ADDIS ABABA UNIVERSITY INSTITUTE OF TECHNOLOGY CIVIL AND ENVIRONMENTAL ENGINEERING, ADDIS ABABA, ETHIOPIA

Damage Identification and Adaptation of Maintenance Techniques for prestressed Concrete bridges

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Damage Identification and Adaptation of Maintenance Techniques for prestressed Concrete bridges

A Thesis Submitted to Addis Ababa University, Institute of technology Civil and Environmental Engineering in partial fulfillment of the requirement for the Degree of Master of Science in Railway Civil Engineering.

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Abstract

This study indicates that, bridges reacts to the forces imposed by passing traffic with damage to, or loosening of fastenings, crack on the deck, crack on the abutment, crack on the girder, crack on the pier, rebar exposure, scour, bulging, wearing of rail, ballast fouling on the deck, rail failure on the bridge, sleeper displacement, missing of fastener, rail joint failure, expansion joint failure, bearing failure, hand rail missing, hand rail bulging, embankment failure, erosion of the underside of the sleepers, fouling of ballast etc. All these defects should be properly identified and propose the viable maintenance solution to keep the bridges services as required.

All bridge defects, which are defined in the above, must need proper inspection and appropriate maintenance techniques. Since ERC is re-established as a new firm and has not yet settled maintenance techniques, this study could serve as a draft guide. In this research maintenance technique for damaged prestressed concrete bridge was adapted from other countries experience, Ethiopian experienced company's like ERA, manuals books and other available sources.

In developed countries it has been common practice closely follow up of their structures and treat according to its damage. The use of damage identification and maintenance is to control infrastructures health condition and to keep their service life safe.

The clear advantage of damage identification and maintenance technique is discussed in this study.

Bridge monitoring and inspection is the best techniques that internationally practiced for identifying of bridge damages. For varied damages/defects, each is having its own maintenance strategy and procedures which practiced in different countries. The most common defect maintenance technique has been indicated in this study.

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Abbreviations and definitions

AREMA=American Railway Engineering and maintenance of way Association

AASHTO= American Association Of State Highway and Transportation Office

ASTM=American Society for Testing and Materials

BMS= Bridge Management System

CFRP= Carbon Fiber Reinforced Polymer

CMS= Culvert Management System

CWR = Continuous Welded Rail

ERA= Ethiopian Roads Authority

FRP= Fiber Reinforced Polymer

NBIS =National Bridge Inspection Standard

NCHRP=National Cooperative Highway Research Program

PTCFRP= Post tensioning Carbon Fiber Reinforced Polymer

1. Introduction

1.1Background

Before a century Railway Company was established in the name of **The Imperial Railway Company of Ethiopian** to connect the eastern Ethiopia from port of Djibouti to the capital city Addis Ababa. Although the railway line offered a huge socio-economic development of the country, because of many reason it could not continue with the necessary services. However, now a day, Federal Democratic Republic of Ethiopia aims to strengthen its transportation network a lot of efforts and funds go toward constructing railway system. This plan mainly covers reconstructing the former railway line that is from port of Djibouti to the capital city Addis Ababa, in a standard gauge and a number of new railway lines are also going to be constructed to the Regional Cities.

Light rail transit in Addis Ababa, freight rail transport system to the main port of Addis-Dewole, Modjo-Konso-Weito, A.A-Jimma-Bedele, Lijaje-Nekemte-Assosa-Kumruk, Awash-Kombolcha-Mekele-Shire, Fnoteselam-BahirDar-Worota-Woldia-Semera-Elidar, Worota-Azezo-Metema, and Adama-Indeto-Gasera are currently under construction and planed projects.

Ethiopia has very difficult and extremely variable topographic nature, up to 4600m above sea level in the case of Ras- Dashen Mountain and abound 500m below sea level in Afar Region. Consequently the under construction railway line consider a number of bridge structures across these terrains. Accordingly after the construction completed these structures should oblige close follow up to keep its service life by providing appropriate maintenance for the expected damages after the construction stage. However, since the railway sector is newly under construction, it has not maintenance technique and the required professionals before. Therefore, the expected damage should be identified and the viable maintenance technique should be settled for the railway sector.

Thus, this study is intended to identify bridge damages and adaptation of the required maintenance technique for Ethiopian Railway Corporation. During the study, reference to researches made to a number of railway bridge defects, study some countries railway bridge maintenance technique and experience and methodologies, study different countries railway bridge manuals such as inspection manual, maintenance manuals, training manuals as appropriate, in addition to case study by visual inspection of existing and newly constructed bridges for highway and railway.

1.2Objectives and Scope of the Study

1.2.1 Objectives

The main objective of the study is identification of bridge damages and adaptation of maintenance technique for Pre stressed concrete Railway Bridges for practice in in our country, where the following are of concern:

a) Predict types of damage that will occurred in the under constructing prestressed concrete railway bridges in Ethiopia,
b) Show the types of inspection and its importance as methods of identifying,
c) Adapt maintenance techniques for damaged prestressed concrete bridges
d) Adapt maintenance procedure.

The study considers: the prevailing common prestressed concrete defects that are incorporated in various literatures and manuals, traditional and advanced inspection techniques, maintenance techniques and the results of a survey of current practice research in National Cooperation Highway research program report No. 226 and 280, survey of current repair practice in the United States and Canada, and adapt Ethiopian Roads Authority concrete bridge maintenance practice.

1.2.2 Scope of the study

Generally the study covers causes of major defects, their maintenance solution on pre-stressed concrete structures. Since most of Railway bridges in Ethiopia are Prestressed Concrete Bridges, the study does not address Steel, Masonry Arch, Suspension and Truss Bridges. In terms of components of bridges more focus given to super structure. In this regard the study is made to develop maintenance techniques and identifying major damages only for the pre-stressed concrete railway bridges to the Ethiopian practice.

Thus, the thesis is concerned to railway bridges made up of pre-stressed concrete but the study result can be also functional for conventional Reinforced concrete bridges. However, the study:

- Does not consider cost or other way of comparison.
- Partially covers on track components which have direct influence on Railway Bridge such as sleepers and ballast.
- Focused on the super structure of pre-stressed concrete railway bridges
- Does not conduct any laboratory testing.

Hence, it covers the major defects and their maintenance technique from different Railway bridges related manual and case study observed from existing bridges but, does not include Policies, Strategies and Plans of maintenance activities

1.3 The Structure of the thesis presentation

The content of this thesis includes six chapter, where the

- First chapter is devoted to introducing background, objective of the study; the structure of the thesis is also presented.
- Chapter 2 addresses the necessary background information regarding prestressed concrete member inspection, General Damages, Deterioration and Causes of Damages & Deterioration, Design Consideration of Limits on Deterioration of Prestressed Concrete and maintenance and rehabilitation techniques,
- Chapter 3 is made to include damage types and causes of damages on Prestressed Concrete Railway Bridges,
- Chapter 4 reveals Prestressed Concrete Bridge maintenance techniques,
- Chapter 5 describes the Ethiopian practice in bridge inspection and maintenance.
- Chapter 6 summarizes the work presented in this document, suggests conclusion and provides recommendations and future research options.

2. Literature Review

2.1Prestressed concrete bridge

Prestressing is a method of inducing known permanent stresses in a structure or member before the full or live load is applied. This stresses are induced by tensioning the high-tensile strand, wire or rods, and then anchored to the member being Prestressed by mechanical, hydraulically, electrical/thermal or chemical means. Prestressed concrete is ideally suited for the construction of medium and long span bridges, by the use of high strength material; it improves serviceability and durability of structure. Since the development of Prestressed, the material has been found extensive application in the construction of long span bridges, gradually replacing steel which needs costly maintenance due to the inherent disadvantages of corrosion under aggressive atmospheric condition. Prestressed concrete is ideally suited for long span continuous bridges in which pre-cast box girder of variable depth are used for spans exceeding 50m. Prestressed concrete has been widely used throughout the world for simple supported, continuous, balanced cantilever, suspension, hammer head and bridle-chord type bridges in the span range of 20m to 500m. Bridle-Chord is a combination of a long beam (usually a trussed girder) partially supported by steel wire from a tower at one end or at each end, [1].

The Prestressing counteract the stresses produced by subsequent loading on the structure, thus extended the range of stress to which a structural member can safely be subjected. This also improve the behavior of the material which the member or structure is composed. For instance the concrete which have relatively low tensile strength, shall behave like a member having high tensile strength, after Prestressing.

2.1.1The concept of Prestressing

The concepts of prestressing exist before the application to concrete. Some applications were lead to the development of prestressed concrete structures. The following are main examples:

1) Force-Fitting of metal bands on wooden barrels it induces a state of initial hoop compression, to counteract hoop tension caused by filling of liquid in the barrels.
2) Pre-tensioning the spokes in a bicycle wheel. The pre-tensioning is to such an extent that there will always be a residual tension in the spoke.

Why Prestressing required?

The concrete tensile strength is only 8-14% of its compressive strength. Due to this crack develop at early stage of loading in flexural members such as beam and girder. To prevent such crack, compressive force can be suitable applied in the longitudinal direction, either concentrically or eccentrically (linear prestressing).

Prestressing enhances the bending, shear and torsional capacity of the flexural member. In the early time attempts of prestressing were not completely successful. It was observed that the effect of prestressing reduced with time. After the hardening of the concrete release the tension in the rods. The rods will try to regain their original length, but this is prevented by the surrounding concrete to which the steel is bonded. Thus, the concrete is now effectively in a state of pre compression (capable of counteracting tensile stress). Now the load resisting capacity of the beam was limited under sustained load, the beams were found to be failed. Under sustained loads, concrete under goes creep strain. The reduction in length due to creep and shrinkage is also applicable to the embedded steel result in a significant loss in the tensile strain. The residual strain and hence the residual Pre-stress was only about 10% of initial value.

- Adopt high strength steel with much higher original strain. This leads to the requirement of high prestressing force.
- Adopt high strength concrete to with stand the high prestressing force.

In Ethiopia the prestressed concrete bridge construction has been established in a few years back, especially long span box girder bridges. According to the information gathered from ERA bridge management team, until the present day, excluding the railway new bridges, less than 20 prestressed concrete bridges are found in the federal highway network. Some of this are: 2 bridges in Debere Markos, 2 in Adigrat, 1 in Alemgena, 1 in Combolcha, 5 bridges in DireDawa, 3 bridges in Nekempte, and 2 in Sodo Districts of the Federal Road networks. Most of them are prestressed deck girder but one PC box girder bridge has been found in DireDawa and the second is found in Debre Markos, Awash Bridge which is completed in 2015 and the Hidasie Bridges, respectively. However, due to the new government policy, numerous amounts of prestressed concrete bridges have been constructed in the railway sector.

2.1.2 Application of Prestressing

In structural member, when the span length is very high with low rise and low structural height, the application of reinforced cement concrete shall be virtually impracticable. In such a case, Prestressing is used to achieve a light weight elegant looking and much economic structure with high durability. Prestressing, therefore, is widely used for long span beam and bridge.

In building structure also Prestressing method is very effectively used to achieve lighter beams and slabs; thus reduce the dead load considerably as compared to reinforced concrete structures. Application of Prestressing in building construction also facilitates a large space between the columns; thus reduce the numbers of columns and then make the structure more versatile for interior planning.

Prestressing is also very widely used in the construction of Mega structures, storage tankers, cement silos, chimney, dams and rock anchors etc.

2.1.3Prestressed concrete system

Pre-stressing system has developed over the years and various companies have patented their products .Detail information of the systems are given in the product catalogues and company brochure.

The prestressing systems and devices are described for two types of prestressing, pre-tensioning and post-tensioning separately

2.1.3.1 Pre-tensioning System

In pre-tensioning the tension is applied to the tendon before casting of the concrete. The stage of pre-tensioning are described below.

In pre-tensioning system, the high strength steel tendons are pulled between two end abutment's (also called bulkhead) prior to the casting of concrete. The abutments are fixed at the ends of prestressing bed.

Once the casted concrete attains the desired strength, for the transfer of Pre-stress, the tendons are cut loose from the abutments. The Pre-stress is transferred to the concrete from the tendons due to the bend at the interface. During the transfer of Pre-stress, the member undergoes elastic shortening. If the tendons are located eccentrically, the member is likely to bend and deflect upwards (camber).

The various stages of the pre-tensioning operation is summarized as follow

1) Anchoring of tendons against the end abutments
2) Placing of jacks
3) Applying tension to the tendons
4) Casting of concrete
5) Curing of concrete
6) Cutting of the tendons

2.1.3.2 Post-Tensioning

The tension is applied to the tendons after hardening of the concrete

The stages of post tensioning

In post-tensioning system, the tendons are pulled in ducts after the casting of concrete. The purpose of the duct is preventing bond between concert and the tendons during the tensioning operation. The profile of the tendon is similar to the profile of duct.

Unlike pre-tensioning, the tendons are pulled with the reaction acting against the harden concrete. The post tensioning member either bounded or unbounded.

If the ducts are grouted (neat cement paste or a sand-cement mortar containing suitable admixture) then it is known as bonded post-tensioning.

In unbounded post-tensioning as the name suggests, the ducts are never grouted and the tender is held in tension solely by the end anchorages.

The purpose of unbounded post tensioning is to avoid the grouting process. It is applicable for slabs which have a small duct holes where grouting is difficult. In such case, corrosion may be a problem.

After the placement of tendon, in order to fix the tendons apply anchorage at one end and, in order to induce prestressing force, apply Jack on the other end. Then applying required amount of prestressing force on the tendons and set a wedge. The tensioning of tendon and recompressions of concrete occur simultaneously.

During the jacking of the tendon, the member develops upward deflection, camber, due to cambering the member hogged up from the casting bed. In this stage the elastic shortening develops.

Here, the whole process are self-equilibrate because the tension in the steel and the compression in concrete are balance each other for this reason in post tensioning no need of any special foundation during tensioning process. Finally, cut the extreme end of the tendon.

In short the stage of post tensioning is summarized as follow:

1. Placing of conduits with the tendons
2. Casting of concrete
3. Placement of the anchorage black and jack
4. Applying tension to the tendons
5. Setting of the wedges
6. Cutting of the tendons

Advantage of post-tensioning over pre-tensioning

- Suitable for heavy cast in place members
- Less waiting period in casting bed
- Transfer of pre-stress independent of transmission length where as it depends on the anchorages block which transfers most of the Pre-stress.

The transfer of Pre-stress depends on not only in the bond between the concrete and steel but also it is the anchorage block at the end which transfer most of the pre-stress from the steel to the concrete.

Disadvantage

Require anchorage device and grouting equipment

In pre tensioning the anchorage device are small, there are usually truck assembly as the individual strand but post tensioning use heavy duty anchorage block to transfer from concrete to steel.

Transportation of grouting equipment is also has its own risk.

Devices that are used for post tensioning

1. Casting bed –no need of sophisticating bench
2. Shuttering –
3. Ducts
4. Anchorage device
5. Jacks
6. Couplers(optional)
7. Grouting equipment(optional)

Post tensioning Anchorage Device

In post tensioning member the anchorage devices transfer the Pre-stress to the concrete, the devices are based on the following principles of anchoring the tendons

a) Wedge action
b) Direct bearing
c) Looping the wires

Wedge action:- Producing a frictional grip on the wires. The anchorage device based on the wedge action consists of an anchorage block and wedges. The strands are held by wedges in the anchorage block

Direct bearing from rivet or bolt heads or button heads formed at the end of the wire

The B.B.R.V post – tensioning system and the person system are based on this principle

Looping the wire around the concrete the Baur- Leonardet system, Leoba system, and also the Owidag single bar anchorage system are working on this principle. This principle works for only a single device at a time.

The most widely used method for prestressing of structural concrete element is longitudinal tensioning of steel by different tensioning device. Prestressing by the application of direct force between abutments is generally used for arch structures, while flat jacks are invariable used to impart the desired force. Circular prestressing is applicable for circular structure such as tanker and pipe.

2.1.3.3 Tensioning device

Prestressing tensioning device can be categorized in to four namely, [1];

i. **Mechanical device** generally used include weight with or without lever transmission, gear transmission in conjunction with pulley block, screw jack with or without gear device and wire winding machine this device are employed mainly for prestressing structure concrete components produced on a mass scale in the factory.

ii. **Hydraulic jacks** being the simplest means of producing large prestressing forces are extensively use as tension device.

iii. **Electrical device**: - the steel wires are electrically heated and anchored before placing concrete in to the mold. This is often referred to as thermo-electrical Prestressing.

iv. **Chemical method**: - expanding cements are used and the degree of expansion is controlled by varying the curing condition. The expansion action of cement is induced tensile force in the tendon and compressive stress in concrete.

2.1.4Types of Pre-stressing System

Pre-stressing can be classified in several ways. Such as based on source of prestressing force, external or internal prestressing, pre tensioning or post tensioning, linear or circular prestressing, full or limited or partial prestressing, and uniaxial, biaxial or multi-axial.

2.1.4.1Based on source of pre-stressing force

This classification is based on the method by which the prestressing force is generated. There are four sources of prestressing forces.

1) **Mechanical prestressing**: -in this types of prestressing the device include weights with or without lever transmission, gear transmission in conjunction with pulley blocks, screw jack with or without gear drives and wire-winding machine. This types of prestressing adopted for mass scale production.
2) **Hydraulic prestressing**: - this is the simplest types of prestressing that used to producing large prestressing force. The hydraulic jack used for the tensioning of tendon, comprises

of calibrated pressure gauges which directly indicate the magnitude of force developed during the tensioning.

3) **Electrical prestressing: -** in this types of prestressing the steel wires are electrically heated and anchored before placing the concrete in the molds. These types of prestressing also known as Thermo-Electrical prestressing.
4) **Chemical prestressing**: - in this types of prestressing expansive cements are used and the degree of expansion is controlled by varying the curing condition. The expansive action of cement is restrained while setting. This generates tensile forces in the tendons and compressive stresses in concrete.

2.1.4.2 External and Internal prestressing

External prestressing: - when the prestressing is achieved by elements located outside the concrete member, for example cable lying outside a beam, it is called external prestressing. External prestressing technique is adopted in repair and strengthening works, such as retrofitting of bridges.

Internal prestressing: - when the prestressing is achieved by elements located inside the concrete member (Commonly, by embedded tendon) it is called internal prestressing. Most of prestressing systems are internal prestressing.

2.1.4.3 Linear and Circular pre-stressing

Linear prestressing: - the prestressing members are straight or flat, in the direction of prestressing, the prestressing is called linear prestressing. Prestressing of beams, piles, poles and slabs are examples of linear prestressing. The prestressing cable may be curved or straight.

Circular prestressing: -the prestressing members are curved in the direction of prestressing is called circular prestressing. Circumferential prestressing of tanks, silos, pipes and similar structures are examples of circular pre-stressing

2.1.4.4 Pre-tensioning and Post tensioning

Pre-tensioning system:

In the pre-tensioning systems, the tendons are first tensioned between rigid anchor-blocks cast on the ground or in a column or unit–mold types pre-tensioning bed, prior to the casting of concrete in the mold. The tendons comprising individual wires or strands are stretched with constant eccentricity or a variable eccentricity with tendon anchorage at one end and jacks at the other. With the forms in place, the concrete is cast around the stressed tendon.

Post-tensioned systems:

In post-tensioning the concrete unit are first cast by incorporating ducts or grooves to house the tendons. When the concrete attains sufficient strength, the high-tensile wires are tensioned by means of jack bearing on the end of the face of the member and anchored by wedge or nuts. The forces are transmitted to the concrete by means of end anchorage and, when the cable is curved, through the radial pressure between the cable and the duct. The space between the tendons and the duct is generally grouted after the tensioning operation.

Most of the commercially patented prestressing systems are based on the following principle of anchoring the tendons:

1. Wedge action producing a frictional grip on the wire.
2. Direct bearing from the rivet or bolt heads formed at the end of the wire.
3. Looping the wire around the concrete.

Table 2-1 comparison of Pre-tension Vs. post tension member

Pretension member	Post-tensioned member
1. In pre-tensioned Pre-stress concrete, steel is tensioned prior to that of concrete. It is released once the concrete is placed and hardened. The stresses are transferred all along the wire by means of **bond**.	1. Concreting is done first then wires are tensioned and anchored at ends. The stress transfer is by **end bearing** not by bond.
2. Suitable for short span and precast products like sleepers, electric poles on mass production.	2. Suitable for long span bridges
3. In pre-tensioning the cables are basically straight and horizontal. Placing them in curved or inclined position is difficult. However the wires can be kept with eccentrically. Since cables cannot be aligned similar to B.M.D. structural advantages are less compare to that of post-tensioned.	3. The post tensioning cables can be aligned in any manner to suit the B.M.D due to external load system. Therefore it is more economical particularly for long span bridges. The curved or inclined cables can have vertical component at ends. These components will reduce the design shear force. Hence post-tensioned beams are

	superior to pre-tensioned beams both from flexural and shear resistances point.
4. Pre-stress losses are more compare to that of post-tensioned concrete.	4. Losses are less compare to pre-tensioned concrete

2.1.4.5 Full, limited and partial prestressing

Full prestressing: - the level of prestressing is such that no tensile stress is allowed in concrete under service loads is called full prestressing. Type 1, as per IS: 1343-1980

Limited prestressing: - the level of prestressing is such that the tensile stress under service load is within the cracking stress of concrete is known as limited prestressing. Type 2, as per IS: 1343-1980. Her tension is allowed in the concrete but below its tensile strength.

Partial prestressing: - the level of prestressing is such that under tensile stress due to service load, the crack width is within the allowable limit is called partial prestressing. Type 3, as per IS: 1343-1980

2.1.4.6 Uniaxial, Biaxial and Multi-axial Prestressing

Uniaxial prestressing: - the prestressing cables are parallel to one axis is known as uniaxial prestressing. Longitudinal prestressing of beams are examples of uniaxial prestressing.

Biaxial Prestressing: - there are prestressing cables are parallel to two axis is called biaxial prestressing. Prestressing of slabs are examples of biaxial prestressing.

Multi-axial prestressing: - the prestressing cables are parallel to more than two axis is known as multi-axial prestressing. Prestressing of dome structures are examples of multi-axial prestressing.

2.2 INSPECTION OF BRIDGE

The primary purpose of bridge inspection is to maintain the public safety, confidence, and investment in bridges. Inspections can be carried out for the maintenance or for the acceptance of bridges at the existing condition. The object of inspections is to identify any defect which may cause an unacceptable safety or serviceability risk or require a serious maintenance in order to preserve the bridge structure.

2.2.1 Types of inspection

According to the *AASHTO Manual for Condition Evaluation of Bridges*, there are five types of bridge inspections:

1. **Initial Inspection--**After the completion of a new bridge structure, this inspection will be done in order to collect all structure inventory data and to determine the baseline structural conditions and to identify current or potential problems.

2. **Routine Inspection—**It is performed on a regular interval of time (usually every 2 years), based on the requirements prescribed by the NBIS, to determine the physical and functional condition of the bridge and to identify changes since the last inspection.

