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Port Hueneme, California 93043-4370

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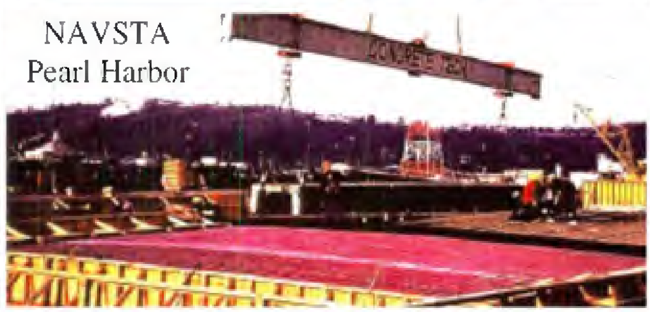
NAVY ADVANCED COMPOSITE TECHNOLOGY IN WATERFRONT INFRASTRUCTURE

1997-98 COMPENDIUM OF PUBLICATIONS



December 1998

FOREWORD

In fiscal year 1991 critical U.S. Navy waterfront deficiencies amounted to a total of $1.3 billion, of which $613 million were related to pier upgrades and repairs, and an estimated $219 million were needed for new construction. About 75% of all Navy piers and wharves were over 40 years old and required increased repair and maintenance. Most of the deficiencies were due to corrosion, in particular to corrosion of steel reinforcement. To prevent this corrosion, the use of galvanized and epoxy-coated reinforcing bars (rebars) was investigated. A recent alternative was the use of corrosion-resistant fiber reinforced plastic (FRP) components including reinforcing bars, prestressing tendons, structural shapes, and unidirectional or woven fabrics. All of these were already commercially available but were used with restraint due to the limited knowledge of their long-term behavior and the lack of uniform codes or design guidelines. In the meantime research was advancing rapidly and several demonstration structures had been constructed in Europe. In Japan, a government-sponsored national research project on the use of new materials in construction was focusing on the use of continuous fiber products in concrete construction.

In 1992 the Naval Facilities Engineering Service Center (NFESC) started investigating the potential of these new materials for Navy waterfront infrastructure applications. FRP reinforcing bars for concrete reinforcement were first evaluated.

In 1994 NFESC established the Advanced Waterfront Technology Test Site (AWTTS), to enable U.S. researchers to demonstrate their state-of-the-art composite applications on a waterfront reinforced concrete structure. This structure represents a typical Navy pier designed to withstand the highest existing Navy crane loads. It uses FRP materials as structural members or components. A parallel effort involved the assessment of structural upgrades using carbon FRP sheets.

In 1995 all components of the AWTTS were completed. Prestressing technology using graphite tendons was demonstrated in pile and deck construction. The feasibility of all-composite structural components was proven. Mechanical properties of FRP reinforcing bars had been evaluated. Structural upgrade using CFRP sheets was very successful and was scheduled for full-scale field applications. A first compendium was published in December 1995 covering research results through that date.

In 1996 half scale tests of composite upgrades were completed on the AWTTS reinforced concrete decks. Composite upgrades of operating piers at NAVSTA Norfolk, San Diego, and Pearl Harbor, were started. Additional work was completed in the areas of composite fender piles, composite hangers, and epoxy coatings for rebars. This work was included in the second compendium published in December 1996.

The two first composite upgrades were completed in 1997 (NAVSTA Norfolk) and 1998 (NAVSTA San Diego). These efforts indicated that the mechanical characterization and use of these materials had matured. However, it became apparent that long-term characterization was still inadequate, and the 1998 technological development effort concentrated on durability issues. The present compendium of publications gives an overview of NFESC's technology in advanced composites for infrastructure applications for calendar years 1997 and 1998.

L.J. Malvar, PhD, CE, MBA
Research Materials Engineer
Shore Facilities Department

NFESC TEAM

The NFESC team included:

SHORE FACILITIES DEPARTMENT

Mr. Steve Ehret	Department Director	(805) 982-1227
Mr. Preston S. Springston	Program Manager	(805) 982-1225

WATERFRONT STRUCTURES DIVISION

Mr. Robert Odello	Structures Division Director	(805) 982-1237
Dr. George E. Warren	Project Technical Leader	(805) 982-1236
Mr. Duane Davis	Civil Engineer	(805) 982-1248
Mr. Chris M. Inaba	Research Structural Engineer	(805) 982-1261
Mr. Steve Harwell	Research Structural Engineer	(805) 982-1269

WATERFRONT MATERIALS DIVISION

Mr. Don E. Brunner	Materials Division Director	(805) 982-1070
Dr. L. Javier Malvar	Research Materials Engineer	(805) 982-1447
Mr. David Hoy	Materials Engineer	(805) 982-1062
Mr. Doug Burke	Research Materials Engineer	(805) 982-1055
Dr. Tom Novinson	Chemist	(805) 982-1056

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NAVAL FACILITIES ENGINEERING SERVICE CENTER
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Site Specific Report
SSR-2285-SHR

DEMONSTRATION PROGRAM FOR WATERFRONT STRUCTURES REPAIR AND UPGRADE: SITE NO 1: PIER 11 NAVSTA NORFOLK

by

G. E. Warren

Sponsored by
Office of Naval Research

January 1997

Executive Summary

The first of a series of three demonstration projects to apply external reinforcing to upgrade the strength of existing Navy piers was completed in December, 1996. The project was executed on a deck span of Pier 11 at Naval Station Norfolk. The project consisted of a load and condition assessment of the existing deck slab, the design of a graphite reinforced epoxy laminate composite overlay for the underside of the deck, preparation of the concrete surface, installation of the upgrade overlay, installation of monitoring sensors, and a load assessment of the upgraded deck slab. The entire project was executed while the pier continued in service for USS Kearsarge (LHD-3), USS Stennis (CVN-74) , and USS Enterprise (CVN-65).

Pier 11 was designed for 70-ton truck mounted cranes and limited use by 90-ton cranes. A recent A&E study identified deck slabs in the portable crane operating lanes in the 22-ft spans to have shortfalls that limited 70-ton crane service. The goal of the upgrade was to reinforce two crane operating lanes between bents 50 and 51 so that restrictions on 70-ton crane service would be removed.

Proof load tests verified the upgrade reinforcement to be integral with the deck and there should be no restrictions placed on operating 70 or 90 ton cranes in the upgraded span. The laminate overlay had little effect on the uncracked stiffness of the deck slab but will increase the service (cracked section) stiffness by as much as 5 percent, increase the strength by 10 percent while restricting crack growth and protecting the steel reinforcing from salt water corrosion. We expect the upgrade life to be approximately 20 years. The project demonstrated that graphite/epoxy laminate overlays can be used to extend the useful life of existing piers at substantial savings compared to deck replacement. composite laminate overlays. Given sufficient deck thickness, strength upgrades of more than 40 percent can be realized with external reinforcing of graphite/epoxy laminate.

Intermittent test and evaluation of the upgrade will be conducted over the next two years. Health and load monitoring sensors are in place and functioning under the deck for future tests.

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DEMONSTRATION PROGRAM FOR WATERFRONT STRUCTURES REPAIR AND UPGRADING: SITE NO. 1: PIER 11 NAVAL STATION NORFOLK

The Upgrade Program

This project is part of an ongoing Naval Facilities Engineering Service Center (NFESC) program to demonstrate advanced technologies for increasing the strength of existing Navy piers. The program defines and demonstrates preparation and application techniques for upgrading pier decks and piling using fiber-reinforced plastic laminate systems and demonstrates sensor systems that provide measurements of performance. This project demonstrates a strength upgrade to correct an original design shortfall. In addition, the site provides an opportunity to evaluate the upgrade materials and methodology exposed to freeze-thaw cycles. With the assistance of Public Works Center (PWC) and Naval Station (NAVSTA) Norfolk Staff Civil Engineer, Pier 11 at NAVSTA Norfolk was the first site chosen to provide a platform for demonstration of pier deck upgrade and health monitoring instrumentation. The Pier 11 upgrade consisted of hand-laid, uniaxial graphite fiber-reinforced epoxy laminate applied to the deck underside of a selected span to increase flexural strength.

The application of external reinforcement to existing reinforced concrete slabs and beams was proven in an earlier NFESC advanced research project. Advanced materials such as graphite reinforced epoxy laminates were shown to be easily and rapidly applied to the underside of existing slabs and would add flexural strength and stiffness, mitigate crack growth, and increase the punching shear strength. Since the laminate was nonferrous and chemically resistant, we would expect the laminate to exhibit a longer useful life than ordinary reinforced concrete and would be capable of extending the useful life of existing waterfront structures that are being hard pressed to serve greater loads than they were originally designed.

Site Description

Pier 11 is located on the northern waterfront area of Naval Station, Norfolk, Virginia. It is a cast-in-place and precast reinforced concrete structure 1,400 feet (427 m) in length and 150 feet (76 m) wide (Figure 1). A nominal cross section is shown in Figure 2. Pier 11 is approximately 14 years old and was constructed from Naval Facilities Engineer Command Atlantic Division Drawings, JO No.542F8113, "Berthing Pier No. 11 Naval Station Norfolk Virginia", 27 Oct 1981. Pier 11 serves nuclear carriers of the Atlantic Fleet. The structure is subjected to freeze-thaw cycles. It is one of the most active and visible piers at NAVSTA Norfolk.

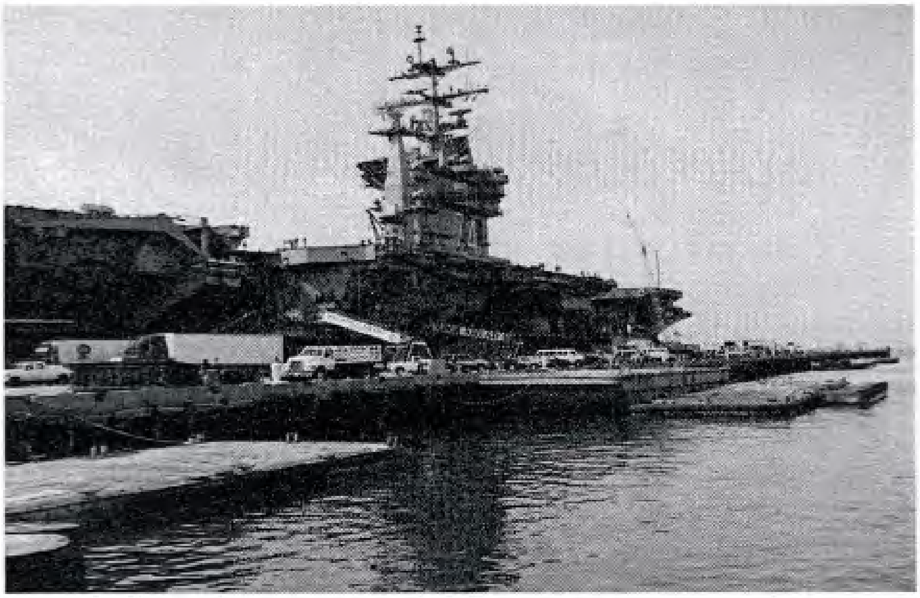

Figure 1. Carrier Pier 11 in the foreground with the USS Stennis (CVN-74) in the background.

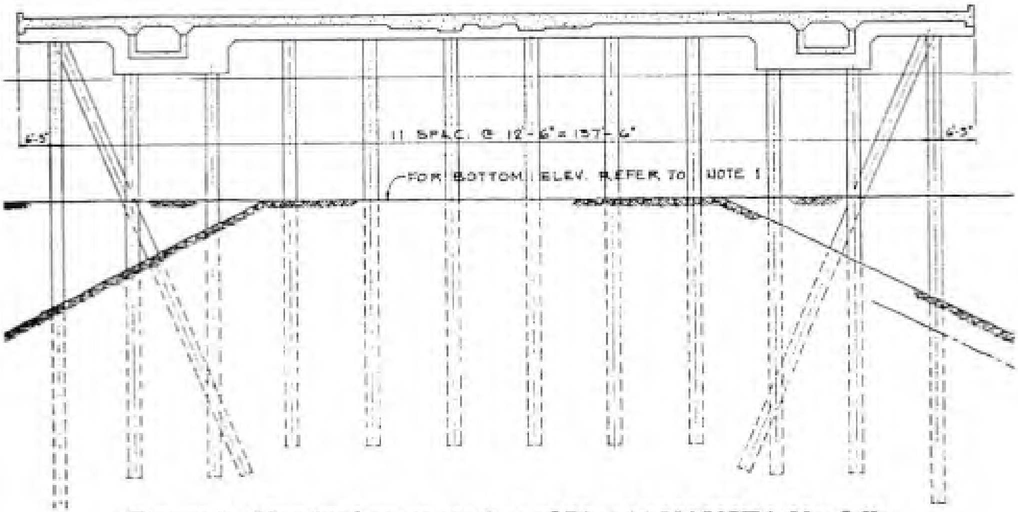

Figure 2. Nominal cross section of Pier 11 NAVSTA Norfolk.

The deck of pier 11 is supported by precast, 24-inch (61 cm) square piles each with a bearing capacity of 170 tons (1,500 kN). The piles are capped by transverse, cast-in-place beams (pilecaps) spaced at 20 feet (6.1 m) and 22 feet (6.7 m) on center that support the cast-in-place deck slab. Two utility trenches run below the deck along the full length of the pier. The design strength of the cast-in-place concrete is 4,000 psi (27 MPa) and the precast concrete is 5,000 psi (34 MPa). The nominal spans are 20 feet (6.1 m); however, there are five, 22-feet (6.7 m) spans where transformer vaults are located below the deck between the utility trenches. The deck slab is nominally 19 inches (48 cm) thick and reinforced with ordinary Grade 60 steel. Clear cover for the reinforcement is 2 inches (5 cm) on the top surface of the deck and 3 inches (8 cm) on the bottom surface. At midspan the deck reinforcement consists of No. 8 bars (2.5 cm diameter) at 4 inches (10 cm) on center on the bottom and No. 7 bars (2.2 cm diameter) at 6 inches (15 cm) on center at the top. Over the pilecap the deck is reinforced with No. 7 bars (2.2 cm diameter) at 6 inches (15 cm) on center at the top (negative reinforcement). Temperature (transverse) steel includes No. 5 bars (1.6 cm) at 12 inches (30 cm) on center both top and bottom of the slab.

As is the case with most of the Navy's berthing piers, pier 11 is heavily trafficked by large trucks and other vehicles which perform dockside maintenance on the carriers. The heaviest vehicles are truck-mounted portable cranes which operate while supported by four outrigger floats. Portable crane operations are generally limited to 25-feet (8 m) lanes adjacent to the curbs (that portion of the deck between the curbs and the second line of piles on each side of the pier). The interior width of the pier deck remains open for moving vehicles. Therefore, the outside lanes are required to support portable cranes servicing ships at berth thereby subjecting the deck slab to its most critical load case.

The pier is currently rated (and specifically designed) for 70-ton (620 kN) truck mounted cranes with limited use by 90-ton (800 kN) cranes. However, a PWC consultant engineering study identified the deck in the crane operating lanes of the 22-feet (6.7 m) spans to have design shortfalls that would limit 70-ton (620 kN) crane service. The 22-feet (6.7 m) span between pile bents 50 and 51 was chosen for upgrade demonstration. This span includes a transformer vault on each side of the pier centerline. Crane loading is not permitted on the vault hatches. Drainage holes (3 inch (8 cm) diameter) are located at midspan near each curb. Two large pipes extend across the center of each area and held in place by ordinary steel hangers suspended from the deck bottom. Photographs of the underside of the deck in span 50 -51 are shown in Figures 3 and 4.

The condition of the reinforced concrete in span 50 - 51 is excellent and there is no evidence of deterioration except some hairline cracking and minor efflorescence . The top concrete surface has minor, random cracking but no concrete spalling or delamination. An inspection of the underside of the deck in March, 1996, revealed some minor cracking and one case of minor efflorescence where water seeps through the deck on the north side of the selected span. Some rust stains produced by steel tie wire, form anchors, reinforcement chairs, and the pipe hangers were visible. No biological fouling was observed. The concrete area was sounded for delaminations and was found to be sound and free of delaminations. The most serious detail was the presence of several 5/8-inch (2 cm) deep form transition marks and irregularities (Figure 4).

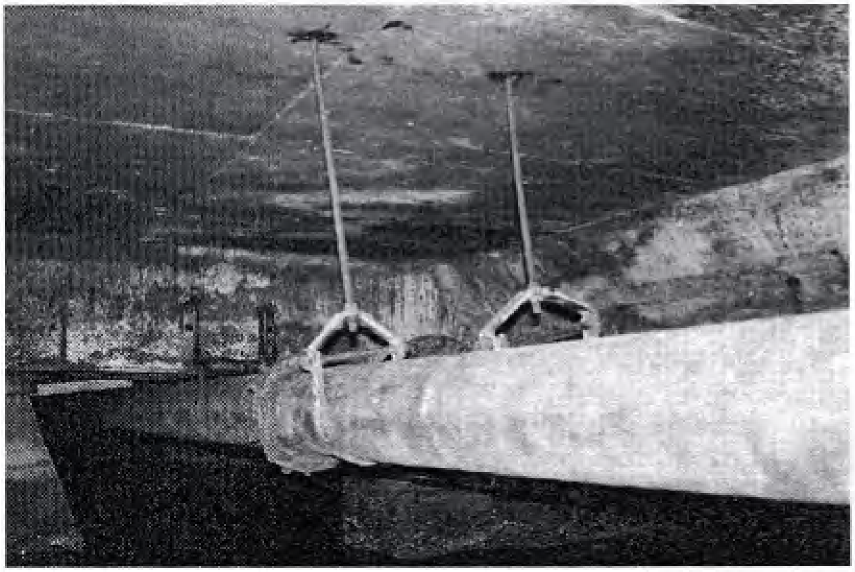

Figure 3. Under side of Pier 11 deck span 50-51 on the south side looking west.

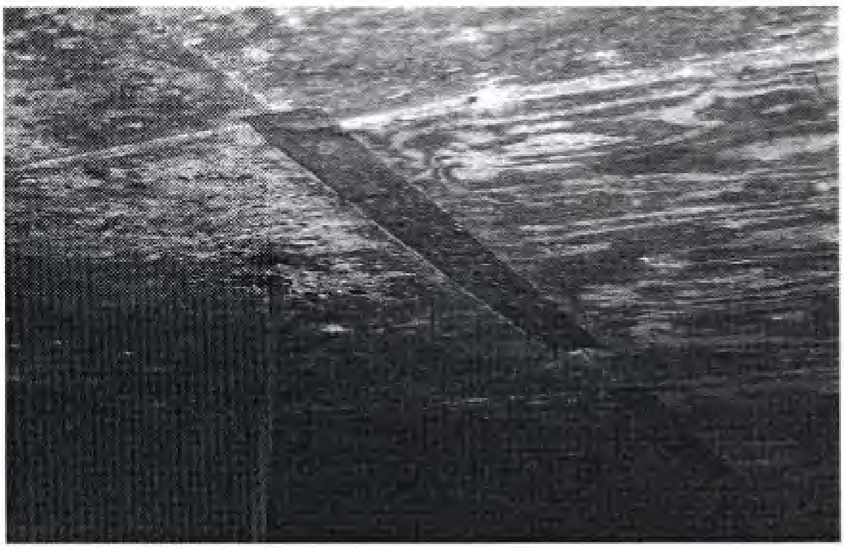

Figure 4. Pipe hanger and deep cut form marks in the under side of Pier 11 north lane.

Pre-Upgrade Load Tests

NFESC engineers conducted a series of impact load tests in March, 1996, on the areas between bents 46 and 54 as part of a load assessment (Figure 5). The purpose of the tests was to compare the structural response of the proposed upgrade area with that of surrounding spans and pile bents. From the structural response we also searched for anomalous behavior indicative of corroded rebar and loss of concrete strength. We applied the measured response to validate a finite element model the deck slab which we used for detailing additional reinforcement. Typical deflection responses for impact loads of 55,000 lbs (245 kN) appear in Figure 6. The tests were conducted in the middle of the crane operating lanes along Line B (9 ft - 7 in (2.92 m) from the north curb) and Line D (9 ft - 7 in (2.92 m) from the south curb). The series of plots in Figure 6 contain the deflection (Z axis) pattern at each load point (pile caps and midspan - Y axis) as a function of distance from the impact load point (X axis). The horizontal axis in the graphic's foreground represents longitudinal line over the deck near the center of the crane lane. The deflection response of the pile caps was equivalent. The response of the proposed upgrade span (50-51) was greater than all neighboring spans. This is indicative of the proportionally less reinforcing for length than that employed in the other spans (slab thickness and concrete strength were the same). This manifests in less structural stiffness and more displacement than neighboring spans. The tests did not reveal any mechanical damage in the area.

Figure 5. Falling weight deflectometer used to load test pier 11.

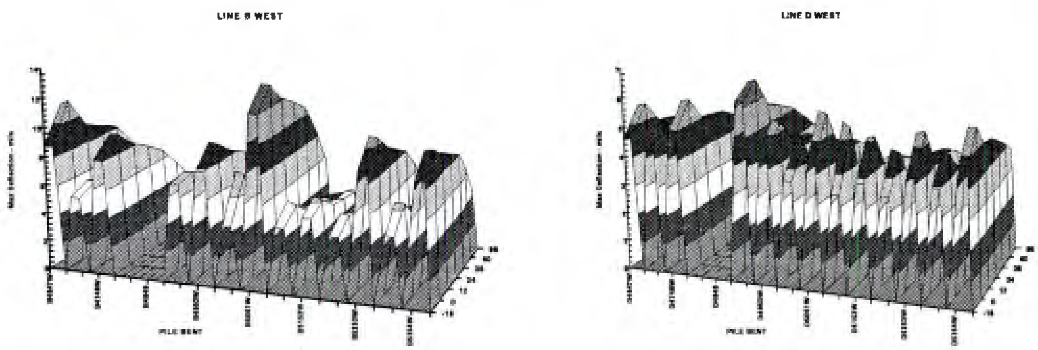

Figure 6. Deflection response at pile caps and midspans in areas of upgrade. The south side is represented on the left and the north side on the right.

The Upgrade

NFESC designed a carbon graphite composite laminate upgrade for two crane operating areas near each curb of the 22-feet (6.7 m) span between pilecaps 50 and 51 to increase the deck strength to the same level as the 20-feet (6.1 m) spans. The required strength increase was approximately 10 percent. NFESC has demonstrated strength increases of more than 40 percent in similar applications but such a dramatic accretion was not needed in this case. The upgrade areas are 19 ft - 2 in (5.84 m) in the longitudinal direction and 15 ft - 11in (4.85 m) in the transverse direction. The areas are adjacent to the curbs and outboard of the utility trenches. These are relatively small areas of less than 35 yds^2 (30 m^2) on each side of the pier. Figure 7 is a detail of the upgrade area on the north side of Pier 11. The maximum outrigger float load is 155,000 lbs (590 kN) applied on a 24-inch (61 cm) pad. The uniform live load requirement is 1000 psf (50 kPa). The upgrade consisted of adding external, biaxial reinforcing on the underside of the deck equivalent to 9.9 kips/foot-width (17.4 kN/cm) (ultimate strength) in the longitudinal (strong) direction and 3.3 kips/foot-width (5.8 kN/cm) in the transverse (weak) direction. Piles are more than adequate to support 70 and 90-ton (620 and 800 kN) cranes plus lateral loads from mooring and berthing. Therefore, the piles did not require upgrading. The utility trench shown in Figures 3 and 7 was also not included in the upgrade. There were no damage or corrosion mitigation requirements for the concrete or reinforcing steel. The calculated bending resistance in the strong (longitudinal) direction of the existing slab was:

Mcr = 342 in-kip/ft width
My = 1931 in-kip/ft
Mu = 2805 in-kip/ft

While that in the weak (transverse) direction was:

Mcr = 342 in-kip/ft
My = 262 in-kip/ft
Mu = 464 in-kip/ft

With the upgrade carbon laminate reinforcement added to the underside of the deck, the bending resistance in the strong direction was increased to:

My = 2168 in-kip/ft (12 percent increase over baseline)
Mu = 3267 in-kip/ft (16 percent increase over baseline)

While the resistance in the weak direction was increased to:

My = 390 in-kip/ft (48 percent increase over baseline)
Mu = 1308 in-kip/ft (180 percent increase over baseline)

The punching shear strength of the existing slab is approximately 310 kips for a 24-inch square outrigger pad. Based on test experience, NFESC expects the upgraded slab to have a punching shear strength of over 350 kips.

