 0	$Z_{5.5.14}$= 1.0+0= 1
Sum group 9		**2.0**

Group 13: Fence

Protective means	S	V	D	
Fence				
Dependencies: element of construction = protective means, fence				
The compound fence - construction of the defect	x	x	x	
There is no fence, there are bumpers at> 50km / h, no pedestrian traffic is planned	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, small building, no pedestrian traffic is foreseen (temporary hazard sign exists)	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, pedestrian traffic planned	x	x	x	
Missing wire in the handrail of the fence, the length of the building is <20m	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, there is a bumper	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, the bumper is missing	x	x	x	
Fence height is not in accordance with regulations (difference ≤ 5cm)	x	x	x	
Fence height is not in accordance with regulations (difference 5 - 10cm)	x	x	x	
Fence height is not in accordance with regulations (difference > 10cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference ≤ 2cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference > 2cm)	x	x	x	
Missing individual fence filling rods	x	x	x	
Missing more consecutive fence filling rods	x	x	x	
Guardrail				
Dependencies: structural element = protection agent, bumper				
Bumper height is not in accordance with regulations (difference ≤ 3cm)	x	x	x	
Bumper height is not in accordance with regulations (difference > 3cm)	x	x	x	
The bumper is partially deformed	x	x	x	
a fence and a bumper are missing	x	x	x	
Weak anchor fasteners of the bumper / fence	x	x	x	
There is a lack of anchoring of protective devices on the length of the move	x	x	x	
Corrosion of protective agents				
Dependencies: structural element = protective agents, damage = surface, metal				
Insufficient thickness of the protective layer	0	0	2	$Z_{5.5.15}$
Local scattering (breaking) of the protective layer	0	0	1	$Z_{5.5.16}$
Scattering (breaking) of the protective layer on larger surfaces	x	x	x	
Local start of corrosion	x	x	x	
Corrosion of large surfaces	x	x	x	
The corrosion of individual support elements of the protecting agents	x	x	x	
The corrosion of more consecutive support elements of the protecting agents	x	x	x	

Summary Group 13

$Z_{5.5.15}$ =2.0	$\Delta Z_{5.5.15}$ = 0	$Z_{5.5.15}$ = 2.0+0= 2
$Z_{5.5.16}$=1.1	$\Delta Z_{5.5.16}$ = 0	$Z_{5.5.16}$ = 1.1+0= 1.1
Sum group 13		**3.1**

Group 11: Road surface

Road surface	S	V	D	
Dependencies: structural element = useful surface				
pavement				
Subsidence of the pavement behind the abutments (≤ 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm), there are warning signs	x	x	x	
Drainage does not work, the risk of drifting	0	2	0	$Z_{5.5.17}$
Paving grooves / indentations, depth <1cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm, there are warning signs	x	x	x	
Pulley grooves / indentations, depth> 3cm	x	x	x	
Paving grooves / indentations, depth> 3cm, there are warning signs	x	x	x	
Bubbles, heights of ≤ 2cm	x	x	x	
Bubbles, height 2 - 5cm	x	x	x	
Bubbles, height 2 - 5cm, there are warning signs	x	x	x	
Bubbles, height > 5cm	x	x	x	
Bubbles, height > 5cm, there are warning signs	x	x	x	
Impact hole, depth ≤ 2cm	x	x	x	
Impact hole, depth 2 - 5cm	x	x	x	
Impact hole, depth 2 - 5cm, there are warning signs	x	x	x	
Impact hole, depth > 5cm	x	x	x	
Impact hole, depth> 5cm, there are warning signs	x	x	x	
A pedestrian hallway	x	x	x	
Erosion of surface layer <2cm	0	1	0	$Z_{5.5.18}$
Erosion of surface layer ≥2cm	x	x	x	
Erosion of surface layer ≥2cm, there are warning signs	x	x	x	
The layers break and fall in pieces	x	x	x	
Slipping risk	x	x	x	

Summary Group 11

$Z_{5.5.17}$ =2.0	$\Delta Z_{5.5.17}$= +0.1	$Z_{5.5.17}$ = 2.0+0.1= 2.1
$Z_{5.5.18}$ =1.1	$\Delta Z_{5.5.18}$= -0.1	$Z_{5.5.18}$ = 1.1-0.1= 1.0
Sum group 11		**3.1**

Group 14: Other

Signs	S	V	D	
Dependencies: structural element = Signs				
Missing building designation number	0	0	0	$Z_{5.5.19}$
The load limit sign is missing / incorrect S = 2 - 4, V = 2 - 4	2	2	0	$Z_{5.5.20}$
Drainage of the bridge				
Dependencies: element of construction = equipment, drainage of the bridge				
The fastening of the drainage pipe was corroded	x	x	x	
The fastening parts are missing, outside the traffic area	x	x	x	
The fastening parts are missing, above the traffic surface, V = 1 - 3	x	x	x	
Water leaks from the pipe (above the field)	x	x	x	
Water leaks from the pipe (above the parts of the structure)	0	0	2	$Z_{5.5.21}$
Water leaks from the pipe (above the traffic surfaces)	0	1	2	$Z_{5.5.22}$
Less corrosion damage on drainage pipes	0	0	1	$Z_{5.5.23}$
Significant corrosion damage on drainage pipes	x	x	x	
Missing dilatation of the drainage pipes at the transition of the structure / field	x	x	x	
Rain grid / clogged pipe	0	2	1	$Z_{5.5.24}$
In the raining grid there is a missing catcher of a garbage (pot)	x	x	x	
The drainage grid/ cleaning hole in the hinge plate is not secured	x	x	x	
The drainage grid is broken	x	x	x	
Inspection agents (inspection tools)				
Dependencies: element of construction = equipment, inspection agents				
Cracks on the rail of the inspected vehicle (risk of falling from height)	x	x	x	
Padlock is missing, third parties have unobstructed access to the building site	x	x	x	
Ladders, The distance between rungs is too large (> 280mm)	x	x	x	
Ladders, rungs too close to the building (<150mm)	x	x	x	
Ladder, The distance between the end rungs and the working surface too large (> 100mm)	x	x	x	
Ladders, according to the regulations, the necessary back protection is missing	x	x	x	
Corrosion of inspection agents				
Dependencies: element of construction = equipment, inspection means, damage = surface, metal				
Thickness of anti-corrosion protection too small	x	x	x	
Locally blown (cracked) anti-corrosion protection	x	x	x	
Anti-corrosive protection on large surfaces is blooming	x	x	x	
The beginning of corrosion - locally	x	x	x	
The beginning of corrosion - a large surface	x	x	x	
Corrosion of individual carriers	x	x	x	
Corrosion of several carriers in a row	x	x	x	
(Tools for protection) Means of protection from invading birds				
Dependencies: structural element = protective agents, protection from birds				
protection agents of invading birds are missing / damaged	x	x	x	
Overview of the building site				

Dependencies: damage = review				
The view of the whole building is not possible? a fictitious assessment of the situation is issued	x	x	x	
The building is very overgrown, only partial inspection possible	0	0	2	$Z_{5.5.25}$

Summary Group 14

$Z_{5.5.19}$ = 1.0	$\Delta Z_{5.5.19}$= -0.1	$Z_{5.5.19}$= 1.0-0.1= 0.9
$Z_{5.5.20}$ = 2.3	$\Delta Z_{5.5.20}$= -0.1	$Z_{5.5.20}$= 2.3-0.1= 2.2
$Z_{5.5.21}$ = 2.0	$\Delta Z_{5.5.21}$= 0	$Z_{5.5.21}$= 2.0+0= 2.0
$Z_{5.5.22}$ = 2.1	$\Delta Z_{5.5.22}$= -0.1	$Z_{5.5.22}$ = 2.1-0.1= 2
$Z_{5.5.23}$ = 1.1	$\Delta Z_{5.5.23}$= -0.1	$Z_{5.5.23}$ = 1.1-0.1= 1
$Z_{5.5.24}$ =2.1	$\Delta Z_{5.5.24}$= 0	$Z_{5.5.24}$ = 2.1+0= 2.1
$Z_{5.5.25}$ =2.0	$\Delta Z_{5.5.25}$=0	$Z_{5.5.25}$ = 2.0+0= 2.0
Sum group 14		**12.2**

5.2. LEVEL 2: MAXIMUM DAMAGE

NO.	Group1	Group2	Group9	Group13	Group11	Group14
	8.1	9.4	2.0	3.1	3.1	12.2

5.3. LEVEL 3:

Group	Z	ΔZ	Z_{BG}
Group 1	$Z_{5.5.4}$ = 2.0	ΔZ_2 = -0.1	Z_{BG}= 2.4-0.1=1,9
Group 2	$Z_{5.5.10}$ = 2.2	ΔZ_2 = -0.1	Z_{BG}= 2.2-0.1 = 2.1
Group 9	$Z_{5.5.13}$ = 1.0	ΔZ_2 = -0.1	Z_{BG}= 1.0-0.1=1.0
Group 11	$Z_{5.5.17}$ = 2.0	ΔZ_2= -0.1	Z_{BG}= 2.0 -0.1= 1.9
Group 13	$Z_{5.5.15}$= 2.0	ΔZ_2 = -0.1	Z_{BG}= 2.0-0.1=1.9
Group 14	$Z_{5.5.20}$ = 2.3	ΔZ_2 = 0	Z_{BG}= 2.3 +0= 2.3

Z_{ges}= 2.3 ΔZ_3 = 0 (GROUP 14 THE MAXIMUM Z_{BG})

Satisfactory condition

The stability and traffic safety of the structure are given.

The stability and/or durability of at least one component group can be impaired.

The durability of the structure can be affected in the long term. A spread of damage or consequential damage to the structure, which in the long term leads to significant impairment of stability and/or traffic safety or increased wear and tear, is possible.

Ongoing maintenance required.

Medium-term repair required.

Measures to eliminate damage or warnings to maintain road safety may become necessary at short term.

6. ALSHAAB PORT BRIDGE

6.1. LEVEL 1: Regular bridge inspection

Group 1: Superstructure

Description of damage and defects	S	V	D	
Bridges, concrete / reinforced concrete / pre-stressed construction structures				
Dependencies: type of construction = bridge, basic building element = structure construction,				

material of the spanning structure = concrete				
Graffiti on visible surfaces	0	0	0	$Z_{6.6.1}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{6.6.2}$
Less wear of the protective layer	0	0	1	$Z_{6.6.3}$
The rust on the lower sides of the construction	x	x	x	
Pollution of internal passages of the building (remains of the formwork or other)	x	x	x	
Pollution of internal passages of building (bird feces or other)	x	x	x	
Coarse granularity of concrete of the spanning structure	x	x	x	
The protective layer above the auxiliary rebar for the installation of the main reinforcement is too small	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) poor quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1.0 to 2.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1,0 to 2,9cm) Poor concrete quality	0	0	2	$Z_{6.6.4}$
The protective layer of the main rebar on the underside of the structure is too small (below 1.0cm)	x	x	x	
The carbonate front reached the main rebar	x	x	x	
Visible main rebar on the underside of the structure, the rebar is lightly corroded (without significant reduction of the cross section)	x	x	x	
The main rebar of the spanning structure lies in the area of carbonization and is slightly corroded (it does not apply to the prefabrication reinforcement)	x	x	x	
Visible main rebar on the underside of the spanning structure, corroded rebar (there is a decrease in the cross-section)	x	x	x	
Blooming (water traces) in the area of heavily corroded main reinforcement on the underside of the spanning structure (advanced reductions in cross-section)	x	x	x	
Blooming in the area of heavily corroded main reinforcement on the underside of the spanning structure (the main reinforcement is partly excluded from the load)	x	x	x	
Partial moisture penetration	0	0	2	$Z_{6.6.5}$
Penetration of moisture on large surfaces	x	x	x	
Description of damage / defect	s	v	d	
Surface cracks outside the humidification area (widths) of ≤ 0.1mm in reinforced concrete or prestressed structure (eg cracks from shrinkage)	x	x	x	
Cracks outside the area of humidification (shrinkage) width of 0.1 - <0.2mm in reinforced concrete- or prestressed structure	0	0	1	$Z_{6.6.6}$
Cracks width 0.1 - <0.2mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Surface cracks in the humidification area (widths) 0.2 -≤ 0.4 mm in the RC structure	x	x	x	
Parallel cracks with prestressing of a width of 0.2 -≤ 0.4mm in the area of humidification (squeezing) in the prestressed structure	x	x	x	
Shrinkage widths> 0.4mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (not in the cable extension area)	x	x	x	
Cracks with a width of <0.2mm with prestressed structure (in cable extension)	x	x	x	
Cracks with a width of 0.2 - ≤ 0.4mm at the front of the structure (in the area of cable extension)	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (in the cable extension area)	x	x	x	
Cracks> 0.4mm under load	x	x	x	

Summary Group 1

$Z_{6.6.1}$ = 1.0	$\Delta Z_{6.6.1}$ = -0.1	$Z_{6.6.1}$ = 1.0-0.1=0.9
$Z_{6.6.2}$ = 1.0	$\Delta Z_{6.6.2}$ = 0	$Z_{6.6.2}$ = 1.0+0=1
$Z_{6.6.3}$ = 1.1	$\Delta Z_{6.6.3}$ = 0	$Z_{6.6.3}$ = 1.0+0=1
$Z_{6.6.4}$ = 2.0	$\Delta Z_{6.6.4}$ = 0	$Z_{6.6.4}$ = 2.0+0=2
$Z_{6.6.5}$ = 2.0	$\Delta Z_{6.6.5}$ = -0.1	$Z_{6.6.5}$ = 2.0-0.1= 1.9
$Z_{6.6.6}$ = 1.1	$\Delta Z_{6.6.6}$ = -0.1	$Z_{6.6.6}$ = 1.1-0.1= 1.0
Sum group 1		**7.8**

Group 2: Substructure

Bridges, substructure	S	V	D	
Dependencies: type of construction = bridge, basic building element = substructure				
Graffiti on visible surfaces	0	0	0	$Z_{6.6.7}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{6.6.8}$
Less wear of the protective layer	0	0	1	$Z_{6.6.9}$
Less rinses in the area of water flows	0	0	1	$Z_{6.6.10}$
Cleaning of the bearing bench (moldings or other)	x	x	x	
Invalidation of the benches (bird droppings or other)	x	x	x	
Remains of the formwork that press the construction	x	x	x	
Cleaning the bearing bench with accumulated moisture	x	x	x	
Formwork material (polystyrene) on the connection with the structure has not been removed	x	x	x	
Less dropping of stone linings	x	x	x	
The installation cover is not correct / damaged	x	x	x	
Cracking stone wall	x	x	x	
Partial moisture stone wall / reinforced concrete	x	x	x	
Moisture on large surfaces of stone wall / reinforced concrete	x	x	x	
Bridges, cracks in concrete- / RC substructure				
Dependencies: type of construction = bridge, basic building element = substructure, damage = cracks				
Dry cracks outside the humidification (spinning) area <0.2mm (no reaction sulfuric acid - RSK)	x	x	x	
Surface cracks in the area of humidification (squeezing), cracks can run water, width <0.2mm (without RSK) REACTION SULFURIC ACID	x	x	x	
Dry cracks outside the humidification area (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, 0.2 -≤ 0.4mm wide (without RSK)	x	x	x	
Dry cracks outside the area of humidification (shrinkage) of width> 0,4mm (without RSK)	0	0	2	$Z_{6.6.11}$
Surface cracks in the area of humidification (shrinkage) of width> 0.4mm (without RSK)	x	x	x	
Description of damage / defect	S	V	D	
Cracks in the area of humidification (cracks), cracks can run water, width> 0,4mm, unarmed concrete (without RSK)	x	x	x	

Cracks in the area of humidification (sprinkling), cracks can run water, width> 0.4mm, RC bottom structure (without RSK)	x	x	x	

Summary Group 2

$Z_{6.6.7}$= 1.0	$\Delta Z_{6.6.7}$= -0.1	$Z_{6.6.7}$ = 1.0-0.1= 0.9
$Z_{6.6.8}$ = 1.0	$\Delta Z_{6.6.8}$ = 0	$Z_{6.6.8}$ = 1.0+0= 1
$Z_{6.6.9}$ = 1.1	$\Delta Z_{6.6.9}$ = - 0.1	$Z_{6.6.9}$= 1.1-0.1= 1
$Z_{6.6.10}$ = 1.1	$\Delta Z_{6.6.10}$ = - 0.1	$Z_{6.6.10}$= 1.1-0.1= 1
$Z_{6.6.11}$ = 2.0	$\Delta Z_{6.6.11}$ = - 0.1	$Z_{6.6.11}$= 2-0.1= 1.9
Sum group 2		**5.8**