3. **Damage Inspection—**it is performed when Damages occur due to environmental or human actions and its primary goal is to identify the need for further action.

4. **In-Depth Inspection—**this is a close up inspection which focuses on certain sections of the bridge structure to investigate deficiencies not detected during Routine Inspection.

5. **Special Inspection—**this inspection is conducted to monitor a single known defect or condition.

2.2.2 Frequency of Inspection

The frequency, scope, and depth of the inspection of bridges generally depend on several parameters such as *age, traffic characteristics, state of maintenance, fatigue-prone details, weight limit posting level, and known deficiencies.* Bridge owners may establish the specific frequency of inspection based on the above factors.

National Bridge Inspection Standard (NBIS) requires that each bridge that is opened to public be inspected at regular intervals not exceeding 2 years. However, Ethiopia Roads Authority prepared its inspection frequency for every highway. In ERA bridge inspection manual, there are three types of bridge inspection which are: (a) regular inspection conducted in every year, (b) major inspection conducted in every three years, and (c) emergency inspection conducted when emergency damages have been occurred, [22].

The underwater components that cannot be visually evaluated during periods of low flow or examined by feel for their physical conditions should be inspected at an interval not exceeding 5years. (NBIS)

2.2.3Inspection Techniques

According to national bridge inspection manual and the research prepared by Abeyou Adane Taddes, Graduate College of the Oklahoma State University in July 2011, bridge inspection techniques can be categorized as visual inspection, physical examinations, advanced destructive techniques and advanced non-destructive techniques. In addition, National Cooperative Highway Research program report 20-07(307) was detail explained non-distractive inspection techniques by correlated with prestressed concrete member [2].

2.2.3.1 Visual inspection technique: - is the observation of structural defects either directly with a necked eye or indirectly using visual instrument. It is the primary and most common technique which is carried out before any other inspection technique. Even though this type of technique is quick and economical, the result is depends on the inspectors experience. Detailed assessment of damage such as damage caused by corrosion of strands or the remaining effective Pre-stress was not mentioned [2].

The disadvantage of visual inspection is its local nature. Except in the presence of significant damage, only information about the cover concrete and outermost layers of steel may be assessed. Sections that are chipped during inspection must be repaired afterwards, which might provide a focal point for future corrosion. [2]

2.2.3.2 Physical examination technique: - is another bridge inspection technique which used to detecting delamination or overlay material. Hammer sounding and chain dragging are some example of physical examination technique. It is simple, inexpensive and quick technique. However, the inspection result depends on *hearing capacity and inspector experience* [3].

A high frequency pitch indicates a sound concrete whereas a lower frequency pitch indicates the presence of flaws/weakness. This simple method combined with visual inspection of concrete surface for cracks extending over the reinforcement length may provide preliminary information about the structure. However, it is dependent on the skill level of the operator and does *not provide information about the extent of damage*, [2, and 16].

2.2.3.3 Advanced destructive technique; - it is one of the well-known technique and usually conducted by taking sample from the bridge element. This may carried out either in the laboratory or in the field to detect defects, deterioration or determining material properties.

2.2.3.4 Advanced Non-destructive technique**: -**it is one of the main bridge inspection techniques unlike a destructive technique no need of taking a sample from the bridge element. Using more advanced technology, by applying the system on the required bridge element easily can collect the appropriate data. Surface Potential Survey/Half-Cell Potential Survey, Remnant Magnetism, Linear Polarization, Acoustic Emission (AE), Electrical Resistance, Fiber Optic Sensors, Impact-Echo, Surface, Penetration Radar, Magnetic Field Disturbance are some types of advanced non distractive techniques.

1. **Surface Potential Survey/Half-Cell Potential Survey**

Half-cell or electrode potential mapping is a widely accepted method for detecting corrosion of steel embedded in concrete [16, 2]. The entire surface is mapped by recording the surface potentials with respect to a reference electrode. Locations with higher negative potentials indicate areas of corrosion. However, this correlation does not always hold and surface potentials are greatly influenced by surface conditions. It is not possible to distinguish between reinforcement and prestressing strands [2, and 16].

The corrosion potential of the steel in concrete can be measured as the voltage difference between the reinforcement and the reference electrode in contact with the concrete surface [2]. By moving the reference electrode, a relative potential map can be made which shows areas that are more susceptible to corrosion. This is a quick and inexpensive method that may be used during planning of areas that need repair. However, results may be affected by the degree of humidity of concrete, oxygen content near the reinforcement, existence and extent of micro cracks, or electrical stray currents. Due to these reasons, ASTM has specified certain conditions where the technique should not be applied. It should be recognized that results of this method are not quantitative [2]. Connection of the equipment to the steel reinforcement is required. The surface potential survey/half-cell potential survey is a well-established standardized technique. While cumbersome, it is presently most viable and widely used *in situ* alongside visual and other manual forms of inspection.

2. **Remnant Magnetism**

Remnant magnetism is useful to get information about the location of prestressing steel fractures and the degree of damage to a strand. The prestressing strands are magnetized up to saturation to remove their magnetic history. This process is performed by an electromagnet along the direction of the tendon [2].

Fractures and breaks in the prestressing tendons are detectable but the size of the defect or loss of section is not [2]. Limitations of the method are related to the density of the reinforcement present and the minimum degree of damage that is sought [2]. The method can be applied on the vertical face or the bottom face of a member. Fracture can be detected even if it is screened by other wires or if the resulting gap in the steel is relatively small. Measurement speed can be increased by magnetizing the tendons by using a large mobile magnet and then scanning the magnetic flux of the entire surface.

The tendons were magnetized with an electromagnet placed on the surface of concrete and then the magnetic leakage fields are monitored for any irregularities which might be caused by fractures in wires [2].

Individual fractured wires can be detected even if they are inside a bundle of several intact strands. When testing tendons from the upper surface, it is possible to speed up the process with the use of a large mobile magnet to magnetize several tendons of lengths up to 11.5 ft. (3.5 m) at the same time. The development and the applicability of a test vehicle and the large magnet were shown together with the experimental data. The measuring speed has been increased by several magnitudes which allow testing without severe disruption to traffic. This equipment, in conjunction with specialized software, can provide data which identify the location of the prestressing steel. In doing so, the absence of prestressing steel can be seen, thus indicating a damaged strand. The resolution is approximately 0.8 in. (20 mm). This process can be performed quickly; up to 43,000 ft^2 (4,000 m^2) can be covered in one shift. While there are still some concerns regarding false positive readings (where the equipment suggests a fracture which does not occur), additional time can be spent in these areas and will likely improve results.

When considering the local nature of impact damage, it is likely that this technique will not be needed to review a large area, but rather only the affected girders. Therefore, the area under investigation will be well defined and the possibility of false positives reduced.

Commercially available systems are primarily aimed at detection of flaws/damage in prestressed slabs.

Available systems could be readily adapted to high-speed applications on bridge soffits. Regardless, this technique is very promising in near-term.

3. **Acoustic Emission** (AE)

The acoustic emission (AE) technique is a noninvasive, nondestructive method that analyzes noises (“events”) that are created when materials (i.e. concrete or prestressing steel) deform or fracture [2]. Events occurring inside the structure create waves that are collected at the surface by receivers [2]. Continuous monitoring is carried out with sensors.

Monitoring can be used to identify new cracks but will not provide information on previously existing damage. The method is applicable for real-time health monitoring of a bridge or a girder. AE was very successfully used to quantify and precisely locate damage to two prestressed box girders tested to failure [2].

Additionally, this work used AE testing to evaluate the effectiveness of the fiber-reinforced polymer (FRP) repair used on damaged portions of the girders by comparing pre- and post-repair results. Because support conditions varied between pre- and post-repair conditions, this comparison was unable to draw any conclusions. Regardless, AE monitoring of the girders prior to and after the repair yielded independent, but informative results, confirming the usefulness of AE monitoring for prestressed girders as well as FRP repairs.

AE is a viable method of continuous structural health monitoring. Alternate approaches using known applied loads (trucks) have been demonstrated to be viable methods of inspection (as different from monitoring). The methods are very promising and viable for ‘problem structures’, although some technical hurdles/ obstacles remain before wide-spread deployment is practical. Again, it is important to reiterate/ repeat that AE methods are presently ‘baseline’ techniques, that is, they are unable to capture damage occurring before monitoring is initiated. Some studies have been initiated on ‘non-baseline’ damage assessment using AE-based methods, but these remain in the research stage [2].

4. **Linear Polarization**

In this method, the corrosion rate of a metal is measured from its polarization resistance [2]. This is determined by applying a small DC voltage. This method is widely used both in the field and for research. The reference provides average values for resistance, which are correlated to corrosion. The advantage of this method is that commercial equipment is available to measure polarization (separation) resistance of existing structures in the field and also the results can be obtained quickly.

There are also some limitations to this technique. The whole reinforcing bar is polarized and a single value is determined as the resistance, which is essentially an average result for all steel in the vicinity.

This method also enables rapid corrosion rate measurement and is useful for measuring low rates of corrosion, less than 2.5 microns/year [2], but the problem here is that it assumes corrosion to be uniform throughout the reinforcement section which is not true for the usual case. Therefore, localized corrosion in the form of pitting is averaged over the entire strand length. Also, the method assumes that concrete resistivity is low whereas it is usually high. Especially in conditions where concrete is dry, this assumption may lead to significant error.

The measurement of the polarization resistance in large concrete structures requires further investigation especially in the presence of corrosion mitigation measures that limit current spread in the strand. Additionally, probes cannot distinguish between reinforcement and prestressing steel.

In order to decrease the time required to corrode the strands, either a constant current or voltage was applied, or the specimens were initially produced by using permeable concrete with high w/c ratios or by casting specimens with inclusion of chlorides. Three different techniques were employed to monitor corrosion and determine the effectiveness of several FRP materials on corrosion prevention, which are Half-cell potential measurement, linear polarization measurements and soil resistance. Although unexpected fluctuations were obtained, they are useful for approximate results and in understanding the general trend and increase in corrosion.

5. Electrical Resistance

Electrical resistance of the steel strands is being used to detect corrosion [2]. Since resistivity is constant for a given steel sample, change in electrical resistivity must be caused by a change in cross-section, most probably due to corrosion. As the section gets smaller, electrical resistivity increases. It is assumed that all wires undergo the same amount of corrosion [2]. A drawback of the method is that short circuits at the anchorage points or within the structure (stirrups) affect results.

The resistivity of concrete when exposed to an electrical current is highly correlated to the amount of ions in the concrete and the state of the reinforcement. Highly resistive concrete has little possibility of corrosion [2]. Therefore it is possible to assess the degree corrosion by determining the resistivity of the section.

Similar to linear polarization, this method has limited usefulness *in situ* and its use in the immediate future to assess impact (and associated corrosion) damaged prestressed concrete structures in not anticipated.

6. Fiber Optic Sensors

The basic idea of the stranded optical fiber sensor is to measure the degree of attenuation/weakness/ reduction of light due to micro bending [2]. In a wire, as the strain or stress increases, so does the amount of attenuation. Analysis of the ratio of light input to output when compared to a reference wire enables the determination of internal stress levels of the wire. Fiber optic sensors have been demonstrated to be a reliable and durable method for assessing displacement or strain. Hardware is not inexpensive at this time.

7. Impact-Echo

The impact-echo method relies on a stress pulse introduced in the structure by a small impact and the reflected stress wave collected by a nearby transducer. The time of travel is calculated from the reflected stress wave and the corresponding frequencies denote locations of reinforcement or voids. The main difference from radar is the use of low frequency waves (up to 60 kHz) which addresses some problems related to non-homogeneity of concrete. Although the method is relatively successful in locating voids and reinforcement, it is not suitable for the use of assessing corrosion [2]. The impact echo technique with a scanning technique called “stack imaging of spectral amplitudes based on impact-echo” to detect voids in grouted post-tensioning ducts.

As per NCHRP laboratory demonstrations, apparent deterioration of concrete modulus as function of load correlated well with physical measures of accumulated damage (strain and displacement). Despite these successful, the impact-echo method is not practical for field implementation.

8. Surface Penetration Radar

Surface penetrating radar works on the principle of reflection of high frequency electromagnetic pulses from interfaces between materials with different dielectric constants such as steel, concrete and voids [2]. The transducer is passed over the section to locate the position and depth of tendons. However, this technique can only give relative results and is not suitable for detecting corrosion.

It should be used together with destructive methods for an estimation of tendon loss. This technique is not well suited to assessing state of deterioration/damage although appropriate for identifying *in situ* conditions if these are not known.

9. Magnetic Field Disturbance

The magnetic field disturbance method consists of applying a constant magnetic field to the member and scanning all assessable surfaces for any abrupt change in the field which might indicate a flaw in the strands [2]. This method is able to detect corrosion and strand failures but is limited in its resolution and accuracy. Each type of defect: pitting, notches, loss of section, etc. has its own unique signature which enables the user to distinguish between them. However, stirrups and other metallic objects embedded in the structure may hide the actual metal loss or defects and lead to errors [2]. The detectable defect size is 5% loss of section when there are no stirrups. When stirrups are present, there is a significant loss in resolution, and the detectable defect size increases to 40% loss of section when the stirrups are spaced at 400 mm. With further decrease in the spacing of stirrups, the method loses its capacity to give accurate results. A high level expertise is required to interpret the results. Furthermore, *a priori* knowledge of the structure such as locations of reinforcement is required to be able to interpret results. This approach is currently impractical for field application, although improvements in resolution are expected to improve its viability.

As this research title shows that the first step of the research is to identify damages that's why introduced about inspection is desirable. From the beginning of this paper some defects that might be occurs in bridges were explained and those damages could be identify by using one of the inspection technique which are easy available. All inspection should be guided by the *AASHTO Guide Manual for Bridge Element Inspection* (2011). In general, **visual** and **manual** inspection is the only practical triage (assessing or sorting according to quality) tool for impact-affected prestressed concrete girders [1.4]. A significant goal of this triage is to identify locations requiring further nondestructive or destructive evaluation. A skilled inspector, familiar with the structure is able to provide a remarkably accurate assessment of the condition of the structure although is unlikely to be able to accurately quantify many damage types. However since the prestressing technology practiced in Ethiopia has very soon, there is no skilled inspector in this area.

2.2.4 DAMAGE RATING:

Generally after the inspection work has been carried out for prioritization purpose rating is vital. Based on the load carrying capacity bridge attribute could be categorized in to primary members and secondary members.

Primary members

- Deck Slab
- main member (Girder, Truss, Beam)
- Main structure, foundation
- Cables
- Anchorages

Secondary members

- Diaphragm
- Bracing
- Railing
- Expansion joint
- Bearing
- Restrain
- scour protection

- Bank protection
- Wing Wall
- Approach

The following rating system is adopted from [23]

Rating System

Every defect and damage found shall be rated based on the following rating system. Because the rating system reflects the specific conditions and desirable interventions taken into account the present situations in Ethiopia, inspectors shall strictly follow the rating (ranking) description table.

Table 2-0-2 ERA bridges condition rating system

Rating (Ranking)	**Condition**	**Desirable Intervention**
A	Serious/major damages, defects, and deterioration causing reduction of load carrying capacity	Urgent repair
B	Major damages, defects, and deterioration affecting reduction of durability	Repair but not urgent.
C	Minor/no damages, defects and deterioration.	Routine maintenance

The rating system settled only by the defect dimension. The traffic volume shall consider. For the crack case rate for flexure and shear shall have not the same rating. Scour at the abutment and at the pier shall have different rating. These two items are the main defects that lead to fall the bridges.

Table 2-0-3 ***According to Japan Bridge inspection Hand book 2nd Edition, bridges Condition has rated as*:**

Damage rating	% reduction in capacity of the attributes to perform its intended function	
	Main structural part	Secondary structures
Good	0-5%	0-10%

Fair	5-20%	10-40%
Poor	20-30%	40-60%
bad	>30%	>60%

However, as per ERA bridge condition three categories are considered as Good, Fair and Bad.

Table 2-0-4 Damage Rating for Deck Slab [27]

Damage	Rank "A"	Rank "B"	Rank "C"
Cracking	Single crack width of more than 5 mm or multiple cracks width of more than 3 mm, together with water leakage, free lime or salt. In bearing area, single crack width of more than 3mm and multiple crack width of 1 mm which causes reduction in loading capacity.	Single crack width of more than 3 mm or multiple cracks width of more than 1 mm, together with slight water leakage. In bearing area, single crack width of more than 1mm and multiple crack width of less than 1 mm which causes reduction in durability.	Single crack width of more than 1 mm or multiple crack width of more than 1 mm. In bearing area single crack width of less than with no water leakage.
Peel Off	Serious peel off which affects the safety of third parties due to a possibility of concrete falling down. Total area is more than 900 cm2.	Wide range of peel off due to rebar corrosion, which causes a reduction in loading capacity. Total area is between 900 – 400 cm2.	Small range of peel off due to external forces which affects the durability of the deck slab. Total area is between 400 - 100 cm2.
Rebar Exposure	Serious and wide range of rebar exposure together with corrosion which causes a reduction in loading capacity.	Partial rebar exposure with corrosion due to expansion or rebar, peel off or honeycomb, which causes negative effect on loading capacity.	Partial rebar exposure without corrosion, which affects the durability of the deck slab.
Water Leakage	Serious and wide range of water leakage through cracks or voids which may progress and cause a reduction in loading capacity, together with leaching free lime or salt.	Serious and wide range of water leakage through cracks or voids which cause a reduction in the durability, together with leaching free lime or salt.	Localized or partial water leakage, which has negative effects on the deck slab.

Table 2-0-5 Damage Rating for Concrete Girder

Damage	Rank "A"	Rank "B"	Rank "C"
	Single crack width of more than 5 mm or multiple cracks width of more than 3 mm, together with water leakage, free lime or salt. In bearing area, single crack	Single crack width of more than 3 mm or multiple cracks width of more than 1 mm, together with slight water leakage. In bearing area, single	Single crack width of more than 1 mm or multiple crack width of more than 1 mm. In bearing area single

Cracking	width of more than 3mm and multiple crack width of 1 mm which causes reduction in loading capacity.	crack width of more than 1mm and multiple crack width of less than 1 mm which causes reduction in durability.	crack width of less than with no water leakage.
Peel Off	Serious peel off which affects the safety of third parties due to a possibility of falling down.	Wide range of peel off due to rebar corrosion, which causes a reduction in loading capacity.	Small range of peel off due to external forces which affects the durability of the beams.
Rebar Exposure	Serious and wide range of rebar exposure together with corrosion serious peel off or honeycomb, which causes a reduction in loading capacity.	Partial rebar exposure with corrosion expansion of rebar, peel off or honeycomb, which causes negative effect on loading capacity.	Partial rebar exposure without corrosion, which affects the durability of the beams.
Water Leakage	Serious and wide range of water leakage through cracks or voids, which may progress and cause a reduction in loading capacity, together with leaching free lime or salt.	Serious and wide range of water leakage through cracks or voids, which may cause and reduction in durability, together with leaching free lime or salt.	Localized or partial water leakage, which has negative effect on the beams.

Table 2-0-6 **Damage Rating for Curb and Railing**

Damage	Rank "A"	Rank "B"	Rank "C"
Cracking	Single crack width of more than 5 mm or multiple cracks width of more than 3 mm, together with water leakage, free lime or salt. In bearing area, single crack width of more than 3mm and multiple crack width of 1 mm which causes reduction in loading capacity.	Single crack width of more than 3 mm or multiple cracks width of more than 1 mm, together with slight water leakage. In bearing area, single crack width of more than 1mm and multiple crack width of less than 1 mm which causes reduction in durability.	Single crack width of more than 1 mm or multiple crack width of more than 1 mm. In bearing area single crack width of less than with no water leakage.
Peel Off	Wide range of peel off exists and lives, and has possibility to occur in the future, which may cause injury to third parties by concrete falling out.	Peel off which affects seriously in reducing anchorage strength of railings	Shallow peel off exists, which has no effect in durability of curb.
Rebar Exposure	Main rebar exposed in more than 100 cm width, and corroded causing a reduction in loading capacity	Main rebar exposed between 50-100 cm width, and corroded causing a reduction in loading capacity	Rebar partially exposed, corroded slightly.

Deformation	Railing deformed which has a possibility to cause any accidents	Railing deformed seriously, or deformation, which causes inconvenience for pedestrians.	Part of a Railing deformed at a place by external force.
Corrosion	Corroded seriously in reducing loading capacity.	Corroded seriously but locally.	Corroded slightly or no corrosion.
Missing	Components missing which have a possibility to cause any accidents.	Serious missing of components, which causes inconvenience for users.	Components missing partially

Table 2-0-7 **Damage Rating for Concrete Pier, Foundation, Abutment and Wing wall**

Damage	Rank "A"	Rank "B"	Rank "C"
Cracking	Single crack width of more than 5 mm or multiple cracks width of more than 3 mm, together with water leakage, free lime or salt. In bearing area, single crack width of more than 3mm and multiple crack width of 1 mm which causes reduction in loading capacity.	Single crack width of more than 3 mm or multiple cracks width of more than 1 mm, together with slight water leakage. In bearing area, single crack width of more than 1mm and multiple crack width of less than 1 mm which causes reduction in durability.	Single crack width of more than 1 mm or multiple crack width of more than 1 mm. In bearing area single crack width of less than with no water leakage.
Peel Off	Serious peel off, which affects the safety of third parties due to a possibility of concrete falling down.	Wide range of peel off due to rebar corrosion, which causes a reduction in loading capacity.	Small range of peel off due to external forces, which affects the durability of the beams.
Rebar Exposure	Serious and wide ranges of rebar exposure together with corrosion, which causes a reduction in loading capacity.	Partial rebar exposure with corrosion due to expansion of rebar, peel off or honey comb, which causes negative effect on loading capacity.	Partial rebar exposure without corrosion, which affects the durability of the beams.
Honeycomb	Wide range of honeycomb with very serious damage to the rebar, which causes the closure of the bridge	Wide range of honeycomb with seriously corrode rebar which causes a reduction in loading capacity	Wide range of honeycomb which affects the durability of the beams
Void	Wide range of voids with serious and continuous water leakage, which causes the closure of the bridge	Wide range of void with serious water leakage, which causes a reduction in loading capacity.	Wide range of void, which affects the durability of the beams.
Displacement	Settlement or displacement in more than 50mm which may progress and cause a hindrance to adjacent bridge elements, or reduce stability.	Settlement or displacement 50-25 mm which may adjacent bridge elements, or reduces stability.	Settlement or displacement in less than 25 mm which causes no negative effect.

Scour *	Foundation is exposed in more than 1 m in depth from the top of footing, caisson or design datum level for pile bent, which may progress and cause a serious reduction in stability of foundation. Major scour problems to element in river confirmed.	Foundation is exposed up to the top of footing or caisson, which may progress and cause a reduction in stability, when left as it is. Major scour problem to element in river suspected.	Settlement or inclination of foundation is identified, however strengthened already. Minor scouring to element in river suspected.