Upgrade construction was completed in November, 1996. The form marks and surface discontinuities were knocked down by removing concrete with a small, compressed-air-driven hammer. The concrete surface was ground smooth and cleaned by sand blasting. Protruding tie wires, reinforcing chairs, and rust stains were also removed by grinding. Large surface indentations that remained after grinding were filled and smoothed with hydraulic cement grout. After surface preparation an epoxy primer was applied to the concrete and allowed to cure. Epoxy void filler (putty) was then used to fill in small voids and other surface irregularities larger than 0.04 inch (1 mm). On the south side a thin layer of putty was trowelled over the entire surface area while on the north side putty was applied in small patch areas. Installation of carbon fiber composite laminate began within an hour of applying the putty. The composite consisted of uniaxial carbon graphite fiber sheets and epoxy resin matrix (saturant). The laminate was hand laid and cured without the aid of external heating or vacuum bags. Tonen Corporation's Forca Tow Sheet FTS-C1-30 ™ was installed by Structural Preservation Systems (SPS) on the south side of the pier and Mitsubishi Chemical Company's Replark 30 Type ™ system was installed by SCI Services Group, Incorporated, on the north side. The carbon fiber sheets of each system are very similar. Both have a tensile strength of 3.3 kips/inch-width (5.8 kN/cm-width) and areal fiber weight of 0.06 lb/ft^2 (300 g/m^2). Both are epoxy impregnated so that individual fibers are coated with resin. Tonen sheets are 20 inches (50 cm) wide while Mitsubishi sheets are 13 inches (33 cm) wide.

SPS covered the entire surface with three layers of Tonen graphite sheet: Two layers were aligned longitudinally (strong direction) and one layer was aligned transversely (weak direction). In addition, another 20-inch (50-cm) sheet was added longitudinally near the curb for additional longitudinal reinforcement. Epoxy saturant was applied by hand roller to the carbon fiber sheet prior to placement. The first layer was applied longitudinally directly to the uncured putty. The second layer was applied in the transverse direction. Successive sheets were applied by rolling epoxy on the sheet as well as to the previous layer. Excess saturant and bubbles were brushed, squeegeed, rolled, and otherwise "worked out by hand" from each layer. A 3-inch (8 cm) diameter hole was cut in the composite at the drain located near the curb at midspan. Gaps were also cut near midspan for the pipe hangers. Fibers were cut and sheets were lapped near midspan. Sheet laps were 4 inches (10 cm) or more in length. An outside layer of saturant was rolled over the entire area. The finished Tonen laminate thickness was approximately 0.05 inches (1 mm) and covered the entire area under the crane operating lane (Figure 8).

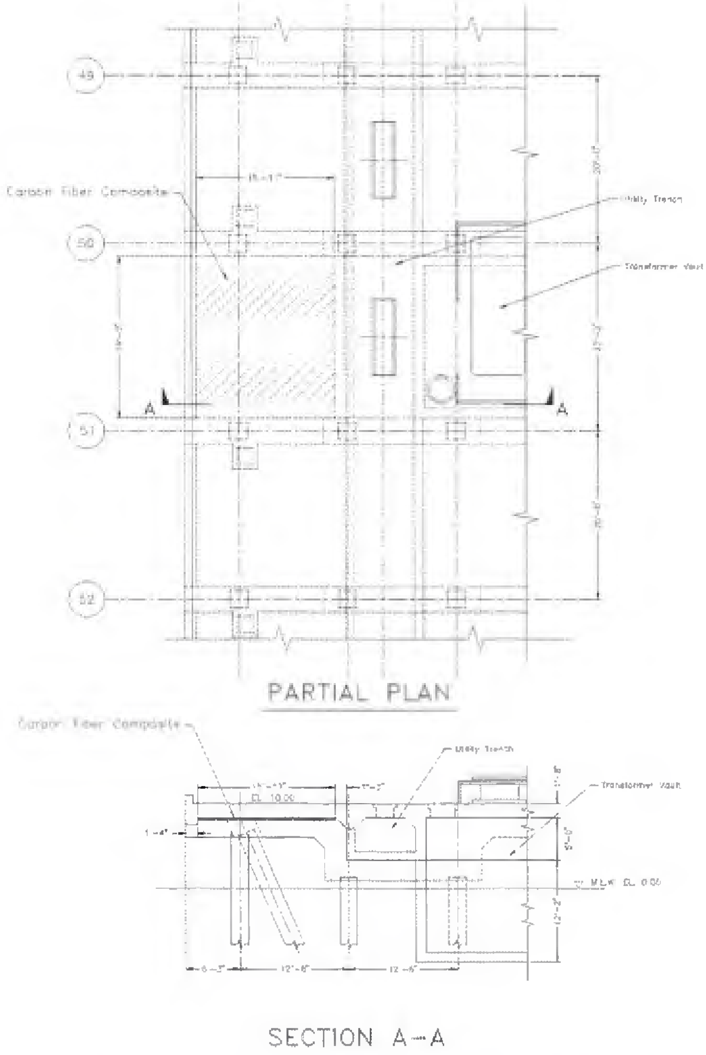

Figure 7. Detail of deck upgrade in north side crane operating area.

Figure 8. Finished installation of graphite laminate on the south side lane.

The installation process on the north side was similar to the opposite side. Rather than coating the entire surface, SCI used epoxy putty sparingly to patch small anomalies that remained after the primer coat had cured. They laid the Mitsubishi graphite sheets (strips) with approximately 6-inch (10-cm) space between them (20 inches (50 cm) on center). Spaces between the strips are to allow water to pass that leaks through the deck. The orthogonal (longitudinal and transverse) reinforcement layout resembled a grid (Figure 9). The number of sheet layers in each direction varied between 1 and 5. There were 12 transverse strips (weak direction). The middle 4 strips had double layers and the remainder were single layers. Nine strips were placed longitudinally in the strong direction. The 4 strips near the curb were 5 layers thick while the 4 strips near the utility trench were three layers thick. The remaining strip near the center was 4 layers. The strip spacing was adjusted slightly to miss the drain hole near the curb and the pipe hangers in the middle of the span. The 19 ft - 2 in (5.84 m) long longitudinal sheets were all continuous. The 15 ft - 11 in (4.85 m) long transverse strips included a staggered, 8-inch (20 cm) lap near midspan. SPS did not pre-apply epoxy saturant to the carbon strips. Epoxy saturant was applied only to the surface overhead and the saturant was worked through the fiber layers with hand rollers.

Figure 9. Finished installation laminate reinforcing grid on the north lane.

Monitoring and Instrumentation

The area on the north side was instrumented to follow the life cycle of the composite and its response to applied patch loads (see impact load tests above). Instrumentation included strain sensors located at grid points to measure maximum load response (Figure 10) and distribution as well as damage sensors along the edges and at the point of maximum load response. The former are traditional wire strain gauges attached to the laminate and aligned with the uniaxial graphite fibers. The gauges are capable of sensing strain in excess of 1.5 percent and will sense flexural cracks as load is applied to the deck.

The second set of sensors is piezoelectric patches attached to the laminate. The patches are sensors for a new impedance-based qualitative health monitoring technique to detect incipient damage in the composite laminate developed by the Center for Intelligent Material Systems and Structures (CIMSS) at Virginia Polytechnic Institute and State University. The technique relies on frequency tracking driving-point impedance of the structure above 30 kHz to identify damage. The qualitative damage detection is made by looking at changes in the structural coupling to a piezoelectric sensor/actuator by monitoring the change in electrical impedance of the bonded transducer. CIMSS engineers epoxy-bonded 25 piezoelectric actuator/sensors to the laminated area: 5 at each corner and 5 near the center at load point B identified in Figure 10. The patches excite and sense high frequency signatures in the near vicinity and will provide real-time structural health monitoring by sensing initiation of edge delaminations and structural damage in the laminate. Sensor data is collected by a central data logger and impedance analyzer at the site where it can be accessed and evaluated. The data logger can also be upgraded to be accessed by telephone modem for remote structural assessment.

After installation baseline measurements were taken by CIMSS. Converting the real time data to the frequency domain yields real and imaginary components. Only the real values will be used for health monitoring. Figure 11 is an example of real impedance from sensor Group C on the southeast corner of the north lane laminate area. The real impedance response of all sensor groups exhibited low dynamic activity (few peaks and valleys) over a large frequency range. This behavior was expected due to the high stiffness of the structure and the damping nature of the composite laminate. This behavior is an advantage when monitoring debonds and delaminations since such defects will quickly induce new dynamic activity over the frequency range that will be easy to detect.

Post-Upgrade Load Tests

NFESC conducted load tests on the upgraded section in December, 1996, using the same methodology used to for the original structural assessment. The composite laminate upgrade had not yet been subjected to freezing temperatures. Maximum load levels were less than 60 kips (270 kN) applied to an 11-inch (28 cm) diameter platen. The structural response was less than the cracking strength of the reinforced concrete. We determined that the graphite composite was acting integrally with the reinforced concrete. An inspection of the underside of the pier revealed that the composite had fully cured although some discoloration had appeared. We determined that the response of both sides were equivalent. Figure12 is the longitudinal strain and deflection distribution response to a patch load of 58.5 kips (260 kN) applied at the midpoint of the upgraded north crane lane. Figure 13 is the patch load-deflection response at midpoint on the upgraded south lane. Comparing previous load response without external reinforcement, we found little change in slab stiffness after adding the external reinforcement (Figure 13). The longitudinal strain response at the load point was linear in the range of measurement because the concrete remained uncracked (Figure 14). We expect the concrete to crack at about 100 microstrain (100 x 10^{-6} in/in (m/m)). If the response is extrapolated beyond cracking, the expected strain in the reinforcement would still be well within a service range of less than 200 microstrain responding to an applied patch load of 155,000 lbs (590 kN), the maximum outrigger load (Figure 15). The steel reinforcement will be stressed to much less than half its yield strength and cracks that are formed during loading should close after the load is removed.

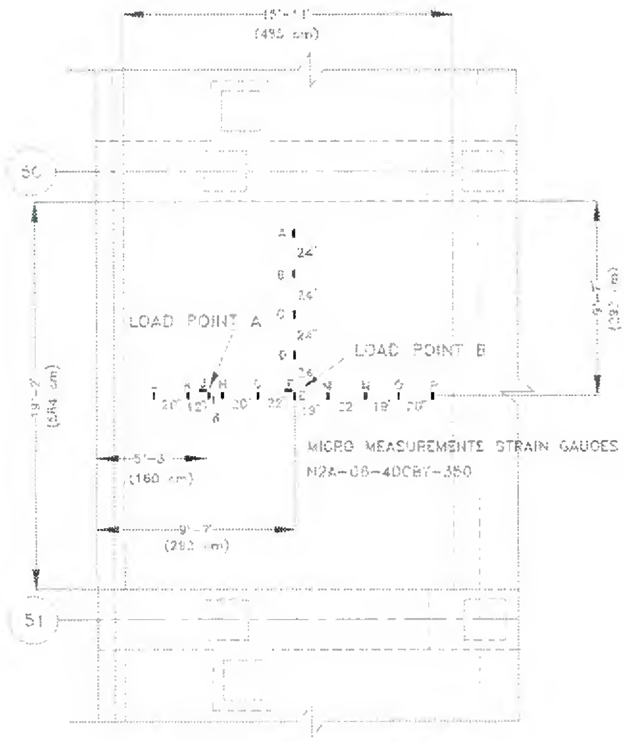

Figure 10. Strain gauge layout of the north side upgrade of pier 11.

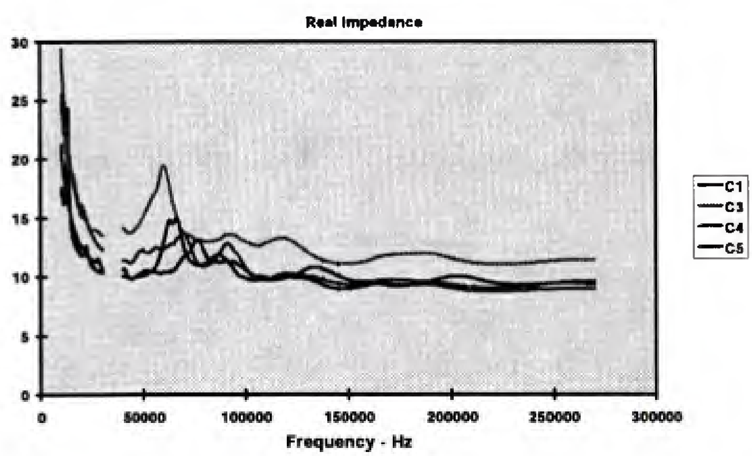

Figure 11. Initial real impedance measurements of the PZT sensors in Group C.

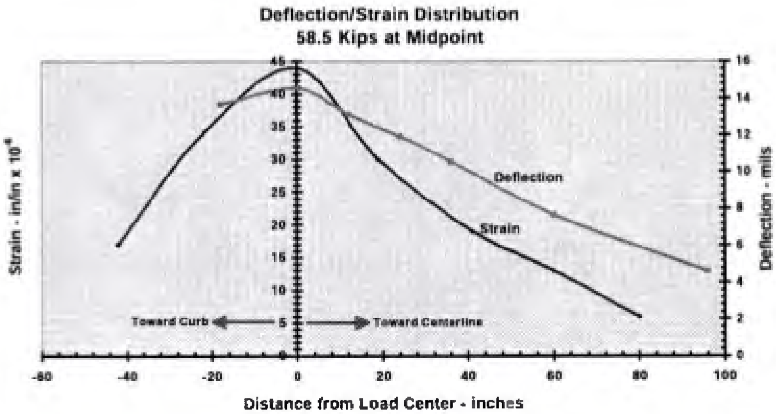

Figure 12. Measured longitudinal strain and vertical deflection distribution about 58.5-kip patch load applied at midspan aligned with Line B (point B near geometric center of upgrade area).

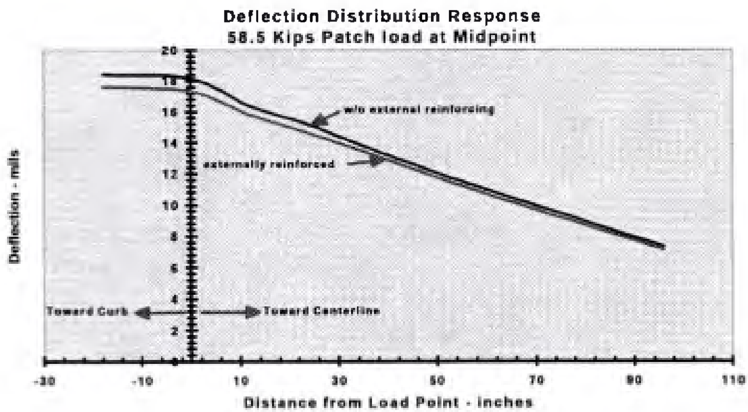

Figure 13. Deflection distribution about load point at midpoint of upgrade area.

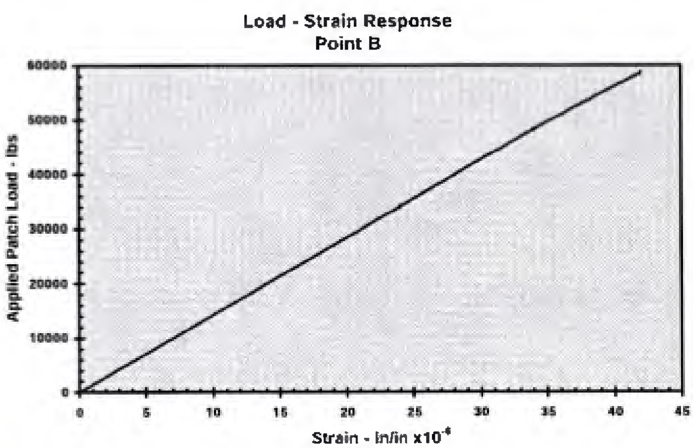

Figure 14. Longitudinal strain response from patch load applied at midspan - Point B

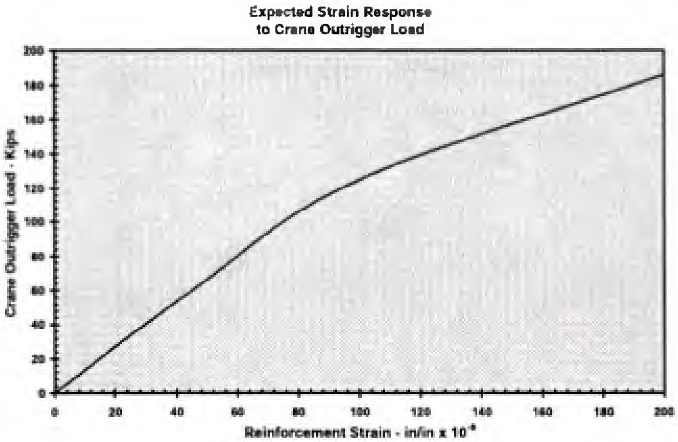

Figure 15. Expected longitudinal strain response in the external reinforcement due to outrigger load placed at midspan.

Conclusions

Adding external composite reinforcement to existing pier decks is a viable means of upgrading the deck strength. In those cases where the reinforcement is added to the underside of the deck, the pier can remain in service during installation. Graphite composite laminating materials are very expensive \$150/ lb (\$70/Kg) or \$82/yd^2 (\$90/m^2 per layer) but the installation time is fast and labor costs are expected to be less than \$55/yd^2 (\$60/m^2). Given that graphite is 8 to 12 times stronger than steel, there is little or no down time of the structure, and we anticipate the life of a graphite composite should be longer than traditional pier construction materials (it is noncorrosive), external composite reinforcement can be an attractive upgrade alternative to demolition and reconstruction. For existing piers with sufficient deck thickness, strength upgrades of more than 40 percent can be realized with external composite reinforcing.

2. Warren, G.E., "Waterfront Repair and Upgrade, Advanced Technology Demonstration Site No. 2: Pier 12, NAVSTA San Diego," Site Specific Report SSR-2419-SHR, Naval Facilities Engineering Service Center, November 1998.

NAVAL FACILITIES ENGINEERING SERVICE CENTER
Port Hueneme, California 93043-4370

SITE SPECIFIC REPORT
SSR-2419-SHR

WATERFRONT REPAIR AND UPGRADE ADVANCED TECHNOLOGY DEMONSTRATION SITE NO. 2: PIER 12 NAVAL STATION SAN DIEGO

by
G. Warren

Sponsored by
Office of Naval Research
800 North Quincy
Arlington, VA 22332

and

Naval Station San Diego
3455 Senn Road
San Diego, CA 92136

November 1998

WATERFRONT REPAIR AND UPGRADE
ADVANCED TECHNOLOGY DEMONSTRATION SITE NO. 2:
PIER 12 NAVAL STATION SAN DIEGO

EXECUTIVE SUMMARY

This project strengthened Pier 12 at the Naval Station San Diego to meet demands of operational changes accompanied by higher vertical loads. We added external, carbon/epoxy composite reinforcing to two areas of Pier 12. These upgraded areas will provide platforms to support outrigger loads up to 100 kip (450 kN) and a uniform load up to 750 psf (36 kPa). To accomplish this we increased the capacity of the deck slab. An upgraded area is located at each berthing of Pier 12. They are suitable for 50-ton mobile truck crane operations with 100-kip (450 kN) maximum outrigger loads.

The project included concrete repair, surface preparation, and strength upgrades for 14 spans. The spans are between pile bents 6 and 14 on the east end of the berthing and 67 and 73 on the west end. The specific project tasks included:

1.Repaired deteriorated concrete of the deck and replaced corroded reinforcing steel.
2. Sealed existing cracks in the deck with polyurethane.
3. Embedded high strength carbon composite reinforcing rods in the top surface of the deck.
4. Bonded wet lay up, high strength carbon laminate to the bottom surface of the 24-inch thick deck section.
5. Bonded pultruded, high strength carbon composite strips to the bottom surface of the 8-inch deck section.
6. Bonded pultruded, fiberglass composite I-beams to the bottom surface of the 8-inch deck section and anchored ends to pilecaps, utility loops, and bollard platforms.
7. Installed fiberglass composite hangers to hold the steam line in the spans between Bents 8 to 11.

This report details the methodology of upgrading Pier 12. It includes the specifications and the as-built drawings.

This project is part of an ongoing Repair and Upgrade Program to demonstrate advanced technologies for strengthening existing Navy piers. The methodology incorporates the placement of a few ounces/ft^2 of high strength composite material in strategic locations versus adding hundreds of pounds of additional concrete and steel.

Naval Station (NAVSTA) Staff Civil Engineer selected Pier 12 as the platform for the post strengthening demonstration. The Naval Facilities Engineering Service Center conducted structural assessment, provided the repair and upgrade design, and selected the materials. Public Works Center, Staff Civil Engineer Office, NAVFACENGCOM Southwest Division, and the South Bay Focus Team provided design input. William P. Young, Incorporated, was the general contractor for the project. Composite suppliers and installers included DFI Pultruded Composites, Fyfe Associates, Sika Corporation, Strongwell Corporation, and Hardcore Dupont Composites. The South Bay Focus Team managed the project to install the upgrades.

Proof tests after completion of the project demonstrated the upgraded areas could support 100-kip (450 kN) outrigger loads as the stress levels in the reinforcement remained well within service limits. The upgrade areas are suitable for 50-ton crane operations.

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WATERFRONT REPAIR AND UPGRADE ADVANCED TECHNOLOGY DEMONSTRATION SITE NO. 2: PIER 12 NAVAL STATION SAN DIEGO

INTRODUCTION

Objective

This project strengthened Pier 12 at the Naval Station San Diego to meet demands of operational changes accompanied by higher vertical loads. We added external, carbon/epoxy composite reinforcing to two areas of Pier 12. These upgraded areas will provide platforms to support outrigger loads up to 100 kip (450 kN) and a uniform load up to 750 psf (36 kPa). To accomplish this we increased the capacity of the deck slab. An upgraded area is located at each berthing of Pier 12. They are suitable for 50-ton mobile truck crane operations with 100-kip (450 kN) maximum outrigger loads.

Unsound concrete was removed and replaced on top and bottom of the deck and all major cracks were sealed on the top surface. Some corroded, steel reinforcing was also replaced by weld splicing new reinforcing bars. Abandoned conduits were removed to provide additional workspace to attach the reinforcement. Steam line hangers were removed in three spans and replaced with fiberglass/vinylester composite hangers.

This project is part of an ongoing Repair and Upgrade Program to demonstrate advanced technologies for strengthening existing Navy piers. The methodology incorporates the placement of a few ounces/ft^2 of high strength composite material in strategic locations versus adding hundreds of pounds of additional concrete and steel. The simplicity of the upgrade allows only minimal interruptions in pier operations as opposed to the pier closing for up to a year for a tradition alteration.

Scope of Work

The Naval Facilities Engineering Service Center conducted structural assessment, provided the repair and upgrade design, and selected the materials. William P. Young, Incorporated, (WPY) was the general contractor for the project. Composite suppliers and installers included DFI Pultruded Composites, Fyfe Associates, Sika Corporation, Strongwell Corporation, and Hardcore Dupont Composites.

The project included concrete repair, surface preparation, and strength upgrades for 14 spans. The spans are between pile bents 6 and 14 on the east end of the berthing and 67 and 73 on the west end. The specific project tasks included:

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Background

Pier 12 is located on the south pier area of Naval Station, San Diego, California. It is a cast-in-place, reinforced concrete structure 1,458 feet (444 m) long and 30 feet (9.1 m) wide. Pier 12 is identical in design to Piers 10 and 11 which are located to the north and was constructed from plans entitled "Fleet Reserve Berthing, Construction of Piers 10 - 11- 12 & 13," Bureau of Yards and Docks drawings numbers 409077 through 409095, 431721 and 431722, dated November 15, 1945, July 13, 1946, and December 23, 1946, respectively. Piers 10, 11, 12, and 13 were constructed in 1946 to berth the mothball fleet stationed in San Diego after World War II. They were incorporated into the Naval Inactive Ship Maintenance Facility until 1975. Pier 13 has been retired while Piers 10, 11, and 12 are currently used for berthing shallow draft ships such LPD's and LSD's (**Figure 1**).

The deck of Pier 12 spans longitudinally between pilecaps spaced at 13.5 feet (4.1 m) on center. The 30-feet (9.1 m) wide deck is comprised of a 24-inch (61 cm) thick central section containing utility tubes flanked by 8-inch (20 cm) thick slab sections on either side with edge beams (curbs) (**Figure 2**). The outside 16 inches of the 24-inch (61 cm) slab is reinforced as an internal slab beam. The central section of the slab is 17 feet (5.2m) wide while the flanking slab sections are 5-1/2 feet (1.7 m) wide and the curbs are 12 inches (30.5 cm) wide. Three vertical bearing piles and two batter piles support the pilecaps. The design strength of the concrete was 3,000 psi (20 MPa) and the reinforcing steel is ASTM grade 40.