Group 9: Transition devices

Transition devices (joints)	S	V	D	
Dependencies: structural element = transitional device				
Contaminated transitional device (moving still possible)	x	x	x	
A highly Contaminated transient device (scrolling limited)	x	x	x	
Loosening the fixing of the profiles in the carpet of the structures, the profile is still held	x	x	x	
Loosening the fixing of the profiles in the carpet construction, the profile loosened	x	x	x	
	x	x	x	
Rubber profile dropped or multiple damaged	x	x	x	
Asphalt crossings, thin asphalt mass (rough, small open cracks)	x	x	x	
The asphalt crossing cracked and depressed	x	x	x	
The transient device is missing, the spanning structure is cracked at the ends	x	x	x	
The transient device is missing, the spanning structure at the ends is cracked and shrugged	x	x	x	

Summary Group 9

Sum group 9	**0**

Group 13: Fence

Protective means	S	V	D	
Fence				
Dependencies: element of construction = protective means, fence				
The compound fence - construction of the defect	x	x	x	
There is no fence, there are bumpers at> 50km / h, no pedestrian traffic is planned	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, small building, no pedestrian traffic is foreseen (temporary hazard sign exists)	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, pedestrian traffic	x	x	x	

planned				
Missing wire in the handrail of the fence, the length of the building is <20m	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, there is a bumper	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, the bumper is missing	x	x	x	
Fence height is not in accordance with regulations (difference ≤ 5cm)	x	x	x	
Fence height is not in accordance with regulations (difference 5 - 10cm)	x	x	x	
Fence height is not in accordance with regulations (difference > 10cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference ≤ 2cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed (difference > 2cm)	x	x	x	
Missing individual fence filling rods	x	x	x	
Missing more consecutive fence filling rods	x	x	x	
Guardrail				
Dependencies: structural element = protection agent, bumper				
Bumper height is not in accordance with regulations (difference ≤ 3cm)	x	x	x	
Bumper height is not in accordance with regulations (difference > 3cm)	x	x	x	
The bumper is partially deformed	x	x	x	
a fence and a bumper are missing	x	x	x	
Weak anchor fasteners of the bumper / fence	x	x	x	
There is a lack of anchoring of protective devices on the length of the move	x	x	x	
Corrosion of protective agents				
Dependencies: structural element = protective agents, damage = surface, metal				
Insufficient thickness of the protective layer	x	x	x	
Local scattering (breaking) of the protective layer	x	x	x	
Scattering (breaking) of the protective layer on larger surfaces	x	x	x	
Local start of corrosion	x	x	x	
Corrosion of large surfaces	x	x	x	
The corrosion of individual support elements of the protecting agents	x	x	x	
The corrosion of more consecutive support elements of the protecting agents	x	x	x	

Summary Group 13

Sum group 11	**0**

Group 11: Road surface

Road surface	S	V	D	
Dependencies: structural element = useful surface				
pavement				
Subsidence of the pavement behind the abutments (≤ 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm), there are warning signs	x	x	x	
Drainage does not work, the risk of drifting	0	2	0	$Z_{6.6.12}$
Paving grooves / indentations, depth <1cm	0	1	0	$Z_{6.6.13}$

Pulley grooves / indentations, depth 1 - 3cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm, there are warning signs	x	x	x	
Pulley grooves / indentations, depth> 3cm	x	x	x	
Paving grooves / indentations, depth> 3cm, there are warning signs	x	x	x	
Bubbles, heights of ≤ 2cm	x	x	x	
Bubbles, height 2 - 5cm	x	x	x	
Bubbles, height 2 - 5cm, there are warning signs	x	x	x	
Bubbles, height > 5cm	x	x	x	
Bubbles, height > 5cm, there are warning signs	x	x	x	
Impact hole, depth ≤ 2cm	x	x	x	
Impact hole, depth 2 - 5cm	x	x	x	
Impact hole, depth 2 - 5cm, there are warning signs	x	x	x	
Impact hole, depth > 5cm	x	x	x	
Impact hole, depth> 5cm, there are warning signs	x	x	x	
A pedestrian hallway	x	x	x	
Erosion of surface layer <2cm	0	1	0	$Z_{6.6.14}$
Erosion of surface layer ≥2cm	x	x	x	
Erosion of surface layer ≥2cm, there are warning signs	x	x	x	
The layers break and fall in pieces	x	x	x	
Slipping risk	x	x	x	

Summary Group 11

$Z_{6.6.12}$ =2.0	$\Delta Z_{6.6.12}$= -0.1	$Z_{6.6.12}$ = 2.0-0.1= 1.9
$Z_{6.6.13}$ =1.1	$\Delta Z_{6.6.13}$= -0.1	$Z_{6.6.13}$ = 1.1-0.1= 1.0
$Z_{6.6.14}$ =1.1	$\Delta Z_{6.6.14}$= -0.1	$Z_{6.6.14}$ = 1.1-0.1= 1.0
Sum group 11		**3.9**

Group 14: Other

Signs	S	V	D	
Dependencies: structural element = Signs				
Missing building designation number	0	0	0	$Z_{6.6.15}$
The load limit sign is missing / incorrect S = 2 - 4, V = 2 - 4	2	2	0	$Z_{6.6.16}$
Drainage of the bridge				
Dependencies: element of construction = equipment, drainage of the bridge				
The fastening of the drainage pipe was corroded	x	x	x	
The fastening parts are missing, outside the traffic area	x	x	x	
The fastening parts are missing, above the traffic surface, V = 1 - 3	x	x	x	
Water leaks from the pipe (above the field)	x	x	x	
Water leaks from the pipe (above the parts of the structure)	x	x	x	
Water leaks from the pipe (above the traffic surfaces)	x	x	x	
Less corrosion damage on drainage pipes	x	x	x	

Significant corrosion damage on drainage pipes	x	x	x	
Missing dilatation of the drainage pipes at the transition of the structure / field	x	x	x	
Rain grid / clogged pipe	0	2	1	$Z_{6.6.17}$
In the raining grid there is a missing catcher of a garbage (pot)	x	x	x	
The drainage grid/ cleaning hole in the hinge plate is not secured	x	x	x	
The drainage grid is broken	x	x	x	
Inspection agents (inspection tools)				
Dependencies: element of construction = equipment, inspection agents				
Cracks on the rail of the inspected vehicle (risk of falling from height)	x	x	x	
Padlock is missing, third parties have unobstructed access to the building site	x	x	x	
Ladders, The distance between rungs is too large (> 280mm)	x	x	x	
Ladders, rungs too close to the building (<150mm)	x	x	x	
Ladder, The distance between the end rungs and the working surface too large (> 100mm)	x	x	x	
Ladders, according to the regulations, the necessary back protection is missing	x	x	x	
Corrosion of inspection agents				
Dependencies: element of construction = equipment, inspection means, damage = surface, metal				
Thickness of anti-corrosion protection too small	x	x	x	
Locally blown (cracked) anti-corrosion protection	x	x	x	
Anti-corrosive protection on large surfaces is blooming	x	x	x	
The beginning of corrosion - locally	x	x	x	
The beginning of corrosion - a large surface	x	x	x	
Corrosion of individual carriers	x	x	x	
Corrosion of several carriers in a row	x	x	x	
(Tools for protection) Means of protection from invading birds				
Dependencies: structural element = protective agents, protection from birds				
protection agents of invading birds are missing / damaged	x	x	x	
Overview of the building site				
Dependencies: damage = review				
The view of the whole building is not possible? a fictitious assessment of the situation is issued	x	x	x	
The building is very overgrown, only partial inspection possible	0	0	2	$Z_{6.6.18}$

Summary Group 14

$Z_{6.6.15}$ =1	$\Delta Z_{6.6.15}$= -0.1	$Z_{6.6.15}$ = 1-0.1= 0.9
$Z_{6.6.16}$ =2.3	$\Delta Z_{6.6.16}$= -0.1	$Z_{6.6.16}$ = 2.3-0.1= 2.2
$Z_{6.6.17}$ =2.1	$\Delta Z_{6.6.17}$= 0	$Z_{6.6.17}$ = 2.1+0= 2.1
$Z_{6.6.18}$ =2.0	$\Delta Z_{6.6.18}$=0	$Z_{6.6.18}$ = 2.0+0= 2.0
Sum group 14		**7.2**

6.2. LEVEL 2: MAXIMUM DAMAGE

NO.	Group 1	Group 2	Group 9	Group 13	Group 11	Group 14
	7.8	5.8	0	0	3.9	7.2

6.3. LEVEL 3:

Group	Z	ΔZ	Z_{BG}
Group 1	$Z_{6.6.4} = 2.0$	$\Delta Z_2 = 0$	$Z_{BG} = 2.0+0=2.0$
	$Z_{6.6.5} = 2.0$	$\Delta Z_2 = -0.1$	
Group 2	$Z_{6.6.11} = 2.0$	$\Delta Z_2 = -0.1$	$Z_{BG} = 2.0-0.1 = 1.9$
	$Z_{2.2.12} = 1.1$	$\Delta Z_2 = -0.1$	
	$Z_{2.2.13} = 1.1$	$\Delta Z_2 = -0.1$	
Group 9			$Z_{BG} = 0$
Group 11	$Z_{6.6.12} = 2.0$	$\Delta Z_2 = -0.1$	$Z_{BG} = 2.0 -0.1= 1.9$
Group 13			$Z_{BG} = 0$
Group 14	$Z_{6.6.16} = 2.3$	$\Delta Z_2 = -0.1$	$Z_{BG} = 2.3 -0.1= 2.2$

$Z_{ges} = 2.2$ $\Delta Z_3 = 0$ (GROUP 14 THE MAXIMUM Z_{BG})

Satisfactory condition

The stability and traffic safety of the structure are given.

The stability and/or durability of at least one component group can be impaired.

The durability of the structure can be affected in the long term. A spread of damage or consequential damage to the structure, which in the long term leads to significant impairment of stability and/or traffic safety or increased wear and tear, is possible.

Ongoing maintenance required.

Medium-term repair required.

Measures to eliminate damage or warnings to maintain road safety may become necessary at short term.

7. ABDUL SALAM AREF BRIDGE

7.1. LEVEL 1: Regular bridge inspection

Group 1: Superstructure

Description of damage and defects	S	V	D	
Bridges, concrete / reinforced concrete / pre-stressed construction structures				
Dependencies: type of construction = bridge, basic building element = structure construction, material of the spanning structure = concrete				
Graffiti on visible surfaces	0	0	0	$Z_{7.7.1}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{7.7.2}$
Less wear of the protective layer	x	x	x	
The rust on the lower sides of the construction	x	x	x	
Pollution of internal passages of the building (remains of the formwork or other)	x	x	x	
Pollution of internal passages of building (bird feces or other)	x	x	x	
Coarse granularity of concrete of the spanning structure	x	x	x	
The protective layer above the auxiliary rebar for the installation of the main rebar is too small	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (3.0 to 3.9cm) poor quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1.0 to 2.9cm) Good quality of concrete	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (1,0 to 2,9cm) Poor concrete quality	x	x	x	
The protective layer of the main rebar on the underside of the structure is too small (below 1.0cm)	x	x	x	
The carbonate front reached the main rebar	x	x	x	
Visible main rebar on the underside of the structure, the reinforcement is lightly corroded (without significant reduction of the cross section)	x	x	x	
The main rebar of the spanning structure lies in the area of carbonization and is slightly corroded (it does not apply to the prefabrication reinforcement)	x	x	x	
Visible main rebar on the underside of the spanning structure, corroded rebar (there is a decrease in the cross-section)	x	x	x	
Blooming (water traces) in the area of heavily corroded main reinforcement on the underside of the spanning structure (advanced reductions in cross-section)	x	x	x	
Blooming in the area of heavily corroded main reinforcement on the underside of the spanning structure (the main reinforcement is partly excluded from the load)	x	x	x	
Partial moisture penetration	0	0	2	$Z_{7.7.3}$
Penetration of moisture on large surfaces	x	x	x	
Surface cracks outside the humidification area (widths) of ≤ 0.1mm in reinforced concrete or prestressed structure (eg cracks from shrinkage)	0	0	0	$Z_{7.7.4}$
Cracks outside the area of humidification (shrinkage) width of 0.1 - <0.2mm in reinforced concrete- or prestressed structure	x	x	x	
Cracks width 0.1 - <0.2mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Surface cracks in the humidification area (widths) 0.2 -≤ 0.4 mm in the RC structure	x	x	x	
Parallel cracks with prestressing of a width of 0.2 -≤ 0.4mm in the area of humidification (squeezing) in the prestressed structure	x	x	x	
Shrinkage widths> 0.4mm in the area of humidification (shrinkage) for RC structure	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (not in the cable extension area)	x	x	x	
Cracks with a width of <0.2mm with prestressed structure (in cable extension)	x	x	x	

Cracks with a width of 0.2 - ≤ 0.4mm at the front of the structure (in the area of cable extension)	x	x	x	
Cracks with a width of> 0.4mm with a prestressed structure (in the cable extension area)	x	x	x	
Cracks> 0.4mm under load	x	x	x	

Summary Group 1

$Z_{7.7.1}$ = 1.0	$\Delta Z_{7.7.1}$ = -0.1	$Z_{7.7.1}$ = 1.0-0.1=0.9
$Z_{7.7.2}$ = 1.0	$\Delta Z_{7.7.2}$ = -0.1	$Z_{7.7.2}$ = 1.0-0.1=0.9
$Z_{7.7.3}$ = 2.0	$\Delta Z_{7.7.3}$ = -0.1	$Z_{7.7.3}$ = 2.0-0.1= 1.9
$Z_{7.7.4}$ = 1.0	$\Delta Z_{7.7.4}$ = -0.1	$Z_{7.7.4}$ = 1.0-0.1= 0.9
Sum group 1		**4.6**

Group 2: Substructure

Bridges, substructure	S	V	D	
Dependencies: type of construction = bridge, basic building element = substructure				
Graffiti on visible surfaces	0	0	0	$Z_{7.7.5}$
Visible changes on concrete from the effect of weather conditions	0	0	0	$Z_{7.7.6}$
Less wear of the protective layer	x	x	x	
Less rinses in the area of water flows	x	x	x	
Cleaning of the bearing bench (moldings or other)	x	x	x	
Invalidation of the benches (bird droppings or other)	x	x	x	
Remains of the formwork that press the construction	x	x	x	
Cleaning the bearing bench with accumulated moisture	x	x	x	
Formwork material (polystyrene) on the connection with the structure has not been removed	x	x	x	
Less dropping of stone linings	x	x	x	
The installation cover is not correct / damaged	x	x	x	
Cracking stone wall	x	x	x	
Partial moisture on stone wall / reinforced concrete	x	x	x	
Moisture on large surfaces of stone wall / reinforced concrete	x	x	X	
Dry cracks outside the humidification (spinning) area <0.2mm (no reaction sulfuric acid - RSK)	x	x	x	
Surface cracks in the area of humidification (squeezing), cracks can run water, width <0.2mm (without RSK) REACTION SULFURIC ACID	x	x	x	
Dry cracks outside the humidification area (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) 0.2 -≤ 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, 0.2 -≤ 0.4mm wide (without RSK)	x	x	x	
Dry cracks outside the area of humidification (shrinkage) of width> 0,4mm (without RSK)	x	x	x	
Surface cracks in the area of humidification (shrinkage) of width> 0.4mm (without RSK)	x	x	x	
Cracks in the area of humidification (cracks), cracks can run water, width> 0,4mm, unarmed concrete (without RSK)	x	x	x	
Cracks in the area of humidification (sprinkling), cracks can run water, width> 0.4mm, RC bottom structure (without RSK)	x	x	x	

Summary Group 2

$Z_{7.7.5}$= 1.0	$\Delta Z_{7.7.5}$ = -0.1	$Z_{7.7.5}$ = 1.0-0.1= 0.9
$Z_{7.7.6}$ = 1.0	$\Delta Z_{7.7.6}$ = -0.1	$Z_{7.7.6}$ = 1.0-0.1= 0.9
Sum group 2		**1.8**

