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Evaluation of Conventional Repair Techniques for Concrete Bridges

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Principal Investigator: M. Arockiasamy, Ph.D., P.E.
Professor and Director
Center for Infrastructure and Constructed Facilities
Department of Ocean Engineering
Florida Atlantic University
777 Glades Road
Boca Raton, FL 33431-0991

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1 GENERAL

Bridge structures, like any other structure, deteriorate with time due to the inadequacy of design detailing, construction and quality of maintenance, overloading, chemical attacks, atmospheric effects, abnormal floods and erosion. Maintenance needs to be done to preserve the intended load carrying capacity of the bridge and safety of the public using it. Rehabilitation refers to restoring the bridge to the service level it originally was intended to have.

The U. S. Bureau of Reclamation (Bureau) identifies the following seven steps for standard concrete repairs of both construction defects in new concrete as well as old concrete damaged by long exposure to field conditions: i) determine the cause of damage, ii) evaluate the extent of damage, iii) determine the need to repair, iv) choose an appropriate repair system, v) prepare the old concrete, vi) apply the repair system, and vii) cure the repair properly.

i) Cause of damage: It is essential to correctly determine the causes(s) of the damage to the concrete. If this is not done, or if the determination is incorrect, the same cause will most likely attack and deteriorate the repair. The money spent for such repairs is, thus, totally lost and larger replacement repairs become necessary at much higher cost.

ii) Extent of damage: The objective of this step is to determine how much of the structure is damaged and how extensive the damage is.

iii) Need to repair: Not all damage to concrete requires repair. Repairs should be undertaken only if they will result in longer or more economical service life, a safer structure, or necessary cosmetic improvements in the structure. This step also includes determination of when the structure can be taken out of service for repairs, an estimate of how long the repairs will take, and how to budget the costs of the repairs.

The Bureau states that these first three steps are the major components of a condition survey. Only after they have properly been performed should one proceed with selecting and installing the repair materials.

iv) Choice of an appropriate repair system: Upon completion of the first three steps, an appropriate repair system can be selected that takes into consideration the many factors essential to a successful repair.

v) Preparation of the old concrete: The most common cause of repair failure is improper or inadequate preparation of the old concrete prior to application of the repair material. Even the best of repair materials would give poor service life if bonded to weakened or deteriorated old concrete. It should be noted that each repair material has special preparation requirements.

vi) Application of the repair system: Each standard and non-standard repair material has application procedures specific for that material. For example, the procedures used with replacement concrete are vastly different from that necessary for polymer concrete or epoxy bonded concrete. It is essential that proper application techniques be identified.

vii) Curing of the repair: The second most common cause of repair failures is improper or inadequate curing. Each repair material has specific curing requirements. As an example, replacement concrete benefits from long periods of water curing while latex modified concrete requires 24 hours water curing followed by drying to allow formation of the latex film.

2 CONCRETE REMOVAL AND PREPARATION

All damaged, deteriorated, loosened, or unbonded portions of existing concrete should be removed by water blasting, bush hammering, or jack hammering. The perimeters of repairs to concrete that involve concrete removal and subsequent materials replacement should be saw cut perpendicular to the repair surface to a minimum depth of 1 inch. Any microfractured surfaces resulting from the initial removal process should be eliminated by shotblasting, wet sandblasting, or water blasting. The surfaces should be cleaned and allowed to dry thoroughly (unless the specific repair technique requires application of materials to a saturated surface). The use of acids for cleaning or preparing concrete surfaces for repair should not be permitted.

All loose scale, rust, corrosion byproducts, or concrete should be removed from exposed reinforcing steel to completely expose reinforcing steel for more than one-third of its perimeter circumference to provide 1-inch minimum clearance between the steel and the concrete. Damaged or deteriorated reinforcing steel should be removed and replaced.

After the concrete has been prepared and cleaned, it should be kept in a clean, dry condition until the repair has been completed. Any contamination, such as oil, solvent, dirt accumulation, or foreign material should be removed by additional cleaning as stated above.

3 PATCHING

3.1 Patching Materials

Depending on the size, location, and the general function of bridge components, various materials are available for repair. The following influences selection of materials: a) compatibility of the material to the original concrete, b) environmental considerations, including aesthetics, c) cost effectiveness, d) expected service life, e) availability, and f) familiarity of the contractors with the product.

In repairing a concrete spall, the following requirements should be satisfied:

i) Properties of the repair material should be as close as possible to the existing concrete, particularly with respect to the coefficient of expansion and the modulus of elasticity.

ii) Strength should be at least as high as that of the original concrete.

iii) Repair material should have low shrinkage, low permeability, and a low water /cement ratio to prevent moisture and chloride penetration.

iv) Repair material should adhere to the concrete substrate, either by applying a rich cement mix or epoxy bonding compound to the prepared concrete surface before placing the new concrete.

v) Color and texture should match the original concrete as much as possible. Selecting materials that meet all necessary properties established by conditions and requirements is difficult. Most materials used for deep repairs use portland cement binders and well proportioned aggregates. Durability for these materials can be increased using special pozzolans (microsilica), polymers Oatex), or admixtures that reduce permeability. Most modified concretes and mortars can be easily used once one has experience in how these materials behave during placement and while curing.

The use of portland cement based repair materials requires special attention to shrinkage and curing. All repair materials used should have low shrinkage properties, and proper curing is critical in reducing early shrinkage and for future long-term performance.

In structural applications, it is important to understand the repair materials' response to loads. Two important properties for load sharing applications are elastic modulus and compressive creep. In understanding the materials properties, it is important to understand the exposure and service conditions to which the selected materials will be subjected. For instance, it has been demonstrated that the addition of latex to modify cement based repair materials causes the flexure creep value to soar under high humidity conditions, but most of the reported material properties are evaluated under at low relative humidity and therefore may appear acceptable.

The use of experimental materials or materials that contain unknown ingredients that could lead to unnecessary problems should be avoided. For instance; the use of materials containing gypsum results in uncontrolled expansion and extremely low durability when subject to moisture. Also, the high heat of hydration of high exothermal materials such as magnesium phosphate based materials can cause thermal cooling stresses.

Some materials are sensitive to the method of application. Latex modifiers have proven exceptional in bridge deck overlays, but, when used in some applications involving dry mix shotcrete, have resulted in interbond failure. Latex films forming on unfinished surfaces caused the failure.

Polymer concretes and mortars are the other major class of materials used to repair concrete surfaces. Epoxies and acrylics blended with graded aggregates produce strong and chemically resistant materials. They can be used for thin application or thick applications where the service exposure conditions do not cause dimensional incompatibility problems. Polymer materials have high thermal coefficients as compared to concrete. Except for thin surface coating systems, they should not be used in solar exposure situations.

The following are the most common repair materials: latex modified concrete and mortar, epoxy patching compounds, polyester resin, acrylic concrete and mortar, polymer-modified cement based materials, pozzolanic modified concrete, high alumina cement compounds, magnesium phosphates, molten sulfur, calcium sulfate based materials, non-shrink quick setting mortar cement based polymer concrete, and pneumatically applied mortar (shotcrete).

3.2 Patching Case Study

Griffen Road Over 175, Florida.

This bridge is located in Broward County, Florida, and carries east and west bound S.R. 818 (Griffen Road) traffic over S.R. 93 (I-75). This structure has five spans for a total length of 404 feet, eight lanes for a curb to curb width of 130 feet; and was designed for HS20-44 loading. The superstructure is constructed of precast prestressed concrete I beams with a cast-in-place reinforced concrete deck. The bridge was constructed in 1984.

The repair under consideration for this case study is a pressure grout patching repair. The deck of this bridge was constructed using prestressed concrete stay in place forms which were placed on the beams with a thin felt pad to provide an even distribution of the load. The deck was then poured over the stay in place forms. The felt pads were not removed, and deteriorated over time. This caused the stay in place forms to rotate and the deck to crack. To remedy this situation, the gap left by the deterioration of the felt pads was cleaned thoroughly and the remaining cavity pressure grouted as shown in Figure 1. This type of repair was done on several bridges in this area that used the same deck construction technique.

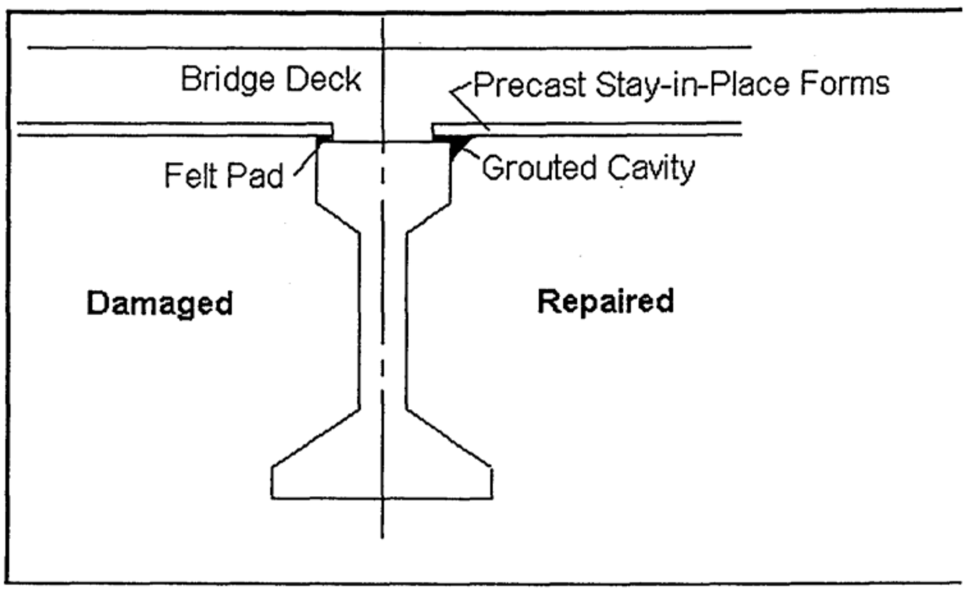

Figure 1 - Griffen Road Repair, Florida

This repair has been in place for 10 years and is working very well. A visual inspection of bridge performed on 4/7/98 found the repair in good condition. In fact, the existence of the repair is barely discernible from the original construction. Figures 2, and 3 show the repaired area under the deck.

Pressure grouting the cavity in this case was an effective way to remedy a poorly constructed bridge feature. District 4 of the Florida Department of Transportation performed this repair and stated that in order for this repair to be effective it is imperative that the area to be grouted be cleaned thoroughly. Also, all of the voids have to be filled during grouting process.