Figure 1. Pier 12 San Diego Naval Station

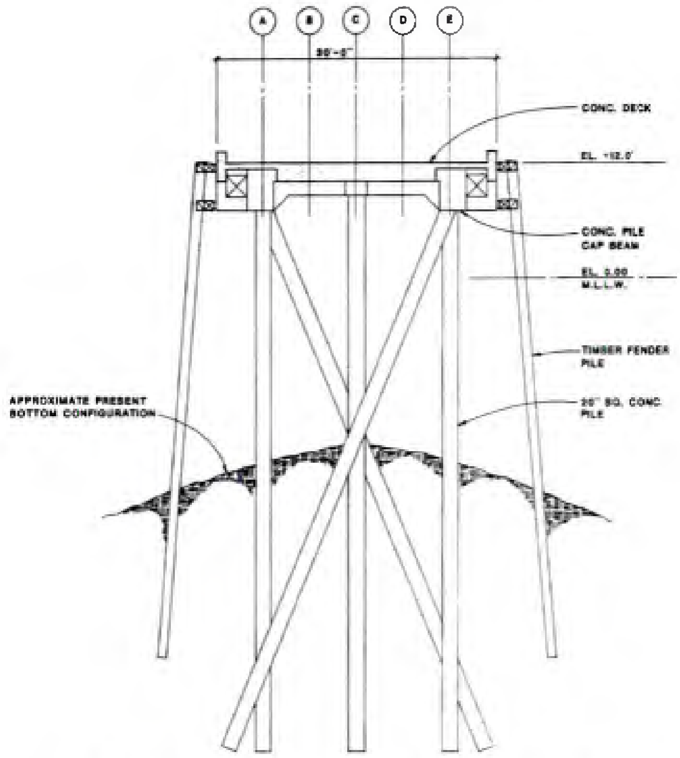


Figure 2. Typical cross section of Pier 12.

EXISTING CAPACITY AND UPGRADE REQUIREMENT

Only 30-Ton truck mounted cranes were employed on the original structure and they were limited to the pilecaps and the central deck for setting outriggers. A colored marking system has been painted on the decks to locate safe areas for crane operations. The 30-ft (9.1 m) wide deck presents a narrow working area for portable cranes and forces outrigger pads to be placed adjacent to, or onto, the 8-inch (20 cm) section of the deck (**Figure 3**). Since the LPD's and LSD's require a longer reach than can be provided by 30-ton cranes, NAVSTA Staff Civil Engineer selected Pier 12 to be upgraded for 50-ton cranes. The upgrade is to endure for at least 10 years at which time the pier is scheduled for replacement.

NFESC engineers conducted load tests and finite element model analyses to verify the current strength of Pier 12. We designed a strength upgrade that will support 50-ton cranes in the areas where outriggers are placed. The Upgrade analysis and design methodology is presented in **Appendix A**. Lanes to support crane wheel loads and outriggers on each side of the deck were strengthened to serve 50-ton cranes or a uniform load of 750 psf (36 kPa). The maximum crane loads are 100,000 lbs (450 kN) on 24-inch (60-cm) square outrigger floats.

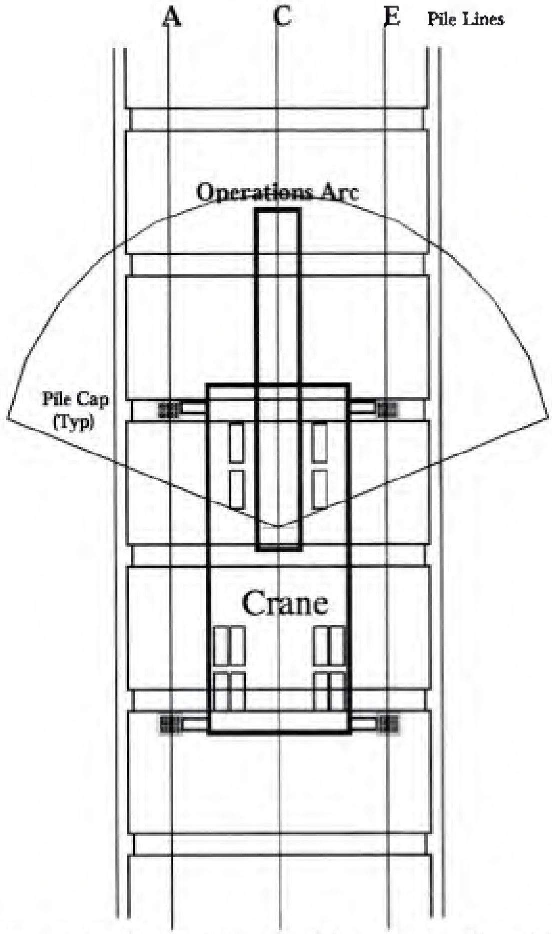


Figure 3. Positioning portable crane on Pier 12.

The pier deck is minimally reinforced (**Figure 4**). The 8-inch (20-cm) deck section is reinforced with steel areas of 0.13 in^2/ft (2.8 cm^2/m) and 0.24 in^2/ft (5.1 cm^2/m) in the lateral direction on the top and bottom respectively and 0.11 in^2/ft (2.3 cm^2/m) and 0.15 in^2/ft (3.2 cm^2/m) respectively in the longitudinal direction. The 24-inch slab (61-cm) is reinforced longitudinally at midspan with 0.27 in^2/ft (5.7 cm^2/m) and 0. 13 in^2/ft (2.8 cm^2/m) laterally. At the pilecap the 24-inch (61-cm) slab has 0. 53 in^2/ft (11.2 cm^2/m) negative reinforcement and 0.13 in^2/ft (2.8 cm^2/m) laterally. The pilecaps are also lightly reinforced with 2.21 in^2 (14.26 cm^2). The complex shape and large depth-to-span ratio of the pilecaps make them difficult to analyze by tradition methods. Finite element analyses showed that the pilecaps will fail in shear before bending because of their relative large depth to span ratio. The failure mode of outrigger loads is punching through the pilecap between the piles. Adding upgrade reinforcement to the deck was attractive because the steel ratios are so low. **Table 1** contains the original strength of the deck cross sections and the upgraded strength provided by additional composite reinforcing. The upgrade strengths were determined by finite element analyses to be sufficient to meet 50-ton crane requirements. In addition to the composite laminate on the bottom surface, fiberglass/vinylester I-beams were bonded to the bottom of 8-inch deck to provide an additional 2,400 in-kip to the flexural capacity.

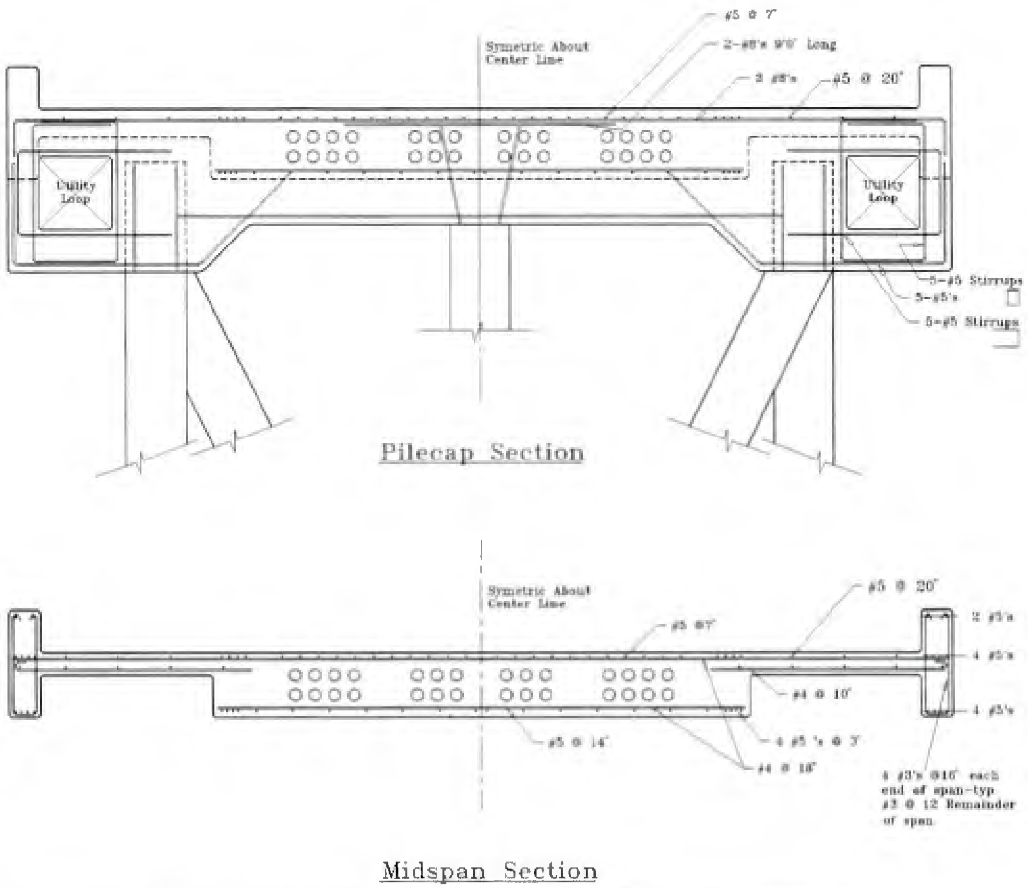


Figure 4. Cross-sections through the pilecap (top) and deck of Pier 12 showing reinforcing steel.

Table 1. Flexural Capacities of Pier 12 Deck Sections

Section	*Moment location*	*Current in-kips/ft (kN-m/m)*	*Upgraded In-kip/ft (kN-m/m)*	*% Increase*
24-inch (61 cm) deck (top)	pilecap	670 (250)	2,905 (1,085)	333
24-inch (61 cm) deck (bottom)	midspan	345 (130)	3,005 (1,120)	770
8-inch (20 cm) deck (top)	pilecap	50 (19)	420 (155)	740
8-inch (20 cm) deck (bottom)	midspan	72 (27)	480 (180)	566

The punching shear strength of the 8-inch deck is more than 200kips (900 kN) for a 24-inch 61 cm) outrigger pad.

SITE INVESTIGATION AND CONDITION ASSESSMENT

The site investigation and condition assessment included that part of Pier 12 to be upgraded (**Figure 5**). The 50-year old concrete has areas of degradation marked by spalling, efflorescence, and chloride contamination. There is ongoing rebar corrosion, especially along the joints, cracks, and where the concrete cover over the rebar is insufficient.

Previous inspections and condition assessments of Pier 12 were conducted in 1994, 1995 and late 1996. The first by Suboceanic Consultants, Inc., concentrated on the piles and pilecaps, the second by NFESC assessed the load carrying capacity of the deck, and the third by NFESC materials engineers focused on the state of the concrete. The Suboceanic study (NFESC Report No. 55-95(055) of June, 1995) concluded that cracking and spalling concrete in some of the piles, pilecaps, edge beams and utility loops has "not significantly" reduced the structural capacity of these elements. They continued by stating that the deterioration was progressive and would eventually lead to structural problems after some unspecified time. This report was immediately followed by a load capacity and repair study by Blaylock Engineering Group leading to design drawings for repair of Pier 12. This repair project has not been undertaken. The repair project does not address the load limitations, which are severe:

3,100 lbs (13.8 kN) for crane wheel loads,
5,950 lbs (26.4 kN) for outrigger loads,
300 psf (14.4 kN/m^2) uniform live load on the 8-inch (20 cm) slab,
14,293 lbs (63.5 kN) for crane wheel or outrigger loads,
400 psf (19.1 kN/m^2) on the 24-inch (60 cm) slab.

A load test and evaluation of Pier 12 was conducted in January 1995 and reported in NFESC report SSR-2132-SHR. The report concluded, while Pier 12 was of generally sound construction, the 8-inch (20-cm) deck cannot support portable cranes. The reason for load limitations is the deck seriously lacks flexural reinforcing. A follow-up series of impact load tests to the 1995 study was conducted in September 1996, on the areas of bents 7 through 18 and 67 through 85. The purpose of the latter tests was to obtain a well-defined structural response pattern of the proposed upgrade area. From the structural response we also searched for anomalous behavior indicative of corroded reinforcement and loss of concrete strength. The responses of the pilecaps were very solid and essentially equivalent. The tests did not reveal any major mechanical damage in the decks of the upgrade areas.

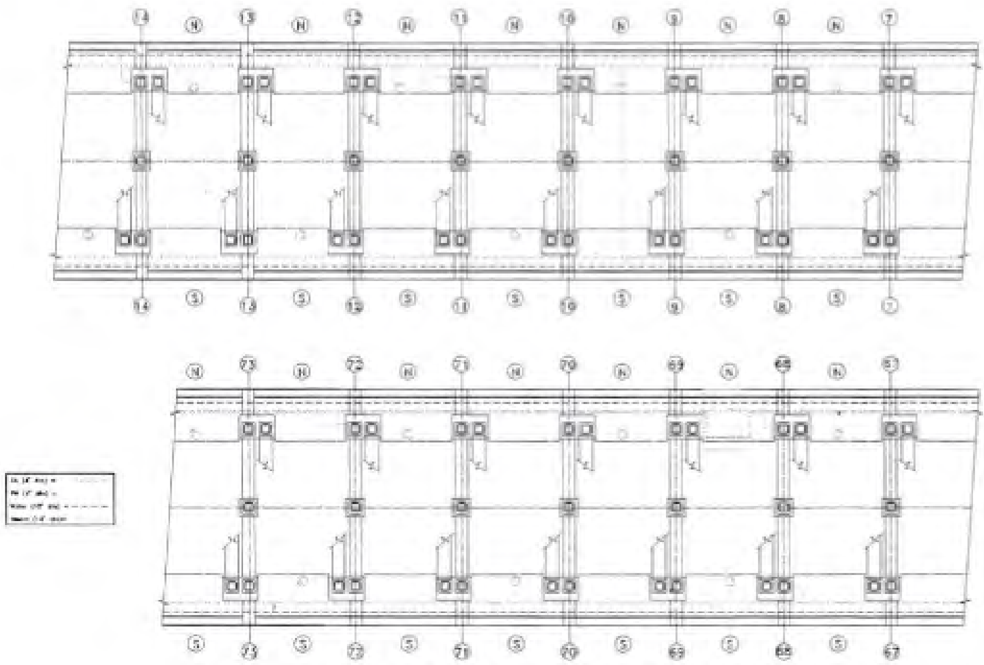


Figure 5. Upgrade locations on Pier 12.

NFESC materials engineers conducted visual inspections of the underside and topside of Pier 12 in late 1996 and early 1997. Overall the concrete in the pier deck was in very good condition given its age and service demands. Cracking and spalling were found in many of the piles between the low waterline and the pilecap. The tops of piles also exhibited similar, but less, damage. The utility loops on the ends of the pilecaps and the bottom of the pilecaps exhibit spalling and cracking particularly around previous shotcrete repair areas. Cracks and spalls were caused by steel reinforcement corrosion. Considerably less corrosion was found in the deck slabs. The corrosion on the underside of the deck appears to be the result of insufficient reinforcement cover while that in the piles and pilecaps is the result of chloride ion migration. The undersides of the decks were marked with efflorescence and rust stains around some cracks and rust stains around tie wires that protrude through the concrete. Transverse cracks appeared at midspan of many of the spans and well-defined form marks (fins) traverse the under side of the deck at 4 to 6-inch (10 to 15 cm) spaces (**Figure 6**).

NFESC materials engineers mapped the cracks in the pier deck and measured the deck reinforcing steel depths and measured the chloride content of the concrete (NFESC Special Report SP-2022-SHR). In addition, cores were taken from the top deck for petrographic analysis. Powder samples were taken from the top and bottom deck to measure the degree of chloride contamination at the depth of the rebar. A pachometer was used to measure the depth of concrete cover over the rebar. The surfaces were sounded for delaminations. Moisture vapor emission tests were conducted and surface adhesion tests were performed.

Twenty-three, 0.75-inch (2 cm) diameter holes were drilled to obtain powder samples for chloride ion contamination measurements. The chloride ion contamination at the depth of the rebar (1.5 inches)(4 cm) averaged 1.6 pounds per cubic yard (0.55 kg/m^3) for the top surface of the deck and 1.2 pounds per cubic yard (0.42 kg/m^3) for the bottom surface.

An Elcometer portable "pull-off" adhesion tester was used to determine the deck concrete's tensile strength of spans between pilecaps 10 and 11 and between 69 and 70 using ASTM D 4541 (Standard Test Method for Pull-Off Strength of Coatings Using Portable Adhesion Testers). Three pull-off tests were performed in each span with values ranging from 310 psi (2,140 kPa) to 400 psi (2,760 kPa). Standard concrete in good condition will exhibit tensile strengths between 300 psi (2,070 kPa) to 500 psi (3,450 kPa). The tensile strength results in the areas tested were indicative of sound concrete.

A Schmidt Hammer (ASTM C 805 Standard Test for Rebound Number of Hardened Concrete) was used to determine the deck concrete's approximate compressive strength of the span between pilecaps 69 and 70. Thirty rebounds were taken to reveal compressive strengths ranging from 3,400 psi (23.4 Mpa) to 6,500 psi (44.8 Mpa) with an average reading of 5,100 psi (35.2 Mpa). The compressive strength results in the areas tested were indicative of sound concrete.

Oil and chemicals on the concrete surface can prevent laminate adhesion. In one area of a known oil spill, a 2-inch (5 cm) diameter by 2-inch (5 cm) long concrete core was extracted to visually reveal hydrocarbon (grease and oil) penetration to a maximum depth of 1/16 inch (2 mm) from the top surface. The concrete's water-to-cement (w/c) ratio was moderately high (0.55 to 0.65) and the depth of carbonation on the top of the deck was 0.2 to 0.7 inch (5 to 18 mm). The cement paste was moderately soft.

Concrete vapor emission can impede the adhesion of laminate to the deck surface. Eight vapor emission test kits were placed on the bottom surface of the deck for a maximum of 43 hours. Vapor emissions between 3.3 and 4.0 lbs/1,000 ft^2 (16 and 20 gm/m^2) were obtained from the bottom surface. Six vapor emission test kits were placed on the top surface of the deck for a maximum of 43 hours. Vapor emissions between 1.8 and 3.0 lbs/1,000 ft^2 (8 and 15 gm/m^2) were obtained from the top surface.

Steel reinforcing cover was measured to be less than 1-inch (2 cm) to over 3 inches (8 cm).

Batter piles are immediately adjacent to vertical piles (**Figure 7**). Many of these have spalled and delaminated concrete. However, in the areas of upgrade, only the batter pile on the north side of Bent 7 and the batter pile on the north side of Bent 72 have documented delaminations (near the high water mark). Many others have shotcrete applied at the intersection with the pilecap.

Figure 6. Deck underside at the thickness transition showing form marks (fins).

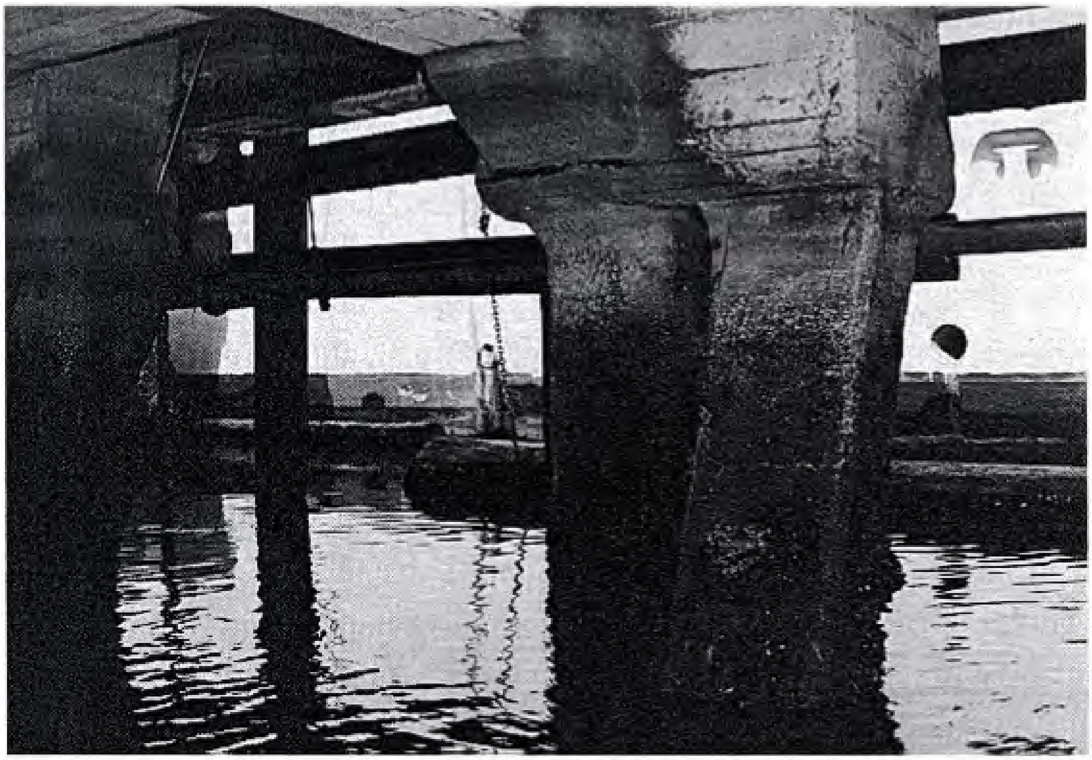


Figure 7. Marine fouling on vertical and batter piles under Pier 12.

UPGRADE DESCRIPTION

NFESC selected the following upgrade methodologies:

Bottom surface of 24-inch (61 cm) deck section:	**Uniaxial carbon fiber sheets with in-situ field (wet) impregnation resin saturator**
Bottom surface of 8-inch (20 cm) deck sections:	**epoxy-bonded, high strength carbon/epoxy pultruded unidirectional laminate strips**
	epoxy-bonded, 12-inch (30.5 cm) deep, pultruded, fiberglass reinforced, vinyl ester structural I-sections anchored on each end to the pilecaps
Top surface of deck:	**3/8-inch (9.5 mm) diameter, high strength, carbon rods embedded with epoxy into slots cut into the concrete**
Batter pile confinement reinforcing:	**Pre-formed cylindrical shells of fiberglass in a vinyl ester matrix filled with shrink resistant grout.**

Upgrade Design

Schematics of the deck upgrade designs are shown in **Figures 8 through 14**. **Figures 8 through 11** show nominal layouts of post strengthening reinforcement of the deck. The external reinforcement was custom fit to each span because the span dimensions varied and contained utilities, small vaults, bollard bases, and drain holes. The as built drawings (for each span) are provided in **Appendix A**. The design required an external reinforcing upgrade equivalent to 16 kips/inch (28 kN/cm) longitudinally on the bottom of the 24-inch (60 cm) deck section **(Figure 8 and 11)**. The design required an upgrade equivalent to 16 kips/inch (28 kN/cm) longitudinally to the bottom of the 8-inch (20 cm) deck section plus the support of two lengths of 12-inch composite I-beams (structural I-sections) to each side of the pier (**Figures 8, 9, and 10**). The I-sections were attached to the pilecaps or the bollard bases with stainless steel anchor bolts **(Figures 12 and 13**). The I-sections were field cut to length to fit precisely across each span at specified locations. The design required additional longitudinal reinforcement on the top surface of the deck equivalent to 16 kips/inch (28 kN/cm) over each of 14 pilecaps **(Figure 15)**. Twenty vertical and batter piles in the bents listed above were selected to be reinforced between the low tide waterline and the pilecaps to increase strength and ductility (**Figure 14**). These included piles in bents 8 through 13 and bents 68 through 71.

Design Parameters

Material design limits were established by NFESC based on finite element analyses (FEA) of the pier responding to outrigger loads of 100,000 pounds (**Appendix B**). Composite materials were selected that would minimize crack growth to protect existing reinforcing steel from corrosion. NFESC engineers set the maximum service (working) strain and a minimum service stress for carbon fibers in an epoxy matrix. Composite systems were selected after performance evaluation of laboratory tests on candidate materials and half scale system load tests at the Advanced Waterfront Technology Test Site (AWTTS) in Port Hueneme (**Appendix B**).

Upgrade reinforcing laminate material areas on the bottom of the deck (**Figures 8 and 10**) were based on laminate of high strength carbon fibers with a minimum service stress of 250 ksi at a strain of 0.83 percent and a minimum ultimate strength of 450 ksi. Carbon fibers provide a high service stress limit with sufficient stiffness to arrest crack growth in the original concrete. Laboratory tests of the selected materials verified the service stress level to be a practical and achievable (**Appendix B**). The reinforcing materials on the top of the deck (**Figures 8 and 11**) are based on high strength carbon/epoxy pultruded rods with a carbon fiber minimum service stress of 200 ksi at a strain of 0.67 percent and a minimum ultimate strength of 350 ksi. Pile confinement (circumferential) reinforcement was designed to develop an equivalent of 2 kips/inch at a strain of 0.2 percent and an ultimate strength of 4 kips/inch. All upgrade reinforcing materials were required to be compatible with ordinary concrete and polymer concrete with 12.5 pH.