Group 9: Transition devices

Transition devices (joints)	S	V	D	
Dependencies: structural element = transitional device				
Contaminated transitional device (moving still possible)	x	x	x	
A highly Contaminated transient device (scrolling limited)	x	x	x	
Loosening the fixing of the profiles in the carpet of the structures, the profile is still held	x	x	x	
Loosening the fixing of the profiles in the carpet construction, the profile loosened	x	x	x	
	x	x	x	
Rubber profile dropped or multiple damaged	x	x	x	
Asphalt crossings, thin asphalt mass (rough, small open cracks)	0	0	0	$Z_{7.7.7}$
The asphalt crossing cracked and depressed	x	x	x	
The transient device is missing, the spanning structure is cracked at the ends	x	x	x	
The transient device is missing, the spanning structure at the ends is cracked and shrugged	x	x	x	

Summary Group 9

$Z_{7.7.7}$ = 1.0	$\Delta Z_{7.7.7}$ = -0.1	$Z_{7.7.7}$ = 1.0-0.1= 0.9
Sum group 9		**0.9**

Group 13: Fence

Protective means	S	V	D	
Fence				
Dependencies: element of construction = protective means, fence				
The compound fence - construction of the defect	x	x	x	
There is no fence, there are bumpers at> 50km / h, no pedestrian traffic is planned	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, small building, no pedestrian traffic is foreseen (temporary hazard sign exists)	x	x	x	
There are no fences or segments of the fence outside the end of the bridge, pedestrian traffic planned	x	x	x	
Missing wire in the handrail of the fence, the length of the building is <20m	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, there is a bumper	x	x	x	
Missing wire in the handrail of the fence, the length of the building is ≥20m, the bumper is missing	x	x	x	
Fence height is not in accordance with regulations (difference ≤ 5cm)	x	x	x	
Fence height is not in accordance with regulations (difference 5 - 10cm)	x	x	x	
Fence height is not in accordance with regulations (difference > 10cm)	x	x	x	
The distance between the vertical fences filling rods is greater than the regulation allowed	x	x	x	

(difference ≤ 2cm)				
The distance between the vertical fences filling rods is greater than the regulation allowed (difference > 2cm)	x	x	x	
Missing individual fence filling rods	x	x	x	
Missing more consecutive fence filling rods	x	x	x	
Guardrail				
Dependencies: structural element = protection agent, bumper				
Bumper height is not in accordance with regulations (difference ≤ 3cm)	x	x	x	
Bumper height is not in accordance with regulations (difference > 3cm)	x	x	x	
The bumper is partially deformed	x	x	x	
A fence and a bumper are missing	x	x	x	
Weak anchor fasteners of the bumper / fence	x	x	x	
There is a lack of anchoring of protective devices on the length of the move	x	x	x	
Corrosion of protective agents				
Dependencies: structural element = protective agents, damage = surface, metal				
Insufficient thickness of the protective layer	x	x	x	
Local scattering (breaking) of the protective layer	x	x	x	
Scattering (breaking) of the protective layer on larger surfaces	x	x	x	
Local start of corrosion	x	x	x	
Corrosion of large surfaces	x	x	x	
The corrosion of individual support elements of the protecting agents	x	x	x	
The corrosion of more consecutive support elements of the protecting agents	x	x	x	

Summary Group 13

Sum group 9	0

Group 11: Road surface

Road surface	S	V	D	
Dependencies: structural element = useful surface				
pavement				
Subsidence of the pavement behind the abutments (≤ 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm)	x	x	x	
Subsidence of the pavement behind the abutments (> 2cm), there are warning signs	x	x	x	
Drainage does not work, the risk of drifting	0	2	0	$Z_{7.7.8}$
Paving grooves / indentations, depth <1cm	0	1	0	$Z_{7.7.9}$
Pulley grooves / indentations, depth 1 - 3cm	x	x	x	
Pulley grooves / indentations, depth 1 - 3cm, there are warning signs	x	x	x	
Pulley grooves / indentations, depth> 3cm	x	x	x	
Paving grooves / indentations, depth> 3cm, there are warning signs	x	x	x	
Bubbles, heights of ≤ 2cm	x	x	x	
Bubbles, height 2 - 5cm	x	x	x	
Bubbles, height 2 - 5cm, there are warning signs	x	x	x	
Bubbles, height > 5cm	x	x	x	
Bubbles, height > 5cm, there are warning signs	x	x	x	

Impact hole, depth ≤ 2cm	x	x	x	
Impact hole, depth 2 - 5cm	x	x	x	
Impact hole, depth 2 - 5cm, there are warning signs	x	x	x	
Impact hole, depth > 5cm	x	x	x	
Impact hole, depth> 5cm, there are warning signs	x	x	x	
A pedestrian hallway	x	x	x	
Erosion of surface layer <2cm	0	1	0	$Z_{7.7.10}$
Erosion of surface layer ≥2cm	x	x	x	
Erosion of surface layer ≥2cm, there are warning signs	x	x	x	
The layers break and fall in pieces	x	x	x	
Slipping risk	x	x	x	

Summary Group 11

$Z_{7.7.8}$ =2.0	$\Delta Z_{7.7.8}$= 0	$Z_{7.7.8}$ = 2.0+0= 2
$Z_{7.7.9}$ =1.1	$\Delta Z_{7.7.9}$= -0.1	$Z_{7.7.9}$ = 1.1-0.1= 1
$Z_{7.7.10}$ =1.1	$\Delta Z_{7.7.10}$= -0.1	$Z_{7.7.10}$ = 1.1-0.1= 1
Sum group 11		4

Group 14: Other

Signs	S	V	D	
Dependencies: structural element = Signs				
Missing building designation number	0	0	0	$Z_{7.7.11}$
The load limit sign is missing / incorrect S = 2 - 4, V = 2 - 4	2	2	0	$Z_{7.7.12}$
Drainage of the bridge				
Dependencies: element of construction = equipment, drainage of the bridge				
The fastening of the drainage pipe was corroded	x	x	x	
The fastening parts are missing, outside the traffic area	x	x	x	
The fastening parts are missing, above the traffic surface, V = 1 - 3	x	x	x	
Water leaks from the pipe (above the field)	x	x	x	
Water leaks from the pipe (above the parts of the structure)	x	x	x	
Water leaks from the pipe (above the traffic surfaces)	x	x	x	
Less corrosion damage on drainage pipes	x	x	x	
Significant corrosion damage on drainage pipes	x	x	x	
Missing dilatation of the drainage pipes at the transition of the structure / field	x	x	x	
Rain grid / clogged pipe	0	2	1	$Z_{7.7.13}$
In the raining grid there is a missing catcher of a garbage (pot)	x	x	x	
The drainage grid/ cleaning hole in the hinge plate is not secured	x	x	x	
The drainage grid is broken	x	x	x	
Inspection agents (inspection tools)				
Dependencies: element of construction = equipment, inspection agents				
Cracks on the rail of the inspected vehicle (risk of falling from height)	x	x	x	

Padlock is missing, third parties have unobstructed access to the building site	x	x	x	
Ladders, The distance between rungs is too large (> 280mm)	x	x	x	
Ladders, rungs too close to the building (<150mm)	x	x	x	
Ladder, The distance between the end rungs and the working surface too large (> 100mm)	x	x	x	
Ladders, according to the regulations, the necessary back protection is missing	x	x	x	
Corrosion of inspection agents				
Dependencies: element of construction = equipment, inspection means, damage = surface, metal				
Thickness of anti-corrosion protection too small	x	x	x	
Locally blown (cracked) anti-corrosion protection	x	x	x	
Anti-corrosive protection on large surfaces is blooming	x	x	x	
The beginning of corrosion - locally	x	x	x	
The beginning of corrosion - a large surface	x	x	x	
Corrosion of individual carriers	x	x	x	
Corrosion of several carriers in a row	x	x	x	
(Tools for protection) Means of protection from invading birds				
Dependencies: structural element = protective agents, protection from birds				
protection agents of invading birds are missing / damaged	x	x	x	
Overview of the building site				
Dependencies: damage = review				
The view of the whole building is not possible? a fictitious assessment of the situation is issued	x	x	x	
The building is very overgrown, only partial inspection possible	0	0	2	$Z_{7.7.14}$

Summary Group 14

$Z_{7.7.11}$ =1.0	$\Delta Z_{7.7.11}$= -0.1	$Z_{7.7.11}$ = 1.0-0.1= 0.9
$Z_{7.7.12}$ =2.3	$\Delta Z_{7.7.12}$= -0.1	$Z_{7.7.12}$ = 2.3-0.1= 2.2
$Z_{7.7.13}$ =2.1	$\Delta Z_{7.7.13}$= -0.1	$Z_{7.7.13}$ = 2.1-0.1= 2
$Z_{7.7.14}$ =2.0	$\Delta Z_{7.7.14}$=0	$Z_{7.7.14}$ = 2.0+0= 2
Sum group 14		7.1

7.2. LEVEL 2: MAXIMUM DAMAGE

NO.	Group1	Group2	Group9	Group13	Group11	Group14
	4.6	1.8	0.9	0	4	7.1

7.3. LEVEL 3:

Group	Z	ΔZ	Z_{BG}
Group 1	$Z_{7.7.3}$ = 2.0	ΔZ_2 = 0	Z_{BG}= 2.0+0=2.0
Group 2	$Z_{7.7.5}$ = 1.0	ΔZ_2 = 0	Z_{BG}= 1.0+0 = 1.0
	$Z_{7.7.6}$ = 1.0	ΔZ_2 = -0.1	

Group 9	$Z_{7.7.7} = 1.0$	$\Delta Z_2 = -0.1$	$Z_{BG} = 1.0-0.1 = 0.9$
Group 11	$Z_{7.7.9} = 2.0$	$\Delta Z_2 = 0$	$Z_{BG} = 2.0 + 0 = 2.0$
Group 13			$Z_{BG} = 0$
Group 14	$Z_{7.7.13} = 2.3$	$\Delta Z_2 = 0$	$Z_{BG} = 2.3 + 0 = 2.3$

$Z_{ges} = 2,3$ $\Delta Z3 = 0$ (GROUP 14 THE MAXIMUM ZBG)

Satisfactory condition

The stability and traffic safety of the structure are given.

The stability and/or durability of at least one component group can be impaired.

The durability of the structure can be affected in the long term. A spread of damage or consequential damage to the structure, which in the long term leads to significant impairment of stability and/or traffic safety or increased wear and tear, is possible.

Ongoing maintenance required.

Medium-term repair required.

Measures to eliminate damage or warnings to maintain road safety may become necessary at short term.

8. RANKING OF ANLYSED BRIDGES AFTER REPAIR

By evaluating the seven bridges after repair, it was found that the bridges that showed damage were:

- Al Sreem Road Bridge Damage Rate Was 2.3
- Abdul Salam Aref Bridge Damage Rate Was 2.3

As these bridges need continuous maintenance and repair is required in the medium term.
As the bladder of the skeleton can be affected in the long term, causing the damage to aggravate.

The bridges are then arranged in terms of the percentage of damage and repairs are carried out after giving priority to maintenance for the previous two bridges:

- Souk Athultha 1 Bridge Damage Rate Was 2.2
- Bab Bin Gheshir Road Bridge Damage Rate Was 2.2
- Alshaab Port Bridge Damage Rate Was 2.2
- Souk Athultha 2 Bridge Damage Rate Was 2.1
- Alseeka Road Bridge Damage Rate Was 2.0

According to the calculated rating all bridges have same damage category and belong to the group of structures with „satisfactory condition“ (2,0-2,4), for which the following description is given in german BMS:

- The stability and traffic safety of the structure are given.
- The stability and/or durability of at least one component group can be impaired.
- The durability of the structure can be affected in the long term. A spread of damage or consequential damage to the structure, which in the long term leads to significant impairment of stability and/or traffic safety or increased wear and tear, is possible.
- Ongoing maintenance required.
- Medium-term repair required.
- Measures to eliminate damage or warnings to maintain road safety may become necessary at short term.

CHAPTER XI

ANALYSIS AND DISCUSSION

CHAPTER XI

ANALYSIS AND DISCUSSION

1. COMPARATIVE ANALYSIS OF RESULTS OF ASSESSMENT OF BRIDGES BEFORE REPAIR

In the aim of making general conclusion on condition of all tested bridges it is necessary to find out the main causes of deterioration and damage appearance. For this purpose, several comparative analyses were performed. The first group of analyses covers the in-situ testing results, such as:

– Carbonization depth,
– Concrete compressive strength
– Concrete tensile strength
– Concrete density and
– Chloride ion content

The second group of analyses encompasses the results of visual inspection i.e. defects and damages, some of which are:

– Concrete cover depth,
– Reinforcement corrosion,
– Spalling off of concrete, etc.

The comparison within the same property/damage/defect is done in accordance with bridge element.

1.1 Analyses of the in-situ testing results

In order to do mentioned analyses, the table XI-1 is formed. The all-necessary data are summarized by name of bridges and by the element of the bridge for the chosen property.

Table XI-1. Data for comparative analyses of in situ tested properties of concrete

		Souk Athulatha 1	Souk Athulatha 2	Al Sseka road	Bab bin Gheshir road	Al Sreem road	Al Shaab port	Abdul salam aref
Number of bridge		1	2	3	4	5	6	7
carbonation depth (mm)	Deck ceiling	50	27	60	42	10	60	37
	Ceiling beam					18	46	
	Supporting wall	12	26	16	46			26
	Abutment			79	38			75
compressive strength	Deck ceiling	22	34	-	-			
	Lateral beam	31,75	38,50	23	23	25	28,33	34,33
	Supporting wall	31,33	27,14	21,86	21,86			
Pull-off method (MPa)		Smaller than	Smaller than	Smaller than	smaller than	smaller than	Smaller than	Smaller than

		minimum required value	minimum required value	minimum required value	minimum required value	minimum required value	minimum required value	minimum required value
Density (kg/m^3)	Deck ceiling	2111	2239	-	-	-	-	-
	Ceiling beam	-	2272	2181	2274	2273	2290	2290
	Lateral beam	2271	-	-	-	-	-	-
	Supporting wall	2292	2288	2185	2225	-	-	-
	Abutment	-	-	-	-	-	-	2356

CARBONATION

The process of carbonization of concrete has already started in all concrete bridge elements. The average, minimum and maximum values of carbonization depth for deck ceiling slabs, supporting walls, abutments and ceiling beams are shown in charts (Fig. XI-1 to XI-4) respectively.