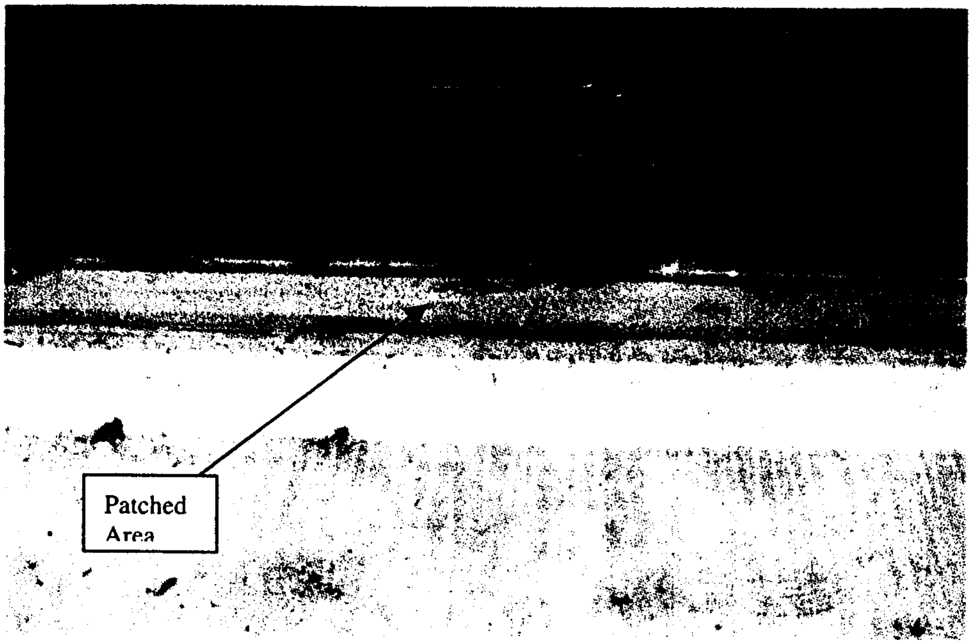

Figure 2 - Griffen Road Repair, Florida

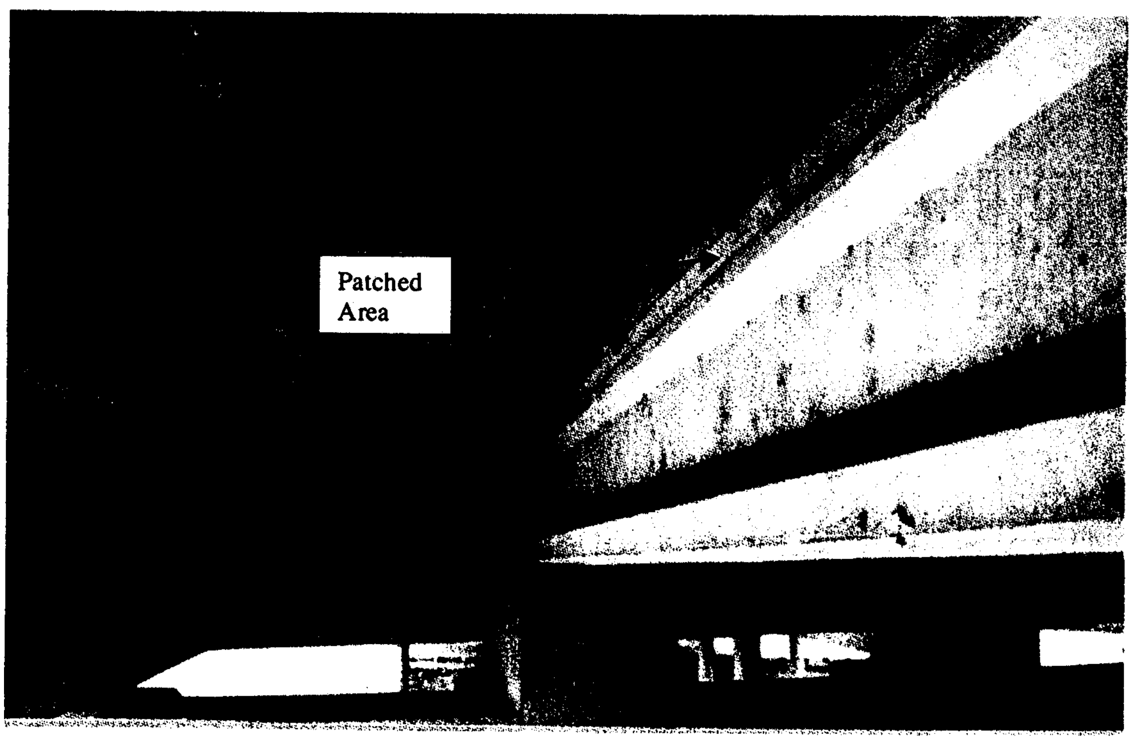

Figure 3 - Griffen Road Repair, Florida

3.3 Deck patching

Patching methods are used to replace localized areas of deteriorated concrete (spalls and delaminations). For decks, the depth of deterioration may include the top layer of reinforcing steel or both the top and bottom layers of reinforcing steel. If only the top reinforcing mat is corroding, a partial-depth repair would be used. For partial-depth deck repairs, the deteriorated concrete is removed to the depth required to provide a minimum of 0.75 in clearance below the top layer of reinforcing steel. Maximum depth of removal for a partial-depth repair should not exceed half the deck thickness. Corrosion of both the top and bottom layers of reinforcing steel requires full-depth repairs. For a full depth repair, the concrete within the delineated area for the entire deck thickness, normally 8 in, is removed. Once all the unsound concrete has been removed, the cavity should be blasted clean to remove all loose material and provide a dust-free surface. Partial-depth deck patching materials include portland cement concrete, quickset hydraulic mortar and concrete, and polymer mortar and concrete. Portland cement concretes are used for full-depth deck patches.

Deck patches have a relatively short service life because they do not address the cause of the problem, corrosion of the reinforcing steel, but address only the symptoms; spalling and delaminations.

When concrete contaminated with chloride beyond the threshold level is left in place in the area surrounding the patches, the patches themselves often accelerate the rate of deterioration of the surrounding concrete. The patch concrete area acts as a large noncorroding site (cathodic area) adjacent to corroding sites and increases the rate of corrosion.

4 DECK REPAIR

4.1 Deck Overlays

4.1.1 Description

Overlays are used to restore the deck-riding surface to as-built quality and to increase effective cover over the reinforcing steel. Overlays include latex-modified concrete (LMC), low-slump dense concrete (LSDC), and hot-mix asphalt concrete with a preformed membrane (HMAM). These overlays are considered repair methods because the chloride-contaminated concrete is left in place. The overlay has some influence on the service life of the repair, but the amount and degree of the chloride-contaminated concrete left in place remains the most important factor.

4.1.1.1 Advantage

The analysis of the cost shows that the latex-modified concrete overlay is more costly than the other overlay materials. However, this overlay increases the deck performance without reducing the load rating since the thickness required is less than other overlay materials. Moreover, when the delay, inconvenience, and safety costs of lane closures due to deck deterioration are factored in, overlays are not only warranted, but also mandated.

4.1.1.2 Disadvantages

LMC, LSDC, and HMAM overlays may increase the dead load and thus decrease the live load capacity of the bridge. The influence of the overlay on the live load capacity of the bridge must be evaluated before one of these overlay systems is specified.

LMC, LSDC, and HMAM overlays should not be placed on decks where the existing concrete may be susceptible to alkali aggregate reactions (silica or carbonate) unless low-alkali cement is used or other preventive measures have been taken.

4.1.2 Case studies

4.1.2.1 Passaic river bridge of the New Jersey turnpike

Passaic River Bridge was constructed in 1951. About 10 percent of the slabs had to be repaired after 8 years of the construction, and an average of about 1 percent more have been repaired every year by the different method. The most recent repaired method is a latex-modified concrete overlay. This method has performed very well since 1991.

4.1.2.2 Rip Van Winkle Bridge

The Rip Van Winkle Bridge is a 5,000 foot combination thru- and deck-truss toll structure over the Hudson River thirty miles south of Albany, New York. The New York State Bridge Authority built the bridge in the early thirties. The bridge was designed to carry concrete trucks, which resulted in a very solid, and potentially long-lasting, structure. However, in the mid-1980's, the Rip was in danger of becoming unusable. A latex-modified concrete overlay was used to repair the deck of the bridge. Even with the overlay, the condition of the deck continued to deteriorate. In 1989 the Authority was asked to study a **possible deck widening and replacement project. The result of that wide ranging study was a recommendation to replace the deck with a new cast-in-place concrete** deck, and widen the deck so that the new roadway would extend the full width available in the thru-truss area.

4.1.2.3 Overboda bridge over Dalalven, Sweden

The Overboda bridge is a part of national road 76 and has average daily traffic (ADT) of 7,600 vehicles. Ten percent of the ADT is heavy vehicles, mainly loaded timber trucks, at high speed. The bridge is 12 m wide and has two lanes. Completed in 1942, it is a zero hinged concrete arched bridge. The concrete slab is reinforced in two directions and had a thickness of 170 to 230 mm in 1942, including an original bonded concrete overlay. The bridge has four main spans and two approach spans. In 1984, after 42 years in service, the end beams were suffering from scaling due to freeze-thaw action and weathering. The old concrete did not have an adequate air-void system. The increased use of deicing agents accelerated the deterioration. The top of the bonded concrete overlay (from 1942) was delaminated but the concrete at the reinforcement was sound. The concrete bridge deck was water jetted to remove damaged concrete and a bonded overlay of steel fiber reinforced concrete (SFRC) replaced the removed concrete and the asphalt wear course. Observation from the water jetting indicated that the concrete had good strength. The water jetted surface was carefully cleaned both after the water jetting and prior to overlay placement. In 1995, the overall impression from Overboda bridge was that the overlay had performed very well. The compressive strength has not degenerated. No contamination was found at the interface in any cores. Bond testing indicated a good bond between the overlay and the existing concrete.The failure took place mainly in the cement paste below the large aggregates and reinforcing steel, and in the aggregate in the old concrete. Moreover, the freeze-thaw resistance of the overlay was tested, and showed that a steel fiber reinforced concrete (SFRC) had very good freeze-thaw resistance after 56 and 112 cycles. The repaired deck is shown in Figures 4 and 5.

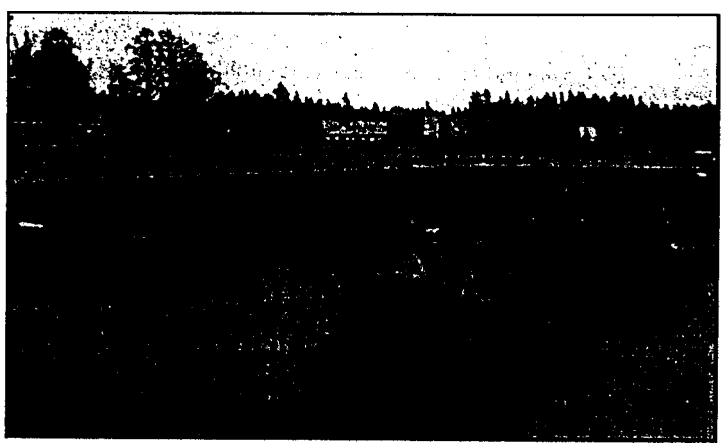

Figure 4 - Overboda Bridge at the repair in 1986

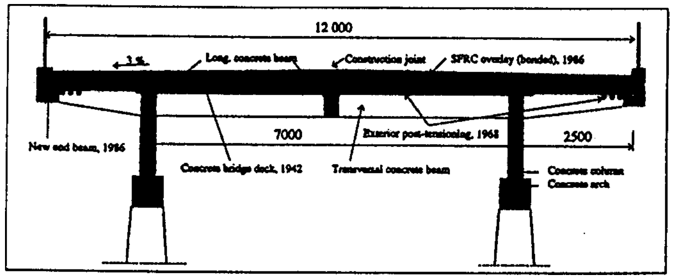

Figure 5 - Cross-section of Overboda Bridge after repair in 1986

4.2 Expansion Joint Repair

There are ten types of deck joints that are often used in the expansion joint. A typical deck joint rehabilitation (or replacement) procedure starts with the removal of the deteriorated or damaged joint, and the debris and dirt from the deck joint opening. Then the deck substrate is repaired to ensure a firm support and anchorage of the joint system. The repair ends with the replacement of the joint components. More detailed descriptions of the rehabilitation procedures for several joint types are presented in the following:

4.2.1 Open joints

Deteriorated deck slab concrete along the deck joint opening is removed to expose sound concrete. The exposed reinforcement would then be cleaned and epoxy coated followed by reconstructing deck slab comers using quicksetting patch materials or high early strength concrete. Alternatively, deck slab is removed to half of its depth along the entire length of the joint. The exposed reinforcement is then cleaned and epoxy coated and the deck slab reconstructed by installing angles for protection of slab corners. As a minimum measure, debris is removed from joint opening and drainage trough. If a trough system does not exist, one is installed to prevent intrusion of water, debris, and deicing salts to the parts of the bridge below the deck level. (Figure 6)

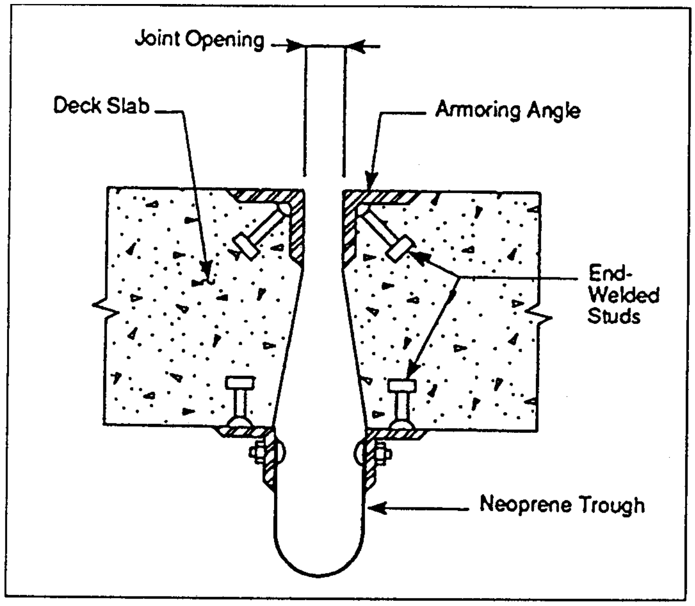

Figure 6 - Open joint.

4.2.1.1 Disadvantages

Although the existing open deck joints are paved to achieve watertight joints during deck overlay work, cracking and deterioration in the overlay can allow the joints to continue to leak and promote deterioration at the pier ends of girders and bearings. Open joints are prone to the intrusion of deicing salts and water, creating costly repairs on surrounding bridge components in the long run. Open joints are seldom used in new bridge structures, and are often worth replacing by other types during rehabilitation of existing structures.