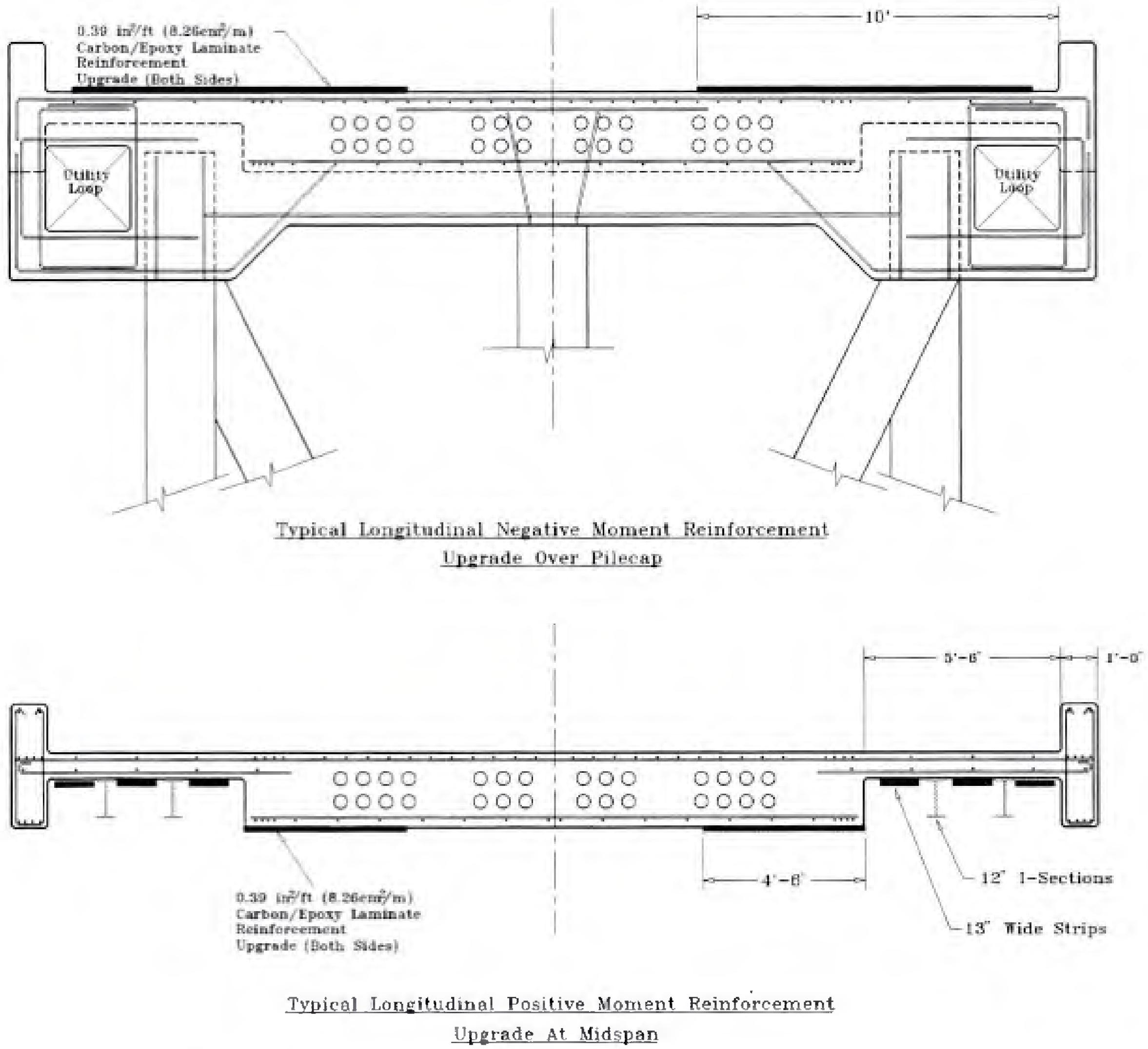


Figure 8. Typical post strengthened cross sections of carbon laminate and composite I-sections.

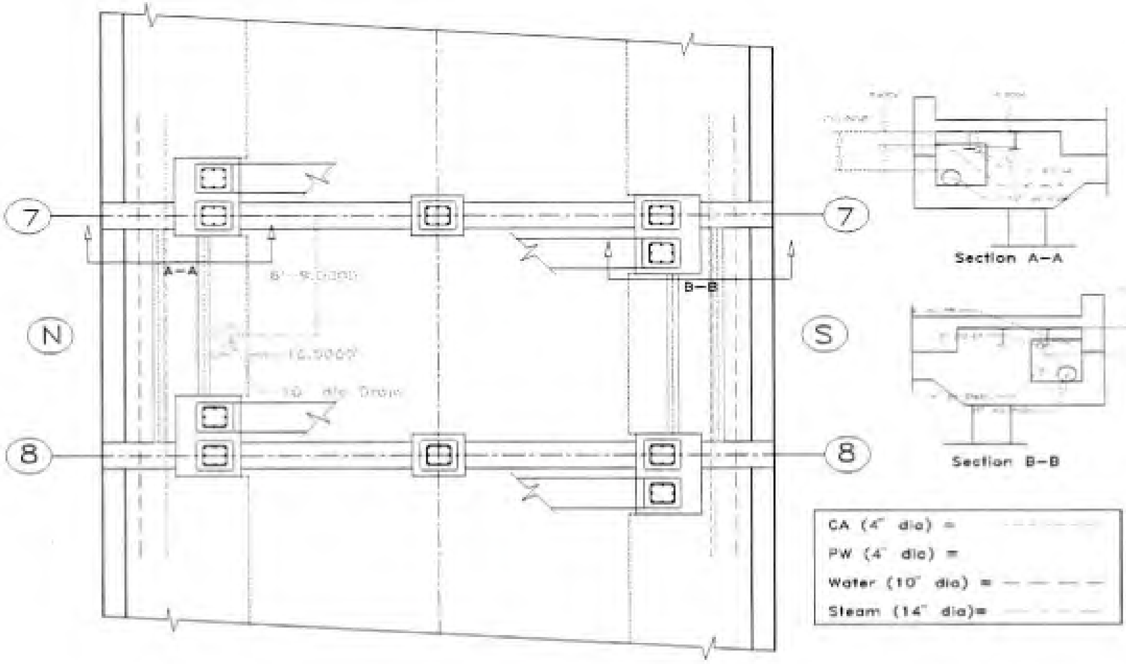


Figure 9. Nominal locations of Structural I-sections supporting 8-inch deck section.

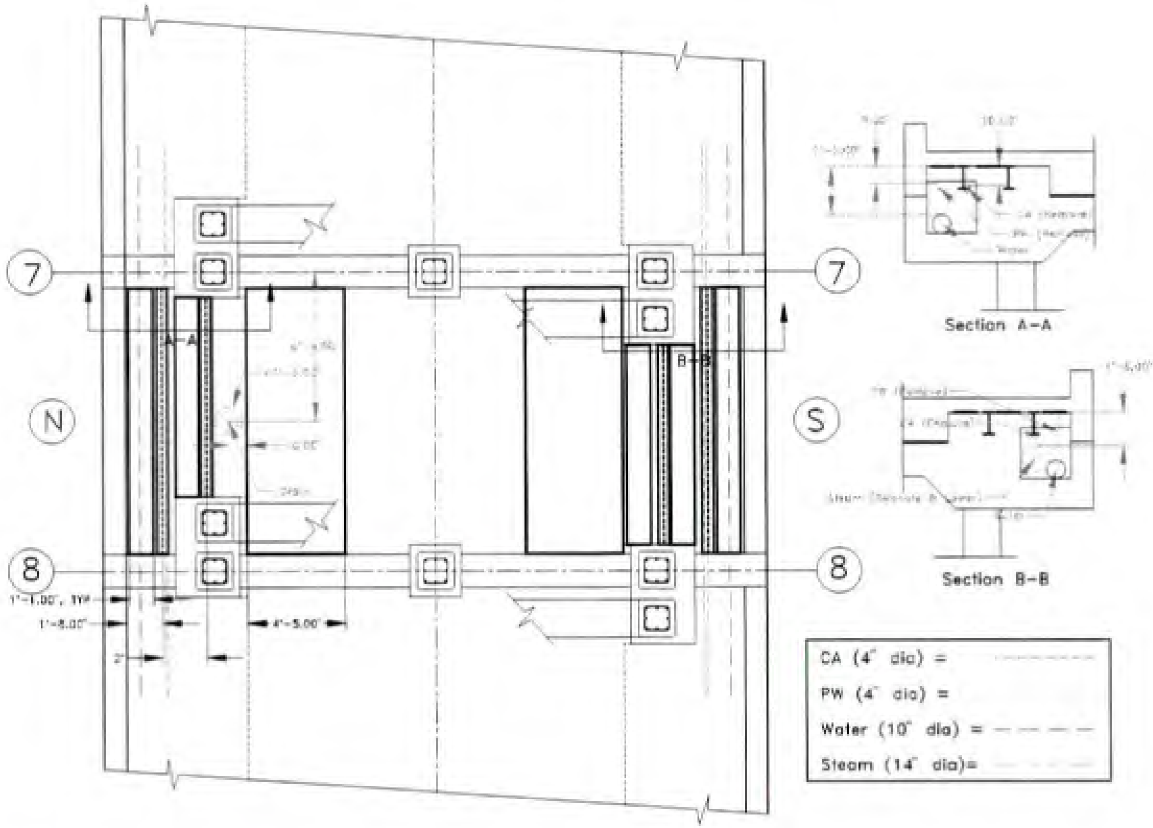


Figure 10. Plan of typical underside deck reinforcement – structural I-sections and carbon laminate.

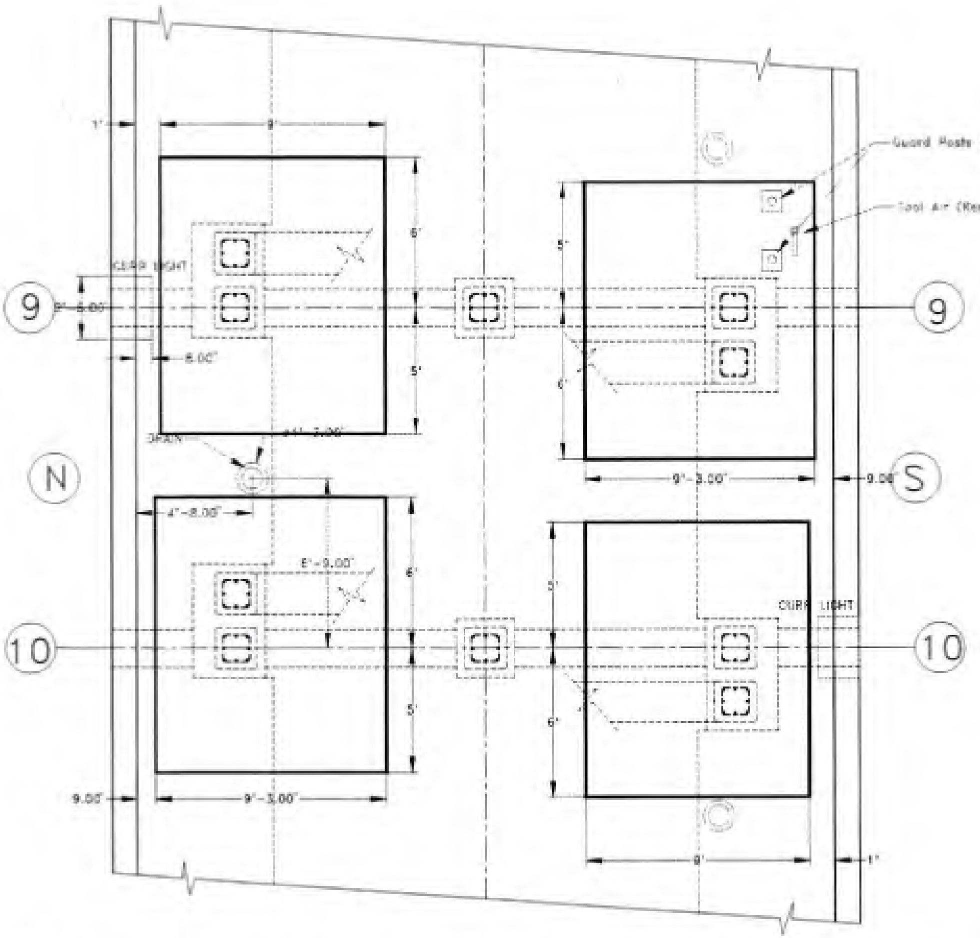


Figure 11. Top deck surface longitudinal reinforcement areas for embedded carbon rods.

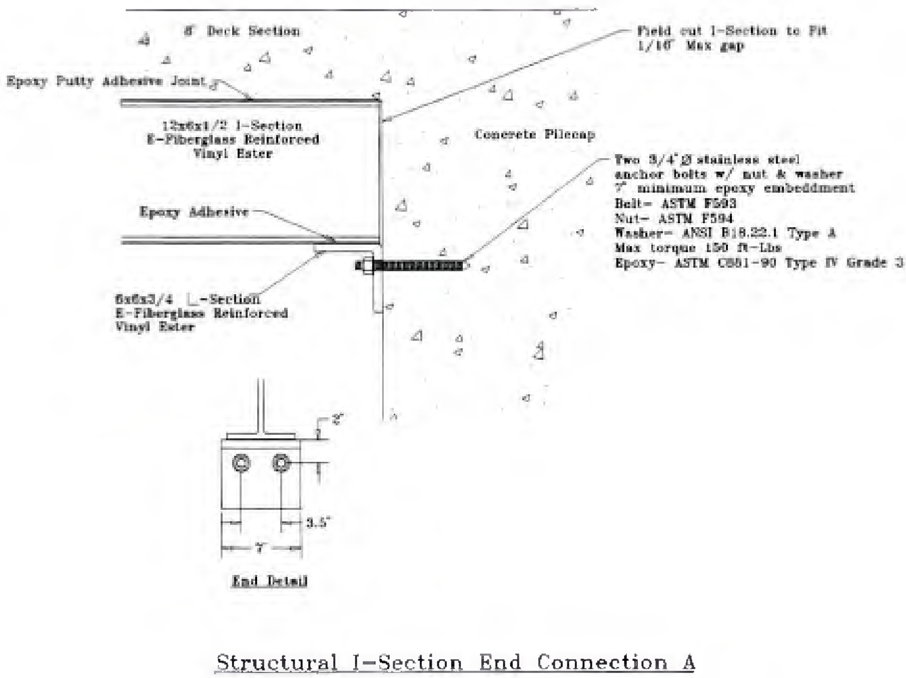


Figure 12. Pilecap connection Detail "A" at the ends of fiberglass composite I-section.

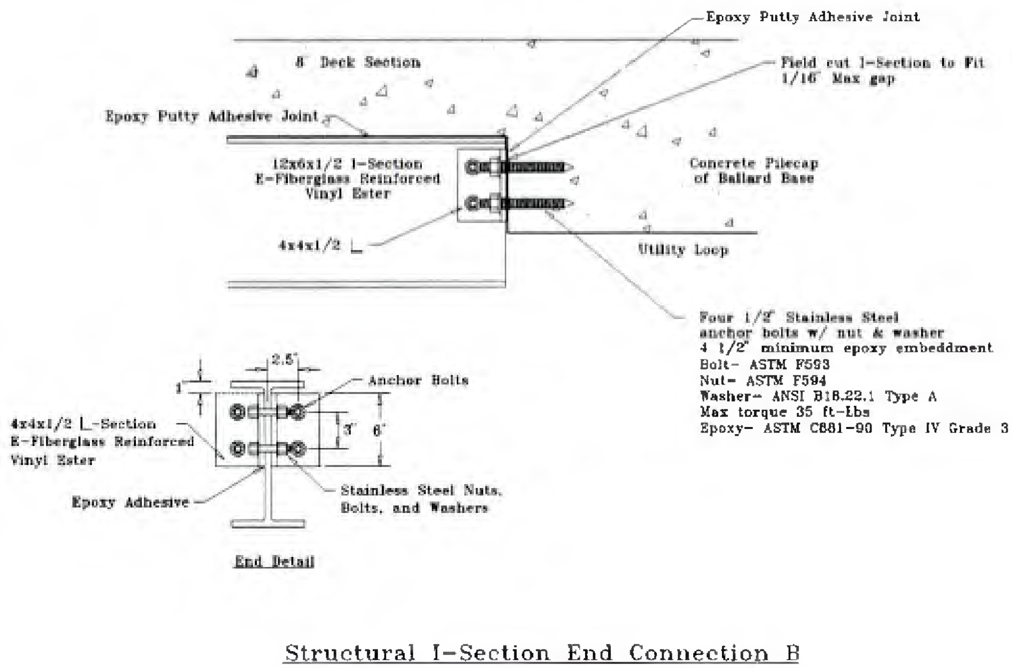


Figure 13. Pilecap of bollard base connection "B" at the ends of fiberglass composite I-section.

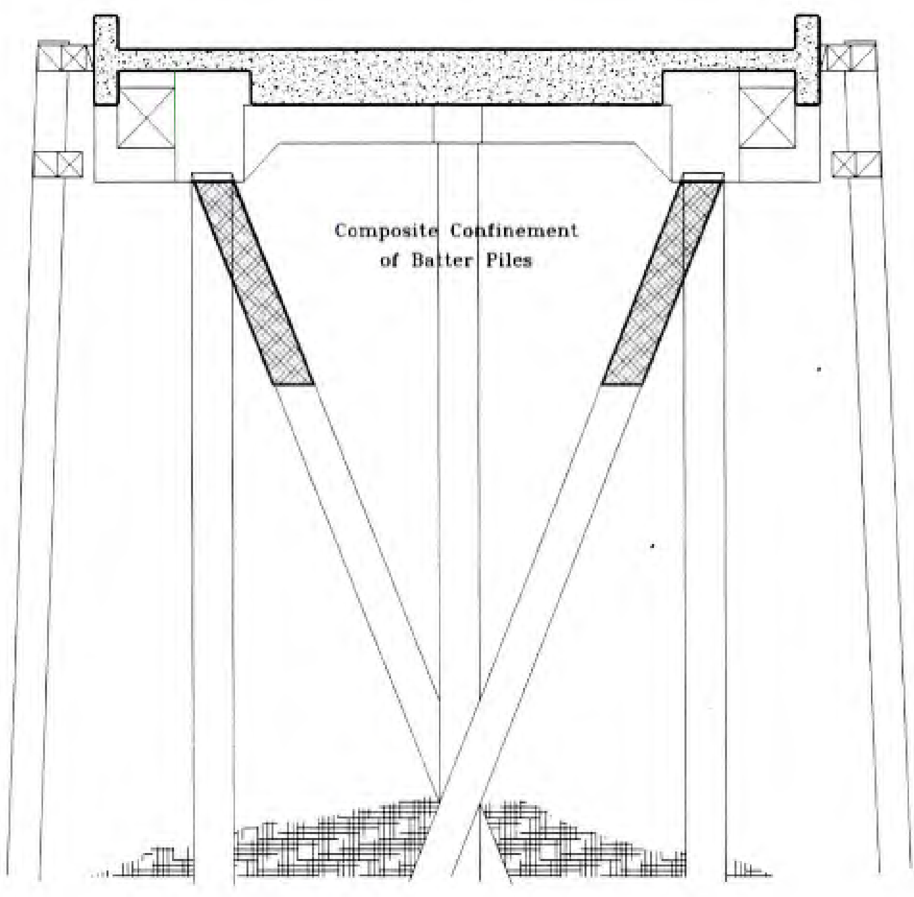


Figure 14. Schematic of batter pile upgrade.

CONCRETE REPAIR AND PREPARATION

Preparation of the concrete is extremely important to insure that the concrete develops a strong bond with the epoxy adhesive or the carbon graphite laminate. The concrete must be sound and undamaged or the added reinforcement will not be effective. Concrete replacement and repair was completed prior to initiating the upgrade procedures in order that the composite reinforcement did not cover a developing problem. All concrete restoration was conducted in accordance with the specifications in **Appendix C**. This damage included: unsound and loose material, impact damage, defects in the formed surfaces, chloride contamination, efflorescence, spalling, drilled and cut holes, and delaminations. An average of 4.5 square feet (0.4 m^2) of deck area required repair in each span.

In preparation for repair and upgrade scaffolding was hung under the deck in the areas of construction. We removed several abandoned utility lines to clear areas for upgrade repair and upgrade. These utility lines are located under the 8-inch (20 cm) deck section running through "utility loops" that are formed in the pilecaps outside the piles (**Figure 8**). An abandoned 4-inch (10 cm) diameter "air" line and an abandoned 4-inch (10 cm) diameter "water" pipe with attached pipe hangers (**Figure 15)** at the top of the utility loop were removed and pipe hanger holes were repaired. The two abandoned 3-inch (8 cm) diameter electrical conduits that exit the "telephone pull box" in the north side of the spans between bents 11 and 12 and between bents 73 and 74 were also removed.

Figure 15. Utility hangers anchored to Pier 12 deck.

Utility Hangers. The 14-inch (36 cm) outside diameter steam pipeline near the bottom of the utility loop crosses under the deck from the north side to the south at midspan between pile bents 9 and 10. The steam pipeline consists of an inner steel carrier pipe surrounded by an insulating fiberglass conduit. As it crosses under the deck it passes within ½-inch (1.3 cm) of the 24-inch (61 cm) deck section. WPY removed the existing hangers on this line at 5 locations in the spans between pile bents 8 and 11. WPY rehung the steam line with fiberglass reinforced vinyl ester composite pipe hangers after the upgrade installation (**Figures 16 and 17**). The clevis hanger was used at six locations between bents 8 and 11. It is a hand lay up laminate of E fiberglass in a vinyl ester matrix from Bedford Reinforced Plastics, Inc. The laminate cross section is ¼-inch (6 mm) thick and 5 inches (13 cm) wide and has ultimate tensile strength of 12,600 psi (87 Mpa). The clevis hanger has a design load of 600 pounds (2.67 kN). The second hanger type was site-constructed for the center of Span 9-10 only. It was made of 6-inch x 6-inch x ½-inch (15.2 cm x 15.2 cm x 1.3 cm) fiberglass structural angle from Strongwell with the same properties as the I-beams that were installed to support the 8-inch (20 cm) deck section. Both hanger types used 5/8-inch (1.6 cm) diameter, threaded, fiberglass/vinyl ester rods anchored in the deck. These rods have a measured ultimate strength of over 3,000 lbs (13.3 kN) and should be suitable for a design load of 600 lbs (2.7 kN). The rods were embedded 4inches in the deck and anchored using a 5000-psi (35 Mpa) strength epoxy anchoring paste (Adhesive Technology ANCHOR-IT® HS-200).

Figure 16. Fiberglass composite angle hanger at midspan between Bents 9 and 10.

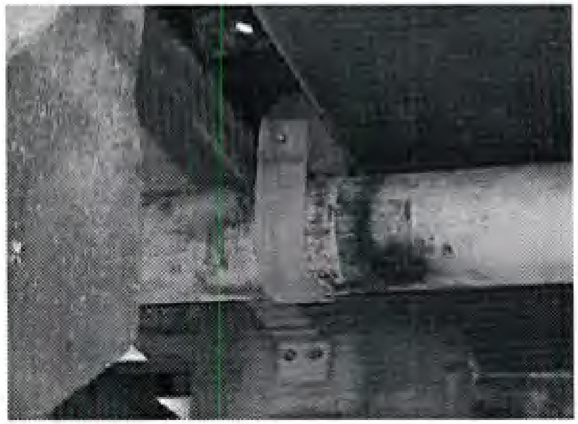


Figure 17. Fiberglass composite clevis hanger.

Crack Sealing. Cracks in the top surface of the deck were sealed to better protect the upgrade reinforcement from moisture passing through the deck. Cracks less than 1/16-inch (2-mm) wide were pressure injected with WEBAC 157®, a hydrophobic grout based on a methylene-diphenyl-isocyanate polyurethane (**Figure 18**). The resin is solvent free and 100 percent solids. The 157 resin forms flexible closed-cell foam while expanding its volume up to 15 times. Moisture or dryness should not shrink or swell the cured 157. Cracks greater than 1/16-inch (2-mm) width and 2 feet (0.6 m) long was filled with a 2-part flexible urethane polymer joint sealer (Sikaflex 2C NS®) after concrete repair was completed. These cracks were saw cut to ½inch (1.3 cm) wide by 1-inch (2.5 cm) deep. The crack surfaces were coated with Sika's penetrating urethane primer (**Figure 19**). A backer rod was installed to contain the crack filler (**Figure 20**) before filling the crack with polyurethane (**Figure 21**). Most of the cracks were "working cracks, so we used flexible, polyurethane sealers to avoid recracking a brittle crack sealant material such as epoxy.

Figure 18. Injecting polyurethane into cracks on deck surface.

Figure 19. Applying polyurethane primer to construction joint crack.

Figure 20. Installing backer rod to seal crack.