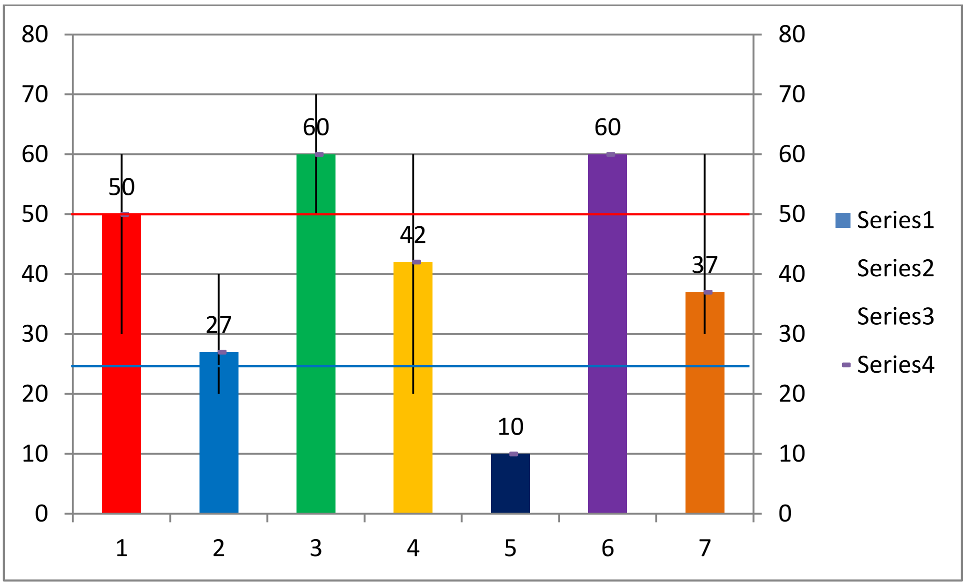


Figure (XI-1) Average, minimum and maximum values of carbonation depth for deck ceiling slabs

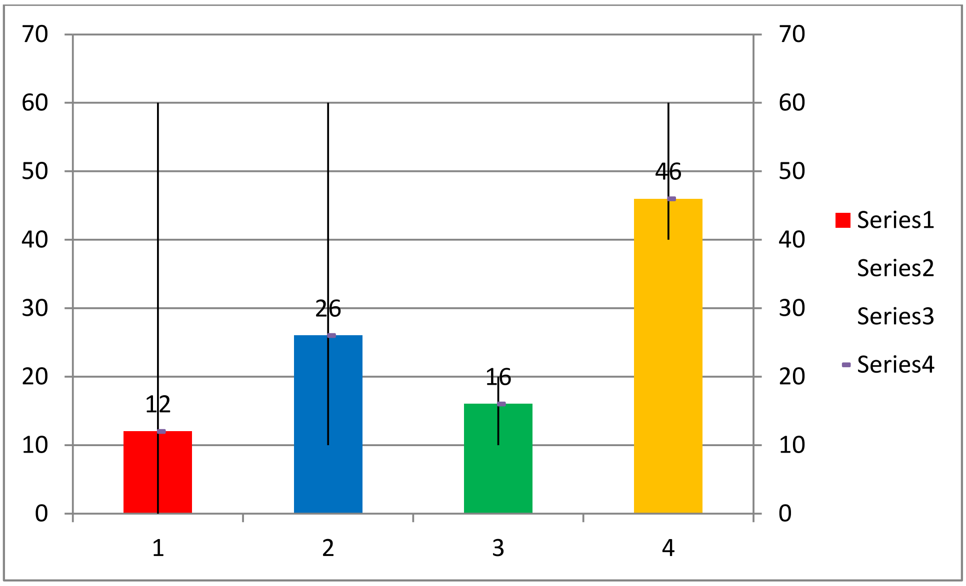


Figure (XI-2) Average, minimum and maximum values of carbonation depth for supporting walls

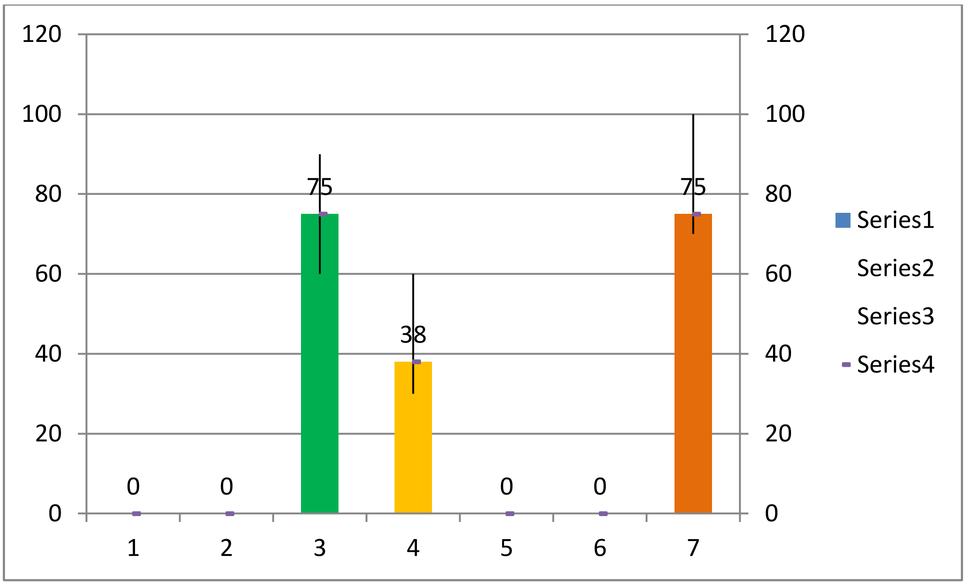


Figure (XI-3) Average, minimum and maximum values of carbonation depth for abutments

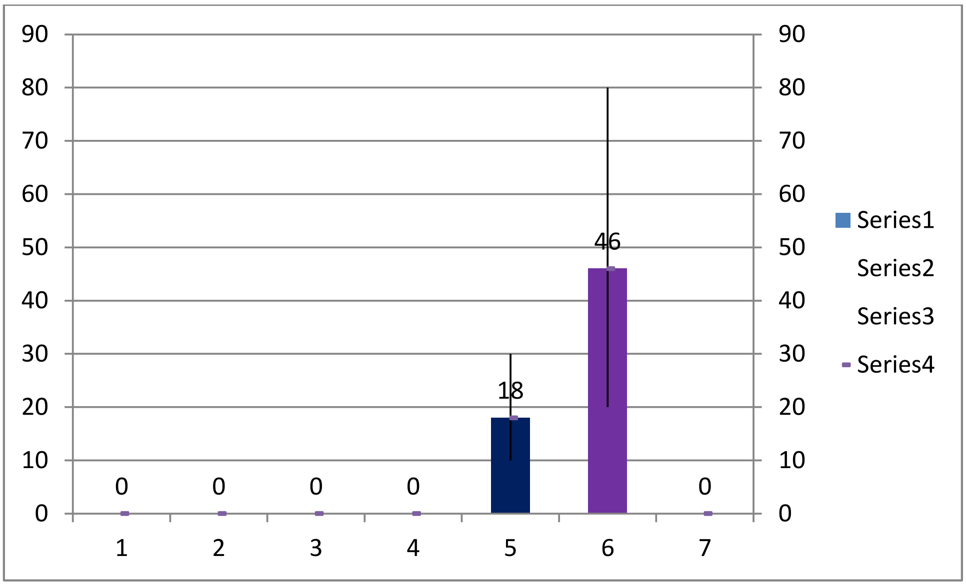


Figure (XI-4) Average, minimum and maximum values of carbonation depth for ceiling beams

On the basis of shown results, it can be seen that the depth of carbonization has a very large dispersion for the same bridge element within the same bridge, as well as among all tested bridges. As a criterion for analysis of carbonization depth the following limit value is chosen:

- Lower limited value: 0,5mm/year x 50years= 25mm
- Upper limited value: 1mm/year x 50years= 50mm

The summarized conclusion obtained by comparing the data shown in figures XI-1 to XI-4 with chosen limit lower and upper values are given in table (XI-2).

Table XI-2. Results of analysis of fulfillment of posted criteria

	Deck ceiling slab	Supporting wall	Abutment	Ceiling beam
Lower limit value	6/7	2/4	3/3	1/2
Upper limit value	3/7	0/4	2/3	0/2

By analyzing the data from table XI-2 it can be seen that:

The lower limited value is achieved or even exceeded in 12/16 (75%) cases.

The upper limited value is achieved or even exceeded in 5/16 (31%) cases.

The carbonization is most expressed in deck ceiling slabs and abutments. In both cases the depth of carbonization is larger than the upper limit value.

Finally, it can be concluded that the progress of carbonization in tested concrete elements in all seven of Tripoli's bridges is within the expected limits. However, in 25% the depth of carbonization overpasses the expected values.

COMPRESSIVE STRENGTH

The average, minimum and maximum values of compressive strength for abutment, deck ceiling , lateral beam and supporting walls are shown in charts (Fig.XI-5 to XI-8) respectively.

The criterion for this analysis is: Compressive strength lower limit value is 30MPa.

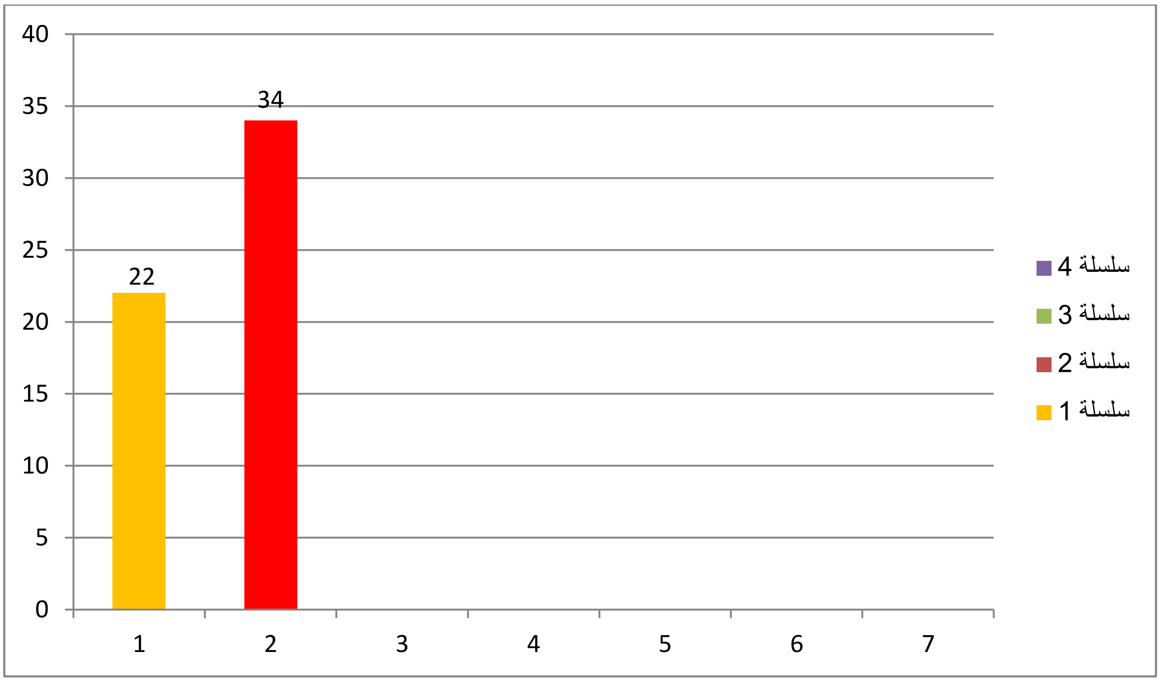


Figure (XI-6) Average, minimum and maximum values of Compressive strength for deck ceiling

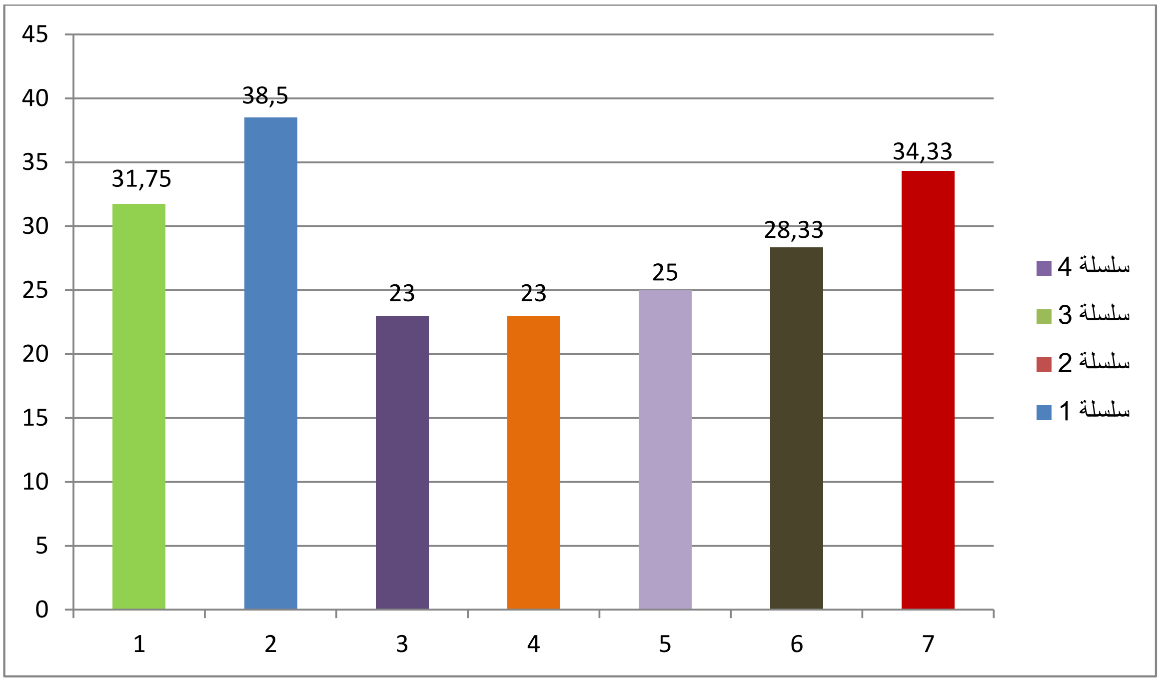


Figure (XI-7) Average, minimum and maximum values of Compressive strength for lateral beam

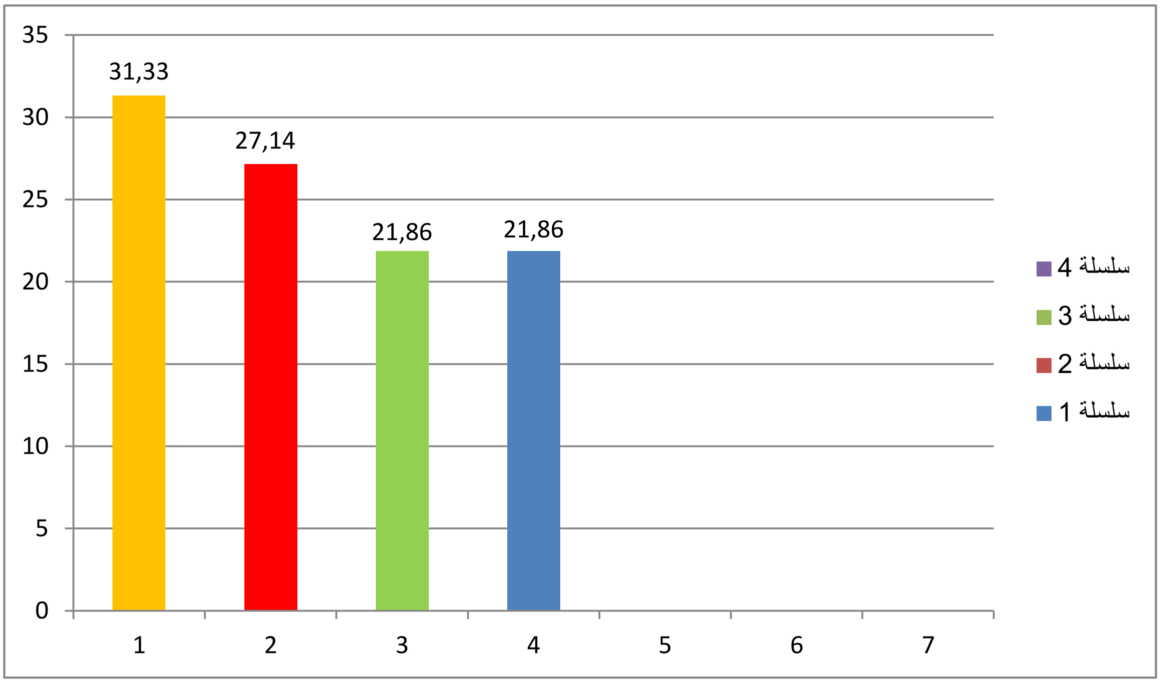


Figure (XI- 8) Average, minimum and maximum values of Compressive strength for supporting wall

Through the chart for the compressive strength, it was noticed that the highest value was in the Souk athulatha 2 Bridge in lateral beam. It was recorded 38 MPa which is higher than the minimum compressive strength 30MPa.

CHLORIDE ION CONTENT

By the analysis of data for chloride ion content in concrete for all seven bridges the following conclusions were derived:

- All testing results were smaller than the criteria value.
- Chloride content in concrete is not hazardous to embedded reinforced bars.

PULL-OFF

The concrete tensile strength was tested by a pull-off test. Through analysis of all obtained results, it was noticed that all values of tensile strengths did not satisfy the required criterion (>1.5MPa). The surface layers of built-in concrete in all tested bridges have bad quality. The main causes might be the usage of dusty aggregate or inadequate curing of concrete during hardening.

DENSITY

The average, minimum and maximum values of density for deck ceiling, ceiling beam, lateral beam, supporting walls and abutment are shown in charts (Fig.XI-9 to XI-13) respectively.