4.2.2 Filled joints

Spalled deck slab corners are repaired along the length of the joint, following the procedure described for open joints. Additionally, if the existing sealant is separated, cracked, shriveled, or bulged, or contains embedded incompressible materials, the sealant is replaced by a new one.

There are two types of joint sealers, which are fieldformed sealer and preformed sealer (Figure 7 and 8). Fieldformed sealer is often used to seal small joint opening. Preformed sealer can seal a wider gap than fieldformed sealer. Preformed sealers are somewhat newer and hence have a shorter record of proven service than fieldformed sealers. An important advantage is quick installation time and less interruption to traffic. Most commonly used types of preformed joint sealers for deck joint rehabilitation projects are of extruded shapes made of elastomeric material.

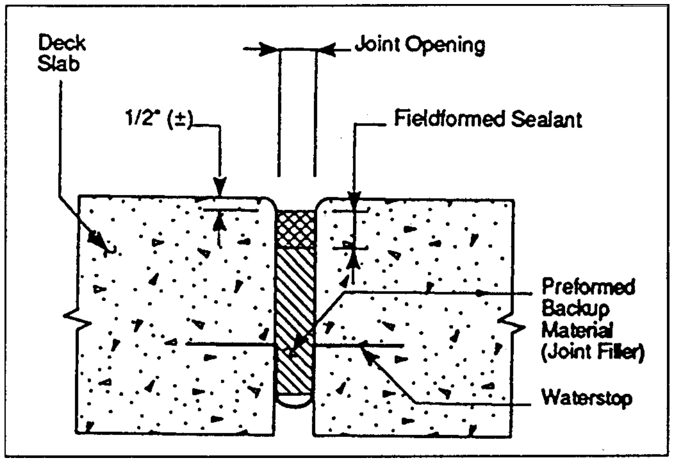

Figure 7 - Filled joint with fieldformed

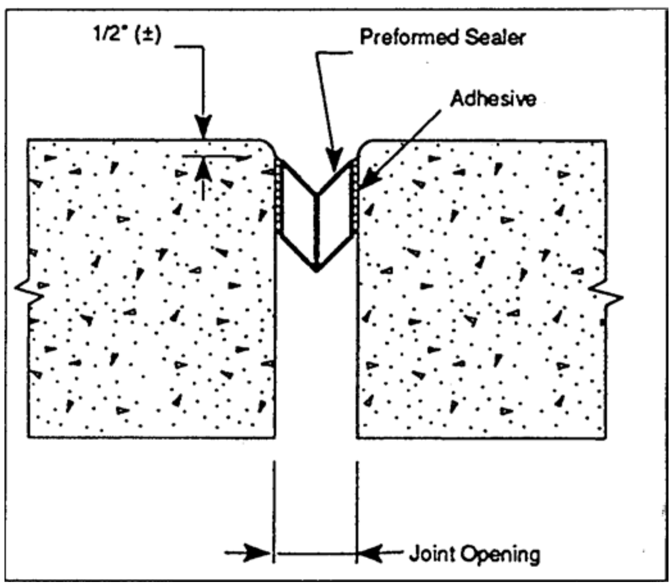

Figure 8 - Filled joint with preformed sealer

4.2.2.1 Advantage

During joint rehabilitation work, it is relatively easy with either type of sealer to change the direction of a filled joint or to extend the joint into a barrier or curb The best result is obtained when the sealer is extended into the curb at the level of the deck slab.

4.2.3 Sliding plate joints

Spalled deck slab corners are repaired along the length of the joint by following the procedure provided for open joints. The corroded, bent, jammed, or cracked plates and anchorages are removed and replaced. Also, debris accumulation from the joint and drainage trough could be removed. Where a trough does not exist or the existing one has deteriorated, a new trough (Figure 9) is installed. Sliding plates are fastened at one end and free to move at the other, providing a simple way to cover moderate-sized gaps.

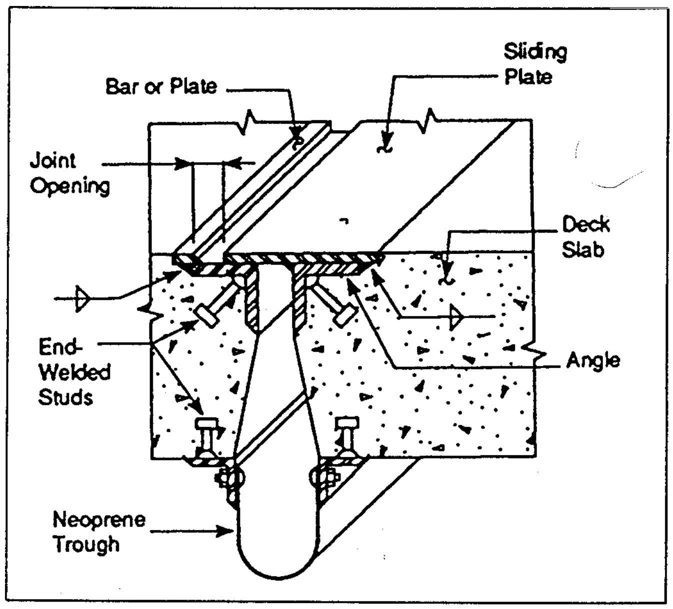

Figure 9 - Sliding plate joint.

4.2.3.1 Advantage

Sliding plate joints can accommodate up to 4 inches of total movement. Direction changes in the joint that may be required during deck joint rehabilitation can easily be achieved by welding the steel plates and shapes at such location.

4.2.4 Finger plate joints

The repair procedures are the same as for sliding plate joints (Figure 10). The relatively long projections of finger plates can accommodate total movements as large as several feet. Finger plate joints are permitting water to leak through and deteriorate surrounding bridge elements.

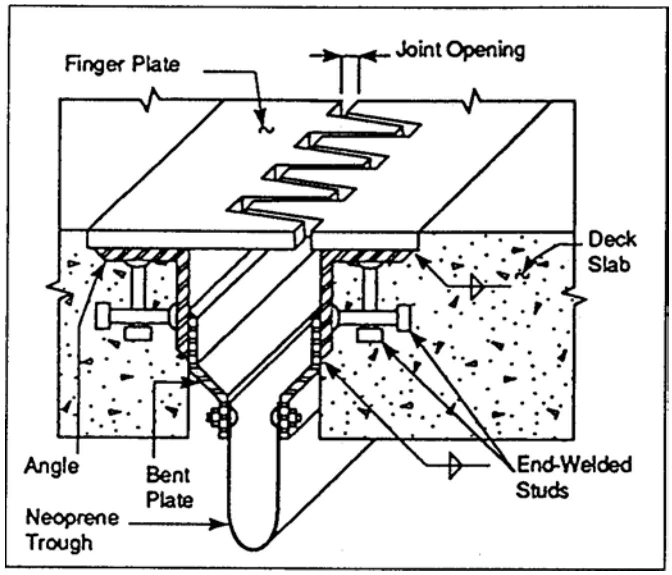

Figure 10 - Finger plate joint.

4.2.5 Sawtooth (serrated) plate joints

The repair procedures are the same as for sliding plate joints (Figure 11). Sawtooth plates are used for moderate joint movements of about 3 inches, and have shorter and broader projections than finger plates. The relatively long projections of sawtooth plates can accommodate total movements as large as several feet like finger plates, but sawtooth plates have shorter and broader projections than finger plates.

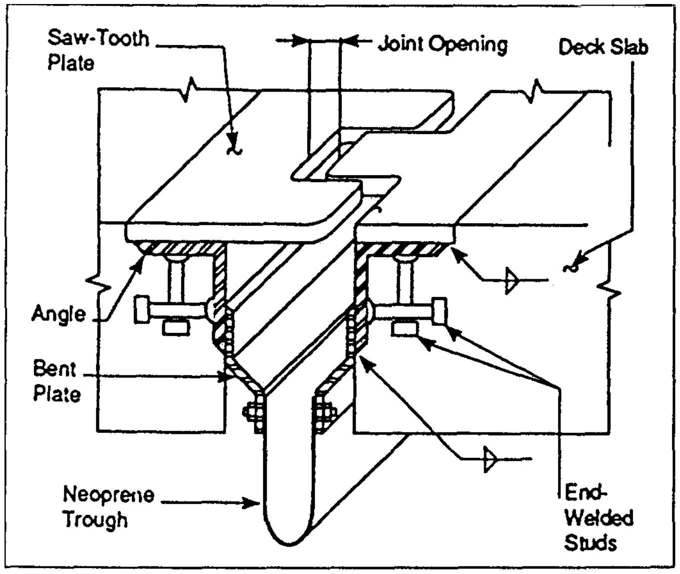

Figure 11 - Sawtooth plate joint.

4.2.6 Compression seal joints

Spalled deck slab edges are repaired following the procedure given for open joints. The corroded, broken, bent, and cracked portions of joint armoring and anchorages would be removed for replacement. Additionally the debris accumulation from the seal would also be removed. If the splice location in the existing seal is leaking, the particular region would be cleaned and sealed with adhesive (Figure 12). The compression seal joint is designed to remain always in compression; these seals have become quite popular, particularly with neoprene as the seal material.

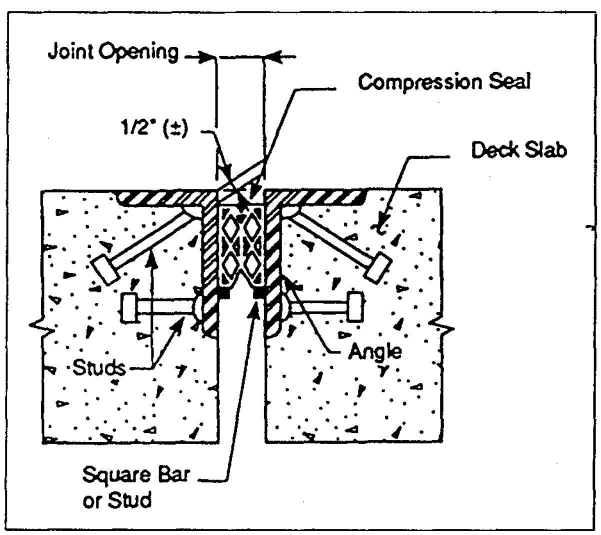

Figure 12 - Compression seal joint

4.2.6.1 Advantages and disadvantages

By using neoprene as the seal material for the compression seal joint, there are several advantages; such as, a large variety of choices in movement ranges, water tightness, relative ease of installation, and cost effectiveness. Compression seals that are manufactured from ozone-sensitive neoprene compositions have been known to lose their elasticity and harden after several years of service.

By using neoprene composition seal, the water-tightness of such a hardened joint will fail when bridge spans contract at low temperatures and cause the hardened seal to pull away from the sides of the joint until the high-solids urethane adhesive debonds.

4.2.7 Strip seal joints

The repair procedures are the same as these for compression seal joints (Figure13). Strip seals combine elastomeric material with metal supports for a locking seal, often favored at locations where differential movements are anticipated.

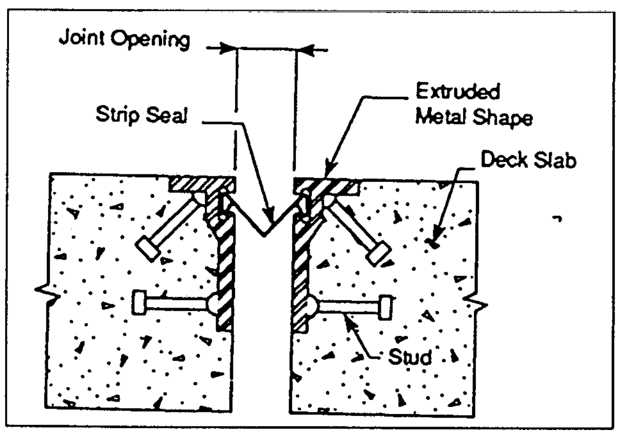

Figure 13 - Strip seal joint.

4.2.7.1 Advantage

Because of the locking nature of the seal, the strip seal joint performs better than a compression seal at locations in which transverse slab movements are **anticipated and provide a superior seal against water leakage.**

4.2,8 Sheet seal joints

Spalled deck slab corners along the length of the existing joint are repaired following the procedure described for open joints. Damaged hold-down bars and anchorages are removed for replacement and the debris accumulation cleaned from the seal (Figure 14). These elastomeric seals can accommodate changes and skews in joint configuration but depend on close contact and may loosen under repeated live loads.

