REHABILITATION
OF
CONCRETE BRIDGES

by

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ABSTRACT

This thesis presents the various methods of assessment and repair of damage to concrete bridges. The current repair and replacement techniques are identified and evaluated. Plausible repair-in-place techniques being used presently are reviewed along with the relevant details. Guidance is also provided for inspection and assessment of damage from accidents, fire, design and construction defects, and other causes.

Suggested guidelines for damage assessment and for selection of repair methods are presented along with some examples from the existing practice. The repair methods have been evaluated on the basis of load requirements, speed of repairs, durability, relative costs, aesthetics, materials, methods, and engineering solutions.

The thesis also deals summarily with the subject of bridge management systems, highlighting the need for effective maintenance and repair management strategies. Examples of two software packages being used currently for management purposes are presented. The case histories presented highlight the various current practices of rehabilitation and replacement

RÉFECTION DES PONTS EN BÉTON

par

Kulbhushan S. Tullu

SOMMAIRE

L'auteur fait une revue des diverses méthodes de détection et de reparation de dommages structuraux dans les ponts en béton. Les methodes présentement utilisées sont decrites, comparées et évaluées, de même que les techniques de réparation sur place L'auteur discute également l'inspection des ouvrages en vue de l'évaluation des dommages dus à des causes diverses telles les collisions de véhicules, les incendies, les défauts de construction et de conception, etc.

L'évaluation des méthodes de réparation est faite suivant les critères suivants exigences structurales, rapidité des travaux, durabilité des réparations, coût relatif, apparence visuelle, qualité des matériaux utilisés, fiabilité des techniques utilisées, et concept structural

L'auteur discute aussi brièvement la problématique de gestion de la maintenance des ponts et insiste sur l'importance de l'efficacité des stratégies de réparation Les principales caracteristiques de deux logiciels de gestion de la maintenance des ponts en usage sont decrites Enfin, trois cas practiques illustrent plusieurs des techniques de refection et de remplacement decrites dans la these.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1
INTRODUCTION

1.1 General

1.1.1 Concrete Bridges

The term concrete bridges, as used in this thesis, refers to all bridge structures with a concrete deck, hence it includes all kinds of composite bridges, concrete deck and prestressed or reinforced concrete or steel girder bridges. For brevity, all of these composite bridges will be termed "bridges".

1.1.2 Need for Repair and Rehabilitation

Bridges are designed to provide satisfactory service over a long period of time. The life of any bridge depends on the preservation of the physical integrity of both the superstructure and the substructure. Whilst bridges are generally required to have a design life of over 100 years, a number are suffering durability problems at a fraction of this age. Of the 572,000 highway bridges built before 1940 in the United States, over 42 percent are classified as structurally deficient because of deterioration or distress, or are functionally obsolete requiring rehabilitation and replacement estimated to cost more than $ 50 billion. Twenty percent of 60,000 bridges on motorways and trunk routes in the United Kingdom suffer from decay and corrosion. The U.K spends £ 35 million on bridge maintenance and repairs annually. In Germany, a survey of its 30,000 prestressed concrete bridges showed that 600 bridges were damaged and the repair cost was estimated at DM 200 million. France with 6700 bridges (span larger than 5 m) plans to allocate $ 40 million annually for 20 years, of which one third is for routine maintenance and the remaining two third for rehabilitation and strengthening of about 25 percent of the stock A recent survey of the bridges on national highways in India showed that of the 6500 bridges surveyed in 23 states, about 1100 showed distress and some 50 percent of these are in need of replacement.

Recently, considerable emphasis has been placed on bridge rehabilitation. Rehabilitation simply means restoring a bridge to its proper condition. A global strengthening or upgradation is not a part of the standard rehabilitation process, although individual components may be strengthened or replaced. Technically speaking, rehabilitation is a modification, alteration, retrofitting or improvement to a bridge structure in order to correct defects or deficiencies to ensure a reasonable service life (33).

Rehabilitation engineering is a highly specialized field requiring special skills that are beyond classical design engineering Over the years, various repair systems, testing techniques and proposed remedies to damaged, deteriorated, or distressed concrete bridges have been developed However, they do not always provide consistent and predictable results which has necessitated the development of standard inspection, testing and repair techniques through proper classification of rehabilitation needs.

1.1.3 Scope of the Problem

The scope of the problem is broadly classified into two aspects: Replacement and Rehabilitation. The decision between them seems to be rather difficult to make and essentially depends on the severity of damage to the bridge. The damage may be either due to structural defects and concrete deterioration or it could be an accidental damage. Replacement of whole or a part of the bridge is necessary if the damage is beyond repair as is the case when a complete bridge or some part collapses. The method of selecting a particular repair method is based on several factors including the nature of damage, durability of repairs, inconvenience caused during repairs, speed of repairs as well as economic and aesthetic considerations.

1.1.4 Research and Development

At present, the decision to rehabilitate or replace a damaged bridge and the techniques used are determined by evaluation of the situation by local engineering authorities with very little published information available for guidance. The main objective of this research program is therefore to identify the various procedures being used for investigation, assessment and repair / rehabilitation. The research has been accomplished by categorizing the various bridge components, identifying the inherent problems and suggesting possible remedies that have been applied in practice. Economical replacement systems have been identified where repair or rehabilitation is not feasible.

The repair and rehabilitation procedures described are applicable to common structural and functional deficiencies found in bridges. They have been generalized to apply to a broad range of bridges. Nevertheless, it is expected that additional procedures will be developed through future research and development.

1.2 Rehabilitate/Replace Decision

The decision between rehabilitation and replacement is made only when external constraints have not dictated that the bridge must be replaced Cost is usually the governing criterion and it is generally accepted that a rehabilitation project with a cost of more than half the cost of a new bridge should not be built but instead it should be replaced Two reasons supporting this statement are presented from Reference 8.

"1) The amount of work to be done when rehabilitating a bridge inevitably grows during construction Even with the best in-depth inspection and evaluation, some items that require repair may be overlooked, some items cannot be found until the construction uncovers them, deterioration continues during the period between the inspection and the start of the construction work, and the rate of deterioration is difficult to estimate and account for in the plans and estimates Further, those responsible for the construction of bridge rehabilitation projects must constantly walk the line between doing too little work and not achieving the design life of the project, and doing so much that the cost of the project escalates dramatically during construction This problem can be reduced by spending more on the in-depth inspection

2) The bridge, after rehabilitation, is unlikely to perform as long or as economically as a new bridge built at the same time The rehabilitated bridge will be a hybrid of the new and old design concepts, detailing concepts, and material types and as such cannot be expected to perform as well as a new bridge that reflects the current state-of-the-art. Also, the inevitable dilemma of when to stop removing deteriorated concrete or steel during construction results in compromises that may affect the useful life of the rehabilitated bridge"

Another important criterion that needs consideration is the ability of the bridge to be readily rehabilitated. Rehabilitation should not be carried out on low strength or partly deteriorated concrete that cannot be easily restored. Consideration also needs to be given to the vertical and horizontal geometry and load carrying ability of the bridge. For example, a bridge with poor geometrics and bad accident record should not be retained but instead it should be replaced Also, a bridge requiring load restrictions even after rehabilitation should be replaced unless the load restriction will not cause any economic disadvantage to the area served by the bridge Exceptions to the above mentioned criteria should be made only after detailed investigation and evaluation. Moreover, these criteria can be applied before detailed inspection

1.3 Rehabilitation Strategies

Tracy and Fling (63) have developed a rehabilitation matrix which attempts to simplify the rehabilitation process. As shown in Figure 1 1, the matrix emphasizes the implementation of three aspects necessary for a renabilitation program, namely, a detailed condition survey, structural investigation and selection of a repair program The condition survey identifies the defects in the design assumptions, analysis, calculations or detailing It also identifies the amount and nature of damage which may be due to fire, earthquake, flood, foundation settlement, impact, abrasion, wind, long term overloading, sustained high temperatures, etc. Furthermore, it can identify the amount and nature of deterioration which might have been caused due to freeze-thaw related scaling, leaching, popouts, alkali-silica reaction. cracking, joint deterioration, etc. Investigation of structural aspects involves checking the feasibility of tne design systems to be implemented such as cast-in-place conventionally reinforced or plain concrete, cast-in-place post- tensioned concrete, precast prestressed concrete, combination systems, etc. It identifies the type of the bridge including its traffic condition, serviceability considerations, load carrying capacity and functional adequacy, extent of repairs needed, maintenance history and projected future needs, etc. It also serves to identify the constraints in the movement of workers, materials and equipment, as well as the maintenance of current operations and work scheduling. After a full assessment of all the above parameters, the action plan to be implemented for rehabilitation can be adopted.

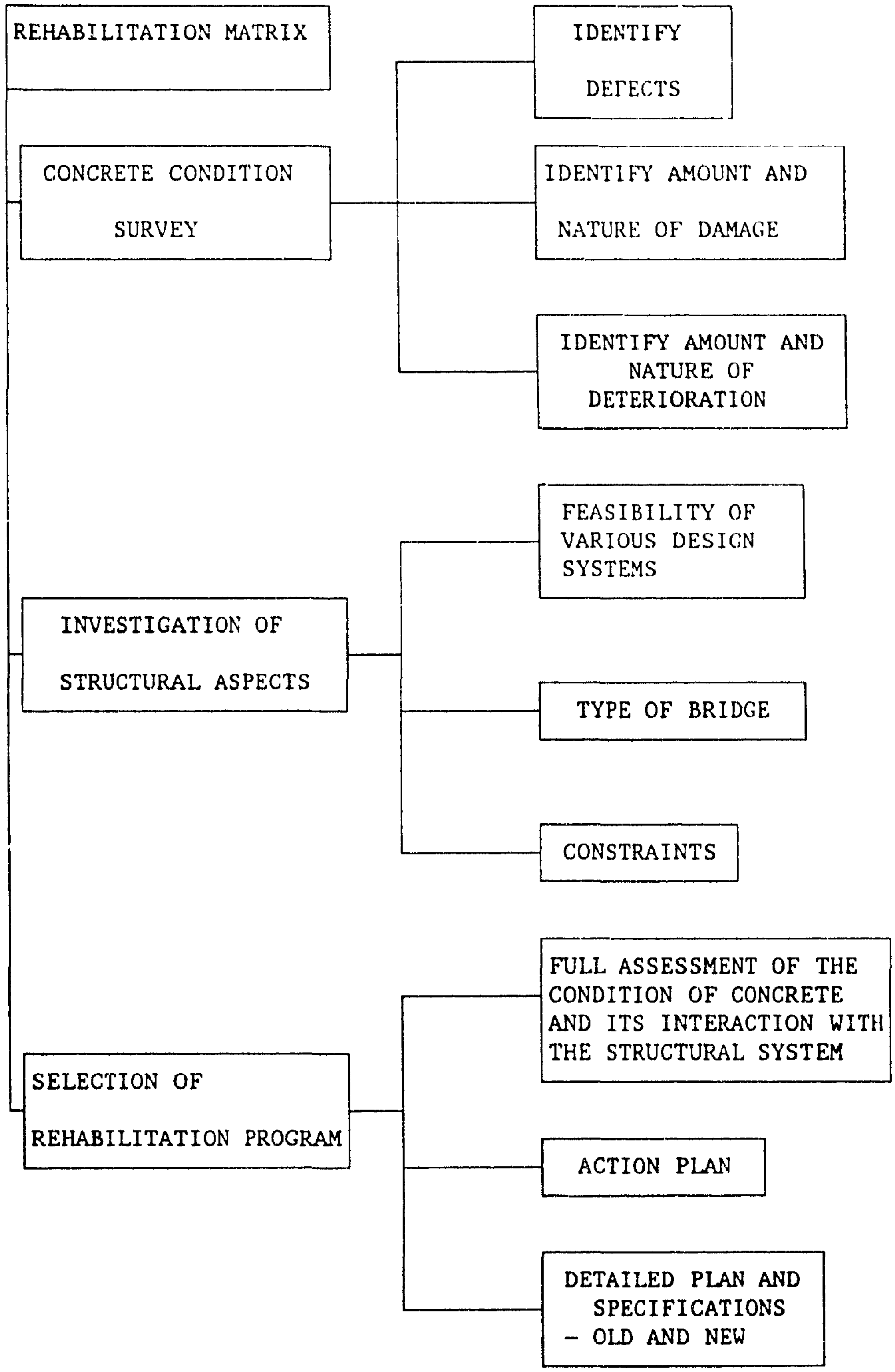


FIGURE 1.1 Rehabilitation Matrix (63)

CHAPTER 2
DECK SYSTEMS

2.1 Nature of the Problem

The phenomenon of premature deterioration of concrete bridge decks has continued to be the major problem in the recent years. This is because the bridge deck environment is one of the worst imaginable exposure conditions for the concrete. It is well known that a vertical surface is more durable than a horizontal surface, and that alternate wetting and drying is a more severe exposure than total submersion, and that freezing and thawing is more damaging than constant freezing It is also known that a minimum concrete cover of at least 50 mm is necessary to protect the reinforcing steel against corrosion by salt water in a marine environment. Yet, bridge decks have been subjected to frequent application of deicing salts on a horizontal surface, alternate wetting and drying, and freezing and thawing, and also the concrete cover used has been much less (around 40 mm) even for a severe type of exposure.

In addition to the above, bridge decks are also subjected to severe temperature changes and high live load stresses, including fatigue and impact. In lieu of the severe congestion of reinforcement, it is necessary to use highly workable concrete. The quality of concrete is usually the worst at the surface of the deck due to finishing and bleeding.

All of the above factors lead to the premature deterioration of the deck. Problems commonly encountered are **scaling**, **delaminations**, **spalling**, **cracking**, and **wear and polishing**. A brief discussion of these problems follows.

2.1.1 Scaling

Scaling is defined as the flaking of hardened concrete at the finished surface. Decomposition of the cement paste occurs gradually starting from the surface and progressing inward. Thus, scaling initially starts in the form of small local patches which may merge with passage of time and extend to form large areas. Scaling could be of three types: Light, Moderate, and Severe. Light scaling does not expose the coarse aggregates. However, moderate scaling exposes the aggregates and may involve loss of up to 3 mm to 10 mm of the surface mortar. In case of severe scaling, the mortar fraction of the concrete is completely broken down which loosens the aggregates. Cases of Moderate and Severe scaling are shown in Figures 2.1 and 2.2, respectively.

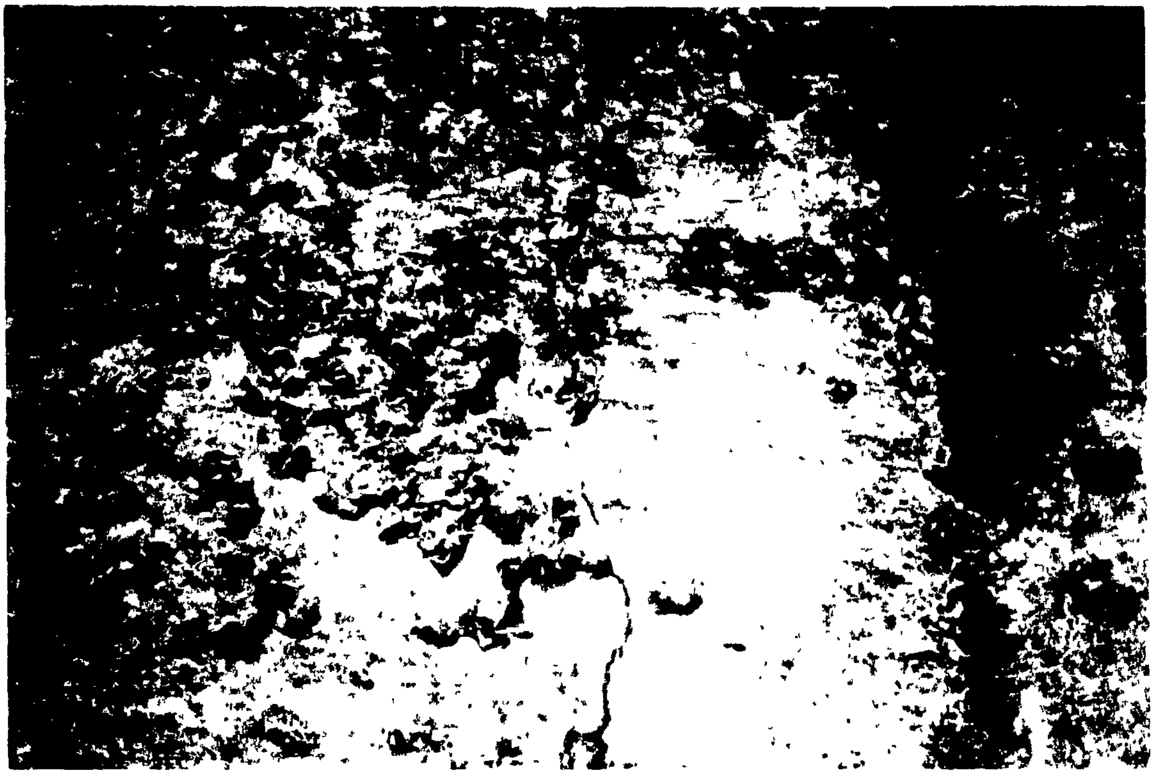


Figure 2.1 Moderate Scaling

Figure 2.2 Severe Scaling

Scaling occurs basically because of frost action and the presence of deicing salts (NaCl or $CaCl_2$) Due to frost action, there is a combined creation of dilative pressure due to accretion and osmotic pressure in the pores which can cause cracking or mechanical damage in the cement paste. The reader may refer to Reference 15 for detailed information regarding the mechanism of scaling However, this can be reduced by the use of properly air-entrained concrete with an appropriate amount of air-entraining admixture. ACI Committee 201 (3) has recommended the following air contents for deck concrete to increase its frost resistance.

Maximum nominal aggregate Size in (mm)	Average Air Content (%) for Severe Exposure(*)
⅜ (9 5)	7½
½ (12.7)	7
¾ (19)	6
1½ (38 1)	5½

** A reasonable tolerance for air content in field construction is ± 1.5 %.*

Air entrainment, however, does not always give complete protection against scaling. The presence of deicing salts increases the degree of saturation of concrete and after getting nearly saturated, air entrained concrete becomes as susceptible to frost action as normal concrete.

Scaling occurs basically on account of the following:

a) Use of non-air-entrained concrete.

b) Application of calcium or sodium chloride deicing salts.

c)If finishing operations are performed while there is bleed water on the surface, this results in a very high water-cement ratio and hence a low strength top surface layer.

d) If curing is not done properly or if it is insufficient, the surface of the deck weakens and scales when exposed to freezing and thawing in the presence of moisture and deicing salts

As recommended by the National Ready Mixed Concrete Association, Maryland, U.S.A., the following rules should be followed generally to prevent scaling:

a) For moderate to severe exposures, air-entrained concrete of medium slump (3 to 5 in) should be used and cured properly. In cold weather conditions, low slump concrete with an accelerator should be used

b) If late fall placement cannot be avoided in moderate to severe climates, deicers should not be used for the first winter and the surface should be sealed with boiled linseed oil.

c) All finishing operations should be carried out in time

d) Proper air-entrained mix should be selected to match the placing conditions

Repair

For light to medium scaling, it is sufficient to seal the surface superficially by applying surface water repellents to minimize the effect of deicing salts. This is the most economical treatment and can be used both for non-air-entrained as well as air-entrained concrete placed in the fall which is subjected to the application of deicing salts during its first winter The most effective and economical solution is the application of two coats of a 50/50 mixture of boiled linseed oil and mineral spirits to the previously cleaned and dried deck However, since different concretes have different porosities, the actual application rate should be determined from a test section on each deck. Very light and very severe applications should be avoided as this may adversely affect the skid resistance of the deck. This treatment should be repeated after every one to three years, if necessary.

For severe scaling, overlays are used instead of coatings. However, prior to installation of an overlay, it is necessary to carry out extensive tests for determining the half cell potentials, chloride ion concentration, etc.

2.1.2 Delaminations

Delamination is the separation of the concrete layers at or near the level of the top or the outermost layer of reinforcing steel and is usually parallel to the surface of the concrete member As mentioned earlier, repeated application of chloride deicers or a marine environment are the most severe exposure conditions for decks. The ingress of chloride ions corrodes the top mat of the reinforcing steel which results in expansion of the concrete above it, thereby causing delaminations. Corrosion can be worse in concrete decks with inadequate cover

Delaminations can be easily detected by use of a rod or hammer which produces a hollow sound when tapped on the deck surface. The extent to which delaminations have occurred is usually determined by a chain drag or by proprietary mechanical devices like the **Delamtect** or by acoustic methods. This has been explained in Section 2.2.

Delaminated concrete should be repaired immediately, otherwise it may grow to a spall Common repair procedures include epoxy injection with the delaminated concrete in place or using spall repair procedures explained in the next section.

2.1.3 Spalling

Spalling may be defined as the separation and removal of the surface concrete in the form of fragments. Unlike scaling which is a surface phenomenon, spalling results from corrosion of the embedded reinforcing steel It also results from the presence of chloride ions from the deicing salts which permeate into the concrete to the reinforcing steel. It is the next stage of deterioration after delamination. As the chloride ion concentration exceeds the peak value, corrosion of reinforcement begins in the presence of oxygen and moisture. The extent of chloride ion penetration depends on the quality of concrete, thickness of the concrete cover, and the size and spacing of the reinforcing steel.

Spalling in bridge decks can be detected in the form of horizontal cracks above the corroding bars. If the concrete quality is poor, the reinforcing bars restrain the sedimentation of high water content concrete after placing and finishing. This produces a weak fracture plane at the level of the steel which when subjected to vehicular loading and ice formation, results in the formation of a pothole.

NCHRP Report 57 (20) defines corrosion as follows:

"*Corrosion, an electrochemical reaction, requires an anode, a cathode, an electrolyte, and the presence of oxygen. Moisture in the concrete serves as the electrolyte and oxygen is generally available. Points on the reinforcing steel serve as the anode, where ions are discharged and corrosion occurs, and the cathode, where ions are received. Anodic and cathodic areas form at points on a reinforcing bar where there are differences in surface conditions or in the environment around the bar.*"

The formation of iron oxides as a result of corrosion of reinforcing steel cause cracking of the concrete since iron oxides occupy a very large volume of upto 13 times the volume of the original metal creating pressures as high as 32 MPa. Conical spalls occur if the corroding area of the reinforcing bar is large. Corrosion of steel may be accelerated in open spalls if atmospheric corrosion occurs and could result in severe pitting of the steel with significant loss of concrete section. Once initiated, it is extremely difficult to halt the corrosion process and permanently repair the damage.

Prevention

Corrosion of reinforcing steel can be prevented by using concrete having low permeability, i.e., having quality materials, minimum water-cement ratio consistent with the placing conditions, good consolidation and finishing practices, and proper curing. Corrosion protection may also be achieved by increasing the concrete cover which should be consistent with the water-cement ratio. The report of the American Concrete Institute Committee 201 (3) recommends a minimum of 50 mm cover for bridge decks with a water-cement ratio of 0.4 and 65 mm for a water-cement ratio of 0.45.

Repair

Before undertaking any repairs, it is essential to carry out a detailed evaluation to determine the chloride ion concentration, extent of delamination and the half cell potential. The entire planning necessary to carry out repairs is described in Reference 20.

It is necessary to thoroughly examine the cause and extent of deterioration of the deck since the area of active steel corrosion and chloride contaminated concrete is usually much larger than the area of spalled or delaminated concrete (4). It may, therefore, not be enough to just repair the area of spalled or delaminated concrete since this way the deck may need continued repair. Instead, it will be more economical and durable to remove the concrete supporting corrosion and providing a waterproof membrane, bonded topping or overlay. The type of repair usually depends on the extent of damage and may range from patching of small isolated spalled areas to the application of polymer cement concrete or latex modified cement concrete.

A method to determine the amount of concrete to be removed is to base it on the percentage of unsound areas in the deck surface as followed by the Minnesota Department of Transportation (46). The most ideal solution to repair a bridge deck permanently is to remove all of the chloride contaminated concrete and prevent any further ingress of deicing salts. Moreover, deck replacement or removing the concrete below the reinforcing layer is very expensive and alternate cost effective measures as suggested by the Minnesota Department of Transportation (46) should be used.

Another repair method for repair of surface scales uses a 50/50 mixture of boiled linseed oil and mineral spirits applied to the deck surface in two coats. However, this procedure results in penetration depths of less than 3 mm and has little or no effect in preventing spalling. For repair of spalls, deep impregnation with sealants is carried out which immobilises the chlorides present and prevents any further ingress of water, oxygen and deicing salts into the concrete

The deck concrete is first dried using suitable trailer mounted catalytic infrared heaters to a temperature of at least 110^{0}C (230^{0}F) to remove moisture. It is then cooled for about two hours Portable, reusable, impoundment dams in conjunction with a suitable sealing or gasket material are constructed for containing the impregnant which is then introduced into the impoundment after the surface temperature drops to 50^{0}C. This temperature range is necessary to optimize the opposing effects of temperature and evaporation of the mineral spirits on the viscosity, and hence, the penetration rate of the impregnant. The area is then covered with a plywood sheathing and tarpaulin for protection from weather. Impregnation depths up to 100 mm can be achieved by this technique. Corrosion potentials of actively corroding reinforcing steel in salt contaminated concrete slabs are considerably reduced by this method.

Selection of an appropriate method of repair depends on the amount of the chloride content and the electrical resistance of the deck concrete. This can be developed based on the local conditions. A flow chart relating these factors developed in Nebraska, U.S.A (1), is reproduced in Figure 2.3 for completeness.

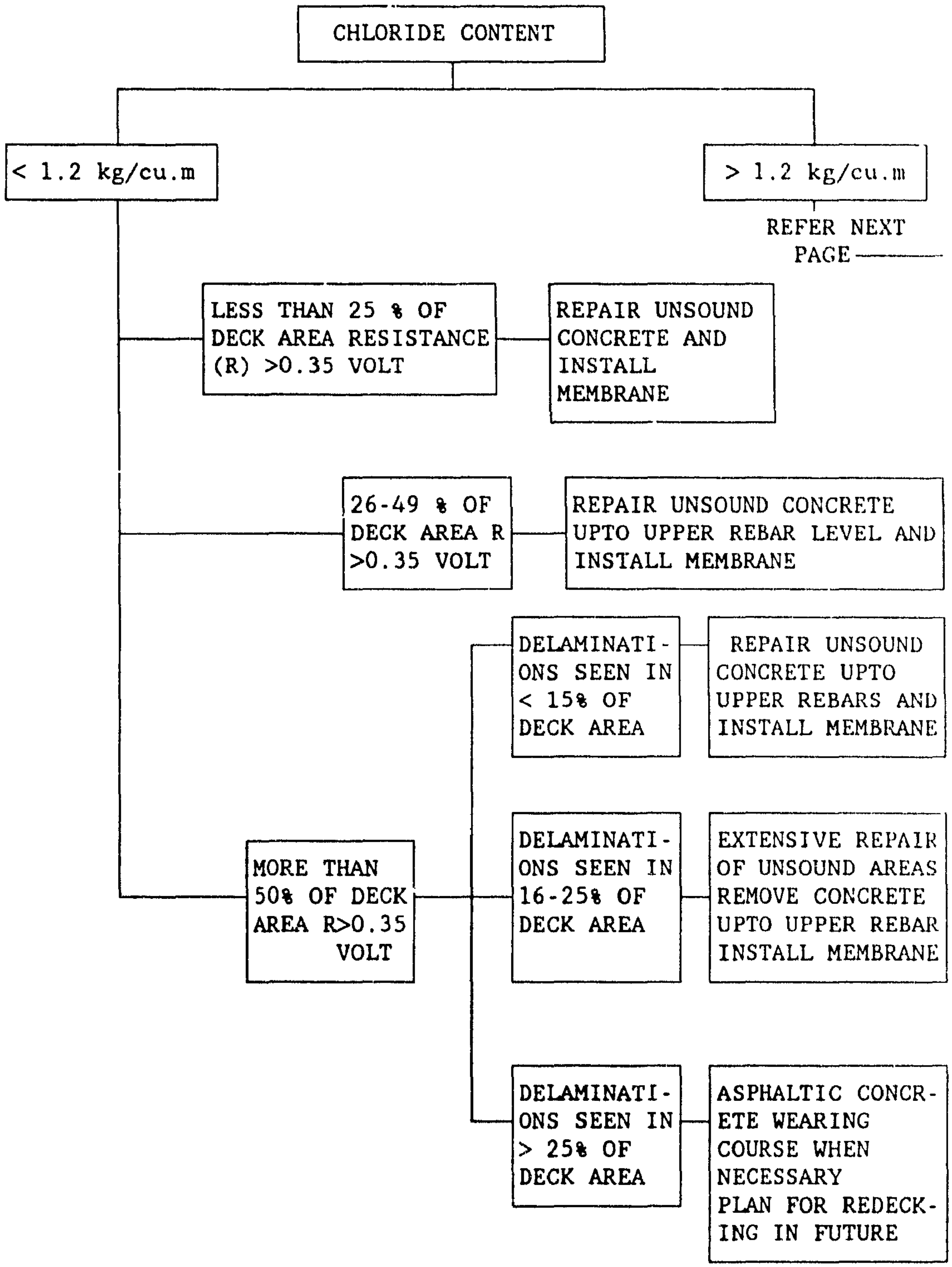


Figure 2.3 Repair of Decks Based on Chloride Content (1)

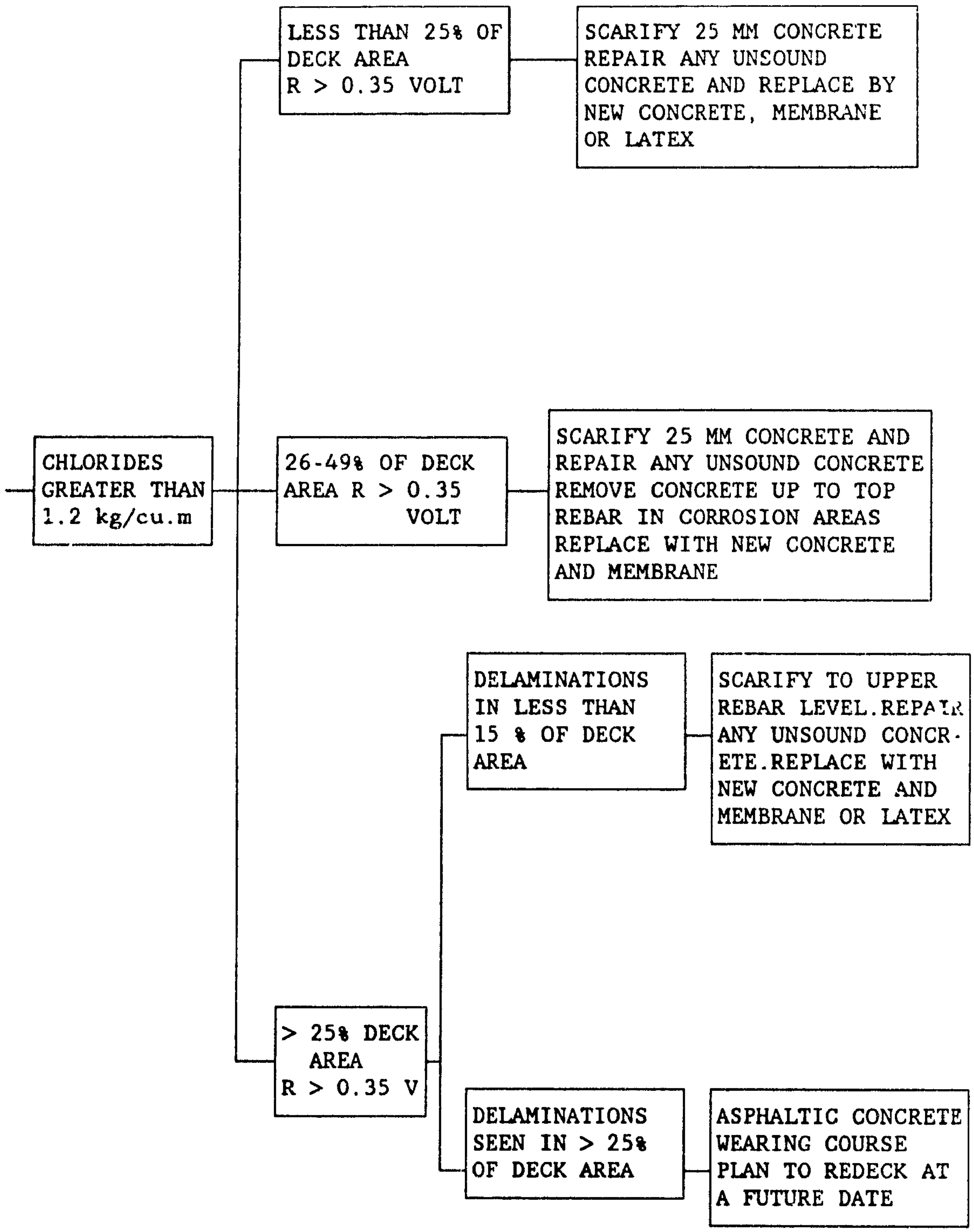


Figure 2.3 (continued)

2.1.4 Cracking

Concrete has low tensile strength. Also, it responds to changes in temperature and humidity by relatively large changes in volume. Thus, cracking of bridge decks can be caused by tensile forces which are in excess of the capacity of the concrete or by shrinkage during construction Cracking usually occurs after spalling and may be transverse, longitudinal, diagonal, or random

Cracks which appear at the time of construction due to plastic shrinkage or settlement of the falsework are usually fine and insignificant. Development of these cracks can be minimized by taking appropriate measures prior to and during construction, such as timely curing and proper quality control. These cracks develop basically because the rate of evaporation of the surface moisture exceeds the rate at which it is being replaced by bleed water which causes shrinkage of the surface while the underlying concrete remains of the same volume It is also observed that during early hydration, all of the cement paste shrinks thereby producing microcracks which grow in size with the increased rate of evaporation.

The other type of cracking which is of more concern is the pattern or map type cracking. It consists of interconnected cracks forming networks of any size and is geometrically similar to the type seen on mud flats. It results from the use of reactive aggregates and may occur after several years and increase in magnitude and intensity leading to total disintegration of the deck concrete The only solution in this case is to replace the entire deck.

Crack widths are estimated using microscopes or feeler gauges. It is also possible to evaluate the extent of cracking using pulse velocity techniques or by coring, if necessary. Structural cracks may depend on the design criteria and are difficult to repair especially when there is a significant crack movement. However, they can be minimized by controlling the strength of the concrete and also by taking care to ensure proper consolidation of the concrete around the reinforcement during construction.

Cracking is one of the factors which leads to corrosion of the reinforcement, the other factors being the depth and quality of the surface concrete. Cracks perpendicular to the reinforcing steel accelerate corrosion of the intercepted bars by allowing ingress of moisture, oxygen and chloride ions to the reinforcement. It has been observed that narrow cracks with widths up to 0.3 mm have little influence on the overall corrosion of the reinforcing steel. Wider cracks accelerate the onset of corrosion but over a period of years, the crack width has very little effect on the extent of corrosion. Cracks that follow the line of reinforcing bars need serious attention since not only is the corroded length of the bar roughly equal to the length of the crack, but the crack reduces the resistance of concrete to delamination.

Repairs

Localized cracks induced by traffic which open and close with the passage of traffic can be filled with a flexible joint sealing compound after routing. However, if the cracks are spread over the entire deck, they can be sealed by applying a polymer or bituminous concrete overlay with a waterproof membrane It is advisable not to apply overlays on decks that are exposed to freezing climates, because the overlay acts as an impervious vapour barrier which causes moisture transmission from the subgrade to condense under the barrier, leading to critical saturation of the concrete and rapid disintegration under cycles of freezing and thawing. Cracks due to delaminations in the deck can be repaired with epoxy resins and other types of adhesives.

2.1.5 Wear and Polishing

Wearing of concrete occurs due to constant friction against the vehicular traffic. It can cause loss of skid resistance because of aggregate polishing. Differential wear in the wheel tracks usually occurs when heavy trucks and automobiles with studded tyres or chains are permitted. This can cause ponding of water leading to deterioration of concrete.

In order to develop skid resistance, a bridge deck must have adequate microtexture and macrotexture. A sharp microtexture is necessary to provide friction between the tyre and the deck surface, while a deep macrotexture is necessary to provide channels for escape of water from beneath the tyre and deck surface thus preventing hydroplaning.

Following are the commonly used techniques for improving the skid resistance of existing bridge decks:

1) Tinning

In this method, grooves of approximately 3 mm $\times$ 3 mm deep and spaced at 20 mm centres are formed on the deck by dragging a tinning device over the surface of the plastic concrete. Tinning is the most economical technique for increasing microtexture, but its use is limited to new decks or overlays that are constructed with non-polishing aggregates and with mixtures and construction procedures that provide for good surface workability at the time the texture is applied.

2) Sawcut Grooves

The disadvantage of tinning operation is that it delays the application of the curing materials which may cause cracking due to plastic shrinkage. To prevent this, the grooves can be sawcut into the surface of decks and overlays after curing operations. The advantage of sawcut grooves is that the deck can be opened to traffic immediately. However, it is necessary to examine the deck condition for adequate cover before making the grooves. If the cover to the reinforcement is not adequate, a layer of bituminous concrete, polymer concrete, or plain or modified Portland cement concrete is generally needed. If a bituminous concrete overlay is to be used, a waterproof membrane should first be applied to the deck surface This is because bituminous concrete allows chloride ions to penetrate in the deck which may cause progressive deterioration and eventual failure of the slab. The thickness of the overlay depends upon the capacity of the bridge to carry additional dead load and is around 35 mm to 50 mm for a bituminous concrete overlay while only around 10 mm for a polymer concrete overlay

3) Shotblasting and Sealing

This is an economical method of repairing deck surfaces constructed with non-polishing aggregates. Shotblasting of concrete may, however, reduce the cover over the steel bar. Also, since concrete upto a depth of 6 mm is removed by shotblasting, it may increase the permeability of the concrete to chloride ion penetration. This can be prevented by applying a penetrating sealer and curing for about an hour before opening the deck to traffic Shotblasting increases the microtexture by removing the mortar between the coarse aggregates which gives adequate bond strength and improved skid resistance. It also increases the microtexture by abrading the surface of the coarse aggregate. The advantage is that the deck surface can be opened to traffic once the sealers are tack free.

4) Latex Modified Portland Cement Slag Slurry

It has been observed that the microtexture of the concrete is lost gradually over the years as the coarse aggregates get polished due to traffic. A latex modified slag slurry can be placed on the deck to provide adequate surface texture after shotblasting. Brooms should be used to brush the slurry into the shotblasted surface and the slurry should be struck off and pulled forward with gauge rakes set to provide a 4 mm to 6 mm thick slurry. For increased skid resistance, slag should be used on the struck-off surface, and a liquid curing material should be applied to prevent the evaporation of water. The deck can be opened to traffic in two to three days.

Advantages of the slag slurry are increased cover over the top mat of steel reinforcement, high and economical microtexture compared to 50 mm thick polymer cement concrete overlays, 35 mm thick latex modified concrete overlays and 6 mm thick polymer overlays.

5) Multiple Layer Polymer Overlays

The overlay consists of two layers of epoxy, polyester methacrylate prime coat, or epoxy urethane and clean, dry, angular-grained, silica or basalt aggregate applied to the top of a Portland cement concrete deck to provide a 6 mm thick wearing surface. The polymer is applied with brooms or squeezed uniformely over the surface of the deck. Usually, within the first hour, a layer cures sufficiently to permit vacuuming of the excess aggregate prior to placing the subsequent layer.

The polymer cement concrete has an advantage over other deck protection systems because it can be constructed in stages during off-peak traffic periods. The first layer of resin and aggregates can be applied to a lane that has been closed and shotblasted, and after a minimum of three hours of curing, the lane can be opened to traffic. The second layer can be placed on the next day or night during off-peak traffic period.

An additional advantage of overlays such as latex modified slag slurry and polymer is that new decks can be constructed with polishing aggregates, thereby extending the diminishing supply of aggregates. Thin overlays can be applied to provide adequate skid resistance.

The application of a multiple layer concrete overlay is a more expensive technique for increasing the microtexture and macrotexture of hardened concrete surfaces constructed with polishing or non-polishing aggregates. The higher cost can be justified when additional protection against the infiltration of chloride ions is needed and a short lane closure time is necessary.

2.2 Condition Surveys of Existing Concrete Decks

Evaluation of an existing deck consists of carrying out a condition survey which may range from a quick overall survey to a detailed survey depending on various factors like nature of damage, type of repair, etc. In addition, a routine condition survey should normally be done at regular intervals of say every two years. A detailed discussion of the various condition surveys can be found in References 2 and 20.

The condition of a bridge deck in terms of cracking, delaminations, scaling and wearing can be evaluated easily by carrying out a visual survey. However, evaluation of a spalled bridge deck is much more cumbersome and necessitates carrying out a complete deck survey, because there are areas of delamination (i.e. where the overlying concrete has not been dislodged) in addition to spalled areas (where the overlying concrete has been removed).

A pachometer survey is normally carried out to measure the thickness of the concrete cover and its adequacy to protect the reinforcing steel from deterioration.

The determination of the chloride ion concentration is another important aspect of deck evaluation because its increase beyond a certain limiting value can cause corrosion of the reinforcing steel. The number of samples required for chloride ion analysis depends on the variation of the chloride ion content within the deck. The location is usually determined using a pachometer which avoids drilling through the reinforcing bars. Samples for testing are usually obtained by taking cores using drills. The testing involves determination of water soluble chlorides and the total chloride content (13).

In addition to determining the chloride content, it is equally important to determine the corrosion potential of the half cells (i.e. anodic and cathodic areas on the reinforcing bars) to detect areas of active corrosion. The reader may refer to the standard test procedure given in References 29 and 57.

Areas of delamination can be detected by carrying out a chain survey or more recently by acoustic methods. Chains are commonly used and they are usually two meters long with a 50 mm link made from 10 mm diameter steel. The chain is dragged along the surface of the deck from side to side in a swinging motion A dull sound is produced when delaminated area is encountered. The length of the chain in contact with the deck is shortened which helps in identifying the extent of delamination. Alternatively, a commercially available portable electronic device for detecting delamination called Delamtect mentioned in Section 2.1.2, developed by the Texas Highway Department in co-operation with the Texas Transportation Institute (37,38), can be used. However, it is less accurate as compared to the chain drag method.

All of the above tests are absolutely necessary for evaluating a deteriorated slab, because the affected areas may not overlap. Also, repair of spalls and separated areas is only a temporary solution. Additional patches may be needed in a short time as the areas of active corrosion progress and new corrosion cells develop between the steel in the patch and that in the adjacent chloride contaminated concrete. In any case, a way of permanent repair is to remove the effects of chloride in the concrete by application of a cathodic protection system.

2.3 Repair Techniques

Before carrying out any repairs, it is necessary, to evaluate objectively the damage to the deck. The damage may be due to faulty design, poor workmanship, wearing and polishing, scaling, spalling, or cracking. The type, nature and extent of repairs chosen depend on the above factors affecting the condition of the deck. This is the most difficult step and requires thorough knowledge on the part of the engineer. For example, if the damage is due to moderate exposure of an inferior quality concrete, then it can be repaired satisfactorily using a good quality concrete. However, if a good quality concrete is destroyed, then the situation becomes complicated since it requires either a very superior concrete or change in the exposure conditions.

Repairs to spalls which occur due to corrosion of reinforcing bars needs a careful evaluation of the entire deck. For example, if the deck has been exposed to deicing salts, the electrolytic conditions will change due to the application of the new concrete, the consequences of which must be considered before any repairs are undertaken.

Some repair techniques normally used are reviewed in the following sections:

2.3.1 Patching

Patching is generally undertaken to repair delaminated areas. In this type of repair, the deck is first surveyed by sounding in order to detect the delaminated areas, which are removed first by sawing their boundaries upto the level of sound concrete. The area to be patched and a band of at least 150 mm width surrounding it is dampened to prevent absorption of water from the patching mortar. After cleaning any exposed reinforcement, a bonding grout or a coat consisting of a 1:1 mixture of cement and fine sand is brushed on the exposed concrete. A premixed patching mortar is then applied and properly consolidated in place and struck off to leave the patch slightly higher than the surrounding concrete surface. In order to allow for some initial shrinkage, the patch is left undisturbed for at least one hour before final finishing. The concrete in the patched areas is cured for at least seven days. The concrete in patches well supported by the underlying deck is normally cured until the material has a compressive strength of at least 7 MPa before opening to traffic. For full depth patches, the minimum strength after curing should be at least 21 MPa (20).

The most common patching material used is the conventional Portland cement concrete. However, other types of patching materials like concrete containing accelerators, fast setting cements, polymer compounds and polymer concrete composed of polymer and aggregates are also used to provide rapid development of strength and allow the deck to be opened early to traffic.

However, it should be emphasized that patching is not a permanent repair. Often newly delaminated areas develop next to the patched areas. Therefore, patching can be used only as a temporary measure until more extensive repairs are performed. Moreover, patching can provide a substantial service with the subsequent installation of a waterproof membrane.

2.3.2 Epoxy Injection

This a temporary but effective method for repairing delaminated decks before more permanent repairs can be made. In this method, an epoxy resin adhesive is injected into the cracks. The procedure for repairing delaminated decks consists of the following steps:

1) Identifying the delaminated areas on the deck by a sounding hammer or by a chain drag

2) Sealing the leakage points within the delaminations with an epoxy paste

3) Locating the steel using the pachometer.

4) Drilling holes 50 to 75 mm deep between the reinforcing bars upto the bottom of the delamination, using hollow-stemmed carbide-tipped drill bits which are connected to a vacuum cleaner to clear the dust during drilling

5) Pumping epoxy into the delamination under an operating pressure of around 140 to 280 kPa depending upon the area and thickness of the crack and the viscosity of the epoxy. The two components of the epoxy resin should be brought together and mixed at the injection nozzle thus maximizing the pot life of the epoxy and minimizing the material wastage

6) Any excess epoxy is scraped off and the exposed epoxy is sprinkled with sand

The Transportation Research Board (20) has provided the following guidelines and suggestions for epoxy injection:

"Epoxy injection is effective and achieves good crack penetration The technique is suited to the repair of bridges in which delaminations have developed but have not progressed to open spalls, provided that the delaminations are free from dirt. It is also preferable that the concrete be dry The epoxies normally used for injection will tolerate the presence of moisture, although bond strengths are reduced. Epoxy injection is no more than a continuing maintenance method of extending the life of a bridge deck before permanent repairs are made, because it does not prevent the subsequent development of further delaminations in injected areas at a different depth below the deck surface. Epoxy injection should not be used prior to the installation of cathodic protection because the epoxy insulates the underlying steel from the cathodic protection circuit"

2.3.3 Electrochemical Method of Chloride Removal

This is a possible method of repair of chloride contaminated deck slabs. Application of an electrical potential gradient causes the chloride ions in the concrete to migrate through the bridge deck concrete and into an electrolyte contained above the bridge deck. The potential gradient is produced by applying a direct current of around 100 volts between the reinforcing steel and the electrode present in the electrolyte. The chloride ions are collected by an ion exchange resin present in the electrolyte at the deck surface. This prevents the evolution of chlorine gas and minimizes the corrosion of the anode. After the treatment, the electrolyte resin solution is pumped off and the resin is returned to the laboratory for regeneration. Laboratory tests (20) have shown that calcium hydroxide solution (0.1 N) can be used as a suitable surface electrolyte, platinized titanium constitutes an optimum anion-exchange resin to capture chloride ions emerging from the concrete. Figure 2.4 shows the set-up necessary for the method.

Actual tests have shown that this test is capable of removing upto 90 % of the chloride ions from above the steel and from the concrete immediately adjacent to the steel. However, its overall efficiency is low due to the presence of the negatively charged hydroxyl (OH) ions present in the concrete. Also, large temperatures of around 200°F (93°C) are generated in the concrete which may lead to cracking. At the same time, it is expensive in terms of the equipment needed to treat a full-size deck slab.

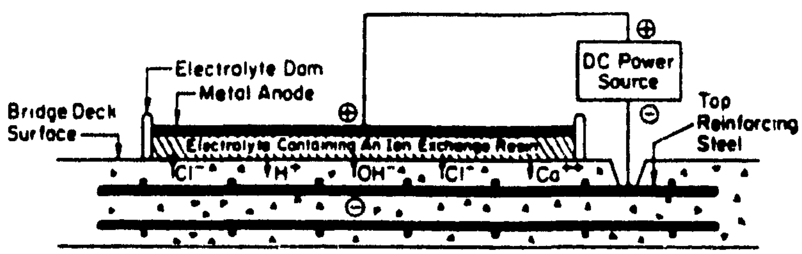


Figure 2.4 Electrochemical Method Set-Up (20)

2.3.4 Cathodic Protection

As mentioned earlier, corrosion of steel is an electrochemical reaction. During the corrosion process, numerous electrochemical cells are formed on the surface of the steel. At the anodic areas, oxidation reaction takes place and ferrous ions are released, while at the cathodic locations, a reduction reaction takes place and the electrons are consumed. The fundamental theory of cathodic protection of steel in concrete is to apply a direct current in a direction such that the corroding anodes on the steel are prevented from discharging ions, i.e., instead of behaving as current discharging anodes, they behave as current discharging cathodes. In short, cathodic protection consists of making a metal cathodic so that it does not dissolve.

This can be accomplished by making the half cell potential of all of the steel more negative than the most negative of the anodes. Stratfull (59) has recommended that the half cell potential of the steel should be not less than -0 85 volt for a satisfactory cathodic protection. However, according to Vrable (64), corrosion can be halted at a half cell potential of -0 77 volt and hydrogen gas bubbles begin to form at steel surface at -1.17 volt in a high pH solution. It can therefore be concluded that for an effective cathodic protection, the absolute range of values of potentials vary between the above values

Cathodic protection can be applied in two ways: by using sacrificial anode approach and by using impressed current, i.e., either by connecting the metals to the sacrificial anodes and forming a galvanic system which generates its own current or by applying a current from a rectifier using inert anodes.

Sacrificial anode approach involves placing of metal electrodes (commonly of zinc or magnesium) which are higher in the electromotive series than the reinforcing steel. These metals are placed in the deck over the top layer of reinforcement and act as sacrificial anodes when a current is applied between them and the steel. The only limitation of this method is that the sacrificial anodes have a low driving voltage, and, since the resistivity of the concrete is very high, numerous anodes are required. Also, overprotection is not possible and this system is maintenance-free until all of the anodes are consumed.

Impressed Current Cathodic Protection

An impressed current cathodic protection circuit is illustrated in Figure 2.5. High-silicon cast iron or graphite anodes are placed at several points in a conductive bituminous layer on the deck surface to obtain an even distribution of power over the deck. The conductive layer is composed of a mixture of coke aggregate, asphalt, stone and sand and it is covered with a wearing course of asphalt concrete. The impressed current is provided using batteries or a DC rectifier operating on an AC line voltage. A control panel regulates the voltage and the current for each anode, and it is located generally beneath the bridge. The flow of the current takes place from the rectifier through the coke, and then down through the concrete bridge deck to the reinforcing steel and back to the rectifier.

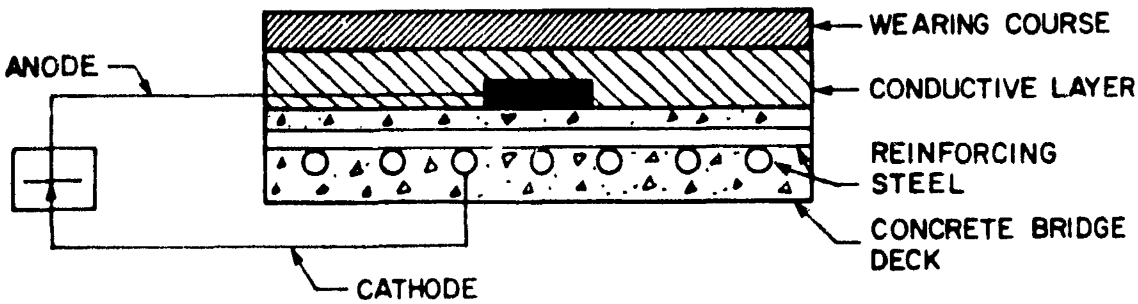


Figure 2.5 Impressed Current Cathodic Protection Circuit (20)

There is danger of overprotection with the impressed current system which necessitates periodic monitoring to ensure that the polarized potential of the steel is maintained within the maximum prescribed limit usually of -1.1 volt (64).

An important precaution to be taken in the method is to insulate the coke mixture from any bare reinforcing steel, deck scruppers, expansion joints, and the like to prevent a direct short circuiting of the bridge steel. This is done by covering the exposed bars with epoxy and surrounding the deck scruppers and expansion joint assemblies with conventional, non-destructive bituminous concrete.

Recently, electrical hardware has been installed such that the wearing course and conductive layer can be replaced without damage to the electric circuitry. The electrical hardware is protected by recessing the anodes and the voltage and the corrosometer probes in the deck surface. The wires for the hardware are also recessed and carried in sawcuts to the curb where all of the wires are cast in a concrete strip along the base of the curb face.

Cathodic protection has a distinct advantage in its ability to halt the corrosion process without removing the chloride contaminated concrete. However, consultants are needed for installation of the system. It is important to note that decks to be treated by this method should not be repaired by epoxy injection as this insulates the steel. Also, it is inadvisable to use polymer bonding agents and polymer concrete patches in repairing the deck prior to installation of a cathodic protection system. As mentioned earlier, periodic monitoring of the polarized potential of the steel below the prescribed limits is necessary. Other disadvantages include possible debonding of the overlay and the need for expertise in design, construction, inspection and monitoring of the system.

Cathodic protection is an economical repair procedure although its cost is variable due to the availability of the coke aggregate mix and the willingness of the local batch plants to produce the mix during their normal operations.

A case study of cathodic protection for the Fourth South Viaduct of Salt Lake city, Utah, U.S.A. is presented in Chapter 8.

2.4 Deck Repair and Protective Systems

Deck slabs are frequently subjected to chemical attack by water, acids, alkalies, deicing salts, organic chemicals, etc. The resulting damage may vary from a mere surface roughening or discoloration to a catastrophic damage resulting in the loss of structural integrity. Also, the deck concrete is not totally impermeable to water and may develop cracks after its placement. It is therefore necessary to carry out periodic repairs of such decks by using protective barrier systems such as waterproofing membranes and overlays.

Some of the commonly used deck protective systems are described in the following sections, describing their relative merits and demerits.

2.4.1 Concrete Overlays

This is an expensive but permanent repair procedure for repairing deteriorated deck slabs and is also used for preventive maintenance of newly opened decks without a deck protective system. Concrete overlays have several potential advantages. Their composition can be suited to give the required thickness, durability and resistance to chloride ion penetration to the deck concrete. They prevent build up of vapour pressure beneath the concrete and improve the wear and skid resistance substantially. Furthermore, they fill in the areas of concrete removal, therefore there is no need for a separate placing operation. This can help keeping part of the deck open to traffic during the repair work.

Various types of materials have been used successfully as overlays. These include Low-Slump Portland Cement Concrete, Polymer Modified Concrete and Internally Sealed Overlays. These are reviewed briefly in the following sections.

2.4.1.1 Low-Slump Portland Cement Concrete Overlays

The system of using a low-slump concrete as a repair material was first developed in Iowa, U.S.A., and is frequently referred to as the **Iowa Method for Bridge Decks**. This method utilizes a high cement factor (5 kg/m^3) and a very low water cement ratio (about 0.32), such that a slump of only 25 mm or less occurs. Also, the deck should be dry prior to grouting. The following are the essential steps (20):

a) Removal of the existing deteriorated concrete.

b) Scarifying the concrete surface to remove around a 6 mm layer; removal of all contaminants such as linseed oil and oil droppings from the surface.

c) Sand blasting or water blasting the concrete surface and the exposed reinforcing steel one day prior to placing the concrete to remove the rust from the surface of all exposed reinforcing bars and also any particles loosened or cracked during chipping or scarifying.

d) Scrubbing a mortar binding agent, consisting of a 50/50 mixture of sand and cement in the form of a stiff cream, onto the deck surface at a controlled rate to obtain a thickness of about 50 mm.

e) Finishing of concrete where necessary to close up any irregularities on the surface followed by transverse grooving to obtain the required skid resistance properties.

f) Curing the concrete for at least three days by covering it with wet burlap to provide a cooling effect resulting from the evaporation of water. For the first day, the burlap should be kept continuously wet after which the deck can be covered with a waterproofing covering such as a polyethylene film to hold in the moisture. Water is supplied to hydrate the cement because of the low initial water content of the concrete and it prevents shrinkage cracking. A minimum compressive strength of 21 MPa must be satisfied before the bridge deck can be opened to traffic.

The advantage of low-slump overlays is the use of cheap materials. However, placing operations are tedious and require specialized equipment. Also, for long lasting effects, good quality control and inspection procedures are necessary.