Table 2-0-8 Damage Rating for Embankment

Damage	Rank "A"	Rank "B"	Rank "C"
Depression	Outstanding crack or depression, which may progress and cause a hazard to traffic or third parties. The mean difference is more than 50 mm in elevation and 1.0 m wide.	Serious depression, which may progress and cause a hazard to traffic. The mean difference is 50 - 25 mm in elevation and 1.0 m wide.	Depression of which mean difference is less than 25 mm in elevation.
Erosion	Large scale erosion in the bottom of shoulder to be related in collapse which may progress if left as it is. Large erosion to slope related in collapse of embankment, which may cause a hazard to traffic.	Partial erosion, which may progress if, left as it is. Partial erosion behind abutment in case of open skeletal abutment.	Partial or local erosion, which cause a hindrance to pedestrians.

Table 2-0-9Damage Rating for Bearing

Damage	Rank "A"	Rank "B"	Rank "C"
Main-Damage	Outstanding damage, such as cracking on the main bearing member, which affects the loading capacity on metal bearing Not applicable for rubber bearing.	Partial cracking, corrosion or swelling of main element, which causes negative effect on functions in bearing, rotating and sliding.	Spot cracking, corrosion or swelling of main element, which causes a reduction in bearing, rotating or sliding functions.
	Secondary elements, such as stopper, anti- lift devices, are	Partial cracking, incline or swelling on the secondary	Slight damage.

Parts Missing	broken which reduces bearing function on metal bearing. Not applicable for rubber bearing.	elements which causes negative effect on bearing	
Anchor Damage	Anchor bolts are broken in wide range which loses anchoring functions on metal bearing	Anchor blots are partially broken or corroded on metal bearing. Not applicable on rubber bearing.	Partial corrosion
Bed-Damage	Major splitting of bearing block, which loses bearing function on high block type bed.	Wide range cracking or spalling, which reduces bearing capacity or functions.	Partial cracking or spalling, which causes hindrance to bearing functions. Excessive accumulation of dirt and debris on bearing beds.
Unusual Movement	Excessive and unusual clearance from the original place, which causes a hindrance to rotating or sliding function of the bearings.	Unusual clearance, which reduces rotating of sliding function	Slight unusual clearance.

Table 2-0-10**Damage Rating for Expansion Joints**

Damage	Rank "A"	Rank "B"	Rank "C"
Noise	Excessive noise of metal hammering by Vehicle wheel shock on metal type joints.	Noise by wheel shock on metal or on rubber type joints.	No noise
Water Leakage	Excessive leakage, which causes hindrance to third parties.	Water leakage, which accumulates debris dirt on pier caps, affects seriously and causes a reduction in durability to other bridge components.	Slight water leakage, which causes leakage trace on slab, beam or piers.
Deformation	Deformation or cracking to joint elements, which causes a hindrance to traffic on metal joints.	Hollow or swelling more than 30 mm in depth with length, which has an effect on safe driving.	slight Hollow or swelling which has an effect on smooth driving.
Peel Off	Serious peel off of joint elements which causes a hindrance to safe traffic	Sealing metal is taken away over more than 1 m in length, which has an effect on water leakage and smooth driving.	Seal material is partially taken away, or surface pavement is taken away, on dummy joints.

Missing	Major components missing to joints, which may progress and causes a hindrance to safe traffic.	Joint components missing which has an effect on smooth driving or missing of pavement surface on dummy joints.	Minor Joint components partially missing which causes negative effect on traffic.

2.3 General Damages, Deterioration and Causes of Damages & Deterioration

2.3.1 General Damage

Pre-stressed Concrete girders are frequently damaged by over-height impacts, fire or corrosion [2].In general damage can be categorized in three as minor, moderate and sever. Minor damage to the girders can consist of cracking or shallow spalls, whereas serious damage can extend to concrete, reinforcing steel and prestressing steel. Minor damage can affect the bridge's aesthetic appearance and long term durability, while severe damage can compromise the structural capacity of the bridge. Regardless of the degree of severity of damage, loose concrete from a damaged girder has the potential to fall onto the road and could result in injury to motorists or damage to vehicles.

According to repair manual for concrete bridge elements prepared by Alberta Infrastructure and Transportation Bridge damage can be classified as follow:

1. **High Load Damage**

High load damage may be classified as surface, minor, moderate, or severe. The assessment of the extent of damage to a girder is described below:

1.1. Surface Damage

These are surface scrapes and small nicks less than 6 mm deep. This type of damage does not warrant repairs unless it is associated with other bridge maintenance repairs [9].

1.2. Minor Damage

This is defined as isolated concrete cracks, nicks and spalls up to 30 mm deep with no reinforcing or prestressing strands exposed. Minor damage adversely affects the aesthetics; however the structural capacity is not reduced. It is important to restore concrete cover in order to prevent reinforcing steel from eventually becoming exposed and corroded [9].

1.3. Moderate Damage

Moderate damage consists of concrete cracks and wide spalls exposing reinforcing steel and prestressing strand. There is no immediate effect on the structural capacity. Although cracks and exposed reinforcement can reduce structure life due to corrosion and freeze thaw action [9].

1.4. Severe Damage

This includes exposed and damaged prestressing strands and reinforcing steel along with loss of significant cross section and possible lateral misalignment due to girder distortion [9].

2. Fire Damage

It is important to estimate the flame temperature in order to determine the extent of fire damage. The flame temperature can be estimated by assessing the damage on surrounding material, or it can be estimated more accurately with the use of available computer software [9].

i. Temperature Effects on Concrete

General

Moisture trapped in capillary and gel pores may form superheated steam and create a damaging force when it cannot escape. “Moisture clog spalling” occurs when moisture migrates from hot to cold through a relatively impermeable.

Repair Manual for Concrete Bridge Elements concrete. The resulting bursting pressure causes spalling at the hot surface.

During fire suppression, water quenching exacerbates thermal shock resulting in micro-cracking and strength loss. The chemistry of concrete at various temperatures is described below:

• At 100°C - loss of free uncombined water from cement paste with little damaging effect.

• At 100° to 150°C - loss of chemically bonded water commences above 100°C. Dehydration and paste shrinkage occurs along with thermal incompatibility between paste and aggregate. Aggregate deterioration results in micro-cracking and worsening of physical properties.

• Above 400°C - calcium hydroxide in the cement paste dehydrates to calcium oxide that on contact with water, as in fire suppression, rehydrates to a damaging 14% increase in volume.

• From 400° to 800°C - gradual, irreversible dehydration of silicate in cement paste. The practical end point for concrete is approximately 540°C, beyond which the concrete becomes friable, porous and can usually be broken down easily. [9]

Physical Properties

• Significant loss of compressive strength after cooling from elevated temperature; about 30-40% for siliceous aggregate concrete after heating to 300°C.

• Significant loss of modulus of elasticity; approximately 40% at 300°C and 50% at 540°C.

• Increased creep and stress relaxation at elevated temperature. At 300°C the observed creep rate was three times those at 125°C. Stress relaxation tests under constant strain for five hours resulted in stress reduction of 2, 32 and 74% at 24, 315 and 650°C respectively.

ii. Temperature Effects on Reinforcing Steel

Normal reinforcing steel loses yield strength with increase in temperature; the loss is about 50% at 600°C. The modulus of elasticity also reduces significantly at elevated temperatures. However, the original properties are recovered on cooling provided the maximum temperature has not exceeded the austenitic transformation temperature of 720°C for a period long enough for significant grain size coarsening. (9)

iii. Temperature Effects on Prestressing Strand

Prestressing strands loose ultimate strength more sharply with an increase in temperature as compared to reinforcing steel. The loss is about 50% at 430°C. Creep and the potential for stress relaxation also increase with rise in temperature.

iv. Temperature Effects on Concrete-Steel Interaction

Bond

The bond between steel and concrete may starts to deteriorate above 315°C. When areas of reinforcing steel have been exposed by spalling, bond in the adjacent covered steel may be adversely affected because of the much greater thermal conductivity of steel compared with the cover concrete [9].

Thermal Expansion Consideration

The coefficients of thermal expansion of concrete and reinforcing steel are similar at normal service temperature; however they become increasingly dissimilar as temperature rises. Differential thermal expansion between concrete and steel produces considerable tangential (bursting) stresses in prestressed concrete.

3. Corrosion Damage

It is vital to determine the full extent of corrosion damage on prestressing strands. When corrosion damage is suspected, concrete removal is required to visually assess the extent of damage to prestressing strands. There may be severe pitting or actual breakage of strand wires. Load carrying capacity may be jeopardized with strand damage. Due to stress corrosion phenomena, corrosion of stressed strand may accelerate much faster than corrosion of rebar. Pitting is not acceptable on prestressing strand [2 and 16].

2.3.2Deterioration of pre-stressed concrete bridges

Deterioration of prestressed concrete bridges is more serious than that of the deterioration of reinforced concrete bridges and steel bridges. There are several reasons why infrastructures deteriorate, among them the main reason are weathering, delayed maintenance, aging and over height vehicular impact. Prestressed concrete bridges are constructed with pre-tensioning and post tensioning strand as a primary strengthening system. Therefore, in these section mainly discussed about corrosion of prestressing steel.

Tendon corrosion

In the present time, most structures are constructed using prestressing system. However, they are considered as maintenance free, but not. Since prestressing strands designed to carry a significant amount of load with a small cross section, a much smaller cross section loss from strand or wire will lead to structural failure or strand deboned from the concrete and eventually breaks. The structure may corrode without producing outward evidence such as rust staining, cracking or spalling because the tensile stress that the small cross section of steel generates in the cover concrete are small. Prestressing strands are much more sensitive for corrosion.

Prestressing tendons may corroded by various means some of them were discussed as

1. metal properties
2. the quality of the concrete that surrounds the wire or strand
3. Service conditions

Metal properties

The properties of the metal are determined by its chemical composition, thermal treatments applied to achieve the desired microstructure, and the methods used to draw the wire/bar and to relieve stress after drawing [14].

The extra energy input used to produce their higher yield strength means prestressing steels are generally less corrosion-resistant than the steels used in the reinforcing bar [14].

Nevertheless, the corrosion resistance of any prestressing steel will be satisfactory provided that its mechanical properties and composition, and the process by which the strand or wire is manufactured are optimized. Similarly, any prestressing steel can be susceptible to corrosion if these properties are not optimized. The effects of metal composition, structure and mechanical properties on the corrosion resistance of prestressing steels, including examples of prestressing steel failures, are described by [14].

An increase in carbon content or the presence of other elements known as poisons (such as phosphorus, antimony, tin, Sulphides and arsenic), particularly as inclusions at metal grain boundaries, can increase the amount of hydrogen entrapped in the steel lattice, increasing the possibility of hydrogen embrittlement. Hydrogen embrittlement may increase the risk of failure if the wire or strand is exposed to a corrosive environment. High strength steels are particularly susceptible to hydrogen embrittlement because of their high carbon content [14].

Concrete quality

Steel in concrete is normally protected from corroding by the cement paste’s high alkalinity and its relatively low permeability to moisture, oxygen and chlorides. Corrosion will be initiated if the alkalinity of the cover concrete is reduced by carbonation or if it is contaminated by chlorides [14].

Sufficient depth and quality of concrete cover is essential to protect the prestressing steel from the ingress of moisture, oxygen and chlorides. Selection of an appropriate mix design is important, but cracks and voids will increase the permeability of even the best concrete mix designs. Significant corrosion damage is more likely when the concrete’s permeability is increased by inadequate compaction, inappropriate mix design or insufficient thickness over the steel. Should the prestressing steel fail, deeper cover will reduce the risk of the cover concrete cracking or spalling, and the risk of the strand or wire bursting out of the element.

Service conditions

Although corrosion can be initiated by loss of concrete alkalinity by carbonation, most cases of prestressing corrosion reported in the literature are related to entry of moisture and chlorides caused by poor drainage. This poor drainage results from poor design and poor maintenance of features such as drains and joints [14].

Chlorides are a particular problem because they can cause much localized corrosion pitting, which may reduce the cross-section sufficiently to cause the steel to fail under a normal working load. Acidification of corrosion pits may lead to hydrogen embrittlement. In addition, corrosion may be promoted at lower chloride concentrations than for unstressed steel. Steel may be contaminated with chlorides before being cast into the concrete, e.g. by storage on site in a marine environment, or it may be contaminated during the service life of a structure exposed to seawater, sea spray or de-icing salts. Chlorides may also be introduced into the concrete at the time of construction in the form of accelerating admixtures based on calcium chloride.

Stray currents from electrical or catholically protected services may also induce corrosion. Corrosion induced by stray currents is easily detected by a characteristic appearance [14].

Overloading can cause the premature failure of a wire undergoing general corrosion. Pure overloading is usually characterized by a ductile failure mode, although the relatively lower ductility of prestressing steels may make a ductile failure difficult to detect.

In addition to hydrogen embrittlement and stress corrosion, corrosion fatigue and fretting corrosion may occur, particularly in partially pre-stressed elements or where the bond to the wire/strand has been lost, e.g. by corrosion of the strand surface.

2.3.3 Cause of damages and deterioration

There are a number of causes of pre-stressed concrete structures. Some of them are listed below.

a) corrosion of embedded strand
   - concrete and the passive layer
   - the roll of chloride ion
   - carbonation
   - dissimilation metal corrosion

b) freeze- thaw deterioration
   - deicer scaling
   - aggregate expansion

c) chemical attack
   - acids
   - salt and alkalis
   - sulfate attack

d) Alkali-aggregate reaction
   - Alkali-silka reaction
   - Alkali-Carbonate reaction

e) Abrasion and erosion
   - Traffic surface
   - Hydraulic structure

f) Fire and/or heat

g) Restrain to volume change
   - Plastic shrinkage crack
   - Drying

- Thermal stress

h) Over loading, over height and impact

i) Loss of support: loss of ballast, sleeper

j) Surface defect (rail uniformity)

- Formed surface
- Finished surface

2.3.3.1 Corrosion of embedded strand

Corrosion of strand steel and other embedded metals is the leading cause of deterioration in concrete. When steel corrodes, the resulting rust occupies a greater volume than the steel. This expansion creates tensile stresses in the concrete, which can eventually cause cracking, delamination, and spalling

Steel corrodes because it is not a naturally occurring material. Rather, iron ore is smelted and refined to produce steel. The production steps that transform iron ore into steel add energy to the metal.

Steel, like most metals except gold and platinum, is thermodynamically unstable under normal atmospheric conditions and will release energy and revert back to its natural state—iron oxide, or rust. This process is called corrosion.

Concrete and the Passivating Layer

Although steel's natural tendency is to undergo corrosion reactions, the alkaline environment of concrete (pH of 12 to 13) provides steel with corrosion protection. At the high pH, a thin oxide layer forms on the steel and prevents metal atoms from dissolving.

Because of concrete's inherent protection, reinforcing steel does not corrode in the majority of concrete elements and structures. However, corrosion can occur when the passivating layer is destroyed. The destruction of the passivating layer occurs when the alkalinity of the concrete is reduced or when the chloride concentration in concrete is increased to a certain level.

The Role of Chloride Ions

Exposure of reinforced concrete to chloride ions is the primary cause of premature corrosion of steel reinforcement. The intrusion of chloride ions, present in deicing salts and seawater, into reinforced concrete can cause steel corrosion if oxygen and moisture are also available to sustain the reaction. Chlorides dissolved in water can permeate through sound concrete or reach the steel through cracks. Chloride-containing admixtures can also cause corrosion.

No other contaminant is documented as extensively in the literature as a cause of corrosion of metals in concrete than chloride ions. The mechanism by which chlorides promote corrosion is not entirely understood, but the most popular theory is that chloride ions penetrate the protective oxide film easier than do other ions, leaving the steel vulnerable to corrosion. The risk of corrosion increases as the chloride content of concrete increases. When the chloride content at the surface of the steel exceeds at certain limit, called the threshold value, corrosion will occur if water and oxygen are also available. Federal Highway Administration (FHWA)

Carbonation

Carbonation occurs when carbon dioxide from the air penetrates the concrete and reacts with hydroxides, such as calcium hydroxide, to form carbonates. In the reaction with calcium hydroxide, calcium carbonate is formed:

$$Ca(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O$$

This reaction reduces the pH of the pore solution to as low as 8.5, at which level the passive film on the steel be not stable. Carbonation is generally a slow process. In high-quality concrete, it has been estimated that carbonation will proceed at a rate up to 1.0 mm (0.04 in.) per year. The amount of carbonation is significantly increased in concrete with a high water-to-cement ratio, low cement content, short curing period, low strength, and highly permeable or porous paste.

Carbonation is highly dependent on the relative humidity of the concrete. The highest rates of carbonation occur when the relative humidity is maintained between 50% and 75%. Below 25% relative humidity, the degree of carbonation that takes place is considered insignificant. Above 75% relative humidity, moisture in the pores restricts CO_2 penetration (ACI 201 1992).

Carbonation-induced corrosion often occurs on areas of building facades that are exposed to rainfall, shaded from sunlight, and have low concrete cover over the reinforcing

Dissimilar Metal Corrosion

When two different metals, such as aluminum and steel, are in contact within concrete, corrosion can occur because each metal has a unique electrochemical potential. A familiar type of dissimilar metal corrosion occurs in an ordinary flashlight battery. The zinc case and carbon rod are the two metals, and the moist paste acts as the electrolyte. When the carbon and zinc are connected by a wire, current flows. In reinforced concrete, dissimilar metal corrosion can occur in balconies where embedded aluminum railings are in contact with the reinforcing steel.

2.3.3.2 FREEZE-THAW DETERIORATION

When water freezes, it expands about 9%. As the water in moist concrete freezes, it produces pressure in the capillaries and pores of the concrete. If the pressure exceeds the tensile strength of the concrete, the cavity will dilate and rupture. The accumulative effect of successive freeze-thaw cycles and disruption of paste and aggregate can eventually cause significant expansion and cracking, scaling, and crumbling of the concrete. Local weather records can also be referenced to more precisely determine the severity of exposure.

The resistance of concrete to freezing and thawing in a moist condition is significantly improved by the use of intentionally entrained air. Entrained air voids act as empty chambers in the paste for the freezing and migrating water to enter, thus relieving the pressure in the capillaries and pores and preventing damage to the concrete.

Concrete with low permeability is also better able resist the penetration of water and, as a result, performs better when exposed to freeze-thaw cycles. The permeability of concrete is directly related to its water-to-cement ratio. The lower the water to- cement ratio, the lower the permeability of the concrete.

Deicer Scaling

Deicing chemicals used for snow and ice removal, such as sodium chloride, can aggravate freeze-thaw deterioration. The additional problem caused by deicers is believed to be a buildup of osmotic and hydraulic pressures in excess of the normal hydraulic pressures produced when water in concrete freezes. In addition, because salt absorbs moisture, it keeps the concrete more saturated, increasing the potential for freeze-thaw deterioration. However, properly designed and placed air-entrained concrete can withstand deicers for many years.

In the absence of freezing, sodium chloride has little to no chemical effect on concrete. Weak solutions of calcium chloride generally have little chemical effect on concrete, but studies have shown that concentrated calcium chloride solutions can chemically attack concrete. Magnesium chloride deicers have come under recent criticism for aggravating scaling. One study found that magnesium chloride, magnesium acetate, magnesium nitrate, and calcium chloride are more damaging to concrete than sodium chloride (Cody, Cody, Spry, and Gan 1996). Deicers containing ammonium nitrate and ammonium sulfate should be prohibited because they rapidly attack and disintegrate concrete

Aggregate Expansion

Some aggregates may absorb so much water (to critical saturation) that they cannot accommodate the expansion and hydraulic pressure that occurs during the freezing of water. The result is expansion of the aggregate and possible disintegration of the concrete if enough of the offending particles are present. If a problem particle is near the surface of the concrete, it can cause a pop out.

2.4 Design Consideration of Limits on Deterioration of Prestressed Concrete

There are two main area of interest

- Design which prolong the life of the bridge and prevent deterioration
- Design feature which make routine inspection and routine maintenance

According to maintenance of cable-stayed bridge research paper prepared by Shin-Gia Chen, Massachusetts Institute of Technology Jun 2000, some practical design consideration for better maintenance has been listed below.

i) Elimination of railway irregularities

Irregularities in bridges are important causes for traffic impact. Bridges should therefore be designed to minimize the occurrence of such regularities; joints and change of elevation are the main causes of railway irregularities.

Most bridges need joints to allow them to flex or expansion and contraction. If the joint seal does not fix to the bridges or the joint deterioration easily and cause some discontinuity along the travel way joint will cause irregularities. As all the joint must be brought up to the surface; as well as the joint often shorten life span than other part of the bridge. The best way to eliminate such joint related problem is to have as few as possible.

Change of level often occurs in the point where approach meets bridges. A common condition is that the approach settles to some extent as a result of traffic impact and embankment consolidation but the bridge settles only a little.

Preventive measure

a) Compact equally in the whole stretches of the approach road.
b) Approach embankment should be constructed in advances to preload the ground and the consolidation could be preventing after the super structural part constructed.

c) Abutment and Wing wall should be designed so that compaction machinery could operate effectively.

ii) Design for parts accessible to traffic.
iii) Joint
iv) Sleeper spacing
v) Ballast type
vi) Surfacing
vii) Drainage
viii) Bearing
ix) Durable concrete
x) Steel/strand
xi) Substructure and foundation

2.5 Maintenance and Rehabilitation of PC Bridges

2.5.1. Maintenance

The fundamental objective of Maintenance is to assure the continuous performance of a structure or to preserve the structure in such a way that it will functional satisfactory at the various limit states. The overall objective of prestressed concrete structure is to identify the need for structural integrity, periodical surveillance, repair, rehabilitation and replacement depending upon the local conditions. The maintenance management must also provide guideline and methodologies to enable local engineers to reach rational, cost effective decision regarding maintenance and rehabilitation of distress structures.

Good maintenance practice required periodical surveillance/observation, identification of local damage deterioration and loss of durability of the structure due to environmental and other load effects. In prestressed concrete structure the primary problem encountered in the damage cause to the anchorages and unbounded tendons due to rusting under exposure to humid weather condition. Therefore, periodic inspection, repair and rehabilitation are the primary aspects of good and effective maintenance. These aspects assuming more and more important due to the rapid increasing in the number of prestressed concrete structures have been constructed the last few years in our country Ethiopia.