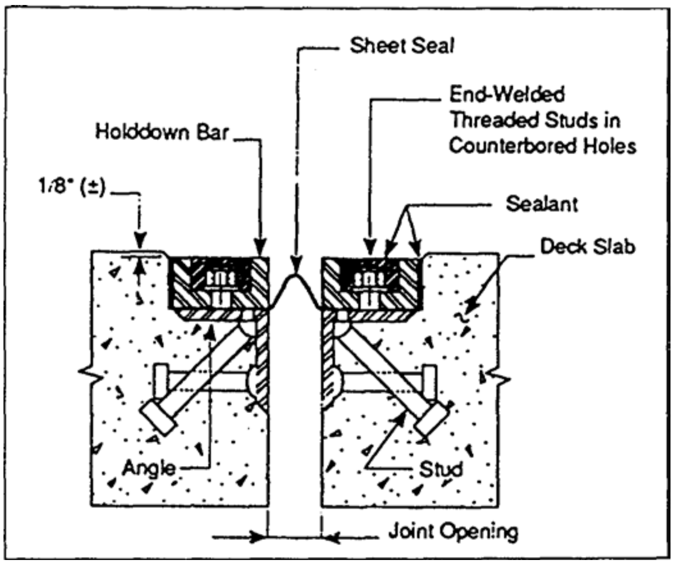

Figure 14 - Sheet seal joint.

4.2.8.1 Advantage and Disadvantage

One important advantage of the sheet seal over the compression seal is the ability to accommodate directional changes and skews in the joint configuration often without any need for a splice in the seal. Sheet seals do not allow water to leak through and deteriorate the surrounding bridge element. Failure of anchorage systems, under repetitive live-load impact, has been a frequently encountered problem with the sheet seals.

4.2.9 Plank seal joints

The repair procedures are the same as those for the sheet seal joints (Figurel5). These molded neoprene sections can accommodate joint movements to 13 inches, but seal anchorages may loosen under prolonged service.

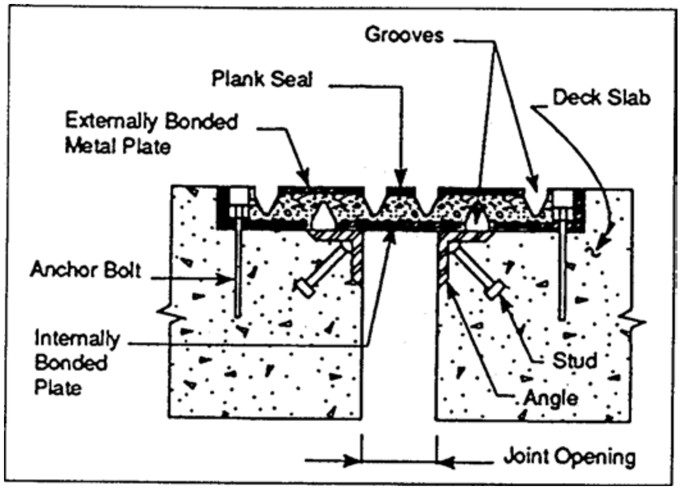

Figure 15 - Plank seal joint.

4.2.9.1 Disadvantage

Leakage at joints between segments, loose anchorages, excessive noise, and snowplow damage have been the problems commonly reported with the existing plank seal joints.

4.2.10 Modular joints

Spalled deck slab corners along the length of the existing joint are repaired following the procedure described for open joints. Damaged seals and separator beams and other accessible hardware are removed for replacement. The debris accumulation from the seals is removed (Figure 16 and 17). Modular joints provide state-of-the-art joints for joint movements up to 4 feet. This type **of joint features an individual support bar at each separator bar. Various sealant materials can be combined with** beams **and support bars.**

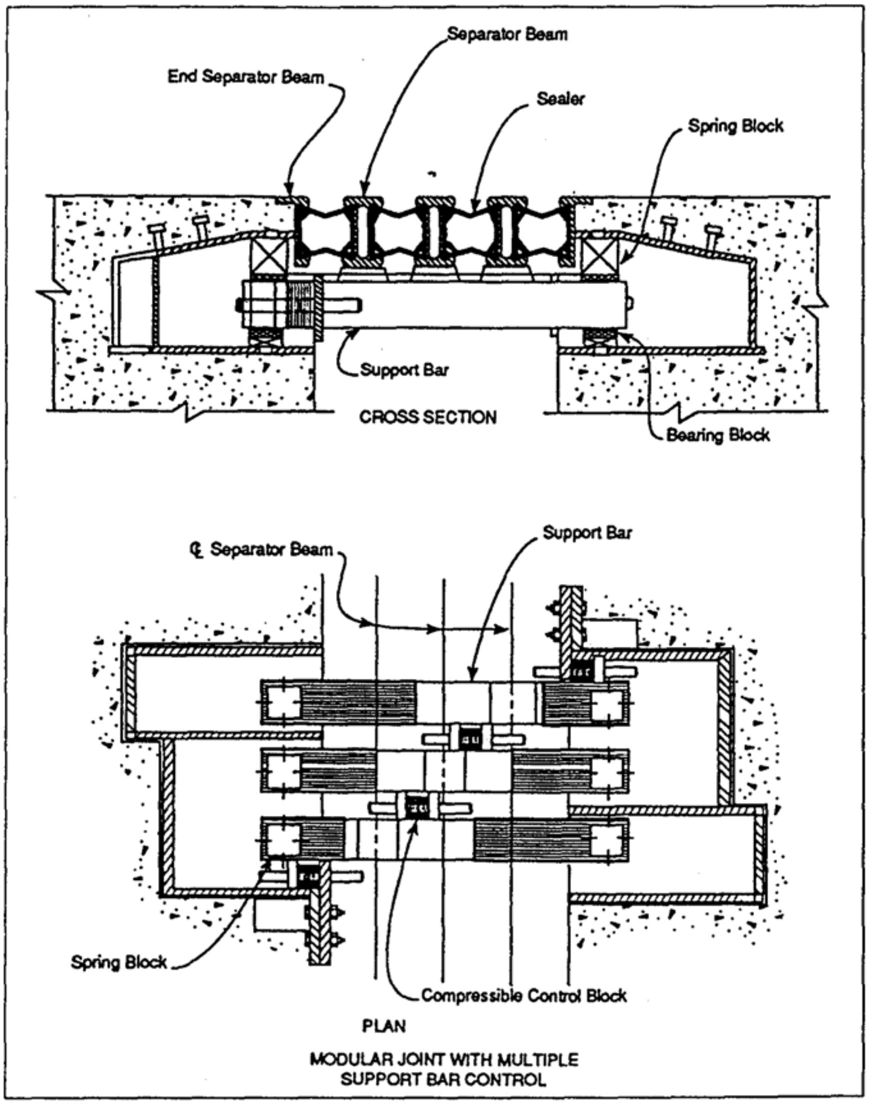

Figure 16 Modular joint with multiple support bar control.

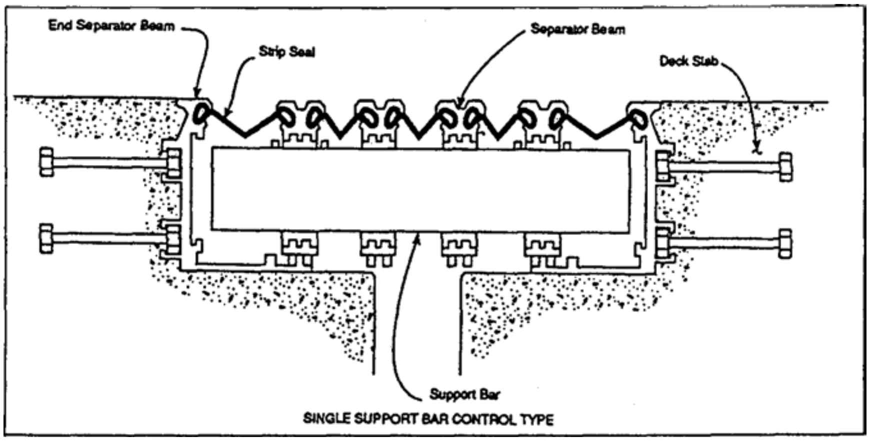

Figure 17 Modular joint with single support bar control

4.2.10.1 Disadvantage

Based on the field performance record of six different modular joint systems, the main points of concern with these joints appear to be the noise under live-load impact, water leakage at seal splice locations, debris accumulation in seals, and snow plow damage.

4.2.11 Case studies

4.2.11.1 Garden State Parkway bridge

The three-span Garden State Parkway bridge is located in New Jersey, over local creeks. The existing obsolete armored open joints are to be replaced with more watertight joints-armored compression seals. Although the existing open deck joints were paved to achieve watertight joints during an earlier deck overlay work, cracking and deterioration in the overlay allowed the joints to continue to leak and promote deterioration at the pier ends of the steel girders and bearings. The planned joint replacement process involves removing the existing overlay and the portions of deck slab over the end diaphragms, then installing of a new joint assembly, consisting of compression seal and armoring angles, and connecting it to the existing end diaphragms using new steel plates and shapes. Finally, placing the concrete around the joint assembly completes the slab and joint reconstruction. The resulting joint gives superior water tightness with minimum slab removal. (Figure 18)

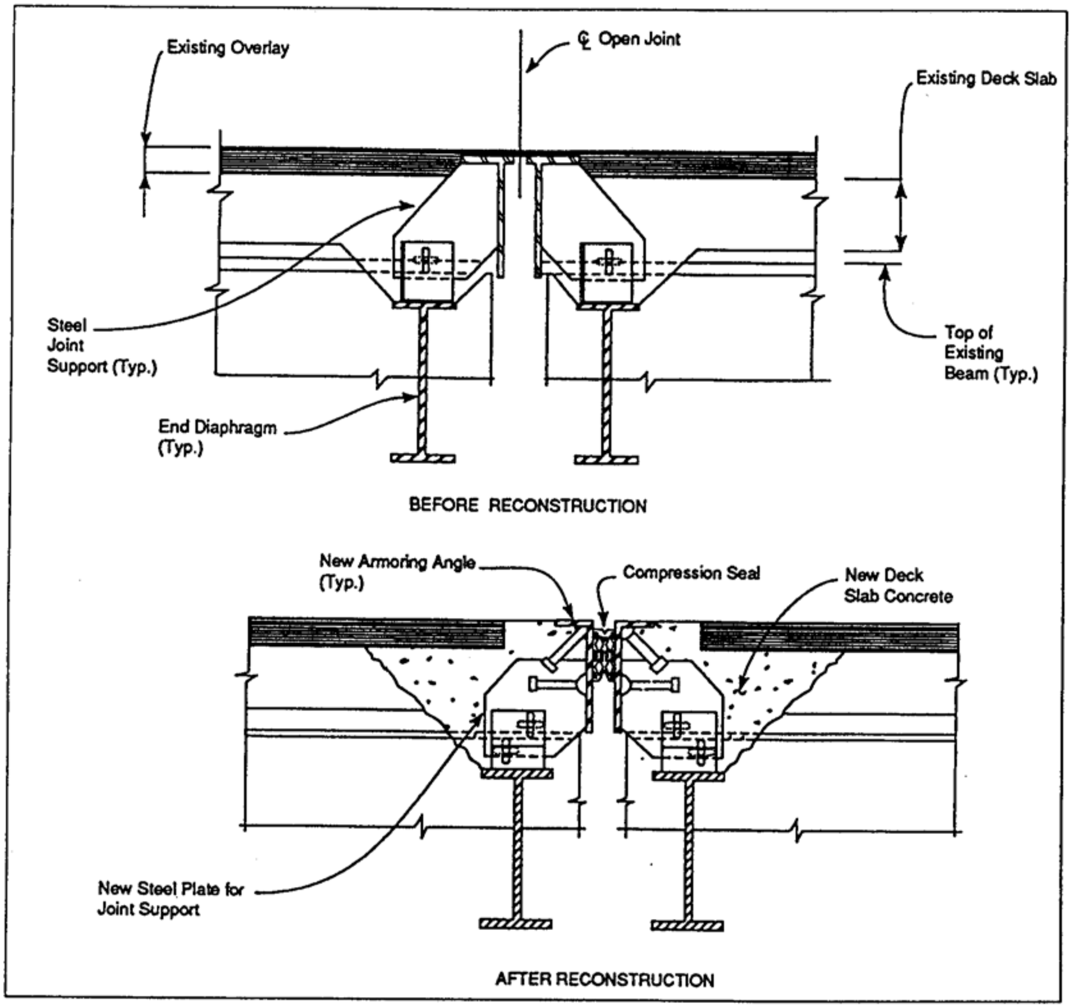

Figure - 18 Replacement of open joint by armored compression seal for joints on the Golden State Parkway Bridge