2.4.1.2 Polymer-Modified Concrete Overlays

The durability and bonding characteristics of concrete can be improved by adding polymers during the mixing operation. Polymerization is normally implemented before mixing. When the polymerized emulsion is mixed with the fresh concrete, the water of suspension in the emulsion hydrates the cement. The polymer enters the structure of the concrete and provides supplementary binding due to its adhesive and cohesive properties. This type of concrete, called latex-modified concrete, is nothing but conventional Portland cement concrete with approximately 15 % latex-modified solids by weight of cement. Styrene-butadiene latexes are commonly used Other types of synthetic latexes in use are polyvinyl acetates, acrylics, and vinylidene chlorides. It is necessary to stabilize the formulated latex modifier during manufacture to be compatible with the alkaline environment of Portland cement and to inhibit formation of excessive air in the composition. It is also necessary to select the particular latex modifier based on anticipated service conditions since it has a direct effect on the strength and durability of the concrete (4). The reader may refer to ACI Committee 546 Report, Section 5 (4), for material properties and quality standards pertaining to the latex-modifiers, Portland cement, aggregates, and the latex-modified mortar. The method of repair using this type of concrete is similar to that using low-slump concrete. However, the following principal differences in the repair procedure should be noted (20):

1) The deck must be kept wet for at least one hour prior to placing the overalay.
2) A separate bonding agent is not always used.
3) The mixing equipment must have a means of storing and dispensing the latex into the mixture
4) The latex-modified concrete has a high slump and is not air-entrained.
5) Conventional deck finishing equipment may be used.
6) A combination of wet and dry curing for at least four days is required.
7) The thickness of the overlay is usually slightly less than for low-slump concrete.

Latex-modified concrete overlays are preferred to low-slump overlays primarily due to the following advantages:

1) The initial wet curing hydrates the Portland cement and prevents formation of shrinkage cracks whereas the drying process dries the latex which then forms a continuous film within the concrete thereby giving the concrete a good bond, flexural strength and resistance to penetration by chloride ions.

2) Although it uses expensive materials, it requires less manpower and can be placed easily by using conventional equipment.

3) The thickness of the overlay required is less than that for low-slump concrete.

2.4.1.3 Internally Sealed Concrete Overlays

This is a type of polymer-modified concrete in which fusible polymeric particles consisting of small wax spheres are added to the concrete at the time of mixing. The mixture is heated to about 185°C which causes the additive to melt and flow into the micropore structure of the concrete, effectively sealing the concrete against the ingress of moisture and chemicals. Heating is usually provided using infrared heaters or an electrical blanket system. The deck temperatures are measured by placing thermocouples in the concrete.

The overlay can be installed in two ways: by applying a bonding agent and placing the overlay on hard concrete, or by placing the overlay and concrete with a time lag of around 30 minutes. After placing, moist curing is required for a minimum period of seven days. Also, heating should normally be undertaken after at least 21 days of placement.

Although internally sealed overlays provide good protection to the reinforcement, their use is still in an experimental stage due to problems such as the use of costly heating and cracking adjacent to the curbs or barrier walls due to the heating operation.

2.4.1.4 Polymer Concrete Overlays

Polymer concrete overlays are normally used for repairing deteriorated slabs. Polymer concrete consists of one part of thermosetting epoxy resin together with four to seven parts of silica sand aggregate by weight. The method of batching and mixing can be found in Reference 4. Useful properties of the resin include low viscosity and a high degree of resilience. Also, the resin is 100 percent polymerizable. Research has shown that polymer concrete overlays offer low permeability to corrosive substances; they are long lasting, skid resistant and provide excellent bond to the concrete substrate. They can be installed quickly during off-peak traffic periods.

Polymer concrete overlays are applied in thicknesses of around 12 mm. Prior to applying the overlay, all major spalls must be repaired and the concrete deck should be shotblasted by a machine equipped with a dust collector which not only recycles the steel shots but also collects the concrete cuttings thereby cleaning the deck.

The following methods of application may be used.

1) A continuous mixing-dispensing equipment is used which lays down a catalyst-promoted mix of resin and sand on the deck in one pass. A levelling equipment is then used to distribute the overlay evenly and finish the surface.

2) Another method of application involves applying a uniform coat of polyester resin to the deck

surface followed immediately by spreading of sand aggregates over the uncured resin. After the resin has cured, the unbonded sand is removed and the process is repeated until four layers of resin and sand have been applied.

The deck can be opened to traffic within approximately three hours after laying of the overlay. Advantages of polymer concrete overlays include low initial cost, improved skid and wear resistance, and durability of up to at least ten years.

Polymer concrete can also be successfully made with methyl methacrylate monomer (MMA) with an initiator and a promoter to polymerize the system in place. Some details of typical monomer systems are provided in Reference 4 which also provides detailed guidelines regarding the aggregate properties and the method of mixing and batching. If user-formulated monomer systems are used, one simple method of mixing and placing is to place premixed aggregate into the repair hole and pour the monomer over the aggregate. Alternatively, the aggregate and the monomer can be premixed before placement.A prepackaged system can also be used, in which case mixing can be done either by hand or by a machine followed by placing the material directly into the repair area. It is then consolidated and finished. Advantages of using a MMA system are its low viscosity, high bond strength to concrete and low cost. Also, curing is rapid at ambient temperatures and compressive strengths of 35 MPa or more can be obtained within two hours of placement.

2.4.2 Waterproofing Membranes

Waterproofing membranes are used to prevent leakage through concrete under service conditions, temperature extremes, vehicular loading, aging, and for crack bridging. Although the various protective systems developed recently such as epoxy-coated bars and concrete overlays offer alternatives, waterproofing membranes are still used extensively in rehabilitation work.

An ideal waterproofing membrane should in addition to preventing leakage, be easy to install, should bond effectively to the substrate and the wearing course, and should be compatible with the substrate, protective layer, wearing course, adhesives and prime coat (20).

The membranes used recently fall into two categories: preformed membranes of sealer and reinforcement type which are unrolled and lapped on the deck surface, and hot-and-cold-applied-in-place liquid systems. Both systems possess excellent stress-relief and crack-bridging capabilities. Surface preparation is much simplified and although much cleaning is required, sand blasting is not always needed. It has been indicated from laboratory screening tests that preformed membranes are much superior to applied-in-place materials in terms of resistance to permeability, crack bridging capability, bond to concrete, and durability under service conditions. However, applied-in-place materials do offer substantial protection against ingress of chlorides even when pinholes and bubbles occur in the coating.

Most of the membranes cannot be used directly as the riding surface of the deck and therefore require an approximately 50 mm thick asphaltic concrete wearing course to provide durability under traffic. Thetefore, the structure must have sufficient reserve strength to carry this additional dead load. Most membranes also require the use of an intermediate protective layer like roofing felt or asphalt-impregnated protection boards around 3 mm thick between the membrane and the wearing course to prevent damage during installation of the hot mix and also to resist puncturing of the membrane by aggregate particles under service conditions.

In practice, there have been several incidents of slip between the deck, membrane and the wearing course. Because of this tendency to slip, membranes should not be used on grades greater than 4 percent, or in areas subjected to rapid acceleration, deceleration, or turning movements.

Before placing membranes on a deteriorated deck, it is necessary to remove all chloride contaminated concrete otherwise the corrosion activity may continue even after their placing. Care should also be taken after placing the membranes to prevent leakage at curbs and at expansion joints, either by placing the membrane up the face of the curb or the joint, or by forming a sealed joint at these locations. Also, vertical slots in the deck drains should be provided for drainage from the surface of the membrane.

Common problems associated with decks provided with waterproofing are blistering and blowholes. Blisters are caused by the expansion of air trapped beneath the membrane or from water vapour pressure developing at the interface between the deck and the membrane due to moisture ingress through the deck and usually occur in areas of poor adhesion. Blowholes may develop usually at the time of installation of applied-in-place membranes. Blisters can be prevented by ensuring that the deck temperature is higher than the ambient temperature at the time of placing the membrane and also where applicable, during the curing period of either the membrane or its adhesives. This can be achieved by applying a black prime coat to the deck which is allowed to heat naturally upto the maximum deck temperature prior to placing the membrane. Blisters may also be prevented by providing venting layers beneath the membrane. A venting layer may be either a perforated sheet of bituminous felt, an open-weave glass or polypropylene fibre, aluminium sheathing, etc, provided to disperse the vapour pressure. However, these are expensive and are still in an experimental stage.

Waterproofing membranes have shown mixed response in numerous installations all over the United States and Canada. Despite the conflicts, the general view has been satisfactory for some types of membranes. However, they are less preferred as compared with latex-modified and low-slump overlays.

2.4.3 Sealants

As mentioned previously, sealants are used primarily for repairing surface scales. A common mixture consists of a 50/50 mixture of boiled linseed oil and mineral spirits applied in two coats after air drying the deck. Linseed oil treatment has been found to be effective in reducing the scaling of improperly air-entrained concrete. However, a depth of penetration of upto only 3 mm can be obtained and the life of treatment depends mainly on the type of traffic wear. Also, if an asphalt wearing course is placed, the linseed oil treatment reduces substantially the bond between the asphalt and the concrete.

Linseed oil treatment is very inexpensive, but it does not last very long and needs to be renewed regularly. Also, it does not reduce the corrosion of reinforcing steel. An alternative is the use of epoxies which are more effective for preventing deck deterioration.

2.4.4 Impregnants

As mentioned earlier, in the case of sealants, the depth of penetration is limited to only about 3 mm. For durability, deeper impregnation is necessary, which can be achieved by using polymer impregnated concrete which is stronger, more durable, and impermeable to deicing salts.

The production of polymer impregnated concrete involves initially casting and curing the concrete followed by drying to remove all of the evaporable water. The concrete is then vacuum soaked in a low viscosity monomer under pressure. A widely used monomer is methyl methacrylate with 5 % by weight of a cross linking agent, trimethylopropane trimethacrylate. The monomer is then polymerized in the voids of the concrete carefully preventing its simultaneous evaporation. Polymerization is accomplished using chemical initiation which involves the use of organic peroxides as initiators. These decompose under the action of heat or a chemical promoter and generate chemical radicals, which cause the monomer units to join together to form a plastic.

For field applications, the deck is initially cleaned by sandblasting, followed by drying. A thin layer of dry sand is then placed on the deck surface after it has cooled. The sand is then saturated with monomer containing an initiator and a cross-linking agent. If necessary, the monomer is ponded in the sand by constructing dikes. The saturated sand is covered with polyethylene sheeting to prevent evaporation of the monomer, and the deck is allowed to soak overnight. If the sand becomes dry, more monomer is added.

Polymerization of the monomer is continued for two hours at a deck temperature between 60°C to 80°C, using steam, ponded hot water or forced air heaters. The water added to the sand inhibits evaporation of the monomer and bonding of the sand to the deck. Any unbonded sand is removed.

2.5 Redecking and Widening

2.5.1 Bridge Widening

In widening or rehabilitation of an existing structure, it is essential to account for several constraints that may affect the overall plan, and design and construction methods. Alignment may be finalized on the basis of the right of way, physical features or the proximity of interchanges. If traffic is to be allowed while widening, the volume and type of traffic should be taken into account in planning the modification, because the concrete in the bridge widening is more vulnerable to the effects of traffic induced vibrations than the concrete in an overlay, especially at the time of the initial set. Due to vibrations, the concrete in the widening of a bridge is subjected to similar problems associated with concrete in overlays. In addition, this concrete may also be subjected to longitudinal cracking and loss of bond with the embedded reinforcement. The negative effects of traffic induced vibrations have been a serious concern. A detailed description of these effects can be found in Reference 21.

There have been numerous suggestions to reduce the amplitude of traffic induced vibrations. These include, for example, use of separate piers for widening or by releasing the diaphragms while placing the widening to reduce load transfer on the newly placed concrete and reconnecting them before placing the closure pour. If existing detours, run-arounds, or temporary structures are available within the right of way, they can be used as barriers to keep the traffic out of the adjacent lane. The most effective way, however, is to maintain a smooth riding surface.

The recent experiences from several installations in the United States (21) can be summarized as follows:

"*1) Widening and new slabs should be attached to the existing structure. Moment transfer should be provided through the joint between the new and existing portions of the deck. Lapped reinforcing bars are preferred to dowels. The laps should be tied securely or welded. Dowels and reinforcing steel should be straight. A concrete keyway is not necessary.*

2) A closure pour is recommended to achieve a smooth surface when an overlay will not be placed on the deck.

3) Where the fascia beam of the existing structure differs from the other beams in either section or camber, it should be removed and used as the fascia beam in the new deck."

2.5.2 Deck Replacement

Replacement of a deck may be necessary in extreme cases of deterioration. In such cases, deck replacement may prove more economical than removing all of the concrete upto the level below the top reinforcement. A deck may be replaced using either cast-in-situ or precast concrete slabs. For cast-in-situ concrete deck replacement using staged construction while maintaining traffic, the same conditions apply as for concrete placed in bridge widenings.

It is important that concrete be placed properly in the construction or repair of a deck. Also, the concrete used should have low absorption characteristics. A low water-cement ratio (around 0.4) should be used with a minimum clear cover of 50 mm to the reinforcing steel. Lower values of water-cement ratio can, however, be used for low-slump and high density mixes. It is also necessary that the concrete be properly air-entrained and consolidated. Also, timely curing is essential to ensure a durable performance.

However, it is very difficult to evaluate the water-cement ratio and the degree of consolidation during the placing of the concrete, and therefore, protective features are often necessary. These may include membrane waterproofing systems and more recently the use of epoxy-coated reinforcing steel as the top reinforcing layer, the latter being preferred for reasons of durability and economy. Furthermore, a low permeability concrete is needed only in the portion of the deck above the top reinforcement while the bottom deck concrete should be of good quality and structurally strong. This can be achieved using a two stage method (46), whereby the structural slab is placed and screeded roughly at a level above the top reinforcing steel. After the slab concrete has gained sufficient strength, a wearing course or an overlay consisting of impervious high quality concrete is placed. The advantage of using the two stage procedure is that the quantity of the costly concrete in the top course is minimized, beyond which , quality control of the smaller volume of concrete is easier. Also, proper cover is more easily obtained because most of the dead load deflection has already occurred and the overlay load acts on the composite section.

Perhaps the most economical and popular solution currently being used is replacement using steel orthotropic plate decks. This has been illustrated in the case study of rehabilitation of the

Champlain Bridge in Montreal, Canada (Chapter 9) and explained in Appendix D.

2.6 Prefabricated Components for Replacement and Rehabilitation

Recently, prefabricated components are being used increasingly for repair and rehabilitation because they are economical and result in reduced closure time of the bridge. Three different methods of construction have been used. These are discussed below.

1) Stay-In Place Forms

This method involves use of thin, precast, prestressed members spanning between girders to act as stay-in-place forms on which the cast-in-place reinforced concrete deck is poured. The precast members are usually designed to behave integrally with the cast-in-place concrete and compositely with the girders. This can be achieved either by roughening the top surface of the precast girders or by projecting reinforcing bars from the surface or by using machine-made ribbed panels with ribs in the transverse direction of the bridge.

2) Full-Depth Precast Concrete Decks on Girders

Full depth precast concrete decks are employed mostly in situations where replacement of deteriorated concrete decks on steel or prestressed concrete bridge girders has to be undertaken. The deck may be covered later by an asphaltic wearing course or a high density polymer cement concrete overlay to prevent deterioration. The precast units are usually connected using standard grout keys and either an epoxy mortar or a cement-sand grout in the joint. This connection is necessary to provide lateral load transfer across the joint and also to prevent moisture leakage. The only disadvantage of this method is that it is costly and should be used only if speedy repairs are necessary.

3) Integral Deck Bridges

In these type of bridges, precast prestressed concrete units having integral decks and girders are used. The deck and the main supporting members may either be the same (such as a slab) or the deck may be cast as a single unit along with the supporting beam or girder. As in the case of full depth precast concrete decks on girders, the integral decks may be covered with a cast-in-place concrete deck or simply an asphaltic wearing course. Various types of sections for the integral decks have been used. These include solid and voided slabs, box beams, channels, angles or double tees, bulb tees, and multi-stemmed units.

Adjacent integral deck units are connected using shear keys with transverse ties for connecting the units laterally to prevent lateral displacement. The shear key is filled with grout which may be either a cement, sand and water mixture, or it may consist of a low viscosity epoxy mortar.

For tying box beams, slabs or channels transversely, bolts and threaded rods are placed in the diaphragms and these are tightened to a specific torque. In some cases, the units have been post-tensioned laterally for improved performance. In cases where stemmed units are used, it is common to use welded plates placed in the flanges of the units.

2.7 Replacement Systems

Widening of a Concrete Deck Bridge

Two methods for widening a bridge deck using precast concrete girders have been presented below. They can be used for widening steel, precast, prestressed concrete, concrete "T" girders, and conventionally reinforced and cast-in-place prestressed concrete box girders.

Method 1

In this method, the piers and abutments are widened first upto the desired deck width Prefabricated prestressed concrete or steel girders are erected for the widening. Formwork for the initial widening is also erected in place. The new reinforcing steel is placed in position and the concrete is poured in place to complete the initial widening pour (see Figure 2.6).

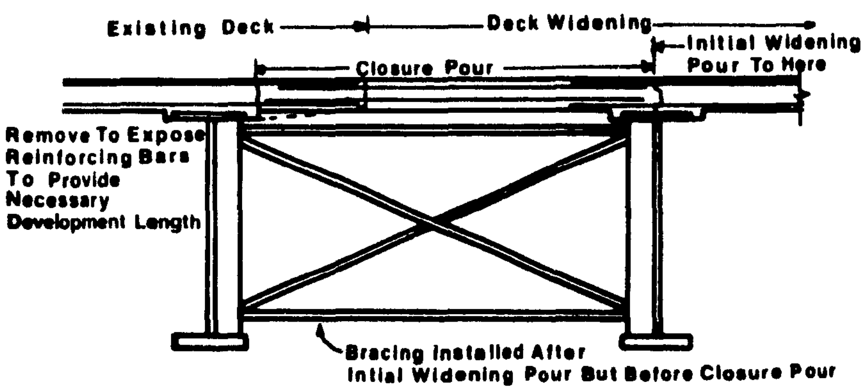


Figure 2.6 Deck Replacement-Method 1 (10)

The concrete in the old deck is removed to expose the necessary length of the reinforcing steel to lap with the new steel. Care is taken to install adequate lateral bracing between the old exterior girder and the new adjacent girder. The new reinforcing steel is attached to the original by lapping both the top and the bottom transverse deck steel. The closure pour is usually implemented three to fifteen days after widening if the bridge has steel or precast, prestressed concrete girders. However, if the bridge has been widened using cast-in-place girders, it is advisable to keep a gap of around two months. Moreover, traffic can be maintained on the bridge during the construction phase except on the lane nearest to the widening. The widened deck can be opened to traffic after the last concrete pour has attained the desired strength.

Method 2

The construction procedure is similar to Method 1 up to placing of the prefabricated girders. After placing the girders, the concrete from the old deck is removed to expose the old reinforcing steel. Adequate lateral bracing is then placed between the old exterior girder and the new adjacent girder. The new deck area is then formed and the new reinforcing steel is then lapped with the exposed transverse steel of the old deck at the top and the bottom. Concrete is then placed in the widened portion. See Figure 2.7 for details.

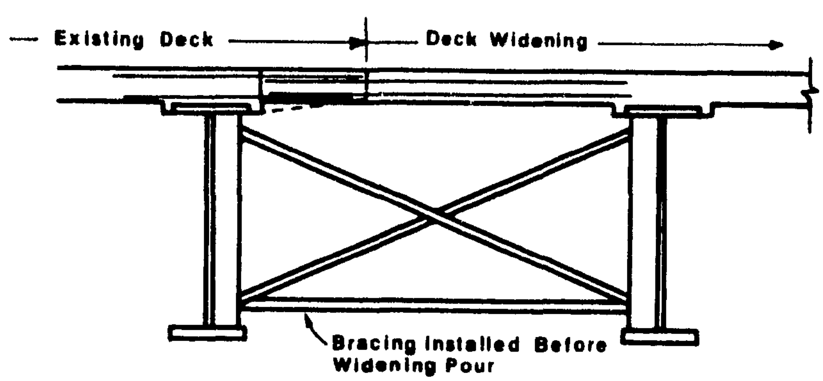


Figure 2.7 Deck Replacement-Method 2 (10)

Both methods use conventional lifting equipment, shoring and formwork. However, skilled personnel are necessary for welding purposes. Method 1 is generally preferred in practice even though it requires a time lag between placing the concrete in the widening and the closure pour.

2.8 Repairs using Hydrodemolition

Hydrodemolition, also known as hydrojetting, hydroblasting, or the water-jet technique was developed in the late seventies in Italy and since then, it has been used widely for repair of deck slabs. Most new rehabilitation projects specify this technique as an alternative to the traditional methods (47).

It is necessary during restoration of deck slabs to remove all traces of deteriorated concrete and also to avoid any damage to the sound concrete and restoration materials. Traditional methods, for example using pneumatic chipping hammers were not satisfactory since it was difficult to differentiate manually between good quality concrete and deteriorated concrete. This led to either incomplete removal of poor concrete or excessive removal of good concrete. This formed the basis of hydrodemolition technique which involves a selective removal of deteriorated portions of the concrete by means of one or more high-speed water jets. After hydrodemolition, the surface becomes rough and clean with no microcracks in the remaining concrete and no damage to the reinforcement which in fact gets cleaned from any rust on its surface (56). Hydrodemolition is not only a more improved method over the conventional methods, but some operations which were once impossible can be easily carried out and restoration is possible even in advanced stages of deterioration.

Mechanism of Hydrodemolition

It is known that concrete is a porous material since it consists of a mixture of aggregates (sand and gravel) and bonding agent (cement), along with gaseous inclusions. Due to the presence of pores, concrete is exposed to deterioration.

Hydrodemolition basically involves three separate mechanisms: direct impact, pressurization of cracks, and cavitation. All three mechanisms reach their peak efficiency when the water jet strikes the bonding agent. The water jet is forced rapidly over the area of concrete to be removed. The excess water is allowed to drain away. For achieving maximum efficiency and economic removal of deteriorated concrete, it is necessary to combine the fluidodynamic, geometric and kinetic parameters as a function of the existing deck situation (i.e. strength of concrete, presence of reinforcing steel, cracks, etc.) and the type of work required (56).

The equipment used, often called as **hydrodemolisher**, typically consists of a mobile unit called a **shrimp** which bears the high pressure water nozzles. The nozzles are mounted on a transverse track that allows a full width movement of about 1.8 m. The nozzles sweep back and forth on the track and the self-propelled shrimp moves in increments over the deck. High-pressure filtered water is supplied to the shrimp through high-pressure hoses from water pumps run by engines.

Modes of Operation

Depending upon different situations existing on the deck, there may be various modes of operation. Two of the main operations are summarized from Reference 47 for completeness.

The first case deals with quite widespread deterioration in terms of surface area, with depths varying from zero to the whole thickness and with potential involvement of rebars. Once the minimum thickness to which the repair material can be applied (e.g. 50 mm) has been established, a few square meters of sound concrete are identified. As a first step, strength of the concrete is determined from samples or, more simply, in-situ using non-destructive methods or pull-out tests. Working parameters are then fixed with the help of diagrams and tables, obtained from the previous tests carried out on a slab of predetermined strength.

An initial attempt is made on about one square meter and, if necessary, the parameters are redefined. The equipment is then moved to the area of worst deterioration (the ideal situation would be an area where deterioration involves the entire deck). Testing is considered successful if, with the same parameters as before, all of the deteriorated concrete is removed.

The second case often encountered is that of decks in relatively good condition but with insufficient cover, leading to delamination. In this case, hydroscarification (5 to 10 mm) of the entire surface is recommended. In this way, the upper part,possibly contaminated, is removed and, at the same time, excellent roughness is ensured for good bonding of repair materials. Hydroscarification also shows up possible areas which have undergone some degree of deterioration, but where delamination has not yet occurred. The boundary areas of the deteriorated parts are marked with regular geometric shapes, if possible, grouping several adjacent zones into a single patch. Deep removal is then carried out. It should be noted, however, that patching is never recommended, except for reasons of economy only, if the deteriorated areas do not exceed 15 to 20 percent of the whole deck.

Advantages of Hydrodemolition

Hydrodemolition offers many advantages over the traditional methods. Additional advantages may be derived by the type and quality of equipment used which influences the quality of repairs as follows:

1) The results are consistent and repeatable and there is guaranteed removal of all of the deteriorated concrete without any damage to the sound concrete.

2) If the top reinforcement is exposed by hydrodemolition, it can be thoroughly cleaned from any rust from the top as well as its lower parts which cannot be achieved by other processes such as sandblasting.

3) A very rough surface can be created which ensures perfect bonding to the repair materials

4) The equipment has very little vibrations and low operating noise levels as well as no formation of dust and fumes.

5) Simultaneous operations, such as casting immediately in adjacent areas, may be carried out

6) It is possible to carry out repair operations even in poor weather conditions and sub-freezing temperatures

2.9 Miscellaneous Deck Elements

These include curbs, sidewalks, and the railings. Metal curbs are designed either to allow water to drain under them or sealed to direct the deck drainage to scruppers. In the latter case, it is difficult to keep the seal watertight. Also, there might be loss of section due to corrosion or misalignment and damage due to collisions with vehicles. Furthermore, the anchor bolts used to fix the curbs to the deck may loosen due to vibrations and temperature changes All of these problems can be avoided by taking proper preventive measures such as epoxy painting and use of sealants to prevent leakage. However, in the event of section loss due to corrosion, a replacement is necessary. The use of cast-in-situ or precast concrete curbs is also quite common, however, it poses more serious problems of deterioration due to freeze-thaw cycles and deicing chemicals, spalling at expansion joints, cracking, and corrosion. In case of deterioration, the entire deteriorated portion should be removed, which in some cases may require removal of the entire curb. Cleaning of the area is performed by sandblasting followed by epoxy-coating of the old concrete surface. The fresh concrete is then cast in forms. In case of spalling of curbs at expansion joints, proper clearance is provided at the joints to accommodate the longitudinal movement of the span. In case of occurrence of cracks, epoxy injection should be used to seal them or if the cracks are large and involve damaged sections, the entire deteriorated concrete should be replaced.

Concrete sidewalks pose more or less the same problems as in the case of deck slabs and similar procedures can be adopted for repair (1). In case when steel plates or gratings have been used, it is necessary to regularly inspect their condition for any corrosion and section loss. Epoxy painting to prevent corrosion and complete replacement in the event of section loss are the commonly used repair methods.

Railings used on bridges are usually of either steel, concrete, aluminum, or timber. The most common problem associated is collision damage from vehicles which may need either repair of the damaged portion or replacement. Cracking or spalling in a concrete railing as seen in Figure 2.8 may occur due to corrosion of the reinforcing steel and should be repaired using procedures used for spalls which include epoxy injection of cracks, removal of damaged concrete, and replacing it with quick-setting concrete. Corrosion and loosening of anchor bolts is quite common with aluminum and steel railings and must be repaired by painting using hot-dip galvanizing. In case of aluminum railings, oxidation protection is required at contact surfaces between the aluminum railing components and the unlike materials to which the railing is attached. Similar problems as with aluminum railings exist for steel railings and the usual methods of repair can be employed.

Timber railings have also been used on decks and need to be checked for any decay, checking and splitting, damage to bolts, and paint condition.

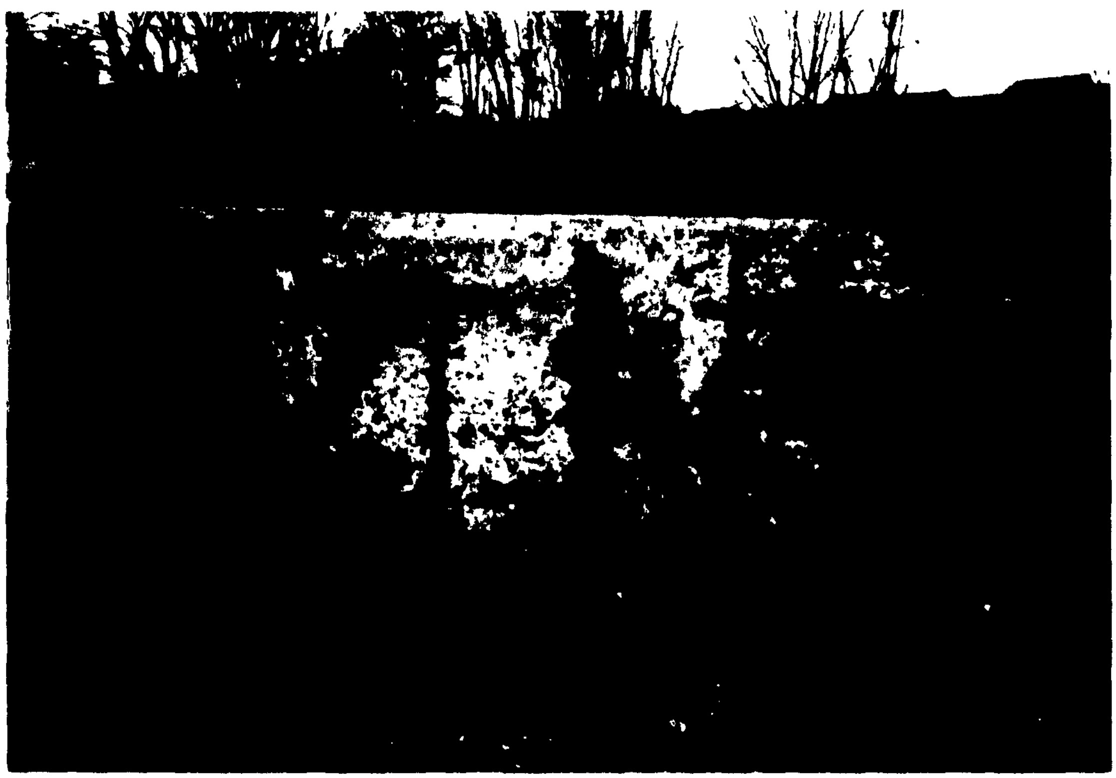


Figure 2.8 Corrosion of Steel in a Concrete Railing

CHAPTER 3
JOINT SYSTEMS

3.1 Introduction

Bridge movements are generally of the form of large-amplitude low-frequency movements in a horizontal plane, in the longitudinal direction, due to thermal expansion and contraction of the deck. Also, the movement of vehicles on the bridge causes small-amplitude, high-frequency horizontal, vertical and rotational movements (48). Creep and shrinkage cause additional longitudinal and transverse movements whereas any substructure settlement can cause additional rotational movements. These movements are normally accommodated by the provision of expansion joints.

Various types of expansion joints have been used depending on the type and magnitude of the movement. However, the related problems are also wide and common which can not only cause damage to the joint, but also to the portion of the bridge beneath the opening. The joint system exists in a very corrosive environment and rusting of the exposed steel portions could become a problem. In fact, more than fifty percent of the problems in concrete bridges are due to structurally deficient joints. A number of methods have been developed over the years for maintenance and repairs of joints like the use of joint sealants to prevent passage of deck drainage and debris, compression seals, membranes, etc. However, none of the methods have been able to solve the problems completely. Preventive maintenance is the best way to ensure efficient functioning of the joint and avoid more costly structural damage.

The various joint systems in use will be reviewed in this chapter. With the numerous and varied designs and materials developed and used over the years, it is virtually impossible to cover every specific system. The scope of this chapter is to categorize the various joint systems according to the type and to discuss the associated maintenance problems and the related correction and prevention needs.

3.2 Types of Joints and Associated Problems

Joints are classified broadly into two types: Open and Closed. Open joints are designed to permit free flow through the joint, while closed joints are designed to be sealed or waterproof.

3.2.1 Open Joints

Since open joints allow free drainage, in order to ensure their satisfactory performance, it is necessary to not only provide an effective drainage system, but also to regulate the flow of surface drainage through the joint and disposal of the runoff from the site. Troughs and drain pipes placed beneath the joint have commonly been used to carry the runoff away from the joint. A common problem with drainage troughs is that they eventually get clogged with debris and cease to function properly. The accumulated debris may spill over the seat below. Due to lack of space under the joint, it is very difficult to clean the troughs. Drain pipes also get clogged over a period or frozen in cold climates and need regular clean up to ensure proper functioning. Commonly used open joints include Butt Joint, Sliding Plate Joint and Finger Joint.

3.2.1.1 Butt Joint

A butt joint or armoured joint does not provide any transition for traffic between adjacent edges of the deck although a minor thermal movement of less than 25 mm can be accommodated. Therefore, it is used in cases where only rotation has to be accommodated. A typical detail is shown in Figure 3.1 and consists of a metal armour plate along the adjacent edges of the deck and secured to the deck by anchor rods. Where asphalt overlays are used, the armour is extended right up to the surface using a joint transitional dam. The plate serves a dual purpose of preventing the concrete surface edge from damage during construction or by traffic during service.

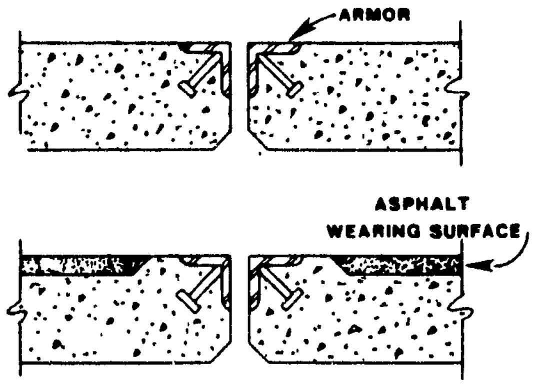


Figure 3.1 Butt Joint (8)

Commonly associated problems with this joint include corrosion of the metal armour and clogging of the joint with non-compressible materials which requires periodic maintenance including cleaning the roadway debris from the opening, painting and roadway surface repair adjacent to the joint

3.2.1.2 Sliding Plate Joint

A typical sliding plate joint as shown in Figure 3 2 consists of a horizontal flat steel plate anchored into the bridge deck along one edge and permitted to slide across an angle anchored to the opposite face of the opening This joint can accommodate movements between 25 and 75 mm.

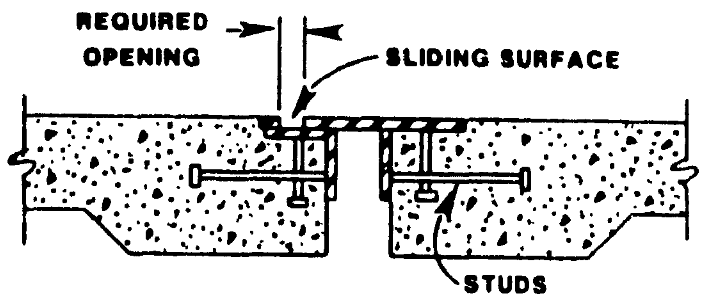


Figure 3.2 Sliding Plate Joint (8)

A common problem which can be attributed to inadequate consolidation of the concrete under and around the plates is the loosening of the plates. This creates a loud sound under the traffic and in the worst case the plates can dislodge from the joint thereby creating a hazard to the safety of the traffic. Other commonly occurring problems are corrosion of the anchors and fatigue under impact loads. Furthermore, the roadway surface around the plate can deteriorate thereby increasing the impact on the joint and dislodging the plates Clogging of the joint with debris is also a common phenomenon. Periodic maintenance and repair procedures common to butt joints can be adopted.

3.2.1.3 Finger Joint

A finger joint system can accommodate movements in excess of 75 mm. A typical detail as shown in Figure 3.3 consists of steel plates anchored on either sides of the deck. The opening between the adjacent plates is shaped in the form of teeth in plan. Finger joints have been installed using various shapes and configurations. However, all of them function identically in that movement is permitted with essentially no stress concentration at the joint face of the deck. Provision for drainage may or may not be provided. If used, it normally consists of a drain trough placed beneath the finger openings or a drain gate and a trough system on either side of the joint opening.

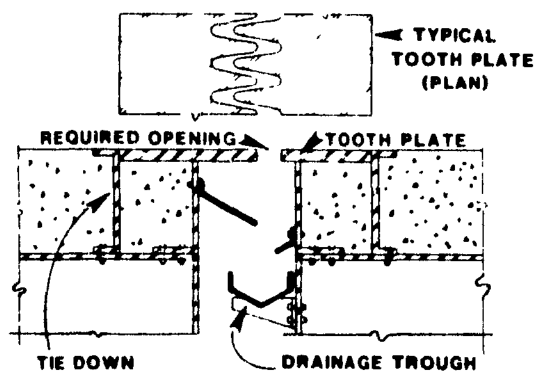


Figure 3.3 Finger Joint (8)

3.2.2 Closed Joints

As the name implies, closed joints are used to seal the surface of the deck, including curbs, sidewalks, medians, and in cases, barrier walls or parapet walls, against the ingress of moisture and debris through the joints. However, their geometry should be such as to prevent any accumulation of surface runoff and debris in the joint. Commonly used joints in this category include filled butt joint, compression seal, membrane joint and neoprene cushion joint.

3.2.2.1 Filled Butt Joint

A filled butt joint is similar to an open butt joint except that it is filled with a premoulded joint filler as shown in Figure 3.4. The concrete surface is power cleaned prior to placing the filler which is attached to one face of the joint or supported from below by an offset in the vertical face of the slab. The opening is then sealed by a hot-poured or a cold-poured sealing compound

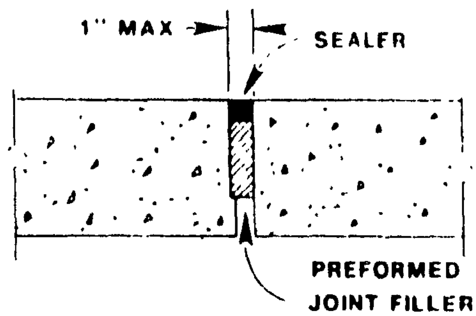


Figure 3.4 Filled Butt Joint (8)

A filled butt joint can accommodate movement up to 25 mm as in the case of an open butt joint and can remain watertight for about two years Problems with this joint include deterioration of the filler and ingress of non-compressible materials into the seal causing it to jam. Periodic maintenance is necessary which includes regular cleaning, replacement of the surface seal and filler as necessary and replacement of the roadway surface adjacent to the joint.

3.2.2.2 Compression Seal

A compression seal consists of a premoulded seal of neoprene or similar material which is squeezed into the joint opening. The seal expands after placing and is compressed with the joint movement. The width of a properly selected seal can cycle between 20 % (in warm temperatures) and 80 % in cold temperatures (31). An adhesive lubricant is applied to the sides of the joint before squeezing the seal into the opening. The adhesive provides for bond between the joint face and the sealant to produce a waterproof system. Single units can accommodate movements up to 65 mm while for larger movements, multiple units have been used. A typical single unit is shown in Figure 3.5.

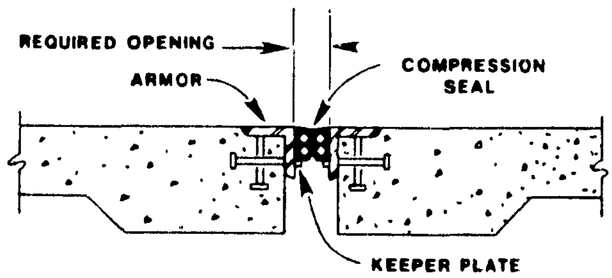


Figure 3.5 Compression Seal (8)

Compression seals can remain watertight for long periods of time only if selected and placed properly. If the size of the opening is too large, the seal can separate from the deck in cold weather, whereas if the opening is too small or kept very near to the surface, the seal can get damaged by the compressive forces in hot weather or by traffic as it pops out due to compression. Furthermore, seals have a tendency to return to their original shape after many cycles of compression after which they loose their watertightness as weathering and fatigue weaken the material. Periodic cleaning of the deck and approaches and roadway repair is necessary to ensure their proper functioning. However, in cases where leakage is detected, it is necessary to immediately repair or replace the seal.

3.2.2.3 Membrane Joint

A membrane joint consists of a U-shaped flexible sheet of neoprene rigidly attached to the two metal faces of the joint as shown in Figure 3.6. The neoprene can flex with the movement of the bridge upto ±150 mm. Larger movements can be accommodated using multiple units.

Although neoprenes are very effective against ingress of moisture, problems have been encountered usually at the gutter lines and areas where breaks in cross-sections occur. As the opening expands, non-compressible materials get lodged in the joint and get wedged in with the subsequent closure of the joint causing its rupture and leakage. Periodic maintenance including cleaning of the debris and sealing or replacing defective membranes is necessary.

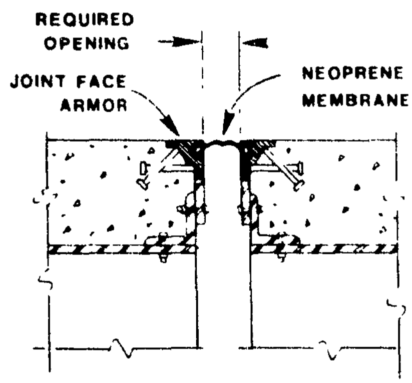


Figure 3.6 Membrane Joint (8)

3.2.2.4 Neoprene Cushion Joint

These joints can accommodate movements in excess of 100 mm. A typical detail as shown in Figure 3.7 consists of a reinforced neoprene pad rigidly anchored to each side of the joint. The anchorage is sealed with caps which are installed with an adhesive which is also applied between the cushion and the concrete to maintain watertightness. The neoprene pad expands or contracts as the joint opens or closes respectively. Reinforcing plates are used within the neoprene cushion to enable it to span the joint. However, as the bridge contracts, high tensile stresses in the neoprene pads may cause failure of the anchorage systems. Also, the adhesives used to maintain water-tightness can break down causing leakage and loss of caps. In areas where snowploughs are used, the joint can be torn apart from the support, or the cushion may get damaged thus requiring extensive repair or replacement of the joint. Maintenance procedures are similar to those for the membrane joint.

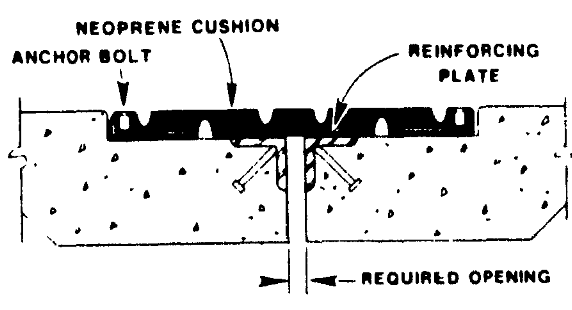


Figure 3.7 Cushion Joint (8)

3.3 Joint Problems-Prevention and Repair

Edge and surface damage and structural breakdown appear to be the most commonly occurring problems with deck joints. A discussion of these problems and the necessary repair techniques follows

1) Edge Damage

Damage to the concrete near the joint edges can occur due to one or more of the following reasons:

a) Inadequate curing of concrete,

b) Excessive pressure on the edges due to heavy steel-wheeled rollers or other equipment during or after construction,

c) Grade difference in the deck between adjacent spans, and

d) Absence of armour plates.

A typical repair procedure involves widening the joint by sawcutting and adding a compression seal. This repair should be done only if the width of the damaged area is narrow and the remaining part of the deck is in sound condition. If the joint needs to be recast, armour plates are added at the edges. Another repair technique is to sawcut the edges and grout them with cement or epoxy mortar.

2) Cracking and Raveling of Wearing Surface at Joint

Generally, bituminous wearing surfaces or overlays are placed continuously over the joints. In such cases, deterioration is frequently encountered in the form of cracking and raveling of the wearing surface at the joints. Initially, a fine crack originates in the nearside lane which propagates along the carriageway and subsequently progresses to multiple cracking and potholing. Also, debonding may occur at the joint in the wearing surface. It is advisable not to seal the cracks in such cases as it might cause new cracks, which may induce lateral cracking.

Where wearing courses or overlays are required, the joint should be redesigned to accommodate the change. Joint transition dams are used to elevate the joint opening to the level of the new surface. This is accomplished by placing the new surface ignoring the joint, and then removing the material over the joint. Finally, the top of the dam is installed to match the grade of the new surface.

3) Loosening of Joint Plates

As mentioned in Section 3.2.1.2, sliding plate joints have a general tendency of loosening of the plates from the anchoring system. In such cases, the opening must be redesigned to include a waterproof seal. This is accomplished by adding a lip to hold the seal in place, injecting epoxy to fill voids, or removing and replacing a portion of the deck around the joint.

4) Closure of the Joint

In the case of finger joints, excessive movement of the substructure may reduce the available joint spacing causing it to close in extremely hot weather or, insufficient provision for pavement expansion may cause joints to be closed by the roadway pavement exerting large forces at the bridge ends. Stresses can build up at the curblines and at the deck ends resulting in the concrete curbs and the deck crushing at the finger joint, or at an adjacent expansion joint. It is necessary in such a case to reverse the shifting of the substructure originally causing the closed condition. Otherwise, the expansion joints can be trimmed or the entire joint system can be removed, repositioned and reinstalled. If such a closure is due to pavement thrust, a pavement relief joint may be cut in the approaches as described in Section 3.5.3.

5) Deterioration of Joint Filler

Premoulded fillers have a tendency of deteriorating frequently causing them to become loose and fall off. A generally adopted solution to this problem is to remove a portion of the deck and to recast it to accommodate the joint armour and anchorage. A compression seal is then installed after cleaning the opening.

6) Cushion Seal Failure

In situations where cushion seals have been used, frequent failure of the anchorage systems can occur causing loss of major sections of the joint. The only solutions available in this case is to either install a new seal or replace the entire joint.

3.4 Results of Joint Inadequacies

A proper design, installation and maintenance procedure is necessary to ensure proper functioning of the joint. The problems that may be encountered due to joint inadequacies are discussed below.

1) Damage to End Diaphragms

End diaphragms are exposed to the moisture and salts leaking through the joints which may penetrate through the concrete and corrode the reinforcing steel thereby initiating spalling of the concrete and section loss of the reinforcing steel (see Figure 3.8). This can be avoided by providing a drip bed at the bottom of the deck between the joint and the end diaphragm.

Figure 3.8 Spalled Concrete and Loss of Steel Section

2) Deterioration of Bearings

Bearings can be exposed to moisture and debris leaking through the joints which if allowed to accumulate can cause corrosion in them. This can lead to freezing of the bearings which in effect transfers the stresses to other members of the structure that have not been designed to take these forces This can cause distress in members such as beams, bridge seats, or substructure elements. As a result, diagonal cracking in beams at the bottom at the bearing ends has been observed. These cracks propagate upward and backward and have also been observed in pier and abutment caps Frozen bearings can also cause transfer of movements to the other bearings, thereby, jamming the joints or causing them to open excessively Furthermore, cracking and tilting of substructure elements can occur and in case of skewed bridges, alignment of the superstructure can get altered.

3) Deterioration of Deck Soffits, Bridge Seats, Pier Caps and Columns

Accumulation of moisture, deicing salts and debris over the bridge seats and pier caps leaking through the joints can cause deterioration of concrete and initiate corrosion of reinforcing steel The top layer of the reinforcing steel expands due to the corrosive forces and lifts the concrete, leading to cracking. Discoloration of the sides of pier caps and columns can also occur, which may eventually cause spalling of the concrete. The underside of the deck slabs can also corrode due to leakage of moisture and salts through the joints. Typical examples of accrustation and cracking of deck soffits is shown in Figures 3.9 and 3.10, respectively.

Figure 3.9 Accrustation of Deck Soffit

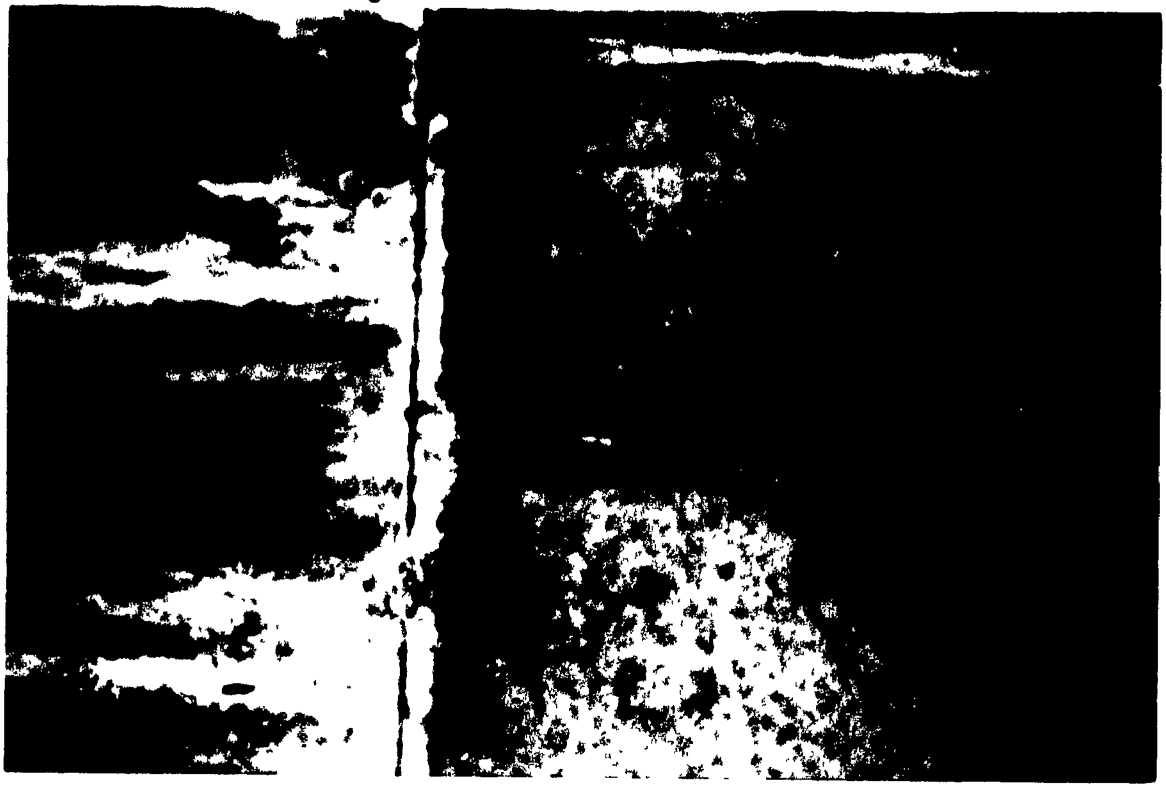


Figure 3.10 Cracking of Deck Soffit

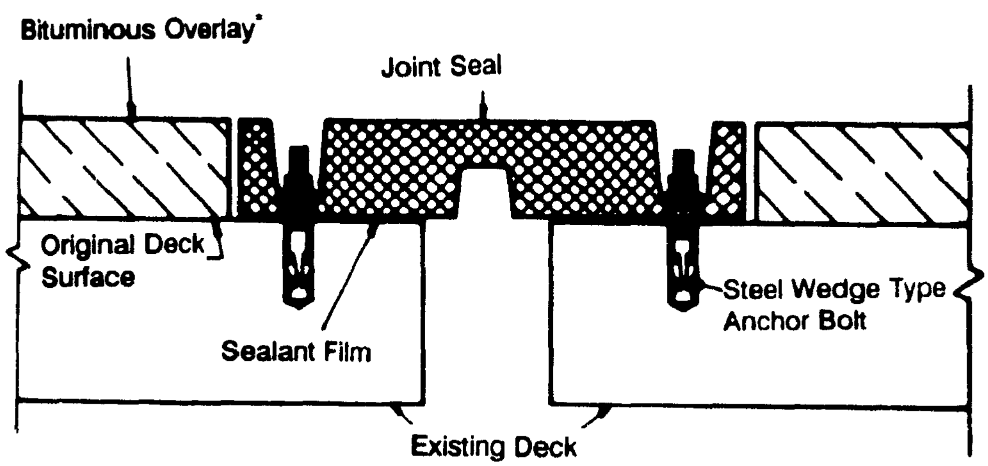
4) Embankment Erosion

Drainage through joints at abutments can cause erosion of the soil embankment which if not prevented can undermine the footing and expose the piles.

3.5 Typical Repair Procedures

3.5.1 Reconstruction and Modification of Existing Expansion Joints

Before using this system, it is necessary to first determine the size and type of system needed to accommodate the existing joint (10). The deck surface on each side of the joint is then prepared by removing the uneven areas. Anchor bolts are drilled on each side of the joint at predetermined depth and intervals. A sealant is applied to the deck surface to serve as bearing area for the joint. The joint sections are then installed and tightened securely by the anchor bolts Finally, the joint is covered with a bituminous overlay to give a smooth riding surface (see Figure 3.11).



* Other types of overlay systems such as dense concrete or latex modified concrete can be used.

Figure 3.11 Typical section of a Joint Sealant System (10)

3.5.2 Sealing of Sliding Plate Joints

This method involves installation of a premoulded compression seal in the existing joint The size of the compression seal and dam plates is determined first depending on the size of the opening The existing sliding plate is cut to clear the weld at the heel of the angle as shown in Figure 3 12 The dam plate assembly is then installed and welded in place. Finally, the compression seal is squeezed in and the joint is covered with a bituminous concrete overlay. The advantage of this procedure is that prefabricated dam plates and supporting strut plates can be used which enables the repairs to be carried out during traffic operations. However, this repair is only limited to joints having sufficient opening to accommodate the assembly. Also, the concrete should be sound to anchor the existing joint. More importantly, this repair procedure should be used only where the revised joint opening is adequate to accommodate the existing expansion, otherwise the repair procedure in Section 3.5.1 should be used.

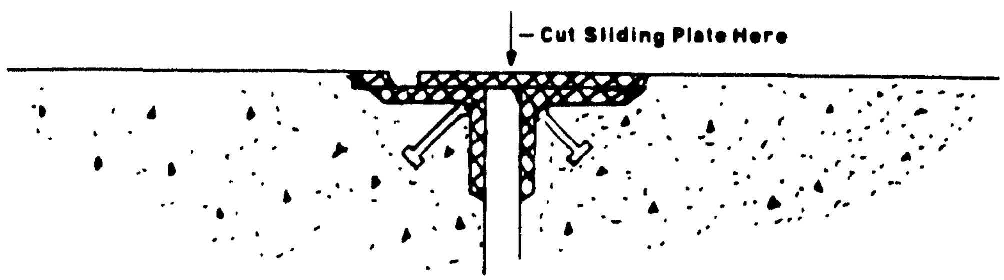


Figure 3.12 Typical Sliding Plate Joint Before Repair (10)

3.5.3 Installation of Pressure Relief Joint in Concrete Approach Pavement

This repair procedure has been commonly used to relieve the pressure on the abutment backwall due to expansion at the concrete approach pavement. The pressure relief joint is positioned beyond the bridge depending on the pavement condition (see Figure 3 13). The existing concrete pavement is removed The reinforced concrete slab is cast in sections using longitudinal joint key between pours Two layers of building paper are placed over the entire top surface of the slab followed by placing of an asphaltic concrete layer The deck surface is finally topped by a bituminous wearing course The concrete is cured sufficiently until it gains adequate strength before opening the bridge to traffic.

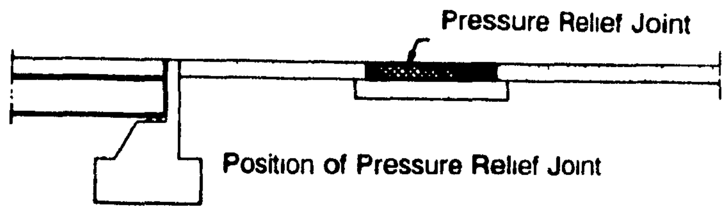


Figure 3.13 Details of Pressure Relief Joint (10)

3.6 Integral Conversion Techniques

An ideal approach to reduce the deck joint problems is to eliminate the deck joints wherever possible. The trend of the bridge industry in the current years has been to build continuous bridges with no deck joints unless absolutely necessary. This has been termed as integral bridge construction.

This technique has now been extended to retrofitting simple multiple span bridges to continuous bridges and non-integral abutments to integral abutments. In the case of superstructures, continuity introduces secondary stresses in the deck due to the response of continuous superstructures to thermal gradients, substructure settlement, post-tensioning, etc This is also the case with integral abutments in which secondary stresses are introduced due to the restraints provided by the foundations and backfill against the cyclic movement of the superstructure. However, it must be emphasized that for small and medium span bridges of moderate lengths, the damage caused by the secondary stresses is much less than that due to the use of deck joints. Also, economy is achieved due to the elimination of costly joints and bearings. The above advantages have made integral conversion techniques very popular in recent years. In North America, almost above 85 percent of the transportation departments are using these techniques The following discussion focusses primarily on the various techniques used for integral conversion of existing bridges

3.6.1 Problems in Jointed Bridges

Bridges built with joints at abutments are usually damaged due to pressure generated by jointed rigid pavements. The major problems are fracturing of the backwalls, splitting of the abutment, and in the case of bridges with intermediate deck joints, cracking and fracturing of the piers Use of deicing salts can also add to the problems. For example, in the case of open joints and sliding plate joints of short span bridges and open finger joints of long span bridges, deicing salts penetrate below the deck surfaces and accumulate over the supporting seats, bearings, and beams as mentioned earlier, causing corrosion of the reinforcing steel. This has necessitated increasing maintenance and repair to reduce the problems. On the other hand, bridges with integral abutments have suffered very little damage from pavement pressure. Also, they were unaffected by deicing chemicals and needed very little maintenance and repair.

3.6.2 Retrofit Procedures for Integral Conversions

A procedure used by the Texas Department of Highways and Public Administration is illustrated in Figure 3.14, involving removal of affected concrete as necessary to eliminate the existing armouring. Sufficient negative reinforcing steel is added at the required level near the deck top with an appropriate concrete cover to resist transverse cracking. The slab is then reconstructed with regular concrete to the original grade. Moreover, only the slab portion of the deck is made continuous whereas the beams remain simply supported. Also, one or both of the adjacent bearings supporting the beams at a joint should accommodate the horizontal movement in order to prevent any horizontal forces from being imposed on them due to rotation of the beams and continuity of the slab.