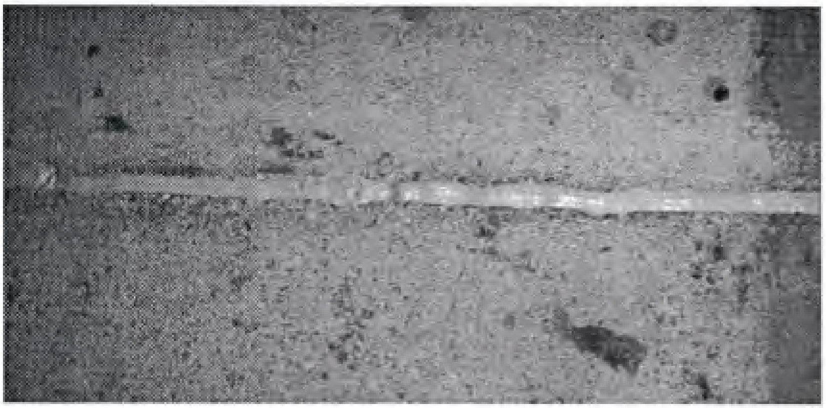


Figure 21. Polyurethane crack sealant.

Damaged, deteriorated, or contaminated concrete was removed by saw cutting the perimeter of the repair area (**Figure 22**) and breaking out the concrete material with a 15-lb air driven hammer (**Figure 23**). Only small tools were allowed to avoid damaging remaining concrete. We used prepackaged shrink-resistant, polymer concrete grouts (SikaTop® 111 on the top of the deck and SikaTop® 123 on the bottom) as concrete repair material. SikaTop 111 is a polymer-modified, portland cement, two-part, fast-setting mortar. SikaTop 123 is similar to 111 but is "non-sag" mortar that will allow its application overhead (bottom surface of the deck slab). The repair material was placed by hand and compacted with small hand tools (**Figures 24 and 25**). The repair material was cured by placing wet carpeting over the patched area for 24 hours (**Figure 26**).

Figure 22. Saw cutting perimeter of concrete repair area.

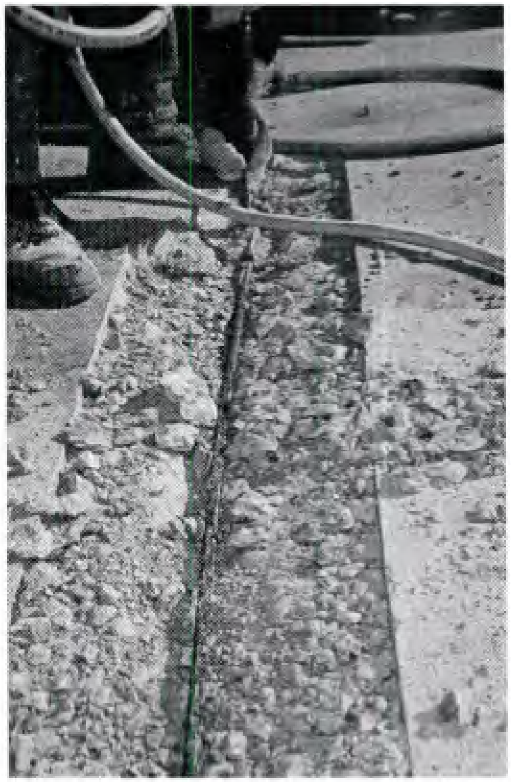


Figure 23. Removing deteriorated concrete with small pneumatic hammer.

Figure 24. Hand packing concrete repair material.

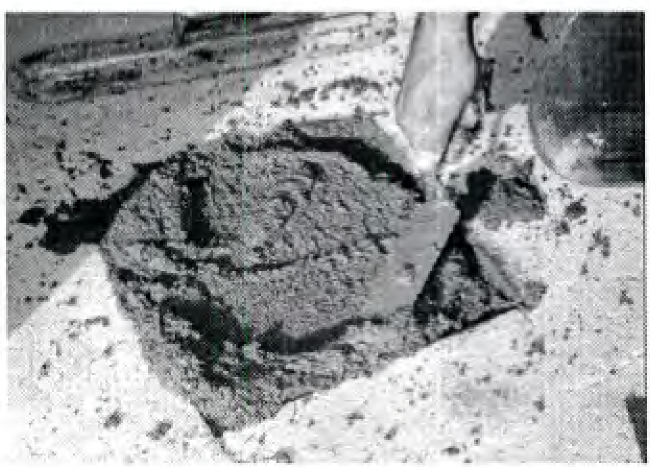


Figure 25. Concrete repair with small hand tools.

Figure 26. Wet curing concrete repair area.

Material Testing. On-site, beam and cylinder test samples of the concrete repair materials were laboratory tested. Table 2 contains test results. Except for one small noncompliance for splitting tensile strength, the properties of the concrete repair material exceeded our specifications. The concrete repair materials were considered satisfactory.

Table 2. Concrete Repair Material Test Results.

Material	Deck Location	Specification	Type of Test	Age	Results	Specified
SikaTop 111	Top	ASTM C157 (Modified)*	Shrinkage	28 days	0.032%	0.05%
SikaTop 123	Bottom	ASTM C157 (Modified)*	Shrinkage	28 days	0.037%	0.05%
Sika Top 111	Top	ASTM C496	Splitting Tensile	28 days	640 psi (4410 kPa)	650 psi (4450 kPa)
Sika Top 123	Bottom	ASTM C496	Splitting Tensile	28 days	800 psi (5520 kPa)	650 psi (4450 kPa)
Sika Top 111	Top	ASTM C109	Compression	28 days	5880 psi (40.5 Mpa)	4000 psi (28 Mpa)
Sika Top 123	Bottom	ASTM C109	Compression	28 days	7650 psi (52.7 Mpa)	4000 psi (28 Mpa)
Sika Top 111	Top	ASTM C882	Bond	28 days	1780 psi (12.3 Mpa)	1200 psi (8.3 Mpa)
Sika Top 123	Bottom	ASTM C882	Bond	28 days	1250 psi (8. 6 Mpa)	1200 psi (8.3 Mpa)

*Modified in accordance with "Supplemental Recommendations for Control of Shrinkage of Concrete" published May 1979 by the Structural Engineers Association of California (SEAOC) to exclude wet curing.

Surface Preparation for Upgrade Reinforcement

After completing crack sealing and concrete repair, the concrete surfaces were prepared to receive the upgrade reinforcement. Concrete fins were removed from the deck bottom surface and ground smooth. All surfaces were cleaned of loose material and sand blasted to remove laitance. Except for 24-inch (61-cm) deck sections, the surfaces were covered with a very low viscosity (less than 100 cps) epoxy penetrating sealer/primer (Sikadur® 55 was used on bottom and Madewell® 927 was used on the top surface). Both of these primers provided excellent penetration into microcracks on the surface of the existing concrete. Both are two-component, 100 percent solids epoxies for porous substrates that have a unique chemistry to displace moisture and increase the tensile strength of the substrates. Some installers do not normally apply a very low viscosity epoxy sealant/penetrant to the concrete surface. A very low viscosity epoxy penetrant was not part of the carbon laminate system bonded to the bottom surface of the 24-inch (61-cm) deck section.

NFESC outdoor application tests of the epoxy primer revealed exposure to the sun with the inherent higher concrete surface temperatures caused evaporation and expanding air within the pores of the concrete. The expanding air outgases through the epoxy primer and causes bubbling and pinholing on the surface. After the primer seals the concrete, outgasing is not a problem for application of additional layers of epoxy. Therefore, primer was applied to the top deck in the afternoon after the concrete had reached maximum surface temperature.

QUALITY ASSURANCE TESTING

NFESC implemented three procedures for maintaining control of the quality of materials and construction methodology. (1) We required submission of 1 percent of all materials to be to NFESC engineers for test and evaluation. (2) WPY was also responsible for testing on the site and by independent laboratories. (3) NFESC project engineers were on the site continuously during construction to monitor procedures and progress.

All submitted composite materials were tested in the laboratory at NFESC. Laboratory tests included coupon uniaxial tensile tests to failure, bond tests, and environmental testing. The results of these tests are presented in the body of this report and in **Appendix B** and were compared to manufacturer's claims and the Specifications. WPY also submitted concrete repair material samples and laminate samples to independent laboratories for testing.

Pull-Off Bond Tests

All the systems adhering composite laminates to the concrete deck surface were required to demonstrate that the laminate-to-concrete interface was sound at the site. On site testing included "pull-off" bond tests (**Figure 27**) and thermographic surveys of the laminate. The thermographic results were negative (a few insignificant flaws were found). The test results of the pull-off tests are in **Appendix B**. WPY conducted the "pull-off" tests on epoxy saturate systems, epoxy putty or grout systems, and other concrete bonding materials to demonstrate the bonding strength. These were conducted in accordance with ASTM D4541 and ISO 4624. The laminate-to-concrete bond was required to develop at least 300 psi (2.1 MPa) and develop the tensile capacity of the concrete.

The pull-off tests resulted in failure at an excess of 300 psi (2.1 Mpa). This insures concrete-to-laminate shear strength of more than 1,100 psi (7.7 Mpa) that is sufficient to develop the tensile strength of the external reinforcing before debonding (**Figure 28**).

Our pull off tests on Pier 12 and in the laboratory showed the low viscosity (100-cps viscosity) penetrating sealer/primer epoxy significantly increased the tensile and impact strength of the first 1/8-inch (3 mm) layer of concrete. The increase in tensile strength is due to epoxy primer being absorbed into the microcracks of old concrete. This is significant for aging concrete marine structures. The pull-off failure was transferred deeper into the substrate where the concrete is more sound (**Figure 29**). NFESC considers application of a very low viscosity (100 cps) penetrating epoxy sealant/primer necessary on the porous concrete surfaces of the Navy's aged piers.

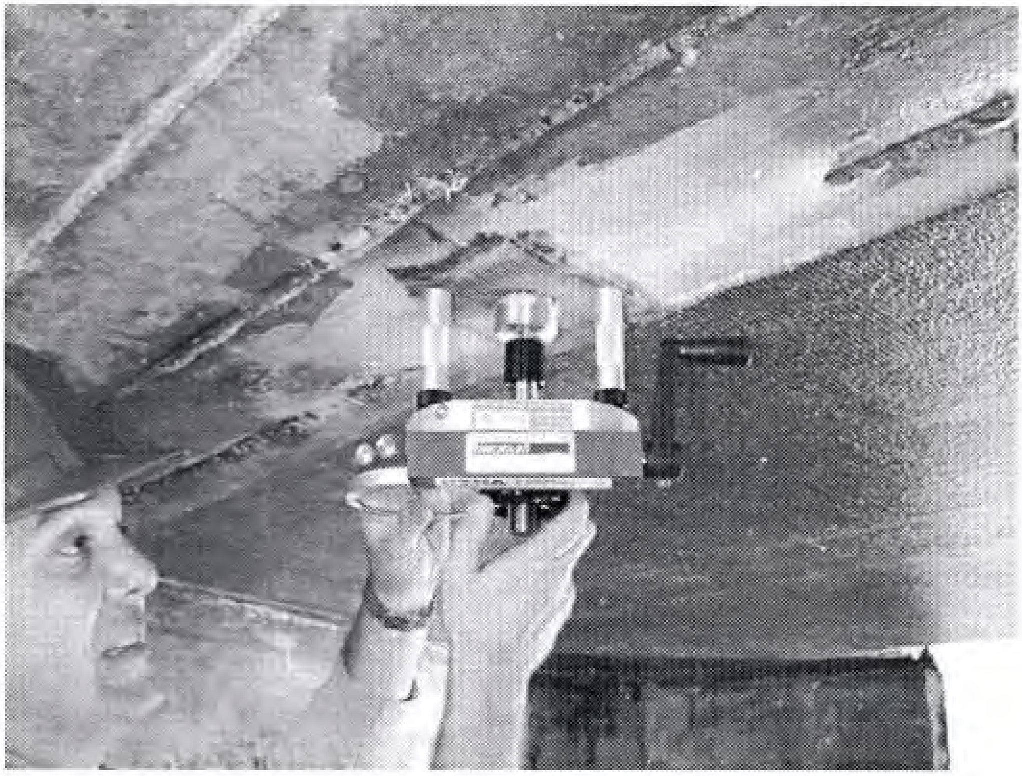


Figure 27. Bond testing with pull-off tester.

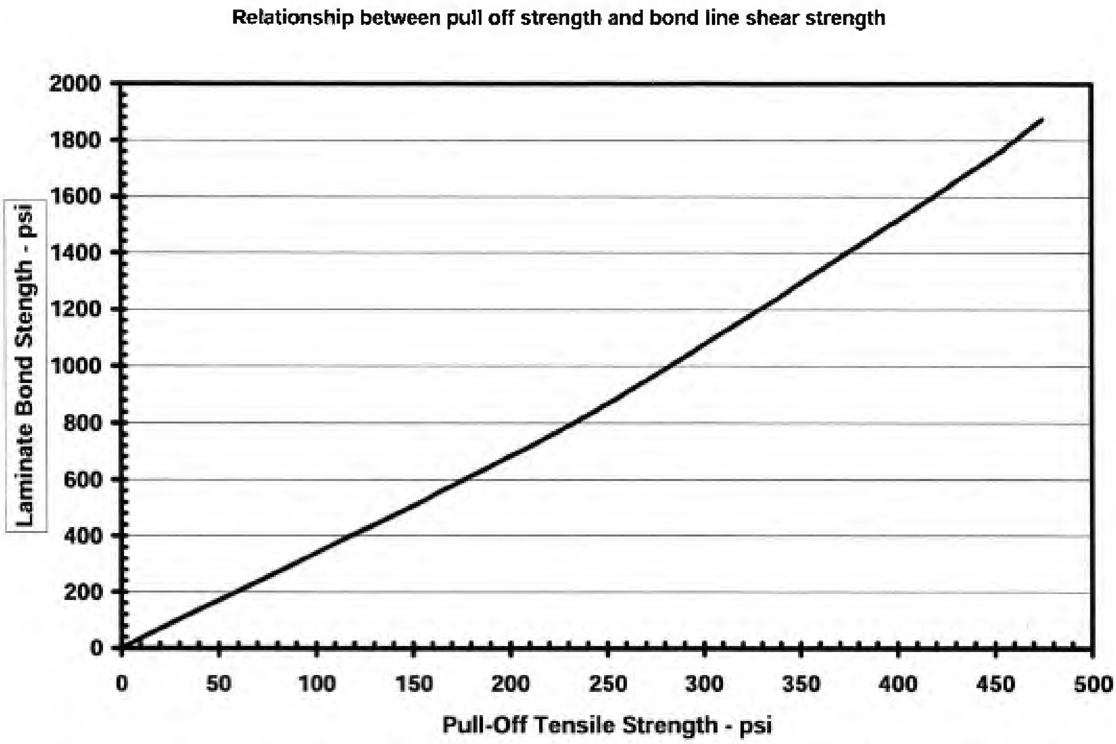


Figure 28. Relationship between pull-off tensile strength and laminate bond line shear strength.

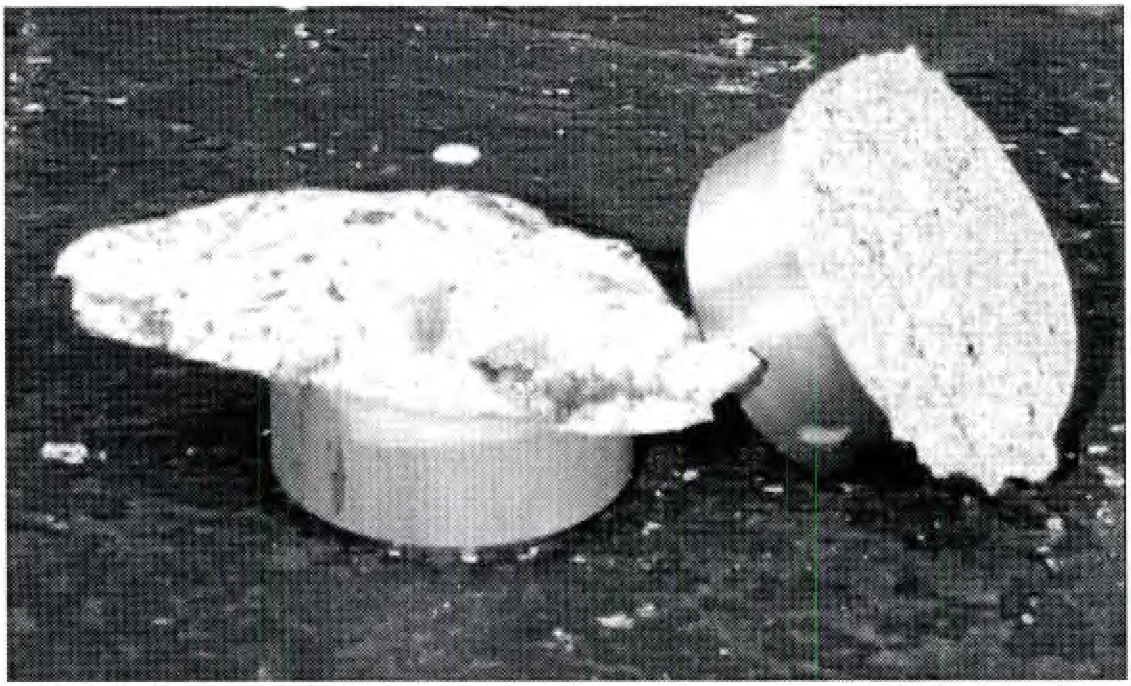


Figure 29. Pull-off test "pucks". Primed concrete surface on left puck shows penetration into microcracks. Unprimed concrete surface on right puck.

EMBEDDED REINFORCEMENT – *Negative Moment Resistance*

Embedding carbon high strength carbon/epoxy reinforcing rods into the deck is particularly attractive on the top surface where external reinforcement would be subject to mechanical and environmental damage and would require protective cover. High strength carbon rods possess desired stiffness, strength, and durability and are compatible with concrete, epoxy sealant, and epoxy adhesives. Material Specifications are provided in **Appendix D**. To meet upgrade moment capacity goals, the required area of high strength carbon fiber was 0.2 in^2 per foot (4.2 cm^2/m) width of deck. The required area was met by 3/8-inch (10 mm) diameter DFI® pultruded rods with 65 percent (by volume) carbon fibers spaced at 4 inches (10 cm) on center. The encapsulating epoxy was Sikadur® 32 Hi Mod, a two-part, 100 percent solids, moisture tolerant, structural adhesive with excellent bonding strength to concrete.

The surface area was primed with penetrating epoxy sealer/primer (Madewell 927®) and allowed to cure overnight. The reinforcing grid was laid out by chalk line and slots were cut in the deck with a concrete saw (**Figure 30**). The depth and width of the slots were determined by the diameter of the reinforcing rods. The slots should allow 1/16 inch (2 mm) between rod and the concrete, and at least 3/8 inch (10 mm) clear cover. The installer cut the slots in the range of 7/8 inch (22 mm) deep and 5/8 to 3/4 inch (16 to 19 mm) wide. The slots were abrasive blasted to roughen the slot surface and air blasted to clean the concrete. The slots were primed before filling with epoxy encapsulate. In the spans between bents 67 through 73, 20 percent 60-grit sand by volume was added to the epoxy. The sand extended the epoxy volume, lowered the epoxy thermal coefficient of expansion, and raised the epoxy glass transition temperature.

Figure 30. Saw cutting slots on deck surface.

Figure 31. Embedded high strength carbon rod in top of deck over pilecap.

The rods were placed in sequence into the slots by hand and pressed to the bottom of the slots using rollers or similar tools (**Figure 31**). After all the rods were placed, and before the epoxy encapsulate began to gel, the slots were filled up to within ¼-inch of the original concrete surface. After the encapsulate was cured the surface was abrasive blasted and a UV protective layer was added up to the top of the slot. The protective layer was two parts (by volume) 60-grit sand and one part Sikadur® 22, a two-component, 100 percent solids, moisture resistant, epoxy resin binder (**See Figure 32**). The sand provides a non-slippery epoxy surface that is UV resistance. The surface is ready for use 24 hours after the installation.

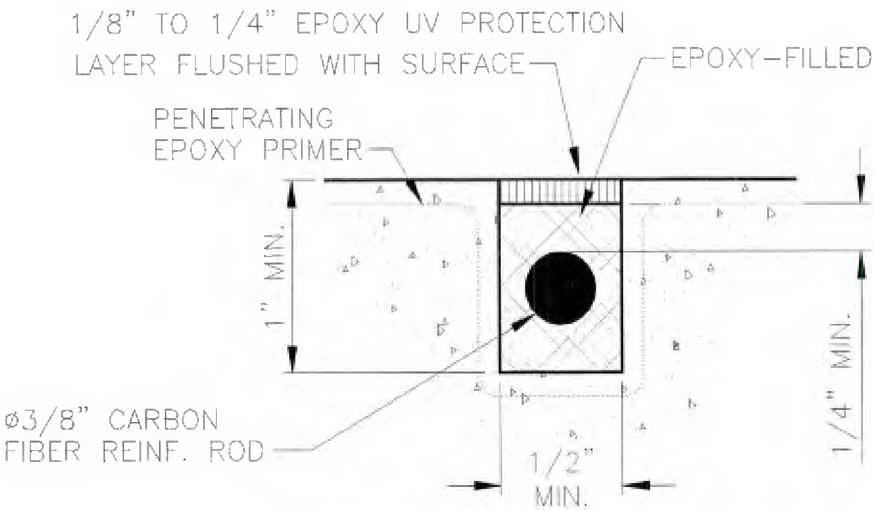


Figure 32. Detail of embedded high strength carbon rod.

Carefully planning is required in laying out the installation sequence when placing large quantities of carbon rods. The entire process must be completed within the pot life of the adhesive to ensure proper bonding and anchoring. The adhesive has a very short pot life (usually about 45 minutes) and it cannot be poured into the slots after it starts to set. Large areas must be subdivided to ensure that there are no adhesive joints where uncured epoxy is placed over, or adjacent to, partially cured epoxy that has exceeded its pot life. Areas as large as 100 ft^2 (9 m^2) can be efficiently installed within the adhesive pot life time limit.

Immediately after the slots are cut in the concrete we recommend that they be thoroughly cleaned using a water jet. This will remove all powdered concrete produced by the cutting process before it can dry and set in the slots. The dried powdered concrete is hard to remove once it has dry set. A dry, concrete-cutting saw system with a vacuum is a more efficient alternative. Cleanliness is very important, the areas of epoxy application must be kept clean of all loose material.

The theoretical strain to failure of the carbon/epoxy composite rods is 1.7 percent at approximately 300,000 psi (2,000 Mpa) assuming 500,000 psi (3,330 Mpa) fiber strength. The measured values from laboratory coupon tests averaged 0.9 percent at 225,000 psi (1,550 Mpa)(**Figure 33**). Misalignment and damage of the fibers during the pultrusion process cause the difference.

The installers cut the slots larger than necessary or required by the specifications. Deeper and wider slots resulted in increased volume of epoxy for embedment. The increase amounted to more than 30 percent. The slot depth was cut unevenly, which, coupled with the undulations in the deck surface, resulted in poor coverage of a few of the reinforcement bars.

One precautionary note should be remembered when selecting an encapsulating epoxy. Epoxies will deform under stress at low temperatures while reinforced epoxies do not. The deflection temperature and the glass transition temperature express this characteristic. The decks of Navy piers can exceed temperatures of 130°F in tropic zones such as Pearl Harbor. An encapsulating epoxy should be selected that has a deflection temperature in excess of this. Adding sand will raise the deflection temperature slightly and conditioning the cured epoxy at high temperatures while unstressed will also drive the deflection temperature up.

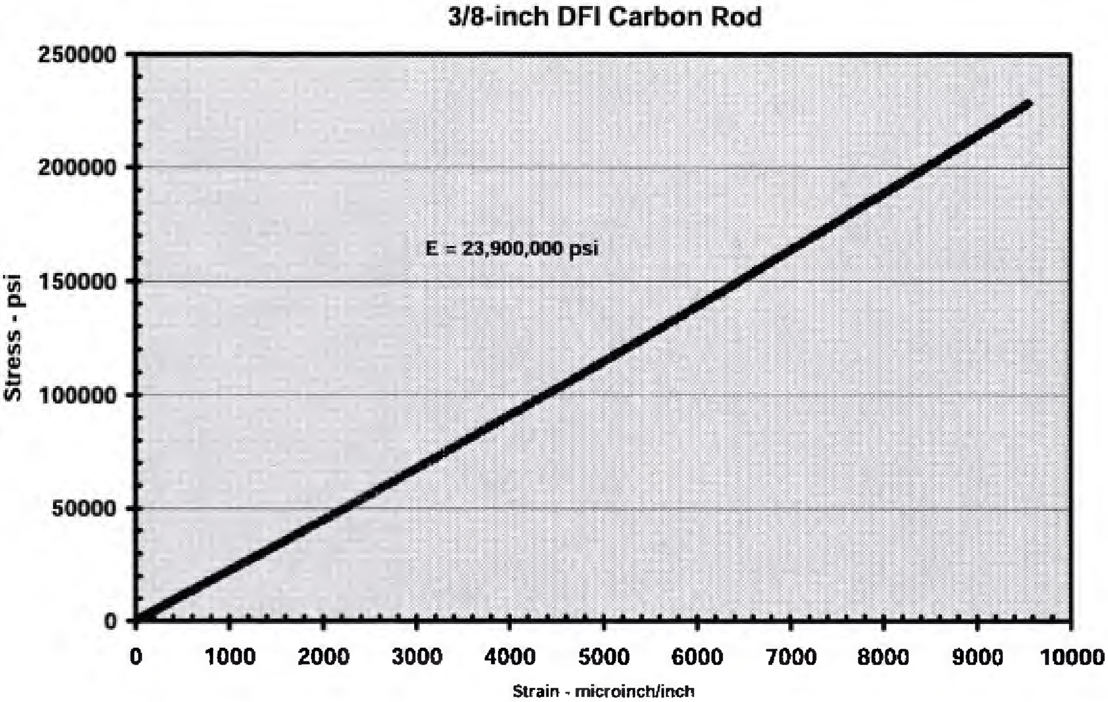


Figure 33. Carbon rod coupon tensile test response.