In general, bridge maintenance contains the following component:

- Initial determination of their actual condition
- Judgment of structure in terms of safety

- Predicting of their future behavior resulting from aging
- Definition of specific maintenance strategy
- Allocation of limit budget to specific item
- Execution of these strategies with adequate men and means
- Back coupling with maintenance strategies and design

2.5.2. PREVENTIVE MAINTENANCE

Preventive maintenance can be defined as the act of keeping a structure in its as-built condition and/or protecting it from inevitable/predictable/expected

Deterioration might be occurring due to environment, traffic vibration and deicing chemicals. In some cases, structures are built with flaws such as cracks in concrete which require action to prevent moisture and chlorides from infiltrating the micro-structure and causing early deterioration [5]

Preventing concrete deterioration is much easier and more economical than repairing deteriorated concrete. Preventing concrete deterioration begins in the design of the structure with the selection of the proper materials, mixture proportions, concrete placement, and curing procedures. Even a well-designed concrete will generally require follow-up maintenance action. [4]

According to ARMY and AIR force maintenance manual, the primary types of maintenance for concrete are surface protection, joint restoration, and cathodic protection of the steel. Surface maintenance involves the application of coatings for protective purposes. Joint problems are usually treated with one of a variety of types of joint sealers, and cathodic protection involves the use of anodes connected to the reinforcing bars which will deteriorate in place of the reinforcing bar.

2.5.3 Rehabilitation

Rehabilitation of structures may be required due to several reasons. Some common causes are:

- Construction deficiency
- Environmental effects
- Design deficiency or design error
- Overloading of structure either due to unanticipated loading or due to accident
- Over height impact

- User changes in the structure during the service life of structure

The problem and rehabilitation methods of structure are variable and unique. Therefore, the uses of common rehabilitation technique for different structures are limited.

As far as maintenance is concerned, several new cementations material and epoxy resins and compounds have been developed. This material has been highly effective in protecting the basic structure from the distractive effect of several exposure conditions in the environment. It is important that maintenance and rehabilitation engineers must study the basic design, history of construction change in loading on the structure, environmental change etc. After carefully analysis of all these factors the engineer will be able to work out a strategy for a long lasting rehabilitation measure for the distressed structure.

3 Damages Types and Causes of Damages on Prestressed Concrete Railway Bridges

3.1 General

Bridges are affected by microclimates which control the nature of deterioration and hence the types of defects that develop in a certain elements. The super structure is the exposed part and is susceptible to moisture sensitive deterioration such as Freeze- thaw, steel corrosion, as well as traffic effects such as abrasion and impact loading. Although less exposed, the sub structure of a bridge is still susceptible to steel corrosion, particularly where exposed to salt-laden wind in costal environments and where the concrete is wetted by driving rain or water leakage through the deck. Shrinkage crack in concrete bridges deck commonly provides a passage for water leak and steel corrosion may result. Bridge pier, pile and abutment may be exposed to soft water attack and to abrasion caused by aggregate movement in the river bed.

The defects that commonly affected prestressed concrete bridges and their causes are described below

Table 3-1 **Causes of Crack, spalling and tendon corrosion:**

Damage type	Cause	Remark
1. Crack 1.1 Based on the internal 1.1.1 flexural 1.1.2 shear 1.1.3 torsion, and 1.1.4 compression cracks 1.2 based on their direction 1.2.1 Longitudinal cracks 1.2.2 Transverse cracks 1.2.3 Diagonal cracks 1.2.4 pattern or map cracking	i) Fatigue ii) Time-Dependent Effects iii) Temperature Effects iv) Over height Impact damage v) Excessive loading vi) Over increasing of traffic **vii)** Other causes	A crack is a linear fracture in the concrete surface [2, 5, 11, 12, and 22].
2. Spalling	• Over height Impact damage • Dynamic loading • Over loading	A spall is a fragment, which has been detached from a larger concrete mass [15]
3. Corrosion of Tendon	• Metal properties	Even if Corrosion

	• The quality of the concrete that surrounds the wire or strand • Service conditions	of pre-stressing steel is relatively rare, it's seriously reduced the structural capacity of the structure [14]

Table 3-2 **Causes of rail damage**

Damage type	Cause	Remark
1. Rails 1.1 Head Defects – Transverse	- Separations normally originate on the gage side of the rail head - Usually a horizontal separation originating from an internal longitudinal seam, segregation, or inclusion. - overheating of the rail surface produced by the friction. - Insufficient electrode contact overheats the rail. -improper welding techniques	
1.2 Head checking	Shallow surface or hairline cracks that appear in the gage corner of rail head, at any angle with the length of the rail. - due to the interaction between the wheels and the rail	
1.3 Flaking	Caused by the concentrated wheel loads, resulting in severe compressive shear deformation of	Flaking is the result of surface metal friction, flow and plastic deformation.

	the rail surface.	
1.4 Spalling	High horizontal wheel-rail creeping forces, transverse friction forces and extreme wheel-rail contact stresses result in micro-cracking, head checking or chipping.	Spalling is cracking and chipping of the rail surface. Spalling is a progression of head checking and flaking.
1.5 Shelling	High contact stresses from wheel-rail interaction, especially when severe non-conformal wheel-rail contact occurs.	Shelling normally occurs on the upper gage face of the rail head, and extends longitudinally.
1.6 Corrugation	Sliding wheel action, tractive forces, braking forces or lateral motion across the rail surface.	Corrugation is repetitive longitudinal pattern of shallow wavelike depressions along the rail surface.
1.7 Corrosion	wet or damp, effects of abrasion, alkali, lime in concrete, salt near coast lines and electrolysis in electrified areas etc.	Corrosion**:** is disintegration of the rail starting at the surface, from chemical decay, mainly oxidation (rusting). [3 and 24]

Table 3-3 **Causes of Sleeper, Ballast and anchorage failure**

Damage type	Cause	Remark
1. Sleepers/ Ties damage 1.1 longitudinal crack, 1.2 Muscle crack, 1.3 Head zone crack, 1.4 concrete decayed, 1.5 concrete tear, and 1.6 Filling of roll plaque. 1.7 Bending cracks 1.8	incorrect performance of screw driving machine, and incorrect vibration, lack of concrete cover behind plaques, Improper concrete mixing plan, Strikes touched to unload sleeper, ballast pollution and reduced track elasticity ,un-permitted load passing and increased loading cycle, Sleeper instability in fastening point, reduced ballast layer thickness, derailment etc.	Concrete sleeper could be damaged (i) Deteriorations during manufacture and coupling (track paneling) (ii)Deterioration during transportation and installation (iii) Deteriorations during use
2. Ballast 2.1 Crushing	-overloading -mechanical maintenance	The following problem may appear due to ballast failure. - Deteriorating track quality

2.2 Abrasion 2.3 Creation of fine 2.4 Ingress of fines 2.5 Ingress of vegetable	(Machine and manual tamping) -tamping -dynamic track movement under traffic -Impaired by the presence of water	-Discreet track geometry faults (e.g. twist & cyclic top) –risk of derailment -Loss of lateral stability -increased risk of track buckling - Formation of wet beds - damage to the sleepers and bearers - Ponding of water -damage to the formation
3. Failure of anchorage	-Uncontrolled cracking or splitting due to insufficient transverse reinforcement. - Bearing failures immediately behind the anchorage plate	In post-tensioned construction, the prestressing force is transferred to the concrete through relatively small anchorage plates behind the anchorage by bearing. This results in a very high concrete bearing stress behind the anchorage plate.

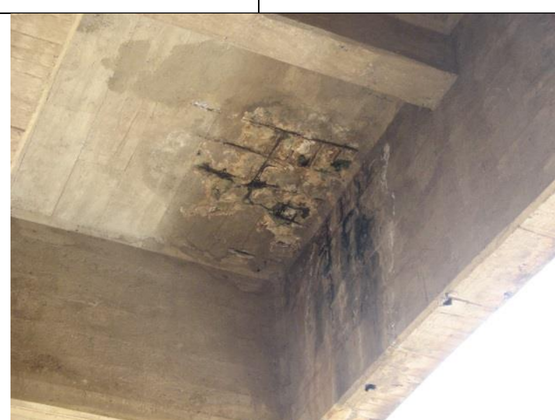
Table 3-4 Possible causes of Bridge bearing failure

Possible Damage type	Causes	Remark
Misalignment, displacement, deformation, damage, spalling, Movement condition etc.	Settlement Pounding effect Excessive loading Seismic activities Improper installation Use of poor concrete for piers and abutments Improper connections and anchorage	Inspection method: Visual inspection -Taking measurements Surveying -knocking off adjacent concrete surfaces in the bearing area, in order to detect hollow spaces
Cracks	Fatigue High stress concentration	Visual inspection Acoustic emissions testing

	Excessive loading Extreme low temperatures Improper connections and anchorage Low bearing capacity	Corrosion sensors Computer tomography Dye penetrant Coating tolerance thermography Radiographic testing Magnetic particle Ultrasonic through method Ultrasonic pulse catch Tensile strength test Chemical analysis Eddy current Ultrasonic testing Robotic inspection
Corrosion	Exposure to salt, de-icing salts etc. Lack of cleaning maintenance Bird nests, excretes and other animal activities Exposure to acidic rainwater, rainwater, snow etc. Ineffective expansion joints Lack or ineffective lubrication Poor or ineffective corrosion protection Bad drainage conditions	Visual inspection Section loss monitoring Coating tolerance thermography

Table 3-5 **Possible causes for Deformation of shape, Scour & erosion and miscellaneous failure**

1. Deformation of shape	-excessive loading, - deterioration of the structural part, - fatigue	Deformation of concrete structure generally induces cracking, [23].

2. Scour & erosion on the underside of sleepers	-improper drainage system -over turning -removal of ballast	-Erosion and scour reduce the embedment of the foundation into the soil, -Erosion and scour increase the un braced length of pile foundations [27]
3. Miscellaneous failure 3.1 Expansion joint 3.2 Railing 3.3 Water leakage and efflorescence	- debonding or being inelastic sealant , movements of angles -aging, collusion and material property. - Cracks, voids, concrete porosity, absence of impermeable wearing course, defect to joint sealants, or blockage of drains are causes.	-On large span bridges steel finger plates and steel sliding plate joints have been used - An extended water leakage leads to the deterioration of the bridge part prone to this specific defect -wide range of water leakage may cause a reduction in loading capacity
 		
4. Design errors	- Incorrect concept - Calculation errors - Drawing/detailing errors	The specification of insufficient concrete cover to reinforcement, which results in cracking and spalling. [23].
5. Construction errors	-Poor materials -Poor workmanship and	Adequate supervision at the construction stage is essential

	equipment -External effects	to ensure that the works comply with the design [23].

Over height Impact damage may causes for the development of spalling, crack corrosion etc. on the prestressed concrete members. Along the Addis- Djibouti road segment there are more than four over pass bridges. The under passing vehicles may have collision or even due to minor contact with the bridge may induce damage. Among the damages one could be spalling, sample can be observed from the exhibited photo below.

Figure 3-1Potential damage on overpass Railway Bridge by under passing vehicular impact, around Welenchiti

Mostly in a railroad system there are three kinds of sleepers, which are concrete, wooden and steel sleeper. Almost all sleepers in the new Ethiopian railroad are using Concrete sleeper types.

Figure 3-2 types of sleeper used on Ethiopian Railroad Bridges

Since all railway bridges in Ethiopia are new so it is difficult to get all the above common defects at these stage. However, to predict future damages on these bridges conducting of condition surveys on highway bridges which are parallel to the railway lines, such as along Sebeta- Dewele rout, is extremely vital.

Therefore, using ERA-BMS output, the common damages, which occurred on concrete bridges along Sebeta- Dewelle route, are identified as follow. Only bridges in fair and bad condition are considered along this road stretch.

Ethiopia Roads Authority Bridge Management Team

Item No	Road Segment	Bridge No	Bridge Name	KM from A.A.	Bridge types	Bridge Length	bridge condition	construction year	current condition
1	Addis -Modjo	A1-1-001	Dengora	25.28	RC Slab Culvert		Bad	1974	Cracking
2	Addis -Modjo	A1-1-003	Unknown	34.75	RC Slab Culvert		Fair	1954	Cracking
3	Addis -Modjo	A1-1-006	Unknown	51.75	RC Slab Culvert	4.8	Fair	2004	Peel Off, Rebar Exposure, Honey Comb, Deck Slab Void
4	Addis -Modjo	A1-1-007	Unknown	51.9	RC Slab Culvert	8.4	Fair	2004	Scour,Rebar Exposure, Honey Comb, Deck SlabVoid
5	Addis -Modjo	A1-1-008	Unknown	52.25	RC Slab Culvert	4.8	Fair	2004	Rebar Exposure
6	Addis -Modjo	A1-1-009	Dengora 2	56.28	RC Slab Culvert	10.4	Fair	2004	Cracking, Peel Off, Rebar Exposure
7	Addis -Modjo	A1-1-010	Udi amelel	58.67	RC Slab Culvert	4.8	Bad	2004	Scour, Erosion, Cracking, Rebar Exposure
8	Addis -Modjo	A1-1-012	Ankara	61.86	RC Slab Culvert	5	Bad	2004	Cracking, Peel Off, Rebar Exposure
9	Addis -Modjo	A1-1-013	Modjo	69.51	RC Deck Girder	74	Fair	1972	Cracking
10	Modjo - Nazret	A1-2-002	Unknown	74.56	RC Slab Culvert	4.8	Fair	1974	Cracking,Peel Off
11	Modjo - Nazret	A1-2-004	Lemma	79.57	RC Box Culvert	4.5	Fair	2003	Cracking, Void on the Deck Slab
12	Nazret - Metehara	A1-3-001	Gibrena	91.89	RC Slab Culvert	6.35	Fair	1974	Scour, Peel Off / Stone deterioration, Cracking,Deck Slab Void

13	Nazret - Metehara	A1-3-004	Denbela	95.5	RC Slab Culvert	4	Bad	1974	Scour, Cracking, Honey comb, Rebar Exposure, Water Leakage, Deck Slab Void
14	Nazret - Metehara	A1-3-007	Teteri-Wacho	117.26	RC Box Culvert	12	Fair	1974	Scour, Cracking, Honey comb, Peel Off , Void on Deck Slab
15	Nazret - Metehara	A1-3-008	Tami Boba	144.5	RC Slab Culvert	4	Bad	1974	Scour, Cracking, Rebar Exposure, Water Leakage
16	Nazret - Metehara	A1-3-009	Crasher	157.5	RC Slab Culvert	4	Fair	1974	Scour, Cracking, Rebar Exposure, Peel Off
17	Nazret - Metehara	A1-3-011	Unknown	175.16	RC Slab Culvert	10	Fair	0	Cracking
18	Nazret - Metehara	A1-3-012	Unknown	175.56	RC Slab Culvert	8	Fair	0	Cracking
19	Bordede - Mieso	A10-1-012	Unknown	255.63	RC Slab Culvert	4.3	Bad	2002	Scour,
20	Bordede - Mieso	A10-1-014	Dire oda	272.05	RC Slab Culvert	11	Bad	1964	Scour, Honey Comb,Drainage Pipe damage
21	Bordede - Mieso	A10-1-018	EJEFAYO	288.48	RC Deck Girder	16	Fair	1964	Rebar Exposure, Curb &Railing Deformation, Drainage Pipe damage
22	Bordede - Mieso	A10-1-021	BECHESA	292.85	RC Deck Girder	16	Bad	1966	Scour, Cracking,
23	Diredawa - Harmukali	C100-001	Lege Hare	506.53	RC Deck Girder	68	Fair	1998	Honey Comb, Water Leakage,Drainage Pipe damage

24	Diredawa - Harmukali	C100-002	Debassay	509.14	RC Deck Girder	19	Bad	1997	Cracking, Water Leakage, Honey Comb, Drainage Pipe damage & block
25	Diredawa - Harmukali	C100-003	Unknown	510.15	RC Deck Girder	18.1	Bad	1997	Cracking, Water Leakage, Honey Comb, Drainage Pipe damage, Rebar Exposure
26	Diredawa - Harmukali	C100-004	Unknown	513.85	RC Slab Culvert	5	Fair	2004	Scour, Cracking, Honey Comb
27	Diredawa - Harmukali	C100-008	Unknown	518	RC Deck Girder	4.4	Fair	2000	Erosion, Rebar Exposure, Honey Comb, Deck SlabVoid
28	Diredawa - Harmukali	C100-009	Butugeder	519.13	RC Deck Girder	17.2	Fair	1997	Rebar Exposure, Honey Comb, deck slab Void, Drainage Pipe damage
29	Diredawa - Harmukali	C100-010	Mure yere	521.49	RC Deck Girder	18.05	Bad	2002	Cracking, Rebar Exposure, Honey Comb, Water Leakage, Curb & Railling Deformation, Drainage Pipe damage
30	Diredawa - Harmukali	C100-012	Degha gebis	526.53	RC Slab Culvert	12.15	Bad	1997	Erosion, Erosion, Cracking, deck slab Void, Drainage Pipe damage
31	Diredawa - Harmukali	C100-014	Unknown	529.1	RC Slab Culvert	6.8	Bad	2002	Erosion, Cracking, Honey Comb
32	Diredawa - Harmukali	C100-015	Unknown	531.76	RC Deck Girder	20.4	Fair	1998	Erosion, Cracking, Honey Comb, Drainage Pipe damage

33	Diredawa - Harmukali	C100-016	Unknown	532.66	RC Slab Culvert	7.4	Bad	2002	Honey Comb, Water Leakage,Drainage Pipe damage, Void on Deck Slab
34	Diredawa - Harmukali	C100-018	Harmukali	540	RC Deck Girder	23.2	Bad	2000	Cracking, Rebar Exposure, Honey Comb, deck slab Void, Drainage Pipe damage
35	Harmukali - Biyikobobe	C100-019	Unknown	550.34	PC Deck Girder	176	Bad	2004	Erosion, Displacement, Cracking, Water Leakage, Rebar Exposure, Honey Comb, Void, Expansion Joint Missing
36	Harmukali - Biyikobobe	C100-022	Unknown	564.37	PC Deck Girder	36	Bad	2004	Rebar Exposure, Honey Comb, Void on deck, drainage Pipe damage, Cracking, Water Leakage, Expansion Joint Missing
37	Harmukali - Biyikobobe	C100-024	Unknown	584.01	RC Slab Culvert	5	Bad	2004	Erosion, Cracking, Rebar Exposure
38	Harmukali - Biyikobobe	C100-026	Dure	588.43	RC Deck Girder	126.4	Fair	2001	Cracking, Water Leakage, Drainage Pipe damage, Expansion Joint Missing
39	Harmukali - Biyikobobe	C100-027	Mefakia	599.76	RC Deck Girder	47.4	Bad	2005	Cracking, Honey comb, Erosion, Peel Off, Rebar Exposure, Expansion Joint Missing
40	Harmukali - Biyikobobe	C100-028	Albinin	617.72	RC Deck Girder	27.6	Bad	2005	Cracking, Honey Comb, Peel Off, Water Leakage, Drainage Pipe damage

41	Harmukali - Biyikobobe	C100-029	Unknown	621.05	RC Deck Girder	15.1	Fair	2005	Cracking, Rebar Exposure, Void on deck
42	Biyikobobe - Dewale	C100-033	Unknown	637.49	RC Slab Culvert	9	Bad	1966	Cracking, Scour, Rebar Exposure, Honey Comb and Wave
43	Biyikobobe - Dewale	C100-034	Unknown	641.88	RC Slab Culvert	5	Bad	1966	Cracking, Scour, Erosion, and Honey Comb
44	Biyikobobe - Dewale	C100-035	Unknown	643.34	RC Slab Culvert	8	Bad	1966	Erosion, Cracking
45	Biyikobobe - Dewale	C100-037	Unknown	674.01	RC Slab Culvert	4.5	Bad	1966	Cracking, Erosion, wave
46	Biyikobobe - Dewale	C100-040	Adelle 1	687.68	RC Slab Culvert	5.6	Bad	1966	Cracking & erosion
47	Biyikobobe - Dewale	C100-042	Unknown	688.48	RC Slab Culvert	4.5	Fair	1966	Cracking, Rebar Exposure, Honey Comb
48	Biyikobobe - Dewale	C100-043	Unknown	690.66	RC Slab Culvert	5.2	Fair	1966	Cracking, Scour, Erosion, Rebar Exposure and Honey Comb
49	Biyikobobe - Dewale	C100-045	Unknown	698.31	RC Deck Girder	29.2	Bad	2005	Cracking, Rebar Exposure, and Honey Comb
50	Biyikobobe - Dewale	C100-046	Unknown	701.49	RC Slab Culvert	4.5	Fair	1966	Cracking, Erosion, Rebar Exposure, and Honey Comb
51	Biyikobobe - Dewale	C100-047	Dewele	714.81	RC Deck Girder	112	Bad	2005	Cracking, Rebar Exposure, and Honey Comb

From the collected data generated the graph shown below

Total No. of defective bridge along Route	51
Cracking	39
Corrosion	25
Deck slab void	14
Peel off/ spalling	8
Scour & erosion	20
Water leakage	7
Honey comb	7
Drainage pipe damage	13
Expansion Joint Missing	4

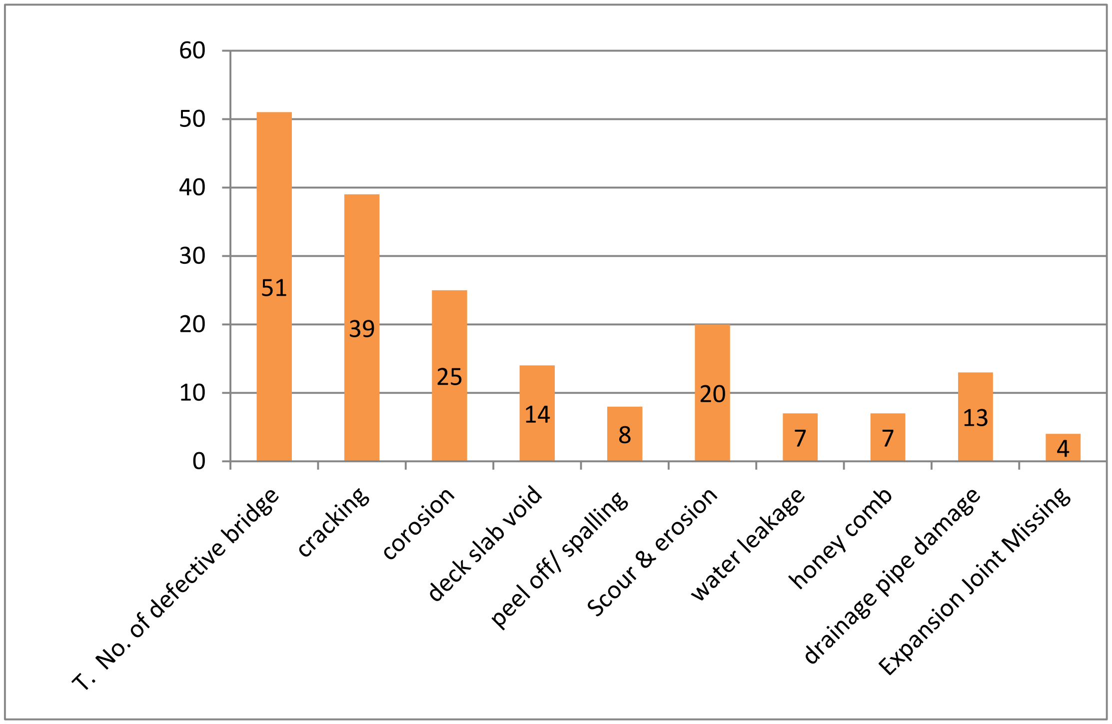


Figure 3-3 Sebeta- Dewele bridges defect chart

As shown in the above chart crack, corrosion and erosion are the most prevalent damages along Sebeta- Dewele route.