4.2.11.2 Route 71 bridge

The bridge is located on route 71 over the Shark River in New Jersey. The existing finger plate joints lacked a trough system and were permitting water to leak through and deteriorate surrounding bridge elements. To prevent further substructure deterioration and improve joint performance, the existing finger plate joints at the abutments and piers were replaced by sheet seals with steel hold-down blocks. First, portions of deck slab above the end diaphragms were removed, including the reinforced concrete encasement around the end diaphragms. The new joint substrate was formed by installing steel angles and placing concrete around them. Then the sheet seals were installed and joint restoration completed. In line with these replacements, the filled joint between the abutment backwall and approach slab was removed and replaced in kind. (Figure 19)

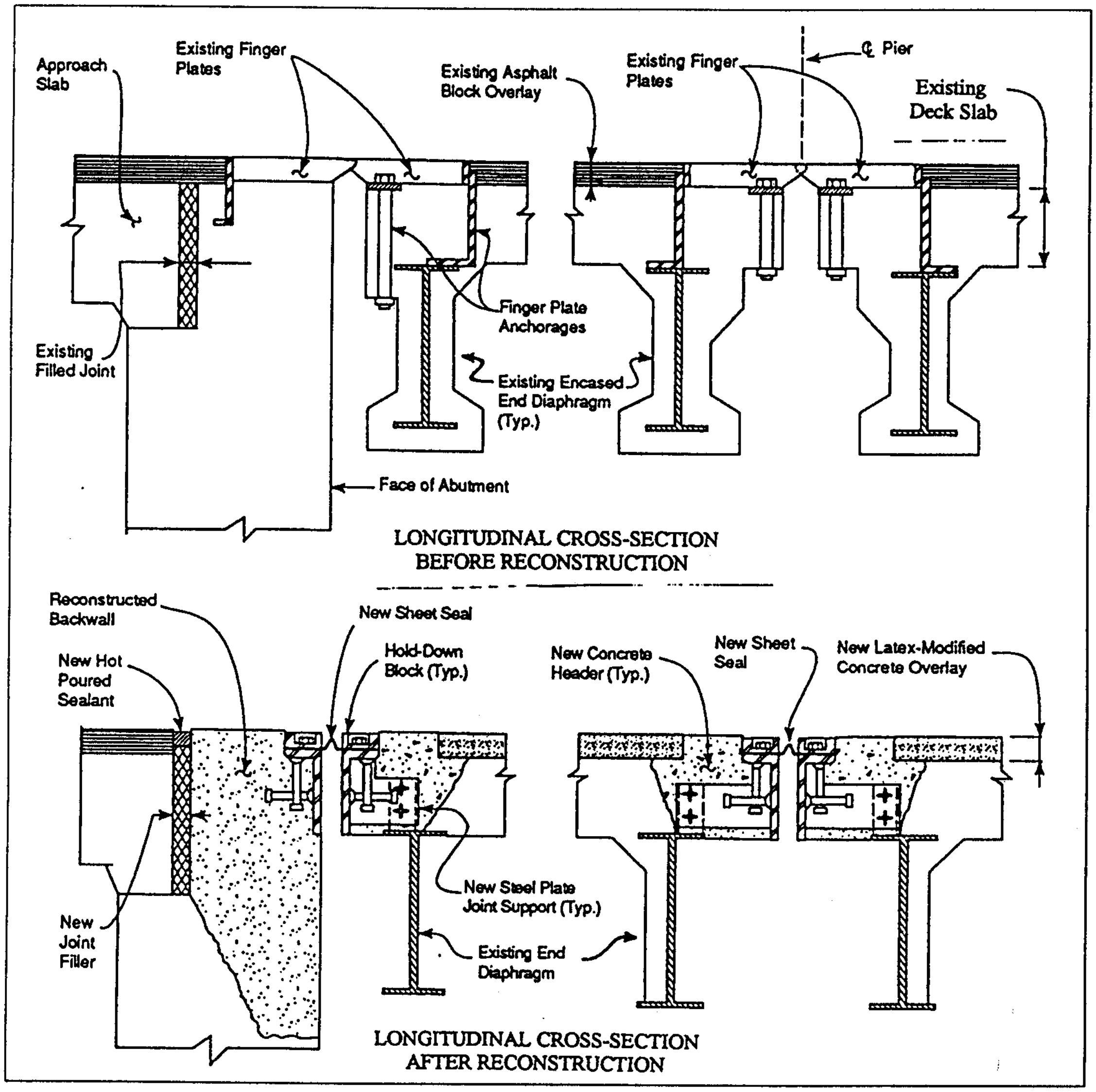

Figure 19 Replacement of finger plate joints by sheet seals on route 71 Bridge

4.2.11.3 Cherry Hill and Littleton Road bridge

The bridge is located over route 80 in northern New Jersey's Morris county. All deck joints on this 400 foot long four-span bridge were compression seal joints. After 20 years of service these seals were aged, brittle, and leaking. This was caused by spalling of the deck slab edge and loss of elasticity and debonding of the seal. Joint replacement was performed concurrently with deck rehabilitation and overlay. After removal of the existing compression seals and the surrounding portions of the deck slab, new compression seals with armoring angles were installed at elevations even with the new deck overlay. The deck slab around the new deck joint was repaired with quick setting patch material before placement of the new deck overlay. (Figure 20)

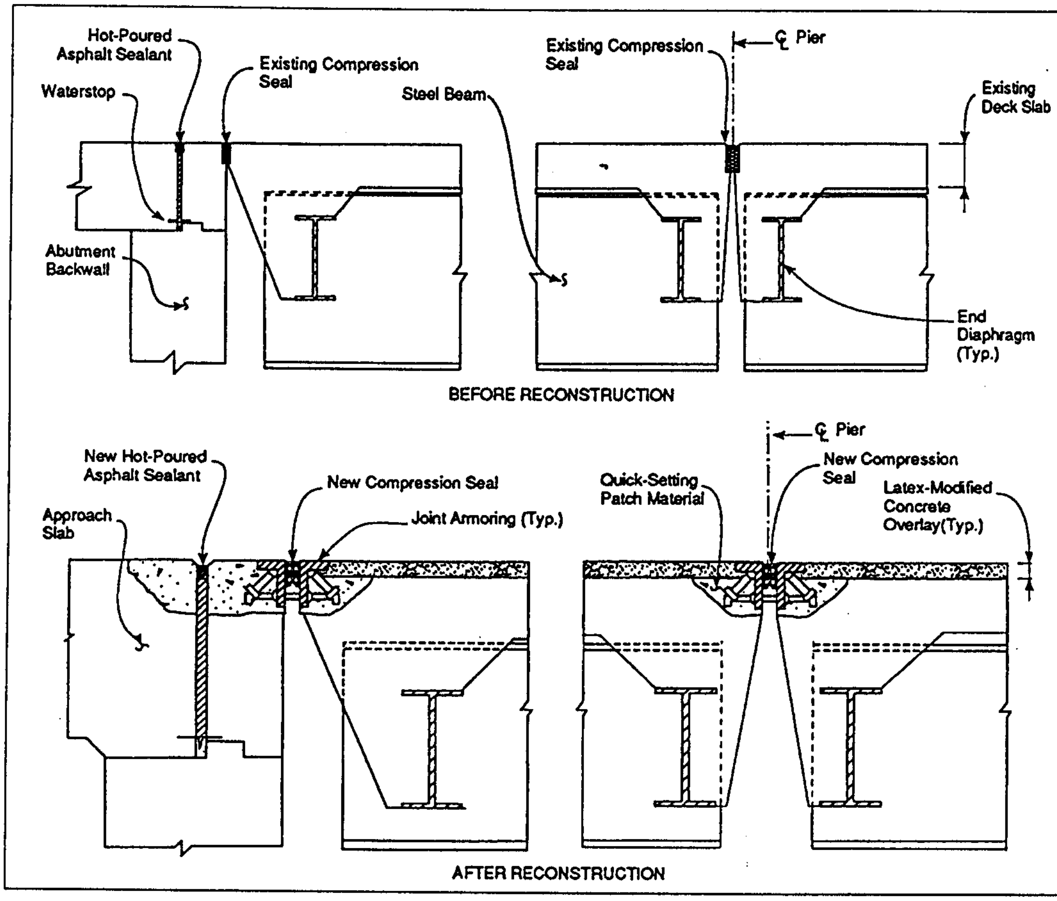

Figure 20 Replacement of compression seals in kind on Cherry Hill and Littleton Road bridge

5 SUPERSTRUCTURE REPAIR

5.1 Longitudinal External Post Tensioning - 270 K Strand or Grade 160 Rod

On either side of the damaged area, in the sound sections of the beam, symmetrical jacking corbels are built and anchored to the bottom flange. Posttensioning tendons (270 K strands, Grade 160 rod) are passed through the corbels and anchored against bearing plates as shown in Figure 21

strong enough, the preloading is removed and the exterior post-tensioning of the beam is applied, simultaneously at both corbels. To protect the bars or strands, they are placed in plastic conduits and pressure grouted.

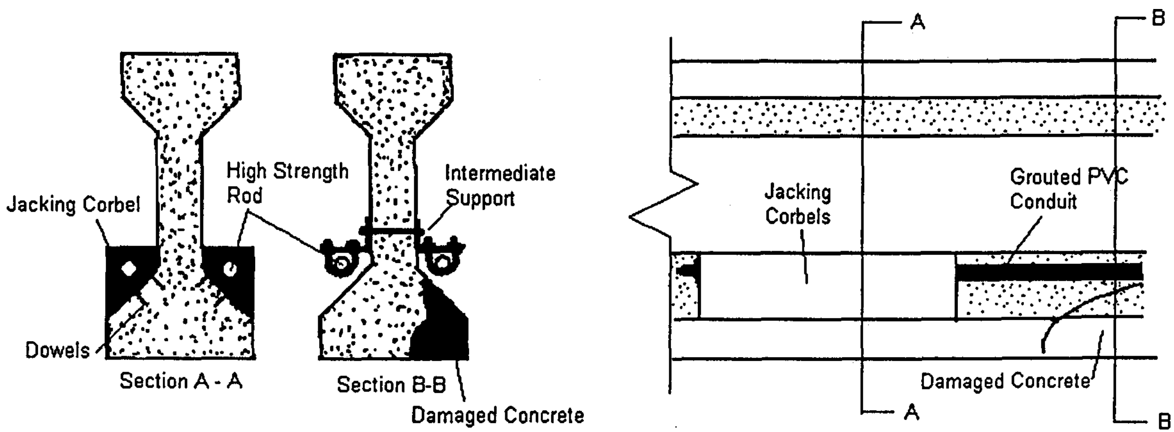

Figure 21 - Longitudinal External Post Tensioning

5.2 Internal Prestress Strand Splice

Opposite ends of severed strands are connected by an internal splice. The splice is torqued to induce a tension in the strand equal to that of adjacent undamaged strands. First, each end of a severed strand is griped by a commercially available chuck, with a threaded coupling screwed to its end. Then a turnbuckle, attached to two pieces of high strength threaded rod, is screwed into the free end of each coupling. The torque is applied by turning the turnbuckle. The concrete is then repaired. Before splicing, a certain amount of preloading of the beam may be necessary. If stresses permit, it is preferable to apply the preloading after stressing the splices and before repairing spalled concrete.

5.2.1 Case Study - Century Road Overpass, Texas

This bridge carries Century Road over Highway 16X in Texas. This prestressed concrete bulb tee bridge was damaged by an overheight load consisting of industrial forestry equipment in 1996. Damage occurred in 15 of the 18 prestressed girders. Five of the damaged girders had exposed or damaged strands. The remaining had various degrees of concrete loss.

The scope of the repair included preloading, splicing and restressing of prestressed strands, removal of damaged concrete, recasting of concrete, crack repair, and concrete sealing and finishing. The damaged strands were repaired

using the internal strand splice type of repair. This enabled the restoration of structural integrity and the reestablishment of the appearance and durability of the structure.

All damaged concrete was removed and the restored areas were sandblasted prior to recasting of concrete. A preload of 44 tons that caused a deflection of approximately 0.5" was used to create compression in the recast areas upon completion of the work. The fractured prestressing strands were completely severed by saw cutting and repaired using internal splices stressed to the initial design tension. The cracks were repaired using injected epoxy resin.

This is a good example of a severely damaged bridge restored to its pre damaged condition with a combination of methods including preloading, splicing and restressing of prestressed strands, removal and recasting of damaged concrete, crack repair, and concrete sealing and finishing.