PULTRUDED CARBON UNIDIRECTIONAL LAMINATE STRIPS

Thin, pultruded composite strips with a 65 percent content of high strength carbon fibers in an epoxy matrix were attached to the underside of Bravo 25. The pultruded strips were Sika CarboDur® attached with Sikadur® 30 adhesive. These strips are 5/64-inch (1.2 mm) thick and are 2, 3, and 4 inches (50, 75, and 100 mm) in width. They arrived at the site in large rolls and were cut to length.

The ideal concrete surface for applying pultruded laminate strips is clean, dry, planar and abraded to 100 grit roughness. Pultruded strips have an inherent stiffness that requires the surface be nearly flat. The bond line between the laminate and the concrete is two surfaces and the carbon fibers are not actually bonded to the concrete. After concrete repair and surface preparation Sikadur® 55 SLV penetrant/sealer/primer was applied to the concrete and allowed to cure. Surface fluctuations and indentations that remained after grinding were filled and smoothed with a grout mixture of Sikadur® 30 epoxy and sand. Indentations in the bonding surface negate the laminate bonding making it susceptible to peeling. Therefore concave surface areas were not allowed. A layer of epoxy grout was troweled over the entire surface prior to installing the carbon strips.

The strips were bonded to the concrete with Sikadur® 30, an amine epoxy adhesive paste. The adhesive has a minimum tensile strength of 4,400 psi with a tensile modulus of 1,900,000 psi and shear strength of 2,500 psi. The adhesive was sufficiently viscous to hold a shape without running or dripping when applied on the bottom side of the pier deck. After a thorough cleaning of the strip, adhesive was applied to the bonded side in a "□" shape. The installers pulled the strip through an adhesive applicator. Sufficient material was applied to form a 1:12 height-to-width adhesive volume on the strip. The strip was laid up under the slab surface and excess adhesive was forced out by uniformly applying pressure with a hand roller (**Figure 34**). No laps or splices were allowed. The strips were continuous across each span.

The theoretical strain at failure for the pultruded strips was 1.7 percent at 25.6 kips/inch (325 ksi)(44.8 kN/cm (2,240 Mpa)) assuming 500,000 psi (3,330 Mpa) fiber strength. The measured values from coupon tests are 1.38 percent strain to failure and 20.4 kips/inch (260 ksi)(35.7 kN/cm (1,790 Mpa))(**Figure 35**). Fiber damage and misalignment during the pultrusion process caused the difference.

Due to their inherent stiffness, the pultruded strips did not conform well to the original underside surface of the deck. This necessitated additional grinding and addition of a significant amount of epoxy grout to obtain a smooth, planar surface for bonding the strips. The thickness of the grout was as much as ½inch 1.3 cm). Nonetheless the bond between the grout and concrete as well as between the grout and the strip was in excess of the required 300 psi (2,070 kPa) measured by the pull-off testing. The pull off tests resulted in failure of the concrete substrate as required. The pultruded strips performed well and as predicted during subsequent proof load tests performed on the deck. Examination of the strips by thermography found no delaminations or gaps in the bonding surface.

Position adjustments were necessary during installation to avoid drain holes and pipe hangers. As-built configurations are shown for each span in the drawings of **Appendix A**. Material Specifications are presented in **Appendix D**.

Figure 34. Installing pultruded carbon strips to the underside of the 8-inch deck section.

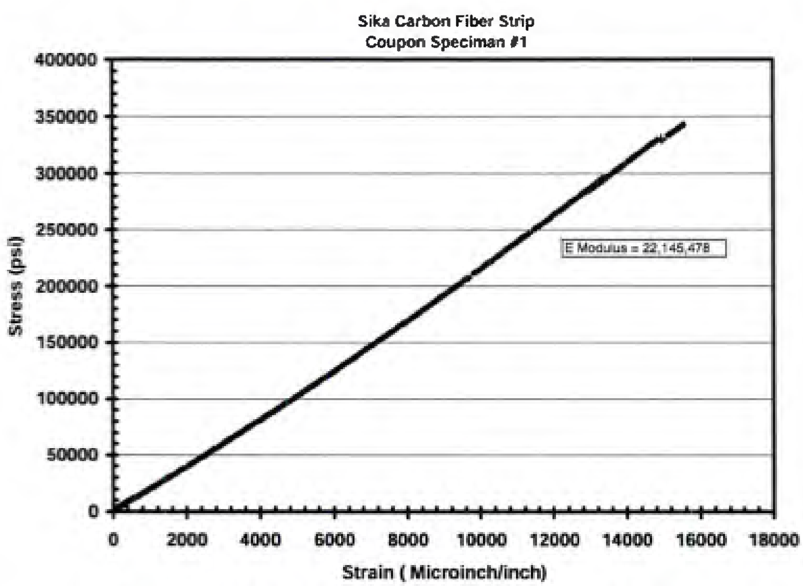


Figure 35. Pultruded carbon laminate tensile test response.

BONDED COMPOSITE STRUCTURAL SECTIONS

The 8-inch (20 cm) section concrete deck section of Pier 12 is too thin to provide adequate moment and shear resistance to the applied crane outrigger loads. Therefore, in addition to surface reinforcement of carbon laminate strips, 12-inch (30 cm) deep composite structural I-sections (Strongwell Extren® Series 625) were bonded and mechanically anchored to the underside of the deck section. The structural sections were aligned with the longitudinal axis of the pier. The I-sections had an area of 11.5 in^2 (74 cm^2), a strong axis moment of inertia of 254 in^4 (10,570 cm^4) a flexural strength of 36 ksi (250 Mpa) and a shear strength of 7 ksi (48 Mpa). Two I-beams were added to each side of the span to support the 8-inch (20-cm) deck section as shown in **Figures 9 and 10**.

Detailed requirements for materials, preparation, installing, and testing the structural section system are provided in the contract specifications. The two I-beams were spaced 2 feet (0.6 m) on center (nominal) and two feet (0.6 m) from the edge of the 24-inch (61 cm) deck section. The lengths were field-cut and custom fitted to each location with a maximum of 1/16-inch (2 mm) space between the end of the I-sections and the concrete pilecaps, bollard platforms, or cleat platforms. As-built configurations are shown in **Appendix A**. Material Specifications are presented in **Appendix D**. Preparation procedure of the concrete surface was the same as for the other laminates including priming and sealing the concrete with epoxy penetrant (Sikadur® 55 with 95-cps viscosity) and leveling the surface with an epoxy paste made of Sikadur® 30 mixed with sand. Sikadur® 30 was also used as an adhesive between the laminate and the epoxy paste. Sufficient bonding material was applied so that the smooth and straight flange of the structural section would adapt to the inherently uneven concrete surface. This adhesive was required to transfer shear between the concrete and the composite section. The thickness of the adhesive layer varied between 1/8-inch (3 mm) and 3/8-inch (10 mm bottom surface of the deck was curved so the). The beam was held in place under constant pressure by temporary anchor rods in the deck while the adhesive cured.

The I-sections were connected to the vertical surfaces of the pilecaps or bollard/cleat platforms. Angle end connections and seats were added after the beam was set. **Figures 12 and 13** are schematics of end anchor details shown in **Figure 36**. The end connections consist of pultruded angle sections with the same structural and material properties as the I-sections. The angles are anchored to sound concrete of the pilecaps or bollard platforms with ¾inch (19-mm) stainless steel anchor bolts and epoxy adhesive (Adhesive Technology ANCHOR-IT® HS-200). The end connections are required to resist the total shear capacity of the 12-inch (30-cm) section or 50 kips (22 kN).

Coupon specimens were cut from example I-beam sections and tested in Universal test machines. The measured strain failure for the fiberglass I-beams was 1.6 percent at 37 ksi (255 Mpa) (**Figure 37**).

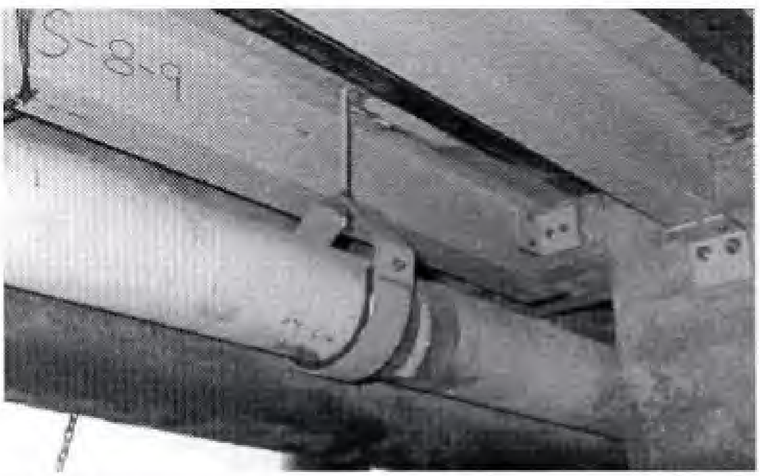


Figure 36. Fiberglass composite I-beams installed on the south side of Span 8/9.

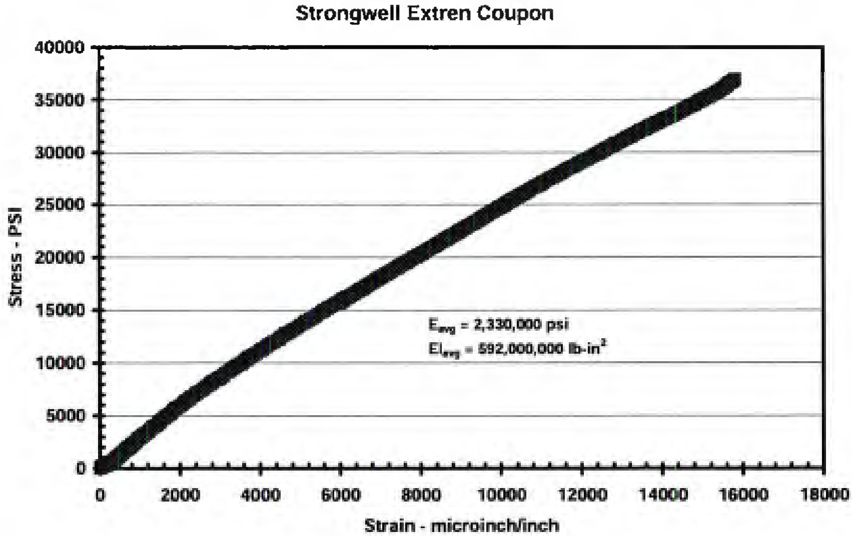


Figure 37. Fiberglass composite coupon tensile test response.

CARBON FIBER SHEETS WITH IN-SITU (WET) IMPREGNATION

Hand laid, field-impregnated, uniaxial, carbon fiber sheets were laminated to the 24-inch (61 cm) deck section. Field impregnation employs a resin saturator (**Figure 38**) to impregnate the fibers. A saturator evenly distributes the epoxy matrix through the fabric and insures consistent wetness of the fibers. This is an efficient method that ensures a more uniform distribution of saturate in the laminate. Material specifications are provided in **Appendix D**. A high strength carbon fiber area of 0.39 in^2 per foot (8.3 cm^2/m) width was required longitudinally under the decks along the outside 53 inches (135 cm) of the 24-inch (61-cm) deck section (See **Figure 10**). The installer applied 4 layers of TYFO® SCH 41carbon fabric that equaled 0.61 in^2 of carbon fiber per foot (12.9 cm^2/m). SCH 41 utilizes a uniaxial continuous tow of 50K carbon fibers (0.0127 in^2/in or 587 gm/m^2). The epoxy matrix was TYFO® S, a two-part, solvent free, general-purpose epoxy.

Applying a polymer impregnated composite laminate to concrete required that the surface be clean and smooth. Fins and concrete surface discontinuities were knocked down with a small, compressed-air-driven hammer. Protruding tie wires, reinforcing chairs, and rust stains were removed by grinding. Laitance was removed by sand blasting. A field mixture of epoxy (TYFO® S) and silica fume (cabosil) was rolled onto the concrete surface as a prime coat. A thin layer of TYFO® TC paste was trowelled over the entire concrete surface to fill small surface voids and smooth the concrete surface for application of the laminate layers. The first epoxy (TYFO® S) impregnated carbon fiber sheet was applied after the paste had cured a minimum of 12 hours (**Figure 39**). Four plies of uniaxial fiber layers were aligned longitudinally (strong direction) on the spans. A layer of TYFO® TC "tackifying" epoxy was applied between each impregnated fiber sheet.

The laminate was air cured at temperatures between the low 60's and high 70's, Fahrenheit. The installers "worked out" excess saturate and bubbles with a squeegees and rollers. Holes were not cut in the composite to circumvent obstacles such as pipes, hangers and drain holes. The laminate fiber sheets were split to circumvent these obstacles. Lap splices (8 inches (20 cm) minimum in length) were permitted in the fiber direction. A protective coating of epoxy (TYFO® WS) was applied over the final layer.

This upgrade system performed well and as predicted during proof load tests. Examination of the laminates found no significant delaminations or gaps in the bonding surface. Pull off tests usually failed in the TYFO® TC epoxy coat between the first layer of laminate and the concrete. All of the pull off tests met or exceeded the minimum required 300 psi (2,070 kPa) strength. None of the pull off tests resulted in tensile failure of the concrete substrate.

The finished laminate consisted of a much greater proportion of epoxy than the other laminate reinforcements. Since the 50K fabric is heavier than the 30K tow sheets used in prior upgrades on Pier 11, NAVSTA Norfolk, additional, more viscous, tackifying epoxy was necessary between the impregnated carbon fabric sheets to hold them in the overhead position. The finished laminate, including the layers of TYFO® TC, was more than 1/2 inch thick in some locations (compared to maximum 3/16 inch thick for the 5 layers of 25K tow sheets used at an earlier installation with the same area requirements). The installer did not use a low viscosity (less than 200 cps) penetrating sealer/primer. The primer, TYFO® S with cabosil, does not penetrate the concrete microcracks found on aged, porous concrete like a low viscosity penetrant. The stress-strain response of the laminate is linear up to failure (**Figure 40**). The theoretical strain to failure of the composite was 1.7 percent. Laboratory coupon tests of the material provided an average failure strain of 1.24 percent. The measured laminate strength was well above the required strength. The average, axial, failure force of the laminate was 20.9 kips/inch (36.6 kN/cm) width. The limiting service strength (defined at 0.83 percent strain) was measured to be 14.5 kips/inch (25.4 kN/cm) width. The strain at the required service strength (8.3 kips/inch (14.5 kN/cm) width) averaged 0.47 percent. The resultant laminate was much stiffer in the service range than the minimum required.

Figure 38. Saturator for in situ epoxy impregnation.

Figure 39. Hand application of in situ impregnated laminate sheet.

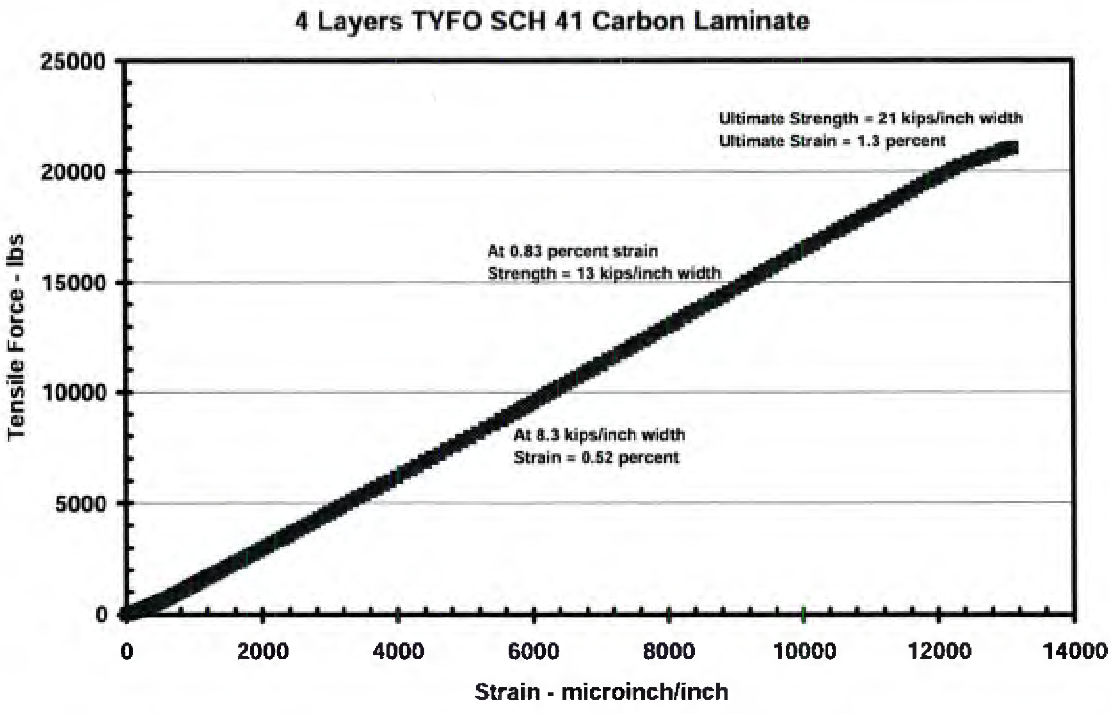


Figure 40. Field-impregnated, carbon laminate coupon load-strain response.

PILE CONFINEMENT – PREFORMED COMPOSITE SHELLS

Two-piece, pre-formed, fiberglass reinforced, cylindrical shells were used to encase the piles (**Figure 41**). Two piles were selected from each bent for confinement. The selected piles included batter and vertical piles that were not encased in large quantities of shotcrete. The shells were designed to increase the confinement strength of the concrete. Hardcore Dupont Composites (HD) fabricated the shells using the SCRIMP process with E-glass in a vinyl ester matrix.

The fabrication included bonding a 30 mil, acrylic exterior layer for UV protection. The acrylic was "rubber toughened" modified to enhance UV resistance. This acrylic was interstitial (chemically) bonded to the vinyl ester matrix as distinct from an after production coating. We do not expect any fading for at least 5 – 10 years and the coating should not chip or flake for the life of the shell. The fabricator selected the acrylic layer over the preferred nexus veil with a resin rich outer layer for UV.

HD used Brunswick WM-4505 unidirectional continuous E-Glass (45 oz/yd^2)(1,526 gm/m^2). Eight plies were used in the circumferential direction and two plies were used in the longitudinal direction. Pile batter disrupted the uniaxial hoop fibers at the intersection with the pilecap. To insure required hoop strength at the intersection of the batter piles with the pilecap, a quasi-isotropic E-glass matrix was fabricated into the laminate at the top 12 to 28 inches of the batter pile shells using Brunswick QM-6408. This is an E-Glass quadraxial mat with 26 oz/yd^2 warp, 16 Weft, 12 at +45° bias and 12 at –45 ° bias. The matrix of the E-glass was Dow Chemical's Derakane 411-45 epoxy vinyl ester resin. The finished shell laminate thickness was almost 1/2-inch and each weighed approximately 400 pounds assembled. The inside diameter of the shells was 30 inches to enclose the 20-inch square pile and leave space for grout to completely encase the pile. The shells were 8 feet long and extended from the pilecaps down into the tidal zone. The material specifications are provided in **Appendix D.**

The required circumferential (confinement) strength of the shells was 4 kips/inch (7 kN/cm) at a maximum strain of 0.2 percent. The required longitudinal strength of the shells was ¼ the circumferential stength. Measured load-strain responses are provided as **Figures 42 and 43**. The stiffness is slightly higher than required (4.33 kips/inch (7.6 kN/cm) at 0.2 percent deformation). However, since 8 fiber plies were used to obtain the stiffness (stiffness of the glass fibers is 1/10 that of carbon) the ultimate strength was more than 10 times that which was necessary.

The half-cylindrical shells were joined with an H-connector joint to facilitate easy pile encapsulation on the site. The connectors were attached to each half-shell at the factory after the SCRIMP process. The H-connector cradled the joining adhesive and prevented it from escaping into the bay water during the installation process. The connectors consisted of 3-inch (7.6 cm) double lap shear union. The joining adhesive was Plexus AO425 two-part methacrylate with measured double lap shear strength of 17 kips per inch (29.8 kN/cm) (**Figure 44**). The adhesive was applied from cartridges in pressurized handguns. Approximately 20 pounds (9 kg) of adhesive was used to join the two cylinder halves. The adhesive achieves 75 percent of its ultimate strength in 90 minutes. Coupon tests showed the connections joined and cured under seawater had 10 percent less strength compared to connections joined under "room conditions".

Marine life and fouling was scraped from the surfaces and loose concrete was removed before encasing the pile. Plywood forms were fabricated and placed around the pile to butt against the bottom of the shells. A reusable rubber seal was placed between the form and the edge of the shell to keep grout material from leaking into the bay water prior to set. The shell and form were clamped in place to allow the joining adhesive to cure for 24 hours. The shells were configured to allow placement within 1 inch (2.5 cm) of the pilecap. For those piles which had layers of shotcrete at the intersection of the pilecap, the shells were placed as close as possible to the pilecaps after some shotcrete was removed.

Prepackaged, shrink resistant grout (SikaGrout® 212 Plus) was pumped into place from the top between the shell and the pilecap. A prepackaged grout was chosen so that a 1-inch (2.5 cm) diameter hose could be used for pumping through a small opening at the pilecap. Sikament® 100SC admixture was added to prevent dilution as the grout displaced the seawater during placement. Cylinder tests results at 28 days averaged more than 6,000 psi (41 Mpa).

Figure 41. Placing cylindrical composite shell around pile.

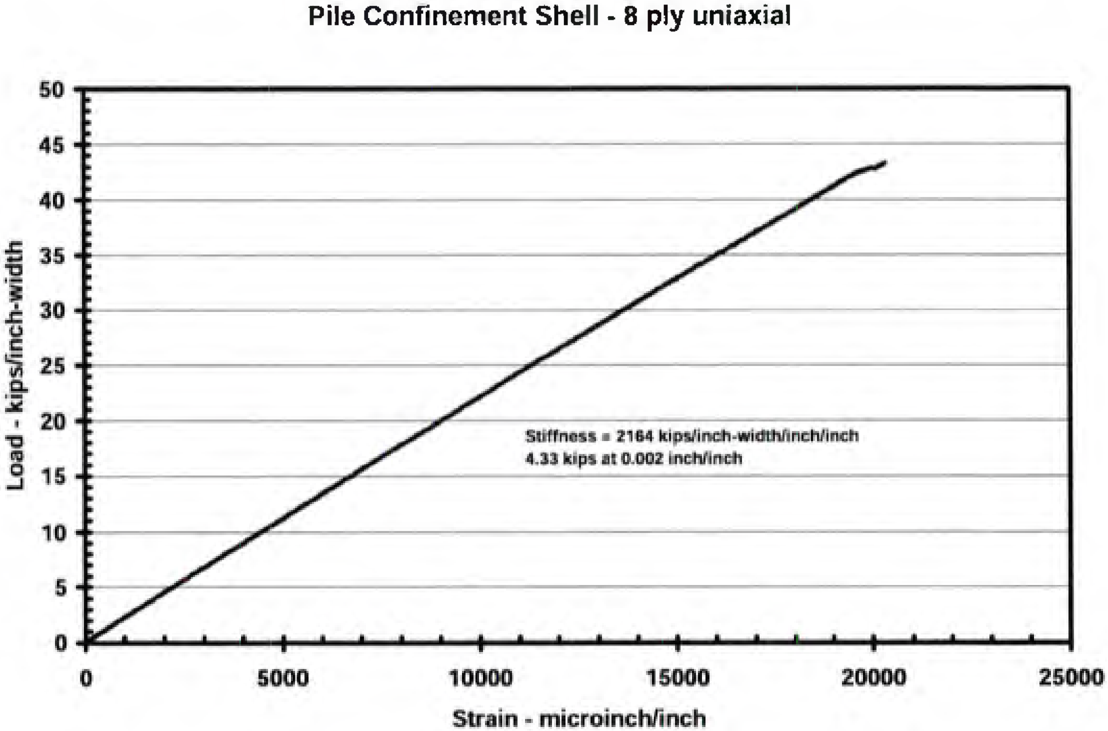


Figure 42. Load-strain response of tensile coupon from 8-ply fiberglass/vinyl ester shell.