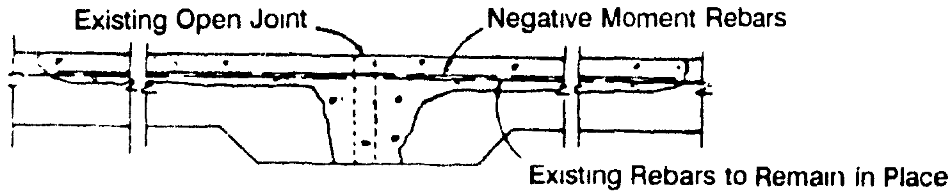


Figure 3.14 Addition of Negative Moment Reinforcement

A similar procedure used by the State of Utah, is shown in Figure 3.15. For deck slabs with a bituminous overlay, a waterproofing membrane can be used to waterproof the new slab section over the piers. However, in this case, longitudinal flexure cracks are induced in the slab due to rotat'on of the beam ends caused by the vehicular traffic. It is normally preferred to have cracks than the long term consequences associated with an open joint or a poorly executed sealed joint.

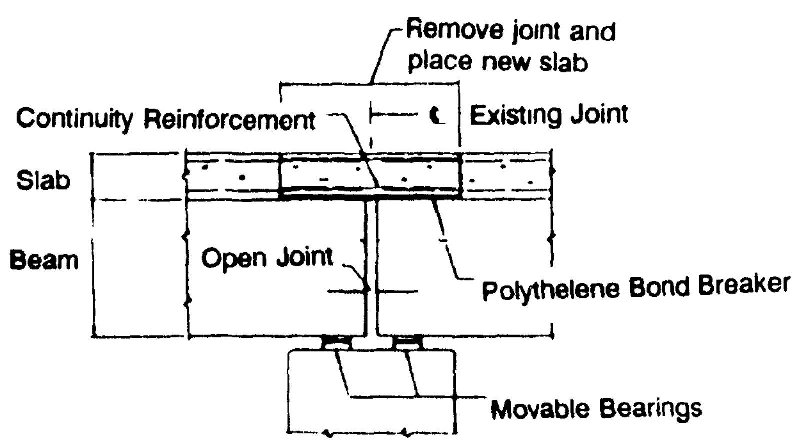


Figure 3.15 Provision of Continuity Reinforcement

A common procedure for converting simple spans to continuous spans for prestressed concrete I- beam bridges is shown in Figure 3.16. A substantial concrete diaphragm is placed at the piers between the ends of simply supported prestressed beams of adjacent spans. The diaphragm extends transversely between parallel beam lines. A reinforced concrete deck slab is then placed to integrate the beams and deck slab, thereby providing a fully composite continuous structure.

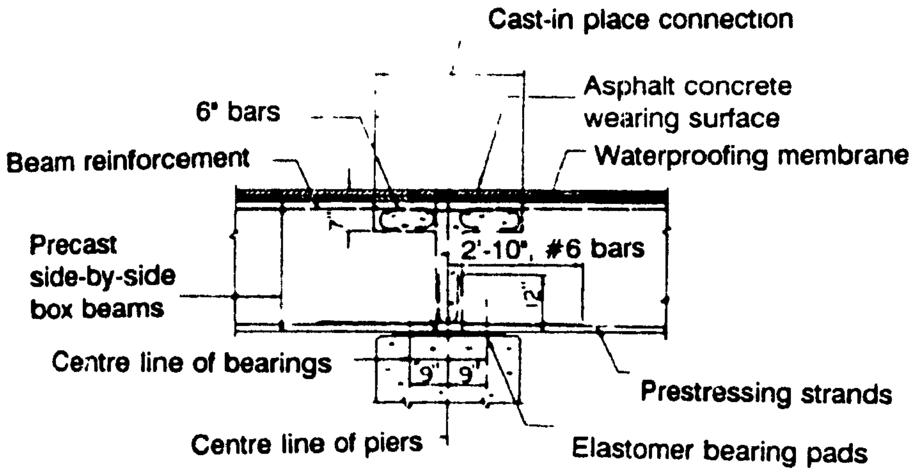


Figure 3.16 Placing of Concrete Diaphragm

A standard procedure used by the State of Ohio, to achieve continuity for simply supported box beams is shown in Figure 3.17. As indicated, the box beams are placed side by side and bolted together. Continuity reinforcement is then placed followed by the concrete closure placement.

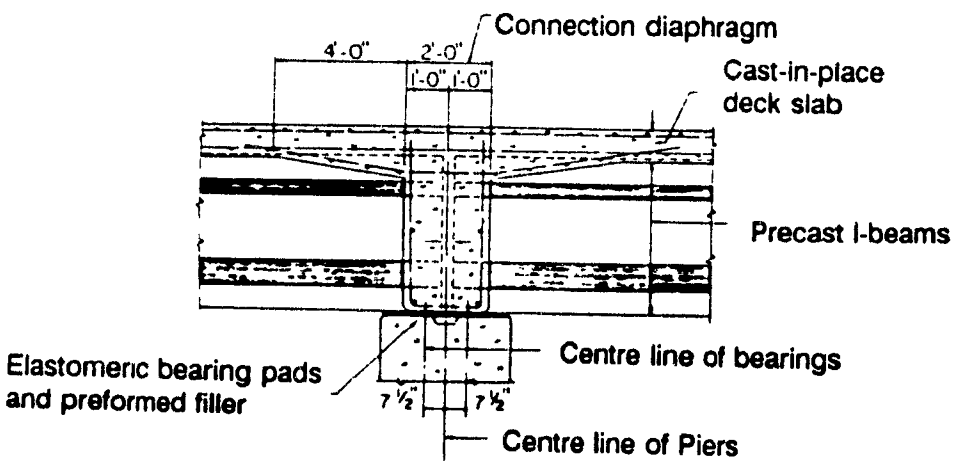


Figure 3.17 Continuity In Box Beams

A detailed description of the various considerations necessary to achieve continuity for bridges with continuously reinforced concrete deck slab on simply supported precast prestressed concrete beams can be found in Reference 22. It should be noted that the above methods provide either partial or complete continuous behaviour for live loads and superimposed dead load. Moreover, continuous and composite behaviour can be achieved for all loads by providing temporary intermediate supports which can be removed after casting the structural elements

CHAPTER 4
PRESTRESSED CONCRETE GIRDER SYSTEMS

4.1 Introduction

Increasing use of prestressed concrete girders in bridges has focussed concern on their service life because of the many problems which may include faulty design, poor workmanship, severe exposure conditions or natural calamities like accidents or fire damage. Errors in structural design, for example, inadequate provision of stirrup steel may lead to girder shear cracking in continuously reinforced concrete deck girder bridges. Poor workmanship may involve use of inferior quality concrete, improper placing of steel, poor consolidation of concrete and several other reasons which may cause a severe impact on the strength of the girders and cause progressive deterioration and eventual failure Girders exposed to severe corrosive conditions are likely to suffer damage due to corrosion of the prestressing tendons. Corrosion of prestressing tendons influences a larger percentage of the cross sectional area than in the case of conventional reinforcement which can lead to serious weakening even when the extent of corrosion is very small This can cause failure of the prestressing steel and consequently of the bridge member.

An in-depth inspection of the damage is necessary prior to selecting any repair procedure. The present practices of assessing damage, the various problems associated with prestressed concrete girders and the current methods of repair and replacement are reviewed in this chapter.

4.2 Damage Evaluation

As mentioned earlier, prior to selecting any repair procedure, it is necessary to evaluate the type of damage and its nature and extent by carrying out a thorough visual inspection of the girders. A wide range of inspection equipment has been used, including mirrors, flashlights, cameras, pachometers, dye penetrants, concrete coring equipment, and ultrasonic testing equipment. A detailed report of the damage is made providing information about the observed and measured damage related to each structural component.

4.2.1 Guidelines for Damage Evaluation

Three major types of damage to girders have been identified as follows:

1) Damage to prestressing strands by corrosion and deterioration of the protecting sheaths (see Figure 4.1),

2) Damage to the concrete (for example, failure of the anchorages - see Figure 4 2), and

3) Loss of structural capacity and rating

Failure sometimes occurs before the tendons are installed or soon after the tendons are prestressed but not covered with concrete It has been seen that pretensioned tendons offer better protection to the reinforcement than post-tensioned bonded tendons The latter are more susceptible to corrosion than any other prestressing tendons except for external uncoated tendons.

According to Novokshchenov (40), the principal reasons of corrosion of reinforcing steel are

"1) *Voids underneath high strength strands*

2) *Lack of passivation provided by concrete due to the decrease of alkalinity caused by carbonation*

3) *Contact between prestressing steel and chlorides , moisture and oxygen*

4) *Water permeable deck overlay, leaking expansion joints, and malfunctioning drains allowing chloride-laden water to have access to the prestressing steel*

5) *Inadequate cover to the concrete*

6) *Permeable concrete surrounding the prestressing steel due to poor consolidation, high water- cement ratio, and incomplete cement hydration*

7) *Cracks in concrete cover promoting penetration of corrosion inducing agents*

8) *Contact between prestressing steel and atomic hydrogen* "

Bonded and unbonded post-tensioned tendons may corrode due to one or more of the following causes (40):

"1) *Contact between prestressing steel and corrosion inducing agents.*

2) *Lack of passivation to prestressing steel in temporary and external tendons*

3) *Excessive local bending of a tendon*

4) *Faulty design of a structure, allowing water or salt laden water to collect in the areas of prestressing steel and penetrate the steel.*

5) *Presence of dissimilar metals, such as aluminium casing at end anchorages*

6) *Defects in grout, such as air pockets formed by evaporated bleed water, poor adherence of grout to prestressing tendons, and inadequate filling of a sheath*

7) *Access of water into sheathing through anchorages.*
8) *Use of grease that is permeable to moisture.*
9) *Lack of grease.*
10) *Damaged sheaths.*

Figure 4.1 Corrosion of Prestressing Strands

Figure 4.2 Corrosion of Anchor Steel Plate

11) *Use of a corrosion-susceptible sheath, which can gradually deteriorate exposing the prestressing steel to corrosion-inducing agents.*

The various types of corrosion are briefly reviewed in Section 4.3

4.2.2 Types of Damage

Damage has been classified into four types. minor, moderate, severe, and critical Minor damage involves damage to the concrete and may consist of extensive spalled areas on the girder without any exposed reinforcement or prestressing steel. Moderate damage involves spalling of the concrete which may expose the reinforcement and/or the prestressing steel The cracks on the concrete surface are wider than those in the case of minor damage However, they are closed below the surface damage and there is no damaged prestressing steel

On the other hand, severe damage involves damage to the concrete as well as the reinforcing and/or the prestressing steel and it may consist of one or more of the following

1) Cracks that extend across the width of the bottom flange but closed below the surface damage.
2) Major or total loss of concrete section in the bottom flange
3) Major loss of concrete section in the web, but not occurring at the same location as the loss of the concrete section in the bottom flange
4) Severed prestressing strands or strands that are visibly deformed.
5) Horizontal and vertical misalignment of the bottom flange.

Critical damage involves one or more of the following:

1) Cracks extending across the bottom flange and/or in the web directly above the damaged bottom flange that are not closed below the surface damage showing evidence of excedance of yield strength of the prestressing steel
2) An abrupt lateral offset as measured along the bottom flange or lateral distortion of exposed prestressing strands.
3) Loss of prestressing force to the extent that the calculations show that repairs cannot be made.
4) Vertical misalignment.
5) Longitudinal cracks at the interface of the web and the top flange that are not substantially closed below the surface damage which indicates permanent deformation of the stirrups.

4.2.3 Damage Assessment - Guidelines

Assessment of damage depends primarily upon its severity. In the case of minor damage, all unsound areas should be located and measured. However, for cases with extensive spalling and damage to prestressing strands, it is necessary to carry out detailed stress calculations to evaluate stresses in the damaged structure which are compared with the design stresses to determine the loss in capacity and provision for temporary supports. Reference 16 details the necessary calculations to determine the loss of the prestressing force and the stresses in the damaged structure It is necessary to review employing not only the traffic restrictions, but also the use of longitudinal steel girders above the damaged girder, or installation of other temporary falsework for safety and facility of the commuters

4.3 Types of Corrosion

Prestressing steel fails mainly due to brittle fracture which can be related to two main causes: pitting corrosion and stress corrosion cracking. Another cause of corrosion of prestressing steel, hydrogen embrittlement cracking, is a variation of stress corrosion cracking. These are discussed below

4.3.1 Pitting Corrosion

This is a localised form of galvanic corrosion. It initiates usually at points of rupture of a passivating surface film and proceeds until the cross-sectional area of the wire is reduced to such an extent that the tensile stress exceeds the ultimate tensile strength leading to failure. Also, pitting corrosion serves to create points of stress concentration. The plastic strain is localized at these points which cannot be redistributed by the high strength steels with low ductility, and as a result, the magnitude of stresses in these areas increases considerably. Furthermore, conditions favourable for hydrogen embrittlement can be created due to the presence of atomic hydrogen.

4.3.2 Stress Corrosion

This type of corrosion occurs where there is a chemical attack in the presence of high tensile stresses. It leads to intercrystalline and/or transcrystalline cracking and finally a brittle fracture of the prestressing steel.

4.3.3 Hydrogen Embrittlement

Hydrogen embrittlement cracking of steel under stress can occur when hydrogen enters the steel structure. Sources of atomic hydrogen may be steel itself, hydrogen sulphide (H_2S) present in the atmosphere, or development of galvanic cell due to water dissociation at the cathodic sites The hydrogen gas atoms are at high pressure and exert a significant tensile force tending to separate the iron atoms. This force coupled with the already existing high tensile force in the prestressing steel can at times exceed the yield strength of the steel causing cracking

4.4 Repair Procedures-Criteria for Selection

Structural integrity is generally the primary criterion for selecting a suitable repair method and involves consideration of the service load capacity, ultimate load capacity, overload capacity, fatigue life and the overall durability of the structure. It is necessary to calculate each of the above capacities for the damaged girder which should agree reasonably with those for the original girder. Typical repair procedures that have been used to increase the structural capacity are described in Section 4.7.

Fatigue of prestressed concrete girders is now being considered as a potential problem. Older bridges designed for lighter vehicles without fatigue provisions are likely to be exposed to high stress ranges with the increasing traffic load causing them to be fatigue-critical. Fatigue life can be determined by conducting a dynamic test on unbonded tendons. However, this is not necessary for bonded tendons unless the anchorage is located or used in such a way that repeated load applications can be expected on the anchorage.

Durability of repairs in the repaired girders is another aspect to be considered The NCHRP Report No. 243 (46) provides the following guidelines:

"1) *All sound concrete shall be removed and surface preparation shall be such that the new material placed will be compatible with the existing material New material shall have equal or better strength characteristics than the original concrete*

2) *Epoxy bonding, epoxy grout and epoxy injection materials and systems shall be fully tested and approved and shall be applied by trained personnel Special requirements concerning ambient temperatures must be observed.*

3) *Additional reinforcement to bond new material to existing surfaces should be considered*

4) *Preloading should be used if necessary (Refer to Section 4 7.2) to ensure that the repair section will not be subject to greater tensile stress under live load than the original section*

5) *Additional prestressing force as required to ensure stress levels in the repaired structure are no greater than the original design stress levels*

6) *To ensure further durability, the repaired areas should be sealed with a proven water retardant*

7) *Where repair desigr dictates, commitment shall be made to perform periodic preventive maintenance*

4.5 Repair-in-Place Procedures

4.5.1 Minor and Moderate Damage

Minor damage involves spalling of the concrete on the girder faces, and hence it is initially essential to remove all unsound concrete. This is accomplished by using either hand or power tools. However, care should be taken to avoid any damage to the reinforcement or the prestressing steel The deteriorated concrete areas are then patched by an approximately 12 mm to 25 mm thick layer of epoxy grout mortar consisting of four to seven parts of clean silica to one part of resin by weight.

Moderate damage repairs can be performed using special concrete mixes in addition to using epoxy grout. Epoxy injection of cracks has also been specified for cracks greater than 3 mils in width. Cracks in the range of 3 mils to 6 mils in width can be sealed effectively by injecting an epoxy resin into them. For wider cracks, a system containing a mineral filler can be used. Injection is normally done using pressure with the two components of the resin system being mixed at the injection nozzle. The pressure injection process is described in Section 4.7.1. It is also recommended to apply a preload prior to injecting and patching the damaged area which increases the durability of the repair.

4.5.2 Severe and Critical Damage

Prior to repair of severe damage, it is necessary to calculate the loss of prestress and the structural capacity after damage This damage should normally be repaired by application of preload as required, and with the addition of prestressing force in addition to the placement of concrete and epoxy injection of cracks

NCHRP Report # 226 (16) describes repair procedures for restoring the strength of prestressed girders in case of severe damage Splices for standard AASHTO I-beam types have been considered Moreover, these splices can be modified readily to use with other shapes Four repair methods are presented using these splices, namely adding external reinforcing, replacing internal prestress, adding external prestress and use of metal sleeve splicing Although the methods are basically for accidentally damaged girders, they can be extended for other cases as well.

Calculations for determining the loss of prestressing force are presented with guidance for a wider range of problems. Most of the splices extend above the bottom flange of the girder However, for bridges with full depth intermediate diaphragms, jacking corbels are placed adjacent to the diaphragms and the diaphragms pierced and used for additional strength For the splices where the diaphragms are an obstruction, concrete is removed or cored out from the diaphragm to allow passage of the splice

For girders subjected to critical damage, replacement is absolutely necessary However, stress calculations should be undertaken prior to replacement Also, as mentioned earlier, it is necessary to install temporary falsework during replacement for safety purposes Replacement is usually performed by cutting the deck slab and the connecting diaphragms, and lifting the damaged girder out of the superstructure. Typical replacement procedures as carried out by Iowa and Minnesota Departments of Transportation, U S A, are described in Section 4 8

4.6 Preventive and Remedial Procedures for Prestressing Steel

The following preventive and remedial measures focus primarily on the type of sheathing, anchorage systems, grease, drainage systems and some miscellaneous corrosion control measures.

1) Sheathing

In the current practice, polyethylene or polypropylene sheaths of thicknesses not less than 1 mm have been used effectively to protect the prestressing steel against the intrusion of chlorides, oxygen and moisture. These sheaths are sufficiently resistant to damage during transportation, storage and installation. Two types of sheathing commonly used are sheathing extruded over a tendon and ribbon type sheathing, the latter being heat sealed after enclosing the prestressing tendon Ribbon type sheathing is less effective due to its vulnerability to corrosion inducing agents through the sealed seam.

2) Anchorage Systems

It is essential to properly seal the ends of anchorage systems after stressing operations to prevent any moisture from penetrating into the tendon. Conventional Portland cement concrete plugs have been found to shrink or crack due to corrosion of the anchor plate, resulting in their loosening from the anchorage pockets.

One of the following two recommendations can be followed:

1) Use of prefabricated concrete plugs - If a prefabricated concrete plug is to be used, prior to plugging the end anchorage pocket, the end of the tendon is thoroughly sealed by applying an epoxy resin compound or corrosion inhibiting grease The protective coat is capped with a screw cap or a plastic lid to prevent it from displacing

2) The anchorage system can be completely isolated from the surrounding concrete by encapsulating it with a plastic sheathing made of high density polyethylene or polypropylene to ensure protection against corrosion.

3) Grease

Grease used for protecting the unbonded tendons should be tested for water permeability, aging and impurities such as chlorides, sulphides or nitrates. It should be applied evenly to the strand and it should fill the sheathing completely. Also, it should be free from air pockets, be continuous over the entire tendon length and it should not become fluid over the anticipated temperature range.

4) Drainage

An effective drainage system should not only remove the storm water drain from the bridge deck but also simultaneously prevent it from contacting with the adjacent substructure elements. A drainage system should have intakes of at least 200 mm diameter, removable intake grids, pipe diameters of at least 150 mm, and a pipe slope of at least 60^0. Intakes should not be located close to expansion joints in a deck slab and should be regularly inspected and cleaned.

5) Miscellaneous Corrosion Control Measures

An epoxy based coating is commonly used to protect structures adjacent to leaking joints against intrusion of corrosion-inducing agents Normally, the casting is installed after sand blasting and priming the concrete surface to prevent wetting and penetration Even if the coating appears to provide good protection, it should not be considered a substitute for a properly designed and maintained joint Epoxy-glued joints in segmental box units appear to provide a good water tight bond between sections

4.7 Present Repair Practices-Some Typical Procedures

4.7.1 Use of Epoxy Injection

Shear and flexural cracks can be effectively sealed by this method. It is a simple procedure of rebonding which involves sealing the external area of the crack and then forcing epoxy bonding through tiny holes perforated in the sealant surface into the crack The chosen epoxy resin system should be compatible with wet conditions for field applications.

4.7.1.1 Repair of Flexural Cracks

Prior to epoxy injection, any loose material in the cracks is blown off using compressed air Injection ports are then placed every 250 to 300 mm along the crack The port spacing is chosen to be approximately equal to the web thickness which ensures adequate epoxy penetration through the cracks. A rapid curing, smooth paste epoxy gel is used to seal the cracks and secure the injection ports prior to pressure grouting. After almost two hours of curing the epoxy gel, a high strength low viscosity epoxy adhesive is pressure-injected into each crack from the lowermost injection port. The injection progresses from port to port as soon as epoxy emerges from the next higher port along the crack or a pressure of 680-980 KPa is sustained at one port for 30 seconds without the flow of epoxy (23). The epoxy is then cured for two days after which the epoxy seal around the cracks is removed.

4.7.1.2 Repair of Shear Cracks

An effective method for repair of shear cracks (60) consists of drilling holes deep down to and intercepting the crack at an angle approximately normal to its surface The holes extend beyond the crack for a distance of about 500 mm An elastic sealant as described earlier is used to confine the crack Epoxy is then pumped through the bottom-most hole and into the crack to fill it. During the process of filling the girder, rebars of at least 900 mm length are placed across the crack and bonded in place with the polymerized epoxy as shown in Figure 4.3 After all of the bars are in place, epoxy pumping is continued until the crack and the drilled holes are completely filled

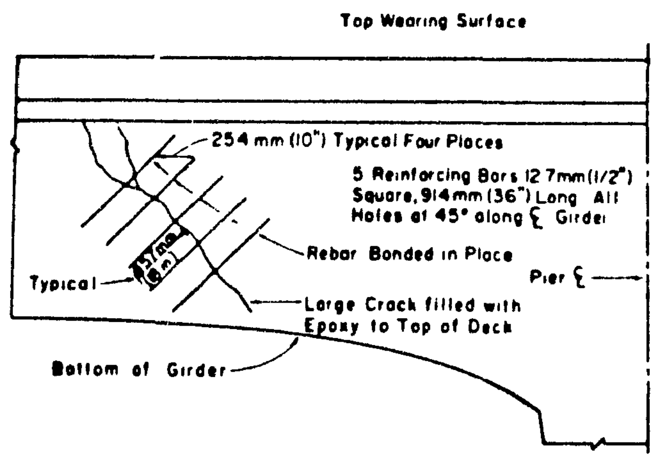


Figure 4.3 (a) Epoxy Injection - Level of Epoxy when Rebar Should be Inserted in the Hole (60)

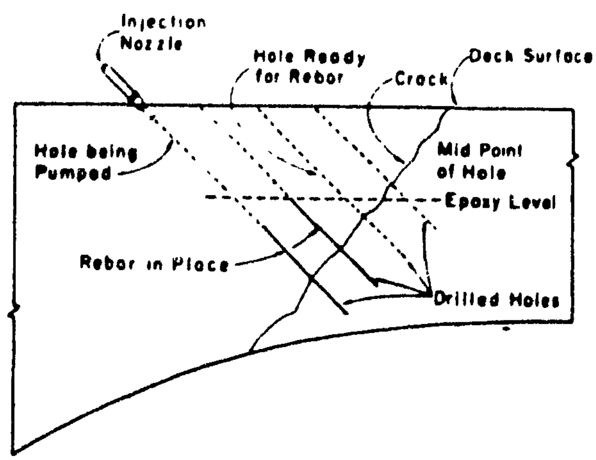


Figure 4.6 (b) Epoxy Injection - Typical Sketch and Description (60)

4.7.2 Preloading

This simple technique involves the application of a temporary vertical load during the repair-in place of a damaged prestressed member. Preloading is applied either by means of a loaded vehicle or by vertical jacking as shown in Figure 4 4 (a and b). The advantage of this procedure is that it requires a smaller roadway width However, if a single jack is used, it is necessary to compute the temporary stresses that will be induced due to transfer of the jacking load through the slab and the diaphragms to the adjacent girders

Preloading can be used in many instances to restore the girder to its original condition without adding prestress. In cases where a significant concrete portion of the girder has been lost, it is necessary to first calculate the section properties of the remaining cross section to determine the required preload. Furthermore, if the maximum allowable compressive stress in the reduced bottom flange is exceeded, enough preload during repair must be applied to bring this stress within the allowable limits

Partial or full prestress to the repaired area can also be restored using preloading This may be needed to reduce or eliminate tension under live load plus impact Durability of repair can be enhanced by applying preload prior to epoxy injection and repair This will result in the live load stresses being no greater than the original stresses. Preloading can also be used to improve the repair of girders with severed prestressing elements In this case, care should be taken to apply the correct preload to avoid any cracking in the remaining portion of the girder

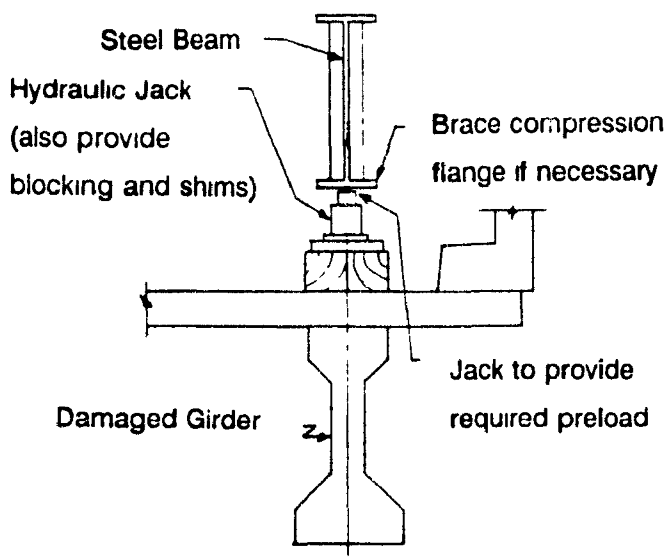


Figure 4.4 (a) Preloading by Vertical Jacking -Section near Point of Damage

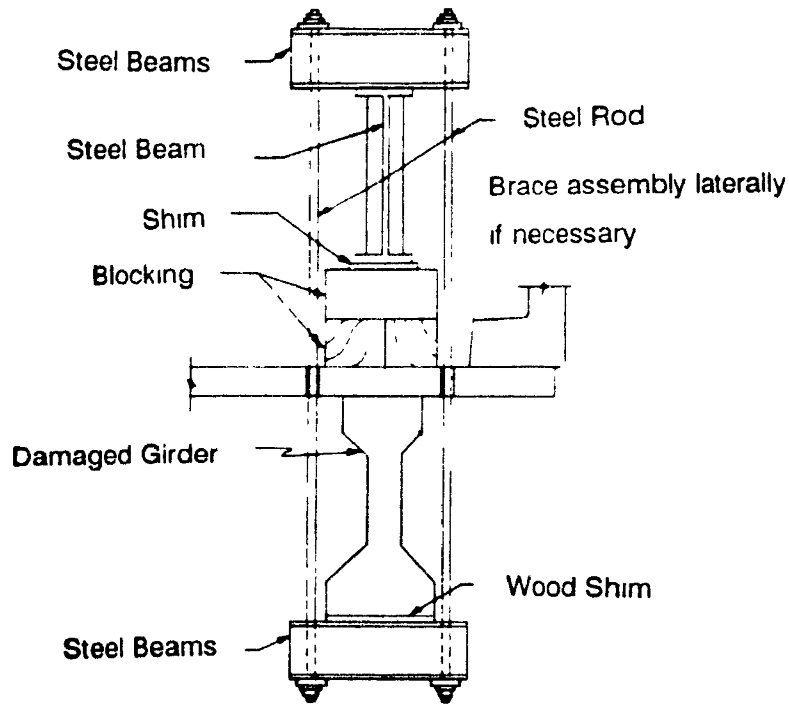


Figure 4.4 (b) Preloading by Vertical Jacking -Section Adjacent to Piers

4.7.3 Use of Precast Concrete Panels

This is a new and innovative approach in which precast, prestressed high strength concrete panels are added to the existing bridge girder in the form of a thin bottom soffit (Figure 4.5). The bottom soffit panel is positioned with galvanized dowel bolts and epoxy grouted to the girder bottom. The panel not only increases the longitudinal flexural capacity, but also improves the transverse load distribution characteristics and bridge aesthetics through a closed bottom soffit.

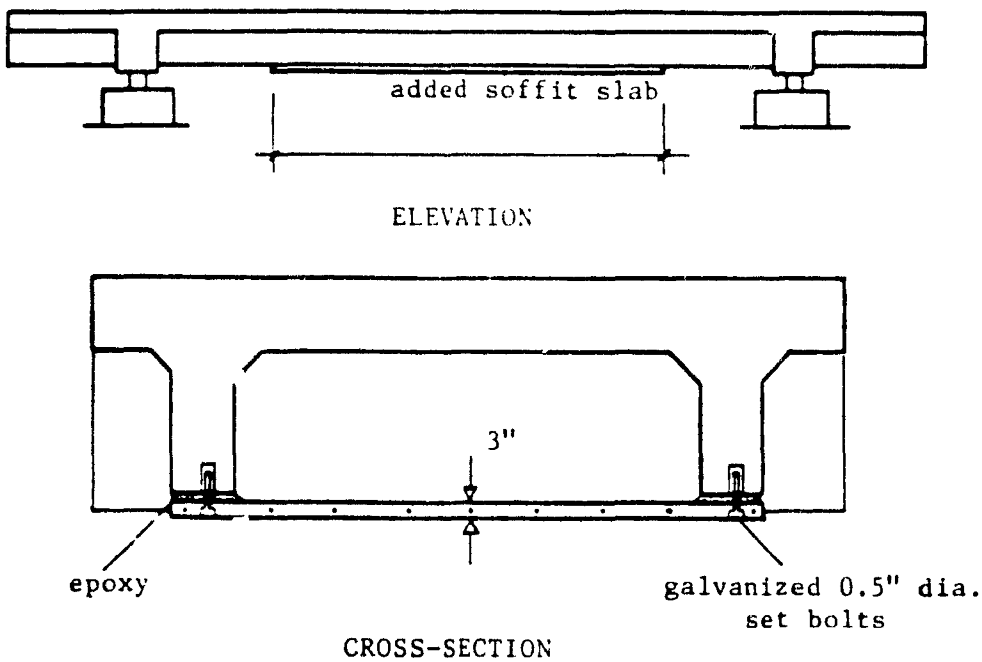


Figure 4.5 Bottom Soffit Slab Panel Addition (23)

4.7.4 External Post-Tensioning

It is well known that conventional prestressing imposes a permanent direct compression and a bending moment which relieves the applied bending moment. For a prestressed concrete deck, the above two effects together allow the deck to carry additional superimposed dead and live load moments without exceeding the permissible bending stresses This technique can also be used to increase the capacity of existing overloaded decks It also finds application in strengthening prestressed girders with severed or damaged tendons.

The direct compression effect of the added prestressing is in general not of any advantage Also, the allowable compressive stresses in reinforced concrete are usually lower than those in prestressed concrete

A most convenient and common way is the addition of external prestressing tendons on the sides of the webs of the girders at the bottom to increase the midspan flexural capacity The external post-tensioning is anchored to the bridge section through anchor blocks in the webs and/or any intermediate diaphragms in the bridge span as shown in Figure 4.6. The anchor block reinforcement is designed based on the shear friction criterion

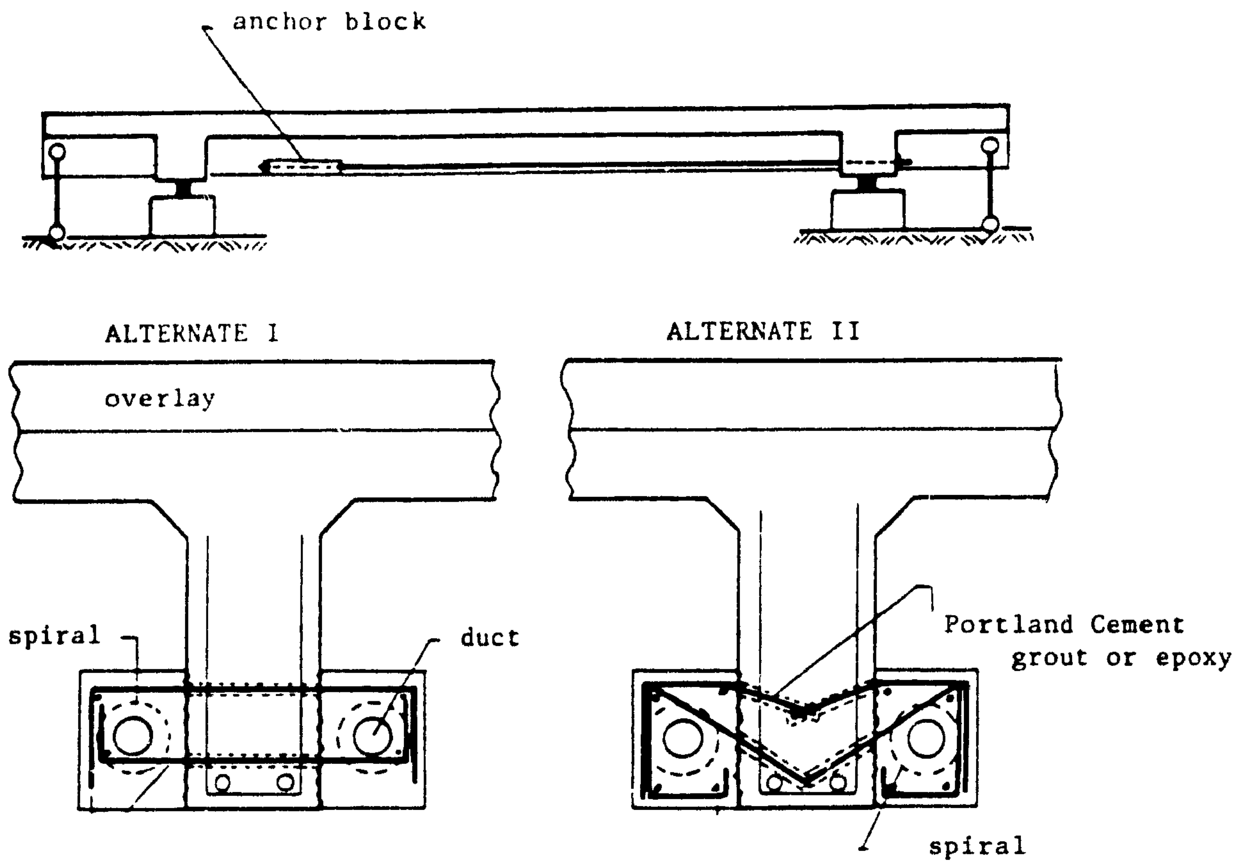


Figure 4.6 External Post-Tensioning (10)

The shear friction reinforcement can be anchored in two ways: through dowels grouted with epoxy in horizontally drilled holes or by setting individual dowels in drilled inclined holes with Portland cement grout on one side and epoxy gel on the other side of the girder as shown in Figure 4.6.

In repairing damaged girders, as shown in Figure 4.7, all of the loose concrete from the surrounding area is removed and the area is thoroughly cleaned with a high pressure water jet. Cracks, if any are chipped in the form of V-shaped groove. A predetermined axle load is placed over the damaged area of the beam. Any formwork necessary to replace the concrete is installed. All cracks in the existing beam are sealed with epoxy as well as the contact surfaces of the existing concrete coated with epoxy resin The new concrete is then placed while the epoxy resin is still tacky. The axle load is then removed from the damaged beam after the concrete has attained the required strength The diaphragm concrete is then removed to allow for passage of the new post-tensioning tendons Shear keys at suitable spacings are chipped into the existing beam along its length to house the new tendons Holes are drilled through the web as shown to accommodate the reinforcing bars to tie the new concrete together The reinforcing bars are grouted into the holes The surface of the concrete beams where the new concrete is to be bonded is roughened and cleaned followed by installation of the post-tensioning ducts, reinforcing steel and the forming for the new concrete The contact surfaces are then coated with an epoxy resin and the new concrete is placed while the epoxy is still tacky After the concrete has attained the design strength, the strands are tensioned to the specified load in a sequential manner to balance the load on each side of the flange Finally, the post-tensioning ducts are grouted with epoxy mortar.

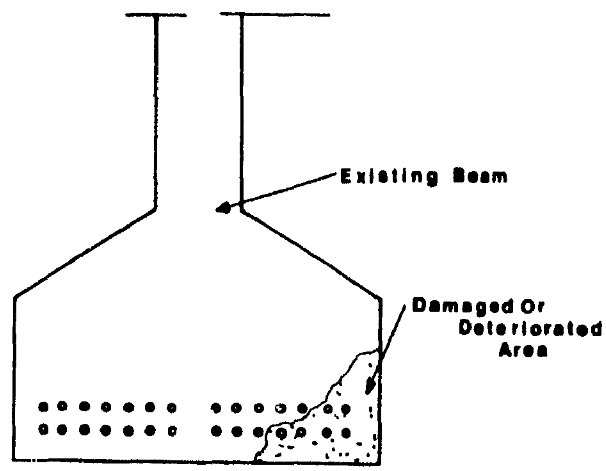


Figure 4.7 (a) Damaged Beam Repair -Section AA (10)

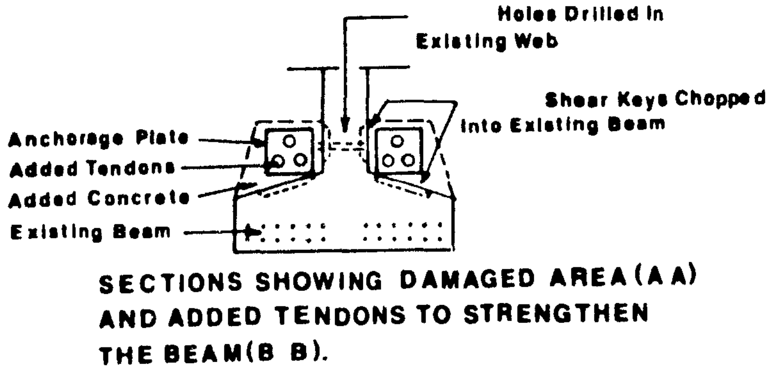


Figure 4.7 (b) Damaged Beam Repair - Section BB (10)

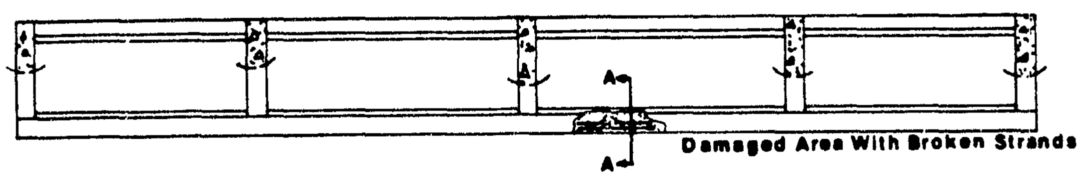


Figure 4.7 (c) Longitudinal Section of Girder Indicating Damaged Area (10)

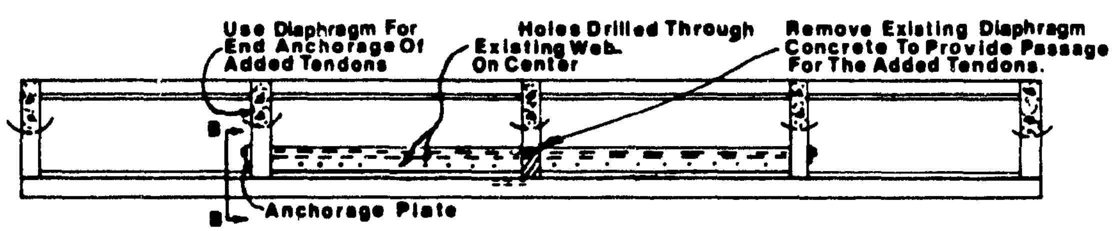


Figure 4.7 (d) Addition of Post-Tensioning Strands (10)

It has been common to extend the new tendons over a length greater than that shown in Figure 4 7. It may also be practical to post-tension between the end diaphragms rather than the intermediate diaphragms In any case, the existing diaphragms should be structurally adequate to support the end anchorages

The above repairs can usually be carried out with one lane of bridge open to traffic However, speed restrictions are specified to avoid problems due to any traffic induced vibrations

4.7.5 Use of Shock Transmission Units (STU)

The existing viaducts usually consist of long simply supported deck spans on high piers This is often seen in major river crossings where high navigational clearances require long approach viaducts as shown in Figure 4 8

The piers under each simply supported span inevitably carry fixed bearings for the one span alongside free bearings for the adjacent span This means that the design longitudinal traction and braking must be applied individually to each deck span through the viaduct Main resistance is offered by the pier carrying the fixed bearings of that particular span.

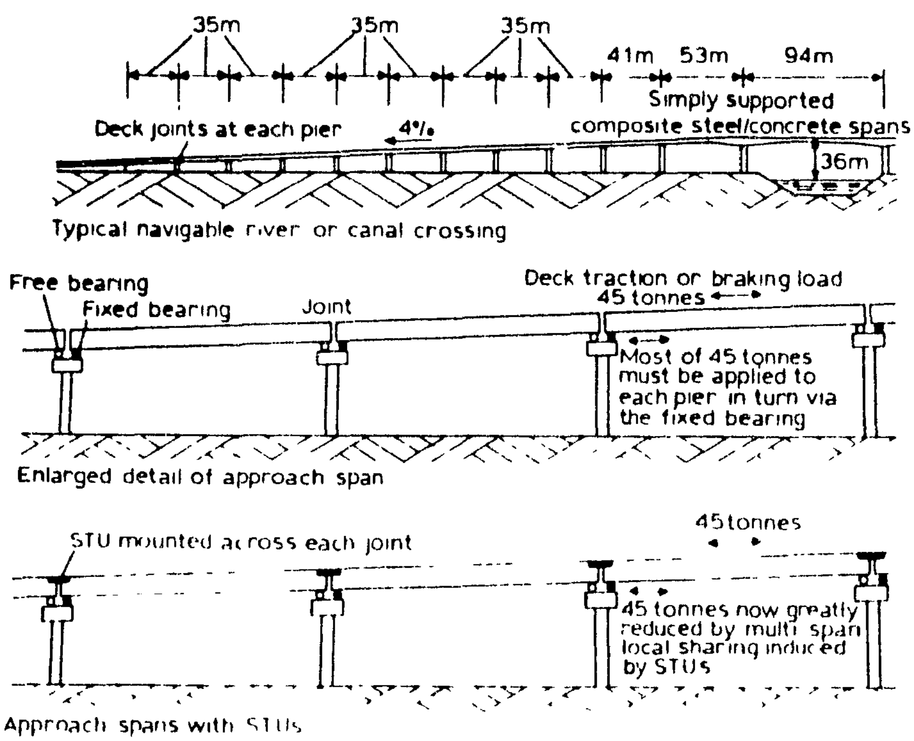


Figure 4.8 Substructure Strengthening using Shock Transmission Units (25)

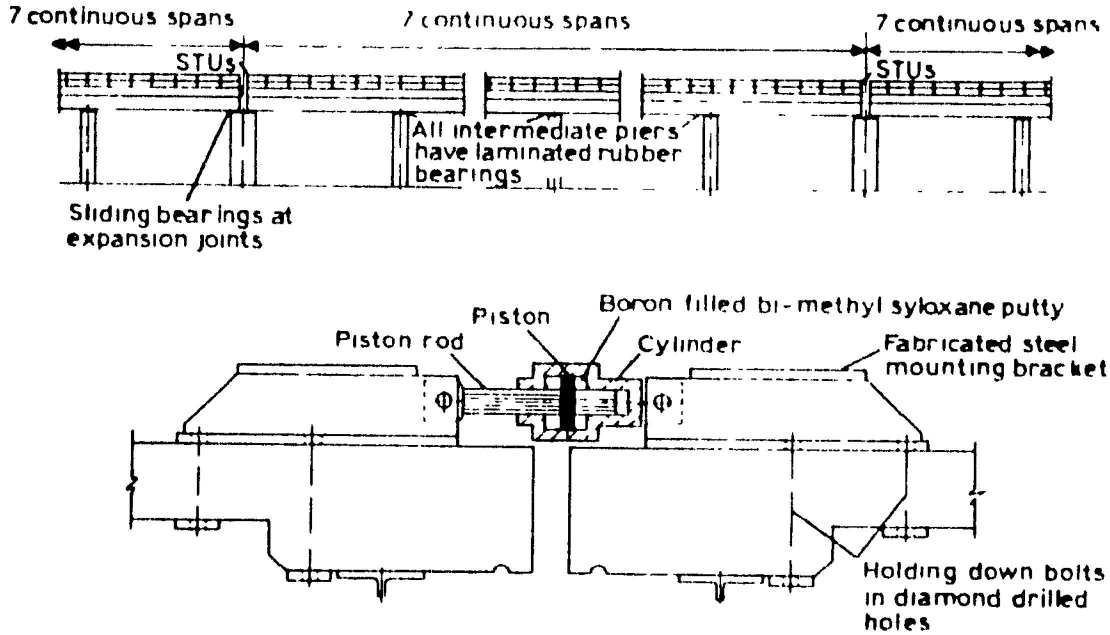


Figure 4.9 Typical Section of a Bridge STU (25)

Assessment of the structural integrity of a number of these viaducts have indicated that the piers are understrength due to increase in deck longitudinal loading from the design loading and also due to deterioration caused by deicing salts, carbonation, etc. For example, a substructure with say ten equal height piers will have a total resistance capacity of approximately ten times the original deck design traction and braking longitudinal loads. This total resistance can be mobilized by providing load transfer shock transmission units across the deck joints.

Shock transmission units are mechanisms which are connected across movement joints between structural elements and transmit slow acting joint movements like temperature and shrinkage with negligible resistance and, when required, transmit momentary impact forces like traction, braking and earthquake with negligible movement (25).

A typical bridge STU developed in the United Kingdom consists of a silicone compound which readily deforms under slow pressure but becomes rigid under impact The unit consists of a steel cylinder containing a loose fitting piston fixed to a transmission rod. The void around the piston is filled with silicone putty which when subjected to a slow movement, squeezes around the piston and displaces from one end of the cylinder to the other as shown in Figure 4.9.

4.8 Replacement Techniques

The choice of a replacement procedure is usually made based on the existing site conditions and may include either replacement from above, replacement from below or by replacement with a precast section that might include roadway deck and curb. The most popular method used to replace a damaged girder has been to remove portions of the roadway slab and diaphragms, allowing the girder to be lifted up and out of the structure It is necessary to leave sufficient length of the reinforcing steel, extending from the slab and diaphragms, to provide for lap requirements. Also, sawcuts are made in the slab prior to the concrete removal to obtain good breaklines in the sound concrete

Two methods of replacement as followed by the Iowa and Minnesota Transportation Departments, U.S.A., have been described Both methods require the use of conventional falsework during the removal operations. The falsework is required to be designed and constructed to safely carry the imposed loads Detailing of the falsework and public protection systems should be carefully included in the detailed replacement plans and specifications. Also, the highway below needs to be temporarily closed during the removal and replacement of the girders.

4.8.1 Iowa Method

In this method (16), the falsework bents are located at the point where the girder is to be cut for removal. The removal involves two stages as shown in Figure 4.10. The falsework bent is constructed so that half of it is moved for the second stage removal This allows for maximum movement of traffic during removal. The falsework bent which remains in place through both stages of construction is sheathed with plywood to serve as a barrier between removal operations and the traffic lane. The falsework bents are razed after both sections of the damaged girder are removed.

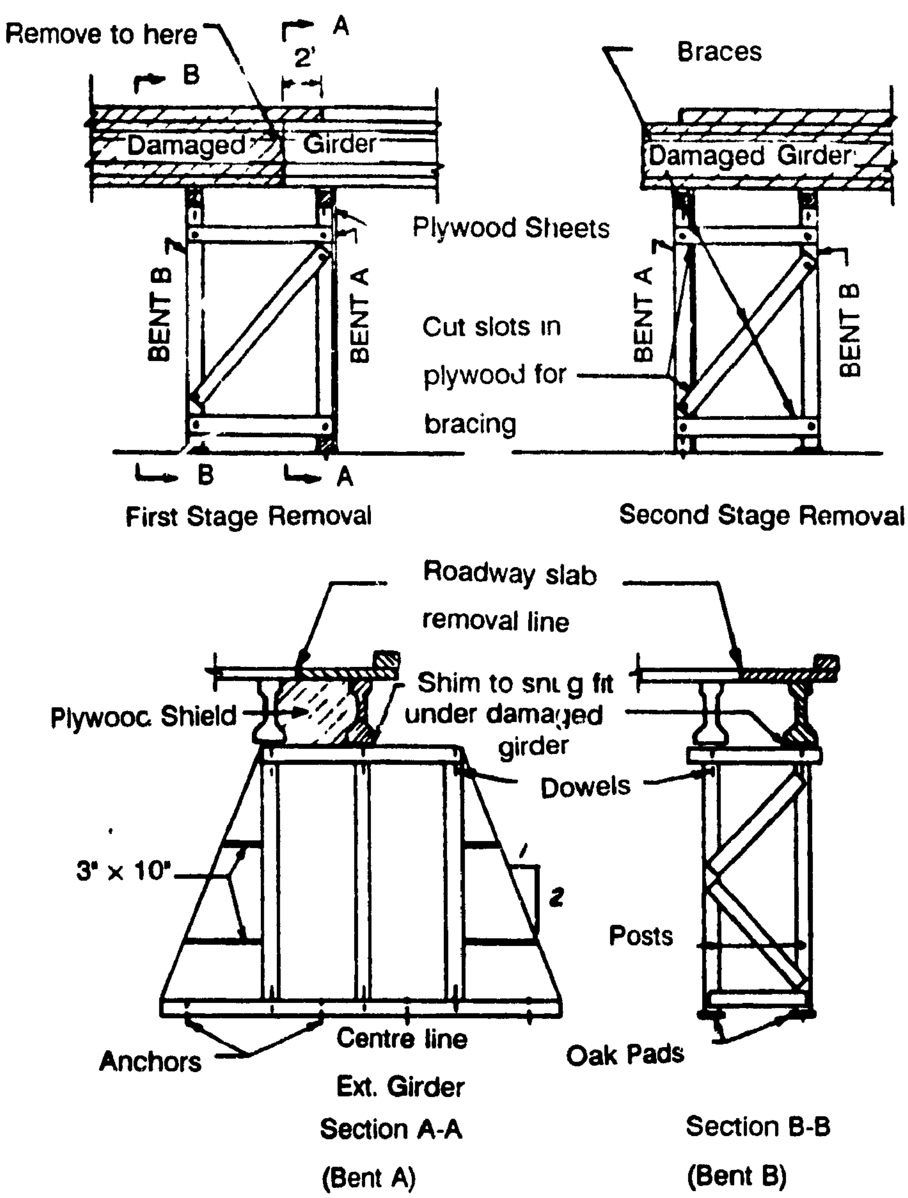


Figure 4.10 Iowa Method (16)

4.8.2 Minnesota Method

In this method, the damaged girders are removed and replaced from underneath the roadway slab (16). Both facia and interior girder replacement are described in detail in Reference 16. In both cases, the girders were fabricated without any shear devices projecting from the top.

4.9 Fabrication, Storage and Construction Defects

1) Concrete Defects

These include minor spalls, rock pockets, hairline cracks, etc. A wide range of repair methods can be used like patching with epoxy mortar or concrete mortar applied to epoxy coated surfaces, minor cracks sealed with epoxy, mortar patching after areas are chipped to sound concrete, final finish by burlap bagging or stone and epoxy injection.

In cases where precompressed areas are to be repaired, the prestress is first released. This may, however, lead to cracking in the repaired areas with the application of additional dead and live loads. Ingress of moisture is not ruled out unless the depth of the defect is very superficial. This may cause corrosion of the prestressing elements. An alternate method to restore compression after release of prestress is using the technique of preloading (see Section 4.7.2).

2) Horizontal Misalignment

This defect occurs during manufacture and may be due to one or more of the following reasons:

1) Prestressing strands not following the centre lines of the girders closely enough.
2) Sunlight on one side of the girder which may cause a permanent set if not rectified.
3) Curing procedures that allow a large temperature difference on opposite sides of the girder.
4) Improper storage that allows the girders to tilt slightly and cause lateral bow.

Excessive horizontal misalignment can be avoided by storing the girders on firm supports, preferably in a north-south position. Also, large differential temperatures on opposite sides of the girder during curing should be avoided and the strands must be placed accurately with respect to the centre line of the girder. Another method to eliminate or reduce lateral misalignment is by shimming under one side of the girders. A last resort would be the use of external force to pull the girders into proper alignment. However, this can create flexural stresses that reduce compression on one side of the bottom flange, increasing the probability of cracking under the action of additional dead and live loads.

3) Rotational Misalignment (Twist)

This is a rare manufacturing defect and can be avoided by carefully aligning the forms. Twist is more critical for wide flange girders that connect each other. For girders with cast-in-place decks, the bearings can be adjusted slightly to compensate for minor twisting. External twisting force can also be used to restore proper alignment. However, this may induce tensile stresses with increasing probability of cracking under the application of additional dead and live loads.

4) Mislocation of Prestressing Strands and/or Reinforcing Steel

Proper care should be taken during placing of the prestressing strands to avoid mislocation, which can be rectified by applying external tensile stresses. This is usually avoided since it increases the probability of cracking of the concrete under the application of additional dead and live loads. Reinforcing steel may also get mislocated due to improper cover and can be corrected by adding a 25 mm coating of epoxy mortar.

5) Mishandling

Mishandling at the plant or during transportation can cause damage to the girders. This may range from minor damage in the form of small spalls to major damage in the form of buckling and complete failure. This can be avoided by taking proper care during handling and transportation such as provision of lateral bracing trusses. Any cracks and spalls arising due to mishandling should be repaired using the established procedures presented earlier.

4.10 Miscellaneous Repair Techniques

4.10.1 Repair of Deteriorated Ends of Girders

Ends of girders may deteriorate either due to excessive shear forces from uneven settlement of the substructure or by chlorides and/or freezing and thawing in cold climate. Deterioration may also occur due to lack of provision for movement from temperature changes.

The repair procedure consists of initially constructing temporary bents for supporting the jacks and blocking as shown in Figure 4.11 (a and b). The jacks are then placed in position. The entire end of the bridge is raised by a fraction of an inch and a thin piece of sheet metal is inserted under the girder seat at the pier or the abutment to serve as a bond breaker for the new concrete.

The deteriorated concrete is removed in steps as shown in Figure 4.11 (b) to provide horizontal bearing surfaces between the new and old concrete. The new reinforcing steel is then placed in position and lapped or welded with the existing steel as shown in Figure 4.11 (c). Epoxy bonding compound is applied to the shear surfaces of the girder ends. Formwork is erected in place and fresh concrete with a non-shrink additive is poured. After the concrete has attained sufficient strength, all of the beams are jacked simultaneously to sufficient height to allow placing of the elastomeric bearing pads. The entire end of the bridge is then lowered uniformly and checked for any possible distress in the repaired area.

The above method should be used only if the deterioration at the girder ends is extensive. In case of moderate cracking, epoxy injection is sufficient and economical.

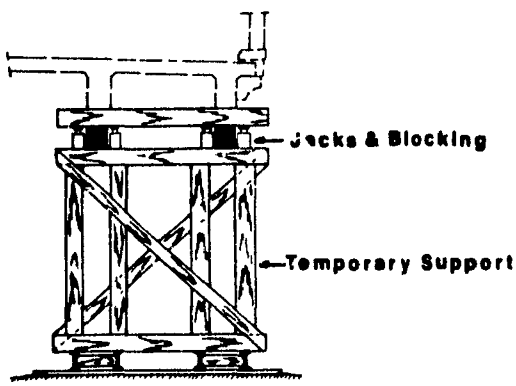


Figure 4.11 (a) Repair of Deteriorated Ends of Girders -Part Transverse Section (10)

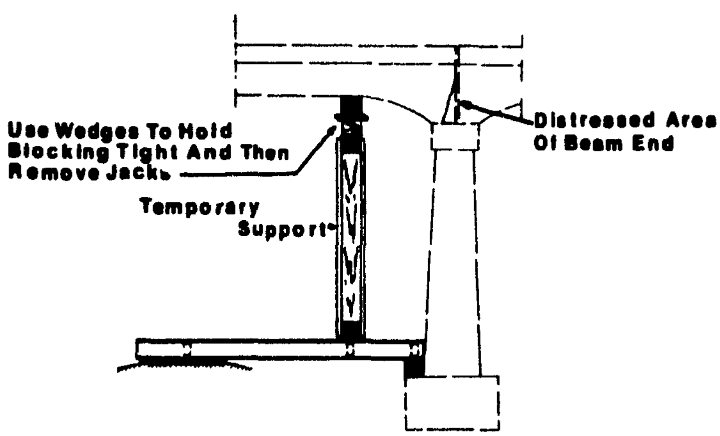


Figure 4.11 (b) Repair of Deteriorated Ends of Girders - Part Side Elevation

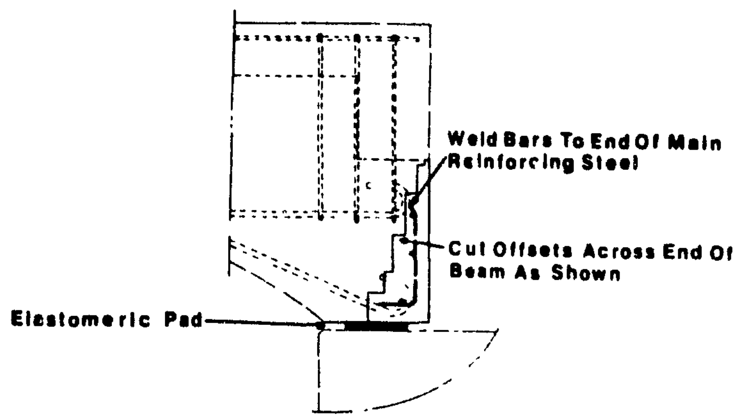


Figure 4.11 (c) Repair of Deteriorated Ends of Girders - Repaired Area (10)

4.10.2 Use of Pier Saddles for Repair of Distressed Girder Bearing Areas

Bearing areas of distressed girders can be repaired either by relocating or by extending them. However, it is advisable only in cases where the repair of concrete beam ends would be inadequate or inadvisable. Furthermore, the dead load reactions from the adjacent spans should be approximately equal and the live load shear should be small to prevent racking of the saddle assembly.