5.3 Metal Sleeve Splice

This is an external procedure for splicing a damaged beam as shown in Figure 22 below. It does not normally restore prestress, although partial or full prestress may be restored by preloading. This splice is often used to restore the beam to its functioning capacity when there are many severed strands, or where a large quantity of concrete is missing. The plates are normally galvanized A-36 metal bonded to the concrete beam by injecting an epoxy grout into a 1/16 inch gap between the materials. Construction normally begins by applying the necessary preloading. Then the concrete is repaired. After the concrete has gained sufficient strength, the preloading is removed and the metal sleeve installed. Alternatively, the preloading may be left in place until after the metal sleeve has been placed and grouted. The latter procedure enhances the capacity of the splice by precompressing it under no live load.

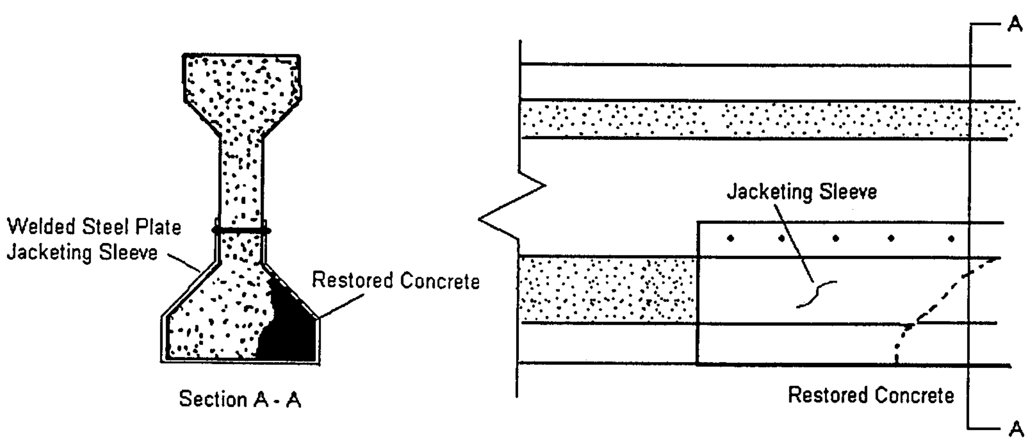

Figure 22 - Metal Sleeve Splice

5.3.1 Case Study- Clint Moore Bridge, Boca Raton, Florida

This bridge is located in Boca Raton, Florida, and carries north bound Florida Turnpike (SR 91) traffic over Clint Moore Road. The repaired bridge is located in Boca Raton, Florida, and carries north bound Florida Turnpike (SR 91) traffic over Clint Moore Road. This three span, prestressed concrete, I beam bridge was built in 1963. The bridge was damaged by an over height vehicle and subsequently repaired in 1996. The damage incurred included concrete section loss and severed prestressing strands. The repair consisted of restoration of the concrete with polymer modified grout and the placement of a 3/4 inch steel plate sleeve that was bonded to the restored section. This repair has been in place for 2 years with the high Turnpike ADTT. The most recent inspection report (12/97) states that, "The beams with impact spalls have been repaired satisfactorily." A visual inspection performed on the repair on 4/15/98 showed no evidence of deterioration. Figures 23 and 24 show the metal sleeve repair.

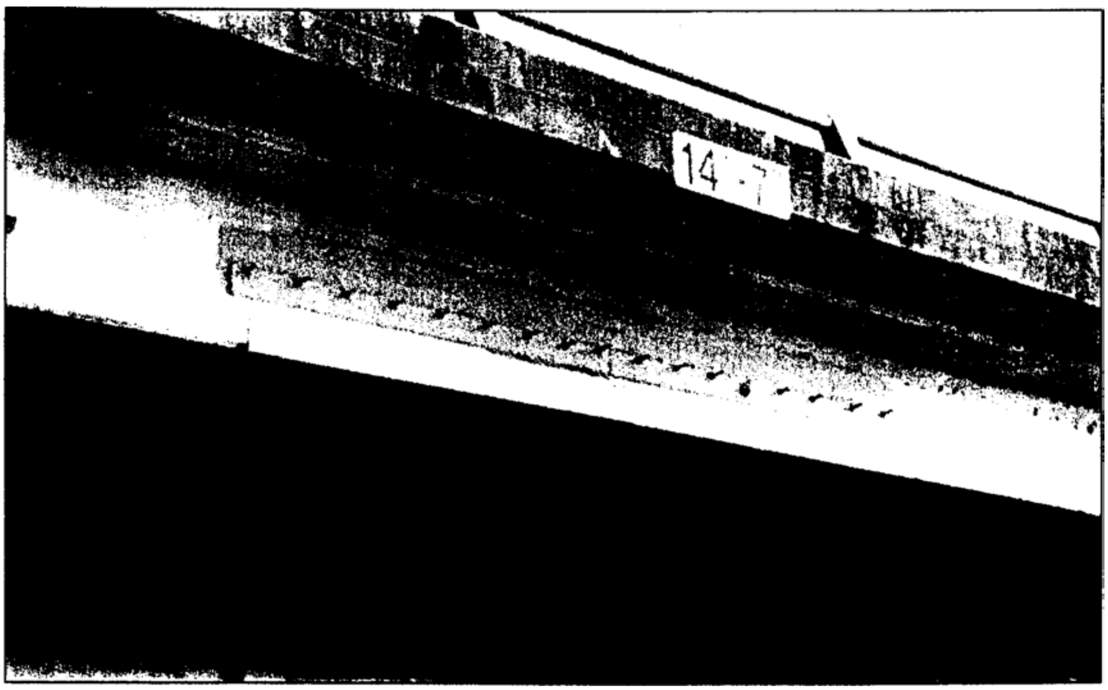

Figure 23 - Metal Sleeve Splice Repair, Clint Moore Bridge, Florida

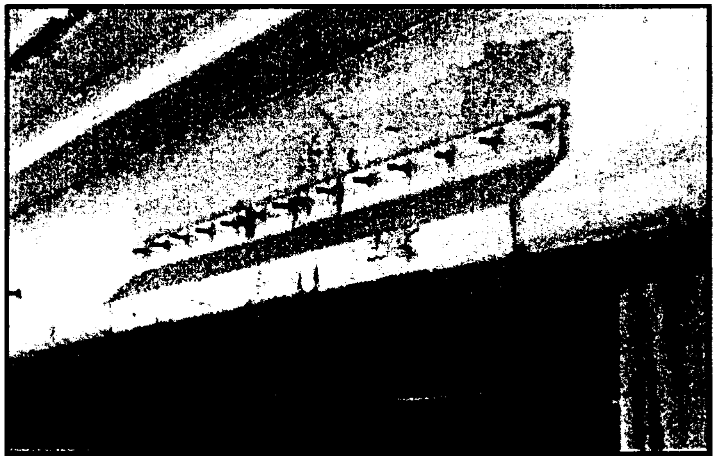

Figure 24 - Metal Sleeve Splice Repair, Clint Moore Bridge, Florida

6 SUBSTRUCTURE REPAIR

In marine environment, wave action, corrosion, UV deterioration, marine organisms and other forces are constantly at work to attack the substructure. Use of concrete in marine environment has received somewhat mixed reviews. Its durability has been well demonstrated on several marine structures, yet examples of concrete vulnerability have also been observed all too frequently. Concrete's principal enemy is corrosion, both in chemical attack of its aggregate/cement matrix and in the corrosion of the steel reinforcement necessary for its tensile strength. Bridge substructures in marine environment are more exposed to corrosion since most part of it is located in the splash zone.

Although a vast revenue resource is consumed in building bridges, 'managing' their maintenance and actually 'executing the maintenance work' can prove even more exacting and costly if what has been built must remain operational for the intended long-term safe use. This requires the following:

a) A thorough examination of the detailed inventories,
b) Carrying out detailed condition surveys and visual and hands-on inspections,
c) Analysis of observations and structures in order to unfold the causes of structural distresses,
d) Carrying out the structural computations and, where called for, the appropriate in situ tests on material samples and on existing structures, and
e) Preparations of specifications for rehabilitation and repair or outright demolition and replacement, as necessary.

Repair and strengthening practice is based only on intuition, and reliable experimental data available on the repaired strengthened reinforced concrete structural member behavior are very limited. However, the subject has recently started to receive considerable attention, and few experiment studies are being carried out in various Europeans and North American countries.

Case studies on bridge substructures are evaluated in this report with emphasis on the condition behavior of substructures under marine environment.

6.1 Underwater Inspection of the World's Longest Overall Bridges Part 1

Twin bridges crossing lake Pantchartrain near New Orleans are each 24 miles in length and are supported on more than 9500 prestressed concrete cylinder piles (Figure 25).

The first bridge was completed in 1955 and is one of the oldest prestressed concrete bridges built in the United States. The second bridge was completed and opened in 1965.

The bridge is supported on three pile bents instead of two pile bents that support the old structure.

The vertical pilings supporting the deck are all 4.5 ft in diameter. They were cast in 16 ft sections.

The method used to make the piling and the procedure used in driving them to a large extent governed how they have performed.

The primary purpose of the current inspection was to satisfy the requirements of the "National Bridge Inspection Standards". The secondary goal of the inspection and in-depth investigation was to provide information necessary to develop a pile rehabilitation program by which the life of both bridges can be materially extended.

6.1.1 Environmental conditions

Connected to the Gulf of Mexico, Lake Pantchartrain is brackish with a salt content that varies with location and season and can reach as high as 5 ppt.

6.1.2 Findings

On the pilings supporting the older bridge the inspection disclosed a number of vertical, mostly hairline cracks spaced around some pilings at about 10 inches. The location of these cracks caused concern since they appeared to occur over the location in the vicinity of prestressing tendons. Another area of concern was with the joints between the 16 ft sections that made up the pile. Grout was missing from a number of these joints. The inspector was able to push a knife blade through several joints to the inside of the pile.

As the inspection on the older bridge proceeded to the south, the number of cracks diminished; after about the first ^{3}/a mile, they were found only infrequently.

Information obtained from the chief engineer of the company that drove the pilings revealed that they had trouble driving the pilings in the beginning. A new hammer had to be designed to eliminate some of the earlier driving problems. Today inspectors believe that most of these cracks were caused by overdriving. There was considerably more cracking in the pilings supporting the newer bridge. These cracks all ran vertically and some were visible above the waterline.

A joint between pile sections in the new bridge occurred a few feet under the water. The divers found numerous vertical cracks starting at this joint and extending downward into the mud. These cracks increased in size so that at the mud line many cracks were more than 1 inch in width. Very few of these cracks extended upward above this joint.

Again, a study of the methods of constructing the two bridges disclosed why, in the inspector's opinion, the newer piling showed more distress. The data collected supported the overdriving theory. A comparison of the driving records of the pilings under the two bridges convinced the inspectors that the problem was caused by driving the pilings to grade on the new bridge, which was less expensive to the contractor than cutting off approximately 1 ft of pile when design refusal was met. The cracking occurred in a number of pilings all across the length of this bridge.

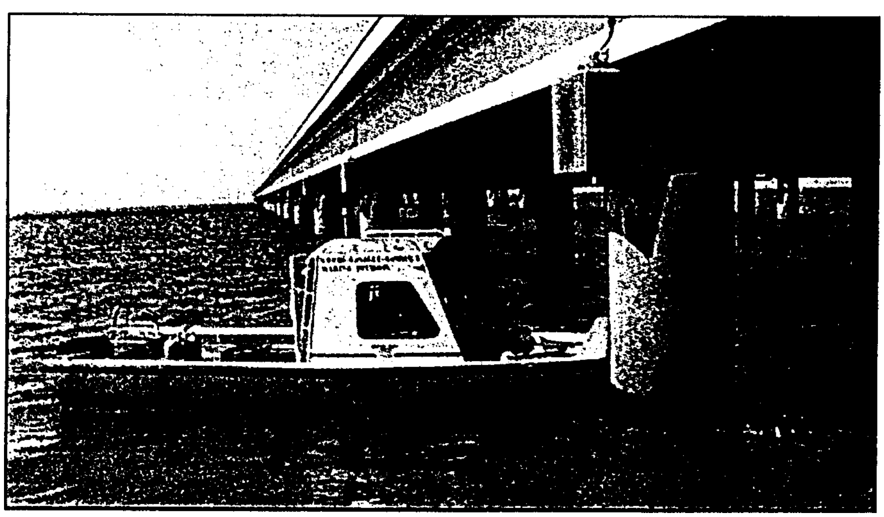

Figure 25 - Pilings of the 24-mi. long bridges crossing Lake Pontchartrain

6.2 Underwater Inspection of the world's longest bridge Part 2

During the inspection of the 9500 piles supporting the twin bridges spanning Lake Pontchartrain, a concurrent study was undertaken into methods that could be used to arrest any deterioration uncovered. This study began with a literature search into techniques used for long term protection of piles and, more specifically, protection of Raymond cylinder piles.