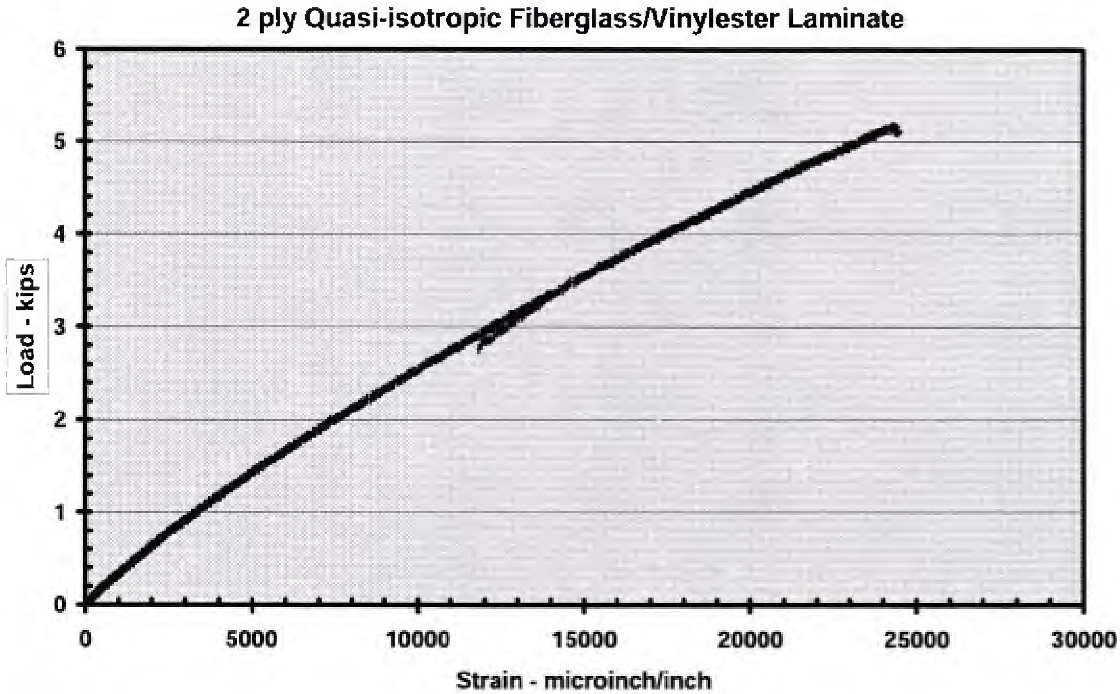


Figure 43. Quasi-isotropic, fiberglass/vinylester, tensile coupon load test response.

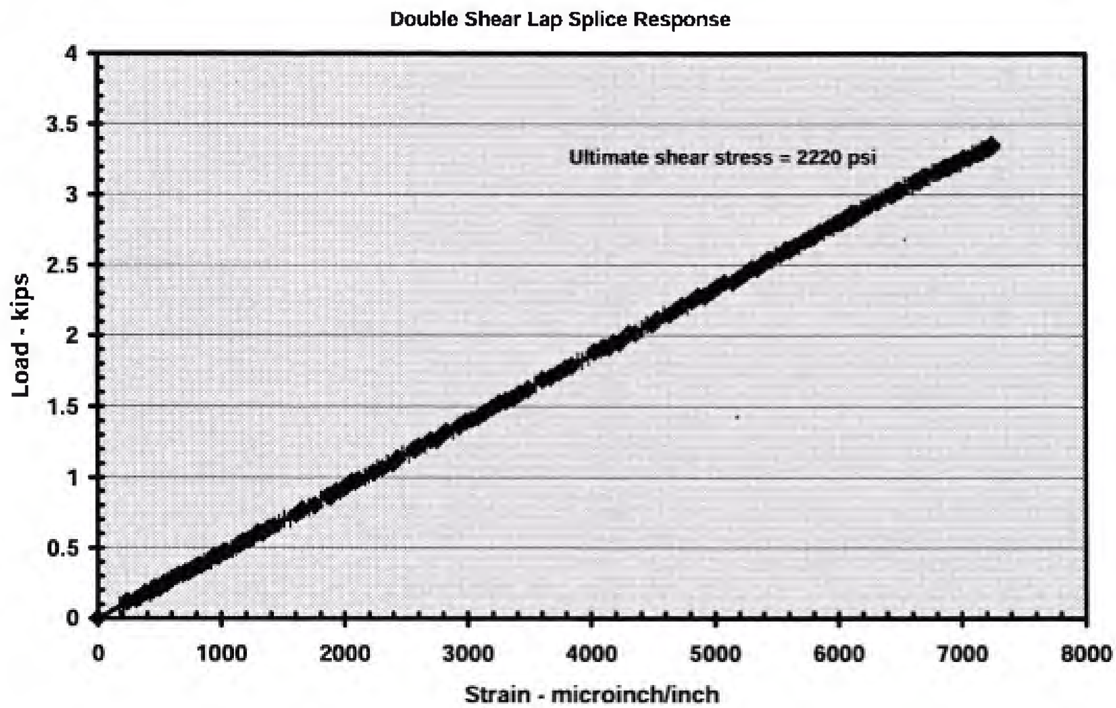


Figure 44. Load response of coupon from double shear lap joint in pile shell.

STRUCTURAL TESTING – PROOF LOADING

After completion of the upgrade we conducted proof tests of four spans using simulated outrigger loads. We monitored strain gages attached to the reinforcing systems. Two gages were placed on each of the systems (carbon rods, pultruded strips, I-beams, and the hand laid laminate) at midspan of each span. The outrigger loads were simulated by placing dead weights onto a 24-inch (61-cm) square plywood pad on the pier deck. The loads were applied at midspan on the edge of the 24-inch section (61-cm) and at the center of the 8-inch (20-cm) section. Weights were placed using a barge crane supplied by the Public Works Center, San Diego (**Figure 45**). The applied, calibrated weights were: 25 kips (112 kN), 50 kips (225 kN), 75 kips (337 kN) and 100 kips (450 kN). The weights were applied individually and held for 1 minute while the strain was monitored. The load-strain response of all the test sites are provided in **Figures 46 through 53**. The maximum measured response in the external composite reinforcing was 500 microstrain. The response is well below the service limit of the composites and should be satisfactory for supporting outrigger loads up to 100 kips.

Figure 45. Applying calibrated weights to the deck for proof loading upgrade areas.

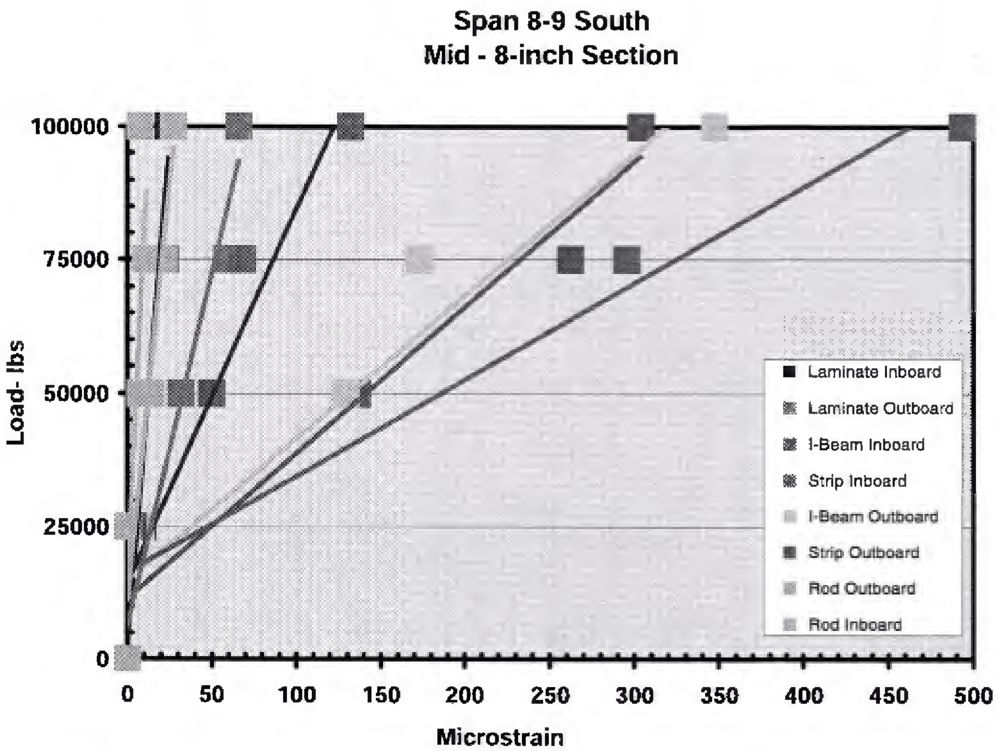


Figure 46. Load-strain response of upgrade reinforcement on the south side of Span 8-9. Outrigger applied at midspan in the center of the 8-inch deck section.

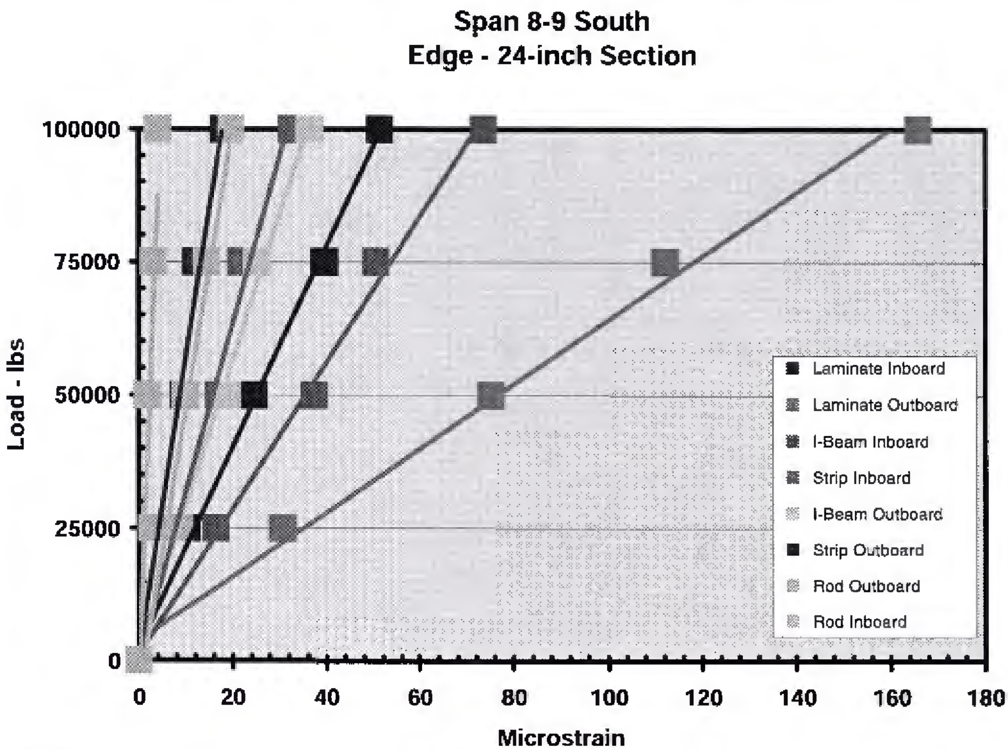


Figure 47. Load-strain response of upgrade reinforcement on the south side of Span 8-9. Outrigger applied at midspan on the edge of the 24-inch deck section.

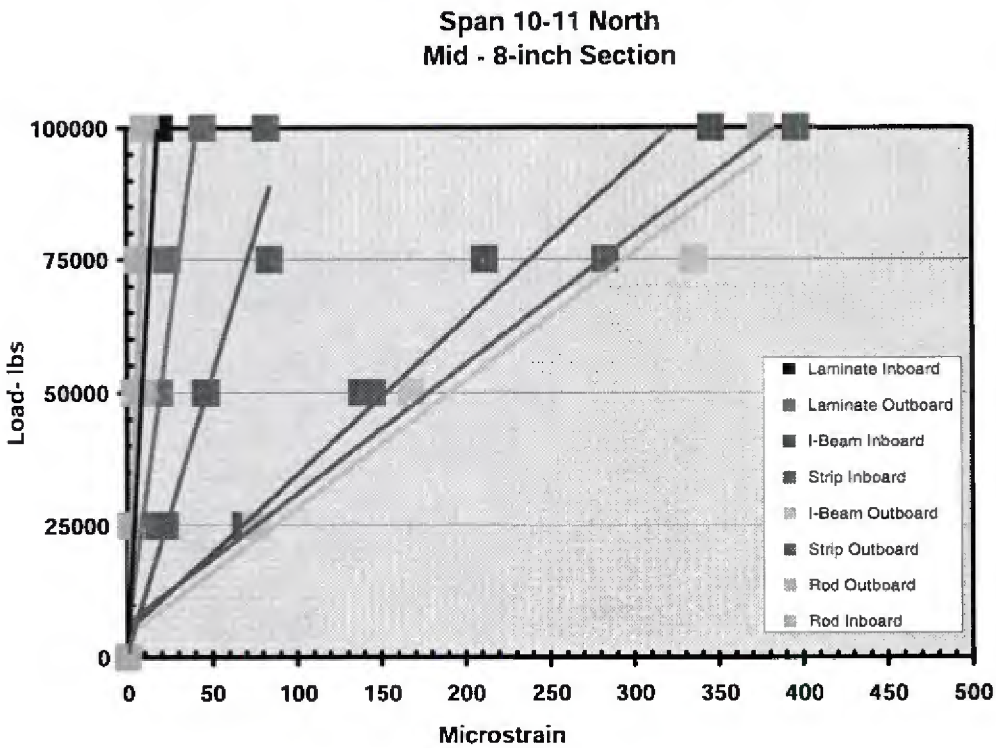


Figure 48. Load-strain response of upgrade reinforcement on the north side of Span 10-11. Outrigger applied at midspan in the center of the 8-inch deck section.

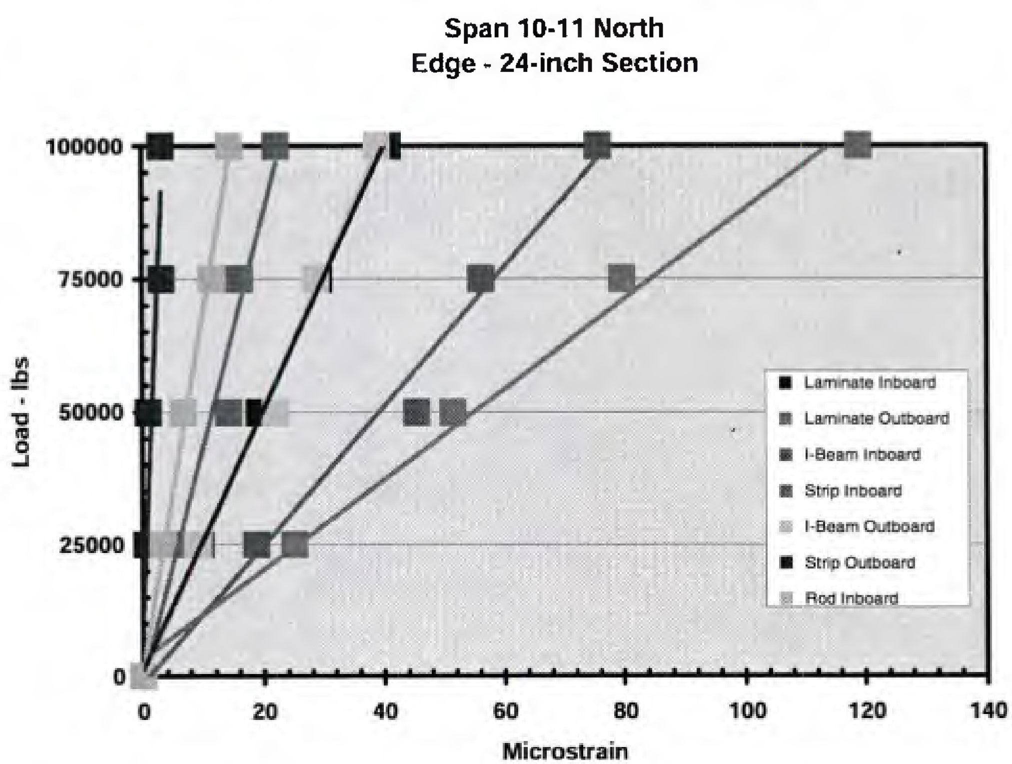


Figure 49. Load-strain response of upgrade reinforcement on the north side of Span 10-11. Outrigger applied at midspan on the edge of the 24-inch deck section.

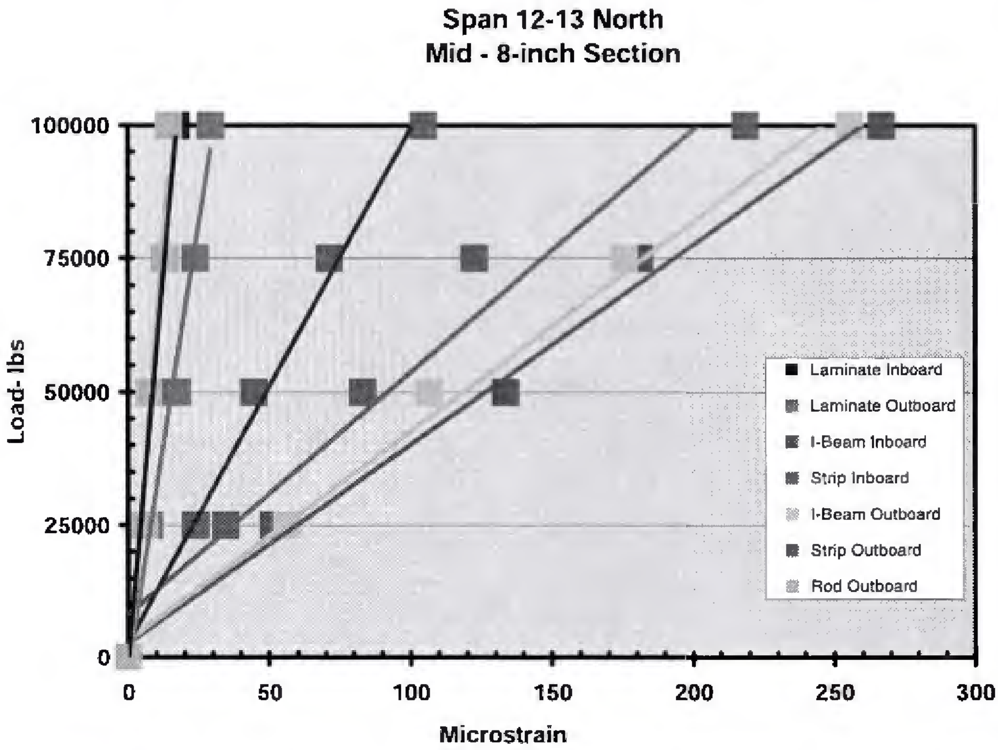


Figure 50. Load-strain response of upgrade reinforcement on the north side of Span 12-13. Outrigger applied at midspan in the center of the 8-inch deck section.

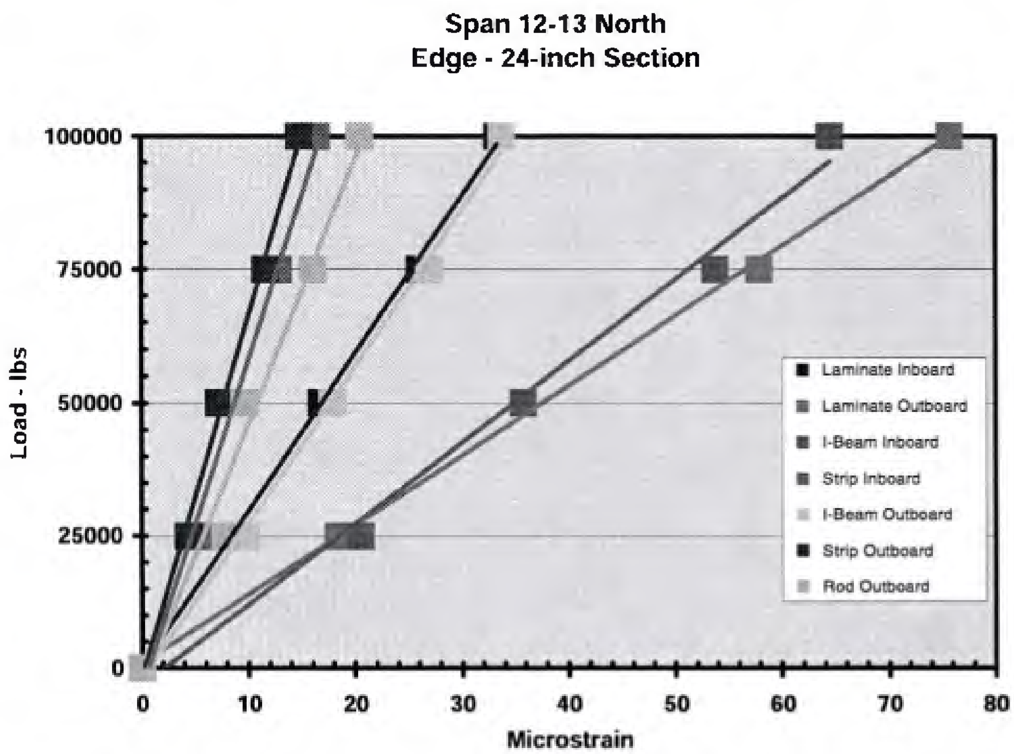


Figure 51. Load-strain response of upgrade reinforcement on the north side of Span 12-13. Outrigger applied at midspan on the edge of the 24-inch deck section.

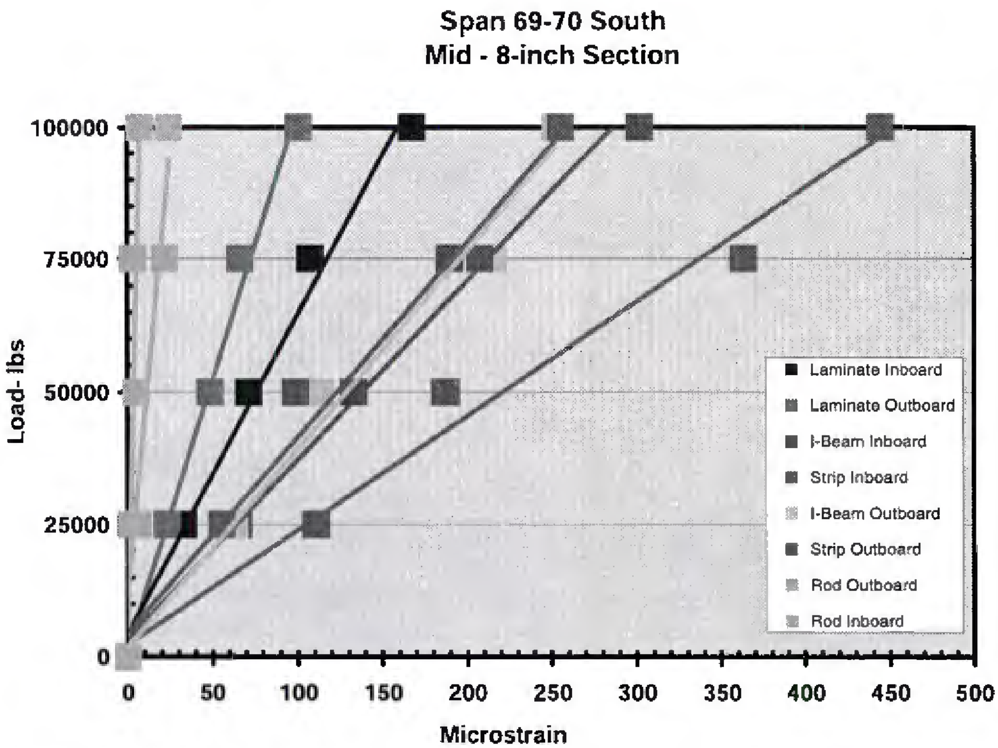


Figure 52. Load-strain response of upgrade reinforcement on the south side of Span 69-70. Outrigger applied at midspan in the center of the 8-inch deck section.