4.10.2.1 Repair of Bearing Areas

This repair procedure involves use of prefabricated channel sections, box beams and other saddle components (Figure 4.12 -a to c). The saddle components are designed for normal girder reactions. A red paint is applied to the pier cap bearing area that will support the channel sections. Three layers of duck are then placed over the bearing area with each layer being thoroughly painted. The channel sections are then placed over the pier cap. Also, hanger straps and draw-up bolts are attached to the supporting box beams. Elastomeric bearing pads are placed on the box beams under the girders after which the beams are pulled up using bolts passing through the lugs provided on the channels and saddle beam hanger straps. The channels are then welded to the hanger straps and the draw-up bolts are removed. The bolts can be left in place as an alternative in which case the welding can be eliminated.

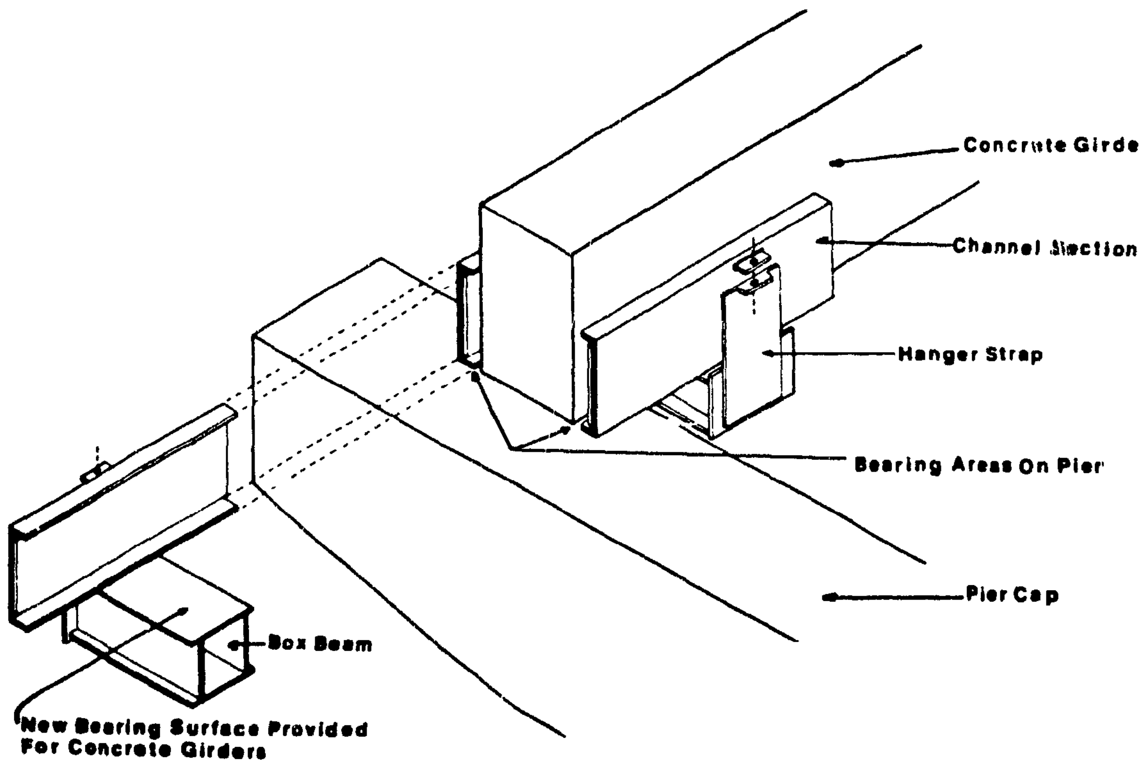


Figure 4.12 (a) Relocation of Bearing Areas -Isometric view of Saddle (10)

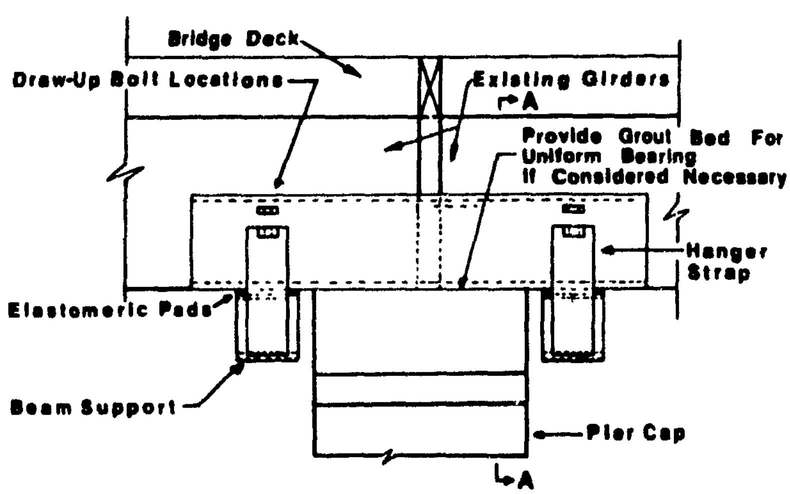


Figure 4.12 (b) Relocation of Bearing Areas - Side Elevation (10)

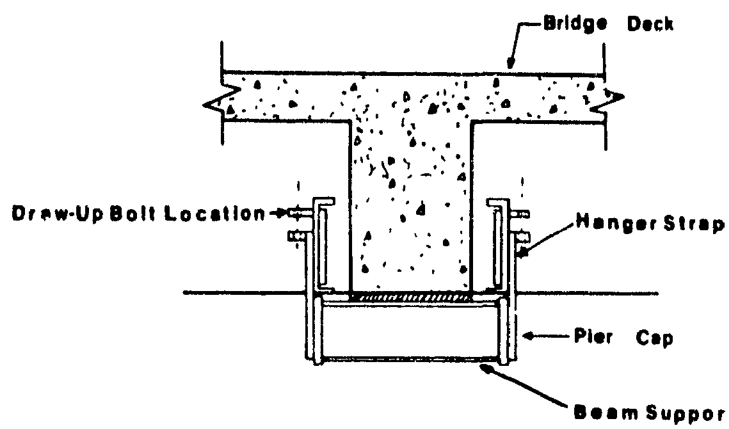


Figure 4.12 (c) Relocation of Bearing Areas - Section A-A (10)

4.10.2.2 Extending the Bearing Areas

The saddle components used in this repair procedure are shown in Figure 4.13 (a and b) and are designed for the maximum girder reactions. Red paint is applied to the bearing area which supports the double angle sections. Three sections of duck are placed over the bearing area and each layer is painted. Angle assemblies are then placed over the pier cap and the hanger straps are attached with the supporting wide flange beams. Field shim plates are then placed and welded to the hanger plates such that a snug fit with the pier cap is obtained as shown in Figure 4.13 (a).

Falsework is erected over the ends of the assembly in which a non-shrinking grout is placed. After the grout has achieved final set, concrete is placed between the angle and beams as shown in Figure 4.13 (b). The forms are then removed. Although this procedure can be used as an alternative to the earlier method, it does not remove the existing dead load bearing forces from the pier cap.

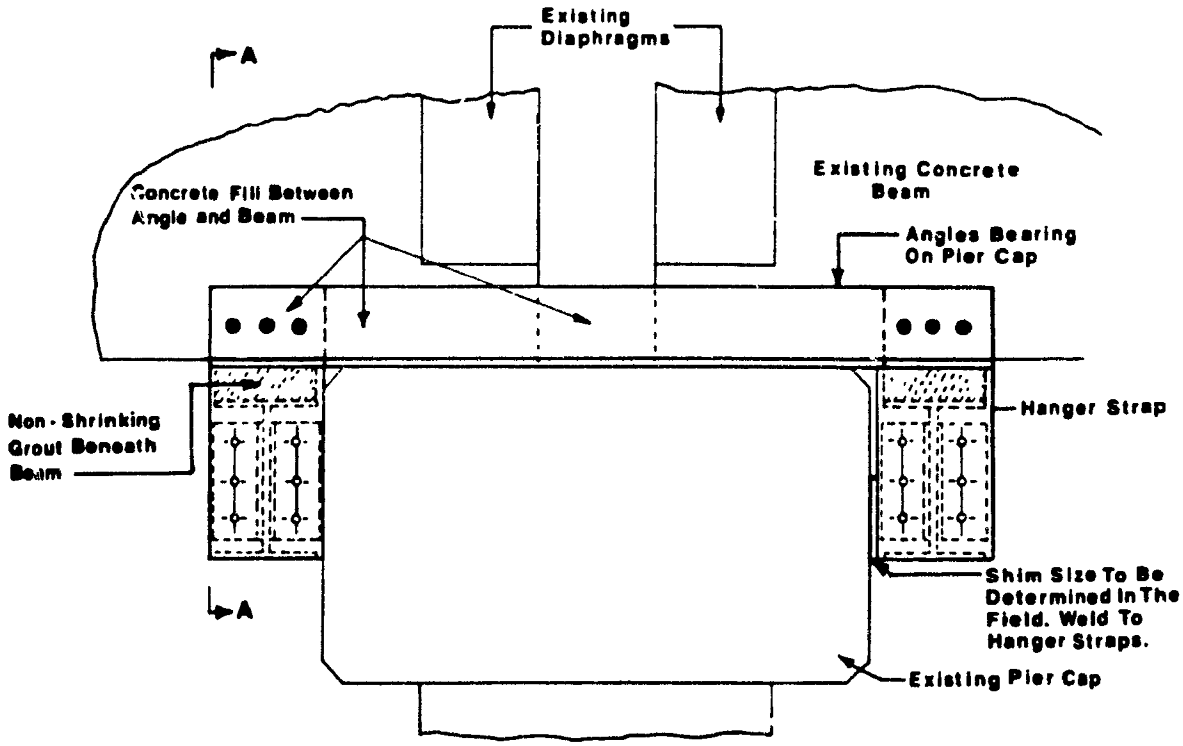


Figure 4.13 (a) Extending the Bearing Areas - Side Elevation of Saddle (10)

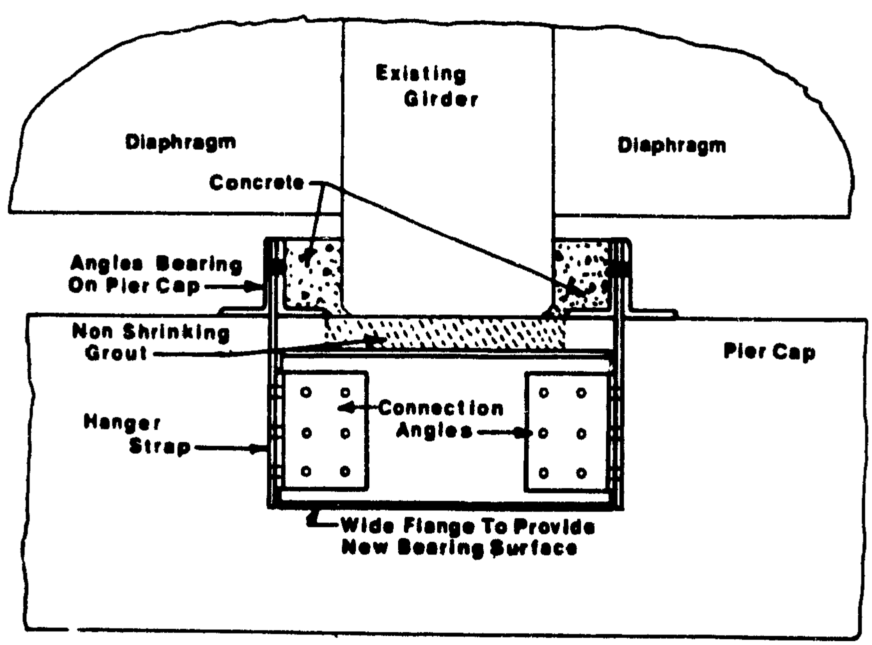


Figure 4.13 (b) Extending the Bearing Areas - Section A-A (10)

CHAPTER 5
BEARING SYSTEMS

5.1 Introduction

Bridge bearings are necessary to transmit both vertical and horizontal loads from the deck to the substructure. The vertical load is usually a compressive load and consists of dead and live loads. It may be tensile in some cases depending on the continuity of the bridge and the loading pattern. The horizontal loads consist of braking forces due to the vehicular traffic, wind forces, centrifugal forces in curved bridges and earthquake loads. Bearing systems have to be designed in such a way that no constraints occur in accommodating the movements between the deck and the substructure for an efficient transfer of vertical and horizontal loads. In earthquake zones, the bearings should provide additional restraint to keep the joint from separating and the span from dropping.

Malfunctioning of bearings can be attributed to accidental damage or displacement, chemical attack, fire, corrosion and the most important being improper installation It is, therefore, necessary to implement a routine bearing maintenance program to keep the bearings protected from the effects of water, deicing salts and debris leaking from the joints. All potential faults should be repaired before they cause any serious damage to the bridge.

The following sections discuss the various types of bearings used, their problems and the current repair and replacement procedures. Recommendations for design, installation and maintenance, and retrofit procedures to withstand seismicity have also been discussed.

5.2 Types of Bearings

5.2.1 Sliding Plate Bearings

These were the earliest type of bearings, made from a variety of materials, with the simplest being steel plates sliding on each other with their surfaces lubricated to allow free movement. For small movements (i.e. spans less than 12 m), an approximately 3 mm thick lead sheet or 1.5 mm thick graphite-impregnated asbestos sheets are used instead, between the plates. However, the steel plates get corroded due to the accumulation of debris on and between the plates leading to freezing of the bearing. Also, the lubricant has a tendency of being forced out which may cause wearing of the plates. This problem is avoided by the use of self-lubricating bronze bearings consisting of a lubricating bronze plate with chamfered edges, either flat or machined to a radius placed between the lower masonry plate and the upper sole plate (9).

Recently, the most preferred type of sliding plate bearing has been the use of polytetrafluoroethylene (TFE or PTFE) sliding on steel (Figure 5 1) PTFE has excellent low friction properties Also, horizontal movement in any direction is possible Due to its low compressive strength and high thermal expansion, it is usually combined with bronze or glass fibre fillers to give it the necessary strength for use in bridge bearings. The following three basic principles should be observed in utilising PTFE (34):

a) The mating surface should be smooth and flat (or it should have the same curvature as the PTFE)-and remain so in service.

b) The PTFE should be mechanically retained on a rigid backing plate in a recess in order to prevent its excessive creep and also to secure the sheet and prevent it from slipping out of the bearing due to any structural movement.

c) Some misalignment can always occur during service due to creep, shrinkage, settlement, etc. It is therefore essential to provide a self-aligning device within the bearing assembly in order to avoid high stresses on the PTFE and the concrete.

The support of the sliding bearing plates is very important since even small deflections can lead to seizure, say, through metal contact between the PTFE retaining plate and the mating sliding plate. This can also be caused by the hydraulic loading resulting from liquid concrete. Any deflection of the bearing top plate would, therefore, become permanent upon the curing of concrete.

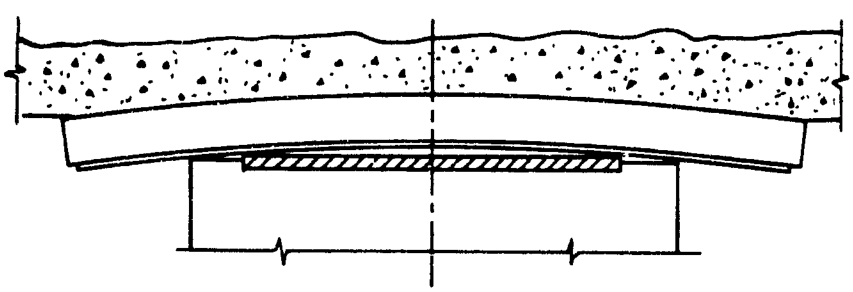


Figure 5.1 PTFE Bearing - Deflection of Bearing Top Plate (34)

5.2.2 Roller Bearings

These are usually made of curved hard steel surfaces which roll over other hardened steel surfaces The rolling surfaces are treated with a hard chrome steel plating to improve the corrosion and abrasion resistance. Some of the roller bearings being used presently are.

1) Roller Nests

A roller nest consists of small diameter rollers (38 mm to 50 mm) closely spaced to form a nest The rollers are kept parallel and separated from each other by providing spacer bars on each end. However, there are many small spaces left which may eventually get clogged with debris leading to their jamming and rusting A common preventive measure is to fill the spaces with grease to prevent ingress of debris and moisture However, the grease can melt due to heat, thereby exposing the spaces. An improvement consisted of the use of a group of larger diameter rollers (up to 360 mm diameter) which are not only free from debris but also give better access for cleaning besides accommodating larger movements.

2) Single Roller

This improved system consists of a single roller of diameter greater than 100 mm. It is kept in place by pintle pins tightly set into the lower element, two each on the top, bottom, and along the line of contact of the roller and plates. The pins project into holes provided in the plates and are loosely fitted to permit backward and forward movement of the roller besides preventing them from straying from their position (see Figure 5.2).

A common problem with a single roller system is again the ingress of moisture into the tight areas at the top and bottom of the rollers leading to corrosion and development of flat spots on the rollers. This can be repaired temporarily by jacking up the bridge and rotating the rollers through 45 degrees to bring new circular sections into the bearing areas. A more permanent repair is subsequently necessary.

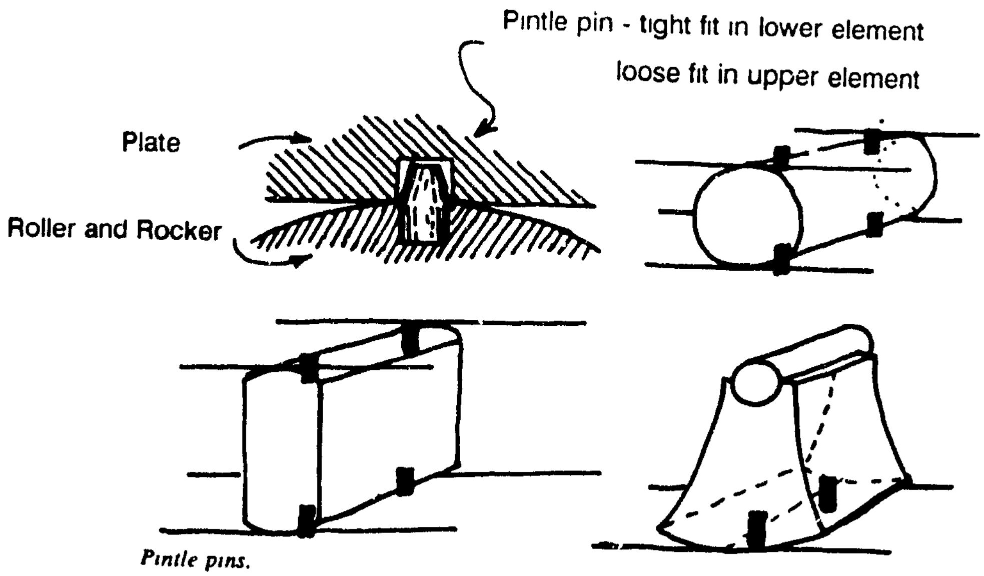


Figure 5.2 Single Roller (9)

3) Segmental Rockers

This bearing has the form of a cylinder with its two sides cut out with concavity inwards, the radii of the two cut-out faces being usually greater than half the depth of the rocker. This roller was an improvement over the conventional roller wherein the unused sides are cut to reduce its weight, besides reducing the space occupied.

4) Pinned Rockers

Recently, pinned rockers have been used more frequently mainly due to the tendency of the pin connection to maintain the correct alignment (Figure 5.3). However, the main disadvantage is the corrosion and freezing of the pins which may cause locking of the joint. They have been used mainly for long spans and heavy loads.

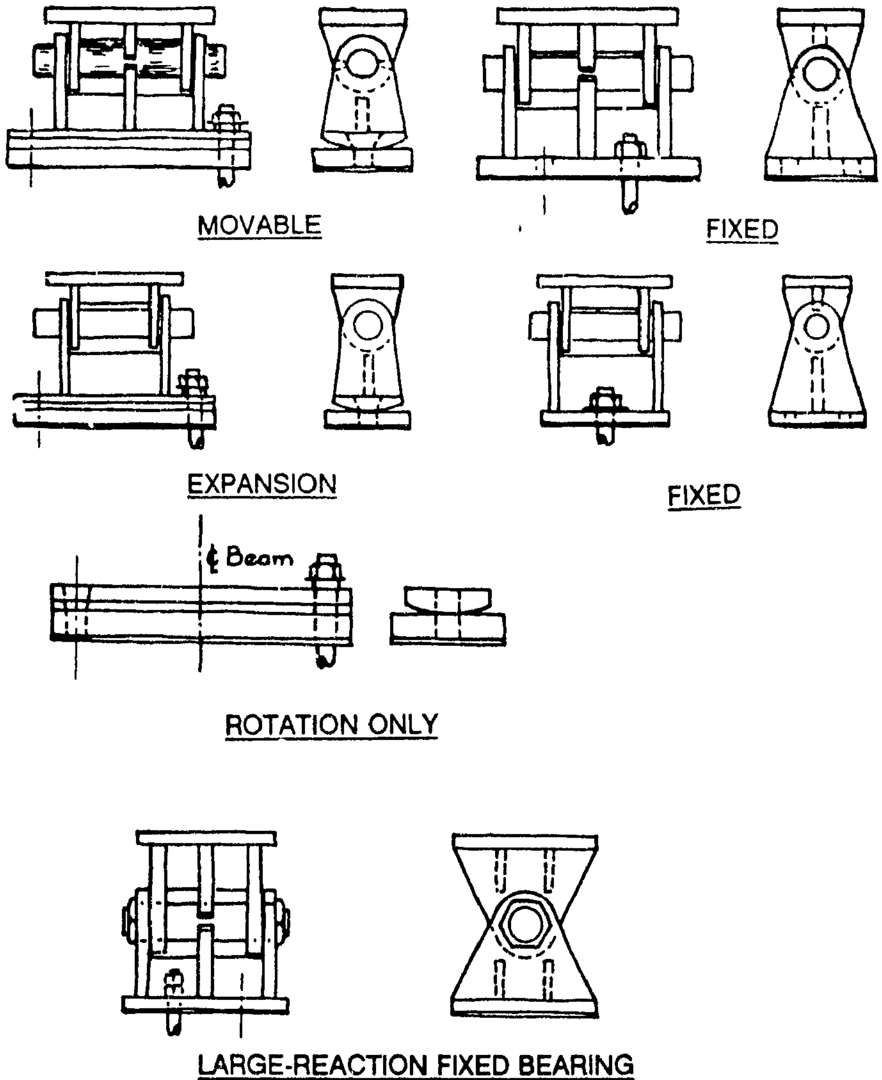


Figure 5.3 Steel Rockers and Pinned Bearings (9)

5) Rack and Pinion Bearings

These bearings have also been used for large spans to cater for very heavy loads. A typical arrangement consists essentially of a pinion rigidly attached to the shaft of the bearing at each end. The pinion has two racks, one above and one below, attached to the superstructure and substructure, respectively. The pinions serve to keep the rollers aligned even for very large loads without shearing off.

5.2.3 Linkage Devices

A common linkage device consists of a simple step bearing with an elastomeric expansion pad as shown in Figure 5.4. An alternate procedure used recently to accommodate large temperature movements has been the use of bridges with tall and slender piers with no intermediate expansion bearings in which case the slenderness of the piers absorbs the movement as explained in Section 3.6.

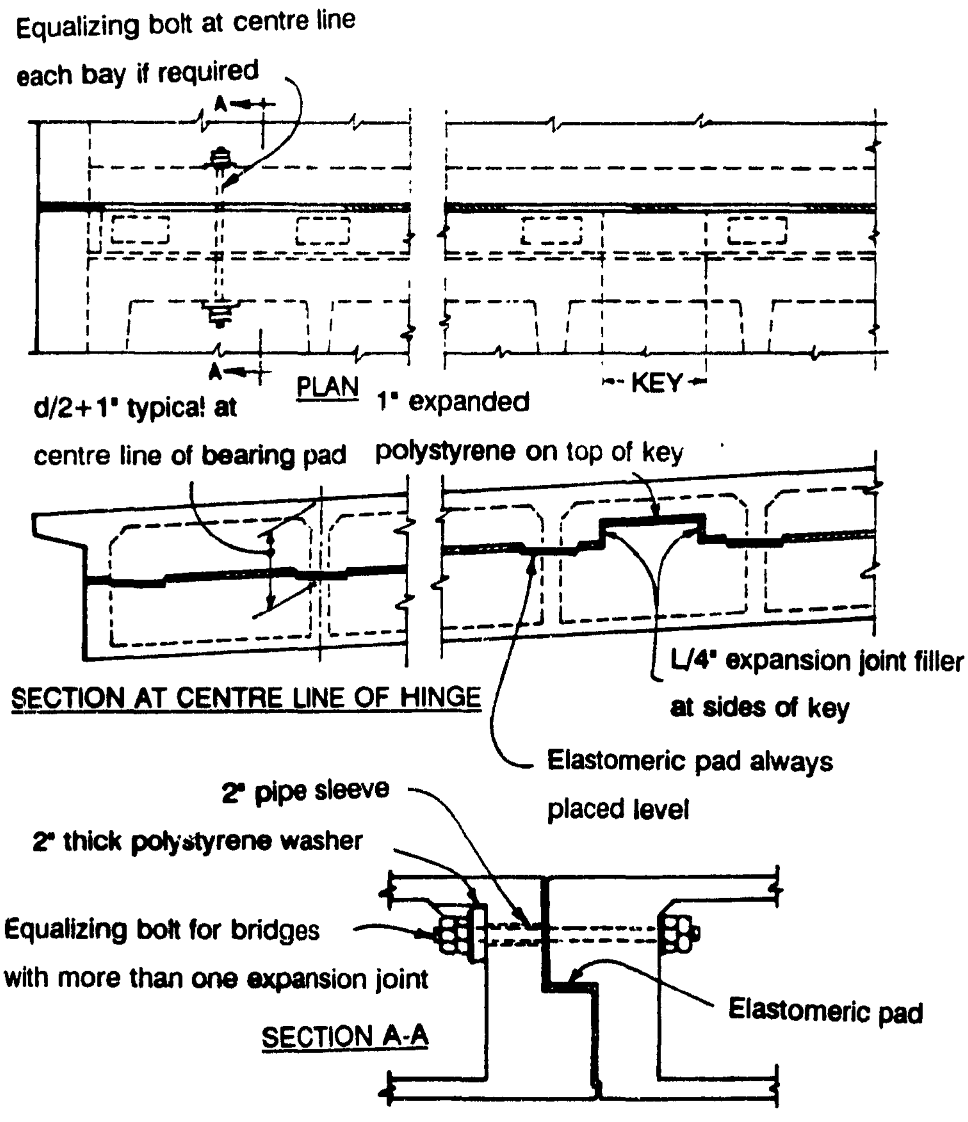


Figure 5.4 Typical Linkage Device for Concrete Box Girder (9)

5.2.4 Elastomeric Bearings

These have been developed lately and found the most suitable compared with other types of bearings. Their advantage is their low installation and maintenance cost. They are practically inert and unaffected by weather. They usually do not have any moving parts and can be used for moderate movements of around 50 to 75 mm. Also, they have rotation handling capacity which can be increased by increasing the number of elastomer layers.

As shown in Figure 5.5, a typical bearing consists of alternate laminations of elastomer and metal or metal and fabric separators bonded together. Commonly used elastomers have been either virgin natural polyisoprene (natural rubber) or virgin chloroprene (neoprene), with the latter being preferred. Rolled mild steel sheets and glass fibres have been used as metal and fabric laminations, respectively. The top and bottom laminations are usually of metal or fabric. Metal laminations at the top and bottom are usually coated with a approximately 3 mm thick sheet of elastomer. The bearings have to be placed on a smooth and level surface of the seat without any adhesives. Generally, a concrete restraining lip is provided around the pad to prevent any slipping during earthquakes.

If quality materials are used, elastomeric bearings can take a considerable amount of rough treatment before failure. However, localized loading, for example, due to uneven seating can break the bond between the elastomer and steel. The resulting shear forces can disintegrate the seats at the edges. This can be avoided by extending the seatings at least 50 mm beyond the edge of the bearing.

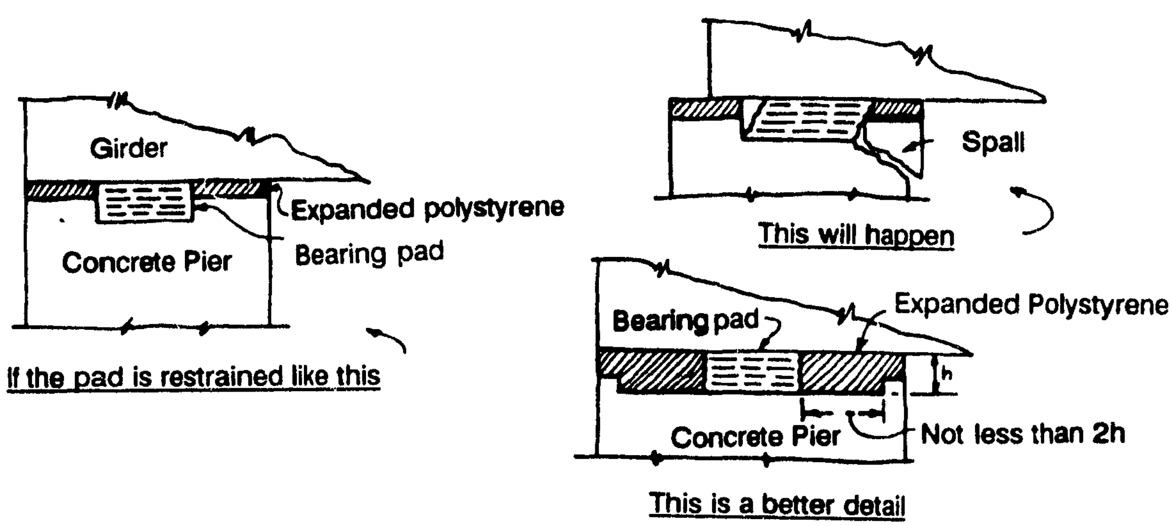


Figure 5.5 Elastomeric Bearing (9)

A special type of elastomeric bearing used to support considerably higher loads is the pot bearing. It consists of a large round elastomeric pad confined by a heavy steel ring, which enables a high load to be resisted by the elastomer. A pot bearing essentially resists rocking movements, and therefore, it avoids the need for precise finishing of the bearing surfaces and other mechanical arrangements. It has also been used to resist horizontal movements, in which case, it is in the form of a sliding pot with stainless steel plates and PTFE pads. As shown in

Figure 5.6, the elastomer is confined in a steel cylinder and a load is applied through a close fitting cutting plate. The steel plate slides on the PTFE to accommodate expansion and contraction. The elastomer within the pot acts as a fluid distributing the pressure equally over the base and also accommodating rotations upto 1:50 (9).

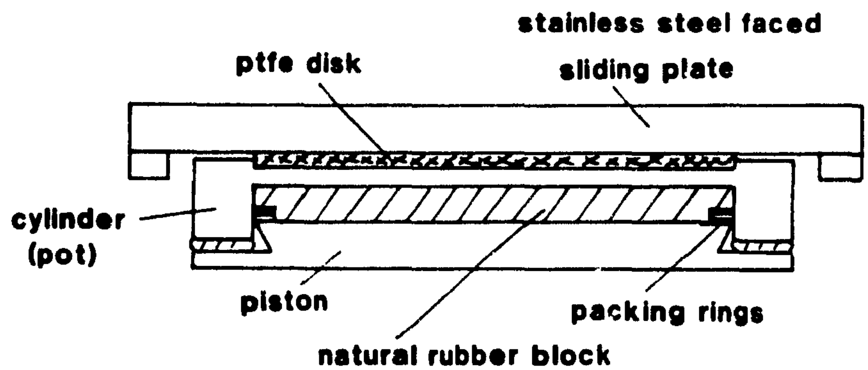


Figure 5.6 Pot Bearing (17)

5.2.5 Spherical Bearings

A spherical bearing is similar to a pot bearing and utilises PTFE concave or convex surfaces as shown in Figure 5.7. The bearing can self-align itself due to the ball and socket arrangement. Also, it has a constant resistance due to the friction on the sliding surfaces and permits large rotations. They are used at the bottom or top of columns to prevent any bending moment being transmitted to the column.

Spherical bearings have had severe problems of freezing and subsequent failure in cold climates. Also, they are difficult to install during erection and the columns as well as the whole bridge requires to be supported to provide stability during erection.

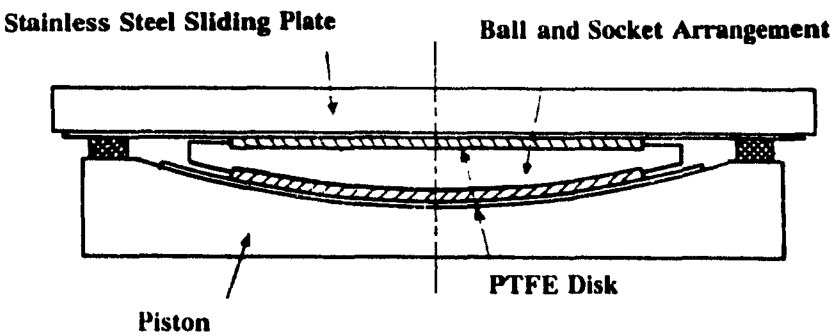


Figure 5.7 Spherical Bearing

5.3 Common Problems with Bearings

Problems with bearings have resulted from a number of causes, some of which are presented below.

1) Freezing

As mentioned earlier, freezing or locking up of the bearings is a common problem for bridges in cold countries. This can restrict the free movement of the structure leading to overstressing of some elements and subsequent failure. Freezing is a common phenomenon in almost all types of bearings and more pronounced in the sliding type. Application of grease on the surfaces has been tried as a possible remedy. However, grease can attract and hold dirt which can eventually find its way between the surfaces.

Freezing of bearings can lead to cracking or spalling of the bridge seats due to overstressing and eventual failure. This failure is called secondary failure and is worse than the initial failure of the bearing.

2) Unequal Resistance to Movements

Since expansion joints serve to accommodate expansion and contraction movements of the deck, their number as well as the design movement to be provided depends upon the span of the bridge. For a long span bridge with a number of expansion joints, the possibility of all of the movement being transferred to one joint due to unequal resistance offered by the bearings cannot be ruled out. This can lead to tearing of the bearing and cracking of the deck. It can be avoided by providing bolts across the joint with compressible material under one end which allows only the design movement after which the bolt tightens transferring the excess movement to the next joint.

3) Effect of Moisture and Dirt

Ingress of moisture and debris has been the most common cause of corrosion of bearings and has been emphasized earlier. Although temporary repairs such as galvanizing is possible, a more permanent repair method is to replace the bearings. Moreover, preventive maintenance is the best alternative to ensure regular functioning of the bearings.

4) Effects of Incorrect Installation

This is perhaps the most important cause of bearing distress. A bearing will perform as desired, only if it is installed correctly. In addition to proper bedding, bearings should be correctly oriented to allow rotation and movement about and along the intended axes.

Selection of bedding material should be based upon the method of installation, the opening size to be filled, the strength and setting time required. Commonly used materials are cement mortar with or without a chemical resin, grout or simply a dry packing. The following preventive measures should be taken during installation of the bearings:

a) All formwork around the bearings should be sealed.

b) While using cast-in-situ decks, care should be taken to prevent any leakage during casting.

c) The bedding should extend over the entire area of the bearing and should be free from any hard spots.

d) Any temporary packing used during erection of the bridge deck should be removed and the voids should be filled with bedding material.

e) Adequate space should be provided around the bearings to allow for inspection and maintenance during service.

f) For abutments or piers that are high or above water, it is convenient to provide a travelling staging in the design to facilitate inspection.

Effect of uneven seating on elastomeric bearings has been explained in Section 5.2.4. Improper preparation of beddings can also cause failure of the bearings. On curved bridges, the expansion and contraction occurs parallel to a chord drawn between the bearings rather than on a tangent to the curve. It is quite possible that the bearings may be installed parallel to the tangent than to the chord which results in failure.

5) Twisting and Wearing

Rocker bearings without pintle pins for the linkage between the rocker and keeper plates suddenly move to one extreme of their movement and lock themselves. In such a case, the bearing is removed and the keeper plates are installed to force the rocker to act and remain in position.

In cases where pintle pins are provided, wearing of the pins due to movement can occur over a period of time. Although this problem is not so serious, it is advisable to either replace the pins or repair the pins by welding and then reboring.

5.4 Recommendations for Design

Although the various aspects involved in the design of bearings are beyond the scope of this thesis, an important aspect of articulation is found to be worth mentioning. Articulation is a means of accommodating the horizontal forces exerted on the superstructure in its design and construction. In other words, it is a procedure of matching the bearing stiffness to the pier stiffness to achieve economic sharing of the longitudinal forces. Day (17) presented detailed procedures for articulation. According to Maurice (34), the following basic principles should be observed:

"1) Where possible, the structure should be free to move with minimum restraint.

2) Bearings must support the vertical loads imposed by the structure and should be positioned so as to permit movement (or restraint as the case may be) without affecting other bearings. If this is impossible, the secondary forces must be taken into account.

3) Structures move transversely as well as longitudinally. For this reason one guide bearing per line of supports is to be preferred to a line of guide bearings. Under transverse loading it is likely that more than one bearing will actually react to the load. Also, if there is any transverse stressing *operation or shrinkage, more than one guide bearing might cause difficulties.*

4) The bearings will only perform as intended if installed correctly Bearings not only need to be properly bedded but they also need correct orientation in order to permit rot. 'ion and movement about and along the intended axis Careful supervision of bearing installation will prevent expensive corrective measures in the future •

5.5 Repair/Replacement Procedures-Criteria for Selection

The decision to implement a particular repair depends on the type of bearing used as well as the nature of distress involved and this can be done readily with the current evaluation procedures and available techniques. However, the decision between repair and replacement is very hard to make and has to be based primarily upon the cause of failure. For example, if failure has occurred due to the inherent characteristics of the bearing, it is more convenient to repair it than replacing it with a similar bearing. On the other hand, replacement is preferred in cases when it is possible to jack up the bridge and slide another bearing and is inevitable when the bearing has served its expected life span. Old bearings are usually replaced with elastomeric bearings. In case the bearing is defective, replacement is absolutely necessary.

Replacement is generally more expensive than repair. It is often necessary to construct a temporary bent in front or on either sides of the pier to jack up the bridge to relieve the bearings. Replacement should be undertaken only if serious deterioration is observed in the bearings, otherwise they can be cleaned, reconditioned and reset back to position.

5.6 Some Retrofit Procedures to Withstand Seismicity

It has been pointed out earlier that during earthquakes, bridges are likely to slide and slip over the bearings or lifted off in the event of seismic loading with a strong vertical component. Robinson et al (51) have developed procedures to prevent uplift at a pier one of which is shown in Figure 5.8. Vertical restraining rods are provided between the girders and the pier wall to limit any vertical separation at the bearings and also eliminate bearing instability and thus loss of support to the superstructure. However, this technique is used only in cases where there is sufficient gap under the girder to accommodate the restrainer details. Also, this technique cannot prevent any possible relative horizontal movement at the bearings.

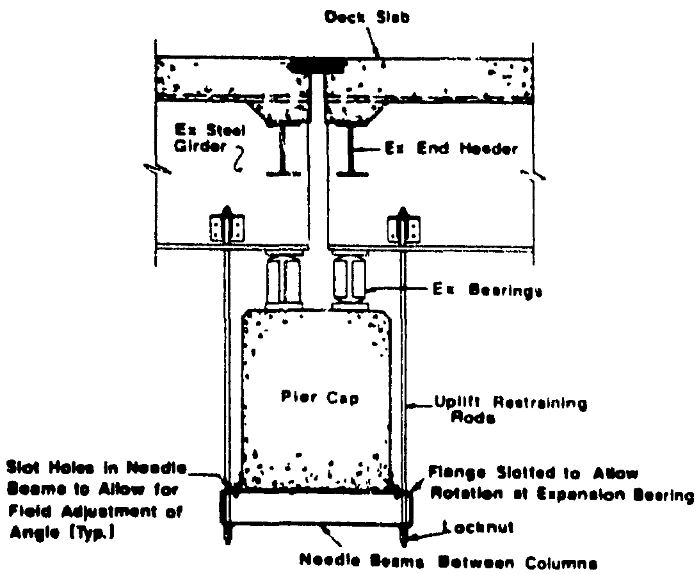


Figure 5.8 Vertical Restraint at a Pier (46)

Another procedure to restrict the relative longitudinal motion of the abutment or pier at the bearing is shown in Figure 5.9. A stopper consisting of a beam or beam grillage is attached by anchor bolts which are drilled and grouted into the abutment or pier. The extent of anchorage and the number of bolts is calculated to develop the required longitudinal force. It is effective only if the concrete in which the bolts are anchored is in good condition.

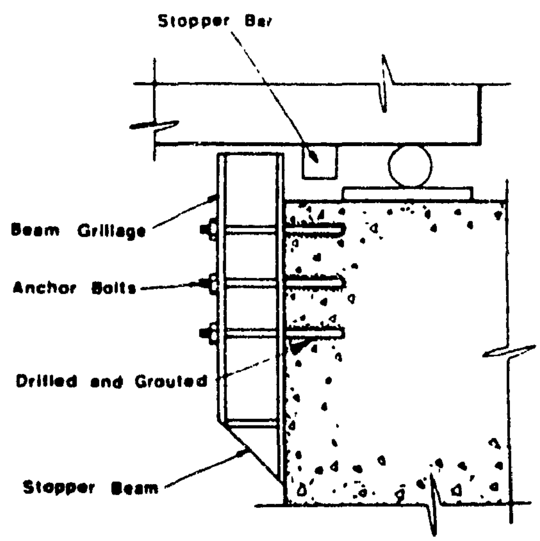


Figure 5.9 Longitudinal Motion Restrainer (46)

In case of box girders, the connection between the adjacent ends is achieved through an internal hinge. The longitudinal motion between the ends at the hinge can be restricted by installing restrainer units as shown in Figure 5.10 (18). A minimum of one 7-cable unit is generally placed in each interior cell at each hinge to provide the necessary resistance to the transverse bending of the superstructure. Concrete bolsters are provided as shown to distribute the concentrated forces of the restrainers thus preventing any damage to the diaphragms. The cells can be accessed either through the soffit to avoid any obstruction to the traffic or if not, through the deck openings.

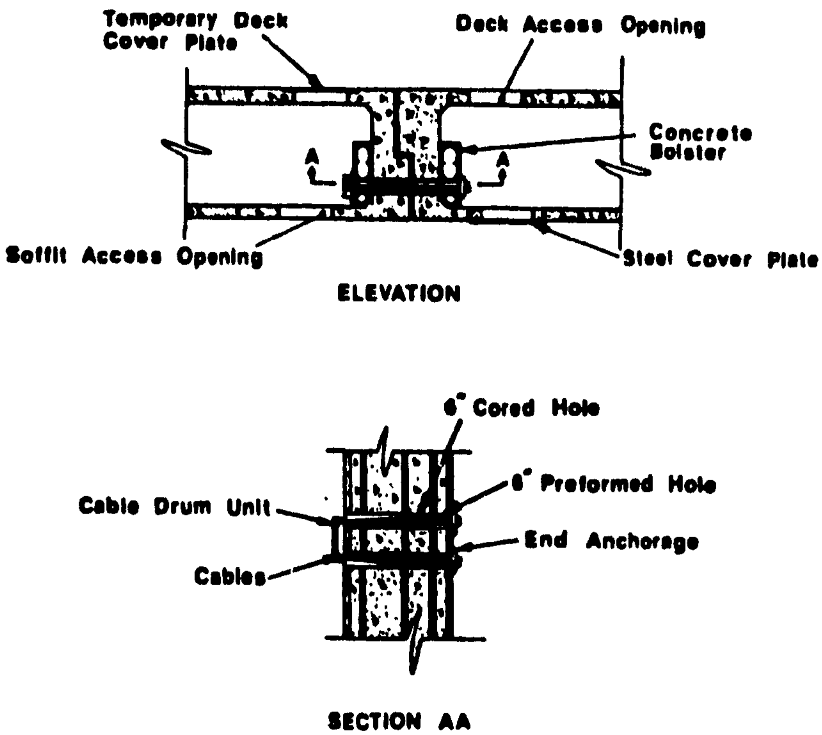


Figure 5.10 Hinge Restrainer for Box Girder (46)

An effective procedure to provide extra bearing width at a support in the event of fall of the superstructure from the bearing is illustrated in Figure 5.11 (50). The extra bearing width necessary is calculated based upon the anticipated longitudinal displacements. As shown, holes are drilled into the existing support to a calculated depth. The reinforcement is inserted and grouted into the holes followed by placing of the forms and pouring of the new concrete. It is necessary for the concrete to be sound for the grouted reinforcement to develop the required strength. Furthermore, this retrofit procedure can be used only where there is sufficient access to the bearing seats.

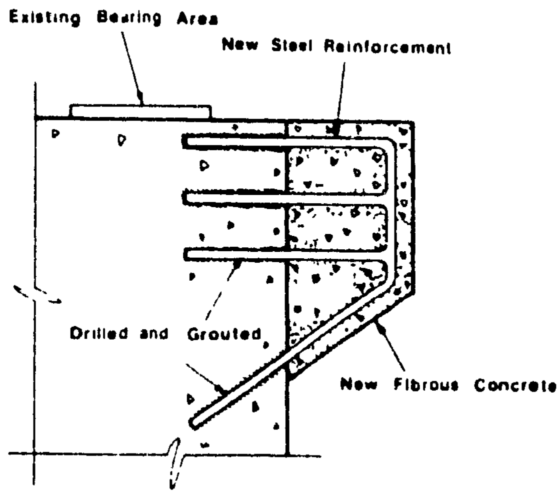


Figure 5.11 Widening of Bearing Area (46)

CHAPTER 6
SUBSTRUCTURE SYSTEMS

6.1 Introduction

A bridge substructure consists typically of piers, abutments, and foundations. The foundations may either be pile or well foundations depending upon the soil conditions. Most of the problems encountered with substructures may be grouped primarily into two major types: natural deterioration and impact from external forces. This chapter deals with the various parts of the substructure along with the associated problems and any recommended repair and/or rehabilitation procedures. Special emphasis is placed on the deficiencies in the substructures below the water surface including scour around piers and abutments, deterioration due to chemical attack, and distress due to structural damage caused by construction, collision, abrasion, or storms. Structural failures resulting from overload, foundation failure, maintenance failure have also been reviewed besides presenting a state-of-the-art of repair techniques for these damaged elements.

6.2 Concrete Deterioration

Deterioration of concrete in bridge piers and abutments is dependent primarily on the location of the water level. Four basic zones have been defined as shown in Figure 6.1 (35).

1) Submerged Zone: This zone lies between the mean low water (MLW) level and the mudline and often a few feet below the mudline, and it is submerged at all times.

2) Tidal Zone: This zone lies between the MLW level to the mean high water (MHW) level. This zone is exposed alternately and submerged twice a day.

3) Splash Zone: This zone lies above the MHW level, and it varies depending upon the size of the waves striking the structure.

4) Open Zone: This zone consists of the remainder of the supporting structure and the superstructure. It may be affected by humidity and water spray.

Deterioration of concrete occurs primarily in a zone above the level of the low tide and below the top level of the splash zone which represents the **Primary Active Zone** (49). A **Secondary Active Zone** consists of the area of concrete at the mudline where deterioration may occur due to abrasion, reaction with the soil, or macrocorrosion.

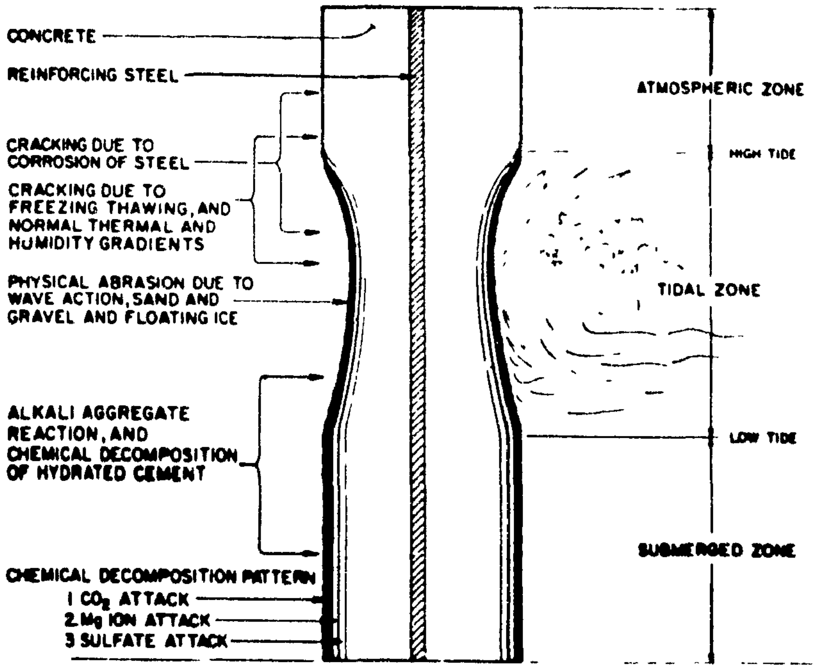


Figure 6.1 Basic Zones of Deterioration (35)

Deterioration of concrete can be attributed to several factors. These can be divided basically into two categories: physical and chemical effects. Mehta and Gerwick (36) have proposed a comprehensive list of all of the physical and chemical causes of concrete deterioration as outlined in Figures 6.2 and 6.3.

A discussion of the common types of deterioration follows.

1) Corrosion

Concrete exposed to a maritime environment can deteriorate mainly due to chloride ingress into the concrete cover to the reinforcement. It is generally agreed that in a situation where no carbonation of good concrete has taken place, there is a low risk of corrosion when chloride ion concentration is less than about 0.4 percent of the cement content. In poor quality concrete, in damp conditions, a chloride ion concentration of about one percent of the cement content can be a high corrosion risk. In saturated concretes with high levels of chloride ions, it has been found that corrosion of reinforcement takes place where the corrosion product volume is unlikely to be sufficient to bring about cracking and spalling of the surrounding concrete. Corrosion can be detected through electropotential observations which are capable of defining areas where chemical conditions are conducive to reinforcement corrosion. Corrosion is not always present in such areas, however, it can take place at a number of isolated locations.

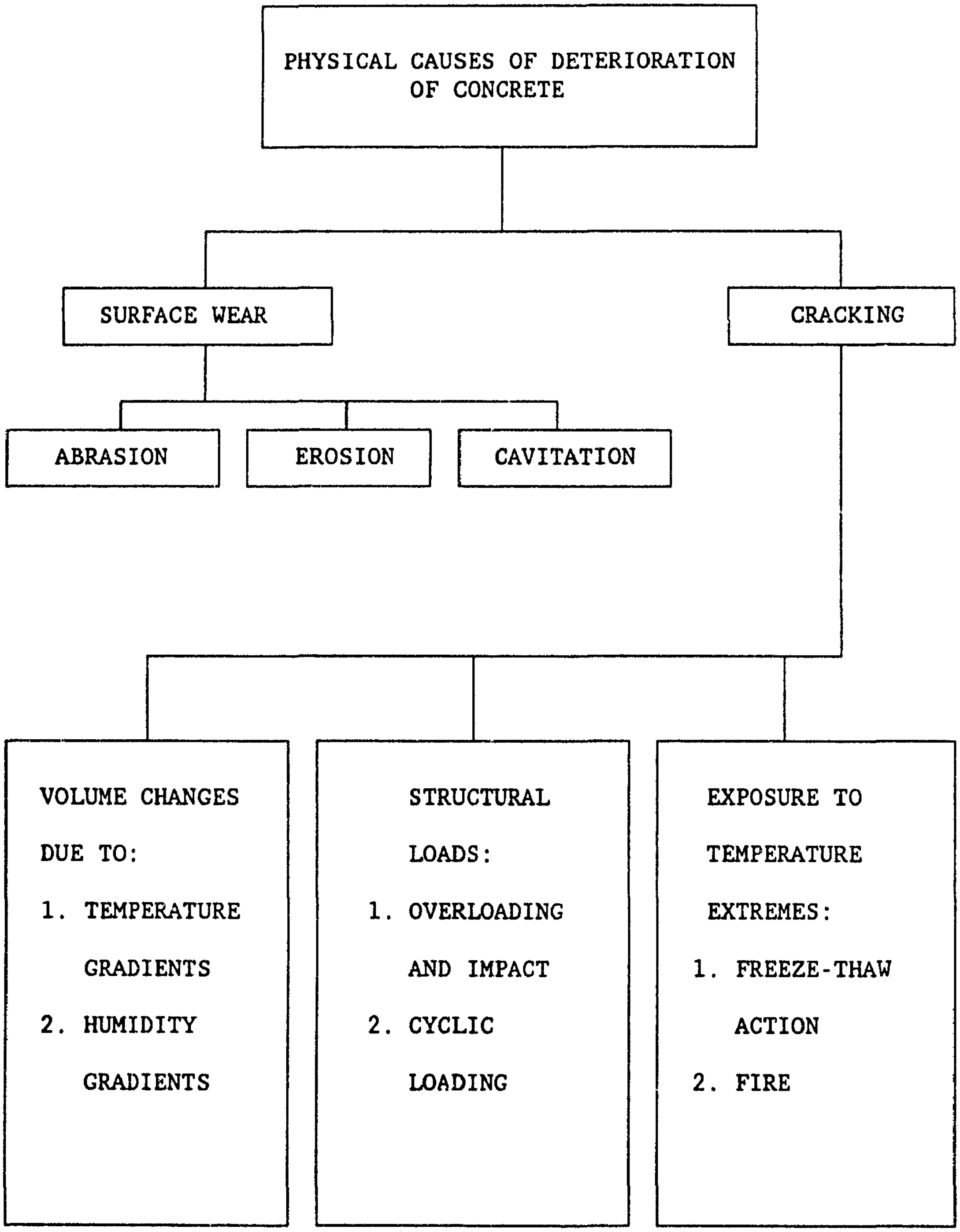


Figure 6.2 Physical Causes of Concrete Deterioration (36)

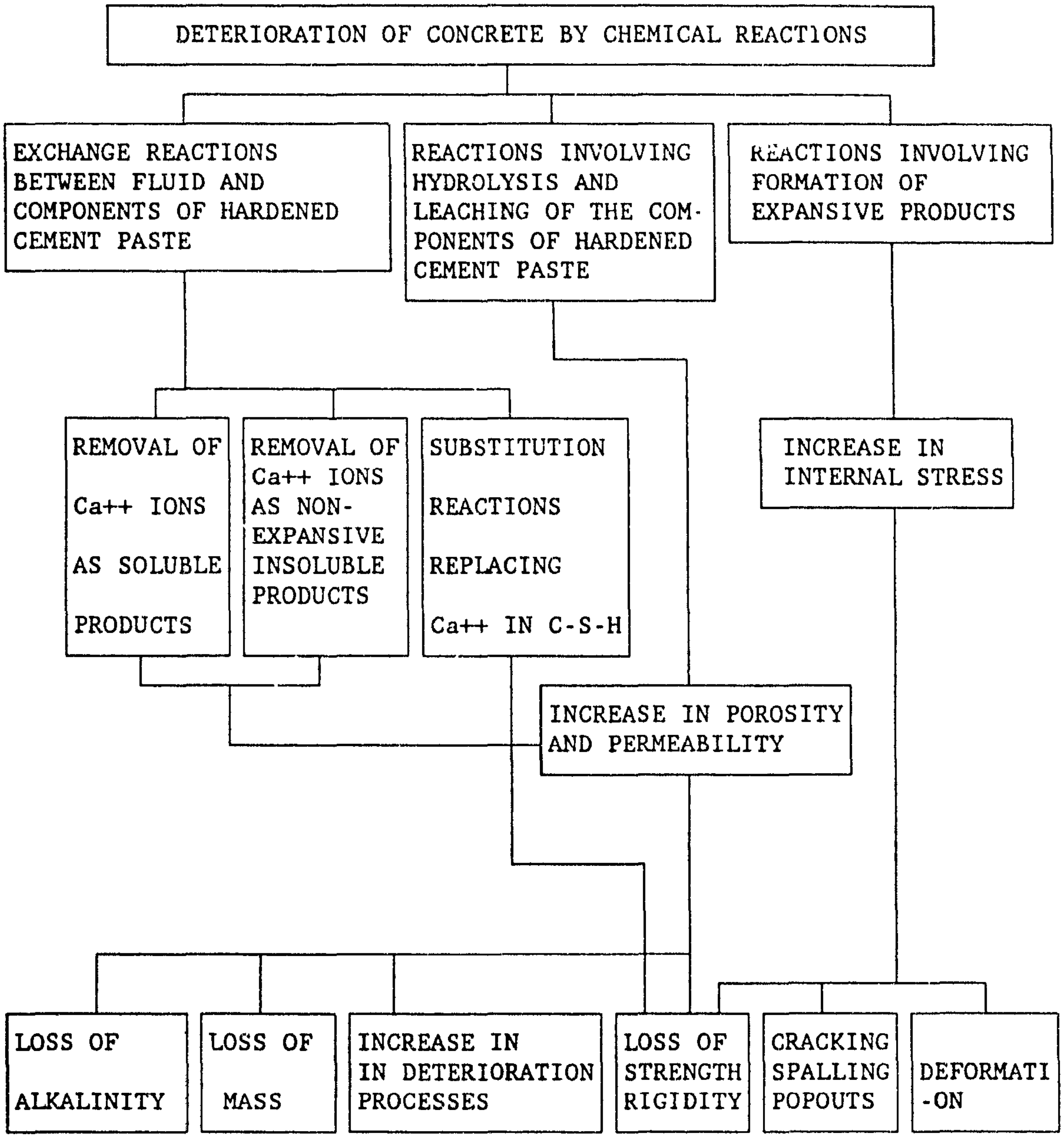


Figure 6.3 Chemical Causes of Concrete Deterioration (36)

Other factors which can influence corrosion of the reinforcing steel are decrease in the electrical resistivity of the concrete cover and the availability of oxygen. In fact, all of the factors interact with each other and determine the time of commencement of corrosion, its rate, intensity and effect.