The study was expanded to include four basic types of pile protection:

a) Wraps with and without mastic infill,

b) Bags with cementitious infill,

c) Rigid jackets with cementitious infill, and

d) Rigid jackets with polymer or polymer grout infill (all-polymer).

Letters were mailed to the manufacturers of encapsulation products requesting information on their product and locations where inspection in the field could be made. The findings are briefly summarized as follows:

6.2.1 Wraps

Several wrap systems were examined and largely, they all employ a flexible outer wrap and a corrosion inhibiting inner liner or mastic coating. The inner liner is placed around the pile, overlapped with a flexible cover, and secured with a roller or bolted seam. Some of these systems have foam or sponge top and bottom seals, and some use stainless bands to hold the wraps in place.

The inspectors found several instances where the soft outer wraps were damaged and saw considerable evidence of seam and seal failure. The chief inspector examined a wrap installation that was less than one year old and most of it was in some state of deterioration. The evidence showed that once the outer wrap, the seals, or the seams are compromised, wave and tidal activity would cause water to pump up and down inside the wrap, allowing fresh chlorides and oxygen to reach the concrete pile.

6.2.2 Bags

Nylon bags are placed around the pile and filled with a mixture of concrete or cementitious grout. The bottom of the bag is usually banded to the pile and the longitudinal seam is closed with a zipper. In some cases the top of the bag is supported with a steel ring.

Concrete or grout is placed into the bag through a hose that enters through a opening at the top of the bag or, in some cases, through openings in the bag. Typically, the annulus between the pile and the bag is 4 to 6 in., adding considerable weight and area for wave forces to act against the pile. Several reports showed that the bags will often stretch, increasing the annulus and weight substantially. The study also found several reports of bag failure.

6.2.3 Rigid jackets with cementitious infill

Rigid fiberglass jackets or forms are installed around the piles and filled with concrete or cementitious grout. The study revealed that the general practice of installing this type of system involved filling the jacket from the top by hose or other means. The evidence is considerable that the infill could be of poor quality due to the concrete or grout falling, at least part of the way, through water. This can lead to water being trapped in the jacket. Because the jackets are opaque, defects in the infill go unnoticed. Like the bags, the annulus is usually several inches or more, adding unnecessary weight and wave force to the structure

6.2.4 Rigid jackets with polymer infill

Rigid fiberglass jackets installed around the piles are filled with either neat epoxy or epoxy grouts. In one process, the grout is poured from the top down. As found with the cementitious infill methods, most of the fiberglass jackets were opaque, making it unlikely that defects in the infill would be detected. The single exception, however, was the A-P-E (Advanced Pile Encapsulation) process that utilizes a translucent fiber reinforced plastic (FRP) jacket and pumps the epoxy grout into the jacket from the bottom up.

Advanced Pile Encapsulation (A-P-E) Process

The glass fiber reinforced polymer (FRP) jackets supplied by Master Builders, Inc. as part of the A-P-E process are made up of marine grade laminate of glass woven roving and mat, impregnated with a clear, UV light-stabilized, polyester resin. The jackets are translucent to allow the progression of grout inside the jackets to be monitored from outside. The jackets are precisely molded to conform to the structure being encapsulated with grout injection ports and integral overlapping seams.

The A-P-E pile grout is pumped into the jackets from the bottom up, through injection ports strategically placed along the length of the jackets. This assures a uniform, dense encapsulation, free of voids and air or water pockets.

6.2.4.1 Advantages

The inspectors had the opportunity for careful examination of approximately 20 A-P-E process installations established 8 years earlier on the causeway. The conclusion was that the encapsulations were as good or better than when they were installed 8 years ago (Figure 26).

A core taken through the encapsulation and 5 in. wall of the pile was sent to the test laboratory, since the inspectors wanted to learn whether the chloride penetration into the pile had been arrested with the installation of the encapsulation. A comparison of the results from the encapsulated pile with those from piles not encapsulated was made. The total acid soluble chloride by weight (percent of sample) from the encapsulated pile was 0.007 % while the total chloride from a non-encapsulated pile was 0.108 %.

Instead of the normal premixing of reactive components or "hot potting," the components are kept separate throughout the batching, mixing, and pumping phases to be blended just before entering the FRP jacket. This eliminates the need to purge the equipment and hoses periodically and, because the grout is not catalyzed, allows plain water to be used for cleanup. Environmentally harmful solvents are virtually eliminated.

6.2.4.2 Specifications

After doing literature search and examining the existing encapsulation systems, the A-P-E process was considered the best available method of protection. A decision was made to encapsulate any pile with three cracks or more and to pressure inject epoxy into those cracks on piles that had two cracks or less.

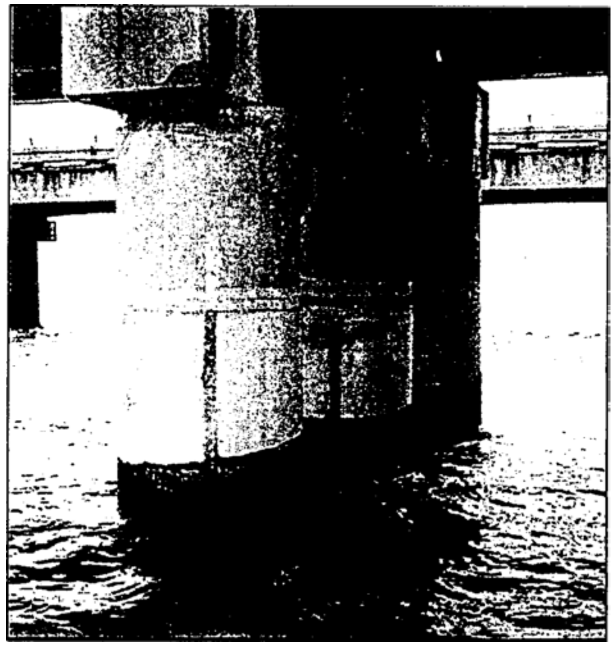

Figure 26 - A trio of A-P-E repairs found to be performing well after 8 years in service

6.3 Case Study in Boston (Master Builders)

Some older marine structures such as Pier 1 at the Boston street terminal in Baltimore, have become virtual museums of pile jacketing failure. At various points of this structure's history, 299 of the 342 concrete piles supporting the structure were jacketed, using a variety of methods and materials. Following are the different cases from a report of substructure Inspection on Pier 1, that categorizes the various types of pile jacketing deficiencies found during the inspection. The report also examined the remaining 43 piles that had no previous jacketing. Their condition is shown in Figure 27 (case 1).

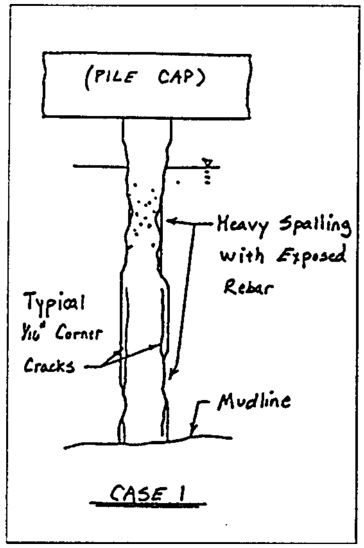

Figure 27 - No Previous

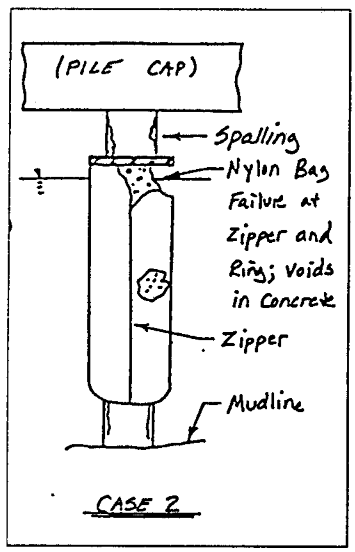

Fig 28 - Nylon Bag

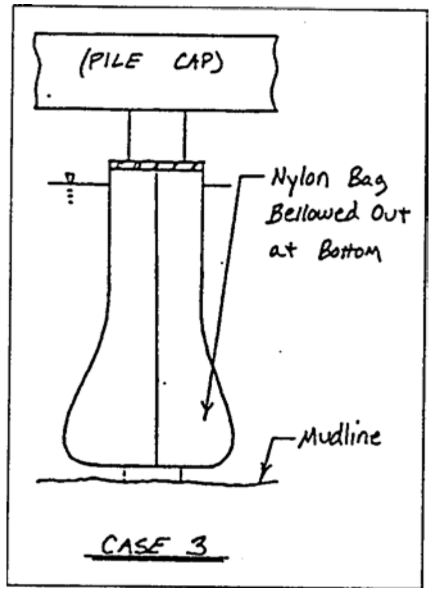

Fig 29 - Nylon Bag Bellowed Out at

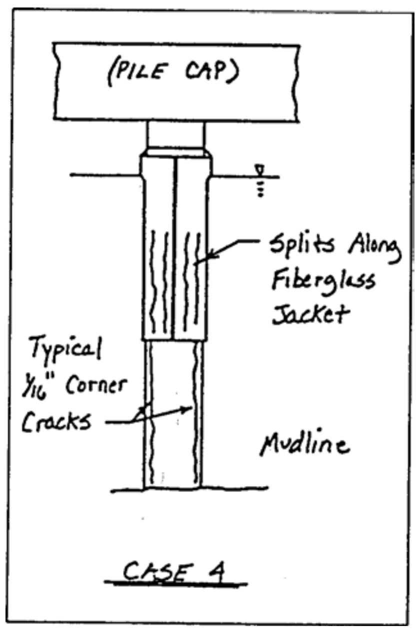

Fig 30 - Fiberglass Failure

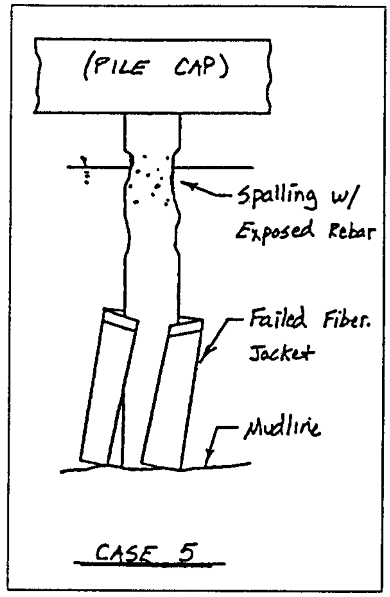

Fig 31 - Failed Fiber Jacket

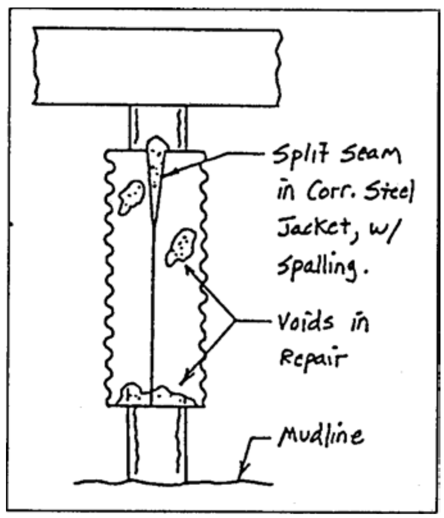

Figure 32 - Split Seam in Steel Jacket

One of the most common methods of pile jacketing, is the combination of fiberglass reinforced polymer (FRP) jackets and Portland cement based grouts or concrete, usually placed into a 3 to 5 inch space between the pile and the jacket. It is also a common practice to place a short lift of polymer grout (6 to 9 inches thick) at the top and bottom of the cement based grout.