BMS software doesn't categorize shear, flexural, torsional…etc. crack, general it call crack. However, from the observed pictures most cracks in this route are shear cracks and others are flexural cracks.

Some damages along this route are illustrated by the following photographs.

Figure 3-4 Corrosion and spaling damages on A1-1-006 River Bridge

Figure 3-5 Peel off, Corrosion and Hony Comb onA1-1-007 river Bridge

Figure 3-6Water leakage and crack on A10-1-018River Bridge

Figure 3-7Shear crack on C100-1-003 River Bridge

Figure 3-8Shear crack, corrosion, spalling, bearing & bearing plate damage on C100-1-018

Figure 3-9: Corrosion, shear crack, spalling damage on C100-1-019 (Harmukali) River Bridge


Figure 3-10 honey comb on C100-1-042 River Bridge


Figure 3-11Shear crack on C100-1-045 River Bridge


Figure 3-12Shear crack, void and spalling on C100-1-047 River Bridge

Condition surveying result for Addis Ababa light rail transport was conducted as follow.

Observed bridge defects on AA LRT bridges

Figure 3-13: Photographs shows Crack, spalling and honey comb on Ayat terminal station bridge

Figure 3-14: Photographs shows spalling **on Railway Bridge Piers around Bambis area**

Figure 3-15 Photographs shows exposed for vehicle collusion **on Railway Bridge piers around Bambis area**

Figure 3-16 Photographs shows exposed for vehicle collusion **on Railway Bridge piers around stadium area**

Figure 3-17: Photographs shows rebar exposure & spalling **on Railway Bridge piers around Firdbet area**

4. Prestressed Concrete Bridge Maintenance Techniques

4.1 General

Damage was classified as minor, moderate, or severe. Minor damage was defined as concrete cracks and nicks, shallow spalls, and/or scrapes and some efflorescence, rust or water stains. Girders with large concrete cracks, and spalls large enough to expose undamaged prestressing tendons, were classified as moderately damaged. Severe damage included exposed damaged tendons, or loss of significant portions of concrete cross section, as well as possible girder distortion resulting in lateral misalignment. Cracks, spallings, tendon corrosion and tendon damage, Rail &Ballast failure on the deck, Sleeper displacement and Loosening of fastenings, problems of Anchorages and bearings, Deformation of shape or Bulging , Support cable and dead man failure, surface erosion and surface deposits/efflorescence, Miscellaneous devices failure and Others are the expected defects of bridge component.

According to different manuals, literatures, researches, brushers, lectures videos, and other related sources the following maintenance techniques for the above defect are adopted.

4.2 Repair technique for concrete structural crack

As discussed in the damage classification section cracks, depends on the severity, can be categorized minor, moderate or severe.

Minor damage is defined as concrete with shallow cracks. Damage at this level does not affect member capacity. Repairs are for aesthetic or preventative purposes.

Moderate damage includes larger cracks that lead to expose strands. Moderate damage does not affect member capacity. Repairs are intended to prevent further deterioration.

Severe damage is any damage requiring structural repairs. Typical damage at this level includes significant cracking that leads to exposed and broken strands.

4.2.1 Minor Damage Repair

A step-by-step outline of the procedure for the repair of minor impact damage to prestressed concrete girders is shown in the following figure. Since minor damage is defined as small cracks, minor damage does not affect the structural performance of a girder. Minor damage is repaired primarily to restore aesthetics and concrete cover. If the durability or aesthetics are not important, the girder not needs to be repaired.

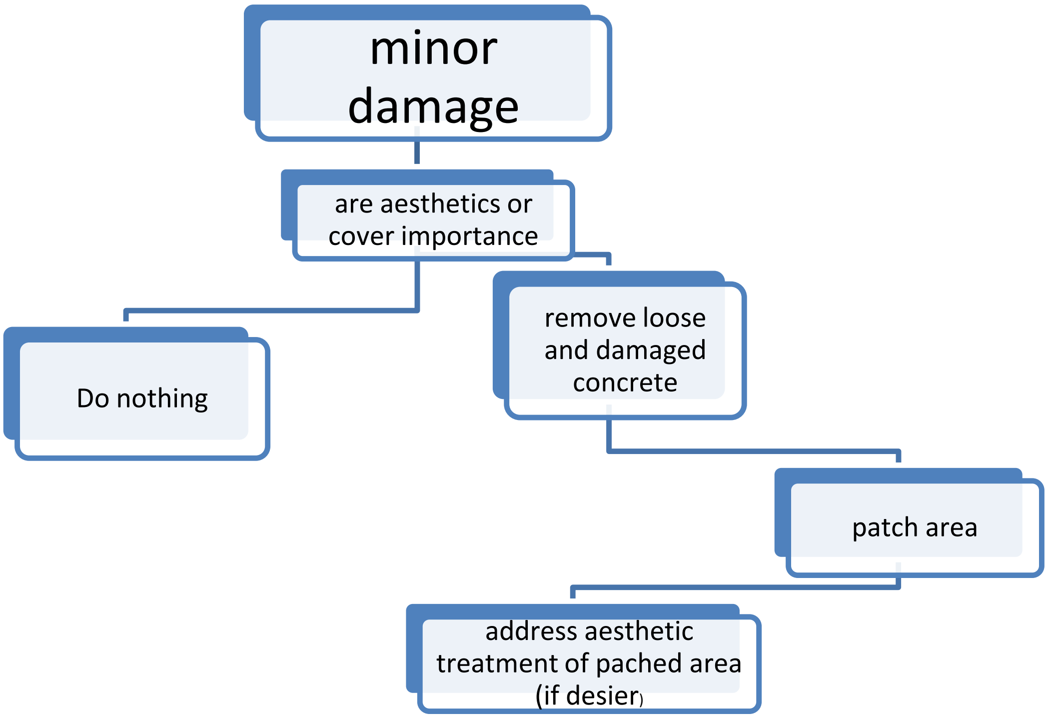


Figure 4-1Minor damage repairs

Minor damage repair procedure

1. Removal of Loose and Damaged Concrete

There is general agreement that before attempting to patch a damaged girder, all loose, cracked, and delaminated concrete should be removed. Only sound concrete should remain to be bonded with the patch material for better bond and performance of the member. It is recommended that the surface be roughened so that coarse aggregate in the base concrete is partially exposed to enhance the bond between the existing concrete and the patching material. After concrete removal the repair surface should be free of dust and foreign materials that can be detrimental to the bond. Cavity edges should be sharply defined.

For minor damage, a chipping hammer can be employed to remove loose and damaged concrete.

Such damage will be superficial, and it is likely that prestressing tendons will not be encountered during the removal process. Extreme care should be taken when using chipping hammers to avoid hitting and/or severing the prestressed strands. Hydro demolition may be a safer alternative since it can be controlled to prevent damage to tendons or to sound concrete.

2. Patching the Damaged Area

Patch materials must be able to withstand the same exposure conditions as the base concrete in a given member. These factors may include: freeze-thaw cycles, exposure to deicing salt, extreme temperatures, rapid temperature changes, and dynamic and static loading. Therefore, the properties of the patch material must be closely evaluated with respect to the properties of the base concrete in a damaged member; otherwise, debonding, cracking, and premature failure of the patch or the surrounding concrete can occur. At times, the repair material may be required to have better durability properties than the base concrete in order to resist environmental conditions which may have been associated with the cause of damage.

The properties that must be considered when selecting a patch material include: compressive strength, rate of strength gain, shrinkage, permeability, and bond strength, generation of heat during curing, and thermal properties as well as durability. A cementitious patch material is most likely to be recommended for use in the repair of impact-damaged girders since its mechanical properties could be made to closely match those of the base concrete. The selected patch material should be suitable for overhead and vertical work as necessary in the repair of impact damaged girders.

Extending the cementitious patch material with coarse aggregate will minimize drying shrinkage and dimensional changes during curing, and will help in controlling thermal properties. To improve its compressive strength, lower material permeability, and reduce material shrinkage, the patch material can be batched with a low water-to-cement ratio. In general, the manufacturer's recommendations for curing should be followed. However, consideration should be given to the internal stress at the interface of the base concrete and the patch material with respect to shrinkage and early-age temperature variations. Typically, the faster the patch sets, the greater the shrinkage after the patch cures, causing higher internal stress at the interface of the base concrete and the patch material, and potentially causing the patch to crack or lose bond from the base concrete.

Hand application of the cementations patching material is often adequate for minor repairs.

Shot Crete, or pneumatically-placed concrete, has the advantage of speed of repair. No forms or form removal is required. Shot Crete is used where traffic under the bridge is heavy. Minor damage is typically shallow, and the spalled areas may not be large enough to warrant shot-creating. Dry packing has the disadvantage that spalled areas may lack sufficient confinement of the space to be dry-packed to permit this process to be used effectively

4.2.2 Moderate Damage Repair

Moderate damage is characterized in this study as large concrete cracks, as it was said. Although moderate damage is still considered non-structural, ensuring long-term durability becomes a significant factor to be considered in the repair process since the tendons are more likely to corrode if the girder is not repaired. Figure 2 outlines a step-by-step procedure for the repair of moderate concrete damage.

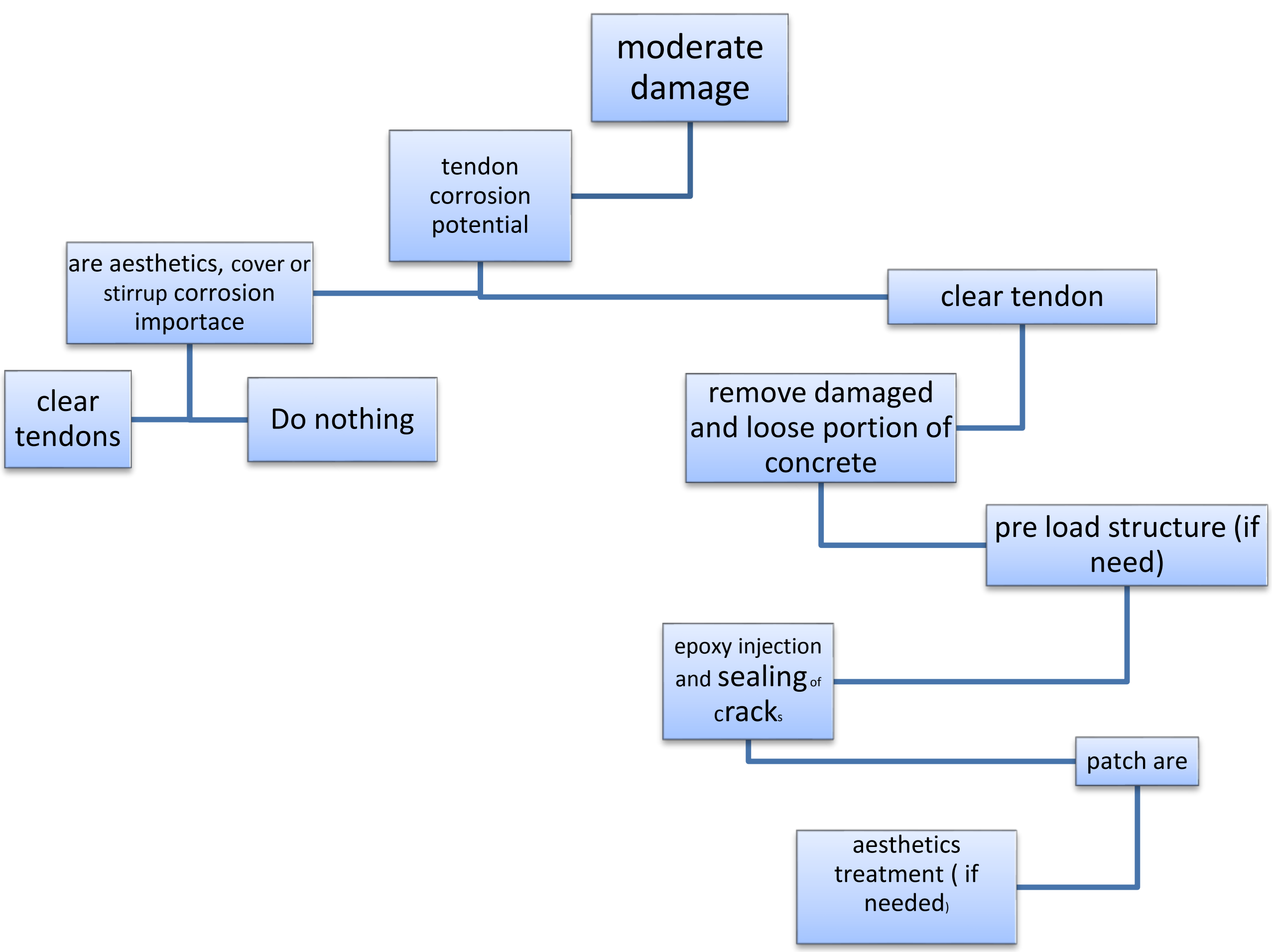


Figure 4-2moderate damage repair

Maintenance procedure for moderate damage

1. Removal of Loose and Damaged Concrete

Loose and damaged concrete should be removed, the same way it should be done for minor damage. However, in areas of exposed reinforcing bars or tendons, it is recommended that concrete be removed to a depth of ¾ in. (20 mm) beyond the back face of any exposed steel.

Remove all feather edges by shallow saw cutting perpendicular to surface. Since prestressing strands will be encountered during the concrete removal process, care should be taken to avoid damaging the strands. Hydro demolition may be considered.

2. Cleaning Exposed Tendons and Stirrups.

A moderately damaged girder will have exposed prestressing tendons, and will likely have exposed stirrups as well. The exposed steel is likely to have been exposed to the environment since the time of the damage.

Loose corrosion products on the prestressing tendons and stirrups should be removed. Damage to girders which results in exposed steel should be made as quickly as possible, particularly if the damaged structure is located in an environmentally harsh area.

In addition, concrete should be removed to a depth of ¾ in. (20 mm) on all sides of the bars.

Removal of concrete around the bar or tendon allows the repair material to surround the bar, providing a more desirable and better anchorage of the patch material to the base concrete.

Stirrups and prestressing strands should be cleaned with high pressure water and power wire brushes. Power wire brushes, however, are not efficient for cleaning the back side of exposed steel. If the concrete surface preparation is done by hydro demolition, the exposed reinforcing bars and prestressing tendons will be cleaned as the concrete is removed. Needle scalars or needle guns should not be used on strands because they may nick and break wires. Likewise, sand or abrasive blasting should not be used

3. Preloading the Structure

The loss of concrete, associated with impact damage, may cause the girder to camber upward.

Applying dead load to the damaged girder prior to repair referred to as preloading the structure may help the concrete patch remain in compression under dead loads and provides some allowance for shrinkage of patch material.

4. Epoxy Injection and Sealing of Cracks

Cracks in damaged girders will reduce long-term durability by allowing contaminants such as seawater and deicing chemicals to enter the damaged member. As a result of these cracks, the corrosion protection of the stirrups and prestressing tendons provided by the concrete is therefore reduced. It has been recommended that cracks wider than 0,008 in (0,2 mm) should be epoxy injected to restore girder durability, while finer cracks should be sprayed or brushed with saline seal to prevent the entry of moisture and deicing salts

5. Patching the Damaged Area

Spalls should be patched as described for minor damage once surface preparation has been executed, noting, however, that moderately sized spalls may have to be formed to restore the original girder dimensions.

6. Aesthetic Treatment/Painting/Coloring of the Girder Surface

If aesthetics of the repaired girder are important, and the patched area is of a different color than the existing concrete, the girder may be painted to obtain a uniform girder color or neat cement grout (cement and water) may be rubbed on the surface of patch materials to adjust the color.

4.2.3 Severe Damage Repair

Severe damage is any damage requiring structural repairs. Typical damage at this level includes significant cracking that leads to exposed and broken strands. Such types of defects required advanced maintenance techniques. External Post-tensioning and CFRP retrofit have been widely practicing in the developed country. Such as in American, Australia, and Canada are some of these countries. Even in Ethiopia Post-tensioning were practiced for strengthen of the **Gibe river bridge in2002 and the Omo river bridge in 2015 are the best evidence.**

4.2.3.1 External Post Tensioning

Post tensioning can be used to help restore pre-stress as well as girder strength.

4.2.3.2 CFRP repair

Carbon fiber reinforced polymer (CFRP) strips bonded to prestressed concrete girders can increase flexural capacity of the girder.

Design of external fiber-reinforced polymer strengthening system

The FRP reinforcement systems will be designed to either correct strength deficiencies that have result from cracking, or prevent strength deficiencies from occurring.

FRP could have the following objective. [24]

- Potential strengthening systems.
- To provide adequate positive bending resistance, as well as adequate shear resistance, regardless of continuity conditions.
- The other objective is to shift future cracking, associated with the restrained deformations of time- and temperature-dependent effects, to a more acceptable location at the face of, or within, the continuity diaphragm.

Note: Based on the maintenance types, The FRP reinforcement shall be selection from manufacturer catalog.

Installation of external FRP reinforcement

The following step should be followed to install EXTERNAL FRP**:**

- Surface preparation,
- Adhesion testing of FRP, for selected FRP type, epoxy on concrete surface,
- Preparation of FRP-composite samples for tensile testing
- Procedures of FRP reinforcement installation process, and
- Painting of the FRP reinforcement that concluded installation.

To conduct adhesion test, we could use epoxy, emacho, or other binding material that can fulfill the specification.

4.3 REPAIR TECHNIQUE FOR CONCRETE SPALLING

Spalling damage can be classified into one of three categories:

Minor damage is defined as concrete with shallow spalls and nicks. Damage at this level does not affect member capacity. Repairs are for aesthetic or preventative purposes.

Moderate damage includes sufficient spalling or loss of concrete to expose strands. Moderate damage does not affect member capacity. Repairs are intended to prevent further deterioration.

Severe damage is any damage requiring structural repairs. Typical damage at this level includes significant spalling that leads to exposed and broken strands.

4.3.1 Minor Damage Repair

Removal of Loose and Damaged Concrete and Patching the Damaged Area are treated as the same as minor crack repair.

4.3.2 Moderate Damage Repair

Removal of Loose and Damaged Concrete, Cleaning Exposed Tendons and Stirrups, Preloading the Structure, Epoxy Injection and Sealing of Cracks, Patching the Damaged Area and Aesthetic Treatment/Painting/Coloring of the Girder Surface are treated as the same as moderate prestressed concrete crack repair.

4.3.3 Sever Damage Repair

Sever damage repair basically spalling damages are structural and it propagates and leads to tendon damage. In prestressed concrete, such damage could be maintained by Fiber Reinforced polymer. Patching and preloading should be carried out. FRP repair procedures for sever spalling damages are similar to sever crack maintenance technique.

4.4 Repair technique for concrete corrosion of tendon

Minor damages have not the direct influence on corrosion of tendons. However, moderate damage becomes the cause for corrosion of tendons. For this reason it needs urgent response.

4.4.1 Severe Damage Repair

According to NCHRP *Report 226*, eleven different repair methods were developed for the severe damage condition and are discussed in detail. Each repair technique was evaluated to provide an overview of the processes and advantages and limitations of the method. Guidelines were proposed based on service load capacity, ultimate load capacity, overload capacity, fatigue life, durability, cost, user inconvenience and speed of repairs, aesthetics and range of applicability.

Evaluation of the repair techniques based on these parameters was conducted using a value engineering process.

Repair methods considered in *Report 226* were external post-tensioning, metal sleeve splicing (or steel jacketing'), strand splicing, a combination of these methods, and replacement.

4.4.1.1 External post-tensioning

External post-tensioning is affected using steel rods, strands or bars anchored by corbels or bolsters which are cast or mounted onto the girder. The steel rods, strands or bars are then tensioned by jacking against the bolster. Due to the externally applied tensioning, preload may not be required. The damaged concrete may be repaired utilizing appropriate patching methods. Protection of the posttensioning system must be considered, [25].

Design of external post-tensioned repair systems is relatively straight forward using a simple plane sections analysis (recognizing that the post-tensioning bar is unbounded). The attachment/ interface of the bolsters, however, require significant attention. These elements are 'disturbed regions' subject to large concentrated compression forces. Additionally, sufficient shear capacity along the interface between the bolster and existing beam must be provided to transfer the post-tensioning force. Effective shear transfer often requires the bolsters themselves to be post-tensioned to the girder to affect adequate 'friction' forces along the interface. Finally, the design of the bolsters and interface must consider the moments induced by the eccentric posttensioning forces.

4.4.1.2 Steel jacketing

Steel jacketing is the use of steel plates to encase the girder to restore girder strength. With this repair technique, post-tensioning force can only be introduced by preloading. Repairs can be accomplished with metal splice sleeves. The damaged concrete areas within the splice area are filled with concrete. Preloading is not required if the stresses at the top and the end of the sleeve are within the allowable, [25]. Generally, this method of repair will also require shear heads, studs or through bars to affect shear transfer between the steel jacket and substrate beam. Steel jacketing is felt to be a very cumbersome/bulky technique. In most applications, field welds will be necessary to 'close' the jacket (since the jacket cannot be 'slipped over' end of beam in most applications). Additionally, the jacket will need to be grouted in order to make up for dimensional discrepancies along the beam length.

4.4.1.3 Strand splices

Strand splices are designed to reconnect severed strands. Methods of reintroducing Prestress force into the spliced strand are preloading, strand heating and torqueing the splice; the latter is most common, essentially making the splice a turnbuckle of sorts. Strand heating is a method whereby the strand is heated, the strand splice is secured to the strand and as the strand is allowed to cool, it shrinks, thus introducing tension back into the strand. Strand heating of conventional high-strength prestressing strand is not believed to be a terribly rational method of affecting any reasonable pre-strain: either a) a long length of strand must be heated; or b) a short length of strand must be heated to a high temperature. The former is impractical in a bridge girder and the latter will affect the material properties of the strand. Strand heating is not recommended. Repair of more than 2-4 tendons by Strand splices method is usually difficult. After tendons are repaired the concrete is repaired, usually under preload, [25].

Note: External post-tensioning and metal sleeve splices/steel jacketing may be used to increase the strength of members.