2) Chemical Attack

Although chemical attack is a slow process, it can destroy the concrete cover over the reinforcing steel. Chemical processes occur frequently in a marine environment, for example, magnesium ions in sea water salts attack concrete by reacting with the calcium. In addition, sulphate solutions react with tricalcium hydrate, a common constituent of concrete, forming calcium sulphoaluminate hydrate, which causes substantial expansion leading to cracking and spalling of the concrete. Other possible causes of concrete deterioration are due to attack from the acid generated by the bacteria and by chemical reaction within the concrete mass. Chemical reactions within the concrete occur between the alkali and the aggregates. There are basically two types of alkali-aggregate reactions: alkali-silica reaction (ASR) and alkali-aggregate reaction (AAR). The ASR is more common and serious, with the reactive form of silica in the aggregate reacting in-situ with the highly alkaline pore solution in the mortar to form alkali-silica gels, which are capable of unlimited swelling as they absorb water. As the water in the pore solution diffuses to the alkali-silica gel, osmotic or swelling pressure exerts stresses which cause disruption of the concrete. Expansion of concrete due to AAR depends primarily on the amount of alkali in the mortar fraction of the concrete, the amount and the nature of the reactive aggregates and the amount of available water. In a marine environment, the available water is always substantial which adds to the problem.

3) Freeze-Thaw Action

This mechanism is commonly encountered in cold climates and is most active at the waterline. It is initiated at low tide when the wet concrete in the tidal zone is exposed. The water in the wet concrete freezes and expands, thereby creating large internal stresses. This cycle is repeated as this ice melts due to the rise of the tide. According to Philleo (44), with the assumption of the use of durable aggregates, the vulnerability of mature concrete (compressive strength of at least 24 MPa) to freezing is a function of the pore structure of the cement paste and the moisture content of the concrete when it is exposed to freezing. Lower strength concretes are also vulnerable for similar reasons, however, they may be even more susceptible to damage because of their higher porosity and lower strength.

4) Carbonation

Carbonation is usually not a significant problem in a marine environment. Carbon dioxide occurs both in the air and in the seawater surrounding the foundation. In most cases, the amount of carbon dioxide in the air is small being of the order of 0.03 percent by volume of the air. In normal seawater, only small amounts of carbonates and bicarbonates are present, being about 10 and 80 parts per million, respectively, along with small amounts of free dissolved carbon dioxide. The average pH value of sea water is around 8.2 when the seawater is in equilibrium with the carbon dioxide in the atmosphere. Under these conditions, some carbonation of the hardened cement may occur accompanied by minor leaching of lime from the concrete surface.

6.3 Steel Deterioration

Corrosion is the primary cause of steel deterioration. It can be either due to atmospheric exposure, water, soil, chemical action or electrolysis. The steel is converted into a compound form through chemical or electrochemical action which falls off subsequently causing a loss of section. Corrosion is seen in the form of a reddish brown rust with a pitted oxidized surface showing loose flakes.

A schematic model on cracking-corrosion interaction is shown in Figure 6.4 (36). It indicates that an increase in concrete permeability either through enlargement of microcracks in the concrete or other causes (leaching of cement paste), causes significant corrosion of the embedded reinforcing steel. Once this corrosion accelerates, expansive products of the corrosion reactions will tend to enlarge the microcracking zone further, thus creating favourable conditions for the progress of corrosion, which can eventually lead to serious deterioration of both steel and concrete.

Atmospheric corrosion occurs above the primary active zone due to the presence of moisture since oxygen is available. Within the primary active zone, corrosion occurs mainly due to the action of chloride ions (in the sea water) and in cold climates because of freeze-thaw action. A chloride ion concentration of about 0.2 to 0.3 percent by weight of cement is generally considered sufficient to depassivate the steel (12). In situations where the structure is below water, corrosion is initiated mainly due to the presence of dissolved oxygen. The rate of corrosion can increase if the velocity of water is high since this brings more oxygen in contact with the steel. Chemical compounds, if present, may also react with the steel causing corrosion. In case of a buried structure, corrosion may be due to either acids, moisture or electrical conductivity. The presence of chlorides can accelerate the deterioration process due to chemical action and freeze-thaw effects.

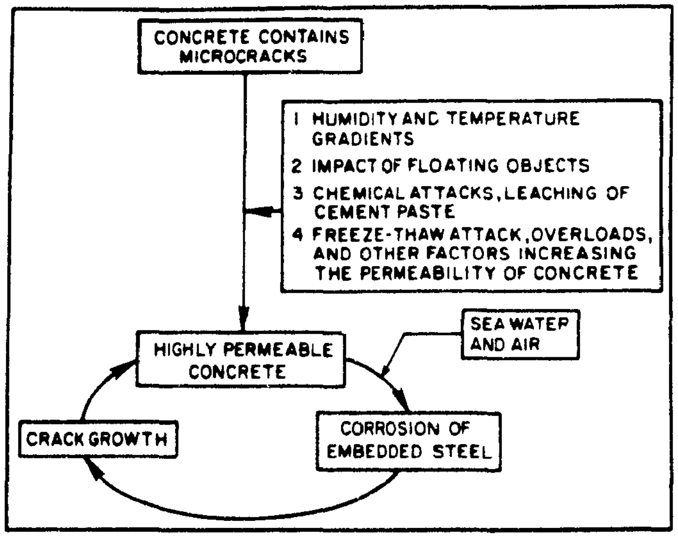


Figure 6.4 Cracking-Corrosion Interaction--Schematic Model (36)

6.4 Characteristics of Deterioration

The characteristics of deterioration of concrete can be seen in the form of its cracking, spalling and decomposition. These may develop individually, simultaneously or in succession and may be present in each zone of deterioration in varying and different degrees of development. Cracks may vary in width and length and may be disastrous at times allowing corrosive elements to enter the concrete and corrode the reinforcing steel. This may lead to spalling of the concrete matrix. Decomposition of concrete occurs where the binding capacity of the cement is lost causing loss of the matrix (cement and sand) leaving exposed aggregate. Ocassionally, this may cause a complete loss of cross-section. A whitish calcium residue can often be seen in the aggregate pockets or depressions.

Cracking and spalling are usually seen in the submerged zone and may be attributed to a poor grade of concrete, chemical attack, impact, faulty construction, etc. The submerged zone may also show occasional abrasion due to scour and decomposition, if the water is polluted with chemicals.

The tidal zone is prone to the most serious problems of deterioration, especially, since it is influenced by tidal and wave action in addition to the problems existing in the submerged zone.

The environment created in the zone results in a more aggressive chloride corrosion, weathering, and ice abrasion. Cracking and spalling can also be seen at an early stage of deterioration.

The exposed zone may also show either a combination or individual effects of cracking, spalling and decomposition which may often be as severe and extensive as those found in the primary active zone

It is necessary to understand the nature of all zones described, with the limitations imposed by repair location, structural configuration, working environment and the available repair techniques, to develop and implement an effective repair method.

6.5 Structural Damage

Structural damage can occur in a bridge substructure due to a number of causes. Accidental impact from vessels can damage the substructure which can range from a minor localized damage to a complete structural breakdown. A fair measure of control over this type of damage can be achieved by providing protective fendering systems around the substructure. These are reviewed in Section 6.9. Moreover, preventive maintenance is the best solution in any case which involves a regular inspection of the substructure to detect any minor or major damage which should be rectified immediately. Improper construction procedures may also lead to occasional damage. Some examples are summarized here for completeness (30).

"*1) Often pieces of steel are left protruding from the finished concrete surface, including hardware used to secure the reinforcing steel and formwork in place or lifting rings not removed from the precast concrete piles. With time, the corrosion of these metals will cause these cracks and spalls in the adjacent concrete area.*

2) During setting of the concrete, cracks will appear in the structure if the formwork moves or as a result of vibrations caused by pile driving, blasting, poor workmanship, etc.

3) The concrete in the substructure, if not properly placed, is subject to cracking caused by differential settlement of the concrete suspension and by initial and drying shrinkage.

4) Stresses resulting from changes in the atmospheric temperature or in the internal temperature of the concrete mass can cause cracking.

5) If the shoring system or framework is removed prematurely, the concrete can crack severely.

6) When the reinforcing steel is placed with insufficient cover, corrosion may occur."

Piles may buckle or crack if overdriven or improperly handled. Damage to foundations due to scour, rotation of piers, etc., may cause a change in grade or alignment of the deck, or the roadway surface.

Another type of substructure deterioration can result from abrasion due to ice, sand, silt, etc. Ice floating up and down with the tides can cause section loss at the waterline The action which causes the loss of concrete surface can vary from simple rubbing or abrading action to the disruption of the concrete surface through microcracking caused by repeated impact from floating objects, followed by loss of the material as the damaged concrete is plucked away by water or floating ice action (26). At the mudline, sand movement can cause a similar problem. In high velocity currents, suspended particles of sand and silt are predominant in causing deterioration. Abrasion may be supplemented by freezing and thawing, chemical attack, carbonation, and corrosion which aggravates the problem.

An important cause of substructure deterioration is due to scouring of the water bed material due to currents. Floating debris carried downstream by the water flow may exert horizontal forces against the substructure. Also, debris accumulation at bridges increases the scour potential by concentrating the flow. Waves generated by strong winds induce hydrodynamic pressures on the substructure components and piles. A detailed discussion of the type and nature of the scour and the related repair measures are summarized in Sections 6.6 and 6.8, respectively.

6.6 Scour Related Damage

Scour is a natural phenomenon that occurs due to the erosive action caused by stream water and by tidal flows and waves in coastal areas. The material from the stream bed supporting the foundations as well as the banks and slopes is removed and carried away seriously disabling the bridge substructure. Chang (11) noted that the bridge abutments followed by the piers and subsequently the superstructure are the most seriously damaged parts. Hopkins et al (27) have categorized scour into three types:

"*1) The **general scour** that would occur in a stream with or without a bridge crossing.*

*2) The **contraction scour** that may occur generally at the bridge waterway because the flow is contracted by the bridge crossing.*

*3) The **local scour** that occurs because of the distortion of the flow pattern in the immediate vicinity of the bridge piers and abutments.*

It has been observed that general and contraction scour contribute significantly at a bridge pier than local scour."

A detailed description of the three types of scour is presented by Jones (28).

Scour causes degradation which may lower the stream bed considerably, thereby affecting the hydraulic design of the bridge. Degradation usually occurs in cases when river control works are installed in which case it is absolutely necessary to check the vulnerability of the bridge. On the other hand, if there is an aggradation or deposition, the water level increases which may cause the hangup of drift on the lower members of the bridge. This restricts the waterway area under the bridge and increases the capacity of the flow locally to transport sediments. This causes local scour which in some cases may be more than aggradation. There is also a possibility of increase of the flood flow encroaching on the flood plain due to deposition in river channel which may add to the local scour at the abutments.

The embankment fills of the highway crossing generally create a severe contraction of the river in flood. This causes the flow to move laterally to the bridge openings. According to Jones (28), "*If the flow returns to the channel largely in a reach of some length upstream from the bridge, there will be general scour over the entire waterway opening If, however, the flow returns along the embankment, there will be severe scour at the abutment and possibly out to the first or second pier, with the general scour taking place downstream from the bridge*" Local scour can also occur at the piers and abutments as mentioned earlier due to their inherent presence which may change the local flow pattern.

Scour related damage needs immediate repairs to avoid any further aggravation of the problem. Various repair procedures have been developed over the years depending upon the nature, type and severity of the damage. These are discussed in Section 6.8.

6.7 Repair Techniques

A wide range of techniques of repair is now available, although careful evaluation of the problem is required before the most appropriate repair technique can be selected. Most repairs involve sealing cracks, patching, depositing concrete or grout to fill cavities, and encasing reinforced concrete members such as piers or piles. The basic steps in any underwater concrete repair are:

1) Remove all marine growth over the deteriorated portion.

2) Remove all deteriorated concrete around the affected bars and all corrosion products from the existing bars.

3) Treat all exposed and new reinforcement with epoxy and restore to original condition.

4) Seal any cracks present in the remaining concrete by injection.

5) Reinstate the concrete section by casting of a new face on the concrete.

6) Apply protective surface treatment to the concrete.

Repairs have been grouped into three basic categories: patch repairs, primary repairs, and primary repairs involving bar replacement. In general, patch repair involves filling of reasonably small areas using a polymer modified sand-cement mortar (a ratio of 3:1 is normally used). If a patch repair requires a very quick setting time because of the tidal restrictions, this can be carried out using more costly epoxy mortar or quick setting cement. Primary repairs involve the implementation of formwork and pouring of a concrete mix to form a new profile. It is now widely recognized that the best protection for steel reinforcement is to have the steel bar covered by sufficient impermeable concrete, or mortar, although in some circumstances other solutions such as waterproof membranes or polymer impregnation may be more appropriate. It is more important to choose proper constituents for the replacement mix which serves as the protective barrier to the reinforcement. A blended cement mix with slag can be effective in retarding the inward advance of carbonation. It also acts as an alkali diluent and improves the sulphate resistance of the concrete along with a reduction in the heat of hydration. Furthermore, the presence of slag decreases the chloride ion diffusion coefficient, thereby improving the resistance of the concrete to chloride attack. It has been noted (52) that the use of separately granulated iron blast-furnace slag, as a separate cementitious material or supplement has produced significant reductions in permeability when the slag replacement level is about 65 percent. A superplasticizer can be used in the concrete mix to reduce the water whilst still maintaining the workability and improving compaction.

An underwater repair may be either **Dry** or **Wet**. A dry repair involves the construction of a portable barrier around the structure with dewatering facility to create a dry working environment for the repair. A wet repair does not require the construction of barrier, however, it requires a high level of competence, sophisticated techniques, and high quality materials unlike for the dry method which is less complicated, and provides greater flexibility in the use of construction materials and repair methods.

Dry methods are classified based upon the type of barrier used which may be a cofferdam, encasement or a caisson. Wet methods can be classified according to the method for placing the concrete which may be a tremie (pipe or bucket), pump, preplaced aggregate concrete, hand placed concrete or bagged concrete. The following procedure is generally followed while placing concrete using a wet method:

1) Concrete should not be placed in running water nor allowed to fall through water to prevent the washing away of the cement from the concrete.

2) The temperature of water during the placing operations should be above 5^0C.

3) The concrete should be workable with slump of around 200 mm and cement contents of upto 390 kg/m^3.

4) The aggregates should be free of fines or other materials which may cause excessive laittance.

An underwater bucket is provided with a deep bottom or roller gate opening which can be opened from above water. The bucket is filled with concrete, covered, and then lowered to the desired position, and then the concrete is allowed to flow The operation is repeated for subsequent buckets with care being taken to ensure that the gate of the bucket sinks into the previously placed concrete to prevent it from falling through the water.

In case of the tremie method, a pipe with a plugged discharge end is used for placing the concrete A funnel is provided at its receiving end. The pipe is gravity filled with concrete through the hopper. The discharge plug is kept sealed until the pipe is completely filled with concrete. The discharge end is always kept immersed in the newly placed concrete during the placing operation to prevent any water or air from entering the pipe (45).

Another method of placing concrete is using prepacked concrete which has been successfully used in situations where concrete placing was difficult to perform. In this method, prepacked concrete is prepared by filling watertight forms with coarse aggregates placed around vertical pipes through which the grout is pumped slowly from the bottom up, displacing water and filling the voids in the aggregate The grout is a slurry mix of fine sand, Portland cement, mixing water, a pozzolanic material to lower mixing water requirements, and an agent designed to increase fluidity and inhibit early stiffening. Specifications for the coarse aggregate and grout are controlled to suit the placing Grout pipes should be spaced not more than 1.5 m. Also, a positive head of at least 1.2 m above the level of the outlets should be maintained in the grout pipes until the forms have been filled and the grout has set.

Perhaps the most popular and simple method of placing underwater concrete involves the use of pumps. In this method, concrete and/or grout is pumped into the form from the bottom or from the top by inserting multiple pipes from the top right upto the bottom. In the latter case, the pipes are raised as the concreting proceeds and the concrete level rises. The forms are filled until they overflow and are allowed to settle for about 30 minutes when they are refilled until overflowing.

Another method of placement is using jute sacks partly filled with concrete. The bags containing a normal mix concrete with a low water-cement ratio are placed in position by divers in such a way that the whole mass is in perfect bond. This method has found use in protecting structures from physical damage as well as scour damage.

Rissel et al (49) have grouped the repair techniques for different stages of deterioration, viz. initiation stage, propagation stage, and destruction stage. In the initiation stage, only the surface concrete is affected by the chemical and the mechanical forces, whereas in the propagation stage, the entire concrete cover is affected. This involves cracking and scaling and combined with the already existing porosity, it allows the chemical and the mechanical agents to reach the reinforcing steel and begin the corrosion process. In the destruction stage, corrosion of the reinforcing steel is accelerated and the structure loses its capacity to sustain any loads.

6.7.1 Repairs at the Initiation Stage

These include the construction of hydraulic training structures which help reduce abrasion attacks on the concrete. An efficient although expensive method is to install protective systems like fenders and dolphins which are capable of absorbing large amount of energy. An alternate cost-effective technique is to place stone riprap around the foundation. However, this repair is applicable only when the abrasion is at the mudline.

Another technique to reduce abrasion is to reduce the velocity of the stream by ponding of water beneath the bridge, either by means of a downstream dam, or widening the stream itself. However, a disadvantage of this technique is that as the velocity of the water decreases, there is more deposition of suspended solids which again silts the pond and alters hydraulic flow beneath the structure.

Several other methods have been used to direct the flood waters away from piers. These include construction of spurs, dikes, jetties, and channelization.

6.7.2 Repairs at the Beginning of the Propagation Stage

These include the application of surface coatings, surface facings, and cathodic protection of steel.

6.7.2.1 Application of Surface Coatings

a) Concrete

Surface coatings are applied to concrete to act as a barrier in separating the concrete from the corrosive environment. The presence of coatings not only prevents the ingress of chemicals and oxygen but also helps in resisting abrasion, chemical attack, and freeze-thaw effects. The coating should have the following characteristics:

1) Its adhesive strength should be greater than the tensile strength of the concrete.
2) Its coefficient of expansion should be compatible with that of the concrete.
3) Its viscosity should be such that it will provide maximum seating of the surface.
4) It should be elastic and should not creep.
5) It should have a long service life.

A surface coating needs proper surface preparation in order to perform well. However, they can be applied **in-the-dry** using portable cofferdams. These are dewatering forms from which water can be pumped out to maintain dry working condition for repair. Following types of surface coatings are commonly used:

1) Asphalt and Tar Coatings

Asphalt has a high degree of resistance to acids and oxidants. Also, it can be applied cold with the use of a solvent. Tar is commonly used for repairing seawater concrete Although it is an excellent water repellant, it offers very little resistance to acids and bases, and it is also poor against abrasion. Both coatings are applied in two coats with the second coat containing a filler compound such as silica for stiffness.

2) Epoxy Coatings

Epoxy coatings have been used widely in recent years primarily due to their good adhesive properties, low shrinkage, and high compressive strength which develops very quickly. They are mostly inert chemically, impervious, and moisture resistant. They are easy to apply (usually with a roller or brush). However, epoxies have their disadvantages as well They require a strict quality control during mixing. The mix must be used immediately upon preparation and is heat-sensitive during mixing and application. Due to rapid hardening, there is a strain incompatibility and the coefficient of thermal expansion differs from that of the concrete. Epoxies have a tendency to creep, relax, and are impact-sensitive. Since they are impermeable, moisture trapped beneath the coating may cause it to spall off in direct sunlight or under freeze-thaw action. As a rule, epoxies should not be used unless the concrete can resist frost action by itself. A low cost alternative to epoxies is polymerized epoxy. In other cases, epoxy polysulphides have been used which are resistant to solvents and have improved impact resistance.

3) Acrylic Rubber Coatings

An highly elastic acrylic rubber-type coating, known as **aron wall** coating has been used successfully to resist chloride intrusion and carbonation, thus preventing depassivation of the reinforcing steel. The details of its configuration, behaviour, and method of use have been reported by Swami and Tanikawa (62). The coating has shown a dual function when applied to concretes having high levels of sodium chloride and additionally exposed to sea water. The coating can act as an effective barrier to external water and chlorides and also permits sufficient mobility of chlorides inside to achieve a more uniform distribution, without peak concentrations at rebar levels. However, the concrete cover should be more than 30 mm to ensure absolute protection to the reinforcing steel.

b) Steel

Exposed reinforcement can also be treated with a protective coating to prevent any further corrosion. Two types of coatings, metallic and non-metallic, have been used. The metallic coatings commonly used are flame-sprayed zinc or aluminium with the latter showing better performance, especially in seawater. The non-metallic coatings may be of organic or non-organic type. Commonly used coatings in this category are coal-tar epoxy, epoxy, vinyl, rubber, urethane, and several others. The steel is first sandblasted to remove any scale and rust. Any oil on the sandblasted surface is removed by solvent washing to establish a perfect bond of the coating to the steel. A primer coat is applied first, except when an epoxy is used. The characteristics of a good primer have been summarized by Lamberton et al (30). In case of epoxy coatings, a single layer of 3 to 6 mm applied by hand is generally sufficient.

6.7.2.2 Application of Surface Facings

These include standard size bricks and tiles of clay, glass or carbon, kilned at high temperatures, which are applied to the concrete surface. The semivitrified facings are very dense and have a low water absorbtion. The bricks and tiles are joined to the concrete with a chemically inert mortar. Three possible types of mortars have been suggested (49).

"*1) Synthetic thermo-setting cements consisting of phenol, furnace blast, epoxy, polyester, and polyurethane;*

2) Potassium silicane cement - This is suitable for acid conditions but performs poorly in alkaline and fresh waters; and

3) For less corrosive environments, modified hydraulic cements, such as natural rubber latex and synthetic resin emulsion, may be used."

6.7.2.3 Cathodic Protection of Steel

This technique has already been discussed in Section 2.3.4 for deck slabs. It can also be used to prevent corrosion of steel reinforcement in a marine environment. The galvanic anode system uses zinc, magnesium, or aluminium anodes that are located either adjacent to the foundations or directly connected to them. In this case, the foundations act as the cathode. No external power source is needed. The difference in potential between the anode and the foundation creates a corrosion cell and allows the current to flow from the anode to the foundation. The anode gets used up after a period of time and needs a replacement. The current output is limited in this system and large number of anodes are needed if the structure is large. However, it can be distributed evenly in case of a long structure. Also, the cathodic interference is very small unlike in an impressed current system.

In the impressed current system, high-silicon cast iron or graphites are used as anodes. The anodes are installed in the electrolyte and are connected to the positive terminal of an external DC power source (Figure 6 5) The negative terminal of the DC source is connected to the foundation to be protected The current necessary in a seawater cathodic protection system is around 5 to 10 mA/ft^2 (54 to 108 mA/m^2) while that for a fresh water or underground application is lower at around 1 to 3 mA/ft^2 (11 to 32 mA/m^2). On the other hand, since seawater has very low resistivity, it requires a low voltage to deliver the current. The most important advantage of using an impressed current system is that it can be designed for a wide range of current and voltage requirements which can be varied to meet the changing conditions of the substructure. However, this system needs to be designed carefully, and inspected and maintained periodically.

A recent development of the impressed current system has been the use of different anodic materials, including conductive coatings and zinc-metallized coatings. These are proprietary water based coatings consisting of a blend of specially treated carbon dispersed in an acrylic resin The properties of this coating have been reported by Clemena and Jackson (14) and have indicated that it is as durable as the best organic-based conductive coating for protecting inland concrete piers. The coating is applied in two layers on the concrete surface with brushes or rollers. In order to eliminate regular inspection and on-site monitoring (as in the case of usual cathodic protection systems), a microprocessor-based data acquisition device has been tested with the system. This device allows convenient monitoring of the condition of the electric components of the system from anywhere using a phone, modem, or a personal computer.

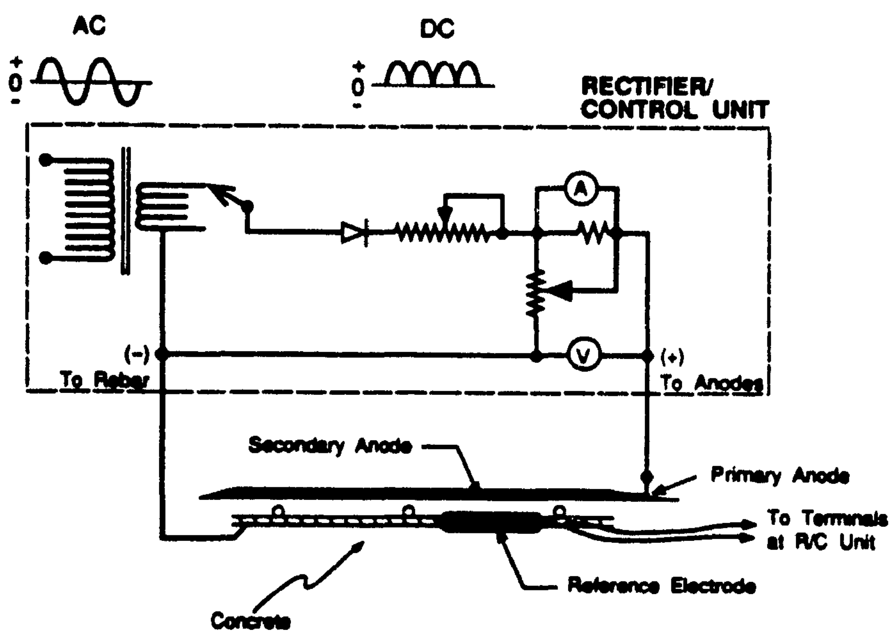


Figure 6.5 Typical Cathodic Protection System (14)

6.7.3 Repairs at the End of Propagation Stage

In cases where the deterioration of concrete has reached the level of the reinforcing steel, it becomes necessary to protect the steel from corrosion, otherwise the structure could lose its capacity to withstand the loads. In pile foundations, plastic or rubber sheets are wrapped around the pile tightly with metal clamps at top and bottom of the pile for insulation from the environment. This consumes the residual oxygen within the trap and stops any further deterioration. Commonly used wraps include neoprene, butyl, nitrile, polyisobutalene, polysulphide, and chlorosulphonated polyethylene. Alternatively, resin-impregnated fibre glass cloth wrap soaked with brushed-on epoxy can be used

Another method of repair is the use of reinforced jackets around the foundations. These are relatively cheap and quick to install. Several types of jackets can be used such as fabric, non-metallic, steel or sheet pile jacketing

A fabric jacket is assembled above the waterline and then clamped around the pile at the bottom. A reinforcing cage is usually placed before clamping. The jacket is then filled with concrete. Fabric jackets are generally durable except in cases of currents and waves when they may deform and reduce the cover over the reinforcement in the jacket. An improved method is the use of non-metallic jackets of fibreglass which are lightweight and easy to handle. These come as a one-piece or two-piece unit which can be bolted or interlocked at the seam(s) (53). They are filled with either epoxy mortar or a combination of cementitious and epoxy mortars. They can be removed or left in place for added protection in which case they are sealed at the top and bottom with epoxy to provide total protection of the pile. Another repair technique is the use of steel jackets which are similar to the fiberglass type. However, in both cases, the jacket may separate from the grout filler material and even get dislodged. The most effective way of jacketing is the use of sheet piling with a backfill. The backfill can be either a concrete grout or earth with a concrete cap.

6.7.4 Repairs at the Beginning of Destruction Stage

Destruction stage is characterized by widespread cracking and spalling of the concrete. The cracks may be either dormant or live. Whereas live cracks may be subject to further movement, dormant cracks are unlikely to open, close, or extend further and may be either fine (up to 1 mm wide), wide (1 mm to 6 mm wide) or fracture cracks (over 6 mm wide). Following types of repairs are common.

1) Pressure Injection of Cracks

In this type of repair, low viscosity epoxy is pressure-injected into the cracks. Another effective method used currently is the use of resin grouts which not only displace the water inside the crack but also react with it. Because of this reaction, there is no residual film of water left between the inside concrete surface and the injection material.

For an above water epoxy-injection repair, the crack is first sealed on the outside using epoxy or tape. Vent ports are inserted at regular intervals to ensure full depth penetration. Injection is started from the lowermost vent port until the epoxy appears at the higher vent port and is completed when the material in the control hose maintains a constant level

In the case of an underwater epoxy injection repair, the cracks are prepared by cleaning with blast water. An epoxy surface sealer is used to seal the cracks. The temperature of the water should not be less than 4°C. The vent ports are installed at regular intervals. A low viscosity epoxy adhesive is injected under pressure into the cracks using a surface mounted pump which mixes the two components of the epoxy at the injection head at a constant pressure. The epoxy achieves full strength in approximately 7 days. According to Lamberton and Sainz (30), "*Cracks upto 6 mm wide are sealed with pure (unfilled) epoxy resin. For wider cracks, the addition of fine aggregate is generally required for a more substantial filler material and improved economy*"

2) Use of Sacrificial Concrete Collar

This method involves casting of a plain concrete collar around the foundation at the waterline which protects the structural concrete by acting as a barrier. However, the collar is subject to deterioration and needs to be replaced repeatedly.

3) Resurfacing Using Grout

In this method, epoxy or concrete is used to resurface large deteriorated areas as well as spalled areas. Commonly used grouts are made with Portland cement concrete along with modifiers and mixtures such as latex modified concrete slurry, high alumina cement, sulphur-impregnated concrete, epoxy mortar grout, gun applied mortar or preplaced aggregate concrete. They can be used along with jackets or formwork. In case of patching areas with exposed reinforcing steel, it is usual to coat the inside of the cavity as well as all reinforcing steel with epoxy bonding compound to electrically insulate the wet patch

4) Repairs to Exposed Reinforcing Bars

Reinforcing bars exposed for up to two years can be prevented from further corrosion by connecting them with a sacrificial zinc anode of approximately 2 to 5 kgs in weight. This is termed as passive cathodic protection.

6.8 Repairs related to Scour Damage

Following are some of the typical repair procedures in use for correcting scour damage.

6.8.1 Concrete Jacketing and Riprap

This repair is suitable for scour under pile foundations supporting pier footings and it involves the following steps.

1) Construction of a cofferdam around the pier and pumping out all of the water inside it.
2) Construction of formwork for a concrete subfooting around the original footing and piling.
3) Casting of the subfooting concrete.
4) Removal of the cofferdam and placing of stone riprap around the subfooting as shown in Figure 6.6. In cases when the scoured material is fine grained, a properly designed gravity filter should be placed prior to installing the riprap.

Care should be taken during placing the riprap. Firstly, it should be placed in even lifts around the foundation to avoid any unbalanced forces against the structure. Also, the structure should be analyzed to check whether or not it can withstand the additional load of the riprap. The riprap should be placed carefully to avoid any damage to the concrete. Finally, whenever possible, riprap should not be placed above the original stream bed since this may obstruct the stream flow.

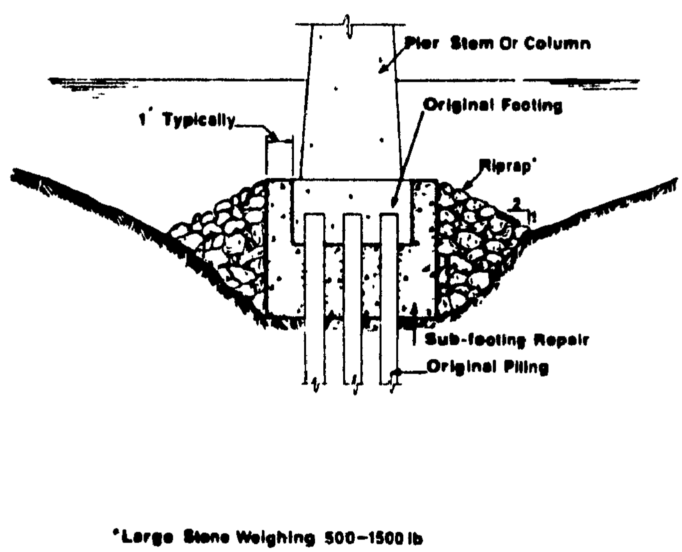


Figure 6.6 Concrete Jacketing and Riprap (49)

6.8.2 Flexible Tube Forms

In this technique, flexible nylon tubes filled with structural mortar are used. The tubes are cut in suitable lengths and joined together by a high tensile nylon stretching. They are then wrapped around the scoured area of the footing besides being extended beyond. Grout injection pipes are installed as shown in Figure 6.7 (a and b). Mortar is pumped into the void under the footing or foundation. After filling the larger voids, the smaller voids between the footing and the form are plugged using smaller fabric forms as shown in Figure 6.7 (b). The voids between the footing and the stream bed are also filled by pumping mortar through the injection pipes. It is advisable to provide several pipes to allow water to escape from the void during placement of the mortar.

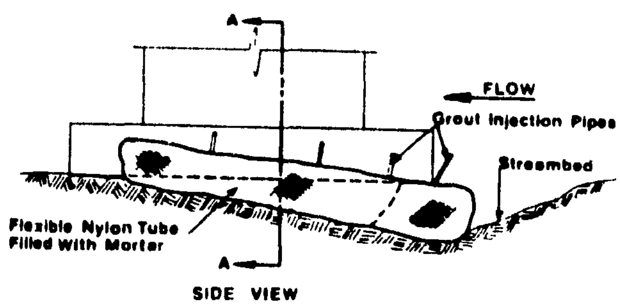


Figure 6.7 (a) Flexible Tube Forms (49)

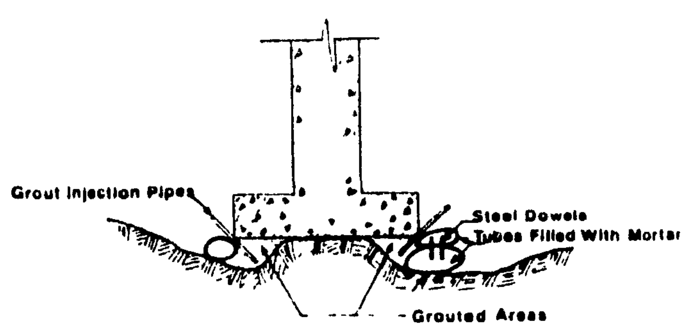


Figure 6.7 (b) Section AA (49)

6.8.3 Tremie Concrete Subfooting and Concrete Riprap

In this technique, a cofferdam is built usually around the scoured footing and the area is dewatered. Concrete riprap in bags is placed around the damaged area upto a level slightly below the existing footing. The resulting form that is created is filled by pumping tremie concrete as shown in Figure 6.8 (a to c).

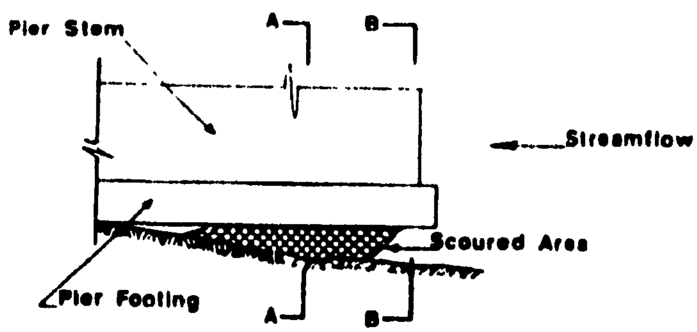
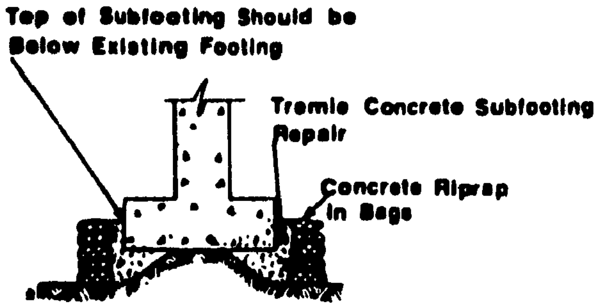


Figure 6.8 (a) Concrete Riprap and Tremie Concrete Subfooting-Partial Elevation (49)



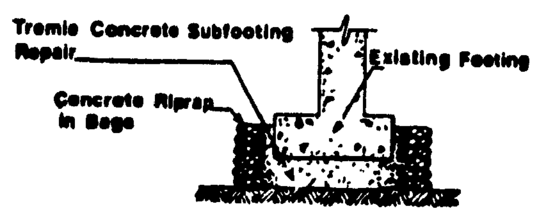


Figure 6.8 (b) Section AA -After Repair (49)

Figure 6.8 (c) Section BB -After Repair (49)

6.8.4 Repairs for Heavily Undermined Streambeds

It is possible that changes in the characteristics of the stream flow may occur which may at times cause the whole stream bed to erode as seen in Figure 6.9. This happens when, for example, during the peak runoff periods, the relief of a downstream constriction in the stream channel can increase the velocities at the upstream end resulting in erosion. The repair procedure involves rebuilding the entire eroded stream bed with a half a meter deep crushed stone subbase topped with a heavier stone riprap upto the level or the original streambed. However, the use of this size of crushed stone may be prohibitive if the stream currents are strong. Flexible tube forms can also be used if required.

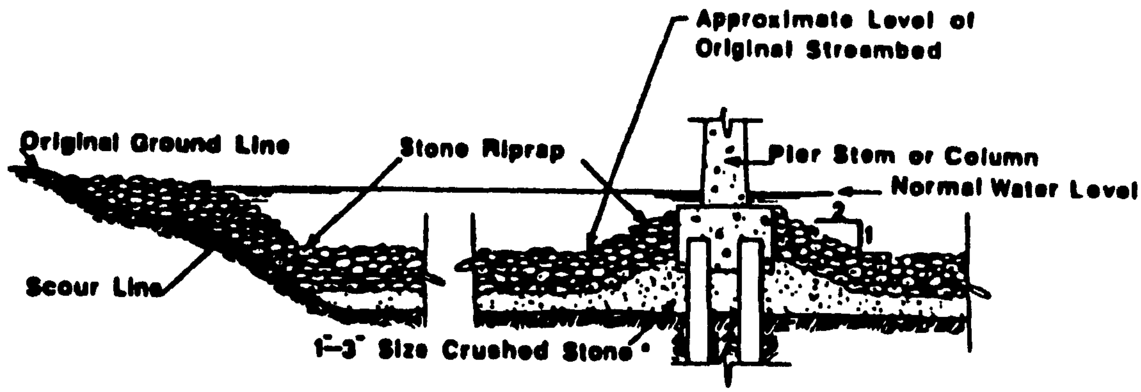


Figure 6.9 Rebuilding of Undermined Stream bed (49)

6.8.5 Repair of Abutment Foundations

Abutments are normally positioned at higher elevations than pier foundations and often need repairs in the case of undermining at the pier foundations. For abutments bearing on soil, a new concrete subfooting is cast surrounding the original footing and tied to the original footing by driving machine bolts through predrilled holes at around half meter centres as shown in Figure 6.10 (a). Bolting is not necessary for abutments on pile foundations as shown in Figure 6.10 (b). However, in both cases, stone riprap can be used to protect any future scour.

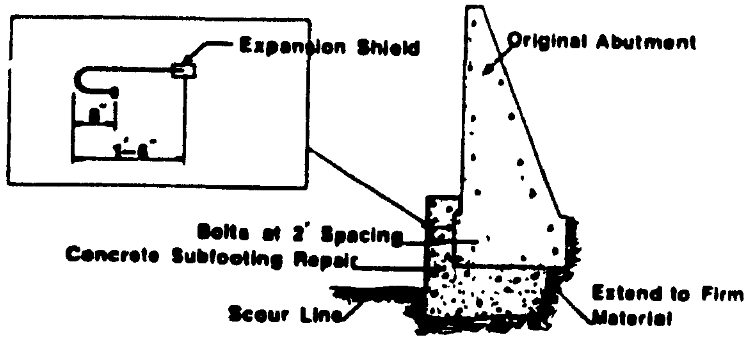


Figure 6.10 (a) Abutment Repair-Soil Bearing Type (49)

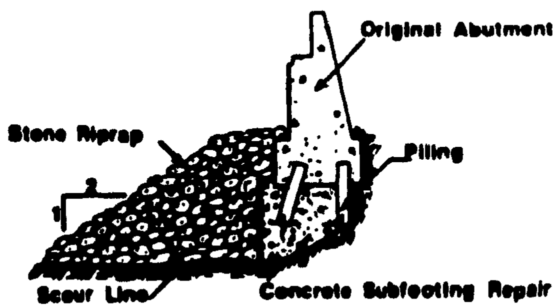


Figure 6.10 (b) Abutment Repair-Pile Bearing (49)

6.8.6 Repair of Concrete Fill Slopes

If there is no undermining of the slopes, it is usual to place heavy stone riprap as shown in Figure 6.11 (a) to prevent any further erosion. However, in case of severe undermining of the slopes, the scoured area is filled with sand and gravel upto a level to enable the extension of the original slope protection over the filled portion. The void underneath the original protection is filled by dropping concrete through holes punched in the slope as shown in Figure 6.11 (b). Additional protection against scouring along the edge of the slope can be provided by constructing a concrete end wall and placing stone riprap as shown in Figure 6.11 (d).

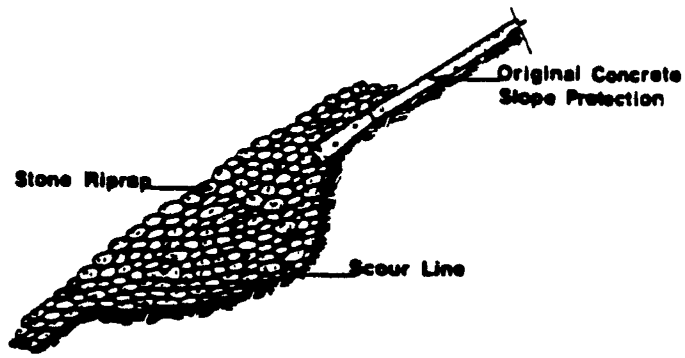


Figure 6.11 (a) Repair of Concrete Slope-No Undermining (49)

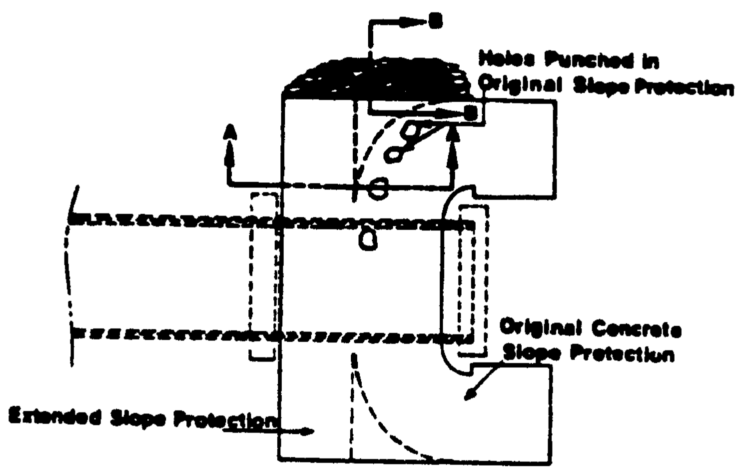


Figure 6.11 (b) Repair of Undermined Concrete Slope (49)
-Plan View of Concrete Slope Protection

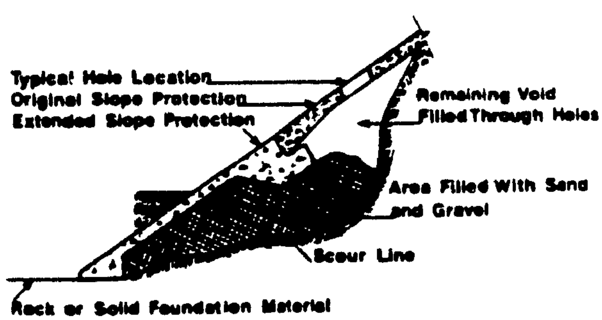


Figure 6.11 (c) Section AA (49)

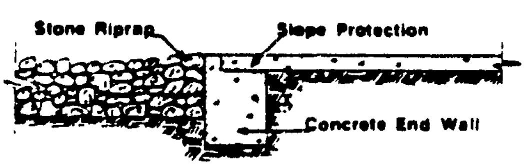


Figure 6.11 (d) Section BB (49)

6.8.7 Repair to Tilted Piers

Piers may tilt as a result of severe scour and may cause distress in the superstructure leading to failure. Before carrying out any repairs to the pier, it is necessary to first repair the footing using any of the previously discussed techniques. A first stage repair procedure in this case involves drilling of a number of holes through the deck and into the pier as shown in Figure 6.12 (a). Grout and steel dowels are then inserted into the cavity to resist further tilting of the pier until the footing is repaired. The pier is then enlarged by constructing a new concrete section upto an elevation above the waterline and the concrete is allowed to gain strength. It is then used as a seat to jack the superstructure to its original level. The superstructure is supported on steel shims until Stage 2 repairs are made as shown in Figure 6.12 (b). In this stage, additional holes are drilled through the deck to gain access to the top of the pier. These holes are closed monolithically with the second stage concrete placement and the steel shims are left in place. This repair should be undertaken only if the damage is acceptable and can be repaired.

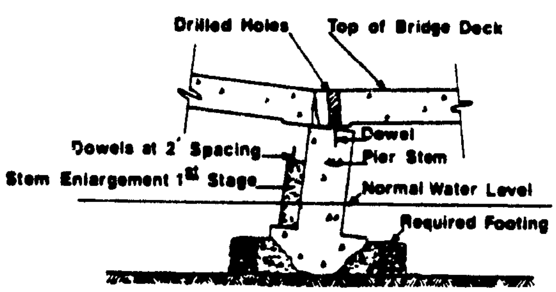


Figure 6.12 (a) Repairs to Tilted Piers - First Stage Repair -Section through Pier Stem (49)

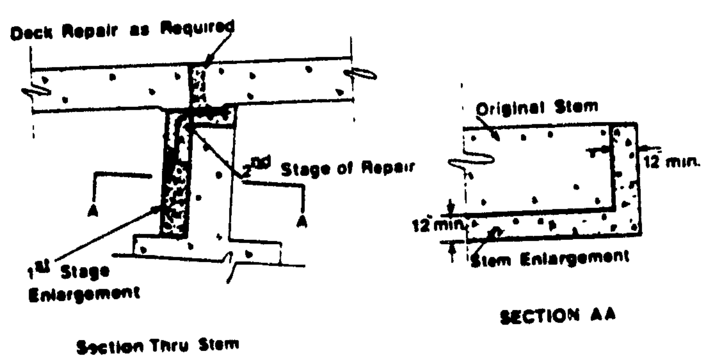


Figure 6.12 (b) Repairs to Tilted Piers - Second Stage Repair (49)

6.9 Repairs as a Result of Damage

Damage to bridges in navigable zones may occur due to impact from vessels and is of serious concern in view of the resulting physical disability which is inflicted on the substructure. Whilst the codes of practice and recommendations can provide the engineer with the tools to design structures which will withstand the normal loads imposed on a bridge in a marine environment and even allow for a degree of abnormal loading, it would naturally be unrealistic to design for the magnitude of loading which would be imposed by such incidents as impact from vessels.

Damage to a bridge can vary from superficial damage to the fendering systems to complete failure of the bridge. It may include either spalled or cracked concrete and damaged or failed piling systems. The cracks in pile caps can be treated by resin bonding. Surface evidence of cracks are first sealed by hand, either by surface application or by first cutting out along the crack and placing sealing compound in the groove so formed. At strategic points along the cracks, filing nipples are installed and once the network of cracks is satisfactorily sealed, resin is pumped into successive filling points using the adjacent points as an indicator of the full penetration of the resin. This method has been used successfully with repairs remaining intact during subsequent incidents despite damage occurring to the adjacent and previously sound concrete.

When damage to a bridge is greater and extends to the supporting piles, more substantial repairs are necessary. Failed or damaged piles can no longer provide support so it becomes necessary to provide additional piles in such locations so that their effect is maximized. Driving new piles beneath the bridge is difficult unless the cap is removed or the piles are driven at rake from outside the perimeter. Even then it is very difficult to avoid existing piles and the connection to the bridge is very complicated. In case it is unavoidable to drive additional piles around the perimeter of the bridge, the size of the cap should be increased as an alternate solution. New concrete should be firmly bonded to the existing structure with tie bars and thorough preparation of the concrete surface.

Failure of piles as a result of flexure often occurs at or near the soffit of the cap or at the point of entry into the water bed. Pile failures immediately under the cap can be repaired by conventional site repairs of plating and stiffening. Repairs at or near the water bed are somewhat difficult as access for inspection and repairs is far more difficult. One technique which increases the strength of the suspect pile groups by decreasing the effective length is to cast a second cap at the bed level. The bed is cleared of debris and prepared by placing a mat of granular material over the area of the pile group. A new cap is cast under water using traditional methods or utilizing proprietary additive such as hydrocrete. The effect is to greatly stiffen the pile group at its midpoint and to spread the forces generated in the upper section evenly over the buried lower section. The solution can be beneficial in remoter regions of the work where mobilization of piling equipment and procurement of replacement pile can be both time-consuming and expensive.

Where a substructure is damaged to a degree where it is basically sound but no longer capable of withstanding the full design load, it is possible to reduce the load imposed and provide a fendering system which absorbs a greater amount of energy and thus transmits less reactive force to the bridge. Many bridges have been provided with rubber fenders which are cost effective, but have low energy absorption characteristics in general. Advancements in fender technology have been extensive in recent years and fender types such as arch, cell, and pneumatic fenders have provided a very practical fendering system with low reaction forces. A drawback of such systems is that they are relatively expensive. However, in view of the serious nature of damage, these systems seem practical.

According to Derucher and Heins (19), basically seven types of fendering systems are being used currently. These are summarized below for completeness.

1) Floating fender

2) Standard pile-fender system
   - a) Timber pile
   - b) Hung timber
   - c) Steel pile
   - d) Concrete pile

3) Retractable fender system

4) Rubber fender system
   - a) Rubber in compression (Seike)
   - b) Rubber in shear (Raykin)
   - c) Flexible
   - d) Rubber in tension
   - e) Pneumatic

5) Gravity-type fender system

6) Hydraulic and hydraulic-pneumatic fender system
   - a) Hydraulic dashpot
   - b) Hydraulic-pneumatic floating fender

7) Spring-type fender system

6.10 Prestressed Concrete in Marine Environment

Prestressed concrete has been used commonly in marine environment and needs additional care in view of the problems before and during repair. Periodic on-site condition surveys should be undertaken to detect any signs of distress before the prestressing tendons are exposed to the corrosive influences of the seawater. Repairs should be initiated early in the deterioration process before the tendons are damaged. Dry methods of repair, as used for reinforced concrete structures, are preferred since the repair requirements are most critical. For example, extreme care should be taken during chipping and cleaning to avoid any damage to the tendons. Special techniques are necessary to repair or replace damaged tendons.

CHAPTER 7
BRIDGE MANAGEMENT SYSTEMS

7.1 Introduction

In recent years Bridge Management Systems (BMS) have been recognized as vital tools necessary for proper management of a bridge population. These systems enable transportation agencies to develop an inventory for the bridge population with all relevant information about these structures that allow for the systematic determination of present and future needs for maintenance, repair, rehabilitation or replacement and the prioritizing of remedial actions to optimize on the available budget funding.

Bridge management systems are similar to pavement management systems. However, unlike the latter which are fairly well established, bridge management systems are relatively new. This chapter reviews the essential requirements of a bridge management system (since they relate to hardware/software useability, rating of structures and financial evaluation techniques used), concluding with a review of two established systems.

7.2 Objectives of Bridge Management

It is necessary for every transportation department to exercise effective control over the state of bridges in its jurisdiction. Such a control can be achieved through implementation of an effective bridge management system which with a proper database serves to perform the following objectives:

1) Administrative Planning Function
    - Establish Needs Policy
    - Categorize Bridge Needs and Funding Sources
    - Prepare Statewide Annual Budgets
    - Prepare Annual Work Program
    - Generate System Wide Reports
2) Programming Function
    - Select Candidate List of Bridges
    - Prioritize Candidate Bridges in Various Action Categories
    - Analyze Cost Effectiveness of Programs
    - Flag Candidate Bridges for Maintenance Activities

3) Implementation Function

- Perform Structural Analysis and Analyze Cost Effectiveness of Project Level Alternatives
- Prepare Plans, Specifications and Estimates
- Perform Necessary Actions
- Analyze Cost Effectiveness and Maintenance Activities
- Perform Maintenance Activities
- Collect Structure Inventory and Appraisal Data
- Maintain BMS Data Base

In short, one must have a proper BMS dealing with activities related to bridges right from their commissioning until reconstruction. Thus, BMS provides benefits to the administrators and engineers at all levels to select optimum solutions to achieve the desired levels of service within the funds allocated and to assess the future funding requirements.

The logical development of a Bridge Management System would include the following six major modules (see Figure 7.1).

- Data Base Module
- Network Level Major Maintenance, Rehabilitation and Replacement (MR & R) Selection Module
- Maintenance Module
- Historical Data Analysis Module
- Project Level Interface Module
- Reporting Module

The Network Maintenance, Rehabilitation and Replacement (MR & R) Selection Module enables the bridge managers to make more effective programming and budgeting decisions. This module includes the following four technical sub-modules:

- Ranking
- Specific MR & R Action Selection
- Life-Cycle Costing
- Optimization

At this level, the impact of the state of the bridge population is dealt with respect to the actions that need to be taken to achieve an intended state of condition and the associated cost. This most commonly involves the need to prioritize the remedial works of individual bridges so that those in most need and have the greatest economic impact are carried out first. To achieve this, economic and technical evaluations have to be undertaken to assess the rankings.

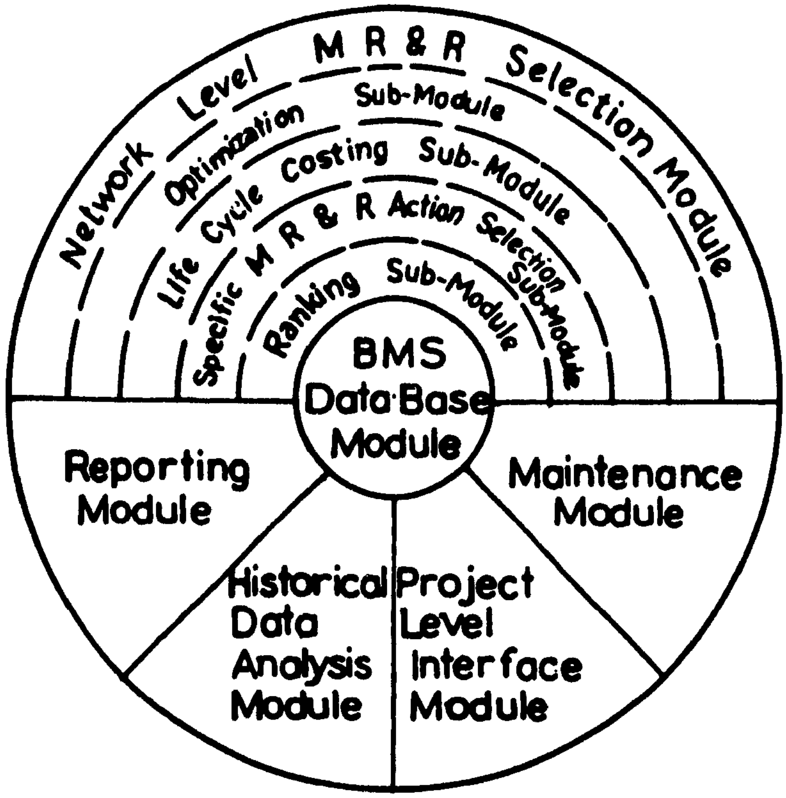


Figure 7.1 Modules and Submodules Comprising the BMS Module (7)

A Bridge Management System (BMS) is a rational and systematic approach to organizing and carrying out all of the activities concerning bridges, such as predicting bridge needs, defining bridge conditions, allocating funds for construction, replacement, rehabilitation and maintenance, identifying and prioritizing bridges for remedial actions and finding cost effective alternatives for each bridge, scheduling maintenance, monitoring and rating bridges and maintaining an information data base. At the network level, the entire bridge population is dealt with globally, i.e., the number of deficient bridges on a particular route are more important a consideration than the condition of a span in a specific location.

A more advanced BMS model can include data on all structure types, bridge sizes, different construction materials, network level considerations, life cycle costing models, prioritization procedures, maintenance, rehabilitation and replacement alternatives, and automation considerations.