If installed properly, pile jackets of this type would perform well for several years, but a large percentage of those observed have failed. The apparent causes of failure range from improper selection of materials to faulty installation techniques and, in many cases, to inadequacy of the construction specifications. The following presents the various mechanisms that cause failure:

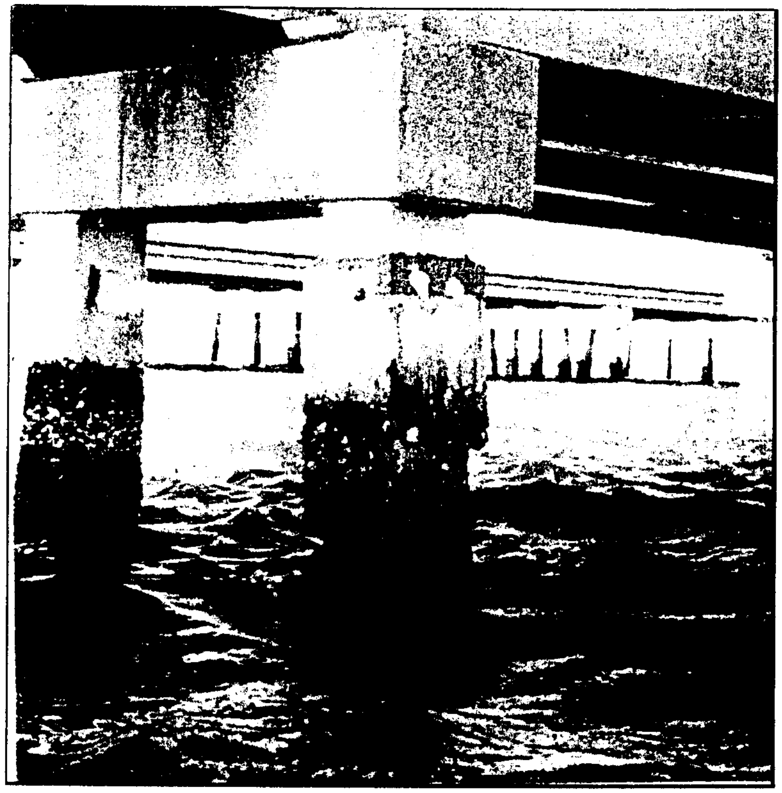

Figure 33 - FRP Jacket Missing

Figure 33 shows that the FRP jacket is completely missing and the cement grout is discontinuous in the splash zone. Lack of bond between the jacket and the grout is the principal cause of jacket failure and the presence of water in the jacket during the placement of the grout is the apparent cause of grout failure. This is typical of installations where the grout is poured into the top of the jacket, causing the grout to fall through water standing in the jacket. The specifications on this project require a 6" thickness of epoxy grout at the top and bottom of the cement grout, which was apparently not done correctly. Because the jacket was opaque, the defect could not be seen at the time of installation. Usually the first visible sign of failure is the jacket disbonding from the encapsulation grout . This sign of failure can be observed in (Figure 34). This lack of bond between the FRP jacket and the grout can be caused by many factors, such as, improper surface preparation of the jacket or marine micro-organisms on the jacket surface, but the most common cause is placement of the grout into the jacket from the top down, with at least some of the grout passing through water. Portland cement based grouts have a much lower bond potential with FRP jackets than polymer based grouts and, when poured through water, have virtually no bond at all. Once bond is broken, wave action quickly overstresses the jacket at the seams or corners and the jacket fails.

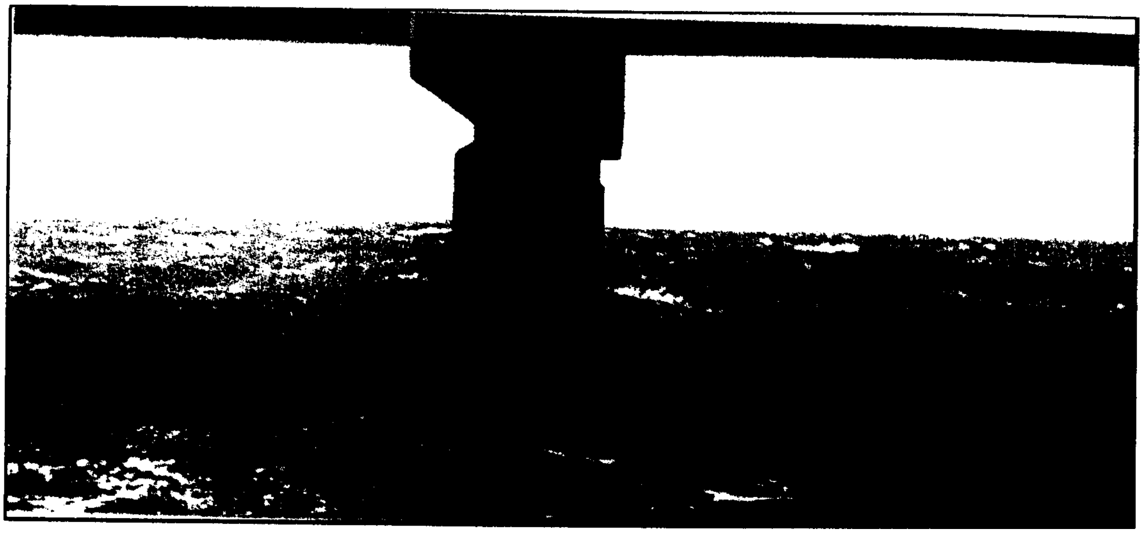

Figure 34 - Jacket Disbonding from the Encapsulation Grout

Once a jacket fails, the encapsulation grout is vulnerable to wave action and the other forces of deterioration. When the grout has been poured from the top, (Figure 35), the grout is of low quality and quickly disbonds from the pile. The grout that remains is also very permeable. The chaulky consistency of the grout (laitance) at the bottom near water surface can be observed that is indicative of pouring through water.

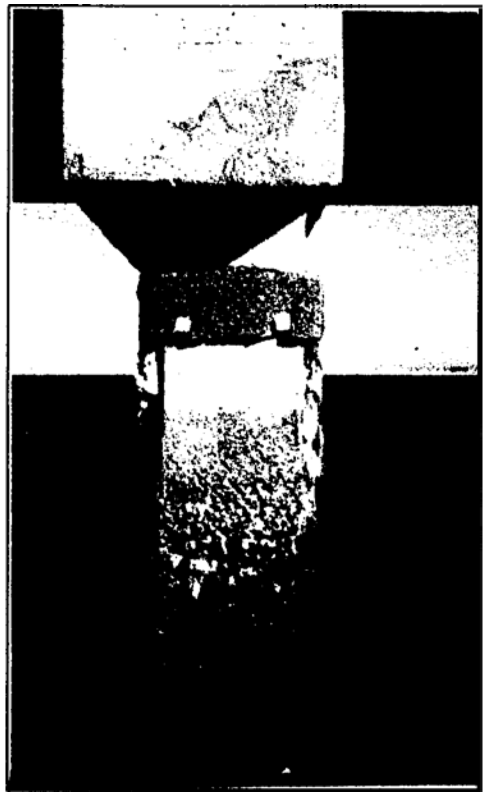

Figure 35 - Grout Disbonding from the Pile

Sometimes, encapsulations fail, but the jackets remain in place and at least partially disguise the failure. Deterioration of the encapsulation grout can be very severe at or near the waterline, yet the jacket is held in place by the polymer grout cap that bonds more tightly with the FRP jacket. One such failure is shown in Figure 36. In Figure 37, the jacket is missing and the discontinuity of the grout at the waterline is shown. These were 8 ft encapsulations, with approximately 4 ft above and 4 ft below the waterline. The original deterioration of the pile that necessitated the encapsulation, still continues, unabated.

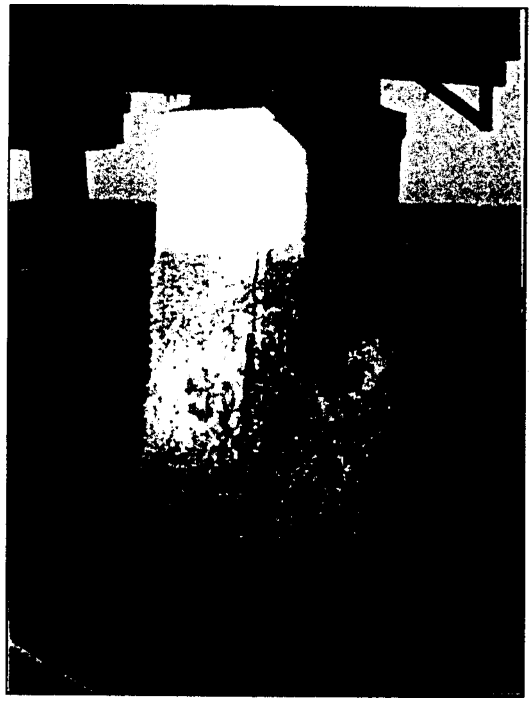

Figure 36 - Deterioration of the Encapsulation Grout

Figure 37- Jacket Missing and Discontinuity of the Grout is Shown

The specifications for the installation of the encapsulations shown on Figures 36 and 37 did not stipulate the method of grout placement, either for the bottom seal polymer grout, the Portland cement based grout or the polymer top off grout. Subsequent discussions with the specifiers, indicated that, with half of the jacket length underwater, they expected the contractor to tremmie place the bottom seal and the cement based grout. What was apparently overlooked, was the fact that the proximity of the pile cap to the top of the jacket virtually prohibited proper tremmie placement. Also, some specifications call for wire mesh that is intended to hold the grout together, but this is usually self defeating, because it interferes with any effort to tremmie place the grout.

The illustration below (Figure 38) summarizes some of the more common causes of failure found in encasement systems consisting of fiberglass jackets and Portland cement based grouts or concrete These observations were made over a five year period of time, involving numerous structures and hundreds of piles.

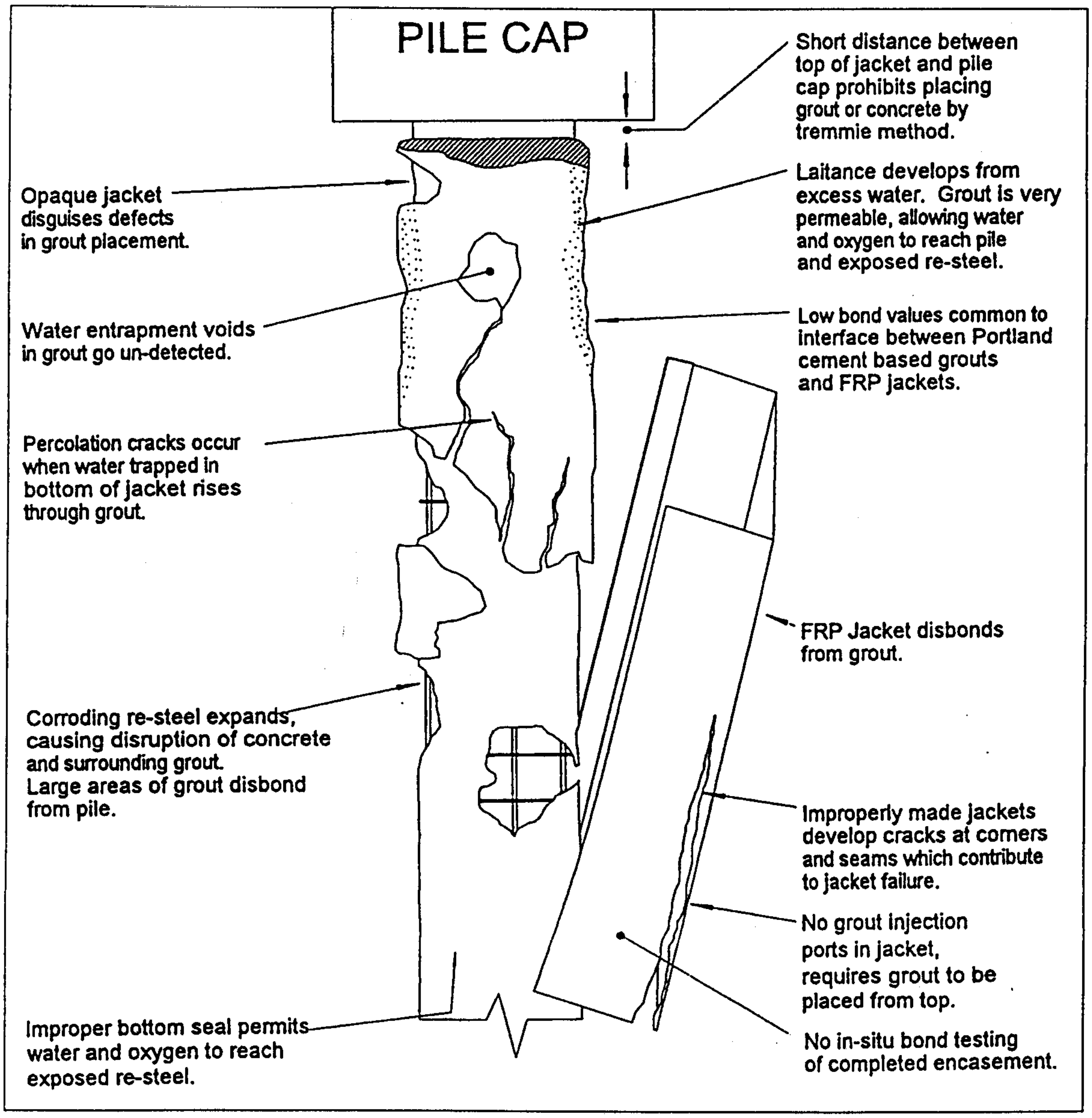

Figure 38 - Summary of the Most Common Causes of Failure

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