4.4.1.4 Combination of repair techniques

Repair techniques may be combined. Combination of repair techniques will allow the user to employ the advantages of each repair. The post-tensioning addresses girder serviceability while the steel jacket reinforces the girder's ultimate capacity. Most repairs proposed in *Report 226* make use of preloading during girder repair. Preload is the temporary application of a vertical load to the girder during the repair. The preload is provided by either vertical jacking or a loaded vehicle. If the damage has caused a loss of concrete without severing strands, preloading during concrete restoration can restore the strength of the girder without adding prestress. Because preloading may be used to restore partial or full Prestress to the repaired area, it effectively reduces tension in the repaired area during live load applications. It is for this reason that preloading is suggested for most repairs, particularly those including patching. Care should be taken when preloading a structure so as to not overload the structure or cause damage from excessive localized stresses from the preloading force, [2, and 25].

4.4.1.5 Member is damaged beyond reasonable repair

Replacement of some severely damaged members may be the only solution, [25].

In general**: -**the type of repair must be determined by the extent and type of damage, the time that the structure will be out of service, the repair cost, durability, and the ultimate load capacity of the repair. Combinations of repairs such as internal splicing with external post-tensioning could also be considered.

4.5 Repair in Rail &Ballast failure on the deck, Sleeper displacement and Loosening of fastenings

4.5.1 Rail Repair Technique

As any damages rail damages can be classified as minor, moderate and severe. Minor and moderate damage can maintain by the use of grinding technique. However, if the rail has severe damage replacing the defective part shall recommend.

General consideration should be given when designating the length of the replacement rail to the size, defect type and rail characteristics. Replacement rail is to be certified free of defects. Particular care should be taken to prevent adding additional rail during the replacement process.

Before proceeding with repairs, thoroughly inspect the CWR and track conditions for a sufficient distance to determine general track rail condition, rail anchor performance, ballast condition, track alignment, rail tensions or compression, etc. Any condition found warranting correction should be corrected at that time or the necessary safeguards taken to provide for the safe movement of trains until it is corrected, [25].

4.5.1.1Repair by cutting in a short section of rail and the application of standard joint bars

a) Determine if a CWR temperature adjustment is necessary by consulting rail laying temperature records and other track condition data that may be available as a result of past track inspections or experiences.

b) If necessary, proceed with the adjustment in accordance with standard practice.

c) Promptly secure the CWR ends to prevent further movement. It is recommended that additional rail anchors be applied to the CWR ends for a sufficient distance to protect against rail-end movement in either direction.

d) Saw cut the CWR, or flame cut if approved, on each side of the defect far enough to ensure complete removal and obtain an opening for a short section of rail. It is recommended that the short rail be one-half the standard rail length to 36 (nearly 12m) feet long or at least 3 feet (nearly 1m) shorter than the standard length. If flame cut, the ends shall be saw cut to remove the torch cut end for a distance of at least two inches.

e) Cut a rail to the desired length.
f) Bevel all cut rail ends to the rail owner's requirements.
g) Promptly place the short rail into the opening and secure it in place.
h) Drill bolt holes of standard size. It is recommended that a template be used to inscribe the bolt-hole locations. Drilling through the joint bar holes is not recommended.
i) Dress the edges of the bolt holes in accordance with standard practice.
j) Install standard joint bars fully bolted.
k) Adjust the rail anchor pattern to conform to standard practice.
l) If in track circuit territory, install any necessary bond or connection wires.
m) In a stretch of new rail, if the rail surface has not been sufficiently work hardened, it is recommended that all cut rail ends be hardened at this time.

4.5.1.2 Repair by cutting in a short section of rail and thermite weld the rail ends

a. Proceed as outlined in paragraph a (1) through (5) above, except it is recommended that the short rail be at least 10 feet (nearly 3.3m) long or longer, preferably one half the standard rail length.
b. Promptly place the short rail into the opening, secure it in place.
c. Line up the rail ends to match, and block or wedge rail ends on each side of the joint sufficiently to maintain a good match and specified crown for thermite welding.
d. Proceed with thermite welding in accordance with standard practice.
e. In cutting the opening for the short rail, the rail ends (joints) should fall in the center of a tie crib and/or ties moved as necessary for the free unobstructed application of the thermite weld mold.
f. Adjust the rail anchor pattern to conform with standard practice.
g. If in track circuit territory, install any necessary connection wires.
h. If a CWR adjustment has been made or is not necessary, but conditions do not permit thermite welding at the time, then drill the rail ends for the temporary use of standard joint bars with the exception of the first bolt hole of the rail ends. Omitting these holes will permit thermite welding later without further rail change. Adjust the rail anchor pattern to conform with

standard practice for buffer rail. Follow with thermite welding as promptly as conditions permit.

4.5.1.3 Repair by cutting in a short section of rail and flash welding the rail ends

a. Repair plugs should only be installed when the current rail temperature is below the Adjusted Rail Temperature (ART) for the track under repair, or when it is desirable to raise the ART of the track under repair.

b. Determine if a CWR temperature adjustment is necessary by consulting rail laying temperature records and other track condition data that may be available as a result of past track inspections or experiences.

c. If necessary, proceed with the adjustment in accordance with standard practice.

d. Saw cut the CWR on each side of the defect to obtain an opening for a short section of rail. It is recommended that the short rail be a minimum of twenty feet (nearly 6.45m). In cutting the opening for the short section of rail, locate the cuts so that the rail ends and resulting welds fall in the center of a tie crib.

e. Remove rail anchors for a sufficient distance on each side of the opening to allow sufficient movement for installation of the replacement rail. If it is desired not to change the ART, then the opening shall be allowed to increase by an amount equal to the amount of rail consumed by two welds.

f. Cut the replacement rail to the desired length. If no adjustment to the ART is necessary, the replacement plug length should be the length of rail cut out of track PLUS the total rail consumption (two times the amount of rail consumed by one weld) of the flash butt weld process.

g. Remove the defect rail and promptly install the replacement rail, securing it in to place. Depending on conditions and work procedure, it may be necessary to bar one of the ends of the CWR out of the plates to facilitate installation of the plug.

h. Align and complete the first weld according to standard practice.

i. If the CWR was barred out of place previously, return it to the rail seats. Put the rail puller in place on the rail around the second weld location. DO NOT pull the rail gap closed until the first weld has cooled to below 700 degrees F or the release temperature designated by the railroad.

j. Pull the rail gap closed, align the rail ends and complete the second weld according to standard practice. DO NOT release the rail puller until the second weld has cooled to below 700 degrees F or the release temperature designated by the railroad.
k. Replace spikes, anchors and all track hardware.
l. If a CWR adjustment is necessary but conditions do not permit it at the time, it is recommended that flash welding be postponed. Cut in a short plug rail of approved length. Drill all rail ends for the application of fully bolted standard joint bars. Adjust the rail anchor pattern to conform with standard practice for buffer rails. Follow with CWR adjustment and flash welding as promptly as conditions permit.

4.5.2 Sleeper displacement repair

As well discussed in ‘causes and types of effect of sleeper’ to recognize main defects of concrete sleepers, life cycle defects of sleepers were primarily categorized as: Deteriorations during manufacture and coupling (track paneling), Deterioration during transportation and installation and Deteriorations during use. If the damaged sleeper loses its strength totally replacement of the individual sleeper is the best solution but the defect is nonstructural just aesthetic solution is enough. Nonstructural concrete sleeper damage can be maintained as minor and/or moderate spalling/crack repair.

4.5.3 Repair in Ballast failure

Ballast is a selected crushed and graded aggregate material which is placed upon the railroad roadbed for the purpose of providing drainage, stability, flexibility, uniform support for the rail and ties and distribution of the track loadings to the subgrade and facilitating maintenance. There are distinct differences in the mineral composition of the various aggregate materials used for roadway ballast applications and the respective in track performance of those materials. Likewise, many variations exist in the mineral properties of aggregate materials within the same general nomenclature of the aggregates known as granites, trap rocks, quartzite, dolomites, and lime stones. One particular aggregate material may possess most of the desirable characteristics for a good ballast material while a deposit of apparently similar material located in the same general geographical area will not meet the applicable specification requirements for railroad ballast.

Basically there are two main types of track system. The first one is ballasted track and the second is slab track. Both track system are constructed on bridges in the new Ethiopian railroad network.


Figure 4-3: Ballasted Track Bridge in Adama city, Ethiopia

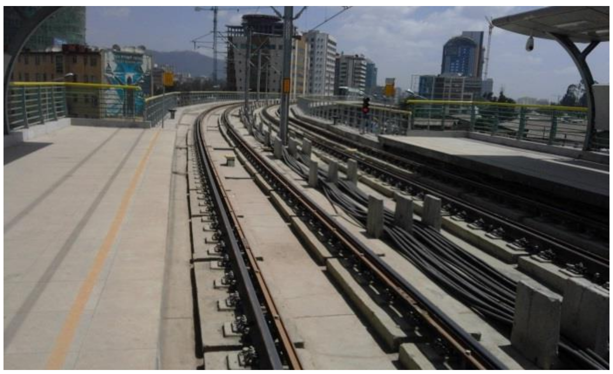



Figure 4-4: Slab track bridge Mexico Addis Ababa, Ethiopia

Thus, when selecting ballast materials it is necessary to define the type of material and the physical and chemical properties which can be measured in the laboratory by specific test methods. It is also most important to consider the field performance and behavioral characteristics of the ballast material. As discussed in third chapter ballast can be damaged/ failed by various ways. Therefore, this failure should be treated in a proper technique.

Objectives of ballast maintenance and renewal

The Principal objectives of ballast maintenance are restoring the drainage and remove the fines particles (including damaged ballast). In addition to this repair any damage to the formation and improve the overall system (for example adds blanketing or geo-grids) has conducted.

Maintenance techniques for defective Ballast

1. Ballast cleaning once done manually
2. Grading of wet beds

–Manual

–Mechanical

3. Shoulder cleaning
4. Weeds praying (residual)
5. Regulating & Brushing
6. Drainage maintenance

Ballast cleaning or re-ballasting once done Manual

This is a very simple technique using manually and good results can be achieved.

Figure 4-5 Manual renewal of ballast

Ballast Regulating

- Can be done manually but uneconomic for other than short lengths
- Largely ceased after privatization but now making a comeback
- Restores correct profile of ballast following settlement or after other work (e.g. tamping)
- Very important for stability of Continuous Welded Track

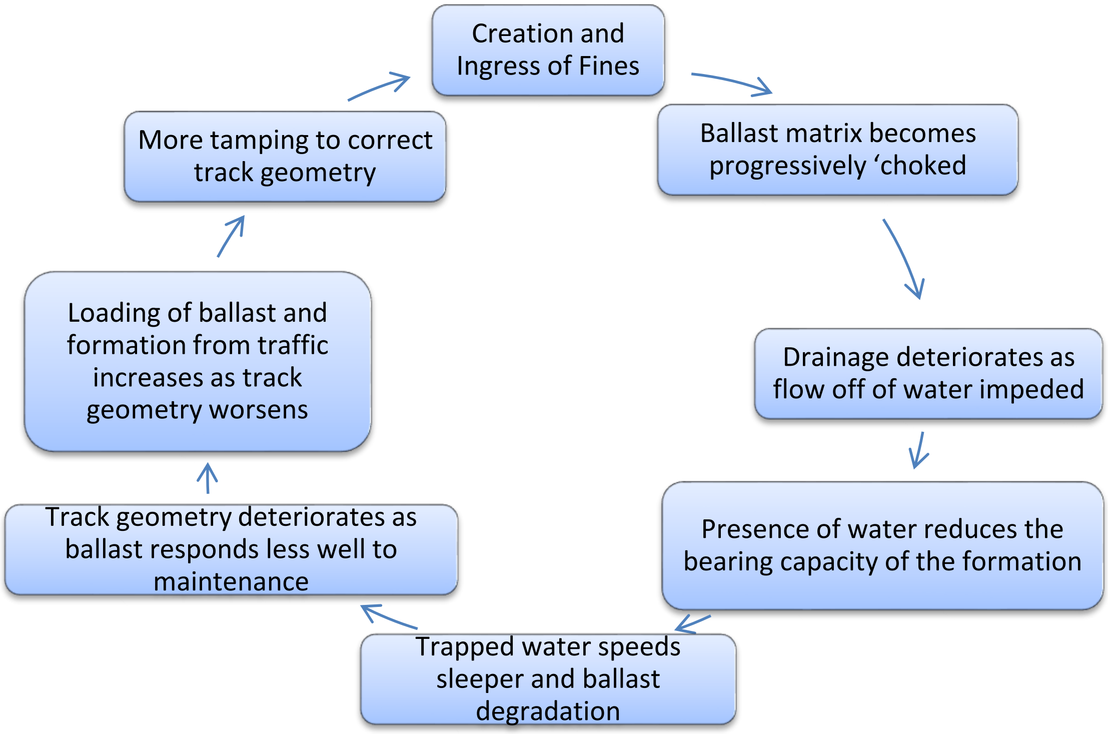


Figure 4-6: Ballast decline cycle

4.5.4Repair of loosen fasteners

There are various reasons to loosen the rail truck fasteners. The common causes are aging, improper handling, lack of maintenance and others. Frequent inspection and maintenance should be carried out. Within a specific period of time make strengthen the loosen fastener by the use of calibrated fastener strength machine. In addition to this, replace the damaged fastener turn by turn.

4.6. Repair in problems of Anchorages and bearings

Anchorage is the main structural component of prestressed concrete bridges. Huge prestressing strands force has anchored and applied here. In post-tensioned construction, the prestressing force is transferred to the concrete through relatively small anchorage plates behind the anchorage by bearing. This results in a very high concrete bearing stress behind the anchorage plate.

Failure of anchorage zone is perhaps the most common cause of problems arising during construction. Such failures are difficult and expensive to repair and might require replacement of the entire member.

Anchorage zones failure due to uncontrolled cracking or splitting of the concrete from insufficient transverse reinforcement.

Bearing failures immediately behind the anchorage plate are also common and may be caused by inadequate dimensions of bearing plates or poor quality of concrete

That is why a proper design, inspection and maintenance of anchorage is very important. Cracks, spalling and/or corrosion Anchorage damage might be occurred in this component. Unlike discussed in the other section if minor or moderate damages observed immediate action should be taken in order to avoid further propagation.

The maintenance shall be carried out by the use of Fiber reinforced polymer. The whole procedure was discussed in the above sever spalling/ crack maintenance section.

4.7. Repair technique for concrete Deformation of shape or Bulging

4.8. Surface erosion and surface deposits

4.8.1 Repair methods for Scouring

Riprap remains the most common countermeasure used to prevent scour at bridge abutments. A number of physical additions to the abutments of bridges can help prevent scour, such as the installation of gabions and stone pitching upstream from the foundation. The addition of sheet piles or interlocking prefabricated concrete blocks can also offer protection.

Trapezoidal-shaped channels through a bridge can significantly decrease *local scour* depths compared to vertical wall abutments, as they provide a smoother transition through a bridge opening. This eliminates abrupt corners that cause turbulent areas. Spur dikes, barbs, groins, and vanes are river training structures that change stream hydraulics to mitigate undesirable erosion or deposits. They are usually used on unstable stream channels to help redirect stream flow to more desirable locations through the bridge. The insertion of piles or deeper footings is also used to help strengthen bridges.

If a pier is found to be suffering from adverse scour conditions, the problem can be corrected by changing the structure (enlarging the footer, strengthening or adding piles, providing a sheet piling barrier around the pier foundation) or replacing the material which has been washed away (broken stone, concrete rip rap,..)[23]

Hydraulic protection techniques include slope protection, foot protection, river bed protection and river realignment depending on the extent and nature of the hydraulic problems encountered such as scour, erosion, and flood flow.

Slope protection is applicable to river banks adjacent to abutments where erosion is observed. **Foot protection** is applicable to footing of the slope protection in order to prevent slope failure caused by scouring action on the river bed.

River bed protection is applicable to river bed surrounding the river piers where local scouring or river bed lowering is observed.

River realignment work is applicable to extremely eroded banks of a meandering river located at the vicinity of a bridge upstream.

Selection of a specific protection technique depends on the stream type, river scale, flood flow velocity, foundation type and geology of each site.

4.9.1.1Gabion protection for scouring

The task involves preventing slope failure caused by scouring action on the river bed, for small scale river and foundation ground is soft.

Rock and Gabion Basket are materials that used for construction of Gabion protection, [23]..

Procedures

1. Traffic warning signs shall be established at both ends of the work section
2. All stumps, roots, debris and rocks shall be removed and disposed of as directed by the Consultant, prior to placing gabions. Excavation to accommodate gabion installation shall be carried out to the lines and levels as specified on the plans or as directed by the Consultant.
3. Gabion cages shall be assembled by tying the appropriate faces together along the self-edges with binding wire. The binding wire shall be tightly looped around every other mesh opening to form a spiral with single and double loops alternating.

Rocks shall be in filled in the assembled cage units either by machine or by hand. When machine is used, minor rock repositioning shall be done by hand to fill the voids between larger rocks and thereby achieve a dense structure. Rocks along visible faces shall be selected and placed by hand from among the larger sizes with a flat face toward the exterior to produce a likely of a masonry structure and a neat and workmanlike appearance, [23].

Backfilling shall be placed and compacted in layers not exceeding 200 mm thickness.

Backfilling shall be compacted to a density not less than 95% of the maximum dry density ratio determined for the in situ material using Standard Compaction. Material directly above gabion boxes shall be placed without compaction.

On completion of the works, the site shall be cleaned of all surplus materials and waste, and left in clean, tidy condition, [23].

4.9.1.2 Spur Dike by stone masonry

The task involves reducing erosion caused by water flowing along the upstream side of the embankment. Scour may still occur with spur dikes, but it will be moved upstream and away from the abutments. Spur dike is applicable for large-scale river.

This item shall consist of providing the necessary labor, materials and equipment to construct spur dike at the vicinity of a bridge upstream including clearing, excavation, driving wooden log piles, and construction the spur dike as specified on the drawing.

Procedures

1. Traffic warning signs shall be established at both ends of the work section
2. Areas on which spur dike is to be constructed shall be cleared, grubbed, and excavated as specified on the plans or as directed by the Consultant.
3. A box like compartment shall be prepared by driving wooden piles of diameter 150mm at 15cm to 30 cm center to center. The piles are secured by wooden bracings. The hollow spacing is filled up by boulders.
4. The dike shall be constructed in successive layers of earth about 45cm thick by mechanical compaction. The layers are thoroughly compacted by rollers of recommended weight and type. When compaction of one layer is fully achieved, the next layer should be laid and compacted. The designed spur dike section hence shall be completed layer by layer.
5. These dikes shall consist segments of dikes located riverward from the existing bank with gaps between the dikes 2 to 2-l/2 times the structure length. The length of the gaps between the dikes shall be about 50 to 60 percent of the length of each vane. The dikes should be placed at a slight angle to the direction of flow, about 10 to 15 degrees, with the downstream end of the dike farther riverward than the upstream end.

The side slopes (upstream and downstream faces) of spur dikes should be maintained on the natural angle of repose of the earth material used to construct the dikes.

6. Slopes shall be finished to a reasonably smooth and compact surface within a tolerance of 15cm from the surface lines shown on the plans. Immediately prior to placement of riprap bedding, the prepared base will be inspected. Riprap or bedding shall not be placed until the prepared base has been approved.

7. The finished slope shall be backfilled with gravel and sand which is spreaded uniformly. Compaction of the bedding material will not be required, but material shall be finished to a reasonably even surface, free from mounds or depressions.

8. Riprap shall be placed over the backfill material in a manner that will produce a reasonably well-graded mass of rock with the minimum practicable percentage of voids. Riprap shall be placed to its full course thickness in one operation and in a manner to avoid displacing underlying material.

Finished riprap shall be free from objectionable pockets of small stones and clusters of larger stones. Hand placing may be required to the extent necessary to secure the results specified and form uniform slopes.

Rock for grouted Riprap shall be thoroughly moistened and any excess of fines shall be sluiced to the underside of the stone blanket before grouting. Stones shall weigh between 50 and 150 pounds each. At least 60 percent shall weigh more than 100 pounds, and approximately 10 percent may weigh 50 pounds or less.

Care shall be taken during placing to keep earth or sand from filling spaces between stones. After stones are in place, spaces between them shall be filled with grout from bottom to top and the surface swept with a stiff broom.

9. Grout shall consist of 1 part hydraulic cement and 3 parts sand, thoroughly mixed with water to produce grout having a thick, creamy consistency.

Care shall be taken during placing to keep earth or sand from filling spaces between stones. After stones are in place, spaces between them shall be filled with grout from bottom to top and the surface swept with a stiff broom. Riprap shall not be grouted in freezing weather. In hot, dry weather, the work shall be protected from sunlight and kept moist for at least 3 days after grouting by the use of saturated burlap.

The grout may be delivered to the place of final deposit by any means that will insure uniformity and prevent segregation of the grout. If penetration of grout is not obtained by gravity flow into the interstices, the grout shall spaded or rodded to completely fill the voids in the stone blanket. Pressure grouting shall not unseat the stones; and during placing by this method, the grout shall be spaded or rodded into the voids.

Penetration of the grout shall be to the depth specified on the project plans. When a rough surface is specified, stone shall be brushed until from one-fourth to one-half of the depth of the maximum size stone is exposed. For a smooth surface, grout shall fill the interstices to within a 1.25cm of the surface.

On completion of the works, the site shall be cleaned of all surplus materials and waste, and left in clean, tidy condition

4.9.1.3 Dumping Stone

The task involves preventing slope failure caused by scouring action on the river bed, for small to Medium River and foundation ground is relatively solid. [23]

Materials

The rock material shall be sound and durable, angular in shape, resistant to weathering and water action.

Procedures

1. Traffic warning signs shall be established at both ends of the work section
2. Areas on which foot protection is to be constructed shall be cleared, grubbed, and excavated. A footing trench shall be provided along the toe of the slope and the trench shall be constructed as detailed in the plans. The river bed should also be cleared and leveled and backfill the river bed in layers using gravel, broken stone or quarry waste up to a min. depth of 0.5 to 1.5 m.
3. Place mass stone with minimum practicable percentage of voids and a minimum thickness of 24 mm. The scoured area to be filled with rock using stone of 10 – 30 kg weight, or heavier, and place the smaller size stone in the lower layer

4. Fill spaces between stones with smaller size stone

5. Continue work layer after layer until final level is reached. The top layer should contain the heaviest stones and have an even flat surface at river bed level

6. When it is not possible to place stones in regular layers due to water flow, the scour area can be filled by random filling of the scour depression. Drop stone blocks into scour depression either from the bridge or from the bank until the depression has been filled.

4.9.1.4 Placing of rip rap

This methodology is applicable in preventing foundation failure caused by scouring action on the river bed, and at the base of abutments and piers.

This item shall consist of providing the necessary labor, materials and equipment to repair foundation scouring including the preparation of the ground surface to receive gabions, the construction of the gabion structures in place, complete with rock filling at locations specified on the drawings, [23].

Procedures

Stakeout the area around the riverbed, the pier or abutment wall where scour has occurred

Excavate the scoured area to the lowest eroded level.

Place riprap stones in layer in the excavated and prepared scoured hole starting with the smaller size stone in lowest layer.