7.3 Computer System Requirements

The effectiveness of managing a bridge population is enhanced by the use of computers and appropriate software. Computers can facilitate the storage of vast amounts of data, quick retrieval and analysis of such data, generation of reports, graphs, etc., on an individual bridge or a group of bridges. In the following section, the requirements of a computer system are outlined for a bridge management system.

7.3.1 Hardware Requirements

These days, it is possible to find appropriate software for many applications. For a data-based bridge management system, a personal computer is sufficient; however, for speed and ease of retrieval of information, a system with sufficient storage and speed is recommended. A computer with 40 Mb disc storage and a minimum speed of 15 MHz should be adequate for most bridge inventories up to 10,000 bridges. The preferred option in system design is to use increasingly, a network system where a number of users can make use of multiple computer workstations simultaneously. They are all linked to the same central processing unit which has expanded storage to handle the increased demand. Storage is normally of the order of 200-300 Mb. The available storage is sufficient to run all application packages available. Thus a BMS may be run simultaneously with other applications in an engineering office without any storage problems.

For offices with a mainframe system, the storage available is so large that use of a computerized BMS, of any size would not require much consideration concerning storage and program execution. However, a mainframe system is quite expensive and should be used only if it was an already existing system.

7.3.2 Software Requirements

The data base is the foundation of the BMS. It is used to store all relevant information about the inventory of bridges (an example of this inventory information is included in Appendix B). This includes such items as the bridge identification number, structure type, etc. In addition, the data base will be required to store information concerning the bridge population such as user cost, unit cost of repair, replacement or rehabilitation of bridge components, etc.

The storage of the bridge data and the need to retrieve it is best achieved with the use of a data base (dbase) type package. Other systems have been developed using programming languages such as FORTRAN, BASIC, COBOL, PASCAL, etc. The programs written in these languages have their data stored as input data files. Additions to the data require editing these input files which is tedious and one has to be familiar with the format of the input file to correctly enter a particular item of data in its proper location. These programs are developed commonly and written in-house or with the expertise brought in, which can require a commitment in personnel, time and money which only a few organizations can afford.

The dbase package is an assembled package purchaseable off the shelf. It is easy to do formatting of screens to produce prepared reports (which is done interactively in dbase), and perform mathematical functions for carrying out financial evaluations, etc. Also, the data base can be readily expanded by entering data in the appropriate screen. These additions are performed while the program is running and the user is able to check his data entry for accuracy immediately. In addition, by the indexing of data files in the main program, one is able to design the BMS so that the file order and searches on bridges may be carried out. This is useful when one wishes to prepare a report on a particular type of bridge or bridges in a specific location only.

7.3.3 Computer System Security

Security for a computerized BMS is provided at two levels: - the software protection where the actual source file (program) that runs/executes commands is protected from manipulation and the protection of data stored on the bridge inventory. The type of security will depend on the type of computer hardware being used. Protection for the above files will be different for a personal computer or a network (and mainframe) system.

Personal Computer Security

The first level of protection which may be provided for a personal computer is to limit the users by allowing only those required to use the computer to have a key to turn the computer on and off. This will require diligence on the part of the users (which becomes difficult to follow with many users), and does not provide total protection for files as the computer can be left running inadvertently. Further protection is provided by the use of a software key, which is a program with a series of commands required by the main program before it starts its own main execution. This may prompt the user for a password without which the main program will not execute. In general, protecting software on a personal computer is difficult so long as the user is able to access the drive on which the program is stored.

A recent development in personal computers applications is the use of software programs which limit user access only to predefined directories. These programs become activated automatically when the computer is turned on, thus they are able to effectively manage user access. One disadvantage of these software programs is that they take up a lot of space on the computer and slow the operations of the system.

Network (and Mainframe) Systems

Networks (and Mainframes) allow the best protection for files using the network utility programs such as the Norton Utilities, which limit access similar to that explained above (personal computers). Programs to be executed are handled in a batch mode and are not available for the user to view. Using only the utility commands designated, user may gain access to run the program and also be able to update the data bases.

One problem that can arise in such a set up is the question of multiple use of the system where one individual is updating a data base while another is producing reports. In this case it could be arranged to have updating done on a specific day or time, while denying access to BMS for anyone else. In summary, it is seen that a computerized bridge management system is afforded the best protection by using it in a network (or mainframe) system. Where use is to be on a personal computer, the security provided leads to inconveniences as stated previously.

7.4 Features of a Bridge Management System

7.4.1 Evaluation of Bridges

A prerequisite for attempting and adopting any scientific system of management of bridge maintenance is to have a proper bridge information system comprising bridge inventory, bridge inspection, bridge rating and data file. The first two are collection of organised data while the third deals with data organisation, storage, retrieval and analysis. Collecting bridge inventory data and inspections are not goals in themselves but tools to achieve the objectives of traffic safety and optimum utilisation of funds for bridges.

A typical Flow Chart of BMS is shown in Figure 7.2.

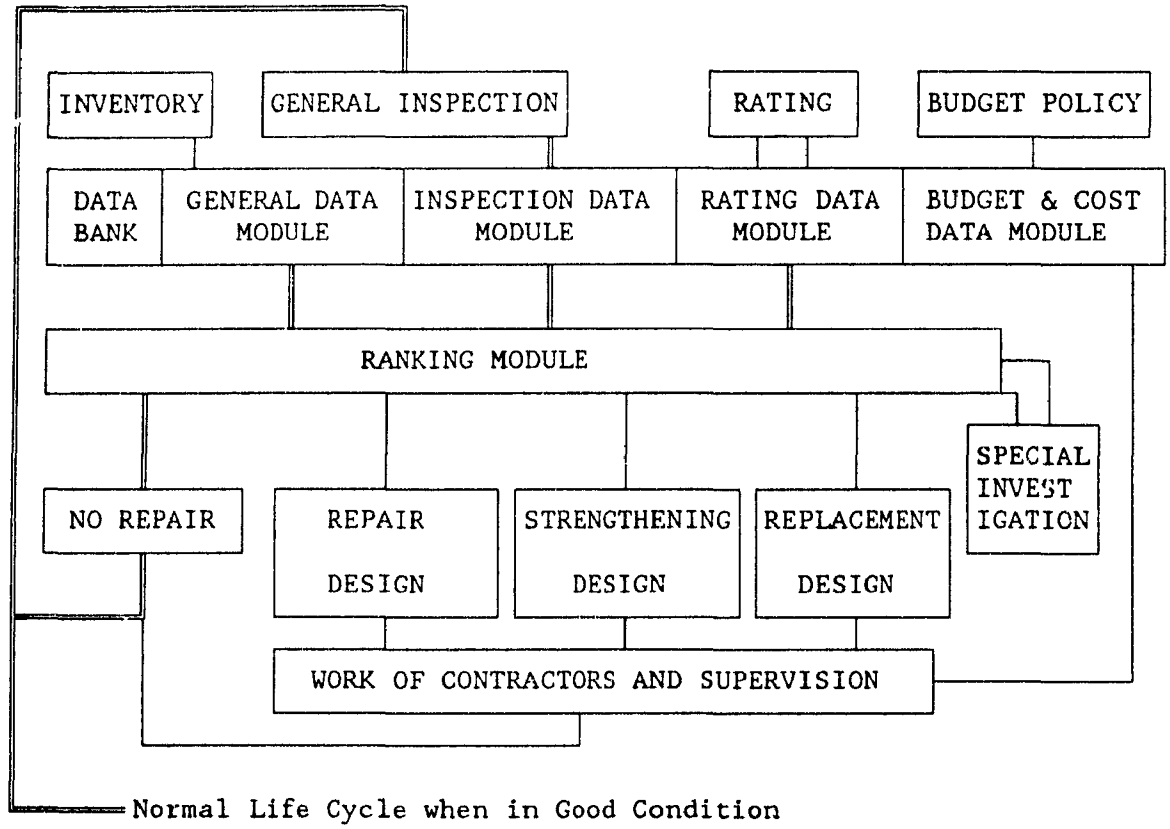


Figure 7.2 Typical Flow Chart of Bridge Management System

7.4.1.1 Bridge Inspection

Bridge inspection is a key aspect to preventative maintenance and to the proper running of any bridge management system, as this is the only means by which the condition of the bridge is determined. In achieving this goal, assessments are made and records are kept of the physical condition of every bridge.

Due to the particularly adverse exposure conditions to which bridges may be subjected, the inspection process for each bridge should be scheduled such that no bridge is inspected later than a specified interval of time which is dependent upon a number of factors, some of which include the following:

1) The average age of structures in the bridge population.

2) The particular exposure conditions in the jurisdiction as a whole or at a particular bridge location.

3) The load history of vehicles in the jurisdiction. Where vehicle load enforcement is not vigorously enforced and overloading is suspected, bridge inspection should be more frequent.

In the U.S., Canada and Britain, bridges are inspected at a maximum interval of 2 years (32). However, the inspections may be undertaken at greater time periods where past reports of performance and exposure conditions (ie. not severe) justify it. As a general rule, structures with no structural redundancies should be inspected more frequently than those with redundancies, as deterioration in members could lead to the collapse of the structure.

The actual inspection of bridges should be performed by a group of suitably qualified or experienced inspectors who are well versed in identifying problems associated with different bridge types (material) and form. The head of such a unit should be a qualified engineer, who can supervise and coordinate the inspections. In jurisdictions where experienced or qualified personnel are not readily available, manuals (2,42) can be used as suitable guides for the training of the inspection staff

The inspection process can be categorized into 5 basic types (32), as outlined below:

1) Inventory Inspection - This inspection is carried out at the time the bridge is added to the bridge inventory file, and should provide the bridge description in detail, e.g., its name, number, etc. A detailed structural analysis is undertaken at this stage to determine the load carrying capacity of the bridge which is determined from the load corresponding to the lowest bridge component capacity. Included in Appendix B is a list of information (32) that should be included in an inventory file.

2) Routine Inspection - Planned inspections (every two years) of bridges to determine if the functional condition of the structure remains as required. If particular deficiencies were highlighted from inventory or previous routine inspections, these should be observed for any change. If warranted by observed deterioration increase, a detailed structural analysis is reperformed (accounting for deteriorated section sizes) to evaluate the actual load carrying capacity for possible load posting of the bridge.

3) Damage Inspection - This is an unscheduled inspection brought about by the need to assess the safety to bridge users and damage to a structure resulting from flooding, high winds or man inflicted damage (fire, vehicle collisions, etc.). Such an inspection may result in the immediate closure, or load posting of the structure, depending on the damage extent. These decisions will be made based on the judgement of the engineer and should above all ensure the safety of the bridge users.

4) In-Depth Inspection - These are required between intervals of five and ten years (32) to detect any bridge deficiencies not readily observed during routine inspections. Besides a closer visual examination of the components of the structure in more detail than a routine inspection, such tests as **tapping and chain dragging** to determine or verify delaminations, and crack width measurements may also be undertaken.

Non-destructive tests and other physical and chemical tests may be needed to supplement the inspections where defects are found. Inspections will lead generally to one of the following:

a) No action being required for the bridge with unrestricted vehicular travel being permitted.

b) Long term monitoring of the structure for crack development, subsidence etc., due to observed defects deemed not serious enough for immediate action.

c) Load posting of the structure due to discovered defects. This will become necessary if repairs have to be deferred to a future date.

d) Recommendations for the corrections of defects.

e) Assessment of the residual life of the affected components.

5) Interim Inspections - This is an ad-hoc ongoing inspection undertaken at the discretion of the agency to monitor a known or suspected deficiency or in response to reports from bridge users.

The above field inspections should be performed in a systematic manner so that the possibility of overlooking a particular bridge component is minimized. The inspection program should be subdivided into stages/ bridge components. The relevant sections are then completed on a pre-prepared inspection form as observations are made. A listing of the these stages/bridge component observations (32) is included in Appendix C.

Where notes are taken to describe conditions, they must be clear and concise as far as possible, utilizing standard engineering terms to describe observed bridge conditions. The notes should also be supplemented with sketches and/ or photographs.

7.4.1.2 Rating of a Bridge

Part of the information required for proper management of a bridge population is an assessment of the load carrying capacity of each structure. This information is used to post bridges where necessary, so that load induced damage and deterioration are not allowed to occur, and for route selection and approval of special permits for the transport of unusually heavy loads. The guidelines for rating evaluation (32) specifies that this load carrying capacity or rating shall be considered at two load levels.

The first level is the operating rating in which the absolute maximum permissible stress at which the bridge will be allowed to operate is determined. This data is used to determine the structural adequacy of the bridge when special permit vehicles are allowed to use it. The second load level is the inventory rating which is the load level (stress) at which the structure can safely operate indefinitely. Both load levels are evaluated using either the working stress or the load factor approach.

In the evaluation of older bridges, contract specifications may not be available to enable the determination of the type of material or its properties. From the age of the structure, whether known or estimated, the material properties (i.e. the allowable stress) can be obtained from tables in Reference 32. These tables provide properties of materials manufactured in specified time periods. Matching the age of the structure being evaluated with the corresponding time period will allow the estimation of a strength value. The tables are made available for both inventory and operating ratings.

In concrete bridge evaluations, non-destructive tests are available to quantify steel reinforcing and its grade. In general, it is accepted that a concrete structure in service under present day traffic, and not showing signs of distress can withstand higher levels of service. Where necessary, this can be verified by load testing.

In the load factor method of approach, for each rating level, bridge components are checked using the appropriate equations set out in Reference 32. These checks are made from both strength considerations and fatigue requirements. The equation used is of the general form:

$$\phi R_n = \gamma_d D + \gamma_l (R.F) L (1+I) \qquad (7.1)$$

or

$$R.F = \frac{\phi R_n - \gamma_d D}{\gamma_l L (1+I)} \qquad (7.2)$$

where R_n = Nominal strength or resistance.

$R.F$ = Rating factor.

γ_d = Dead load factor.

γ_l = Live load factor.

ϕ = Resistance factor (capacity reduction).

L = Live load effect (nominal).

I = Impact or dynamic load amplification factor.

D = Dead load effect.

The live load effects shall be obtained by applying a suitable model which represents loading in the particular jurisdiction, or using the standard AASHTO HS loading or one of the three vehicle types described in Reference 32. Allowance for impact should be included in the live load evaluations.

The load effects are distributed to the different bridge superstructure components using the approach set out in Reference 41. This involves the multiplication of the load effects of one line of wheels (or one half of the lane load) by a distribution factor to obtain the actual effects in components such as floor beams, main girders, etc. In concrete slabs, the method of approach shall be as set out in Section 5.3.3 of Reference 32.

For truss bridges, the standard approach is to assume pinned joints with members subjected to direct axial tension or compression. Influence lines can be generated using statics and load effects calculated from these.

As a result of years of research and field testing in Ontario (5), it has been shown that the simplified methods of analysis used in determining the structural capacity results in gross underestimation of actual load carrying capacity of bridges. In such cases, the structural evaluations can be performed using more sophisticated analytical methods such as the finite element analysis (accounting for three dimensional effects), or load testing. Correction factors (32) are applied to the load effects to account for the inherent inaccuracies of the method of analysis used.

After determining the load effects (shear and moments) for all structural components of the bridge, the rating factor is obtained using Equation 7.2. The lowest rating factor obtained shall be taken as the limiting factor for the bridge. Rating factors less than 1.0 indicate clear deficiencies in the structure and will require posting of the bridge. The load that corresponds to an acceptable rating factor is obtained by simply prorating from that which causes the deficiency. Where it is found that the load required to obtain acceptability is too low, consideration should be given to : 1) replacement 2) active load restriction and/or 3) repair.

The flow chart in Figure 7.3 shows the procedure for the rating evaluation of a bridge. The actual rating factor is calculated for all structural components. The evaluations are undertaken using the load factor method of approach. The procedure is similar if the working stress method is used, except for the load factors and the stress values used.

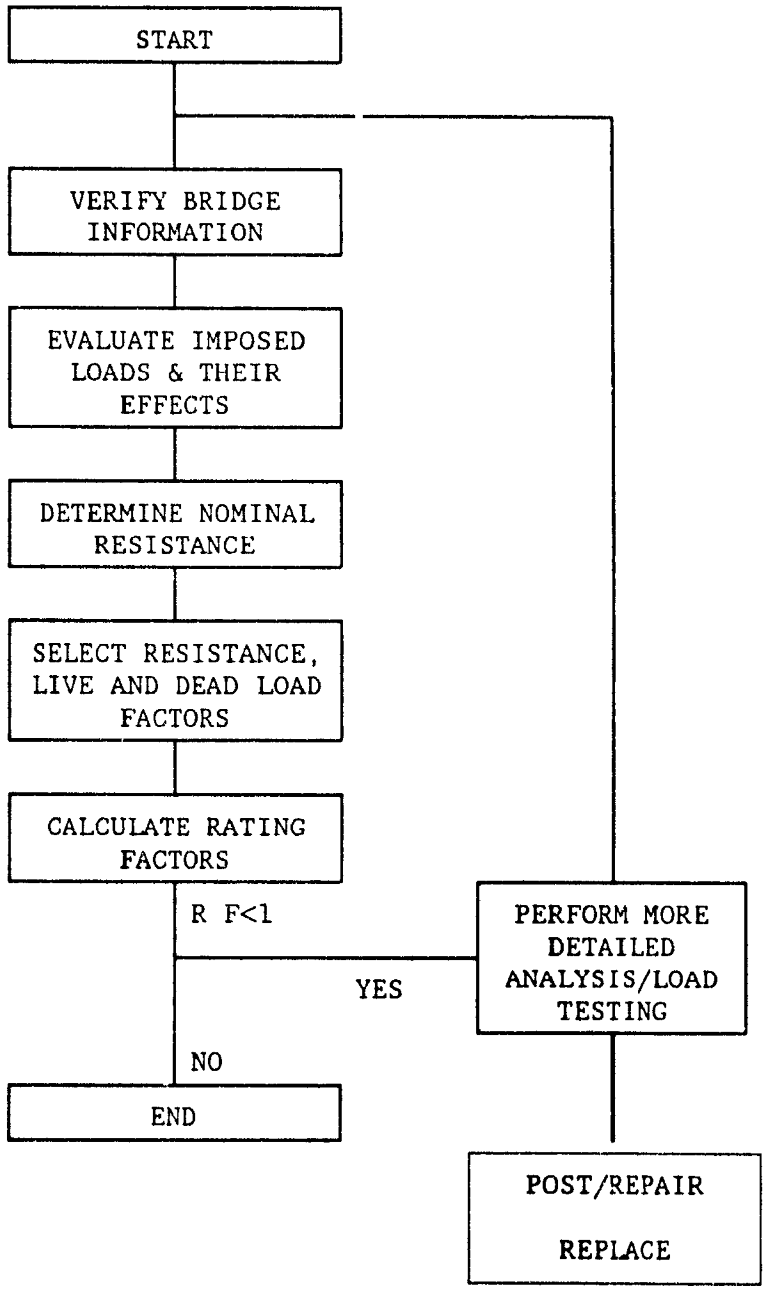


Figure 7.3 Rating Evaluation Flowchart

7.4.1.3 Ranking of a Bridge Population

Bridges which have rating factors less than 1.0 must be flagged for replacement, repair or retrofitting. For rating factors greater than 1, where members are undergoing observed deterioration, the previous three options are again reconsidered along with the added option of load posting only for deficient bridges or those scheduled for repair, etc. It is also recommended (32) that where posting load limit is 3 tonnes or less, consideration should be given to its replacement.

Where a number of bridges in a population fall into either rating category, some means of ranking or prioritizing them for action is required. The process of ranking involves the consideration of factors affecting the state of the bridge material and geometric standards which impact upon the volume of traffic using the bridge. Among the factors that affect the ranking are as follows:

1) Operating and Inventory rating.
2) Extent of deterioration.
3) Residual life of deteriorated component.
4) Number of lanes and their widths, etc.

The above factors are taken from the list in Appendix B. There is presently no set format for the number of factors or which items go into the ranking process. The selection will vary depending on the state of the bridge population and the objectives adopted by the agency. If the bridge population has good user services (such as proper lane widths, desired number of lanes, etc.), but is deficient in load carrying capacity, then the ranking should be such to reflect a preference for operating and inventory ratings. The factors used are given numeric ratings (0-9) which coincide with a qualitative state or condition of the structure (i.e. Good - 7; Fair - 5). An algorithm is formulated to combine the variables, weighted to reflect the importance of each in the function of the structure. The final overall rating is a number from 0 - 100. The rating should be so calibrated that 100 represents a perfect condition, and some cut-off value, say 20 would represent a condition which would require a high priority for remedial action. The ranking is then evaluated with the bridges with the lowest rating numbers requiring more urgent attention.

Calibration of the rating factors to actual bridge condition must be undertaken carefully, giving proper weighting to each component so that the weighting of a component (which may become deteriorated/damaged), which may cause collapse of the structure is not negated and that the structure can be given a reasonable overall rating (and hence ranking in the population) while it is on the brink of failure. In NCHRP Report 251 (49), the authors have proposed a Maintenance

Urgency Index which would allow for flagging of the structures requiring special consideration. A number (0-9) is qualitatively correlated to the various maintenance/ repair actions warranted. Otherwise, the ranking can be designed such that a bridge adopts as its overall rating, the rating of the most deficient structural component. Restricting it to the structural component ensures that bridges with critical problems will be given priority in the ranking

7.4.2 Financial Evaluation

Having highlighted the deficiencies/ defects in the bridge population, and having ranked them according to priority, the next stage is to evaluate the cost of rectifying the problems. This becomes necessary to judge the most cost effective approach of the remedial options being considered and allows the selection of repair candidates amongst a list of bridges.

At the project level, each of the possible remedial options considered have different costs associated with them. Some may have higher initial costs, but smaller maintenance costs, in addition to which one may have a longer life of repair than another. These have to be taken into consideration when evaluating the various options. Other factors which have to be taken into consideration are :

a) The location of the structure in the road network. This will have a pronounced impact on the travelling public if the alternate route has a longer mileage and the cost associated with its use are high. Where the bridge has to be closed during remedial works, the cost of detouring the traffic has to be considered, with the objective of minimizing this cost. When traffic volume is not heavy, temporary bridges (i.e. Bailey Bridges) may be launched adjacent to the existing bridge for the duration of the remedial works. The cost associated with any detouring will have implications for the allowable duration of the project which has to be accounted for estimating liquidated damages.

b) Nature and extent of deterioration of the structure.

c) Anticipated level of service of the structure and its present load carrying capacity.

d) Remaining life of the structure or any future reconstruction planned for that particular location.

f) Local experience in the repair schemes.

g) Number of qualified contractors available.

Not all of these considerations may be compatible to each repair alternative and the evaluation will show differences which will be used to determine the appropriate option.

Evaluations need to be carried out to assess the cost of foregoing repairs to a later date, which will result in a higher cost of action at a future date due to increase in unit cost associated with the repair plus having to account for additional deterioration of the component/structure.

7.4.2.1 Present Value Cost

Earlier techniques in bridge management simply addressed one or more options and the scheme with the lowest cost was normally chosen. Provisions were then made to carry out the necessary works. Where multi-year costs were encountered, the out-of-year costs were converted to a present day equivalent value, accounting for the time value of the funding.

The present value (PV) of a multi-year expenditure is given by the expression:

$$PV = \frac{C_n}{(1 + r)^n} \qquad (7.3)$$

Where C_n = capital expenditure for year n

r = rate of discount

The need to address the benefits which accrue from the remedial works has resulted in less use of this method of approach and the use of the benefit/cost (b/c) ratio method, which is presented in the following section.

7.4.2.2 Benefit / Cost Ratio Evaluation

In the previous section, only the cost of remedial works was considered. The approach used frequently is the benefit to cost ratio method, which is facilitated by attributing a numeric value to the benefits which accrue due to the improvement works. Associated with every bridge is a user cost, which comes from the various parameters associated with the traffic on the structure, cost of accidents, vehicle repairs, etc., associated with the defects of the structure.

The benefits are assessed by the reduction in the user cost incurred after improvements are made. For example, posting a bridge prohibiting heavy trucks and commercial vehicles causes these vehicles to detour along a longer route and results in an increase in the operating costs to these users. If these vehicles are allowed after the improvements are made, it helps to reduce their operating costs thereby supplementing the benefits offered by the improvement.

Consideration of the above enables assessment of the true costs and the benefits accrued from carrying out a particular action. Where these are multi-year benefits and costs, present day equivalent values are obtained as before. The available alternatives are judged on the basis of the highest benefit/cost (b/c) ratio. An alternative with a b/c ratio less than one is not considered feasible as the costs outweigh the benefits

7.4.2.3 Incremental Benefit Cost Evaluation

In the evaluation of repair schemes, various repair options are considered with each option offering a different level of improvement. For example, a concrete overlay offers a better measure of improvement over a waterproofing membrane with bituminous concrete wearing course in a deck rehabilitation. In addition, construction practices and costs vary. Thus after the various b/c ratios have been evaluated, they can be attributed to a desired level of improvement.

It was noted earlier that the b/c ratio was an improvement on the present value of cost method. The b/c ratio, however, does not adequately reflect the incremental benefit to cost ratio per level of improvement. In order to achieve this refinement, the incremental benefit/ cost (ib/c) ratio is used. The ib/c ratio is the ratio of the additional benefits realized in utilizing one alternative over another, divided by the corresponding difference in the costs of the two alternatives. This ratio is actually the rate of increase in benefit (from one option to the other) to cost, and gives a measure of the relative improvements for corresponding levels of improvements. Thus, the user has the opportunity to optimize the level of expenditure to a level of improvement (only feasible options are judged, i.e., the conventional b/ c ratio is greater than one). This is explained by the fact that the relationship follows the law of diminishing returns as shown in Figure 7.4. Thus, some of the options may be shown to be uneconomical for the additional investment. The most feasible option is the one with the highest ib/c ratio, which corresponds to the maximum point on the curve shown in the figure.

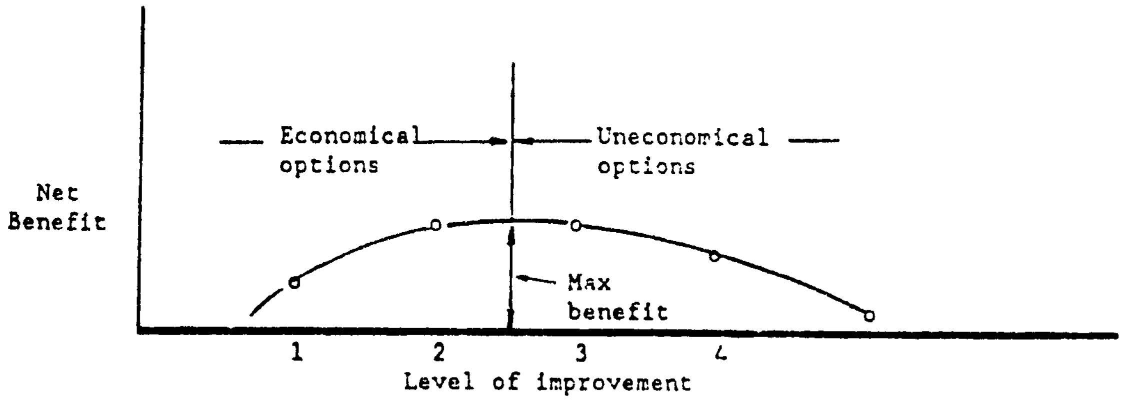


Figure 7.4 Net Benefit vs. Level of Improvement

7.4.3 Network Appraisal - Prioritizing Bridge Selection

At the network management level, bridges which were previously ranked according to urgency of repair needs, etc., and based on which the different repair options were considered, are chosen to evaluate the amount of funding required. The final approval of funding is normally outside the control of the bridge agency and often it does not match that required.

Therefore, the objective is to reorganize the remedial work that will yield the most benefit for the funding provided. One means of optimizing the selection process is through a further ranking according to their ib/c ratios. Alternative bridge sites are chosen based on the highest to the lowest ib/c ratio.

7.5 Review of Two Existing Bridge Management Systems

Two bridge management programs are reviewed in this section. One was developed in Canada and the other in Denmark. The two programs are :

1) The Bridge Rehabilitation, Inventory and Maintenance Management System (BRIMMS).

2) DANBRO bridge management and maintenance system (DBMMS) which is currently used in Denmark (for the state railway company) and Thailand(for their Department of Highways).

BRIMMS

This system has the capability of:

1) Storage of all the data necessary for the maintenance of the bridge population.

2) Allow for the analysis of the data so as to identify those bridge sites where replacement, rehabilitation or maintenance is nece.:sary.

3) Define cost effective rehabilitation and maintenance programs allowing for current and multi-year expenditure.

The system is microcomputer-based and consists of four databases and a main program (which retrieves data from databases by user specified reports) to analyze and correlate the relevant data to carry out :

1) Bridge rating.

2) Prioritizing remedial work.

3) Produce remedial work plans.

BRIMMS Data Bases

The four data bases in BRIMMS comprise the following.

1) Physical bridge data - This comprises the physical parameters of each bridge.

2) Traffic data - This takes account of the Annual Average Daily traffic (AADT).

3) Component condition data - Each component inspected is given a material and urgency rating. These are similar to that described in Section 7.4.1.2. The material rating is based on the level of deterioration of the component and its ability to carry its intended function. The material rating is either G (Good), F (Fair), P (Poor), or V (Very Poor). The urgency rating (UR) indicates how soon a component is estimated to need remedial action. Tables 7.1 and 7.2 summarize both parameters.

4) Service level data - This includes information on the load capacity, hydraulic requirements, number of bridge lanes, widths, etc.

Main Program

The BRIMMS program makes use of its database information to generate six parameters, on which a bridge condition can be judged. The parameters used are:

1) Bridge Condition Rating (BCR)

From the field inspections conducted, each component of a bridge is given a material rating and an urgency rating. From the calibrations performed (Table 7.3) field ratings are converted to numerical equivalents to give a component condition rating (the scale is 100 - 0, where 100 represents perfect condition while any value of less than 30 indicates that the component has reached the end of its useful life).

A component group rating is calculated by taking the weighted average of the individual component group ratings. Component groups comprise foundations/ substructures, superstructures, decks, structural and non-structural accessories. If any critical component in any of the above groups has a rating less than 30, then this is automatically taken as the governing rating, overriding the component group rating. This allows flagging of any serious defects which warrants special attention.

The Bridge Condition Rating (BCR) is obtained by taking the weighted average of component group condition rating (GCR). Typical weightings are illustrated in Table 7.4. Again, any rating within a component group with a value of less than 30, supersedes the final rating (BCR), not withstanding the particular weighting of the group.

2) Bridge Service Level Rating (BSR)

This rating is assigned a number on a scale 0 - 100, and it reflects the level of service or functional adequacy of the bridge. The level of service is dependent on a number of factors which follow :

a) Load Capacity
b) Number of Lanes
c) Width of Lanes
d) Horizontal Curvature
e) Bridge Traffic Railing
f) Vertical Curvature
g) Vertical clearance under bridge
h) Vertical clearance over bridge
i) Approach road narrowing
j) Hazardous utilities
k) Flooding
l) Hydraulic opening available under bridge

Each of these is assigned a value which is compared to a desirable and mandatory value. The mandatory and desirable values are based on the design standards. The actual values are stored in the physical parameters database in BRIMMS.

UR	DEFINITION
0	CRITICAL REPAIRS TO BE CARRIED OUT IMMEDIATELY
1	URGENT REPAIRS TO BE CARRIED OUT WITHIN ONE YEAR
2	HIGH PRIORITY REPAIRS TO BE CARRIED OUT WITHIN TWO YEARS IF THIS IS DEEMED ACCEPTABLE BY AN ENGINEER
5	LOW PRIORITY REPAIRS THAT CAN BE DEFERRED FOR UP TO FIVE YEARS
N	NO ACTION REQUIRED

Table 7.1 Urgency Rating Definitions

MATERIAL RATING	MATERIAL DETERIORATION	LOSS OF FUNCTION
G (GOOD)	NONE	NONE
F (FAIR)	MINOR	NONE
P (POOR)	MAJOR	MINOR
V (VERY POOR)	SEVERE	COMPLETE

Table 7.2 Material Rating Definitions

CONDITION RATING	FIELD CONDITION RATING			
100	GN	FN	PN	VN
90	G5			
85		F5		
70	G2		P5	
60		F2		
50	G1			V5
40			P2	
30	G0	F1		
25				V2
20		F0	P1	
10			P0	V1
0				V0

Table 7.3 Table for Converting Field Ratings to Numerical Equivalents

COMPONENT GROUP - FOUNDATIONS/SUBSTRUCTURES		
COMPONENT	WEIGHT	GROUP WEIGHT
FOUNDATION STABILITY	0.20	0.35
FOOTINGS, PILES	0.15	
COLUMNS/WALL PIERS	0.15	
PIER CAPS	0.15	
WINGWALLS	0.10	
ABUTMENTS	0.15	
ABUTMENT SEATS	0.05	
BEARINGS	0.05	
COMPONENT GROUP - CONCRETE SUPERSTRUCTURE		
COMPONENT	WEIGHT	GROUP WEIGHT
MAIN CONCRETE BEAMS	0.40	0.30
CORBELS, DAPPED ENDS	----	
TRANSVERSE SYSTEM	0.15	
SECONDARY BEAMS	0.10	
REINFORCEMENT	0.25	
BRACING, DIAPHRAGMS	0.10	
COMPONENT GROUP - STEEL SUPERSTRUCTURE		
COMPONENT	WEIGHT	GROUP WEIGHT
MAIN STEEL FLANGES	0.20	0.30
MAIN STEEL WEBS	0.20	
STEEL BEARINGS	0.05	
MAIN STEEL CONNECTIONS	0.15	
TRANSVERSE FLOOR BEAMS	0.10	
LONGITUDINAL FLOOR BEAMS	0.10	
SECONDARY CONNECTIONS	0.05	
DIAPHRAGMS, BRACING	0.05	
PAINT SYSTEM	0.10	

Table 7.4 Weights for Calculating Component Group and Bridge Condition Ratings

COMPONENT GROUP - DECK		
COMPONENT	WEIGHT	GROUP WEIGHT
CONCRETE DECK SLAB	0.30	0.20
DECK REINFORCING	0.25	
PROTECTION SYSTEM	0.05	
SIDEWALK	0.10	
JOINTS	0.15	
DECK DRAINS	0.15	
COMPONENT GROUP - OTHER STRUCTURAL COMPONENTS		
COMPONENT	WEIGHT	GROUP WEIGHT
BARRIER WALLS	0.05	0.10
PARAPET WALLS	0.10	
RETAINING WALLS	0.25	
EMBANKMENTS	0.25	
APPROACH SLABS/PAVEMENT	0.15	
GUIDERAILS	0.10	
HANDRAILS	0.10	
COMPONENT GROUP - OTHER NONSTRUCTURAL COMPONENTS		
COMPONENT	WEIGHT	GROUP WEIGHT
OVERALL DRAINAGE	0.04	0.05
SLOPE PROTECTION	0.50	
LIGHTING	0.09	
UTILITIES	0.01	

Table 7.4 (continued)

If the actual value is less than the mandatory value, a maximum point reduction is assigned. This is implemented for each element individually. The point reduction for a particular element can have three scenarios:

1) No point reduction - The actual value is equal to or greater than the mandatory.

2) Full point reduction - The actual value is worse than mandatory.

3) The actual value is equal to or better than the mandatory value but less than the desirable value.

The point reduction is given by:

$$Point\ Reduction = 0.5 \times MPR\ [\ 1-(\ ACT - DES\ )\ /\ (\ DES - MAND\ ) \qquad (7.4)$$

where MPR = maximum point reduction.

The bridge service rating (BSR) is then determined by summing all point reductions (for all elements) and subtracting them from 100. Maximum (default) reduction is 100, which would correspond to a BSR of 0.

3) Overall Bridge Rating (OBR)

The OBR is a combination (weighted sum) of the Bridge Condition Rating (BCR) and the Bridge Service Level Rating (BSR) given by,

$$OBR = T(\ 0.7\ BCR + 0.3\ BSR\ ) \qquad (7.5)$$

where value T is the traffic importance factor which reflects the importance of the structure in the network and is dependent upon the volume of traffic (AADT) using the bridge and whether the route is used by particular traffic groups (buses, trucks).

The weighting 70% - BCR to 30% - BSR (Equation 7.5), reflects the emphasis in restoring the condition (correcting defects) of its bridge population. The OBR value is used to rank structures in urgent need of remedial action.

4) Factored Condition Rating (FCR)

The FCR is obtained by combining the traffic importance factor and the status factor (S) to the BCR. The status factor is user defined, intended to assign lower priority for remedial work to structures less than six meters in span.

5) Factored Service Level Rating (FSR)

This is obtained by applying traffic factor T, and status factor S, to the BSR rating.

6) Factored Overall Ranking(FOR)

This is the weighted sum of the FCR and FSR ratings. In the BRIMMS program, bridges are ranked by any of the six parameters to obtain a shortlist. For each bridge component that requires repairs, the program considers four options, which are:

1) No Action (N)
2) Minor Action (A)
3) Major Repairs (C)
4) Replacement (D)

The program uses unit costs in the database to carry out a life cycle cost analysis, selecting remedial action with the lowest life cycle cost. In addition, it produces a parameter equivalent uniform annual cost (EUAC) which is used to show the cost of deferring the remedial action to some future date.

The BRIMMS calculates how well the action plan compares to a no action plan through an index called the cost effectiveness index (CEI). A CEI greater than one indicates that the action plan is more economical than the no action plan and a value of less than one shows that the no action plan is better. The return on investment (ROI), of the plan can be displayed. This is obtained by multiplying the CEI by the discount rate.

The CEI can be used to compare between different bridges where remedial action is limited by funding. The repair schemes are ranked according to the highest CEI value, given by the following expression:

$$CEI = \frac{PVE - PV + Workplan\ Cost}{Workplan\ Cost} \qquad (7.6)$$

where PVE = Present asset value for no action

PV = Present asset value for chosen repair option

Workplan Cost = Cost of remedial action.

The BRIMMS facilitates network management of the bridge population by the production of rating factors that relate to the condition and service of the total population. These factors include the bridge population condition rating (PCR) and the bridge population service level rating (PSR), which are computed by using the BCR and BSR ratings, respectively. Both the PSR and PCR are used to get an overall population rating (OPR).

The BRIMMS will predict the future PCR in one and five years, assuming that no maintenance or rehabilitation is undertaken. The use of a decay curve (negative exponential curve) of the population condition as a whole is used to make the prediction. The decay curve is a combination of all of the individual decay curves of bridge components.

Using proposed work plans, BRIMMS computes new BCR and PCR ratings. The new PCR ratings should exceed the previous one, otherwise it indicates that the bridge population as a whole is degrading.

DANBRO BRIDGE MANAGEMENT AND MAINTENANCE SYSTEM (DBMMS)

This system was developed for the Danish State Railways and it was commissioned later by the Department of Highways in Thailand. These two agencies are responsible for the management of 2500 and 10,000 bridges, respectively.

The DBMMS system is capable of being run on a personal computer and minicomputers and consists of.

1) A database.
2) Manuals for the program / system.
3) Six Modules: a) Inventory Module.
   b) Inspection and Condition Rating Module.
   c) Load Capacity Rating Module.
   d) Priority Ranking Module.
   e) Maintenance Module.
   f) Budgeting Module.

Database

The bridge database is used to store all relevant information about each bridge. This information is categorized as administrative, geometric, material composition of the structure and its condition.

Manuals for BMMS Program

The main activities covered by the DBMMS are shown below in Figure 7.5. Manuals are provided for all of these activities so that inspection and reports are performed in a consistent and systematic manner to have comparable results for the ranking of the bridges.

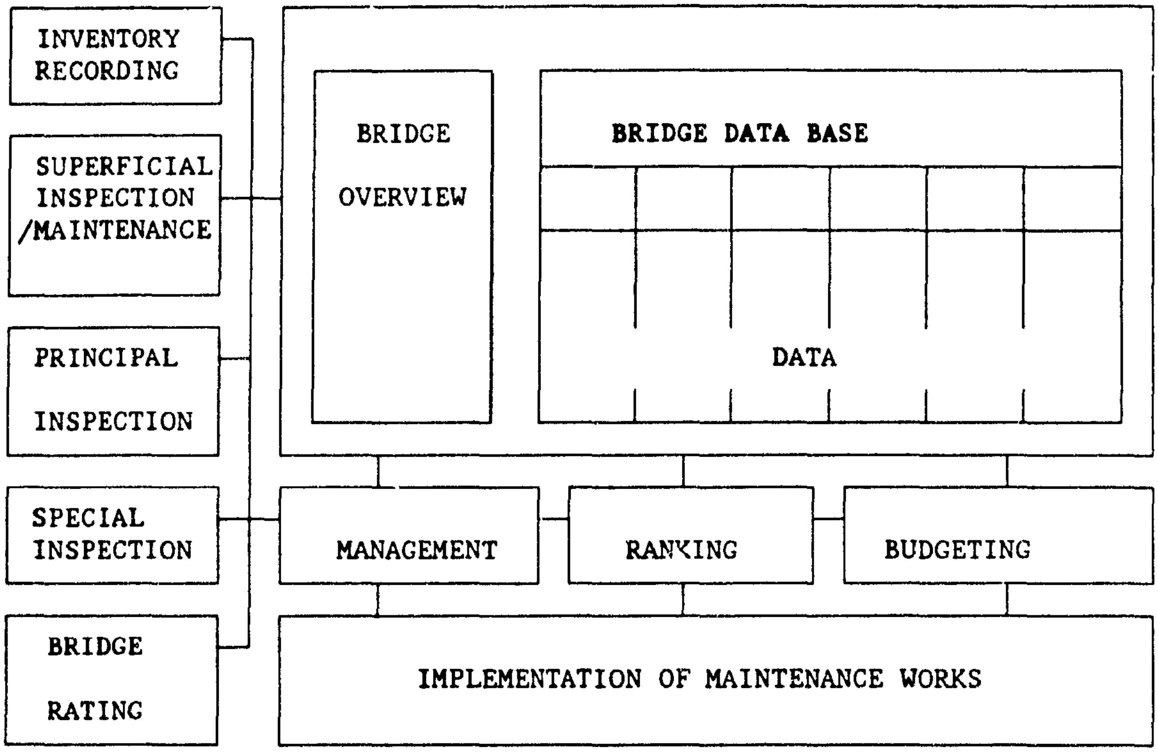


Figure 7.5 Activities Covered by the DBMMS System.

Inventory Module

This module is used to set output and input report formats used in DBMMS. Both formats contain information on the various elements/ components of a bridge. This information is either specifically user-defined or as presented in the list in Appendix B. Through this module, the user is able to view or update the information concerning the various bridge elements and add new bridges to the inventory. All updated or new bridge information is stored in the database, and is performed automatically by the program as the information is entered.

Inspection and Condition Rating Module

Three types of inspections are carried out:

1) Superficial Inspection.

2) Principal Inspection.

3) Special Inspection.

These inspections are similar to those described in Section 7.4.1.1. Superficial inspection is performed visually by trained staff and supervised by a qualified engineer. If the condition of the bridge is deemed to warrant, then special inspections are scheduled, which involve non-destructive testing and other chemical/ electrochemical tests. Superficial inspection covers the following activities:

1) Condition Rating
2) Estimates of the Remaining Life.
3) Cost Estimates.
4) Damage Registration.

The condition mark (rating) is a numerical assessment of the status of the component. The scale used is from 0 -5 (0 -Component with no need of repair, 5 -Component with no useful life remaining).

Estimates of the remaining life for elements/ components are made using deterioration models, which are graphical representations of similar components, whose rates of decay under varying exposure conditions have been monitored. Regression analysis is then used on the recorded data to obtain curves which approximate the decay rates.

The final activities involve estimating the cost of any repair activity. Any damage to the structure is noted. The date of the inspection is also recorded and displayed so that future inspections may be planned. A bridge with known deficiencies/ damage is flagged for inspection on a more frequent basis than one without any deficiencies/damage.

Load Capacity Rating Module

This is a menu driven structural analysis program which is linked directly to the DBMMS database. The direct linkage allows for data to be read form the database and used as input for structural analysis. The load capacity evaluation is performed using the actual condition of the bridge components and this is compared with the actions caused by live loads. The DBMMS is able to use various truck types and loadings (AASHTO, OHBDC 83), the effects of which are expressed in relation to a standard truck. In addition, the user is able to define his own truck loading/ configuration. In the case of simultaneous vehicle occurrence on a bridge, multiple present factors are applied accordingly to the load effects (24,58).

Section Strength Reduction

Strength reduction factors are used to assess the loss in capacity of the component due to its deteriorated condition. For each element, the strength reduction is assessed as a function of the condition mark. These factors are determined within each jurisdiction by a probabilistic approach, taking into consideration the likelihood of damage and its influence on the capacity of the bridge. The function used by DANBRO for the Thailand Highway Department is shown in Figure 7.6.

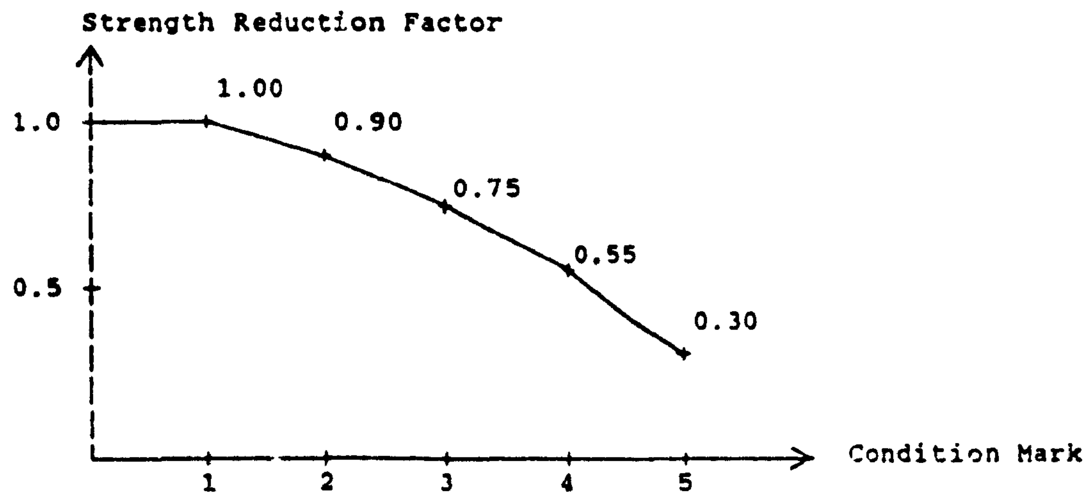


Figure 7.6 Strength Reduction Factor

Live Load

A standard truck is used as a reference in both the evaluation of the component capacity and the load effects from the user-defined trucks. Thus, the effect of any type of loading can be compared with the capacity of the components. This is beneficial while evaluating legal vehicle loads where the permit load is expressed in terms of the standard truck and compared directly with the bridge capacity (the component of the bridge that limits its carrying capacity).

The program analyzes a structure using either a simple two dimensional analysis or a detailed three dimensional analysis. In addition, it considers three limit states in its evaluation : ultimate, serviceability and fatigue limit states.

Two rating levels are considered, both corresponding to those outlined in Section 7.4.1.2. The inventory rating is used for evaluating new structures to be added to the database and the operating rating for the evaluation of the in-service structures.

The load factors used for both dead and live loads are obtained from the AASHTO Standard Specification for Highway Bridges (58). The resistance factors are calculated using an algorithm based on geometric and functional data (24) of the bridge from the inventory file, the AASHTO Code (58), and the condition data for the bridge. These factors are determined for all user defined sections.

Evaluation Results

Two parameters are output from the Load Capacity Module, namely, the load carrying capacity (in terms of the standard truck) and the load capacity rating mark. For each component, the sectional forces are compared with the corresponding strengths. Where they are not equal, the rating factor is adjusted (increased or decreased), until the load effects equal the sectional strengths. This final factor represents the margin of safety in the component limiting the bridge capacity.

The load capacity rating mark is used to prioritize bridges for repair. This rating mark reflects the ability of the structure to carry the actual truck loads. Where the ability of the structure to carry loads is reduced, this rating mark is increased by preprogrammed algorithm within DBMMS. Structures with higher rating marks are given a higher priority for repair.

Maintenance Module

DBMMS incorporates guidelines for material testing and maintenance strategies, which are used to evaluate repair options for a particular component deficiency. Included in the database are contractor costs, user costs from traffic surveys (43) and the owners costs. These are used to calculate the net present value of costs for each strategy to select the best option.

Budgeting Module

Rating marks as described earlier are used as the basis to prioritize bridges in need of remedial action. Bridges with higher ranking points have higher priority for repair. The importance of the function of each element in the bridge and the importance of the route (traffic class and volume) is included in an algorithm that evaluates rankings.

Shortlisting of the rating list is performed to reflect the ability of the agency to provide qualified staff to effectively monitor and manage repairs. From this, a budget is prepared. The repair budget may be for one or more years ahead, depending on the program requirements of the agency (e.g. a five year plan to upgrade the bridge inventory condition).

Where budget limits are imposed (typically outside the agency's control), DBMMS is able to rationalize the repair cost per year to fit within the restrictions. This is achieved through a feature called reverse budgeting facility. The budget limit is input, and DBMMS automatically evaluates the different combinations of jobs which will fit within the limits.

At the network level, DBMMS produces a network of routes within the jurisdiction where heavy vehicles can pass the bridges safely. Alternative routes, capable of accommodating these trucks are also included. This is used to more effectively manage routes in order to best preserve the bridge population, thus a bridge which is deficient is not allowed to be traversed by traffic which may cause this deficiency to be accelerated (e.g. from fatigue damage).

Over time, the agency can judge whether it wishes to extend the number of routes capable of accommodating heavy loads. This may be executed in response to the trucking industry requests. DBMMS will facilitate the review of the network system, determine the routes available for the intended loading, or if necessary, those bridge sites that would need upgrading/ new construction to allow the passage of such traffic

7.6 Conclusions and Recommendations

A bridge population represents a substantial investment in the transportation infrastructure, and a proper management system is required to safeguard this investment, with the following features:

1) A facility to store the relevant data for the bridge population, allowing easy up to-date review of the bridge population status.

2) Systematic evaluation of the bridge population condition, resulting from the standardized inspection reports for computer data entry.

3) A facility of generating reports (in fixed formats) for bridges.

4) A facility to rank and shortlist bridges for remedial work.

5) A facility to priority-rank repair plans, carry out financial analysis, and provide budgets for the remedial works.

6) Accrual of savings due to minimizing of unexpected repair works.

7) A means of maintaining a bridge population at a desired level of service and condition.

Review of the two programs highlight not only how much bridge management systems have advanced but also the different levels at which they are used to fulfil the requirements of the user agency.

The DBMMS provides the above features and in addition, it is able of interfacing different software programs to its database, enabling the system to perform structural analysis, evaluation and rating of bridges. In BRIMMS, this evaluation is performed independently by the engineer and the sophistication of the analysis is then dependent upon the engineer. However, the evaluation as performed by DBMMS and BRIMMS may differ, with the result that one structure may obtain a more favourable capacity check than another, though both may offer the same state of service.

Bridge management systems will find increasing use in the future, as transportation agencies seek more efficient means of storage, retrieval and evaluation of bridge data. The recommendations for future development of bridge management systems follow:

1) Standardization of technical terms and evaluation procedures is needed to make management techniques more uniform.

2) Standardization of financial evaluations and terms used.

3) Increase uniformity of design (evaluation) codes, and recommendations (1) and (2) will allow BMS packages to be standardized and hence purchaseable off the shelf, resulting in lower costs and quicker implementation.

4) The production of a standard document or manual of practice which encompasses :

a) Condition surveys (2) for the evaluation of the various bridge types, based on material of construction and type of structural system (precast concrete, steel girder, etc).

b) Test methods and procedures for chemical, electrochemical and physical non - destructive tests for both concrete and steel structures.

c) Repair strategies for concrete (61) and steel bridges.

This manual would then be used to design modules which could be interfaced with an existing bridge management system such as DBMMS or BRIMMS, thus allowing a bridge agency to fully automate its activities.

CHAPTER 8
CASE STUDIES

8.1 Repairs to Prestressed Concrete Girders of Thane Creek Bridge, Bombay, India

Salient Features of the Bridge

The bridge is located on a creek joining the provincial highway between the cities of Bombay and Panvel in the state of Maharashtra, India. It is designed to carry four lanes of I.R.C. (Indian Road Congress) Class A vehicles or single lane of I.R.C. Class AA vehicles whichever produces the worst effects. Construction of the bridge was started in 1963 and the bridge was opened to traffic in 1972.

The width of the carriageway is 13.41 meters and total length of the bridge is 1836 meters. The central three spans are provided with a vertical clearance of 9 meters for navigation purpose (see Figure 8.1). The foundation consists of open wells for six piers on the Panvel side and bored piles for the remaining.

Independent reinforced concrete piers are provided on each well for Pier Nos. 1 to 6 while reinforced concrete inclined "V" arms are provided for Pier Nos. 7 to 30. Piers from 30 to 35 are twin piers. Reinforced concrete piers above the pile cap and Pier Nos. 36 to 38 are single piers. The superstructure consists of six precast prestressed and post-tensioned concrete beams resting on rocker cum roller type cast steel bearings. The pier beams of the superstructure on "V" arms serve as tie beams. The deck slab is mostly precast as flanges of the beams and is post-tensioned transversely. Most of the structural components of the bridge such as piles, bracings, "V" arms and prestressed concrete girders are precast.

Distress Observations in Girders

Visual observations of the bridge in 1981 revealed development of longitudinal cracks in the girders. Spalling of concrete was also observed at a few places which exposed the high strength prestressing steel. The problem aggravated when the prestressing steel was found to be corroded and some wires of the cable had ruptured and snapped. However, the mild steel reinforcement was observed to be relatively unaffected.

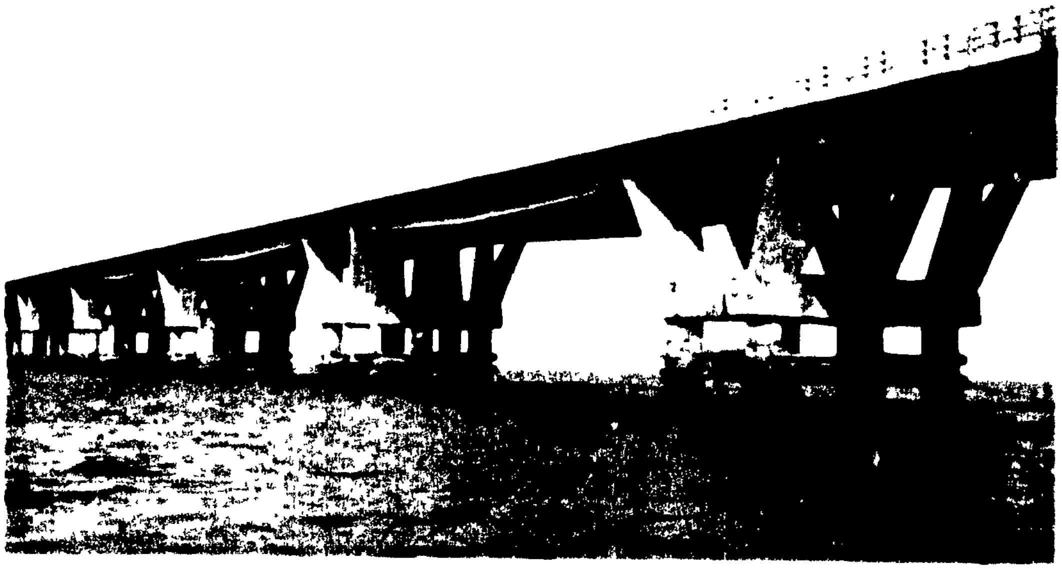


Figure 8.1 View of the Bridge

A detailed evaluation of the bridge superstructure showed that the balance of the prestressing force in the girders was not sufficient to sustain the heavy loads on the bridge after a few years because the rate of corrosion was very high in this region. Also, the bridge was the only direct link between the two major industrial cities and that any further damage would have been detrimental to the vital communication network.

In order to monitor the development of cracks and to identify the extent of the damage and distress in the bridge, it was felt necessary to inspect the bridge in detail (39). The difference between the high and low tide levels is about 4.7 meters. At the lowest bed level, the minimum standing water at low tide is 7.86 meters. The vertical clearance for navigational purposes is 9.15 meters above high tide level for the three central spans. Under such site conditions, it was not possible to organize a close inspection of the bridge. Therefore, the Public Works Department, Government of Maharashtra, decided to import a mobile bridge inspection unit as it was not available locally. This equipment had a maximum vertical working range of 13 meters for the inspection cage measured from the bridge surface in meters. The horizontal working range of the inspection cage from the centre of the vehicle was 10 meters. Also, the inspection unit consisted of four main booms of rigid steel blocks. The unit was mounted on a truck chassis with three axles and it was fitted with a diesel engine developing 216 H.P. at 2300 RPM.

The main unit consisted of a heavy duty box frame consisting of welded steel plates to resist all of the forces caused by the use of the machine (see Figure 8.2). The stabilizing unit comprised of four hydraulic outriggers, with the complete controls situated at the rear end of the vehicle. The movement of each outrigger was controlled individually. The vehicle could be adjusted on an uneven surface by means of a level indicator, provided at the rear end of the vehicle. The outriggers with tilt-up twin wheels enabled mobile inspection. During the operation of the machine, more than half of the road was completely free for traffic. The cradle was of size 0.9 by 2.0 meters , of tubular construction and equipped with two doors and with facility for lateral movement. The safe design working load was 350 kg.