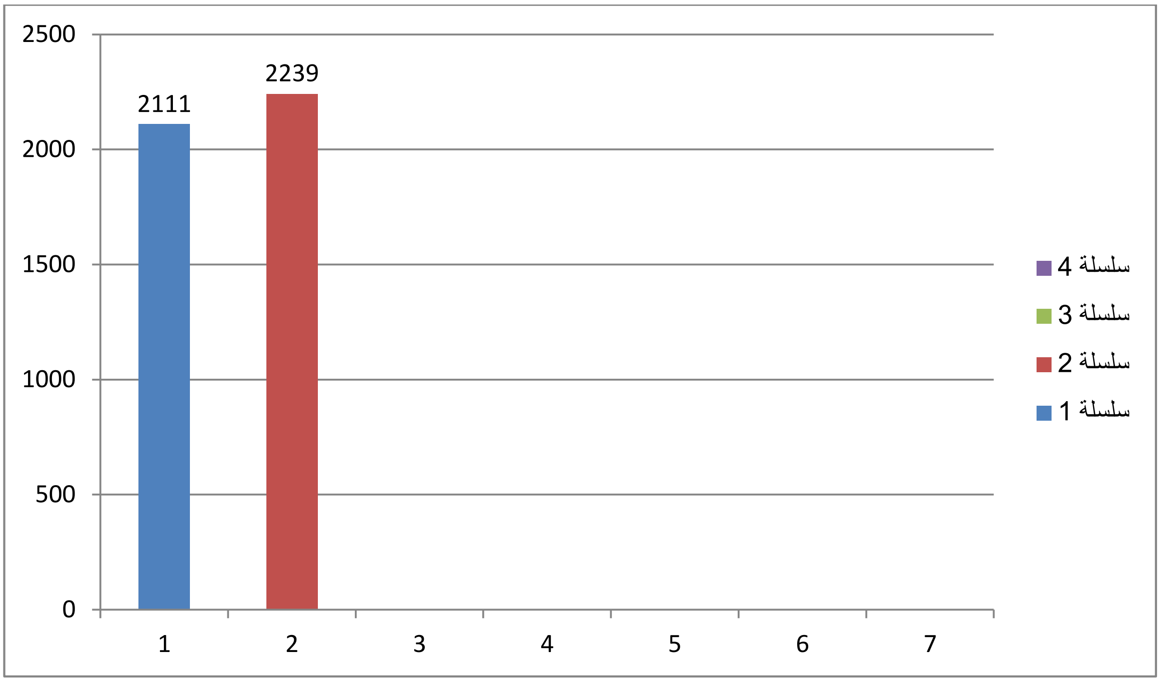


Figure (XI-9) Average, minimum and maximum values of density for deck ceiling

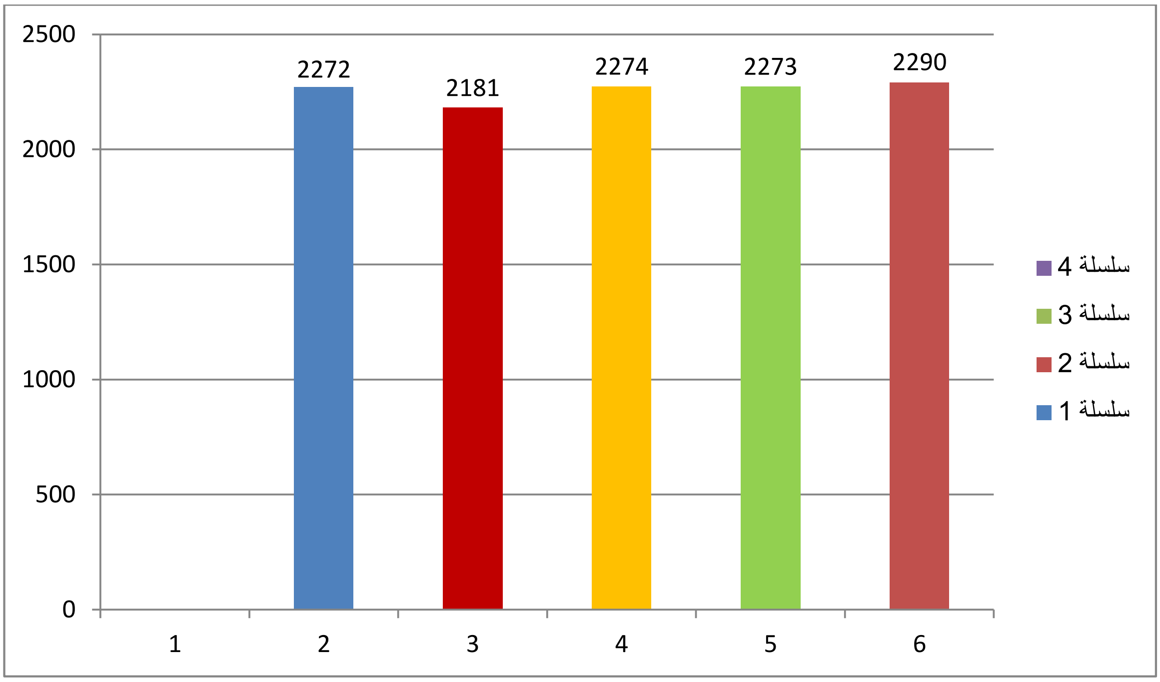


Figure (XI-10) Average, minimum and maximum values of density for ceiling beam

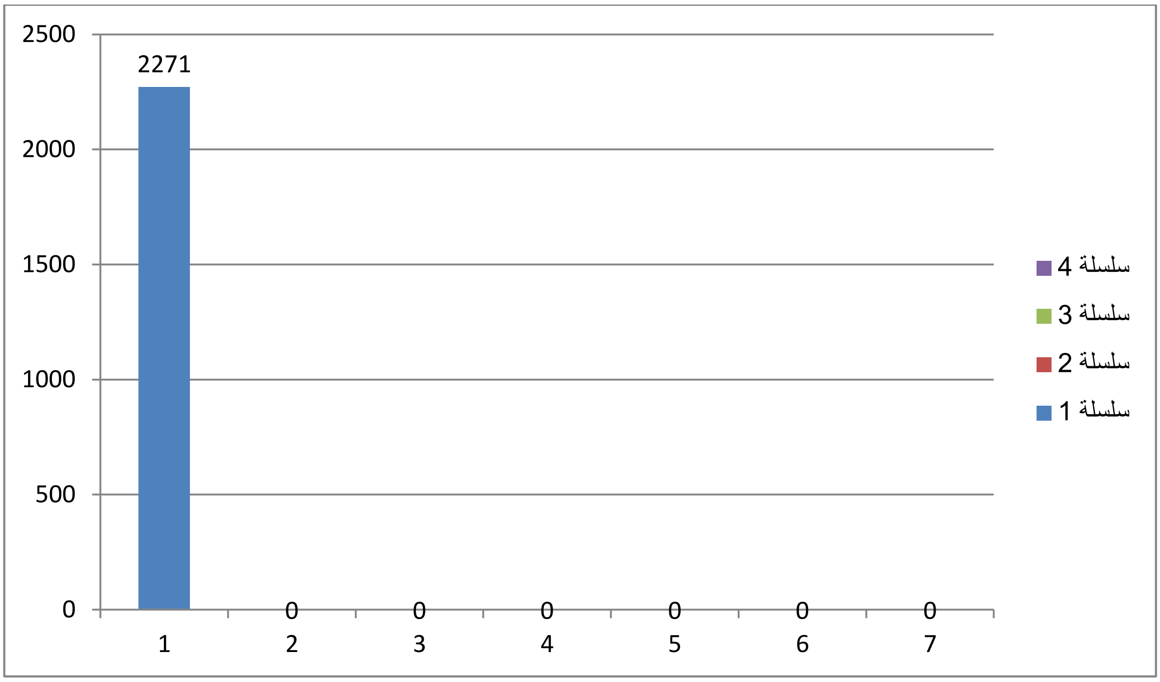


Figure (XI-11) Average, minimum and maximum values of density for lateral beam

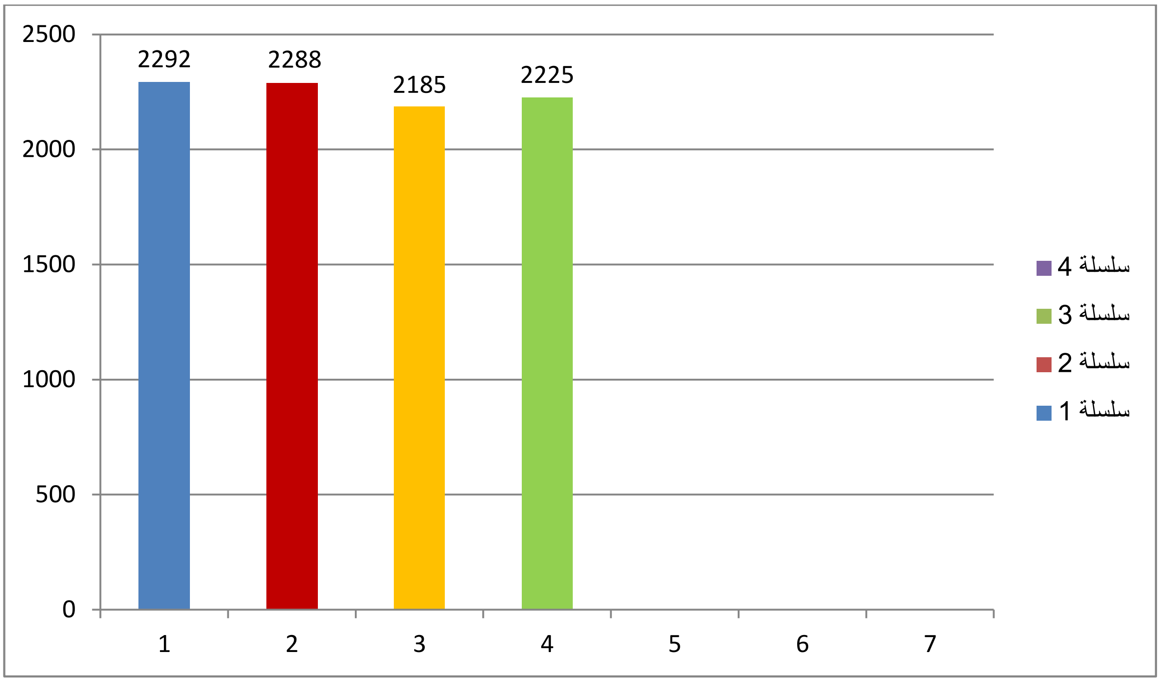


Figure (XI-12) Average, minimum and maximum values of density for supporting wall

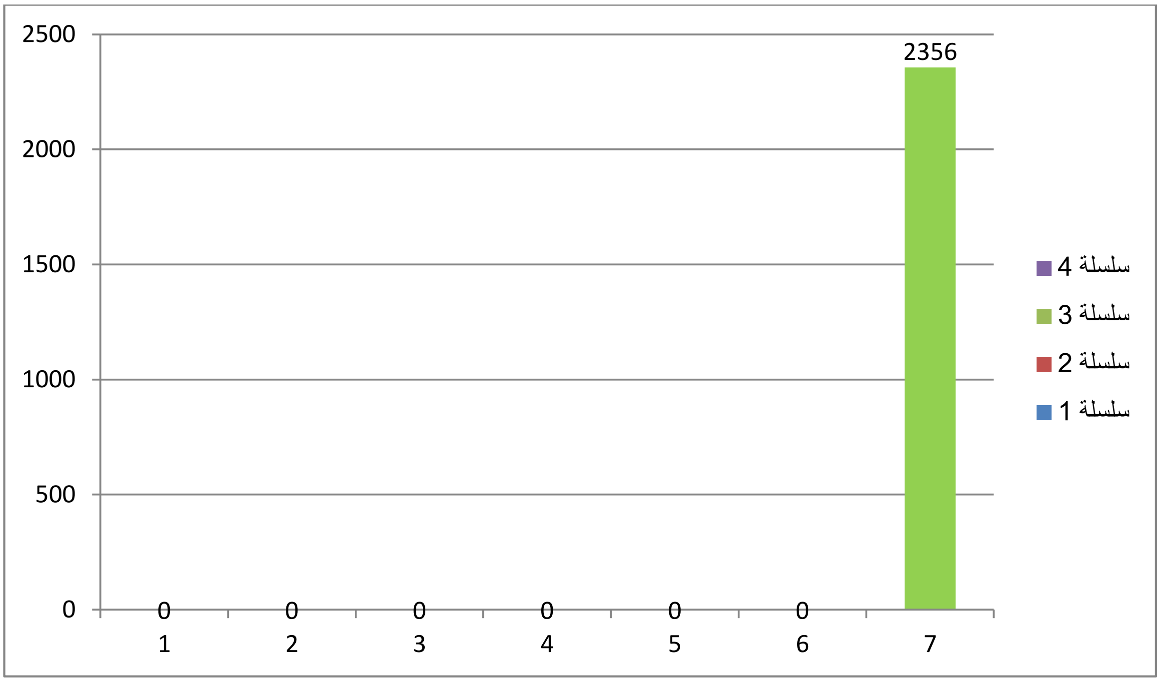


Figure (XI-13) Average, minimum and maximum values of density for abutment

Through the density chart, it was found that all values were less than the minimum value of the concrete density of 2300 kg/m^3 except for the abutment of the seventh bridge, Abdul Salam Aref, the value was greater than the minimum, where it recorded

2356 kg/m^3.

The criterion for this analysis is: Minimum value of concrete density is 2300 kg /m^3.

1.2 Analyses of the results of visual inspection

Table XI-3. Results of visual inspection before repair

Damages	RC elements	Souk Athulatha 1	Souk athulatha 2	Al Sseka road	Bab bin Gheshir road	Al Sreem road	Al Shaab port	Abdul salam aref
Concrete cover depth	Lateral beam	10-70mm	-	-	-	-	-	-
	Deck ceiling Arch cantilever slabs	10-70mm	10-20mm	-	-	-	-	-
	Deck ceiling Simple beam slab	-	-	-	-	-	-	-
	Deck ceiling	-	-	10-20mm	0mm	20mm	5mm	-
	Deck ceiling - Longitudinal supporting beams	-	-	-	10-40mm	-	-	-
	Main and secondary deck ceiling beams	-	-	-	-	10-30mm	-	-
	Longitudinal and transversal supporting (ceiling) RC beams	-	-	-	-	-	5mm	-
	cantilever slabs	10-70mm	-	-	-	-	-	-
	Tunnel ceiling	-	-	-	-	-	-	-
	Supporting walls	20mm	20mm	30-90mm	20-30mm	-	-	-
	Supporting wall (Masonry support walls made of stone)	-	-	-	-	-	-	-
	Abutment	20mm	20-50mm	-	30-60mm	-	-	100mm
	Abutment wall (Masonry wall made of stone and covered by plastering)	-	-	-	-	-	-	-
	Abutment Additional RC layer	-	-	-	-	-	-	-
	Support column	-	-	-	-	-	-	50mm
	Expansion joints	-	-	-	-	-	-	-
Reinforcement corrosion	Lateral beam	+	-	-	+	-	-	-
	Deck ceiling Arch cantilever slabs	-	-	+	-	-	-	-
	Deck ceiling Simple beam slab	+	-	-	-	-	-	-
	Deck ceiling	+	+	-	-	-	-	+
	Deck ceiling - Longitudinal supporting beams	-	-	-	-	-	-	-
	Main and secondary deck ceiling beams	-	-	-	-	-	+	-
	Longitudinal and transversal supporting	-	-	-	-	+	-	-