Fill space between stones with a cyclopean concrete (1cement: 4 sand: 8stone by volume).

Continue work layer after layer until normal bed level is reached. The top layer should contain the heaviest stones and have a flat even surface at river bed level.

4.10. Repair technique for concrete

Generally, other concrete structural crack and spalling can be treated as prestressed concrete repair techniques that discussed above.

For proper repair works there are at least four basic procedures to be followed; such as Removal of Concrete, Surface cleaning, Substrate Preparation, and Repair Material Application. [23]

Substrate Preparation

Unless directed otherwise, use a bonding agent if cement mortar or concrete is used for the repair material. Use either a cement scrub or epoxy as the bonding agent. Follow the manufacturer's recommendations for bonding agents if pre-packed repair material is used. Avoid using an epoxy bonding agent with rapid setting repair materials. Apply a cement scrub coat bonding agent to the saturated surface-dry substrate by scrubbing, brushing, or other methods approved by the Engineer immediately before placing the repair material. Apply an epoxy bonding agent in accordance with the manufacturer's recommendations. Ensure that any bonding agent used does not set or cure prematurely, creating a bond breaker, [23].

Repair Material Application

Place the repair material in an approved manner ensuring that the repair material is in intimate contact with the substrate and free of voids. Follow the manufacturer's recommendations for pre packed repair materials. Place the repair materials so that the original lines and surfaces of the structure can be restored, [23].

4.11. Repairs in Miscellaneous devices failure

4.11.1 Repair of Expansion Joint

ERA Bridge repair manual discusses varies expansion joint repair methods. Such as, temporary repairs, particularly where decks are 'fixed' to the substructures, but in the longer term replacement with more suitable joint arrangements is the only realistic option.

4.11.1.1 Temporary Repairs

Expansion joints which comprise carriageway surfacing over the joint gap at 'fixed ends' may be repaired by cutting a groove 20 mm wide and 25 to 30 mm deep across the carriageway, directly above the expansion gap, and filling it with rubberized bitumen. It may be necessary to remove any bulged surfacing above the joint first, using a milling machine, or local patching of the carriageway may be required. Before undertaking such a repair the bridge articulation should be verified to confirm the locations of 'fixed ends'. This type of repair is unlikely to be effective at free ends where significant movements are anticipated.

4.11.1.2 Partial Repairs

Joints comprising a rubber seal located in metal rails may be repaired by replacing the seal. However more extensive damage is likely to require at least partial replacement of the joint arrangement as a whole.

Rubber slab joints and steel comb joints are generally bolted down to the bridge deck and curtain wall. The replacement of missing fixings and tightening of loose fixings complete replacement.

4.11.1.3 Replacement of joints

The installation of replacement expansion joints requires special skills, equipment and materials.

4.11.1.4 Treatment of concrete surfaces beneath an expansion joint

When a replacement expansion joint is installed, the concrete surfaces within the expansion gap should be sprayed with bitumen emulsion prior to fixing the joint. This will provide some protection if the joint fails in the future. The failure of a joint to provide a watertight seal may go unnoticed for some time if inspections coincide with dry periods.

4.11.1.5 Replacement of Expansion Joints

If the expansion joint has severe damage, the joint shall be replaced, [23].

4.11.2 Repair techniques of Bearings

Generally, maintenance of bearings can be preventive or corrective. Preventive maintenance is carried out to prevent failure of the bearing. Preventive maintenance is commonly, [22].

- Cleaning
- Painting
- Lubricating
- Inspection
- Safety measures
- Sealing deck joints that allows leakage of rainwater, de-icing salts and dirt to the bearing areas
- Monitoring etc.

Corrective maintenance are characterized by problems that have prevented the functions (rotation, translational movements, transfer of forces etc.) of the bearing and should be carried out immediately to correct failures of the bearing and prevent failure of the bridge structure, [22]. Problems such as:

- Frozen bearing
- Failure in connections and anchoring components (rivets, bolts, welds) that prevents uplift and shear
- Massive corrosion of the bearings causing section loss
- Displacement of the bearing, visible misalignment and loss of components
- Torn out or clearly bulged rubber in elastomeric bearings
- Tilted bearing as a result of high load from the superstructure
- Damaged bearings from seismic activities etc.

Corrective maintenance is in form of replacement, repair, retrofitting, and refurbishment/ renewal.

Monitoring of bridge bearing is important as it provides information on the actual behavior of bearings during service. Monitoring of strains, deformations, corrosion, cracks, forces, dynamic response etc. in bridge bearings helps in predicting and preventing failures.

4.11.2.1 Repair and preservation of bridge bearings

After a detailed bearing inspection have been made, the decision to repair or replace bridge bearings can be made with respect to the report on the inspection which explains the condition of the bridge, bearing problems, condition of the bearing and the repair methods to be used. The main bearing repair and preservation methods are by, [22].

- Cleaning and painting
- Lubrication
- Replacement of worn out components
- Jacking and resetting
- Retrofitting
- Replacement of the entire bearing for fully degraded bearings

a) Cleaning and painting

Cleaning and painting is the most common repair and maintenance method for bridge bearings as virtually all kinds of bearing have steel components. Cleaning is done to remove all kinds of rust, dirt, mill, surface impurities etc. and prepare the surface of the steel for corrosion protection treatments. Cleaning methods are: [22]

- Painting with special rust removal paints
- Solvent cleaning by mineral spirits or turpentine
- Wire brushing
- Pickling with sulphuric acid, phosphoric acid or iron phosphate
- Flame cleaning with oxyacetylene flame
- Sand blasting or steel grit blasting
- Water jetting

Special paints are applied on the steel parts to protect it from rust and corrosion, common paints used are,

- Zinc metal paint applied by hot dip galvanizing or thermal spray galvanizing
- High performance paint coatings such as alkyd, vinyl, phenolic, iron ore paints etc.

The cleaning, primer quality, workmanship, quality of the paint and the application method determines the reliability of the corrosion protection hence good quality control should be ensured during painting. [22]

b) Retrofitting

Retrofitting is the process employed in improving the functions, load capacity, seismic capabilities, movement capabilities, general conditions of the bridge bearing etc. Prior to retrofitting a proper inspection and investigation of the bridge bearings and bridge structure itself should be carried out in order to ascertain the causes of problem in the damaged or underperforming bearing, this will help in determining the right retrofitting measures to be employed. Retrofitting the bearings could mean retrofitting the bridge structure itself and vice versa. Retrofitting can be done by the following, [22]

- use of restraining bars
- use of dampers
- use of shock transmission units

- use of shear keys and restraining brackets
- Installing additional bearings, isolation bearings, lead rubber bearing, friction pendulum bearing etc.

c) Replacement

Bearings that are damaged beyond repair or that can no longer carry out its functions should be replaced. Prior to replacement, a detailed inspection and investigation should have been done. Replacement is done by using hydraulic jacks (standard, flat or computer aided) in jacking up the bridge deck, supporting the bridge deck on temporary bearing, taking out the damaged bearing and replacing it, [22].

Where there is sufficient space around the bearing seat area or top of the pier, an auxiliary beam is erected across the top of the pier to support the temporary bearing, jacking device and to give sufficient work area. In the absence of sufficient space on top of the bearing seat area/ pier top, an auxiliary pier can be mounted from the pile cap, ground or by attachment to the bridge pier to support the temporary beam, jacking device and temporary bearing to effect the replacement. It is a risky process that requires skill, innovation and technique in handling effective load transfer, stability, bearing pressures, deck balancing etc. during the replacement hence only experienced and certified contractors should be contracted to carry out bearing replacement. [22]

4.12 Others

During construction and design phase problems may occur. These problems shall be properly handled. For instance, during construction improper installation of tendons could happen. And this can totally change the behavior of the structure. So to avoid such errors a skilled manpower must followed every construction and maintenance procedure of prestressing system.

In addition, since most railway bridges in cities are Overpass Bridge collusion damages may occur. Therefore, proper handlings of these damages are highly advisable.

5 What is the Ethiopian Practice and Future Works

5.1 Damage identification in Ethiopian practice

In this section damage identification in Ethiopian practice in the past is assessed. Ethiopian Roads Authority practice of damage identification is studied to see possible application to railway bridges. Damage identification by inspection is the initial and major step of bridge management system of ERA. Ethiopian Roads Authority has a department called “Bridge Management Team” which is responsible for performing all activities regarding bridge management and maintenance.

Since 2002, after the in-house developed Bridge Management System (BMS) software, ERA radically transformed in bridge asset management from almost absence of organized bridge asset information to the systematic bridge asset management practice, which is born through challenging efforts truly paid to introduce concept of bridge asset management and develop & implement the computer aided BMS. This task was technically and financially was assisted by JICA.

After having developed BMS software by individually initiated ERA Bridge Engineers the bridge Inspection manual was prepared by JICA experts. The ERA BMS/CMS software contains wide range of information concerning geographical, physical, historical and service condition of bridge and culvert structures.

Bridge damage has been identified of all bridges in Ethiopia in accordance with ERA Bridge Inspection Manual. The inspection technique used is visual inspection and using local tools. However, advanced Inspection techniques such as, Non-destructive testing and destructive load testing are not used for Ethiopian concrete Bridges.

All the collected bridge data at the site, by using standard formats for the purpose, entered in to the BMS software where storage could be made and data analysis processed. The software, after analyzing the data, produces multipurpose reports such as list of prioritized bridges for improvement; estimation of the required budget for the improvement implementation; historical data with pictures, drawings, videos; many kind of information that might be asked by professionals or stake holders.

The branch has developed manual for bridge inspection which helps to inspect, record results and repair based on the result found using software called “ERA-BMS”. In the following paragraphs Inspection trend, classification, rating method and other related concepts are described according to bridge inspection manual of Ethiopian roads Authority.

Bridge inspection is an action to assess correctly the bridge conditions in a standardized manner and is the most important element in the Bridge Management Cycle that is the systematic maintenance process of inspection, assessment, selection of measures, prioritization and repair to keep bridges in satisfactory levels. The purpose of bridge inspection is not simply to collect the latest information about the present bridge conditions but, more importantly, it should be understood that the inspection is to provide essential information for Bridge Management Cycle. The road administration sometimes must take the critical decisions such as closure and reconstruction of the bridge based on the inspection results, [23 and 24].

Bridges cannot be kept in good conditions forever. Bridges start deteriorating soon after their completion because of the actions of weather, traffic and other various causes. If the bridge inspections are not carried out at all, lots of bridges would collapse frequently across the county and would invite social and political troubles without doubt. One bridge collapse can cause casualties and require considerable amount of money and time to reconstruct the new bridge. It is important to understand that a bridge consists of lots of members and elements that have different durability in nature due to the difference of materials, locations, and sensitivities against weather and loads. This indicates that in order to keep the appropriate functions of bridges expected in design, appropriate maintenance is essential in response to the actual deteriorations. Service life of a bridge is usually expected as long as fifty years and longer, [23 and 24].

To do bridge inspection in the standardized manner can provide:

- Basic information to assess the safety of bridges
- Data and information to all the activities of Bridge Management Cycle
- Information on any potential trouble spots
- Information on a consistent maintenance strategy
- Information on the effect of any changes in traffic loads
- Information on the behavior of repair and new strengthening techniques
- Hard facts on the results of new constructions and measure

5.1.1 CLASSIFICATION OF INSPECTION

According to bridge inspection manual, there are three types of inspection in terms of purposes and frequencies, [23 and 24].

1. Regular Inspection

The regular inspection is a planned, periodic, and superficial/shallow inspection to confirm the structural safety and safe traffic condition as frequently as possible. It is also expected that the regular inspections can detect the major/serious defects and damages as soon as possible. The inspections are to be carried out by not only bridge inspection staff but also road inspection personnel.

Inspectors must develop the annual plan for the regular inspection work to cover all the bridges in the District to meet the frequency requirement of once a year.

2. Major Inspection

The major inspection is a planned, periodic inspection to be conducted once in every three years by close visual inspection method. The results of the major inspections are core information of the Bridge Management System and bridge inspectors of the District must conduct the major inspections. Inspectors must develop the annual plan for the major inspection work to cover one third of the bridges in the District to meet the frequency requirement of once three years.

3. Emergency Inspection

The emergency inspection shall be conducted when needed. After natural disasters and severe traffic accidents the emergency inspection may be needed. The purpose of this inspection is to provide information on structural safety and safe traffic condition. If needed, bridge inspectors must do the emergency inspection without delay so as to judge necessity of emergency measures.

The classification is summarized as follows.

Table 5-1 *Bridge Inspection classification according to ERA*

Classification	Type	Purpose	Frequency	Method	Inspector
Regular Inspection	Periodic Superficial	Assessing traffic safety and structural safety. Finding major defects.	Once a year	Visual from ground level. Report on check list	Road inspector, or Bridge Inspector
Major Inspection	Periodic	Assessing	Once in 3	Close visual	Bridge

	Detail	conditions of all the structural components.	years	with equipment. Detailed report with damage ratings	inspector or Bridge Engineer
Emergency Inspection	Non-periodic	Assessing traffic Safety and structural safety	When Needed	Visual	Bridge inspector Or Bridge Engineer

5.1.2 PROCEDURE OF INSPECTION

The standard inspection procedure is as shown below. However, sometimes inspectors may follow the different procedures due to actual traffic conditions and seasonal conditions. Before going out for inspection, the inspector shall check the availability of past inspection records using ERA-BMS and obtain the necessary information on the target bridges. Inspectors shall bring the necessary equipment to manage the appropriate inspections, [23 and 24].

1) Arrive at the Target Bridge and Park at the safe space with necessary traffic safety measures
2) Check the location data, name of the bridge and GPS indication
3) Examine the entire carriageway condition from the abutment side(s)
4) Examine the approach road section
5) Examine the expansion joint
6) Examine the pavement
7) Examine the parapet and railing
8) Examine the drainage inlets
9) Examine the surrounding land condition
10) Move down to underneath the bridge (channel, opening)
11) Examine the abutments
12) Examine the bearing(s) at the abutment(s)
13) Examine the piers
14) Examine the bearing(s) at the piers
15) Examine the high water level
16) Examine the river condition (river bank, sedimentation)
17) Examine the girder(s)

18) Examine the deck
19) Take the pictures of defects/damages
20) Check all the inspection records are correctly filled in the standard form then Leave the bridge

5.1.3 COMMON DEFECTS AND DAMGES TO THE ETHIOPIAN CONCRETE BRIDGES

ERA bridge management branch has developed a manual which shows types of defects and most prone components of the bridge to which the defect could appear based on past experience. The list could help the inspector and the bridge Engineer to suspect most prone components for Ethiopian concrete bridges. This could be important to the railway concrete bridges too. The table below shows the typical defects/damages frequently observed in reinforced girder concrete girder bridges in Ethiopia, [23].

Table 5-2*Common defects to the Ethiopian concrete bridges*

Component	Defect/Damages	Principal Causes
Girder	Flexural Crack, Shear Crack, Spilling, Delamination Re-bar Exposure Material Deterioration Honey come Water leakage	Excessive load, Excessive load, poor design Insufficient cover, Excessive load Insufficient cover, Re-bar corrosion, Concrete deterioration Poor construction Poor construction Excessive load, poor design
Deck Slab	Flexural Crack Spilling, De lamination, Re-bar Exposure, Material Deterioration Honeycomb Water Leakage	Excessive load, poor design, Insufficient Cover, Excessive Load, Insufficient Cover, Re-bar Corrosion, Concrete Deterioration Poor Construction Poor Construction

		Excessive Load, Poor Design
Bearing	Break Failure Dislocation	Poor design, poor construction, Abrasion Poor design, poor construction
Expansion Joint	Abrasion Distortion	Aging Aging
Pier, Abutment	Crack Spilling, De lamination Re-bar Exposure, Material Deterioration settlement, Inclination , Scouring	Poor construction, Insufficient cover, Insufficient cover, Re-bar corrosion, concrete deterioration, Poor construction, Poor design, poor construction, Earth pressure
Parapet,	Deformation	Vehicle collision
Guard rail	Break Failure	Vehicle collision
Clearance	Sedimentation	Poor planning, Lack of maintenance

5.1.4 RATING METHOD

Every defect and damage found shall be rated based on the following rating system. Because the rating system reflects the specific conditions and desirable interventions taken into account the present situations in Ethiopia, inspectors shall strictly follow the rating (ranking) description table, [24].

Table 5-3 ERA Bridge damage rating system

Item No.	Percent damage	condition

1	0-10%	Good
2	10%-15%	Fair
3	>15%	Bad

5.2 Materials and tools for repair of concrete bridges in Ethiopia

5.2.1Materials used for repair of concrete bridges

Materials for repairing cracks, voids, peel off and delamination on concrete, [23]

- **Mortar and concrete:** - the material needed for maintenance by dry-packing is a slurry bond and a low water-content mortar. The cement slurry bond coat consists of equal parts of cement and fine sand and the mortar consists of one part cement and three parts of sand passing a No. 16 sieve and only enough water so that the mortar will stick together when molded into a ball by slight pressure of hands and will not exude the water but will leave the hands dry. If the patch must match the color of surrounding concrete, a blend of Portland cement and white cement can be used. About one-third white cement is adequate for blending. Dry pack mortar shall consist of type I or II Portland cement, clean sand that will pass a 1.18mm (No. 16) sieve, and clean water.[23]
- **Cement mortar: -** used for repair of cracks and voids by shot crating, patching and caulking. All ingredients of mortar, the epoxy resin, cement based polymer and concrete materials shall fulfill the Standard Technical Bridge Repair Manual.
- **Epoxy concrete: -** material used for patching used for repair of cracking and resurfacing delaminated or/and Honey combed areas concrete bridge.
- **Flexible epoxy filler:** such as Epoxy resin, polyurethane resin, Acrylic and others; used for effective repair of nonstructural cracks observed on RC or plain concrete components of bridges. The application is done with injection.
- **Rigid epoxy filler:** Epoxy injection fluid shall confirm the following requirements. The resins for crack injection system shall have a two part: solvent free low viscosity, polymer and; the flexible, low viscosity, polymer, crack injection resin system when mixed in the proportions specified by a supplier, supplied and injected in to cracks in concrete, the resin shall form a

slightly flexible and impermeable barrier in both dry and damp condition, and thus, shall form permanent seal in cracked seal.

- **Cement grout:** Mixture of neat Portland cement and water, Portland cement. Other additives and admixtures (siliceous residue, diatomite) may be added on approval of the Engineer. The proportions of ordinary Portland cement to sand will depend upon the size of the spaces to be filled and will vary from a neat grout to about mix containing one part of cement to two parts of sand. The sand used in 1:2 ratios can be successfully pumped if all sand passes the No. 16 sieve and 15 percent or more passes the No. 100 sieve. Where necessary and approved by the Engineer, admixtures to Portland cement grout mixtures may be added for delaying the setting time, increasing flow ability minimizing segregation and shrinkage. Materials shall conform to AASHTO M6-81, M85-01 and M154 for fine aggregate, Portland cement and Air-entraining admixtures respectively. Water shall be fresh, clean and free from sewage, oil, acid, alkali, salts or organic matter.
- **Elastomeric sealer and fast setting mortar: -** materials used for repair of crack by caulking**.**
- **Latex modified concrete and highly dense concrete: -** are materials used for repair of cracks of concrete bridges by grinding and overlay.
- **Steel plate, Post-tensioning steel and steel pins: -** used for crack repair by methods of stitching and external pre-stressing.
- **Reinforcement**: - high yield strength reinforcement bar reinforcing steel should rely on specification of AASHTO M31. Reinforcement can be used for repair methods of stitching and external pre-stressing.

In addition materials used for replacing concrete surface, a method used for repairing Delaminating/ Honeycomb, are Portland Cement Concrete, Low-Slump, High Dense Concrete, High Alima cement Concrete, Epoxy mortar cement and concrete, Polymer concrete and Latex-modified concrete.

TOOLS USED FOR CONCRETE BRIDGE REPAIR

The following are tools and equipment used for repairing concrete defects and damages in Ethiopia. Most of the tools are used for cleaning, calibration and application of repair. Advanced inspection equipment are not included since it is not applicable in Ethiopian Highway Bridges, [23]. Inspection tools are listed below;

- Tools used for **measurement** are Tape measure, pocket measure, caliper, level, staff, rope, straight line and crack gauge.
- Tools used for **inspection** are Hammer, pen knife, Scraper.
- Tools used for **Visual aid** are Binocular, magnifying glass, handy torch, goggles and ladder.
- Tools used for **documentation** are digital camera, clip board, pen and pencil, chalk, paint, information board, area map and As-built drawings.
- Tools used for **Cleaning**: - Shovel, stiff hand and brush.
- Tools used for **Safety**: - Range pole, hard hat, safety traffic cones

5.3 Ethiopian bridge strengthening practice on GIBE RIVER BRIDGE

Damages and cause

The major damage of the bridge is clearly the shear cracks in the deck beam at both sides of pier2. This damage has been caused by the bomb blast which occurred in the 1987.
Subsequently the bridge has been repaired by casting in situ the missing deck segment after jacking of the two adjacent parts as can be still visible from the different finishing of the concrete surface, [26].

Strengthening and repair

As said above, the bridge needs to be strengthened to increase its load bearing capacity and prevent further deterioration of its present capacity which otherwise is set to rapidly decrease. This operation cannot be delayed if the bridge needs to provide crossing for heavy loading.

Different approaches exist to increase the load bearing capacity of these types of bridges although they tend to fall in either one of the following two, [26].

- Increasing the load carrying capacity by strengthening,
- Increasing the overall load bearing capacity by putting a new resisting element in parallel

For the bridge under consideration, the first approach can be pursued as the piers and foundation are in a relatively good shape and the deck has ample strength reserves if properly repaired. This is confirmed by the fact that the bridge has been used since the repair works carried out after the bombing, [26].

Extraordinary maintenance is also required for bearings and expansion joints at the abutments and water drainage along the deck.

Strengthening by External Unbounded Tendons

Strengthening the deck means first and foremost to reconstitute the integrity of the continuous slab and beam deck now jeopardized because of the above said cracks.

To reconstitute the deck integrity, injection of the cracks with epoxy resin was not sufficient.

Post-tensioning the deck is certainly the best option, after having injected the creak,

In the specific case under consideration, prestressing can be very efficient because of the deck and beam configuration and the diaphragm arrangements.

The numeric analyses detailed in the companion structural report and the deck configurations have suggested the adoption of 6 cables of 12 strands each.

The total prestressing force shall be of 1000 ton giving an average stress on the deck concrete X-section of 1.44 MPa at piers and 2.16 MPa at mid-span, [26].

The 6 cable configurations is found by placing one cable at each side of the beams except for the outer side of the outer beams where no cable are place far aesthetic and durability concerns and also because the anchorages would fall within the side curb.

The cables are continuous all along the deck and jacked from both anchorages near the abutments from above the deck. This gives a total cable length, for each cable, of 120 meters roughly.