The type of work possible with the mobile inspection unit was:

1) Close and safe inspection of all components of the bridge.
2) Detailed inspection of the bridge underside, slab soffits, all sides of the girders, piers, etc.
3) Repairing and greasing of the bearings.
4) Removal of cracked and damaged concrete from slabs, girders, piers and carrying out patch repairs.
5) Painting of soffits of bridge deck and all sides of girders.
6) Taking corrosion measurements and monitoring developments of cracks, etc.
7) Shotcreting, spray painting, sandblasting, or cleaning of concrete surfaces using pneumatic pressure hose system.
8) Measurment of girder strains

The unit was assembled on the deck and shifted to barges with the help of cranes. The barges were then towed and brought under the span and aligned. By manouvering the cranes, the unit was lifted up and the suspender bolts were driven through the holes made through the deck slab for this purpose. After levelling the platform, its load was transferred to the deck by fixing distribution girders and bolts. The suspender assembly was protected to avoid any damage and also to minimize hazard to the traffic.

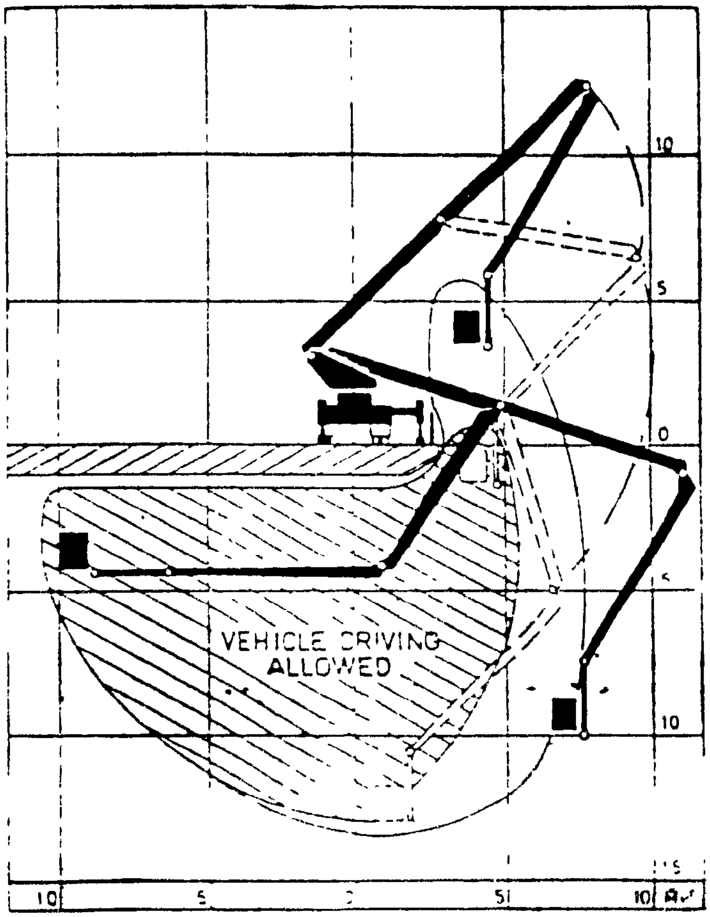


Figure 8.2 Bronto Skylift (39)

The longitudinal arrangement for the mobile unit is shown in Figure 8 3.

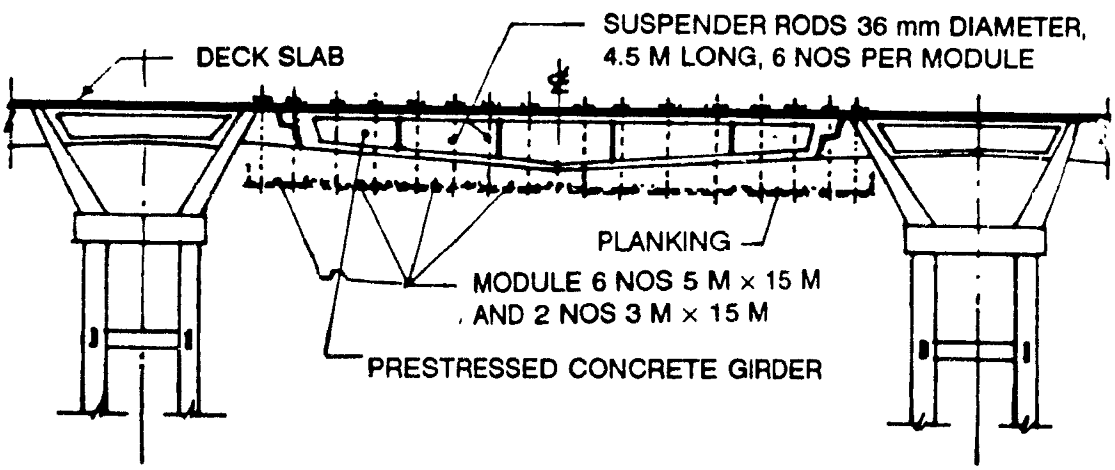


Figure 8.3 Longitudinal Arrangement of Mobile Unit

Immediate Remedial Measures

The following measures were adopted to arrest deterioration.

1) The surface of the prestressed girders was sand blasted. Any crevices, surface openings, pitting holes, etc. were filled using epoxy mortar. A protective coating of epoxy paint was applied uniformly on the surface of the girders.

2) Minor cracks in the girders were sealed by pressure injecting an epoxy resin into them.

3) The prestressed girders were strengthened by providing the required external prestressing.

External Prestressing

This technique has been successfully used on this bridge to compensate at least part of loss of prestressing force. While evolving the rehabilitation scheme, one of the consideration was to determine the prestressing force to be applied to each girder. Calculations were made as mentioned earlier to determine the prestressing force available in each girder accounting for the loss of prestress due to the distressed cables. After inspection of the girders and considering the available sections for transmitting the end anchorage forces, the maximum magnitude of the required external prestressing force that could be applied was about 25 percent of the balance prestress.

In the initial stages, four 12.7 mm diameter strands were placed symmetrically about the vertical axis of the prestressed girders. This provided approximately 15 percent of the required prestressing force. By using 15 mm strand or by increasing the number of strands to six, the magnitude of the prestressing force could be increased to about 25 percent of that required. The latter alternative was implemented. The basic system of external prestressing is shown in Figure 8.4.

The cable was anchored at the deck. For this purpose, necessary pockets were formed in the deck and holes were drilled for the cable to pass. At transition points, deviator blocks were fixed in position. The deviator blocks were fixed to the diaphragms by through bolts ensuring proper bearing against the girder soffit. The high tension strand was placed in a steel conduit to protect it from atmospheric corrosion. The conduit itself was sandblasted and painted with epoxy. The single strand cables were grouted by injecting a neat cement grout. Plasticizers were used for some cables in preparation of the grout to improve workability and to reduce the water-cement ratio.

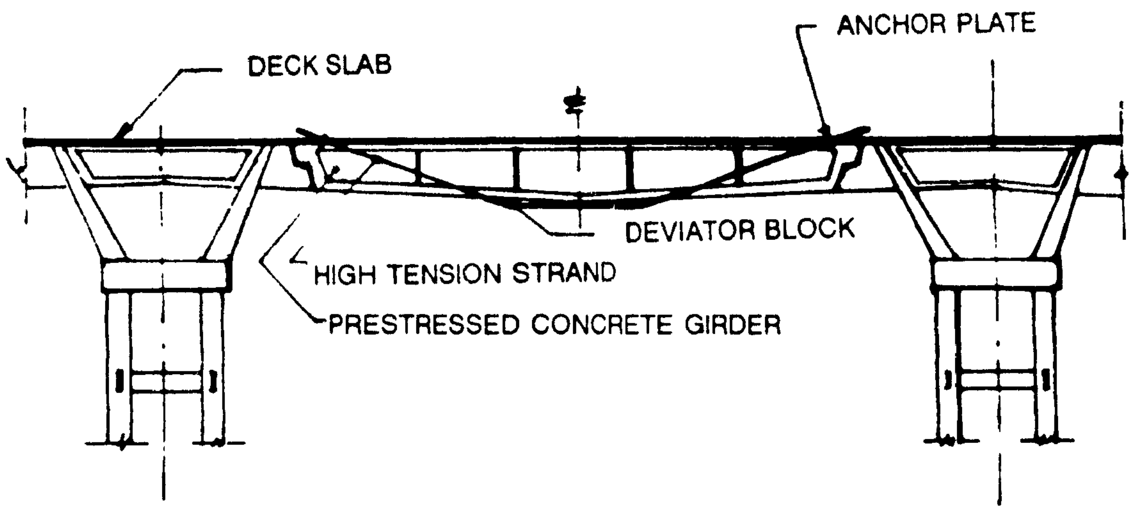


Figure 8.4 External Prestressing

Problems During Execution and Methods Evolved

To reduce inconvenience to public, erection of modules was carried out only during the weekends. However, it was not always possible to have favourable tides on these days Although motor boats were used, control of barges presented difficulties in keeping them in proper alignments. Therefore, the timing of the tides had to be studied and the work had to be done to precise schedule.

The north side girders are provided with steel brackets to carry a pipeline. This required making changes in the profile of the external cables at the site, especially as the pipeline could not be put out of commission. Brackets supporting the pipeline had to be modified by permitting threading of cables through special openings provided in modified brackets. The angle of inclination of the cables had to be modified by changing the size of the deviator blocks whenever the cable was intercepting the structural framework of the pipeline.

In some cases, while preparing the recess for the anchor block, cross cables of the deck were encountered. Therefore, the location of the recess had to be shifted to avoid interference with these cables. As a result, the cable had to be realigned and the deviator blocks had to be modified to accept such realignment.

The work of external prestressing of such magnitude was carried out for the first time in India.

This technique did not only arrest any further distress but it also helped to improve the serviceability of the structure.

8.2 Cathodic Protection of Fourth South Viaduct, Utah, U.S.A.

Location and Salient Features

The Fourth South Viaduct is located in downtown Salt Lake city and spans the main switching yards (26 tracks) of the Denver Rio Grande and Union Pacific railroads. The 1036 meter long concrete and steel viaduct provides critical daily access for 15,000 vehicles. It consists of 25 main spans, four approach spans, a 178 mm thick reinforced concrete deck on concrete girders, and a 203 mm thick reinforced concrete slab on the steel girders crossing the railroad tracks.

Deterioration of the Bridge

The bridge had been subjected to continuous applications of deicing salts during the winter season. Examination of the bridge after 17 years of service revealed that the salt had contaminated the concrete deck and pier caps. The steel reinforcement within the concrete had corroded and this had caused a progressive deterioration of the surrounding concrete. Large amounts of concrete had split off from the supports and the bridge deck had commenced to delaminate. Results from 27 cores analyzed confirmed that the chloride ions had penetrated upto 3 percent on the deck and 19 bents and had penetrated to depths ranging from 50 to 100 mm.

Rehabilitation Options

The following alternatives were considered before rehabilitation.

1) <u>Use of conventional procedures such as patching, membranes and sealers</u>:

Since this was a temporary type of repair, it would add only 3 to 5 years to the life of the structure at a cost of about $ 600,000. Over this period, corrosion of the steel would continue to weaken the structural components, resulting eventually in higher deck replacement costs.

2) <u>Removal and replacement of the old concrete with a new overlay</u>:

Use of this alternative could add 10 years to the life of the structure at a cost of about $ 1 million.

3) Replacement of the complete deck:

Use of this alternative could provide an extended life of around 40 years at a cost of about \$ 8 million. However, it could keep the bridge out of service for at least one year.

4) Arresting corrosion with the Cathodic Protection technique:

Complete refurbishing of the deck and the substructure, including installation of cathodic protection on the deck and bents could cost just under \$ 3 million Also, it could gain the same advantages as total deck replacement resulting in substantially less repair time and cost

The fourth option was chosen since it was considered to be the most appropriate. The proprietary cathodic protection system was developed by Elgard Corporation, Ohio. It uses titanium-based anode mesh embedded in the concrete. Corrosion is arrested by driving a low DC current from anode mesh to the reinforcing steel. The anode mesh remains intact and unchanged with prolonged use and can continue to protect the reinforced concrete deck and bents from corrosion for more than 30 years.

Implementation of Rehabilitation Program

The Project started in late April 1988. It was completed in mid-November and the anodes were energized in April 1989. From June through November 1988, two shifts worked to complete the refurbishing. The bridge was closed to traffic for only four months, from July through November 1988. The actual cost of installing the cathodic protection system was about \$ 650,000 and the entire bridge retrofit project was valued at \$ 2.6 million. The entire rehabilitation program was completed in 2 phases as follows.

Phase 1 - Bent Repairs

In 1984, delamination tests were carried out on the columns and around the bents. The data was plotted to locate areas of weakened concrete. The results indicated that the bent caps were subjected to more delamination and rust than other areas due to the water dripping from the decks.

Refurbishing started in 1988 with hydrodemolition to remove about 4 percent of the damaged concrete on pier caps and along 19 of the 25 supports. The exposed reinforcement was repaired and structural concrete replaced over it. The installation crew then wrapped the bents with 2790 m^2 of anode mesh which was cut with tin snips to fit around the columns and caps. Plastic fasteners held the anode in place and distributer bars were welded to the anode. Wires ran from the conduit on the outside of the bent cap down to enter one of the nine rectifiers. A 38 to 45 mm thick shotcrete overlay was then sprayed over the anode.

Phase 2 - Deck Installation

The delaminated concrete on the deck was removed down to the level of the reinforcing steel. The concrete was replaced after repairing all of the exposed reinforcement. The expansion joints were hydroblasted and repaired, and where required, exposed reinforcing bars were covered with a 12 mm thick mortar patch. The entire surface of the deck was then scarified to a depth of 6 to 12 mm to provide better adhesion for the new concrete overlay.

Around 9290 m^2 of anode mesh was installed on the scarified deck. Mesh sections (supplied in rolls approximately 1.1 m wide $\times$ 81.4 m long), were cut to the required size and fitted around the drains and other irregularities. The diamond pattern of the mesh (76 $\times$ 34 mm) containing thousands of strands not only provided redundancy for the current path, but also assured a uniform current distribution to the steel.

The mesh panels were electrically joined by transverse titanium conductor strips. Anode lead wires were inserted into holes drilled through the deck and connected to power wiring in a junction box located at the underside of the deck. A direct current from the viaducts power source, between 50 to 100 watts of continuously applied power, neutralizes the corrosive chloride ions. The mesh was then overlaid with a 50 mm minimum thickness of portland cement concrete that was grooved for skid resistance.

Monitoring

Initial testing to set the rectifiers was carried out after completion the rehabilitation program. It was necessary to carry out tests after every 6 months to determine if the rectifier settings need to be adjusted. Monthly checks were performed to verify the operation of the rectifier, i.e., to examine if power was on and also recording the operating current and voltage of the rectifier.

In the present case, a reference cell was embedded in the concrete during the rehabilitation program. Two different tests were specified to check that the steel was protected Both tests used the embedded reference cell to monitor the electrical potential of the steel. The electrical potential is measured against a parameter which is nothing but a difference in the tests.

The E versus Log(I) test required the measurement of the steel potential (E) versus the level of the applied cathodic protection current (I). It is a semilog plot used to interpret the level of the cathodic protection current and is often used along with depolarization tests to initially set the rectifier.

The depolarization test measures the change in the electrical potential of the steel with time after cathodic protection current to the steel is stopped. Normally, the job specifications require

100 to 150 mV change over time. This test is usually run for 4 hours. However, it may last longer for structures which are slower to depolarize. The routine test is simple and less subject to interpretation than the E versus Log(I) test explained earlier.

The cathodic protection system is working satisfactorily and has been able to stop further deterioration of the steel. It is, however, being monitored constantly to ensure its proper functioning. Figures 8.5 to 8.8 highlight the bridge and the cathodic protection system used.

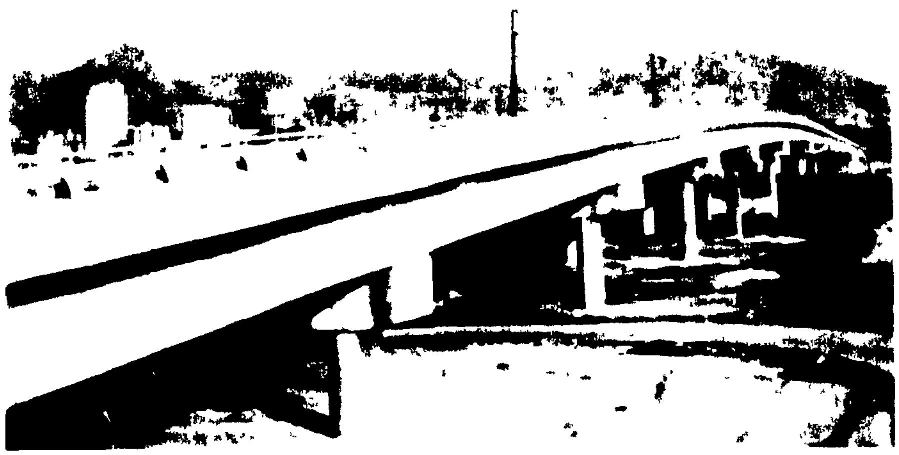


Figure 8.5 View of the Viaduct (65)

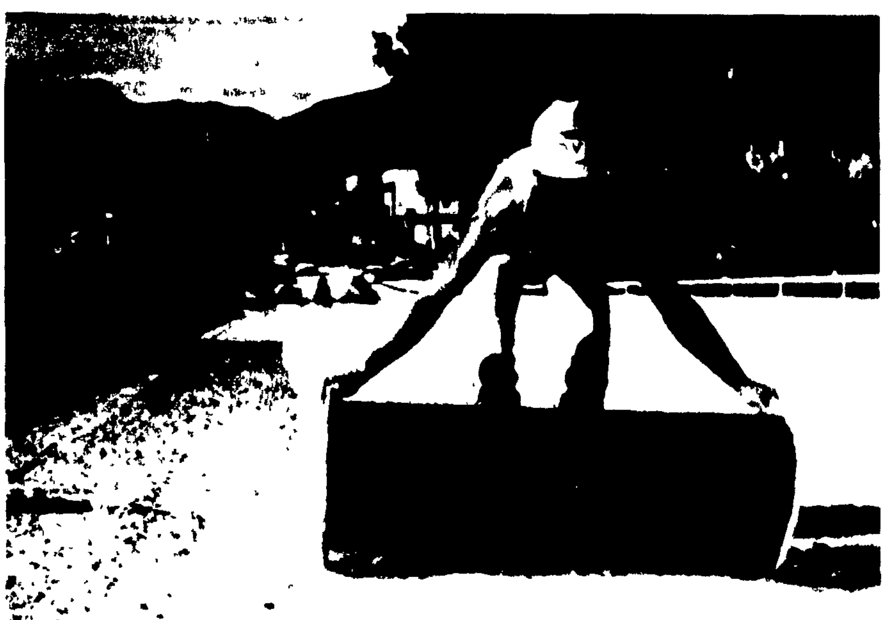


Figure 8.6 Installation of the Cathodic Protection System (65)

Figure 8.7 Spraying of Fibre-Reinforced Concrete Overlay on the Bents (65)

Figure 8.8 Fixing of Anode around Curbs, Beams, Piers, and other Ir-regularities (65)

CHAPTER 9
REHABILITATION OF CHAMPLAIN BRIDGE, MONTREAL, CANADA

9.1 Introduction and General Background

The 3.5 km long Champlain bridge is located over the St. Lawrence River at Montreal and links the South Shore community of Brossard and the Island of Ile des Soeurs and the city of Montreal. It was constructed during the years 1958 to 1962 at a cost of about $ 35 million. The six lane bridge consists of prestressed concrete beams and concrete piers except in the main span, situated over the St. Lawrence Seaway, where the roadway is carried on a three-span steel truss and concrete piers, including a cantilevered steel superstructure with a central suspended span (see Figures 9.1 and 9.2). The total length of the steel superstructure is about 762 m and provides a clearance of about 36.5 m to the shipping traffic below.

The bridge has a large network of approaches which include many satellite structures, such as bridges, viaducts and ramps. The original design of the bridge was performed very close to the design limits with no allowance being made for a future increase in either dead or live loads Also, the deck of the superstructure was designed without any bituminous asphaltic overlay or impervious protecting membrane. The employment of deicing salts over the harsh winter months has been along with the ever increasing traffic (especially trucks), the dominant factor leading to the progressive deterioration of the bridge over the years. When the Federal Government established the Jacques Cartier and Champlain Bridges Corporation in 1978, this organization had a clear mandate as custodians of the bridge to establish a proper maintenance policy. Maintenance activities are now prioritized through in-depth condition surveys of the various structural components of the bridge. However, after examination of the deterioration of the various substructure and superstructure components, it has at times been a difficult task to decide which component should be repaired first. The entire repair program for the bridge is reviewed globally on an annual basis and priorities established by taking into account the cost of work, limited annual funding available, any conflicts or interlaps between concurrent repairs and most importantly by the severity and location of deterioration as it pertains to the integrity of the bridge. Some urgent temporary repairs are performed to protect severely deteriorated members (like localized failure of the deck and failed expansion joints revealed due to the cyclical live loads in the truck lanes), however, these repairs were not sufficiently durable.

The use of deicing salts in combination with increased vehicle live loads has also contributed

Figure 9.1 Aerial View of Bridge

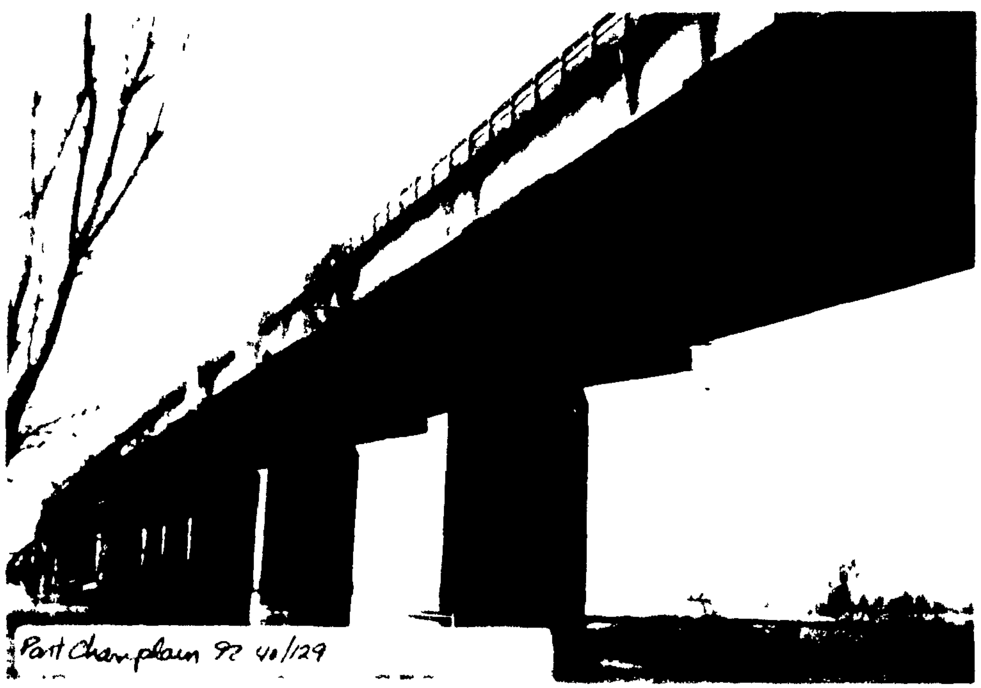


Figure 9.2 Side View of Bridge

to the local delamination of concrete from the top reinforcing steel in the bridge deck. Extensive full depth and partial depth patch repairs were carried out annually since 1983. However, these were temporary repairs, and in many cases, patches were required to repair patches. Over 57 percent of the original deck surface area (of Section 7 of the bridge) scheduled to be replaced, required patch repairs. This tedious work did not only have an impact on the heavy vehicular traffic but also increased considerably the expenditure projected. It was decided in 1989 to replace the deck with an orthotropic steel deck as a permanent solution to the problem.

This case study describes the various requirements set for the condition survey of the bridge, the problems revealed in the various structural components and the repair procedures adopted for their correction and prevention including the future repairs proposed.

9.2 Condition Survey - Specifications for Inspection

The consultant for the inspection of the Champlain bridge and approaches carried out detailed inspections since 1986, based on a four year cycle detailed inspection program that was planned in the year 1985 and later modified to better suit the field conditions, availability of inspection equipments and unforeseen problem areas.

Annual inspections are carried out in accordance to the Ontario Structure Inspection Manual (42), and Manual for Maintenance Inspection of Bridges 1983 (32), in conjunction with the Bridge Inspectors Training Manual 70, U.S. Department of Transportation.

The following guidelines were followed for the inspections:

1) The bridge being strategically important is to be inspected annually. Also, one quarter of the bridge is to be inspected in depth every year.
2) Overhead sign supports are to be inspected each year.
3) The underwater portions of the bridge are to be inspected every five years (next inspection is scheduled for 1994).

Before proceeding with the inspection, the available records, as-built drawings, stress sheets, inspection reports and rehabilitation records are reviewed to evaluate the seriousness of the defects encountered and to determine rehabilitation priorities.

The general annual inspection includes a visual inspection of all structural components of the bridge above the ground level and above the water level using special inspection equipment, where necessary, taking into account the available access features. In addition, these inspections include the following:

1) Hammer sounding of deteriorated concrete, scrapping of corrosion materials and

hammering suspect rivet heads.

2) Measurements to verify expansion bearing performance under extremely cold and extremely hot weather conditions.

3) Monitoring of structurally significant cracks as and when required.

Certain special inspections were also carried out depending on the requirements and are given below.

1) Hands-on inspection and hammer sounding of structural concrete components which are not easily accessible in areas where serious defects are suspected.

2) Chipping and hands-on inspection of deteriorated concrete in distressed prestressed concrete beams.

3) Coring and testing of concrete where required.

4) Measurement of steel cross sectional area to detect any significant loss of metal between as built drawings and field inspections.

5) Ultrasonic and dye penetrant testing of pin connections.

6) Ultrasonic testing of significant cracks in substructures.

7) Surveying of structures when abrupt structural movements are suspected.

Deck condition surveys are carried out to detect any significant deterioration requiring rehabilitation. These included the following:

1) Visual survey of the underside of the deck to check the condition of the joints, floor beams and stringers.

2) Coring and testing deteriorated deck components for chloride content and compressive strength.

Inspection Report

A detailed inspection report of the bridge is made based on the various inspections carried out. This involves a description of the general condition of the bridge besides describing the condition of each and every component including the approaches, concrete components including the deck slab, prestressed concrete beams, piers and the substructure, steel components including the trusses, connections, welds, etc, mechanical components including the bearings, joints, etc, and auxiliary components. The report also gives recommendations based on the bridge condition for its regular inspection and maintenance, special inspections and investigations and rehabilitation program if required. Finally, it proposes a ten year major maintenance and capital plan including cost estimates for the entire bridge. It also presents (in a tabular form), all rehabilitation projects recommended in the order of priority, the proposed timing and the estimated budget for each project. Projections are based on the extrapolation of trends in observed deteriorations, reasonable expectations based on experience in maintaining the bridges and on expected useful life of the components, such as roadway pavements, which require periodic predictable repair or replacement at regular intervals.

9.3 Underwater Repairs and Injection of Piers

Scope of Work

The underwater repair work is based on on-site inspection, previously carried out underwater inspection reports and surveys on videocassettes, and involves the following:

1) Caulking cracks and repairing concrete surfaces in the submerged sections and tidal zones of selected piers.
2) Stabbing and removing sound concrete in addition to damaged concrete in some cases to completely remove the damaged concrete within demolition lines marked on the piers.
3) Performing repair work in places only where the current velocity does not exceed six knots (9.6 ft/sec).

In order to make the above work possible, deflectors (see Figure 9.4) made of metal plates, firmly anchored to the river bed or attached to the existing structure were installed to slow down the current below 0.5 knot (0.8 foot/sec).

Drawings were prepared giving the following details:

1) General layout of the bridge.
2) General layout of the piers.
3) Field survey of damage to the piers (see Figure 9.3).
4) Typical details for reparation.

The cost of the work was evaluated based on the nature and scope of work to be performed, difficulties of access to the site, working conditions, and the equipment needed for the work.

Limits of Work and Materials Used

The work is performed in the area extending from the river bed up to 0 5 m above the water surface. Cracks extending down in the river bed are repaired to a depth up to 150 mm below the river bed (for this, granular and other materials are removed up to a depth of 200 mm below the river bed).

The materials (successfully used in the past) specified by the consultant were:

1) Support product for injection - SIKADUR MARINE 36 : Two component gel adhesive with a low modulus of elasticity.
2) Injection product for cracks - SIKADUR 53-ST-1 : Two component epoxy adhesive for grouts.
3) Caulking product for honeycombing and areas damaged due to ice abrasion, etc., - SIKADUR MARINE 45 : Three component pre-proportioned epoxy grout for underwater application.

Before beginning the repairs, on-site tests were performed to determine precisely the best work procedure to be followed. The following repairs were carried out on the piers:

Cleaning and Drilling of Cracks

Each pier is thoroughly cleaned using water and air jets to locate the cracks and damage as shown on the drawings as well as any damage not detected during prior inspections. All surfaces and cracks to be repaired are cleaned with a high pressure water jet. The pressure is controlled to be high enough to produce an effect equivalent to sandblasting (the minimum working pressure was 10.3 MPa).

The cracks are cleaned and prepared as follows:

For cracks 5 mm wide or less, the crack lips are chipped to form a "V" about 10 mm wide. The surface near the cracks are cleaned up to 25 mm on each side using a high-pressure water jet. The inside of the crack is cleaned with a small-sized nozzle of diameter 1.5 mm to allow the water jet to penetrate the crack and remove all dirt, sand, etc (see Figure 9.5, 9.7 and 9.8).

Injection of Cracks

Holes are drilled at pre-determined positions to place the injection tubes, which are spaced at 400 mm on centres and are secured in place during the injection process. The gel adhesive (SIKADUR MARINE 36) is then applied to close the external faces of the cracks completely. The injection process is started as soon as the gel reached the strength required to withstand the pressure of injection (see Figure 9.6). SIKADUR 53-ST-1 has been successfully used as the injection product. The injection begins at the bottom of the cracks and proceeded upward, ensuring that the injection product appears in the upper tubes. The pressure and speed of injection is controlled to allow the water to drain completely and to be replaced by the epoxy. Some hairline cracks (less than 2 mm width) are also repaired in a similar manner.

Repair of Honeycombing and Spalling

The bridge piers had suffered surface deterioration in the form of honeycombing and spalling resulting from wear due to ice, chemicals and other causes. In areas with honeycombed concrete, repairs have been performed up to a depth of more than 30 mm. The surface of the piers is cleaned using a high-pressure water jet. The formwork is fastened to the pier and the periphery sealed using SIKADUR 36. The epoxy grout SIKADUR MARINE 45 is then injected into the formwork to fill it completely. The grout is then allowed to reach adequate strength before removing the formwork and fasteners. Figure 9.9 shows the details of the repairs.

Repair of Porous Concrete

Repairs were performed where the concrete was porous up to depths of 30 mm. The porous concrete was removed using pick-hammers and the surfaces were cleaned by a high pressure water jet. Formwork was then fastened to the pier and the periphery sealed using SIKADUR 36. The epoxy grout SIKADUR MARINE 45 was injected in the formwork to fill it completely and allowed to set before dismantling the formwork and fasteners as mentioned earlier. Figure 9.10 shows the details of the repairs.

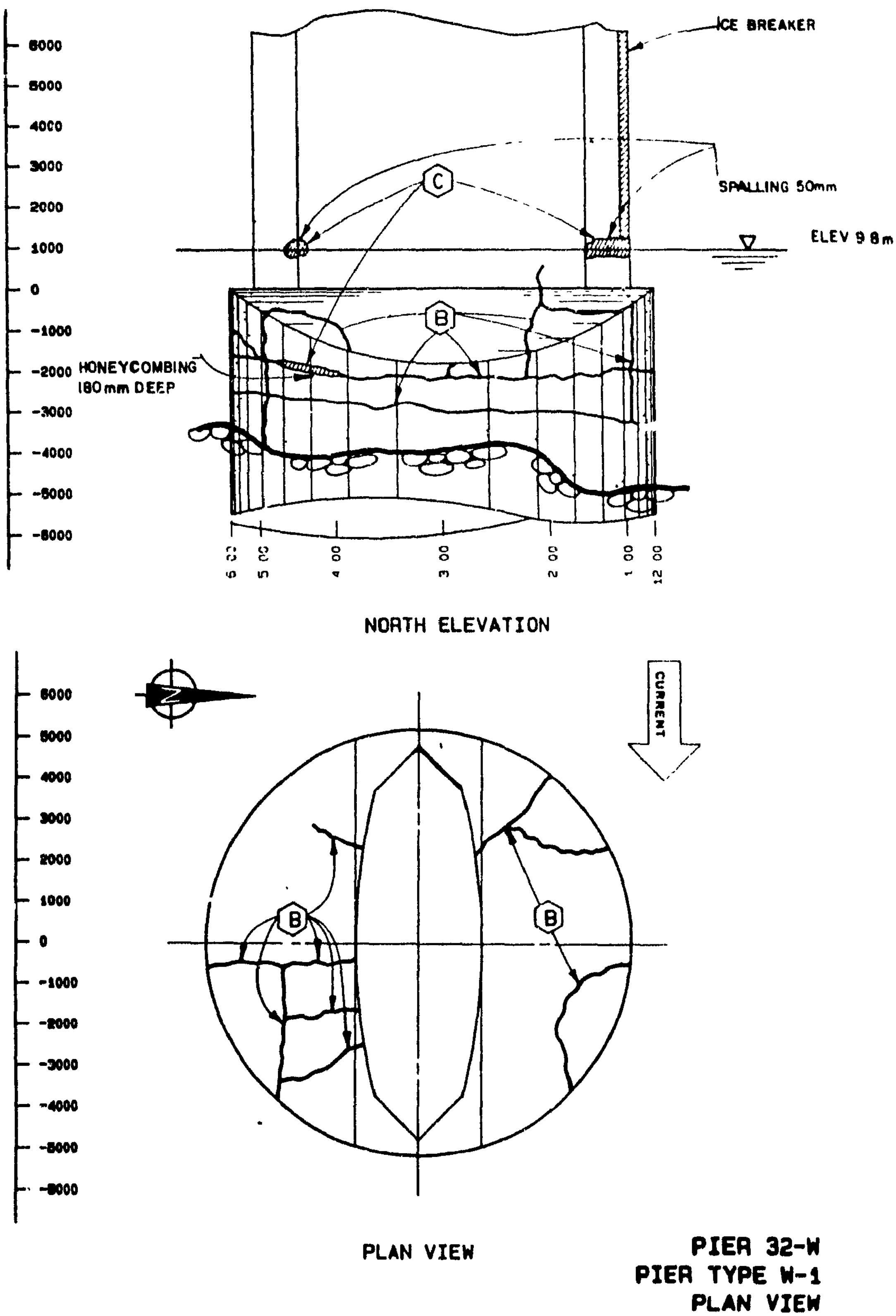


Figure 9.3 Typical Pier - Typical Damage and Deterioration (Field Survey Results)

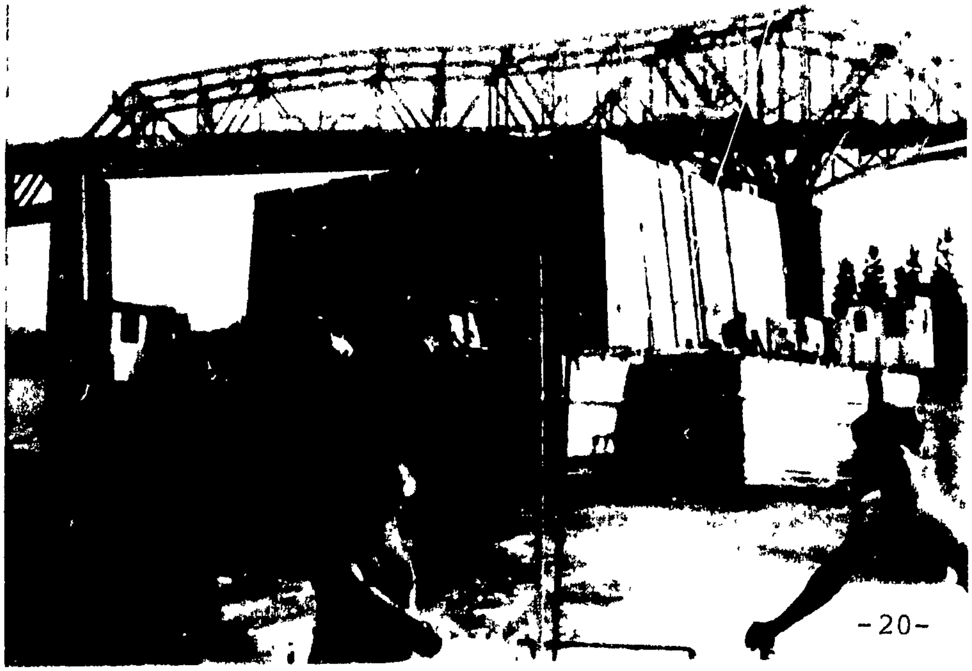


Figure 9.4 Metal Plate Deflectors

Figure 9.5 Injection Cleaning of Cracks

Figure 9.6 Epoxy Injection of Cracks

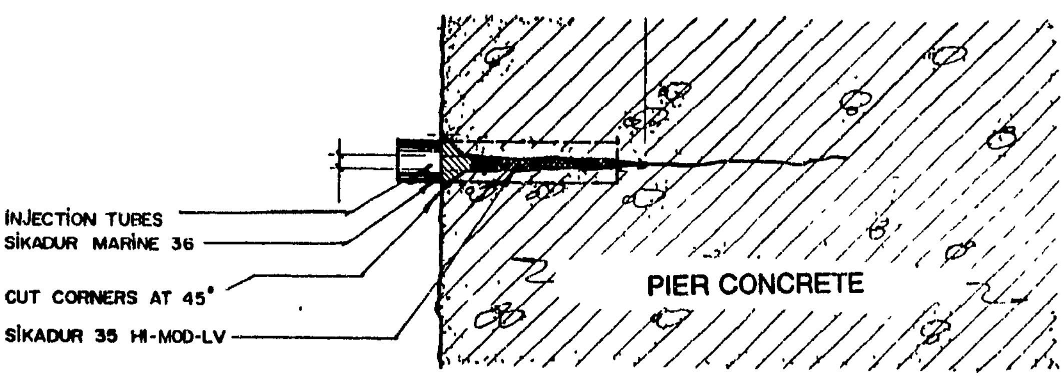


Figure 9.7 Repair of Narrow Cracks

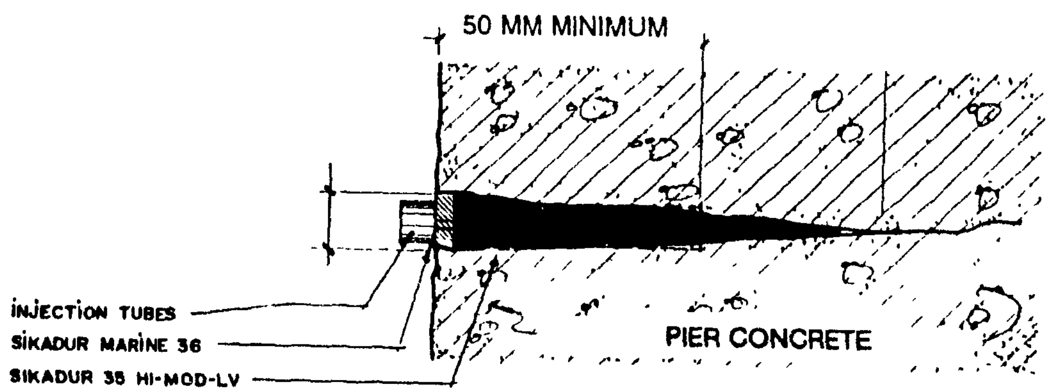
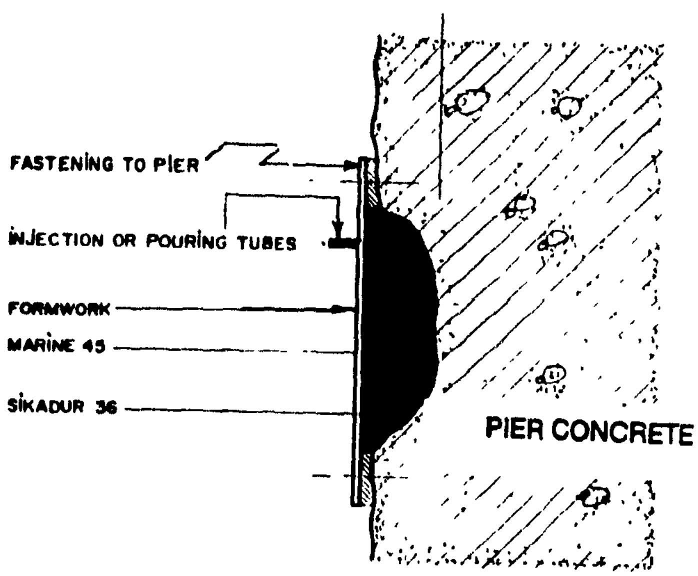


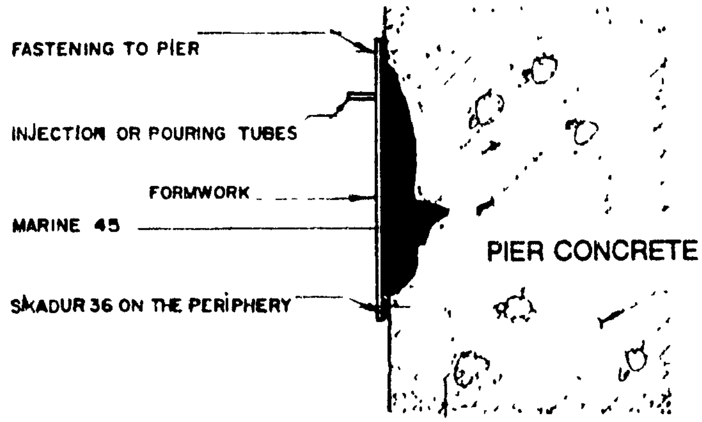
Figure 9.8 Repair of Wide Cracks



REPAIR STEPS

1- CLEAN CAVITY WITH WATER JET
2- INSTALL FORMWORK ON PIER SIDE AND FASTEN SEAL WITH SIKADUR 36 ON THE PERIPHERY
3- INJECT MARINE 45 IN THE FORMWORK SO AS TO FILL HONEYCOMBING COMPLETELY

Figure 9.9 Repair of Honeycombing and Spalling



REPAIR STEPS

1- REMOVE CRUMBLING CONCRETE WITH MECHANICAL TOOLS AND WATER JETS
2- CLEAR CAVITY WITH WATER JETS
3- INSTALL FORMWORK ON PIER AND FASTEN, SEAL WITH SIKADUR 36 ON THE PERIPHERY
4- FILL UP FORMWORK WITH INJECTED MARINE 45

Figure 9.10 Repair of Crumbling

9.4 Repairs to Pier Tops and Beam Ends

The top surface of the bridge piers is susceptible to continuous accumulation of salt bearing water dripping from the open drains which has resulted in considerable deterioration of the concrete and corrosion of steel reinforcement. Similar deterioration was observed at the beam ends as well as to the underside of the deck where spalling of concrete had occurred exposing the reinforcing steel. The repair program was initiated in 1988. A typical repair program performed in 1990 involved rehabilitating concrete on all areas of Pier 3W on the west side and on the top of the piers and on the underside of the slab and at the beam ends at Pier 29W and at the subsequent piers in the eastern direction. In all, 7 piers were repaired beginning with Pier 3W, followed by Piers 29W to 23W (except Pier 27W).

Rehabilitation Work

In certain areas of Pier 3W, chipping and removing of concrete had already been executed, however, additional chipping and removal was required on the above areas and in other areas to be repaired Scaffolding was installed over the full height of Pier 3W since the entire pier concrete was to be rehabilitated Unsound and deteriorated concrete was demolished, removed and disposed Precautions were taken to avoid any damage to the reinforcing steel during chipping The concrete surfaces were prepared by sandblasting all existing concrete surfaces and the reinforcing steel which were to receive the new concrete. Any dust and debris was cleaned using compressed air and water The surfaces were wetted sufficiently, prior to their receiving the concrete, so that there was no excessive absorption of moisture by the concrete Care was taken to ensure that the existing concrete surfaces were made not too wet as to overcome suction, and free water was not allowed to remain on the surfaces Also, the existing reinforcing bars were rigidly secured to the existing concrete to reduce vibration to a minimum during the concreting operations. The exposed reinforcing steel was brush coated with two coats of epoxy patching compound. Pump mix concrete with suitable air entrainment and water reducing admixtures was used to repair the surfaces. The concrete used had a 28-day compressive strength of 30 MPa and the temperature during the placing operations was between 10 to 30 degrees celsius.

A similar procedure was followed for repairing the horizontal concrete areas on top of the hammerheads and a 150 mm height strip at the top of the hammerhead and all around it.

The underside of the deck, diaphragms located on top of the piers, the underside of the beams and the beam ends were repaired where it was possible to erect forms. The forms were injected with a commercial grout Nitromortar EL and allowed to remain until the grout gained sufficient strength. The surfaces were finished after removing the forms.

Figures 9.11 through 9.13 highlight the areas repaired using the above procedure.

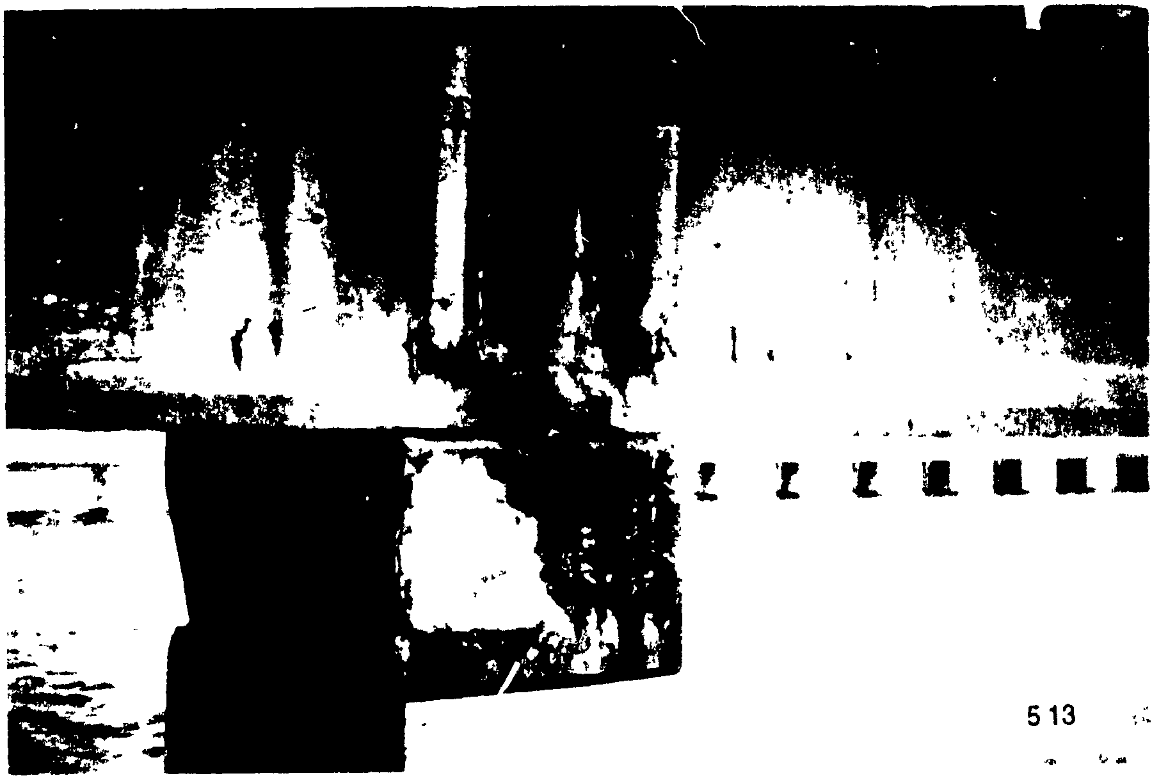


Figure 9.11 Side View of Pier showing Spalled Concrete

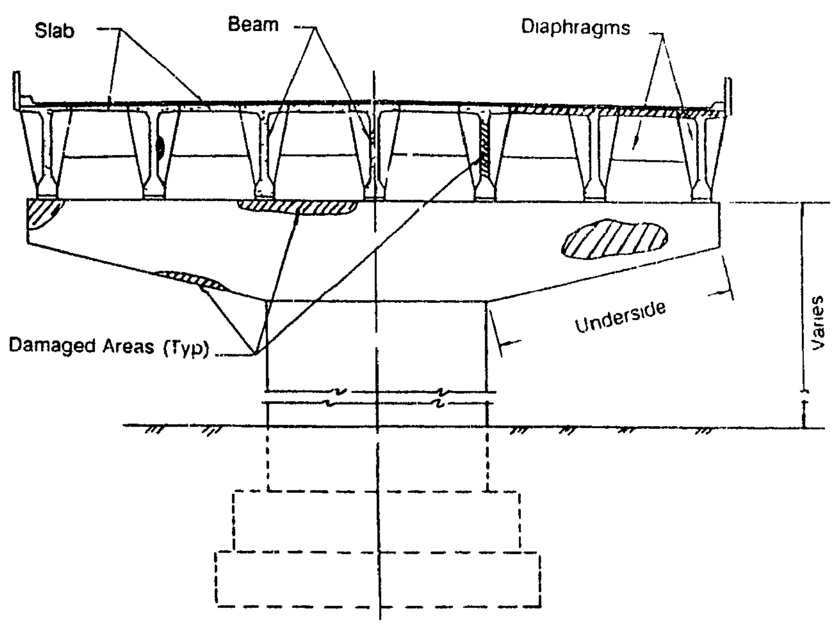


Figure 9.12 Typical Transverse Cross Section of Bridge Indicating Repaired Area

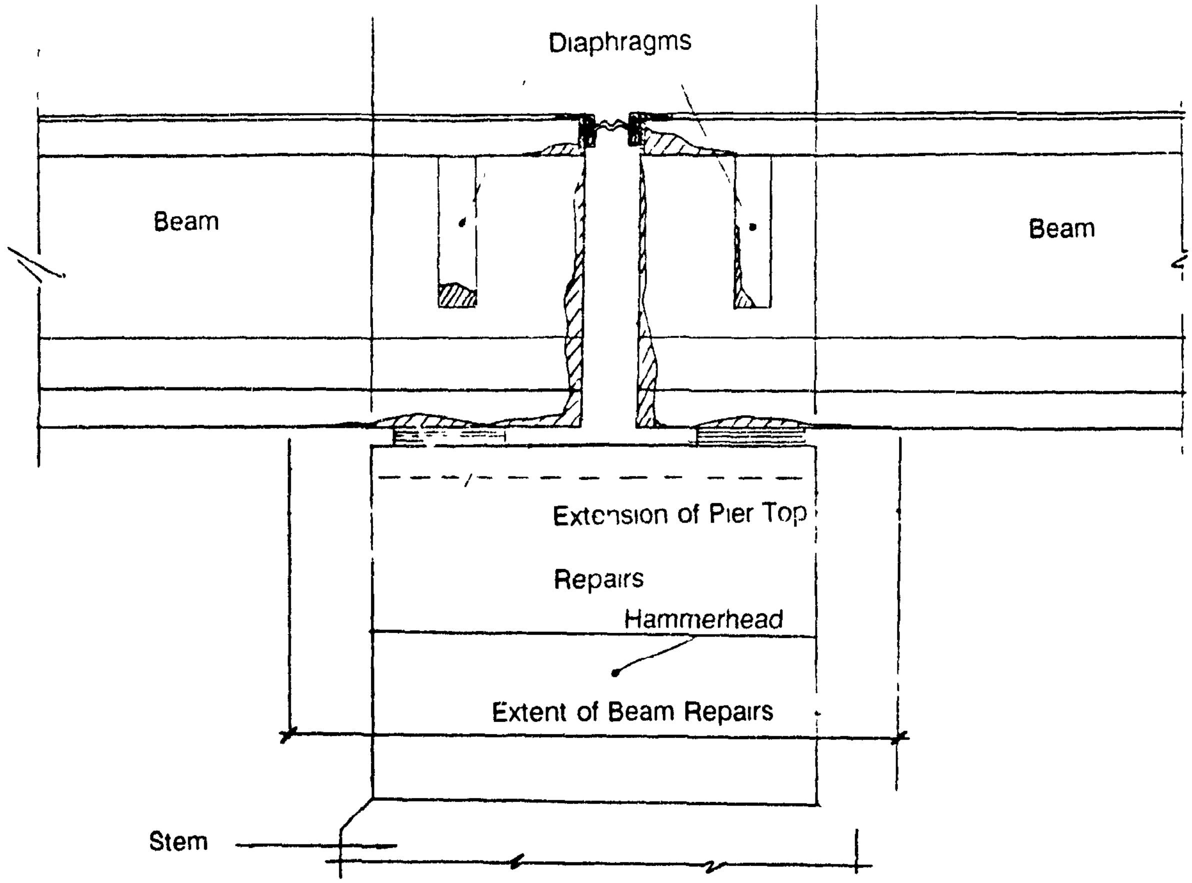
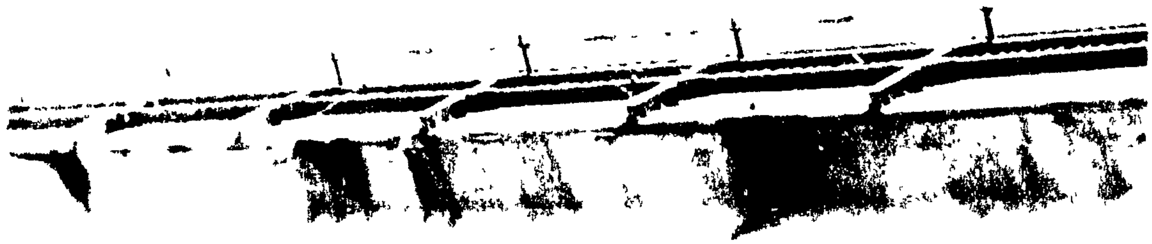


Figure 9.13 Typical Longitudinal Cross Section of Pier Indicating Repaired Area

9.5 Repairs to Prestressed Beams

9.5.1 General Repairs and Application of Impervious Membranes

The post-tensioned beams girders had been subjected to severe chloride attack due to dripping of the salt borne water over them. Spalling of concrete had occurred in several places exposing the reinforcing steel and the prestressing tendons, which had corroded (see Figure 9.14). Several tendons had ruptured and many were close to failure. A general survey of the beams had led to the conclusion of carrying out repairs to selected beams of the bridge needing immediate repairs. A more permanent repair program involving the installation of a cathodic protection system is envisaged with test systems presently implemented and is explained later.



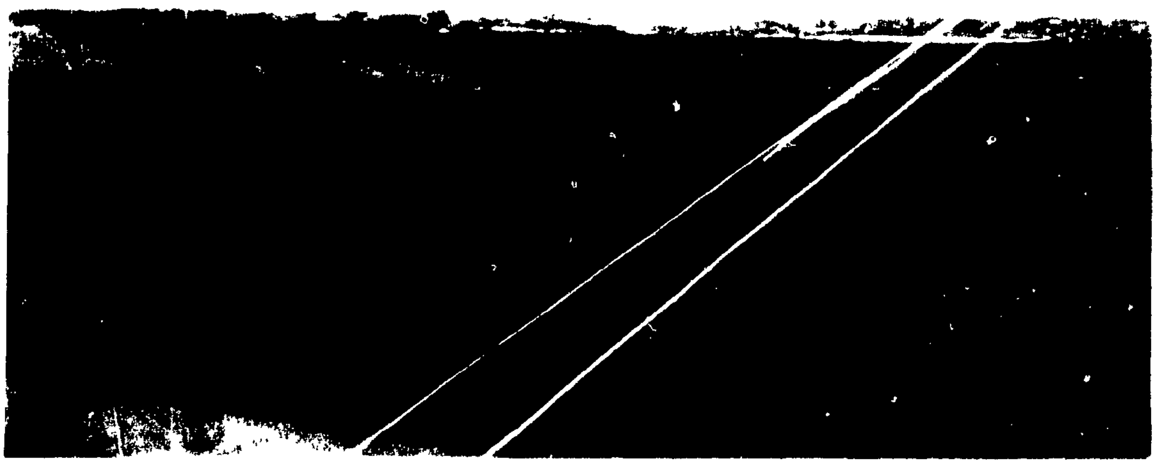


Figure 9.14 View of Underside of Beams Showing Exposed Prestressing Steel

The following general repairs were carried out:

a) <u>Repairs to Underside of Beams</u>

The edges of the prestressed concrete beams were demarcated where deterioration had occurred. These areas were sawcut up to 18 mm depth. The concrete was removed by chipping using power hammers avoiding any damage to the existing tendons and the reinforcing steel during chipping. The undersides of the beams were sandblasted to clean the steel and concrete. The exposed steel was painted with two coats of an epoxy compound. A galvanized reinforcing steel wire mesh (f_y=400 MPa) was anchored to the underside of the beams where the concrete had been removed. Watertight formwork was installed under the beams and then these areas were grouted. The grout was allowed to gain strength before removing the forms and finishing the surfaces. In all, six exterior beams of Section 7 of the bridge were repaired using the above method.

b) Repairs to Diaphragms

Repairs were carried out to all of the end diaphragms on the outside face of seven upstream and seven downstream beams in Section 7 of the bridge. The repairs involved sawcutting the edges of all concrete areas up to 18 mm depth where concrete to be removed was demarcated. The deteriorated concrete was chipped without damaging any of the existing good reinforcing steel followed by sandblasting to clean the steel and the concrete. The exposed steel was painted with two coats of an epoxy compound. A galvanized wire mesh was installed and anchored to the surfaces where concrete had been removed All surfaces were cleaned of any dust and debris using compressed air and water. The surfaces were thoroughly wetted and dried before shotcreting, taking precautions not to overdry the concrete to avoid excessive absorption of moisture from the shotcrete. Shotcrete was then applied to these surfaces. During the mixing operation, 19 mm long polypropylene fibre was introduced in the shotcrete mix at the rate of 1000 kg/m^3. The shotcrete was then cured using two coats of a commercial curing compound at the rate of 5 m^2/litre per coat.

c) Waterproofing

A protective and waterproofing membrane was applied to the bottom part and along the cracks of the outside face of seven upstream beams and an equivalent number of downstream beams. Before waterproofing, the surfaces were sandblasted. It was ensured that the surfaces to be coated were sound, dry, clean and free of all dirt, oil, tar, asphalt, sealer, curing agent, coatings, loose particles, laitance and other contamination and foreign matter which may interfere with the adhesion of the membrane. Spalled areas, if any, were cleaned and made level with the existing surface. Concreting was carried out on the vertical faces whereas the bottom surfaces were shotcreted. A 60 mil thick coat of a proprietary VULKEM 450 membrane, 200 to 250 mm wide, was applied to treated expansion joints, hairline cracks and cold joints. A similar coat was applied over the entire surface to be treated. The membrane was allowed to cure before applying the protective coating which consisted of a 6 to 7 mil thick coat of VULKEM 451. It was ensured that concreting was completed at least 14 days prior to the application of the membrane and where shotcreting was required, a minimum of seven days were allowed prior to applying the membrane.

d) External Prestressing

External prestressing was carried out on some external beams of the approach spans which had shown signs of loss of strength due to the severely corroded prestressing steel. The beams were transversely anchored externally at selected locations (anchor blocks) using diwidag bars. The anchor blocks were reinforced with steel reinforcement. External prestressing tendons within metal pipes were provided along the length of the beams at the bottom. They were anchored at the anchor blocks and were stressed to provide the necessary prestressing force. The anchor blocks were concreted after the stressing operations were completed. Figures 9.15 and 9.16 highlight the repairs carried out.