	(ceiling) RC beams							
	cantilever slabs	+	+	-	+	-	+	-
	Tunnel ceiling	-	+	-	-	-	-	-
	Supporting walls	-	+	-	-	-	-	-
	Supporting wall (Masonry support walls made of stone)	-	-	-	-	-	-	-
	Abutment	+	-	-	-	-	-	-
	Abutment wall (Masonry wall made of stone and covered by plastering)	-	-	-	-	-	-	-
	Abutment Additional RC layer	-	-	-	-	-	-	-
	Support column	-	+	-	-	-	-	+
	Expansion joints	-	-	-	-	-	-	-
-Spalling off of concrete	Lateral beam	+ local	-	-	+ local	-	-	-
	Deck ceiling Arch cantilever slabs	-	-	-	-	-	-	-
	Deck ceiling Simple beam slab	+ local	-	-	-	-	-	-
	Deck ceiling	+ local	+ local	-	-	-	-	+ local
	Deck ceiling - Longitudinal supporting beams	-	-	-	-	-	-	-
	Main and secondary deck ceiling beams	-	-	-	-	-	+ local	-
	Longitudinal and transversal supporting (ceiling) RC beams	-	-	-	-	-	-	-
	cantilever slabs	+ local	+ local	-	+ local	+ local	-	-
	Tunnel ceiling	-	-	-	-	-	-	-
	Supporting walls	-	-	-	+ local	-	-	-
	Supporting wall (Masonry support walls made of stone)	-	-	-	-	-	-	-
	Abutment	-	-	-	-	-	-	+ local
	Abutment wall (Masonry wall made of stone and covered by plastering)	-	-	-	-	-	-	-
	Abutment Additional RC layer	-	-	-	-	-	-	-
	Support column	-	-	-	-	-	-	+ local
	Expansion joints	-	-	-	-	+ local	-	-
Cracks	Lateral beam	-	-	-	-	-	-	-
	Deck ceiling Arch cantilever slabs	-	-	-	-	-	-	-
	Deck ceiling Simple beam slab	-	-	-	-	-	-	-
	Deck ceiling	-	-	-	-	-	-	+
	Deck ceiling -	-	-	-	-	-	-	-

	Longitudinal supporting beams							
	Main and secondary deck ceiling beams	-	-	-	-	-	-	-
	Longitudinal and transversal supporting (ceiling) RC beams	-	-	-	-	-	-	-
	cantilever slabs	-	-	-	-	-	+ Longitudinal crack along the corner rebar	-
	Tunnel ceiling	-	-	-	-	-	-	-
	Supporting walls	-	+	-	-	-	-	-
	Supporting wall (Masonry support walls made of stone)	-	-	-	-	-	-	-
	Abutment	-	-	-	-	-	-	-
	Abutment wall (Masonry wall made of stone and covered by plastering)	-	-	-	-	-	-	-
	Abutment Additional RC layer	-	-	-	-	-	-	-
	Support column	-	-	-	-	-	-	+ vertical cracks
	Expansion joints	-	-	-	-	-	-	-

2. COMPARATIVE ANALYSIS OF RESULTS OF ROUTINE INSPECTION OF BRIDGES AFTER REPAIR

In the aim of making general conclusion on condition of all tested bridges after repair and conclusion on the success of bridge rehabilitation, it is necessary to perform several comparative analyses. The first group of analyses covers the in-situ testing of carbonization depth.

The second group of analyses encompasses the results of visual inspection after repair (damages), some of which are:

- Cracks,
- Pilling off protecting paint,
- Spalling off of concrete,
- Water leakage/traces.

The comparison within the same property/damage/defect is done in accordance with bridge element.

2.1 Analyses of the in-situ testing results of carbonation depth

In order to do analyse of depth of carbonation, the table XI-4 is formed. The all-necessary data are summarized by name of bridges and by the element of bridge for chosen property.

Table XI-4 Data about elements where carbonation depth was measured, after repair

	RC elements	Souk Athulatha 1	Souk athulatha 2	Al Sseka road	Bab bin Gheshir road	Al Sreem road	Al Shaab port	Abdul salam aref
Carbonation								
	Lateral beam	Not measured	Not measured	Not measured	-	-	-	-
	Deck ceiling Arch cantilever slabs	+	Not measured	Not measured	-	-	-	-
	Deck ceiling Simple beam slab	+	+ (in progress)	+	-	-	-	-
	Deck ceiling	-	-	-	+	+ (in progress)	-	+ (in progress)
	Deck ceiling - Longitudinal supporting beams	-	-	-	-	-	+	-
	Main and secondary deck cilling beams	-	-	-	-	-	-	Not measured
	Longitudinal and transversal supporting (ceiling) RC beams	-	-	-	-	Not measured	-	-
	cantilever slabs	Not measured	Not measured	Not measured	Not measured	Not measured	Not measured	-
	Tunnel ceiling	-	-	-	Not measured	-	-	-
	Supporting walls	+ (in progress)	+ (in progress)	Not measured	Not measured	-	-	-
	Supporting wall (Masonry support walls made of stone)	-	-	-	-	Not measured	-	-
	Abutment	-	-	+ (in progress)	+ (in progress)	-	-	+ (in initial phase)
	Abutment wall (Masonry wall made of stone and covered by plastering)	-	-	-	-	+ Not important	-	-
	Abutment Additional RC layer	-	-	-	-	-	+	-
	Support column	-	-	-	-	-	-	+ (in progress)
	Expansion joints	-	-	-	-	-	-	-

Table XI-5. Data for comparative analyses of in situ tested carbonation depth after repair

		Souk Athulatha 1	Souk Athulatha 2	Al Sseka road	Bab bin Gheshir road	Al Sreem road	Al Shaab port	Abdul salam aref
Number of bridge		1	2	3	4	5	6	7
carbonation depth (mm)	Ceiling	1	12.5	10	2	6	3	9
	Supporting wall (east side)	0	8	-	-	-	-	-
	Supporting wall (west side)	5	8	-	-	-	-	-
	Abutment(west side)	-	-	18	5		4	2
	Abutment(east side)	-	-	5	3		0	0
	Abutment (south side)	-	-	-	-	20	-	-
	Abutment (north side)	-	-	-	-	0	-	-
	Support column	-	-	-	-	-	-	5

The carbonization depth values are shown in the following graphs.

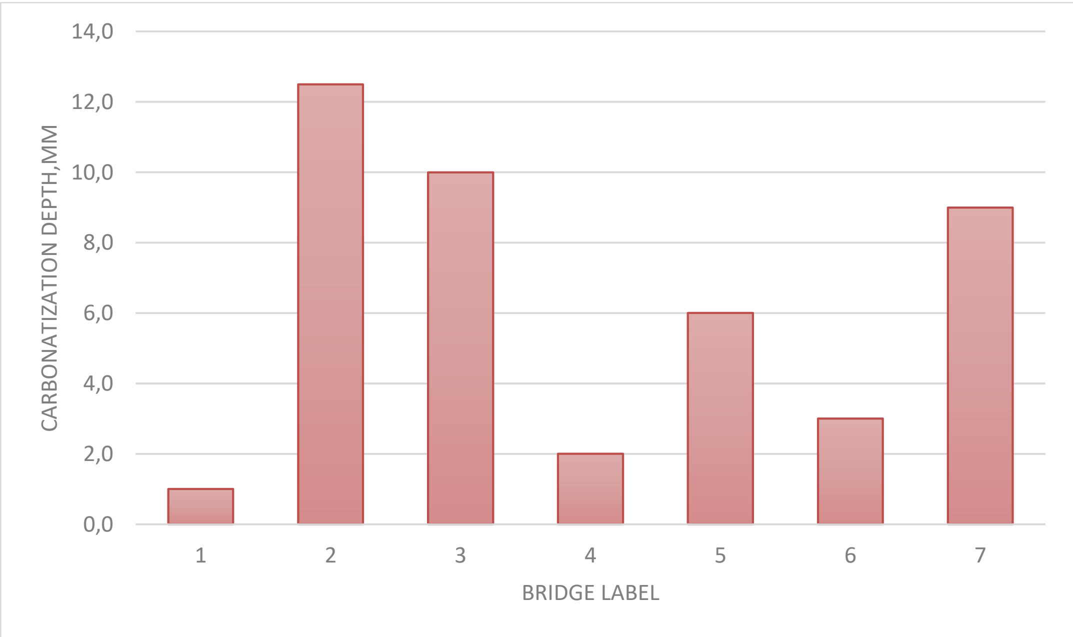


Figure (X-14) Measured values of carbonation depth for ceiling slab

The largest value of carbonation was measured on ceiling (12.5mm).

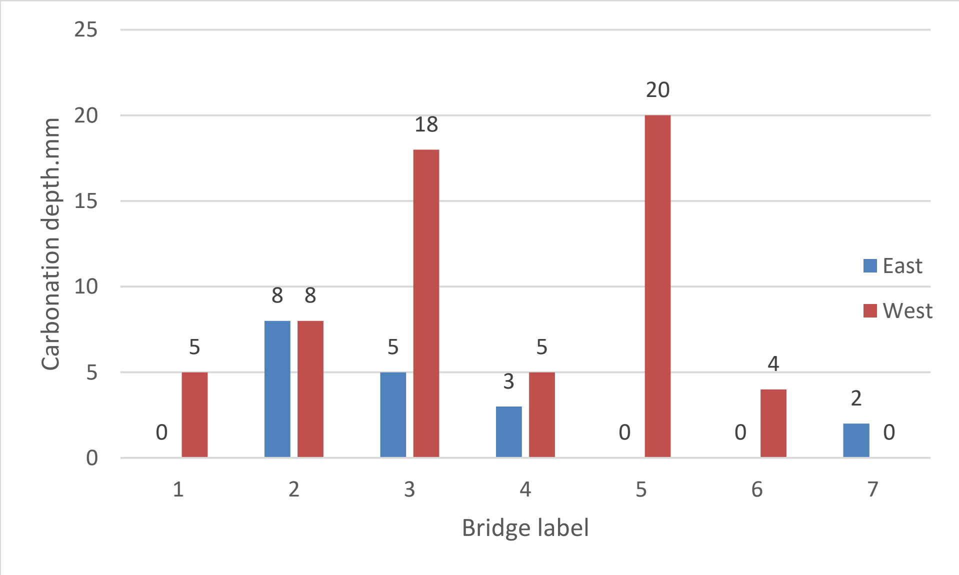


Figure (X-15) Measured values of Carbonation depth for substructure elements

Carbonation was registered on the elements of the super and substructure of the bridges. On the lower surface of the bridge slabs, the depth of carbonation ranges from 1mm to 12.5mm, while on the elements of the substructures it was measured from 0 to 20mm. If we take into account that the highest expected value of carbonation depth for 5 years is 5mm, it can easily be concluded that carbonation is progressing faster than assumed, even though factory-produced repair materials were used for reprofiling, and all repaired elements were additionally treated with protective coatings. This statement once again confirms that carbonation is the main cause of decreasing the durability of RC bridges in hot climates.

2.2 Analyses of the results of visual inspection after repair

In order to analyse of results of visual inspection after repair, the table XI-5 is formed. The all-necessary data are summarized by name of bridges and by the element of bridge for chosen property.

Table XI-5. Results of visual inspection after repair

Damages	RC elements	Souk athulatha 1	Souk athulatha 2	Al Sseka road	Bab bin Gheshir road	Al Sreem road	Al Shaab port	Abdul salam aref
Cracks	Lateral beam	+ Mesh like cracks	+ Mesh like cracks	+ longitudinal	-	-	-	-
	Deck ceiling Arch cantilever slabs	+ Mesh like cracks	+ Mesh like cracks	-	-	-	-	-
	Deck ceiling Simple beam slab	+ Mesh like cracks	+ Mesh like cracks	+ Vertical, simple beam deck	-	-	-	-

	Deck ceiling	-	-	-	+ Net like crack	-	-	+ (on contact with beams)
	Deck ceiling - Longitudinal supporting beams	-	-	-	-	-	+ Longitudinal and transverse cracks	-
	Main and secondary deck ceiling beams	-	-	-	-	-	-	+ (few longitudinal, a lot of trasversal), thin
	Longitudinal and transversal supporting (ceiling) RC beams	-	-	-	-	+ Net like	-	-
	cantilever slabs	+ horizontal and vertical	+ horizontal and vertical	+ Net like, vertical and horizontal	+ Net like, vertical and horizontal	+ net like, not characteristic	+ Vertical and horizontal cracks	-
	Tunnel ceiling	-	-	-	+ Horizontal, vertical and net like crack	-	-	-
	Supporting walls	+ Vertical cracks	-	+ Net like	+ Net like crack	-	-	-
	Supporting wall (Masonry support walls made of stone)	-	-	-	-	-	-	-
	Abutment	-	-	+ Net like	+ Horizontal, vertical and crack of different direction	-	-	+
	Abutment wall (Masonry wall made of stone and covered by plastering)	-	-	-	-	+ Net like	-	-
	Abutment Additional RC layer	-	-	-	-	-	+ vertical and horizontal cracks	-
	Support column	-	-	-	-	-	-	-
	Expansion joints	-	-	-	-	-	-	+ (longitudinal)
Pilling off protecting paint	Lateral beam	-	-	-	-	-	-	-
	Deck ceiling Arch cantilever slabs	-	-	-	-	-	-	-
	Deck ceiling Simple beam	-	-	-	-	-	-	-

	slab							
	Deck ceiling	-	-	-	-	-	-	-
	Deck ceiling - Longitudinal supporting beams	-	-	-	-	-	-	-
	Main and secondary deck cilling beams	-	-	-	-	-	-	-
	Longitudinal and transversal supporting (ceiling) RC beams	-	-	-	-	-	-	-
	cantilever slabs		-	-	-	-	-	-
	Tunnel ceiling	+	-	-	-	-	-	-
	Supporting walls	+	+	+	-	-	-	-
	Supporting wall (Masonry support walls made of stone)	-	-	-	-	-	-	-
	Abutment	-	-	-	-	-	-	+
	Abutment wall (Masonry wall made of stone and covered by plastering)	-	-	-	-	+	-	-
	Abutment Additional RC layer	-	-	-	-	-	-	-
	Support column	-	-	-	-	-	-	-
	Expansion joints	-	-	-	-	-	-	-
Spalling off of concrete	Lateral beam	+ local	+ local	+ Large local damage, spalling off concrete, pulled out and deformed bars	-	-	-	-
	Deck ceiling Arch cantilever slabs	+ local	+ local	+ local	-	-	-	-
	Deck ceiling Simple beam slab	+ local	+ local	+ local	-	-	-	-
	Deck ceiling				-	-	-	-
	Deck ceiling - Longitudinal supporting beams	-	-	-	-	-	+	-
	Main and secondary deck cilling	-	-	-	-	-	-	-

	beams							
	Longitudinal and transversal supporting (ceiling) RC beams	-	-	-	-	+ Local	-	-
	cantilever slabs	-	-	+ local	+ local	+ Local	+ Local	-
	Tunnel ceiling	-	-	-	+ local	-	-	-
	Supporting walls	-	+ plinth	+	-	-	-	-
	Supporting wall (Masonry support walls made of stone)	-	-	-	-	-	-	-
	Abutment	-	-	-	-	-	-	-
	Abutment wall (Masonry wall made of stone and covered by plastering)	-	-	-	-	-	-	-
	Abutment Additional RC layer	-	-	-	-	-	+ Local	-
	Support column	-	-	-	-	-	-	-
	Expansion joints	-	-	-	-	-	-	+ (local)
Water leakage/traces	Lateral beam	+ (traces)	+ (traces)	+ Leakage on the spot of supporting Traces and stains of water through joins	-	-	-	-
	Deck ceiling Arch cantilever slabs	+ (traces)	+ Leakage, rust & white traces	+ Leakage on the spot of supporting Traces and stains of water	-	-	-	-
	Deck ceiling Simple beam slab	+ (traces)	+ Leakage, rust & white traces	+ Traces and stains of water	-	-	-	-
	Deck ceiling	-	-	-	+ Stains of water	+ traces of dust	-	+ (cantilever part)
	Deck ceiling - Longitudinal supporting beams	-	-	-	-	-	+ traces of dust	-

	Main and secondary deck cilling beams	-	-	-	-	-	-	+ (traces, on side surface of external beams)
	Longitudinal and transversal supporting (ceiling) RC beams	-	-	-	-	+ Leakage, traces and stains of water and dust	-	-
	cantilever slabs	-	+ (traces)	+ Traces and stains of water	+ Traces and stains of water	-	+ traces of water	-
	Tunnel ceiling	-	-	-	+ Leakage, Traces and stains of water	-	-	-
	Supporting walls	+ (traces)	+ (traces)	-	-	-	-	-
	Supporting wall (Masonry support walls made of stone)	-	-	-	-	+ Traces and dark stains of water	-	-
	Abutment	-	-	+ Traces and stains of rust	+ Leakage, Traces and stains of water	-	-	+ (traces)
	Abutment wall (Masonry wall made of stone and covered by plastering)	-	-	-	-	-	-	-
	Abutment Additional RC layer	-	-	-	-	-	+ Traces and stains of water	-
	Support column	-	-	-	-		-	-
	Expansion joints	-	-	-	-	Leakage, traces and stains	-	-

The characteristic damage of the down side of lateral beams and deck slab are net like cracks caused by drying shrinkage of repair mortar. They cover the whole down side of deck slab. These cracks are the most serious damage due to the possibility of corrosion of reinforcement. This damage might cause the reduction of durability.