Anchoring blocks at the abutment are cast in the top slab, locally reinforced with an underneath concrete block connected to the beam and slab by steel shear connectors. These concrete blocks are cast using the slots/holes that are cut into the deck slab to allow the cables emerging from below to be tensioned (jacked) from above the deck.

Figure 5-1Side and alignment view of Gibe river bridge

Figure 5-2 The Gibe River Bridge strengthen cable configurations

Cables are deflected using steel tubes with a 2 meter longitudinal radius. These tubes are either grouted in the diaphragm or supported by saddle resting against the main deck longitudinal beams.

As said above the cables are made of 12 strands individually waxed and sheathed. The 12 strands of each cable are contained in a PHDE pipe of 110 mm outer diameter. These pipes are continuous along the bridge thus passing through the steel tube deviators that support an inner diameter of 120mm. The pipes won't be grouted thus simplifying the onsite operation dramatically.

Protection against corrosion remains more than adequate as the strand are individually waxed and sheathed and further protected by the PHDE pipes.

Cable deviation at the diaphragm ensure an evenly distribution of the vertical component of the cable deviation force between the 4 longitudinal beams.

The cables are arranged into two groups of different geometry so to spread these vertical deviation forces between the 1^{st} and 2^{nd} diaphragm at each span.

The increased load bearing capacity of the bridge

The increase in load bearing capacity of the bridge obtained with the proposed measures is significant and allows the repaired bridge to carry load in excess of 150 ton.

Further increase of the shear carrying capacity with respect to the actual (damaged) configuration is provided by the application of the axial force in the concrete deck (longitudinal component of the prestressing force).

<u>Maintenance Specifications and phasing of works of Gibe River Bridge</u>

A detailed description, method specifications and phasing of works is provided in the following

1. **Phase 1-Boring Holes into the diaphragms**

Holes into pier diaphragms shall be bored first. The holes shall be bored as close as possible to the corners where the diaphragms, beams and deck slab meet. A distance of 16cm has been assumed between the holes axes and the deck/beam surface. Holes diameter is 17cm.

Diaphragm are scarcely reinforced nonetheless it is likely that boring will require cutting the existing D12 reinforcement spaced at 40cm center.

Holes into the intermediate diaphragms shall be bored subsequently.

Holes bored in the intermediate diaphragms deviations is applied need to be protected against corrosion of reinforcement with the application of epoxy painting or other equivalent product.

2. Phase 2-Geometry check and steel saddles manufacturing

Once the holes are bored, a geometry check shall be performed using a wire to be tensioned all across the bridge to trace and check the actual cable layout and geometry with respect to drawings

Once accounted for variations and/or tolerances, the steel deviators(tubes, saddles and reinforcing frames) can be shop manufactured and brought to site for positioning.

The supervisors recommend the shop fabrication of 1 prototype per each item so as to check directly on site tolerance and erection requirements.

3. Phase3-Fixing the saddle to the deck

The saddles and steel tubes shall be fixed to the diaphragms and beams using epoxy resin or other similar products. To hold the bottom saddles in place, temporary fastening into the beam can be used as specified in the drawings.

The steel tube deviators for cables type B over the pier diaphragms are cased two concrete blocks to be cast in situ while positioning the above tubes. Technology, material procedures are the same detailed for the anchoring blocks at the abutments described in the following. The two operations can be carried out in trail.

4. Erection of the PHDE Guide Pipes

Once the deviators have been fixed, the PHDE pipe provided by the prestress supplier shall be put into position. These pipes are generally shipped in 6 meters segment. The segments are joined to each other used a purposely made FRP tape provided by the supplier and lay out between deviators.

The PHDE pipes are 110mm outer diameter and shall be continuous over the whole length. Steel tube deviators are 120mm internal diameter although curved. The PHDE shall fit tightly but comfortably into these tubes.

5. Casting the anchor blocks at the abutment

This operation needs the traffic to be diverted on one side of the available platform. Currently the platform has already been narrowed to a single lane to reduce live loading.

With the traffic all on one side the anchor blocks for the 3 cables (1 type B,2 types A) on the other side of the deck shall be casted as follows:

- Drilling and fixing the shear connectors (normal D16 reinforcement chemically bonded in to the existing structure) will be carried out first from below the deck. The works are to be carried out beside the abutment with simple scaffolding from the ground.
- Cutting the slots into the deck slab. This is better performed with a wire/chain saw or conversely can be carried do out with the hammer.
- Assembling the reinforcing cages and the cable anchorages according to drawing and cable supplier specifications.
- Casting the anchoring blocks with ready to use high strength concrete (f_{ck}>45 MPa)

Once the anchoring blocks on one side are finished, traffic shall be deviated onto this side using temporary steel plate to cover the anchoring pits and works performed for the other 3 cables/books.

6. Strand spinning, main cracks injection and cable tensioning

Strand spinning can be carried out as soon as the anchoring blocks are finished strand spinning will require the single lane currently in use to be diverted alternatively on the two sides to allow the strand pushing to be operated from above the deck. Before tensioning the cables, the main shear cracks at pier 2 and any other major vacuum and defect of the deck girders shall be injected with appropriate resin (not expandable, two-component silicon-mineral –organic formula with low viscosity and high mechanical strength).

Bridge external prestressing a strong maintenance/ repair tool for restoring a failed RC and can be applied to prestressed system.

5.4 Ethiopian bridge strengthening practice on OMO RIVER BRIDGE

The bridge consists of 36m Approach Bridge both side and 128m main span steel truss. The major damage of the bridge is loose its load caring capacity. This damage has been caused by accident, during the accident loader carrying lobed vehicle failed in the bridge as a result of which the bridge was then highly damaged on one abutment side.

The whole work, design and post tensioning construction, has been executed within 7 months. The bridge was strengthened by post tensioning of the truss bottom chord 7 strand was used in a tendon. Totally four tendons and 1000MPa jacking control stress were used.

Prestressing strand

The prestressing strand shall be seven-wire super strand of class 2relaxation with the nominal tensile strength of 1860MPa. Prestressing strand shall be stacked on the ground. All material analysis and test certificates for the batches of prestressing steel used in the work were properly recorded and handled.

Anchorages and Couplers

The anchorages were imported from China with a satisfied certificate at exit of the factory. In general, anchorages to be used shall comply with the requirement, constructed from durable material completely free from imperfection and shall not damage, distort or twist the prestressing steel in a manner as will result in ultimate tensile strength reduction. They shall resist, without failure and/ or excessive deformation or relaxation the force in the tendon, the full ultimate tensile strength of the tendon. The anchorage shall effectively distribute the force in the tendon to the structural member and the resulting local stresses and strain in the member shall be so limited as to prevent damage.

Sheaths

Rigid steel pipe was used to protect the prestressing tendon from damaged by weathering or mechanical damage caused by vehicle or machine. In general the property of the sheath material shall be such that no corrosive attack of the prestressing steel will be induced. It is sufficiently flexible to accept the required curvature without kinking, strong enough to retain its cross section and alignment and to resist damage on account of handling, transport and construction operation.

Equipment

All tensioning equipment was properly calibrated before the tension operation was conducted.

Post tensioning process

Prior to the commencement of post tensioning operation, all the prestressing steel was carefully checked and tested and post tensioning equipment has been calibrated.

The tensioning process has been illustrated by the following chart

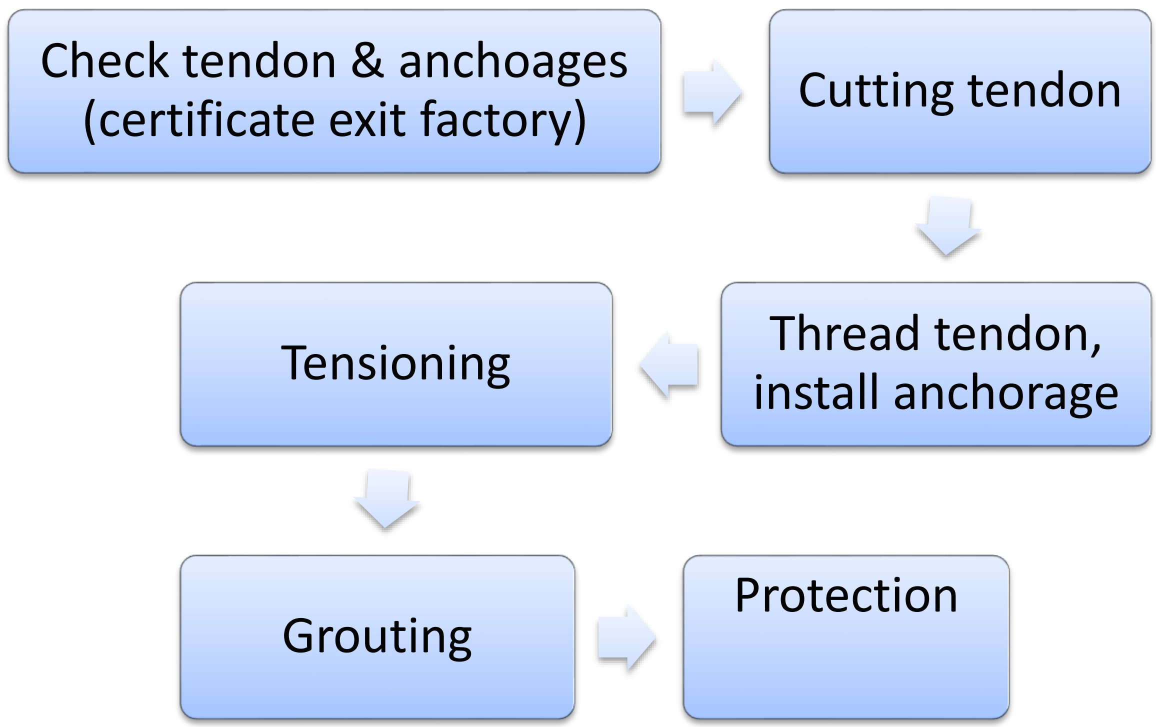


Figure 5-3 tensioning process of the OMO river bridge

Note: checking of tensioned strand and proper curing should be handled.

All tendons were cut by a high speed abrasive cutting disc. The sheaths are steel pipe with enough rigid strength and grout tight and bond force can be transferred from grout to the surrounding structures. All the tendons are accurately located and maintained and secured by steel weld at an interval of 10m which has been supported by bottom level chord and steel plate board fixed with the bridge. The joint of sheaths are tightly fixed and be watertight to prevent the ingress of grout.

The tendon axis is putted perpendicular to the bearing surface of its anchorage and firmly secured in position so as not to move during operation.

Post tensioning

The post tensioning operation has not commenced until the end facilities has been properly installed and checked. And all calibration work has conducted. The extensions of tendons are measured to an accuracy of ± 2%. After the final tension force applied a) the tendons has get the required tension force b) the measured extension of the individual tendons is ±6% of the theoretical value this is almost within the allowable limit.

Duct installation

- Steel plate for duct position was installed within 10m interval
- Each tendon is provided with four number of grout vent port at a distance of 25.6m

Tendon installation

The tendons are pulled through the embedded duct by crane after duct installed.

- A strand is passed through the duct by manpower
- Connect the strand and steel wire which one end fixed on windlass
- Pullout the strand and the steel wire that passed through the duct
- Starting the windlass and dragging the tendon in to the duct after fixing the steel wire with the joint of the tendons.

Stretching of the tendons

The tendon was tensioned from one end and 4 sets of 150T jack used. The jack is moved by 2ton hand chain pulley block.

A steel meter used to measure the extension of tendons. A marker pen is used to make a sign at the end of tendon prior to tensioning. After completing of stressing, the projection tendons are cut by a hand electric saw before the permanent grout caps and there will be 20mm remains after cutting. After passing all this strengthening process the bridge has started to offer service

Figure 5-4 alignment and side view of the Omo river bridge

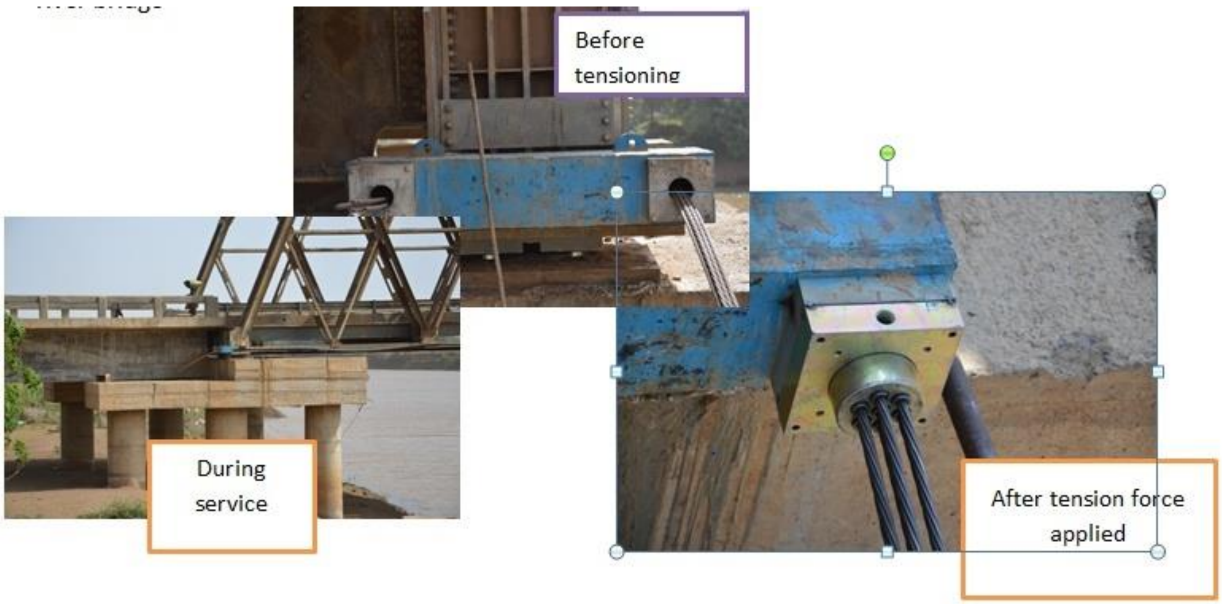


Figure 5-5 tensioning of the strands on Omo River Bridge

6 Discussion, Conclusion and Recommendation

6.1 Discussion

In our country a vast railway network is under construction and the Addis Ababa Light Rail Transit (LRT) has officially started providing service for the community since Sep 20, 2015. In this regard almost all completed and ongoing structures are constructed using prestressed concrete system, specially the super structure part. For this reason, this research has been carried in the area of damage identification and adaptation of maintenance technique for prestressed concrete bridges.

Bridges are key elements of the highway and rail road network by virtue of their strategic importance in connecting vital segments of roads. In case they get damaged due to any reasons, their loading capacities can severely impair, and costly consequences would unfavorably occur.

Railroad bridges are more serious than high way bridges. For instance if the highway bridges come to be fail, even the traffic flow get a lot suffer, there might provide access by constructing detour either in the up or down stream of the bridge. However, if the same scenario will happen in the railway bridge construction of detour is really very difficult task the traffic will completely blocked. This justifies that railway bridges need more care.

Consequently, unlike Reinforced concrete bridges pre-stressed concrete bridges are more sensitive for damage. Since a large amount of load is carried by a small cross-section area, a small damage on the prestressing strand can cause high reduction in its load caring capacity. For this reason, the prestressed concrete structures should have serious follow up and inspection.

Therefore, maintaining a full stock of bridges in good condition to secure the safety of traffic at any time is the utmost important duty for the sector. So such type of study has a vital role to achieve the sector duty.

Through the course of this research, prestressed concrete system have been categorized based on various ways, such as source of prestressing force, external or internal prestressing, pre tensioning or post tensioning, linear or circular prestressing, full or limited or partial prestressing, and uniaxial, biaxial or multi-axial. The concepts of prestressing, Application of Prestressing, Prestressing tensioning device, has been discussed in section 2.1.

To assure the continuity the structures proper monitoring and inspection and maintenance for the defective components are significantly vital. In view of that, purpose of bridge inspection, types of inspection according to ERA and AASHTO manuals, frequency of inspection, inspection techniques was covered. In addition to this, since Ethiopian Roads Authority is one of the most experienced Authorized organizations in Ethiopia its damage rating system has been considered in section 2.2.

Generally Pre-stressed Concrete components can be frequently damaged & deteriorated by one of the following a) High Load Damage or/and over-height impacts, b) Fire Damage or c) Corrosion Damage. For the ease of maintenance these different defects could be categorized in as minor, moderate and sever Damages. Weathering, delayed maintenance, aging and over height vehicular impacts are some of the reasons for infrastructures deterioration.

The main causes of prestressed concrete Bridge deterioration are discussed in section 2.3 of this document. Some of them are corrosion of embedded strand, freeze- thaw deterioration, chemical attack, Alkali-aggregate reaction, Abrasion and erosion, Fire and/or heat, Restrain to volume change, Over loading, over height and impact, Loss of support: loss of ballast, sleeper , Surface defect (rail uniformity).

The fundamental objective of Maintenance is to assure the continuous performance of a structure or to preserve the structure in such a way that it will functional satisfactory at the various limit states. Preventive maintenance can be defined as the act of keeping a structure in its as-built condition and/or protecting it from inevitable deterioration due to environment, traffic vibration and deicing chemicals.

Common damage types on Prestressed Concrete, which are discussed in the research, are Crack, Spalling, Corrosion of Tendon, damage to prestressing strands, Rail failure and Ballast failure on the deck, Sleeper displacement and Loosening of fastenings, defects on anchorages and bearings, Deformation of shape or Bulging, Scour/ erosion of the underside of the sleepers, Miscellaneous failure and others.

Fatigue, Time-Dependent Effects, Temperature Effects, over height Impact damage, Excessive loading, over increasing of traffic, Dynamic loading, and others are the major cause of Damages type on Prestressed Concrete which covered in the thesis.

The main objective of this thesis is identification of damage and adaptation of maintenance technique for Ethiopian prestressed concrete railway bridges. This is done by adapting the AREMA standard, NCHRP report 20-07(307) result, ERA concrete bridge maintenance manual and others countries maintenance practice to the Ethiopian condition. However, to practicing such techniques as presented in the preceding section the availability of advanced technology, materials and equipment and skilled workmanship is expected to be challenge issue. Therefore, to minimize these challenges, it is necessary to develop the capacity of the existing staffs of Ethiopian Railway Corporation and prepare knowledge transfer conference and trainings for the staff.

6.2 Conclusion

Identifying bridges damage is very important task to know the cause and effective maintenance techniques of the bridges. Consequently, the major prestressed concrete bridge defects have been identified by conducting inspection along some road segments of the Sebeta- Dewele route and most parts of AA LRT railway line, referring different manuals, literatures and the previous experience of the country. Therefore, since ERC is newly re-established firm such studies are very vital as a draft document for the institute, to prepare Bridge management and keep its bridges operational.

In general, Pre-stressed Concrete bridge components are frequently damaged by High Load Damage and/or over-height impacts, Fire Damage or Corrosion Damage.

In Ethiopia over height impact is a very seriously observed cause for the occurrence of damage on the over pass railway Bridges along Sebeta –Dewele route. Even if the construction along Sebeta -Dewele route was completed shortly, they have been facing a numerous problems due to over height vehicular impacts.

To avoid over height impact from railway bridges, the future at cross railway bridges shall be constructed underneath the highway roads or sufficient space (clearance) should be considered in design in addition to protection against collision by under passing vehicles. However, for the already completed bridges other safe accesses such as detour may be considered in addition to provision of protection barriers against potential collision.

Even though the major common damages on Prestressed Concrete structure are Crack, Spalling, Corrosion of Tendon and damage to prestressing strands, railway bridges along Sebeta –Dewele road route are affected by Shear crack, Corrosion and scour & erosion. It is obviously known that railway bridges are new and have not any observed defects except some spallings that developed by the result of over height impact. However, from condition survey data, which was collected from Ethiopian Roads Authority Bridge management Team, shows that the most common damage on Highway Bridges along Sebeta –Dewele route are cracking, corrosion, deck slab void, peel off/ spalling , Scour & erosion, water leakage, honey comb, drainage pipe damage, Expansion Joint Missing.

Even though all the above defects were observed crack, corrosion and scour & erosion are the dominant. From the sample bridges taken, about 76.5% having crack, 49% affected by corrosion and 39.2% affected by scour & erosion. Therefore, the railway bridges along Sebeta- Dewelle route will have potential damage of cracks, corrosion and scour & erosion.

Most railway bridge piers in Addis Ababa Light Transit have no protection. Unless protection wall is built around a bridge pier soon, vehicular collusion damage will definitely disturb the overall bridges components and then will become the cause for the development of many defects such as crack, spalling, shifting of rail, derailment etc. In addition, Bridge piers at St. Estifanos and along St. Lideta to Coca cola spalling, rebar exposure and honey comb were observed. Besides, moderate cracks were also observed on Ayat terminal station bridge.

Ease of maintenance, all defects occurring on prestressed concrete structure was categorized as minor, moderate and severs. Since minor and moderate damages are considered as non-structural and maintained for aesthetic aspect, their maintenance technique is similar to reinforced concrete structures. However, sever damage maintenance technique is different.

Severe crack/spalling damage in any damage requires structural repairs. Typical damage at this level includes significant cracking that leads to exposed and broken strands. Such type of defects required advanced maintenance techniques. Post-tensioning and CFRP are the best maintenance solution. CFRP is preferable for flexural and torsional crack maintenance but post tensioning can be implemented for both shear and flexural crack maintenance. The Gibe River Bridge and the Omo River Bridge are strengthened by post tensioning technique.

Sever strand damage has to be maintained by external post-tensioning, metal sleeve splicing (or steel jacketing'), strand splicing, and a combination of these methods. But if the damage has not been repairable replacement should be recommended.

The non-sever defective rail part can be treated by grinding mechanism whereas for sever rail damage cut and replace the damaged part is the solution. Generally, consideration should be given when designating the length of the replacement rail to the size, defect type and rail characteristics. Replacement rail is to be certified free of defects. Particular care should be taken to prevent adding additional rail during the replacement process.

The new Ethiopian railway system has contained both ballasted and slab track system. Sleepers on slab Tracked Bridge shall maintain monolithically with the girder. However, for the ballasted truck damage cleaning and replacing of damaged part is necessary. If the sleeper is severely damaged replacement shall be recommended.

Finally, timely inspection and maintenance is very vital before further propagation of damage.

6.3 Recommendation

The study has addressed potential major damages on prestressed concrete bridges and the necessary maintenance techniques by referring different guidelines and manuals in addition of specific case references practiced in this country of highway bridges.

The following recommendations are set for further researches.

- In this study the viability and the application of carbon fiber reinforced polymer and post-tensioning system are discussed. Sever crack can be maintained by either of the two procedures. As the former is not thoroughly addressed using case study, it is recommended for other researcher to Compare Fiber Reinforced Polymer and Post tension technique for severe crack maintenance.
- Prepare guideline for capacity development of railway bridges.

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