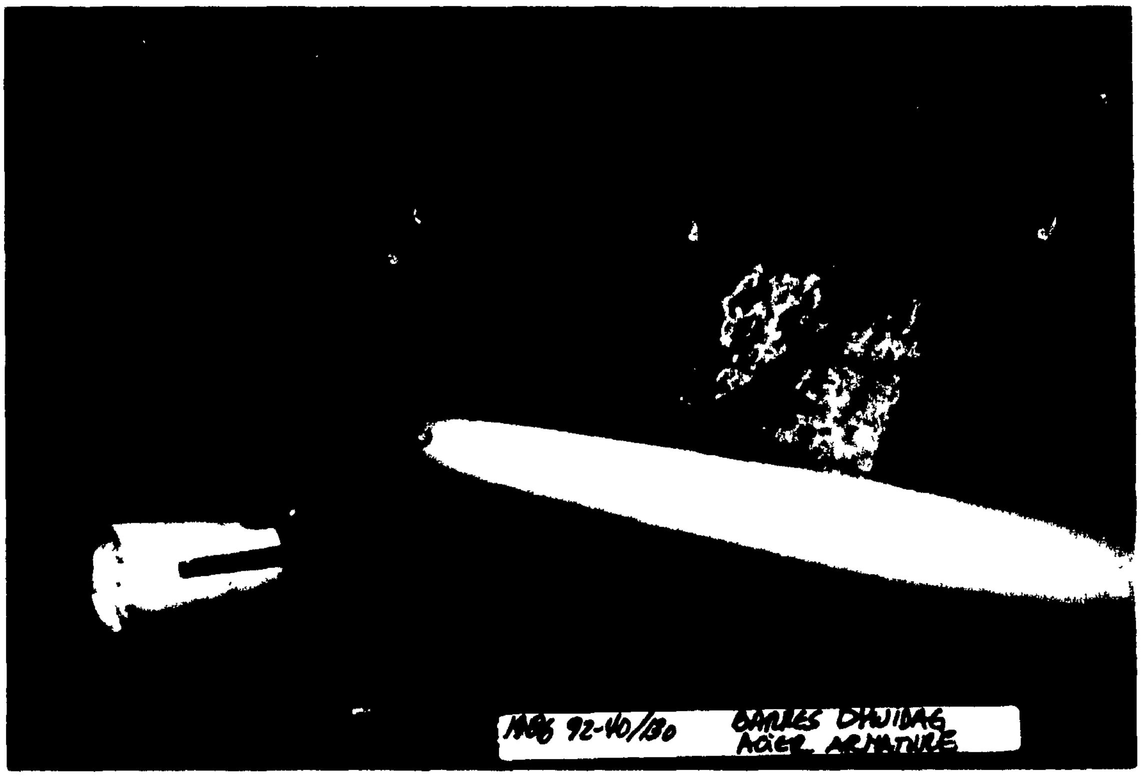


Figure 9.15 Anchor Block Showing Diwidag Bars and Steel Framework

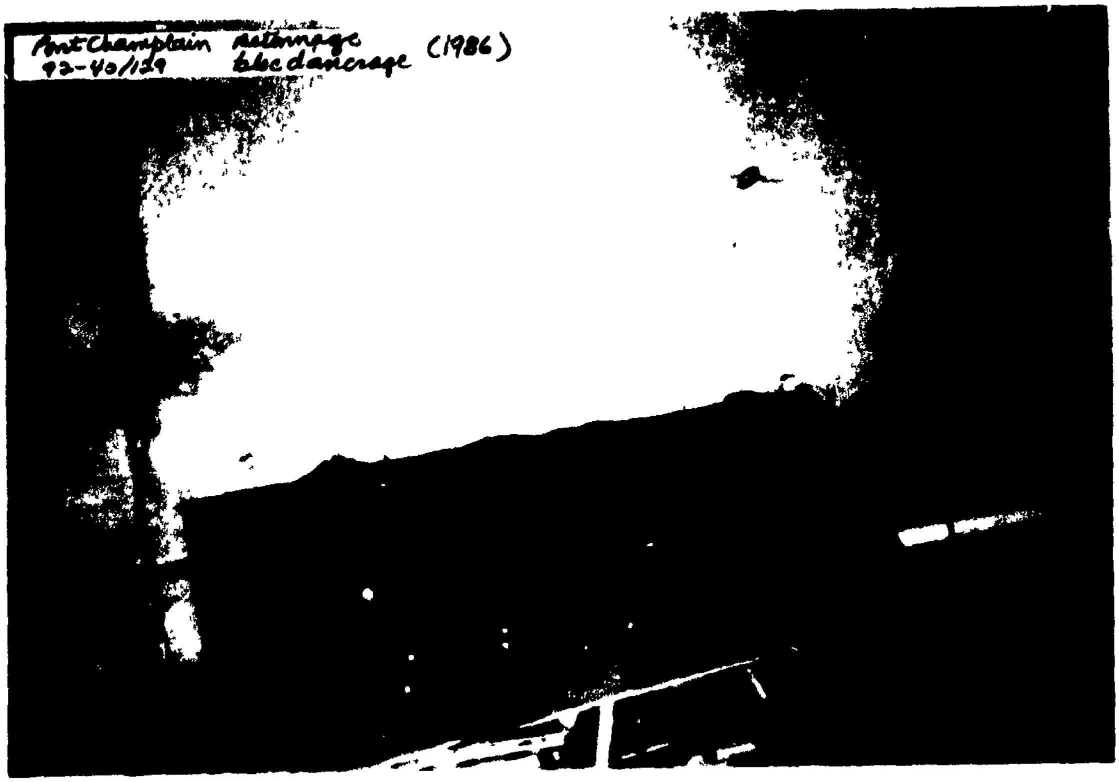


Figure 9.16 Concreting of Anchor Block

All of the above repairs were implemented from below the structures outside of peak hours when traffic was closed on one lane of the bridge.

9.5.2 Application of Cathodic Protection System

As mentioned earlier, the main prestressed concrete beams of the bridge had suffered deterioration due to chloride ion attack from the salt borne water dripping from the bridge. In certain areas like the toll booth area on the Montreal side approach, this was so severe that significant levels of corrosion had impregnated the concrete. In certain areas, tendons had ruptured and were close to failure. External tendons were subsequently added to several exterior beams to re-establish the integrity of the members as explained earlier.

In 1989, Corexco Inc., a Montreal based firm, worked under contract for the Jacques Cartier and Champlain Bridges Inc. to design, supply and supervise the installation of a test cathodic protection system on several beam sections of the bridge. This was a trial system to determine the field performance and feasibility before the system would be installed elsewhere on the bridge.

Components of the System

Three distinct systems were installed on five metre sections of select beams. The difference between them is based on the particular cathodic protection criteria. However, the main components of the systems and their configurations are essentially identical.

The supplementary anode used is an expanded thin, lightweight, diamond-shaped titanium mesh that enables application to many surface shapes. The mesh is wrapped around the sides and bottom of the beam in 900 mm wide pieces to be later covered with a 25 mm overlay of concrete. The mesh is fastened to the surface of the concrete by plastic nails driven into pre-drilled holes. The mesh can be easily built around circular members and cut to fit to provide versatility.

The current is fed to the anode from the positive terminal of the power supply through positive cables and current distributer bars. These bars, also of titanium, ensure an even distribution of the current across the mesh. The distributer bars are welded to the anode mesh at regular intervals. It is crucial that the anode not be in contact with any metal surface, especially the reinforcing cage. To avoid such short circuits, the concrete surfaces are inspected prior to the installation and any metal is isolated from the anode.

Several reference electrodes are included as part of the monitoring system. These electrodes are of pure high quality graphite and are 12 mm in diameter and 100 mm long. As is the case with the anode, it was mandatory that the reference electrodes not be in contact with the steel cage as the current values read would be incorrect.

A standard 110 volt alternating current power supply is fed through the rectifier which provides the direct current mode and the low amperages required. Both the voltage and the amperage of a rectifier are adjustable and in this case the maximum output is 2 volts DC and 20 amperes.

The remote monitoring unit provides immediate access to all of the important electrical information at any time from any office. Not only can the voltage and the current of the rectifier as well as the potential of each reference electrode be read, but the operator is able to switch the rectifier on and off as required. The unit consists of a multimeter, a cellular phone, a modem and a central electronic board that ties the components together. Links from the rectifier and reference electrodes are fed to the box which houses the remote monitoring unit. Inside the box, a multimeter reads the various currents, voltages and potentials and the information is relayed through the board, modem and telephone to a personal computer at Corexco's office. This monitoring unit enables routine checks of the system and steel protection levels from the office and eliminates the need for site visits.

Installation Sequence and Details

Surface Preparation

Initially, the beams were thoroughly inspected for any visible metal such as chairs to separate the reinforcing steel from the forms as well as the ties to the reinforcing steel. The corrosion of the post tensioned tendons of these beams was so advanced that visible lines of corrosion by-products were seen on the underside of the beams. All of the exposed metal and these ferrous traces had to be completely covered with a thin coat of epoxy to ensure isolation from the anode so that no short circuiting would occur.

Continuity Testing and Connection of Negative Cables

While the surface was being prepared, the concrete was carefully chipped away in a designated areas to expose both the post tensioned tendons and the mild steel stirrups. The connection of the negative cable to the steel cage was crucial as only one was achieved. The copper cable was tapped through a large gauge stirrup and properly insulated with tape. Once several steel ducts and bars were exposed, the continuity between the steel elements was tested using a multimeter.

Installation of Reference Electrodes

Holes were carefully drilled into the underside of the beam for the graphite reference electrodes taking care to ensure that no steel was hit with the drill bit. After a prospective hole was drilled, the graphite rod was inserted and continuity was tested with the reinforcing cage using a multimeter. If there had been any contact with steel, a new hole would have to be drilled. Each electrode hole was lined with fresh concrete (mortar) before the electrode was inserted in order to guarantee that the electrode was in proper contact with the electrolyte (concrete). The holes were sealed with the repair mortar after the electrodes were in place. The testing, installation of electrodes and connections was followed by repairs to the concrete with the same mortar.

Placement of Anode Mesh

The next step in the installation sequence was the placement and connection of the anode mesh. The mesh was stored in large rolls. Sections were unrolled, cut and attached to the sides and underside of the beam with plastic fasteners (to avoid short circuits). Enough tension was applied to keep the mesh snug against the concrete surface.

Once the mesh was in place, a titanium current distribution bar was welded to the mesh along the underside of the beam using a lightweight spot welding unit. Subsequently, the positive cables were connected to the current distribution bar and properly insulated.

Shotcreting

For this step, all of the exposed, connected cables were rolled up and protected from the concrete. Shotcreting was carried out on the beam to the desired thickness. The surface was lightly trowelled to enhance the appearance.

Installation of Conduits and Electrical Equipment

All cables, including positive, negative and electrode wires were run externally to the rectifier/remote monitoring unit location, approximately 25 metres away from the beam. All cables were housed in 25 mm poly vinyl chloride (P.V.C.) plastic piping to ensure protection from the surrounding. The rectifier and the remote monitoring unit were installed later and all of the electrical connections were made. All of the components were checked for any problems prior to energizing the system.

Performance

The pilot project has been monitored for about two years and found to be working effectively. It has therefore been proposed to install similar test systems on several complete beams. The monitoring of hydrogen gas formation is planned for the next test phase.

Figure 9.17 highlights the cathodic protection systems used.

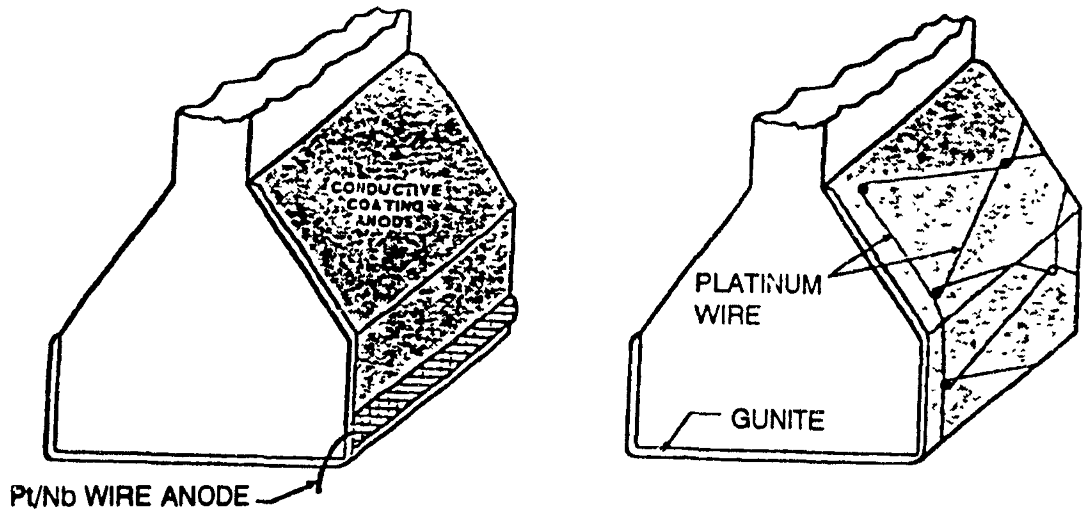


Figure 9.17 Conductive Coating and Platinum Wire Net Systems

9.6 Replacement of Reinforced Concrete Deck by Orthotropic Steel Deck

The Champlain bridge is one of the busiest in Canada with more than 41 million transits per year. As mentioned previously, the reinforced concrete deck constructed in the late fifties, was not designed with any bituminous asphaltic overlay or impervious protecting membrane. This coupled with the considerable increase in live loads and the use of deicing salts contributed to severe deterioration of the deck in the form of local delamination of the concrete from the top reinforcing steel. Since 1983, the deck has required an annual patch work program (full and partial depth repairs) which has contributed not only to an increasing maintenance budget, but also been a constant obstruction to the heavy vehicular traffic.

The idea to replace the concrete deck was initiated in 1989. Various alternatives were examined as possible solutions to the problem. These are summarized below.

1) Replacing the Existing Deck with a New Reinforced Concrete Deck

From the design point of view, the thickness of the new deck could be easily maintained to be the same or smaller depth than the existing deck (165 mm at main span and 190 mm at local approaches). However, this solution was unacceptable due to the increased amount of reinforcing steel required according to the new codes (nearly 50 percent increase in live loads allowed than those at the time of the original bridge design) Also, the new deck would require a bituminous asphaltic overlay of 50 to 65 mm thickness to prevent any deterioration due to the application of deicing agents. Calculations of dead weight of the deck and overlay showed that the new alternative would cause overstressing in the lower steel trusses of the approaches to the main span and cantilevered superstructure Furthermore, the construction would have required the closure of a minimum of three traffic lanes which would greatly disrupt traffic especially during the peak hours. Closure of lanes also would have a negative economic impact on the sectors served by the bridge. The phenomenon of microcracking of a newly cast deck due to vibrations from adjacent traffic presented the risk for premature deterioration of the deck. The above factors led to discarding this alternative

2) <u>Replacing the Existing Deck with a Steel Grid Open Deck</u>

This alternative was found unacceptable due to the following reasons:

a) Corrosion would have accelerated in the structural steel element of the truss due to the salt water escaping through the deck openings.

b) Maintenance of the deck welds was high.

c) Driving on such a deck would be uncomfortable.

3) <u>Replacing the Existing Deck with Concrete filled Steel Grid Closed Deck</u>

Although this alternative was quite efficient due to the composite action with the deck, it was less resistant to deicing agents and freeze-thaw action, thereby causing segregation of the concrete into individual blocks, resulting in the loss of composite action.

4) <u>Replacing the Existing Deck by a Prestressed Precast and Post Tensioned Concrete Deck</u>

This alternative was somewhat cheaper than provision of the orthotropic deck. However, it required considerable vertical and horizontal bracing below the deck during erection presenting difficulties. Also, it had a shorter lifespan than the orthotropic steel deck and therefore, it was ruled out.

5) Replacing the Existing Deck with Orthotropic Steel Deck

This alternative offered several structural advantages of being light weight (nearly 50 percent reduction in the dead weight yielding much higher live load carrying capacity), high degree of flexural and torsional resistance and a good load distribution behaviour due to composite action with the floor members and the longitudinal trusses. Also, the orthotropic deck requires less maintenance and it is economical. It was therefore decided to implement this alternative and a detailed program was worked out for the same. The structural advantages of orthotropic steel plate decks and the design required for redecking are reviewed in Appendix 4.

Design, Manufacture and Installation of Orthotropic Steel Deck

Design

The steel deck is made of high grade steel (300 WT, Category 5) with a thickness of 16 mm. It had a minimum Charpy value of 27 Joules at -20°C which allowed it to overcome embrittlement at low temperatures and absorb the high impact energy due to the moving vehicles.

The nominal size of the deck plate was 19.5 m x 4.6 m at the main span although some spans had an overall length of 9.8 m as required by the bridge geometry and the spacing of the expansion joints. Larger panel sizes could not be used in the main span since the capacity of the cranes was limited due to the low overhead clearance (4.9 m) available at the bridge. At the approaches, there was no overhead clearance restriction and therefore the deck panels were made of a larger size, 19.5 m x 9.2 m. However, to facilitate transportation of these panels, they were fabricated in two sections, each weighing about 25 tons.

In total, 210 orthotropic steel deck panels were estimated to redeck the bridge which represented a total surface area of about 18,590 m^2.

Manufacture of Orthotropic Steel Deck

The steel decks were shop fabricated as follows. The deck plate was cut to size and blast cleaned to remove any mill scale. The 9.8 m long rib plates were cleaned using a wheelabraiter and then cut to size using a plasma cutter. Holes were drilled for the inter-rib bolted connection and the plate was then bent into shape along its length using a 1000-ton press.

The 9.8 m long ribs required for 19.5 m long panels were initially tack welded together to secure the ribs for the automated seam welding operation. The camber requirements were incorporated into the fabrication process. An automatic welding machine with six heads was used to perform the 80 percent partial penetration weld between the ribs and the deck plate. Three ribs were maintained in each of the two passes with two heads simultaneously working on the two sides of a rib to avoid any distortion. A run-on tab was used to stabilize the welding arc before passing the welding machine over the underside of the deck whereas a run-off tab was used to perform non-destructive testing for quality control. The ribs were then hermetically sealed with the end diaphragms to prevent any moisture ingress

The top surface of the panels was then cleaned and an inorganic zinc coat was applied This was followed by a 10 mm thick epoxy tack coat on which trap rock was scattered to provide an abrasive surface for vehicles for about a two year period until the deck receives its final wearing surface (this will be implemented only after all of the 210 deck panels are installed). The exposed underside of the deck and ribs were painted with two coats of vinyl mastic high build paint. It will finally receive a third coat of vinyl mastic high build when field erection of the panels is completed

Installation

The work of installation commenced in the fall of 1990. The panels were transported to the site on trucks as soon as they were ready. The existing deck panels were cut out using a circular saw and lifted using two cranes. The orthotropic steel deck panel was then placed in its final position using the same two cranes. Splice plates were used to join the ribs of the new steel plate with those of the adjoining new panels already in place. The deck plates of the adjoining new panels were welded together using a 30 degree bevelled penetration weld. The plates were preheated before welding to remove moisture. Since the truss portion of the bridge is a cantilevered structure, it requires a balance to be maintained by the anchor arms relative to the cantilever arms of the bridge. Therefore, the sequence of installation was carefully considered. Also, analysis of the balance of the bridge was performed which indicated the need for installing anchors at the ends of the three trusses where they rest at Piers 2E and 2W. A system of counterweights was employed to offset the weight difference (nearly 40 percent reduction in the weight of the panels during installation) basically not to change the load distribution pattern severely and cause overstress in any member.

The work is scheduled to be completed by fall of 1993 at the rate of one panel per day working mainly at night (starting at 20:00 hours closing up to four of the six lanes keeping one lane in each direction and re-establishing the normal traffic flow at 6:00 hours).

The epoxy asphalt wearing surface that will eventually cover the entire deck is a two component mix of epoxy resin and petroleum derived asphalt which reacts chemically while hardening The mix is about 15 percent epoxy to 85 percent asphalt by weight. It has a life expectancy of more than 20 years and it will also act compositely with the steel plate deck thereby increasing its load distribution properties and stiffness.

Figures 9.18 through 9.24 highlight the various aspects of the installation of the orthotropic steel plate decks.

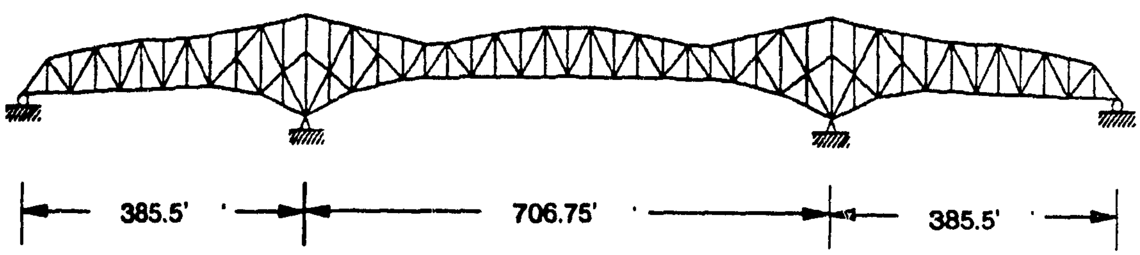


Figure 9.18 Longitudinal Section of Truss Portion

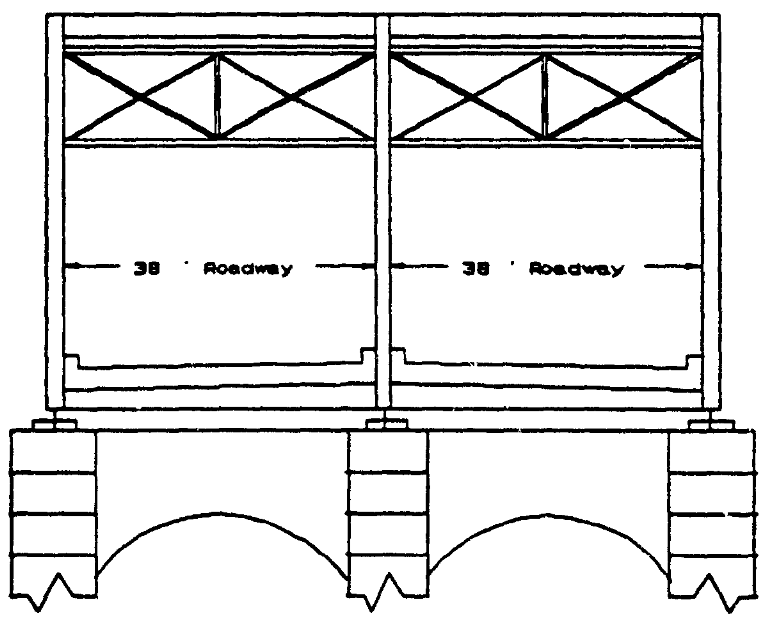


Figure 9.19 Cross Section of Bridge

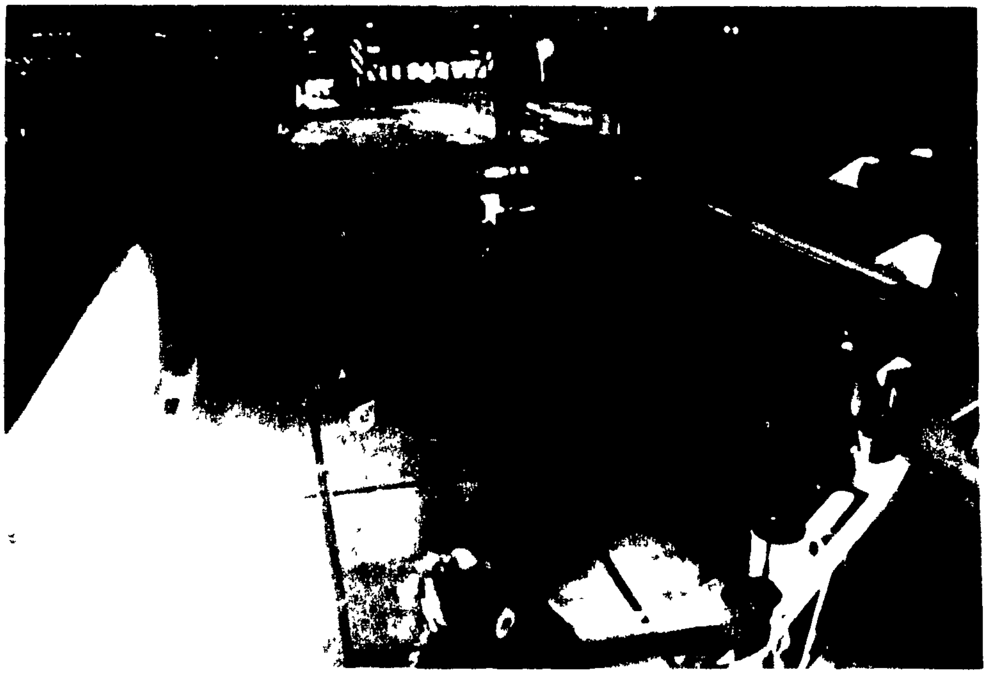


Figure 9.20 View of Underside of Deck

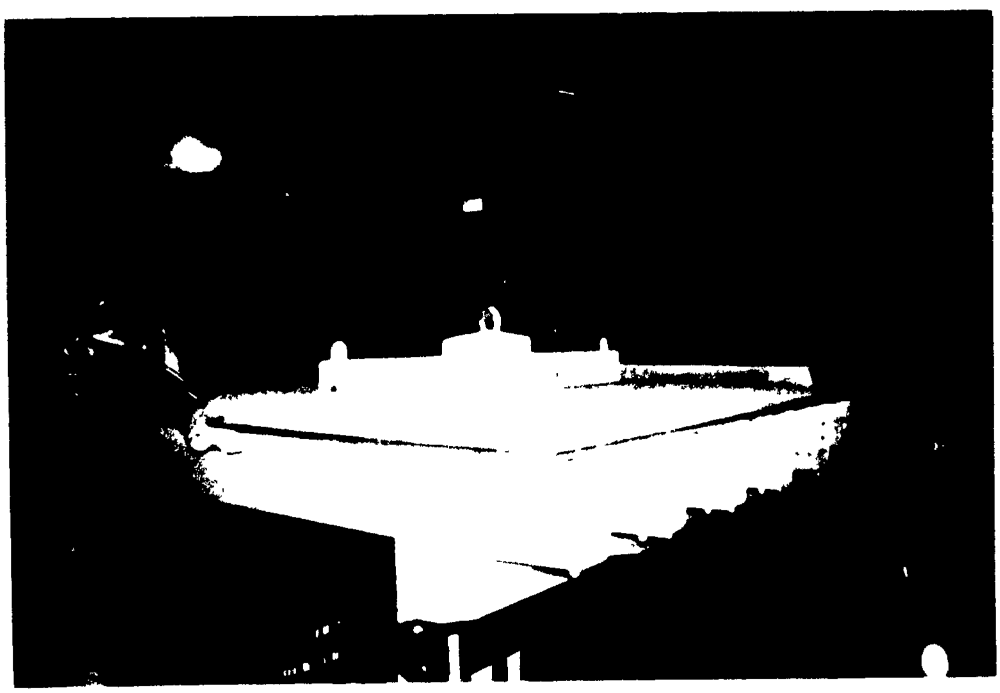


Figure 9.21 Lifting and Placing of Orthotropic Steel Deck

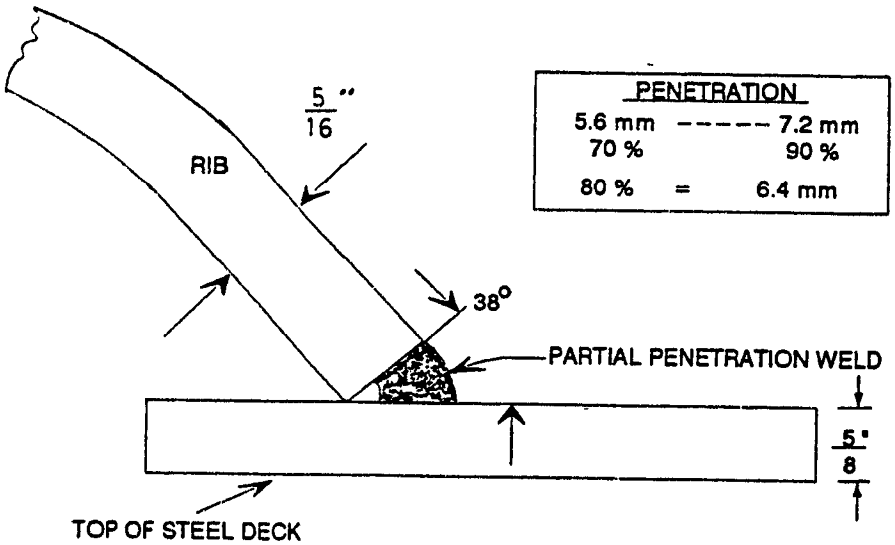


Figure 9.22 Deck to Rib Weld

Figure 9.23 Temporary Finishing of Deck

Figure 9.24 View of Partially Completed Deck

9.7 Replacement of Moulded Rubber/Steel Expansion Bearings

A proposal to replace the moulded elastomer expansion bearings of the bridge by new moulded rubber/steel expansion bearings has been finalized. Some expansion bearings are in a advanced stage of deterioration due to the ingress of chlorides present in the salt borne water draining from the open drains of the bridge and accumulating over the pier tops. It has been proposed to replace the expansion bearings on eight selected piers, which have deteriorated the most

The replacement work will consist of installing jacking equipment (specified in the drawings) which includes the necessary valves, piping, pumps and shims for gauging the elevation at each girder. Freyssinet type flat jacks will be used for jacking purposes. These will be inserted in the gap between the girders and the piers at the desired position. Some local removal of concrete may be necessary to accommodate the jacks since the available space is very limited and not perfectly flat (see Figure 9.25). It is proposed to use a synchronous lift system for simultaneously jacking all girders. Each span will be jacked in steps of 1.5 mm, or less, and the change of elevation will be verified at each girder. Securely attached machined reference pads will be used for measuring the changes in the elevation between each girder and the pier cap.

Each flat jack will be used only for a single jacking. Also, the jacks and the auxiliary equipment will not be used beyond three quarters of their rated capacity. The jacking, blocking and lowering of spans will be carried out during night hours only (00·01 hours to 03.00 hours). The vehicular traffic will be stopped during this period, however, only a maximum of three hours of interruption to the traffic will be permitted The jacking will be performed against the dead load only (maximum loads at a bearing are 115 tons due to dead load and 97.3 tons due to live load).

The existing bearings will be removed without damaging the concrete bearing surface, which will then be chipped, ground and cleaned thoroughly of any adhering parts of the old bearings and loose material. Any deep pits or grooves found shall be filled with a quick setting grout. Galvanized steel plates will then be installed in position over which the new moulded rubber/steel bearings shall be installed. Typical dimensions of the steel plates and the bearings shall be as follows:

Steel Plates	420 × 640 × 6 mm
Bearings	305 × 610 × 41 mm

The bearings shall be installed only at a time when the ambient temperature forecast is around ±4°C.

The spans shall then be lowered on to the bearings in a controlled manner by reversing the jacking procedure. The entire work is scheduled to be completed within about three months.

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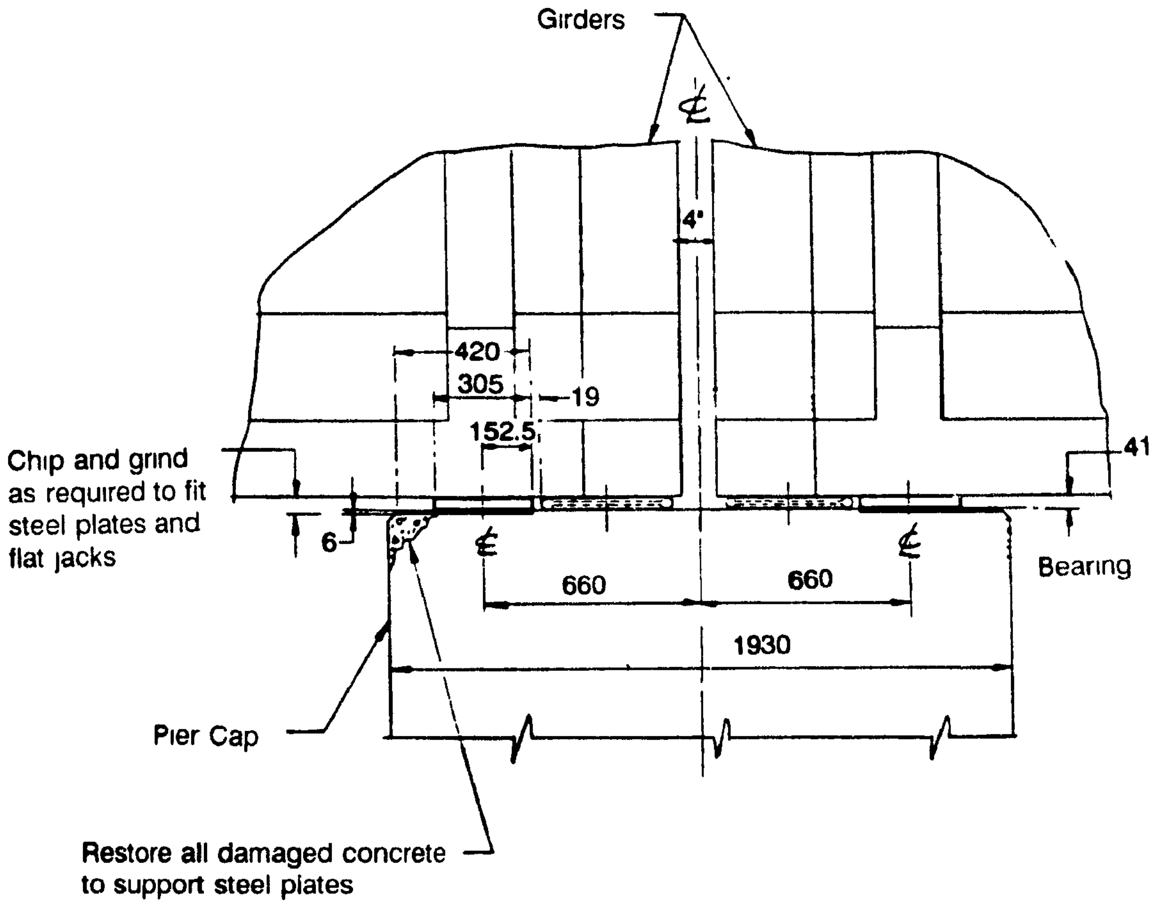

Figure 9.25 Proposed Repairs to Bearings

Other repairs planned in the future include painting of the steel superstructure in 1994-95.

CHAPTER 10
SUMMARY AND CONCLUSIONS

Rehabilitation and strengthening of bridges takes many forms, from simply adding to a members cross section to a complex bridge deck replacement and primary strengthening of members, involving significant secondary effects during construction. This thesis focuses on developing an appreciation of the available alternatives through an in-depth study of the various defects encountered in bridges and the rehabilitation and strengthening techniques used, based on actual bridge retrofit projects. In addition, some topics related to management and rehabilitation of bridges are reviewed, including the following:

1) The basic concepts involved in rehabilitation are reviewed with special applications of repair and rehabilitation of bridges.

2) Deterioration of individual elements of a bridge, their causes, and possible rehabilitation and repair procedures that are most appropriate based on actual practices are discussed briefly.

3) The phenomenon of deterioration of concrete bridge decks is characterized along with the causes and the possible rehabilitation/repair techniques that have been successfully used, including the use of various repair and protective systems like concrete overlays, waterproofing membranes, sealants and impregnants. Currently popular techniques of redecking and widening, deck replacement, use of prefabricated components and hydrodemolition are also described.

4) The various joint systems are reviewed according to the type, design and materials used. Maintenance problems associated with each type are also discussed along with the necessary repairs. Some special techniques for integral conversion of simple span bridges to continuous bridges which have become very popular in recent years have also been reviewed.

5) Deterioration of the commonly used prestressed concrete girder systems is reviewed briefly. The present methods of assessing the damage in girders are identified along with a review of the various repair procedures for deteriorated concrete and prestressing steel which include present repair practices like epoxy injection, preloading, use of precast concrete panels, external prestressing and the use of shock transmission units. Typical replacement and repair techniques used currently are also described.

6) Bearings form an integral part of bridges. The bearings commonly used on concrete bridges are discussed briefly along with the associated problems and the current repair and replacement procedures Certain special retrofit procedures to withstand earthquakes are also described

7) Problems associated with the substructure components of a concrete bridge, especially those below the water surface are reviewed along with the causes of concrete and steel deterioration and the related structural damage caused and associated repair techniques. Damage due to scour and impact from vessels is also summarized along with the associated repair techniques

8) The essential requirements of a bridge management system, rating of bridges and the techniques used for financial evaluation are reviewed. Two established bridge management systems are discussed as examples for completeness.

9) Two case studies are presented to highlight the currently popular rehabilitation techniques used. The first case study involves a four lane prestressed concrete girder bridge which suffered loss of strength due to deterioration of its prestressing cables. The technique of external prestressing was used for rehabilitation. The second case study reviews the role of cathodic protection for rehabilitation of concrete decks.

10) Rehabilitation of the Champlain Bridge in Montreal is presented along with management considerations and an overview of the various repairs performed to the distressed structural components of the bridge, and the repairs planned for the future.

11) The essentials of an efficient bridge management program are reviewed with the following principal features.

a) A proper system of regular inspection of different bridges in the jurisdiction of an organization should be defined with the levels and intervals at which the inspections should be conducted in the prescribed manner. If any distress is observed as a result of a routine inspection, it should be followed by a detailed inspection. Also, while carrying out detailed inspections at longer intervals (two to five years), special investigations should be performed, if required.

b) All of the data collected from the inspections should be systematically classified and recorded for the purpose of evaluation of the structural and functional adequacy of each bridge to enable prioritization of follow-up actions which may include major repairs, strengthening, upgrading or replacement, with due consideration for safety of the users and economy Computer based evaluation techniques should be given preference.

c) Although a bridge is expected to have a long service life (over 100 years), some of its elements require frequent repair and replacement, such as expansion joints, bearings, waterproofing, parapets, wearing course, drainage, etc., accounting for almost 20 percent of the initial outlay on its construction. It is always economical to improve the maintenance techniques to limit the expenditure on major repairs or replacement. The rehabilitation and strengthening must be preceded by condition evaluation and proper assessment of the repair needs. Attention should be focused on the preservation and management of the large stock of bridges built over the past few decades and to rehabilitate and strengthen those bridges which have suffered due to inadequate maintenance.

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APPENDIX A
SEISMIC REHABILITATION TECHNIQUES

Many of the existing bridges are inadequately designed to resist earthquakes. This problem is important especially if the bridge is located in seismically active zones. Failures in the past due to earthquakes have indicated three major problem areas (55).

1) Susceptibility to damage of existing steel roller and rocker type bearings.
2) Inadequate strength and ductility of columns and piers.
3) Inadequate support lengths for girders.

Retrofit measures for the above problems have been discussed in Reference (55), some of which are included here for completeness.

a) Replacement of Vulnerable Bearings

Bearings which are susceptible to damage (see Figure A1.1) and which may cause collapse or loss of function of the superstructure should be replaced using elastomeric bearings or using the technique of base isolation discussed later.

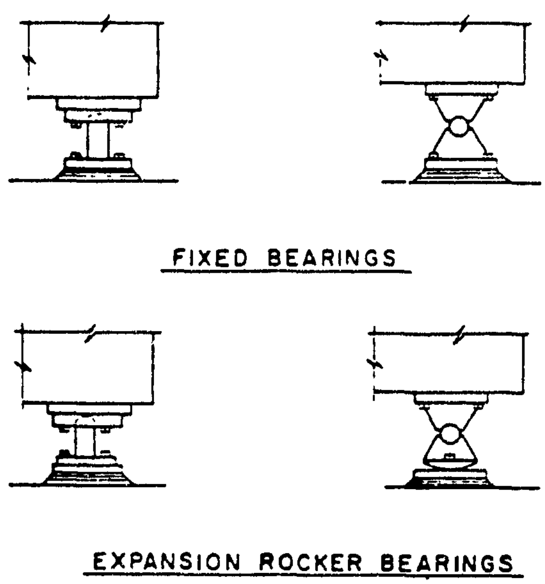


Figure A1.1 Bearings Vulnerable to Earthquakes (54)

b) Retrofit Measures for Columns and Piers

It has been observed that in the event of an earthquake, the reinforced concrete columns or piers together with the footings form a group of interacting elements which may fail if they have inadequate strength and ductility. A number of retrofit procedures have been presented under Reference (55) which include the use of force limiting devices, increased transverse reinforcement for confinement of columns, reduced or increased longitudinal reinforcement, use of infill shear walls between column bents and strengthening of footings.

c) Retrofit Measures for Inadequate Girder Support Lengths

A common retrofit measure (6) that has been used in the United States is the provision of positive cable connections between the superstructure and the supporting substructure Other measures used have been the provision of longitudinal joint and vertical motion restrainers Some recommended concepts have been the use of a bearing seat extension and specially developed bearings and devices that can change the seismic response of the bridge (55)

Base Isolation

Base isolation is a technique of isolating the effects of the superstructure from the substructure. It involves inclusion of base isolators and energy dissipators which can significantly enhance the seismic performance of the bridge while simultaneously economising the repair. A typical base isolation system consists of two basic elements.

1) A flexible mounting to lengthen the period of vibration of the total system sufficiently enough to minimize the acceleration response.

2) A damper to control the relative deflection across the flexible element.

Since elastomeric bearings are flexible, their thicknesses could be increased by adding additional layers to increase their flexibility and the resulting period of vibration However, this may cause large displacements in the substructure. These can be controlled by the addition of a damper which can not only provide hysteretic damping but can also provide additional elastic stiffness to the system (6).

A commonly used energy dissipator is a lead rubber elastomeric bearing shown in Figure A1.2. As shown, it consists of alternate layers of rubber which are vulcanized or cemented to thin steel shims. A lead plug is inserted into a preformed hole at the centre which is confined by internal steel shims and the externally applied vertical load. The weight of the superstructure is taken by the rubber in the bearing while the lead plug provides the necessary energy dissipation by way of plastic deformation.

The outer steel shims with dowel holes are also provided as shown in the figure to transfer the lateral force from the structure to the bearing The vertical stiffness of the bearing (K_v) and thus the vertical load capacity is inversely proportional to the thickness of the individual rubber layers and, therefore, these bearings have multiple thin rubber layers instead of a single rubber layer. Also, the shear stiffness of the bearing is inversely proportional to the overall thickness of the rubber and increases with increased bonded surface area.

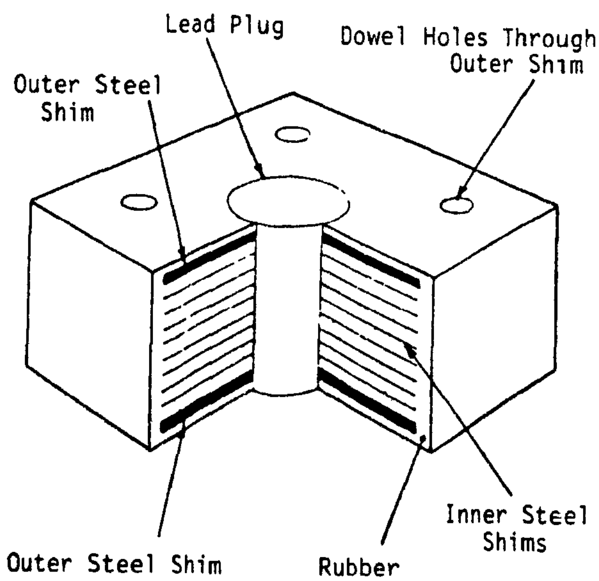
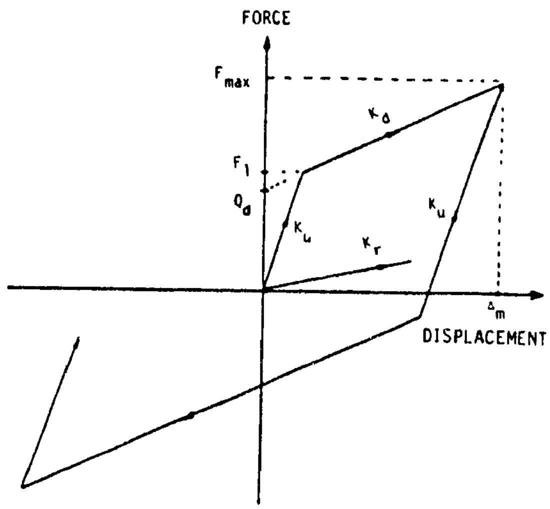


Figure A1.2 Lead Rubber Elastomeric Bearing (54)

These bearings are designed to resist low levels of shear, such as wind and braking loads, elastically with a high initial stiffness until a yield level determined by the characteristic strength, Q_d, is reached. The strength is a function of the diameter of the lead plug(s). Also, the post yield stiffness is kept at a minimum to ensure good energy dissipation and low overall structural stiffness during more severe seismic loading. A bilinear hysteresis curve formed by the two stiffnesses is shown in Figure A1.3. As seen in the figure, the curve has a stable form and a large enclosed area highlighting the energy absorption properties.

It is necessary to design these bearings taking into account the vertical dead and live loads, non-seismic lateral loads (due to temperature changes, creep, wind and braking forces) and seismic loads. Various design procedures using an approximate single degree of freedom concept are given under Reference 54 However, this should be used only as a preliminary estimate prior to a more detailed analysis.



Q_d = Characteristic strength (kips)

F_1 = Yield Force (kips)

F_{max} = Maximum Force (kips)

K_d = Post-elastic stiffness (kip/inch)

K_u = Elastic (unloading) stiffness (kip/inch) ($\approx 6.5K_d$)

K_r = Elastic stiffness of rubber acting alone (kip/inch)

Δ_m = Maximum bearing displacement

Figure A1.3 Hysteresis Loop for Lead Rubber Bearing (54)

APPENDIX B
INVENTORY INFORMATION

A bridge inventory report should contain the following items which comprise the information required for the inventory (52).

1) Bridge number.
2) Date of investigation.
3) Bridge name
4) Location.
5) Description.
6) Skew angle.
7) Number of spans.
8) Total length.
9) Roadway length.
10) Type of roadway surfacing.
11) Sidewalks.
12) Railing.
13) Alignment.
14) Number of traffic lanes.
15) Design live load.
16) Waterways crossed.
17) Other features crossed.
18) Clearances.
19) Date built.
20) Plans.
21) Plans and dimensions.
22) Bridge Inspection report.
23) Restrictions to bridge use.
24) Miscellaneous-Any other relevant information.
25) Stress analysis.
26) Paint record.
27) Signature/Professional stamp-of person making report.
28) Channel profile.

29) Encroachments

30) Environmental conditions.

31) Regulatory agency.

32) Average daily traffic.

APPENDIX C
BRIDGE INSPECTION DETAILS

The general items that need to be inspected for a bridge are as follows.

1) Approaches.
2) Waterway opening adequacy.
3) Piers and abutments.
4) Bents.
5) Stringers.
6) Steel girders.
7) Concrete girders.
8) Expansion joints.
9) Bearings.
10) Deck.
11) Curbs.
12) Sidewalks
13) Bridge railings.
14) Barrier railings.
15) Steel trusses.
16) Timber trusses.
17) Movable trusses.
18) Suspension span.
19) Signs.
20) Encroachments.
21) Aesthetics.
22) General - observations not covered in other sections.

APPENDIX D
APPLICATIONS OF STEEL ORTHOTROPIC DECKS IN REHABILITATION

General

Steel orthotropic decks are becoming increasingly popular for redecking existing deteriorated decks, because they offer many advantages over concrete decks, the most important ones being their suitability to overloads and immunity to weather and chemicals. There have been several installations all over the United States some of which include the Golden Gate Bridge in San Francisco, the Benjamin Franklin Bridge in Philadelphia, and the George Washington Bridge and Throgs Neck Bridge in New York City. This alternative was also preferred and implemented on the Champlain Bridge in Montreal, Canada, the details of which have been presented in Chapter 9. All of these installations have highlighted the excellent adaptability of orthotropic decks. A detailed discussion of the advantages offered by orthotropic decks follows.

Orthotropic Deck

An Orthotropic deck generally consists of a flat, thin steel plate, stiffened by a series of closely spaced longitudinal ribs which are set orthogonally to the floor beams of the bridge. The steel plate is protected against corrosion by a protective coating which is often an epoxy resin, and covered by an asphalt or bitumen wearing surface upto 60 mm thickness. The thickness of the rib plate may vary from 10 mm to 25 mm, depending upon the rib spacing, load requirements and the allowable deflections.

Generally, the rigidity of the floor beams and the ribs is of unequal magnitude. This results in the elastic behaviour of the slab to be different in both of these principal directions. This phenomenon is known as structural anisotropy and the bridge deck system is called as orthogonal-anisotropic or in short orthotropic.

Orthotropic decks may consist of either torsionally soft, open ribs, or torsionally stiff, box-shaped closed ribs. Open ribs are made from flat bars, bulb shapes, inverted T-sections, angles and channels. The torsionally stiff box ribs may be either trapezoidal, semicircular, triangular or combined. Closed ribs are economical since they allow a wider spacing of the floor beams which not only reduces the dead weight of the deck but also reduces the amount of welding needed.

However, closed ribs are more difficult to fabricate than open ribs. Also, they require more complicated field splices and precision fitting at the splices.

Structural Advantages of Orthotropic Decks (66,67)

The main advantages offered by orthotropic decks are as follows:

1) Reduction in Dead Weight

Replacing concrete decks with orthotropic decks can reduce the dead weight of the roadway by upto 50 percent which is quite significant from the point of view of economy. The actual reduction may vary anywhere below this value depending on the design of the orthotropic deck, connections and splice details as well as the thickness of the road surfacing. Due to reduction in the dead load, there is a direct increase in the live load carrying capacity of the underlying floor beams and the main bridge members. This factor gains importance in view of the ever increasing truck weights, and correspondingly higher live load requirements.

Reducing the dead weight also is important in cases where bridge rehabilitation involves widening the roadway to add new traffic lanes or shoulders. Due to addition of the new orthotropic deck, the additional dead loads would in fact be smaller than those if concrete decks were used.

2) Increase in Deck Overload Capacity

Orthotropic decks have a very high local overload capacity. The actual ultimate loads are several times greater than those calculated by the first order flexural theory. Due to their second order elastic behaviour under increased loading, these decks can easily carry occasional heavy wheel and axle loads without suffering any structural damage or permanent deformation.

3) Strengthening of Main Bridge Members

The floor beams and the main longitudinal bridge members act compositely with the orthotropic deck in which the later acts as an added top flange thereby not only stiffening the bridge members but also increasing the structural redundancy of the integrated bridge superstructure.

4) Ease of Installation

Orthotropic steel decks are very easy to install unlike the concrete decks with a minimum restriction to traffic during the installation. Furthermore, the strength of the orthotropic deck is not impaired by installation under traffic On the other hand microcracking usually occurs in a newly cast-in-place concrete deck when traffic is permitted across the bridge in adjacent traffic lanes Vibrations from vehicular traffic can also result in segregation of the aggregates before the concrete has a chance to set. The long term effect of microcracking can also result in maintenance problems as a result of premature deterioration of the deck since infiltration of water takes place through a network of cracks.

5) Maintenance-Free Operation

Although the orthotropic decks require regular maintenance painting, since the deck undersides are well sheltered, the need to repaint is much reduced. In North America, orthotropic decks have been used for over 25 years and have required no maintenance to the deck or to the wearing surface. On bridges where traffic volumes are large and deicing salts are heavily used, maintenance needs have been limited to repairs to rutting in the wearing surface. A lifespan of about 100 years for the deck has been estimated if it is well maintained On the other hand, concrete decks need regular maintenance inspection and patching of the structural element of the bridge roadway and also they may have to be replaced after 30-40 years of service

Orthotropic decks also offer the advantage of continuous construction, which, on long bridge spans, permits eliminating deck joints which always present serious maintenance problems.

6) Durability

Orthotropic decks are immune to damage by deicing salts and weather effects. The durability of surfacings on steel decks with sufficient deck plate rigidity is also quite satisfactory.

7) Economy in Construction

The first construction cost of redecking a bridge in concrete is generally less than that of redecking in steel. However according to Wolchuk (66), a meaningful economic comparison of alternate engineering solutions should be based on annualized life cycle cost basis, which should include in addition to the first cost, the projected yearly cost of maintenance and repairs, as well as the annualized cost of future replacement (including economic effects of traffic disruption due to replacement) at the end of the useful life of the alternative under consideration. On the basis of this comparison, it has been shown that the orthotropic deck solution can give almost 50 percent savings compared with the concrete deck solution.

Design for Redecking

Although the detailed design procedure for orthotropic decks is beyond the scope of this thesis, the following important observations as observed by Wolchuk (66) are found notable to mention:

1) If truss and girder bridges with floor systems of longitudinal stringers supported by transverse floor beams are to be redecked, maximum efficiency and dead-weight saving is obtained if the existing stringers are removed and the new decks are supported on the floor beams. This necessitates the deck rib lengths between supports well exceeding the usual rib spans of existing orthotropic deck bridges (deck spans exceeding 7.5 m are feasible and practical). However, from the design point of view, it is necessary to consider certain characteristics of long-span decks in lieu of the fact that the standard design of orthotropic decks are based on much shorter spans.

2) In the design of open-rib decks, it is economical to assume the long-span ribs, deflecting under the wheel loads, to act as elastic supports of the continuous deck plate as against the earlier assumption of considering them to be rigid. This results in a wider transverse wheel load distribution leading to a reduction in the loads and bending moments acting on the individual ribs.
In the case of closed-rib decks, the capacity for lateral load distribution of the wheel loads is much better than in the open-rib deck systems. As the deck span increases, the lateral load distribution also increases which may cause the effects of two adjacent wheels on the deck to overlap, thereby increasing the loading on an individual rib which should be considered in the design.

3) In the case of long span decks, the live load deflections may be very large and should be limited to avoid undesirable deck springiness. It is, therefore, quite possible that the deflections instead of the strength considerations may govern the design.

4) Although the choice between open and closed rib decks depends on many factors, the latter are usually preferred mainly due to the fact that they are more structurally efficient and require less steel than open ribs. Also, the underside of the deck has less surface area to be painted and is smooth and easily accessible. Furthermore, the total length of rib-to-deck welds of the trapezoidal ribs is one-half of that of the open ribs. However,

closed ribs need to be permanently air-tight on the rib interiors, requiring seals at the rib ends and at field splices. Also, the webs of trapezoidal ribs are subject to flexure under the wheel loads.

5) It is very important to design the connection details between the new deck and the existing steelwork for fatigue under the effects of alternating stresses in the deck at the supports. The supporting diaphragms which are subjected to local bending and twisting must be carefully designed and detailed.

6) In cases where multiple girder or stringer-type bridges are to be redecked (where the original concrete deck was placed on the girder or stringer flanges), the new orthotropic deck must be supported on suitably spaced transverse members between the girders Connection details between the new deck and the longitudinal members must be properly designed to assure structural integration of the deck with these members Live load shears and temperature effects should also be considered in this design. In all cases where the new deck acts compositely with the existing structure, proper allowances for the additional axial tensile or compressive stresses must be made in the deck design.