In the middle part of span, on down side of lateral beam and on simple beam slab, the local spalling off of repair mortars is appeared. Described damages are shallow and no reinforcing bars have been spotted. The deeper spalling off is very local.

The horizontal and vertical cracks are typical damage of cantilever slabs. The horizontal crack was appeared between two layers of concrete and it stretches along the whole length of slab, while the vertical cracks are located on side surfaces. They are very thin and caused by drying shrinkage of concrete.

The traces of water are noticed on lateral beams, in the middle of deck slab and on vertical sides of cantilever slabs.

The typical damage of supporting walls is peeling off of surface protecting paint. It covers of approximately 50% of wall surface. Vertical and horizontal cracks appeared on places where previous openings in supporting wall were closed.

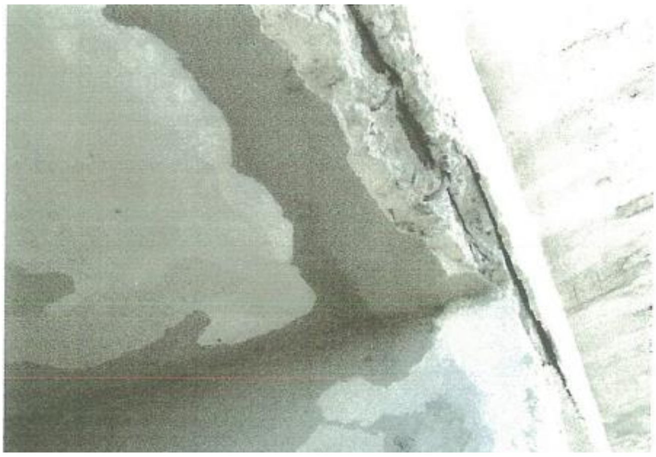
Finally, the stability, bearing capacity, functionality and durability have not been jeopardized, yet. As it was mentioned, damages were spotted on the surface of inspected RC elements, especially on ceiling deck. All damages located in cover (such as mesh like cracks), could be slowed down by some measures like impregnation. The same measures are suggested for RC elements caught by carbonation, but water leakage should be prevented by other methods, due to the risk of progression of reinforcement corrosion. The protective paint on supporting walls should be repainted Local separation and spalling off of repair mortar can be easily re-repaired.

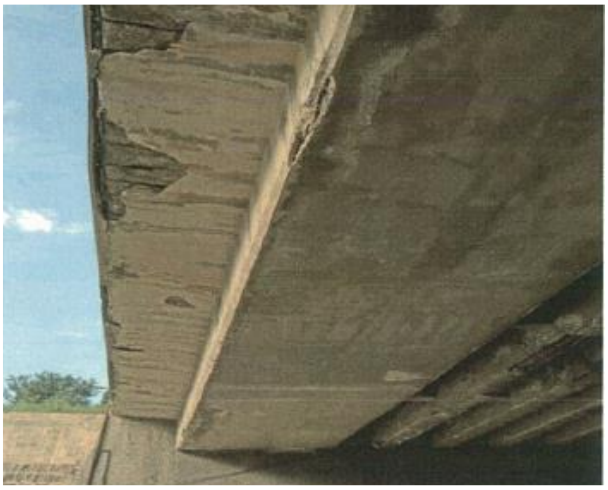
3. CATALOGUE OF TYPICAL DAMAGES OF RC BRIDGE ELEMENTS

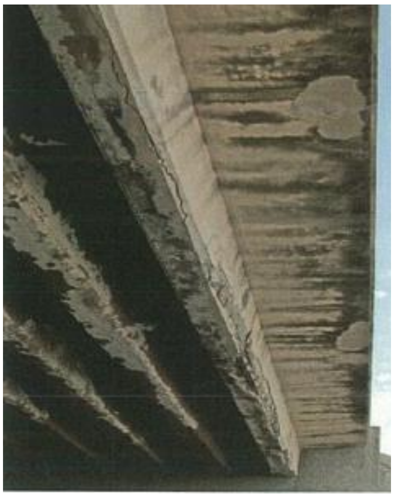
The typical damages of reinforced concrete bridges in hot climate, are given in table XI-6.

Table XI-6. The typical damages of reinforced concrete bridges in hot climate

Damage illustration	Damage description	Possible cause
SLABS		
	**Bridge slab:** Corrosion of reinforcing bars, delamination and falling down of concrete cover, wet stains	Carbonation of concrete and water leakage

	**Cantilever slab:** Bared and corroded reinforcing bars, falling off of concrete cover and plaster layer	Poor concrete works, porous concrete cover and improper repair with ordinary plaster; Carbonation
	**Bridge slab:** Corrosion of reinforcing bars, falling off of concrete cover, reduced adhesion between reinforcement and concrete	Carbonation, Water leakage through cantilever slab
	**Cantilever slab and lateral beam:** Traces of water leakage, corrosion of reinforcing bars, local spalling of concrete	Carbonation, Water leakage

	**Cantilever slab:** Corrosion of reinforcing bars, spalling of concrete cover, delamination of concrete, wet stains	Carbonation and cycling drying and wetting
	**Cantilever slab:** Bad adhesion between reinforcing bars and concrete	Carbonation and cycling drying and wetting
	**Bridge slab:** Net like cracks, local spalling off of repair mortar (cover)	Drying shrinkage of repair mortar

	**Bridge slab:** Local spalling off of repair mortar	Mechanical damage
BEAMS		
	**RC beam:** Bared reinforcing bars, corrosion of bars, falling down of cover and plaster coating	Carbonation and cycling drying and wetting
	**Longitudinal RC beams:** Longitudinal cracks along the corner rebars, dark and white stains	Carbonation and periodical wetting and drying through cantilever slab

	**Lateral beam of arch slab:** Bared and corroded reinforcing bars, falling off of concrete cover and plaster layer	Poor concrete works, porous concrete cover and improper repair with ordinary plaster; Carbonation
	**Edge RC beam:** Bared, deformed and twisted reinforcing bars	Mechanical damage
	**Edge RC beam:** Bared, deformed and twisted reinforcing bars	Mechanical damage
	**RC longitudinal beam:** bared, deformed and twisted rebars, crashed and cracked concrete	Mechanical damage

	**RC longitudinal beams:** Bared and corroded reinforcing bars	Insufficient concrete cover, water leakage and concrete carbonation
	**RC longitudinal beams:** Water leakage through joint between two deck slabs, spalling of mortar layer, white and dark stains	Water leakage

	**Edge beam:** Bared, deformed and broken rebars, crashed and cracked concrete	Mechanical damage
	**RC longitudinal beams:** Delamination and falling off of repair mortar	Poor adhesion between substrate and repair mortar

	**RC longitudinal beams:** Cracks along main reinforcement	Corrosion of main reinforcing bars
COLUMNS		
	**RC column:** Bared reinforcing bars, corrosion of rebars, cracking of concrete along the edge rebars, spalling off of corner concrete	Carbonation and periodical wetting and drying

	**RC column:** Large delamination and spalling off of cover, Bared and corroded reinforcing bars	Poor concrete works. Carbonation and periodical wetting and drying
SUPPORTING WALLS AND ABUTMENTS		
	**Supporting wall:** Cracking and falling off of edge concrete	Mechanical damage

	**RC wall:** Longitudinal cracks near edges due to corrosion of reinforcing bars	Carbonation and periodical wetting and drying
	**Supporting wall:** Net like cracks in (the part above the openings)	Drying shrinkage of repair mortar

	**Abutment:** Leakage through the joint between abutment and deck slab	Poor bridge drainage system

CHAPTER XII

CONCLUSIONS

CHAPTER XII

CONCLUSIONS

1. CONCLUSION

Through analysis of the available literature, it was observed that a large number of studies in the field of the durability of bridges and bridge maintenance management have been performed. In most of these papers, the existing BMSs are analyzed and several new ones are proposed. To obtain the right decision from a BMS, software packages must have high-quality asset information for the system’s various analytical processes. Because of that periodic inspection records are the key resources amongst other information, as historical bridge condition rating data can affect approximately 60% of BMS analysis models. Also, it is emphasized the importance of understanding deterioration mechanisms of concrete and reinforcement for obtaining quality input data.

In this doctoral dissertation problem of deterioration processes in hot climates was particularly emphasized, because the analysis of the available papers showed a lack of research in this field.

The doctoral study encompasses seven reinforced concrete bridges (overpasses) in Tripoli, which were built at the same time and are located on the city's main roads. The bridges have two different structural systems. To assess bridges’ condition, the data obtained by visual inspection and in-situ tests before and after rehabilitation were used. Collected data were also used for software analysis (BMS) to define bridges’ rating.

These bridges were selected for study in the doctoral thesis because they are all reinforced concrete bridges, exposed to similar traffic loads and in the same climate environment, which enabled the application of the comparative analysis to define the characteristic damages.

The main contribution of the research are: the determination of defects and damages by the elements of bridge structures, which are typical for hot climate; creation of a catalog of typical damages of RC bridge elements for a more reliable assessment of bridges during the control survey and collection of data for BMS and improvement of the system of maintenance of bridges in Libya.

By comparative analysis of defects and damages of seven RC bridges in Tripoli it was concluded:

Carbonatizion of concrete

Carbonation of concrete was registered in all elements of the supporting structure, on which the measurement was made. It is equally expressed in the elements of the superstructure and substructure of the bridge. At more than 50% of the measurement points in the superstructure and about 40% of the measurement points in the substructure, the depth of carbonation is greater than 40mm. It is also characterized by a large dispersion of measured carbonation depth (10mm-79mm), both within the elements of one bridge and between all analyzed bridges.

The main reasons for the faster progress of the carbonation front are unfavorable thermo-hygrometric conditions, i.e. relative humidity in the range of 40-60%, relatively poor quality of the concrete cover due to high temperatures, rapid drying, and interrupted hydration.

Taking into account the fact that bridges in hot climates are also designed for a service life of 100 years and that the results of measuring the depth of carbonation in this doctoral dissertation showed that the front of carbonation after 50 years often exceeds the thickness of the concrete cover and that it is known that carbonation of concrete is the main cause electrochemical corrosion of the reinforcement, it is recommended that in the design phase, greater thicknesses of the protective layer of concrete on all elements of the bearing RC structure should be foreseen.

Concrete compressive strength

The results of testing concrete compressive strength (on cores taken from bridge structural elements) indicate the uneven quality of concrete from the aspect of mechanical characteristics. The concrete compressive strength ranges from 22MPa to 38.5MPa. The concrete cast in the elements of the superstructure has a compressive strength greater than 30MPa only in two bridges out of the seven analyzed (< 30% of the total number of bridges). When it comes to the substructures, in only one of the four bridges analyzed, the concrete compressive strength exceeded 30MPa. By the analyzes all the results of testing the concrete compressive strength, a general conclusion was drawn that the concrete cast in the supporting structure of the seven selected bridges has a lower compressive strength than the design one and that it did not decrease over time, but lower quality concrete was placed in. This statement is based on the fact that concrete from the interior of the section, which was not exposed to deterioration processes during exploitation, was used for subsequent compressive strength testing.

Concrete tensile strength

The tensile strength of concrete was tested with a pull-out test. By analyzing all obtained results, it was observed that all values of tensile strength do not meet the required criteria (>1.5MPa). The surface layers of the cast in concrete in all tested bridges are of poor quality. The main causes can be the use of dusty aggregate or inadequate curing of concrete during curing.

Chloride ion content

Analyzing the data on the content of chloride ions in concrete for all seven bridges, it was concluded that all test results are lower than the criterion values and that the chloride content in concrete is not dangerous for the embedded reinforcement.

Concrete density

The unit mass of concrete is one of the more reliable indicators of concrete quality, and that is why in the dissertation a comparative analysis of the unit mass of concrete cast in the structural elements of the selected bridges was done. The unit mass of cement concrete with natural aggregate is usually greater than 2300 kg/m^3 for well-compacted concrete. The results of testing the unit mass of concrete incorporated in the bearing structure of seven selected bridges are in the range of 2111 to 2356 kg/m^3. The average value of the unit mass for the concrete cast in the superstructure of the bridges is 2245kg/m^3, while for the concrete cast in the substructure of the bridges was 2269kg/m^3. It was concluded that the used concrete has a slightly lower value of the unit mass of 2300kg/m^3, and taking into account that these test results refer to the concrete in the interior of the cross-section of the elements (concrete core) where the concrete is of higher density, the concrete cover, and the matrix are certainly more porous, that is, with lower values of unit mass, due to the effect of the wall and the effect of the grid. This conclusion is directly correlated with the depth of carbonation.

Thickness of concrete cover

The thickness of the cover was measured on concrete cores that were taken out from the elements of the supporting structure of the bridges. The thickness of the cover is variable and ranges from 0mm to even 100mm, which indicates that not enough attention was paid to the placement of the reinforcement and that spacers for the reinforcement were not used for certain elements or the reinforcement assembly was not sufficiently attached. The thickness of the cover on the elements of the superstructure of the bridges ranges from 0mm to 70mm, and most often it is from 10mm to 20mm. In the elements of the substructure, the cover is from 20mm to 100mm, and regardless of the wide range of measured thicknesses, the average value is 35mm. The smallest thicknesses of the concrete protective layer were measured on the slab and longitudinal beams of the "Al Shaab port" bridge. Since the thickness of the carbonized layer of concrete was in most cases greater than 40 mm, it is evident that in all elements with a thickness of the concrete cover less than or around the average, the reinforcement was practically unprotected from corrosion.

Corrosion of reinforcement

Corrosion of the reinforcement was registered in both the elements of the superstructure and in the elements of the substructure and is the characteristic damage of bridges. Corrosion intensity ranges from superficial to severe with the peeling of steel. Reinforcement corrosion is frequently followed by impaired adhesion to the surrounding concrete. Most often, corroded reinforcing bars are installed in the lower zone of bridge slabs and beams, as well as in the cantilever parts of bridges. The corrosion of reinforcement was also registered on columns and internal retaining walls but on a smaller scale. Corrosion of the reinforcement in the elements of the superstructure is caused by the insufficient thickness of the concrete cover and the progressive carbonation of the concrete.

Cracking, delamination and spalling of concrete

Cracking, separation, and spalling of concrete are the most common types of concrete damage caused by the increased volume of corrosion products of reinforcing bars. On all seven bridges, the concrete cover was affected by cracking, delamination and spalling of concrete, and in some cases also the zone of corroded reinforcement bars - the cross-section matrix. This statement indicates that the corrosion of the reinforcing bars is the main cause of the described damage to the concrete.

The concrete damage is local, but it was registered in many places on the cantilevers, slabs, and beams in the superstructure of the bridge.

On vertical supporting elements (pillars, external and internal walls), characteristic damages caused by reinforcement corrosion are cracks along the edges and longitudinal bars of the reinforcement.

Six years after the rehabilitation works on seven bridges in Tripoli, a control inspection was carried out again. The inspection included the measurement of carbonation depths and a visual inspection of the elements of the supporting structure of the bridges. Visual inspection revealed: cracks and fissures, flaking of the protective coating, cracking, separation and falling off of repair material and wet zones. The conclusions drawn are briefly presented below.

Carbonation

Carbonation was registered on the elements of the super and substructure of the bridges. On the lower surface of the bridge slabs, the depth of carbonation ranges from 1mm to 12.5mm, while on the elements of the substructure it was measured from 0 to 20mm. If we take into account that the highest expected value of carbonation depth for a period of 5 years is 5mm, it can easily be concluded that carbonation is progressing faster than assumed, even though factory-produced repair materials were used for reprofiling, and all repaired elements were additionally treated with protective coatings. This statement once again confirms that carbonation is the main cause of the reduction of the durability of RC bridges in hot climates.

Visual survey (inspection)

Characteristic damage on the bottom side of repaired bridge slabs and transverse beams are net-like cracks, which are caused by shrinkage due to the drying of repair mortars, which are usually applied in thin layers. In certain areas on the bottom side of the bridge slabs, damage in the form of cracking, separation, and shallow spalling of the thin surface layer of the repair mortar appeared. Thin transverse cracks appeared in the repair mortar on the repaired cantilever slabs, also as a result of shrinkage due to drying. On the load-bearing elements of the substructure (abutments and supporting walls), only damage of an aesthetic nature appeared, in the form of flaking of the protective coating. All the listed damages are another confirmation of the hypothesis that hot climates with their thermo-hygrometric conditions have a great influence on the durability of concrete bridges, even in situations where the bridge structure is repaired with factory-produced repair materials.

Rating of bridges

The rating of the bridges is determined according to the methodology provided by the German BMS. The rating of the bridges before rehabilitation ranges from 2.6 to 2.9. For the calculation of the rating of all seven bridges, damage to the bridge superstructure was relevant, and the characteristic damage was corrosion of reinforcement with or without cross-section reduction. Based on the assessment value, the category of damage to the bridge as a whole was determined. It was concluded that all seven bridges before rehabilitation belong to the same category (2.5-2.9), which is described as "sufficient condition". According to the interpretation of this category, the stability of the bridge is ensured, but the bearing capacity of a part of the bridge may be impaired. The durability of the structure is reduced and damage can be expected to spread, which in the medium term may lead to a significant threat to bridge structure stability and/or traffic safety.

The rating of bridges after rehabilitation ranges from 2.0 to 2.3. For the calculation of the rating of all seven bridges, the relevant factors were the lack of vertical traffic signals or blocked atmospheric sewerage. It was concluded that all seven bridges after rehabilitation belong to the same category (2.0-2.4), which is described as "satisfactory condition". According to the interpretation of this category, the stability of the bridge is ensured, but the bearing capacity of the part of the bridge may be impaired. The durability of the structure over a longer period of time may be reduced.

The assessment of the condition of the bridges, which was made using the German BMS methodology, coincides with the assessment of the condition of the bridges, which was performed based on the analysis of the results of a detailed visual inspection of the bridges. This conclusion points to the fact that the German BMS methodology used is well-coordinated, but that the assessment of the condition largely depends on the quality of the input data, primarily data obtained by visual inspection of bridges, which is why practical experience and theoretical knowledge are necessary. For the realization of the visual inspection phase, the "characteristic damage catalogs" are of great help.

General conclusions

Through a case study, the dependence between the influencing parameters that are characteristic of hot climates and the type and degree of possible damage in reinforced concrete bridges has been proven.

The progress of the carbonation front in reinforced concrete structures in hot climates, with relative humidity in the range of 40-60%, is significantly faster compared to the actual theoretical recommendations on the calculation of this deterioration mechanism. Because of that when analyzing the durability of reinforced concrete structures in hot climates, stricter criteria must be taken for determining the required thickness of the concrete cover.

The catalog of the characteristic defects and damages of reinforced concrete bridges in hot climates, which was formed based on the assessment of the state of a representative number of reinforced concrete bridges in hot climates, will be of great importance for a more efficient and reliable assessment of the condition of RC structures exposed to high air temperature and relative humidity in limits of 40-60%. The application of the mentioned catalog will improve the management system of reinforced concrete bridges in Libya, as well as in other countries with thermo-hygrometric conditions, which are characteristic of hot climates.

CHAPTER XIII

SCIENTIFIC CONTRIBUTION AND FURTHER RESEARCH

CHAPTER XIII

SCIENTIFIC CONTRIBUTION AND FURTHER RESEARCH

1. SCIENTIFIC CONTRIBUTION

By the detailed analysis of previous research in the broader field of durability and maintenance of concrete bridges, it is concluded that this scientific area is insufficiently researched, especially from the aspect of characteristic damages, as well as the connection between the durability and maintenance of concrete bridges in hot climates, so that research with this topic is needed both in the scientific and professional society.

The scientific contribution of this doctoral dissertation is primarily seen through the fulfillment of the set hypotheses:

- It has been proven that the basic climatic parameters of hot climates (seasonal temperatures, air humidity, wind speed, average precipitation, etc.) have a great influence on the type of damage that appears on reinforced concrete bridges during their service life.
- Effective and reliable ranking of bridges using a bridge management system (BMS) is possible only in situations where input data of satisfactory quality has been obtained. From that aspect, the precisely defined type and degree of damage, as well as the causes of their origin in conditions that are characteristic of hot climates, are extremely important for the accurate ranking of bridges in hot climates, using a bridge management system (BMS).
- By analyzing several different models for bridge management, it was confirmed even though all analyzed methodologies have the same goal, the precisely assessed condition of bridges and specific needs for the type of maintenance, the rating, and rank of bridges largely depends on the applied BMS.

The scientific contribution of the dissertation is primarily reflected through the realized case study, which proved the dependence between the influencing parameters that are characteristic of hot climates and the type and degree of possible damage in reinforced concrete bridges.

In the group of own scientific contributions, there is also the conclusion about significantly higher progress of the carbonation front, which resulted from the analysis of own research results on a representative number of RC bridges in hot climates, and which differ from the previous theoretical recommendations on the prediction of the durability of reinforced concrete structures.

As one of the valuable output results of the dissertation, the creation of a catalog of characteristic defects and damages of reinforced concrete bridges in hot climates is highlighted, which will be of great importance for a more efficient and reliable assessment of their condition.

Special contributions of the dissertation are recommendations for improving the management system of reinforced concrete bridges in Libya, as well as in other countries with thermo-hygrometric conditions, which are characteristic of hot climates. The formed catalog of characteristic defects and damages of reinforced concrete bridges in hot climates will enable the improvement of the system for their repair and maintenance.

2. FURTHER RESEARCH

As a part of future research, it is necessary to carry out inspections and in-situ tests of other types of bridges, such as prestressed bridges and bridges with prefabricated elements of the superstructure, which will enable the extension of the database on characteristic defects and damage of reinforced concrete bridges in hot climates.

Also, it would be of particular importance to investigate the speed of the progress of the carbonation front in hot climates, depending on the type of cast-in concrete (concretes with different maximum aggregate grains, self-compacting concretes, concretes with mineral additives, concretes with different strength classes, etc.). The analysis of these data would help to more effectively define the required thicknesses of concrete cover from the aspect of protecting the reinforcement from corrosion caused by carbonation.

CHAPTER XIV

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[27] Australian classification of types of inspection of RC bridges Australasian Transport Research Forum 2011 Proceedings 28 - 30 September 2011, Adelaide, Australia Publication website: http://www.patrec.org/atrf.aspx 1 Proposal of a Methodology for Bridge Condition Assessment Maria Rashidi1 , Peter Gibson2 1 PhD Candidate, University of Wollongong, Northfield Ave, 2522, NSW, Australia 2 A/Professor, University of Wollongong, Northfield Ave, 2522, NSW, Australia mpr223@uow.edu.au

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Овај Образац чини саставни део докторске дисертације, односно докторског уметничког пројекта који се брани на Универзитету у Новом Саду. Попуњен Образац укоричити иза текста докторске дисертације, односно докторског уметничког пројекта.

План третмана података

Назив пројекта/истраживања
КОМПАРАТИВНА АНАЛИЗА ТРАЈНОСТИ И РАНГИРАЊА БЕТОНСКИХ МОСТОВА У ТОПЛИМ КЛИМАТИМА COMPARATIVE ANALYSIS OF DURABILITY AND RANKING OF CONCRETE BRIDGES IN HOT CLIMATES
Назив институције/институција у оквиру којих се спроводи истраживање
Факултет техничких наука, Департман за грађевинарство и геодезију, Универзитет у Новом Саду
Назив програма у оквиру ког се реализује истраживање
-
1. Опис података
1.1 Врста студије *Укратко описати тип студије у оквиру које се подаци прикупљају* <u>Докторска дисертација.</u> <u>Предмет истраживања у докторској дисертацији армиранобетонски мостови, који су током 50 година експлоатације, били изложени специфичним термо-хигрометријским условима, који су карактеристични за топле климате. Кроз теоријска и теренска експериментална истраживања утврђени су карактеристични дефекти и оштећења за армиранобетонске мостове у топлим климатима и дате су препоруке за унапређења система за управљање њиховим одржавањем.</u> 1.2 Врсте података **а) квантитативни** **б) квалитативни** 1.3. Начин прикупљања података а) анкете, упитници, тестови б) клиничке процене, медицински записи, електронски здравствени записи в) генотипови: навести врсту ________________________________

г) административни подаци: навести врсту ________________________

д) узорци ткива: навести врсту__________________________________

ђ) снимци, фотографије: навести врсту______________________________

е) текст, навести врсту ____

ж) мапа, навести врсту ____

з) остало: описати визуелни преглед носеће конструкције мостова и теренско испитивање квалитета уграђених материјала

1.3 Формат података, употребљене скале, количина података

1.3.1 Употребљени софтвер и формат датотеке:

a) Excel фајл, датотека ___________________

b) SPSS фајл, датотека __________________

c) PDF фајл, датотека_________.pdf__________

d) Текст фајл, датотека_______.doc___________

e) JPG фајл, датотека ____________________

f) Остало, датотека ____________________

1.3.2. Број записа (код квантитативних података)

а) број варијабли ______3____

б) број мерења (испитаника, процена, снимака и сл.) ______7 мостова_______

1.3.3. Поновљена мерења

а) **да**

б) не

Уколико је одговор да, одговорити на следећа питања:

а) временски размак између поновљених мера је ___________6 година__________________

б) варијабле које се више пута мере односе се на ___квалитет уграђених материјала___

в) нове верзије фајлова који садрже поновљена мерења су именоване као ___/________

Напомене: __Понављање испитивања није резултат грешке већ чињеница да се одређивано својство материјала мења током времена испитивања ___________________________________

Да ли формати и софтвер омогућавају дељење и дугорочну валидност података?

а) Да

б) Не

Ако је одговор не, образложити ___/____

2. Прикупљање података

2.1 Методологија за прикупљање/генерисање података

2.1.1. У оквиру ког истраживачког нацрта су подаци прикупљени?

а) експеримент, навести тип ____На свежој стабилизацијској мешавини одрђени су Proctor-ови опити. На очврслим стабилизацијским узорцима одређене су: чврстоће при притиску, индиректне затезне чврстоће, као и запреминске масе, бризине и времена проласка ултразвучног таласа. ______

б) корелационо истраживање, навести тип ____

ц) анализа текста, навести тип ____

д) остало, навести шта ____

2.1.2 Навести врсте мерних инструмената или стандарде података специфичних за одређену научну дисциплину (ако постоје).

__Машина за вађење бетонских језгара из конструкције моста, хидрауличне преса (капацитета 150kN) за одређивање чврстоће при притиску, „pull-off“ метода за испитивање чврстоће бетона на затезање, „шмитов чекић“ за одређивање површинске тврдоће бетона.________________

2.2 Квалитет података и стандарди

BS EN 14630: Products and systems for the protection and repair of concrete structures. Test methods. Determination of carbonation depth in hardened concrete by the phenolphthalein method.

BS EN 14629: Products and systems for the protection and repair of concrete structures. Test methods determination of chloride content in hardened concrete.

BS EN 12504-1: Testing concrete in structures - Cored specimens. Taking, examining and testing in compression

BS EN 12504-2: Testing concrete in structures Non-destructive testing. Determination of rebound number

BS EN 12504-3, Testing concrete in structures - Determination of pull-out force

2.2.1. Третман недостајућих података

а) Да ли матрица садржи недостајуће податке? Да **Не**

Ако је одговор да, одговорити на следећа питања:

а) Колики је број недостајућих података? _____/______________________

б) Да ли се кориснику матрице препоручује замена недостајућих података? Да **Не**

в) Ако је одговор да, навести сугестије за третман замене недостајућих података

__________/____

2.2.2. На који начин је контролисан квалитет података? Описати

У овом раду подаци су резултат теренских експерименталних истраживања. Квалитет, тј. поузданост добијених података се обезбеђује одговарајућим бројем узорака тако да се при обради резултата могу применити статистичке методе. Детаљан поступак рада у сваком од експерименталних испитивања је дефинисан одговарајућим стандардом. Квалитет података је контролисан на основу одговарајућих стандарда и упоредних резултата досадашњих истраживања у овој научној области.

2.2.3. На који начин је извршена контрола уноса података у матрицу?

Контрола уноса података је подразумевала физичку проверу, што је био задатак докторанда. Анализа уноса података је такође била задатак докторанда, али је праћена и усмеравана од стране ментора рада. После тога су подаци уношени у матрицу.

3. Третман података и пратећа документација

3.1. Третман и чување података

3.1.1. Подаци ће бити депоновани у Репозиторијуму докторских дисертација на Универзитету у Новом Саду.

3.1.2. URL адреса ______________https://cris.uns.ac.rs/searchDissertations.jsf ______________

3.1.3. DOI ____

3.1.4. Да ли ће подаци бити у отвореном приступу?

а) Да

б) Да, али после ембарга који ће трајати до _____/________________________________

в) Не

Ако је одговор не, навести разлог _____/_____________________________________

3.1.5. Подаци неће бити депоновани у репозиторијум, али ће бити чувани.

Образложење

____/____

____

3.2 Метаподаци и документација података

3.2.1. Који стандард за метаподатке ће бити примењен? ________________________________

3.2.1. Навести метаподатке на основу којих су подаци депоновани у репозиторијум.

____

____

Ако је потребно, навести методе које се користе за преузимање података, аналитичке и процедуралне информације, њихово кодирање, детаљне описе варијабли, записа итд.

____

____

____

____

3.3 Стратегија и стандарди за чување података

3.3.1. До ког периода ће подаци бити чувани у репозиторијуму? ________________________

3.3.2. Да ли ће подаци бити депоновани под шифром? **Да** Не

3.3.3. Да ли ће шифра бити доступна одређеном кругу истраживача? **Да** Не

3.3.4. Да ли се подаци морају уклонити из отвореног приступа после извесног времена?

Да **Не**

Образложити

____

____

4. Безбедност података и заштита поверљивих информација

Овај одељак МОРА бити попуњен ако ваши подаци укључују личне податке који се односе на учеснике у истраживању. За друга истраживања треба такође размотрити заштиту и сигурност података.

4.1 Формални стандарди за сигурност информација/података

Истраживачи који спроводе испитивања с људима морају да се придржавају Закона о заштити података о личности (*https://www.paragraf.rs/propisi/zakon_o_zastiti_podataka_o_licnosti.html*) и одговарајућег институционалног кодекса о академском интегритету.

4.1.2. Да ли је истраживање одобрено од стране етичке комисије? Да **Не**

Ако је одговор Да, навести датум и назив етичке комисије која је одобрила истраживање

_______________________________________/_______________________________________

4.1.2. Да ли подаци укључују личне податке учесника у истраживању? Да **Не**

Ако је одговор да, наведите на који начин сте осигурали поверљивост и сигурност информација везаних за испитанике:

а) Подаци нису у отвореном приступу

б) Подаци су анонимизирани

ц) Остало, навести шта

____

____

5. Доступност података

5.1. Подаци ће бити

а) јавно доступни

б) доступни само уском кругу истраживача у одређеној научној области

ц) затворени

Ако су подаци доступни само уском кругу истраживача, навести под којим условима могу да их користе:

__/____

____

Ако су подаци доступни само уском кругу истраживача, навести на који начин могу приступити подацима:

__/____

____

5.4. Навести лиценцу под којом ће прикупљени подаци бити архивирани.

__/____

6. Улоге и одговорност

6.1. Навести име и презиме и мејл адресу власника (аутора) података

____SAEEDA FURGAN, email: saeedaomran.1980@gmail.com____________________

6.2. Навести име и презиме и мејл адресу особе која одржава матрицу с подацима

_____ SAEEDA FURGAN, email: saeedaomran.1980@gmail.com______________________

6.3. Навести име и презиме и мејл адресу особе која омогућује приступ подацима другим истраживачима

____SAEEDA FURGAN, email: saeedaomran.1980@gmail.com_______________________