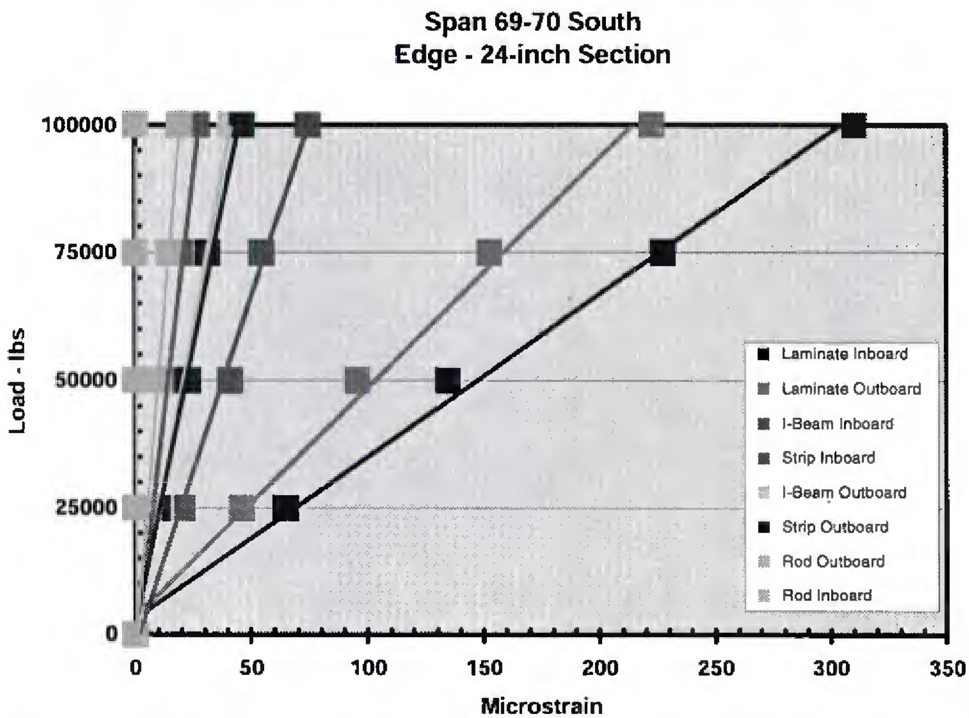


Figure 53. Load-strain response of upgrade reinforcement on the south side of Span 69-70. Outrigger applied at midspan on the edge of the 24-inch deck section.

COSTS

Table 2 contains cost data on the installation of the five external reinforcing systems. These include surface preparation, materials and installation.

Table 3. Unit costs of Composite reinforcing systems

Composite Reinforcing System	Unit Cost
Embedded Carbon Rods	100/ft^2 (\$1075/m^2)
Pultruded Carbon Strips	\$70/ ft^2 (\$755/m^2)
Wet Lay-Up Carbon Laminate	100/ft^2 (\$1075/m^2)
Pultruded Fiberglass Beams	$250 per lineal foot ($820 per meter)
Fiberglass Pile Shell	$960 per lineal foot ($3150 per meter)

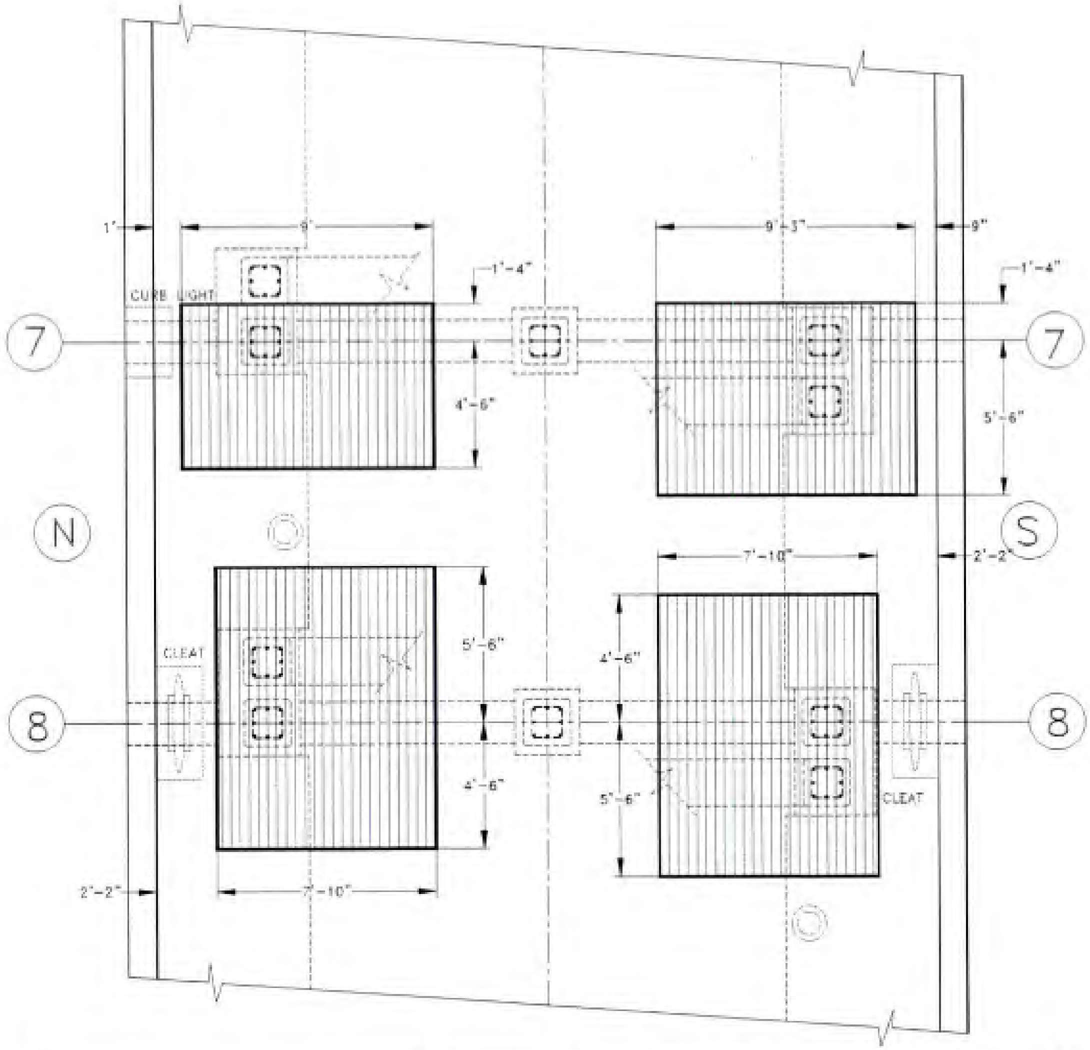
ACKNOWLEDGEMENTS

The Naval Facilities Engineering Service Center provided the repair and upgrade design and selected the materials. NAVSTA Staff Civil Engineer Office, Public Works Center, NAVFACENGCOM Southwest Division, and the South Bay Focus Team provided contract, test, and design requirements support. William P. Young, Incorporated, was the general contractor for the project. Douglas Burke (ESC63) designed the concrete repair and provided on-site quality assurance. David Hoy (ESC63), Steve Harwell (ESC62), and Chris Inaba (ESC62) provided on-site quality assurance and conducted laboratory test.

APPENDIX A:

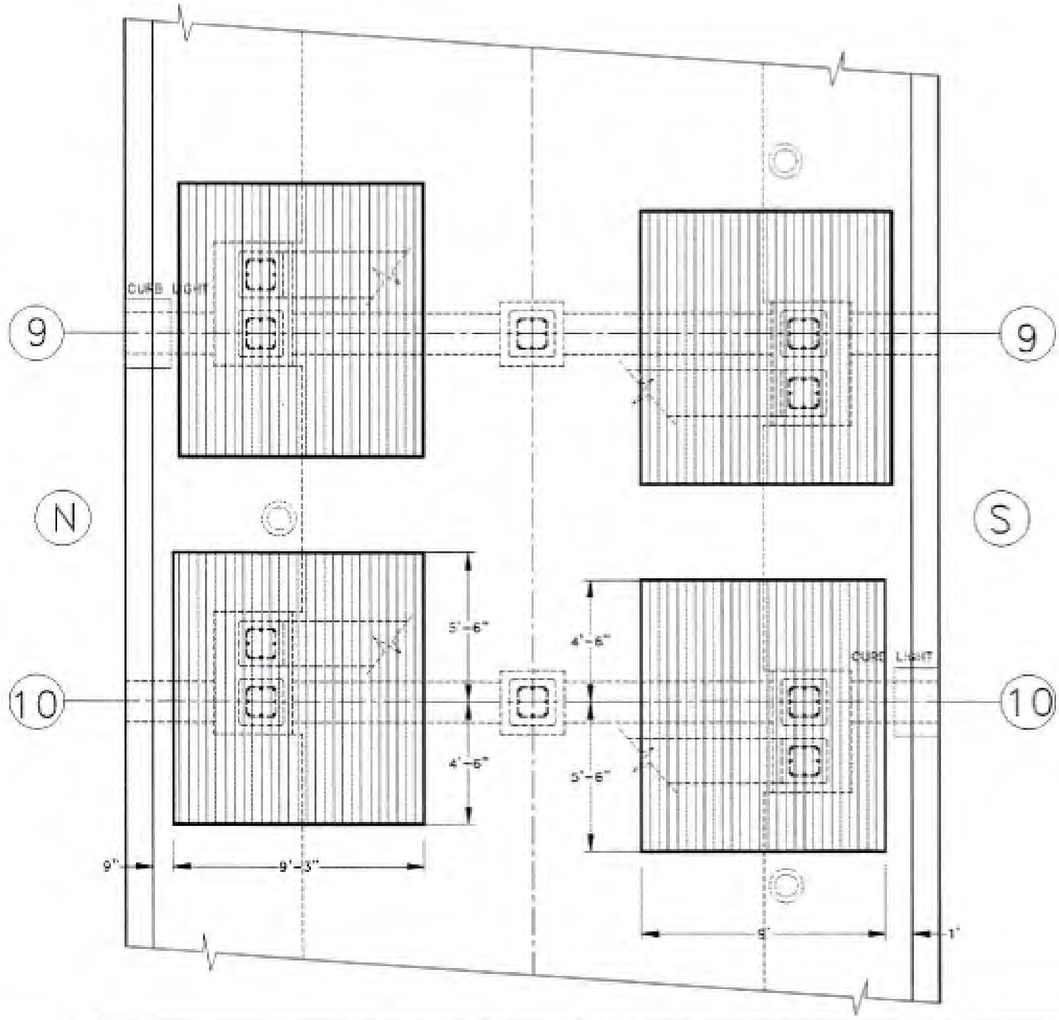
As-Built Drawings: Pier 12 Upgrade NAVASTA San Diego

Upgrade Reinforcement on Top Surface of Deck



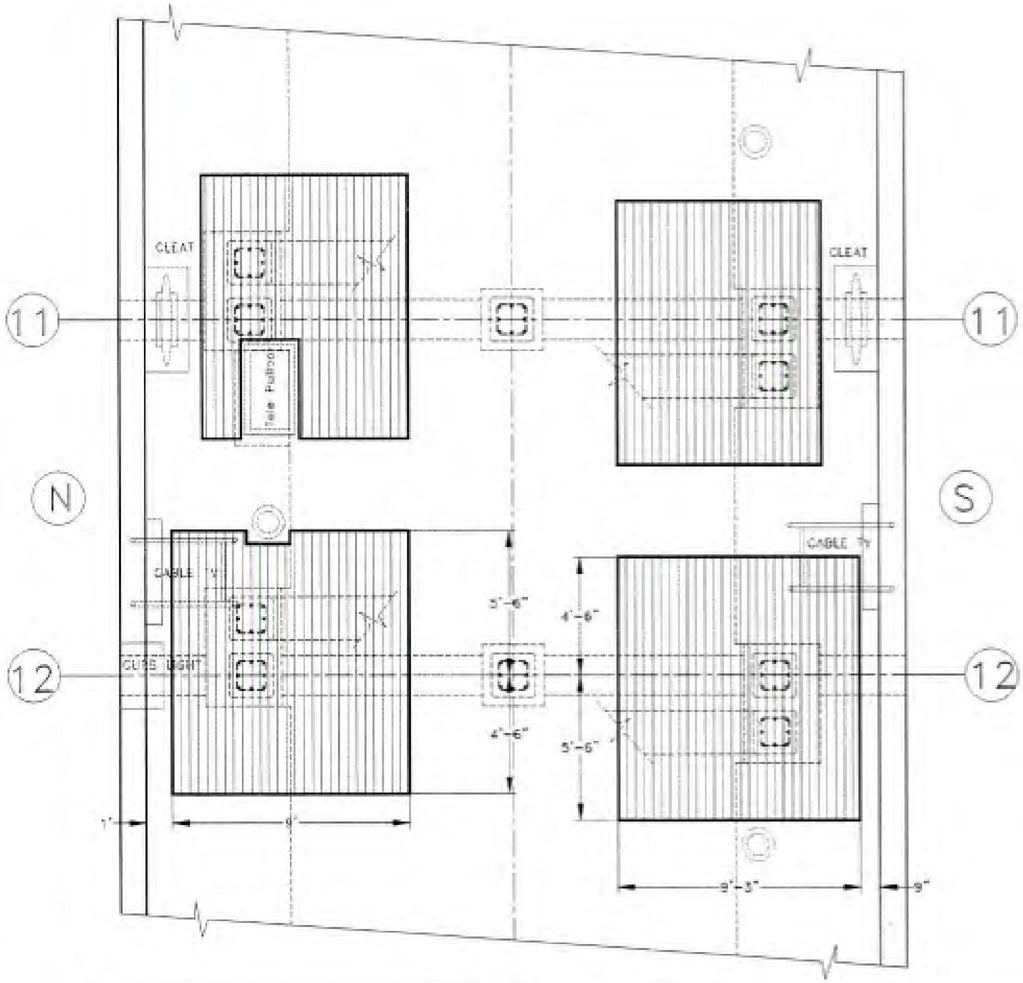
3/8" DFI Diameter Carbon Reinforcing Rods @ 4" O.C	Measured Ultimate Tensile Strength : 220,000 psi @ .92% strain (72.6 kips/in) Recommended Design Limit : 110,000 psi (36.3 kips/in)

Figure A1. Added Reinforcement over Bents 7 and 8.



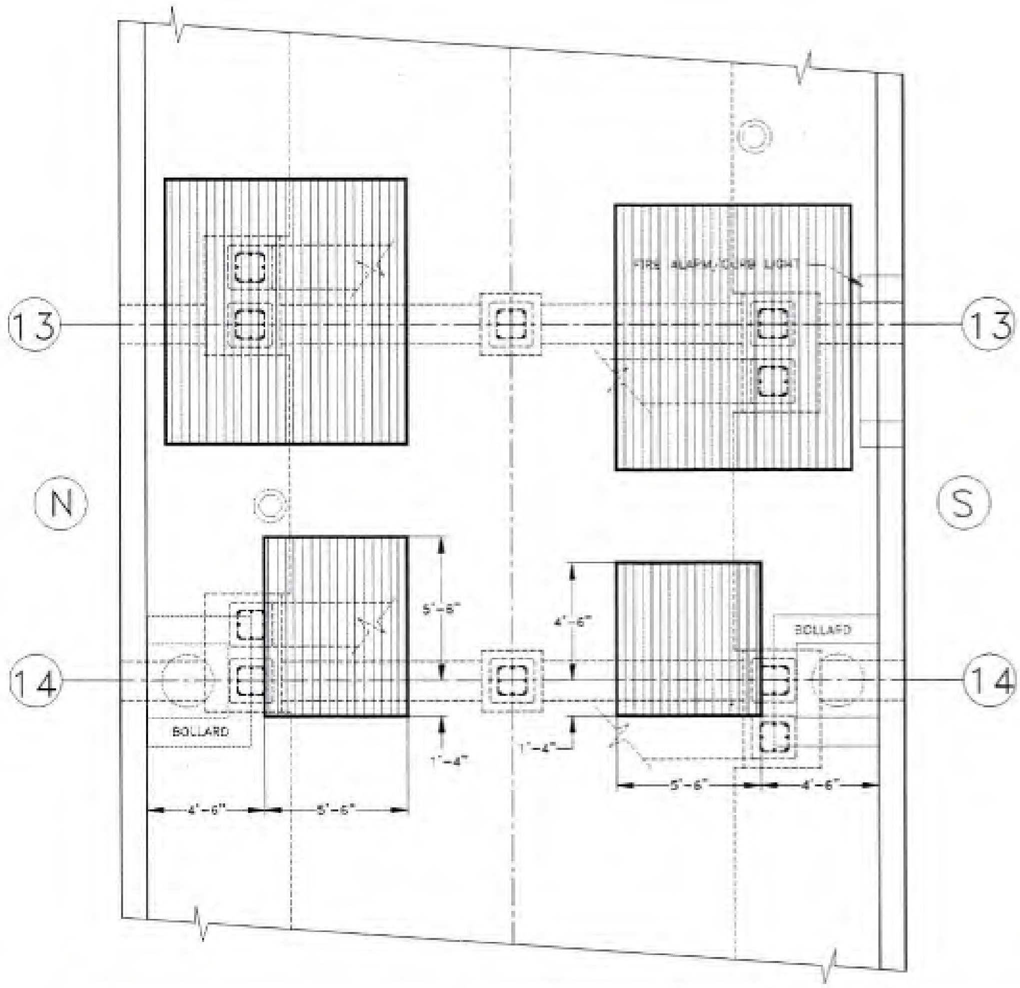
3/8" DFI Diameter Carbon Reinforcing Rods @ 4" O.C	Measured Ultimate Tensile Strength : 220,000 psi @ .92% strain (72.6 kips/in) Recommended Design Limit : 110,000 psi (36.3 kips/in)

Figure A2. Added Reinforcement over Bents 9 and 10.



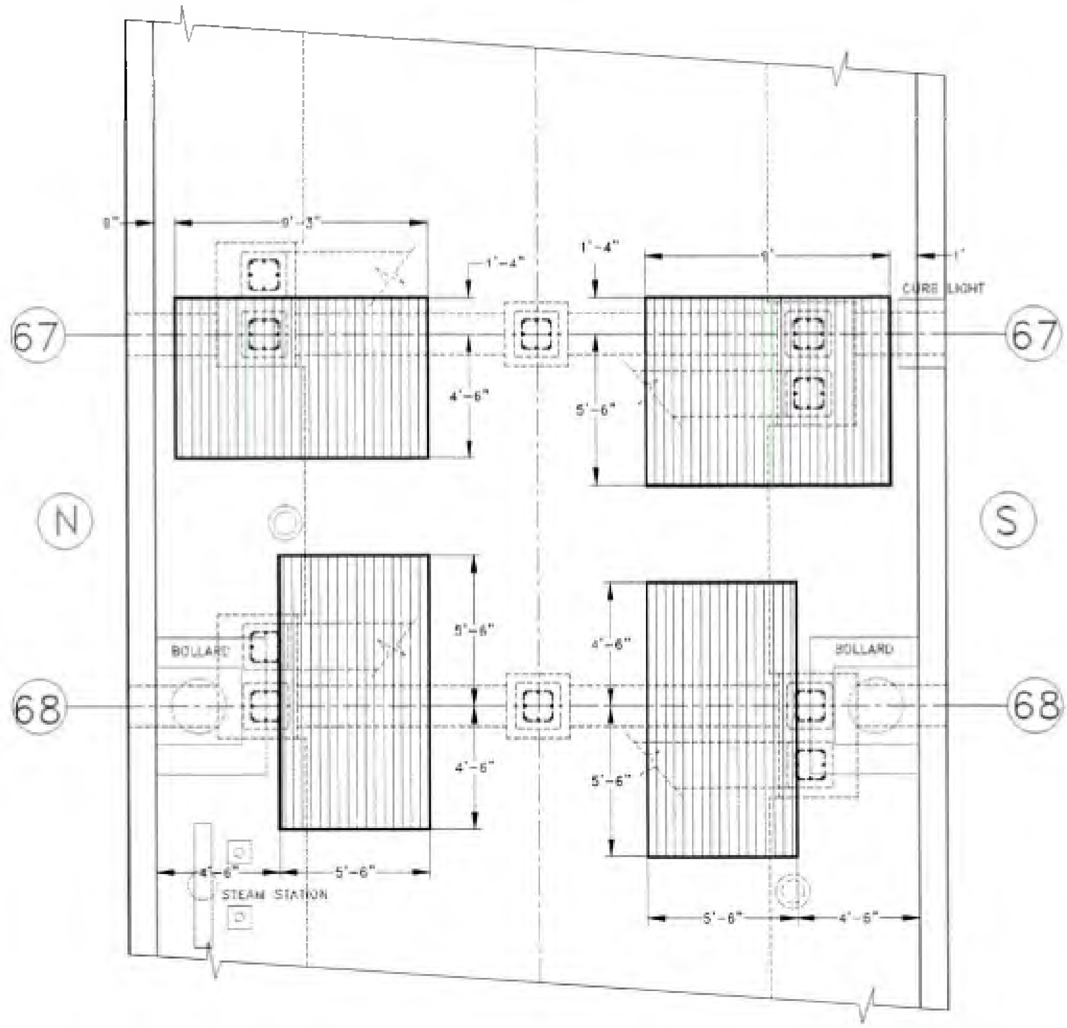
3/8" DFI Diameter Carbon Reinforcing Rods @ 4" O.C	Measured Ultimate Tensile Strength : 220,000 psi @ .92% strain (72.6 kips/in) Recommended Design Limit : 110,000 psi (36.3 kips/in)

Figure A3. Added Reinforcement over Bents 11 and 12.



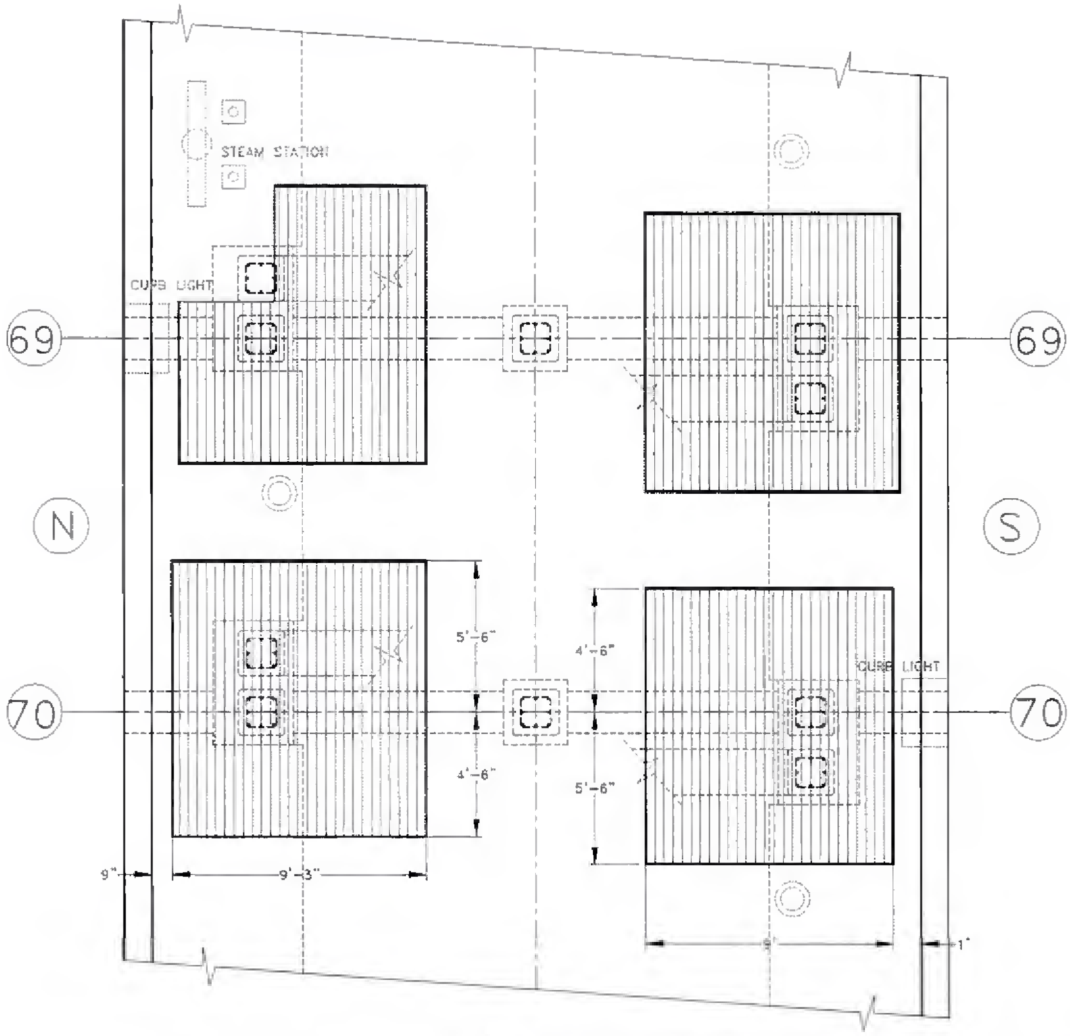
	3/8" DFI Diameter Carbon Reinforcing Rods @ 4" O.C	Measured Ultimate Tensile Strength : 220,000 psi @ .92% strain (72.6 kips/in) Recommended Design Limit : 110,000 psi (36.3 kips/in)

Figure A4. Added Reinforcement over Bents 13 and 14.



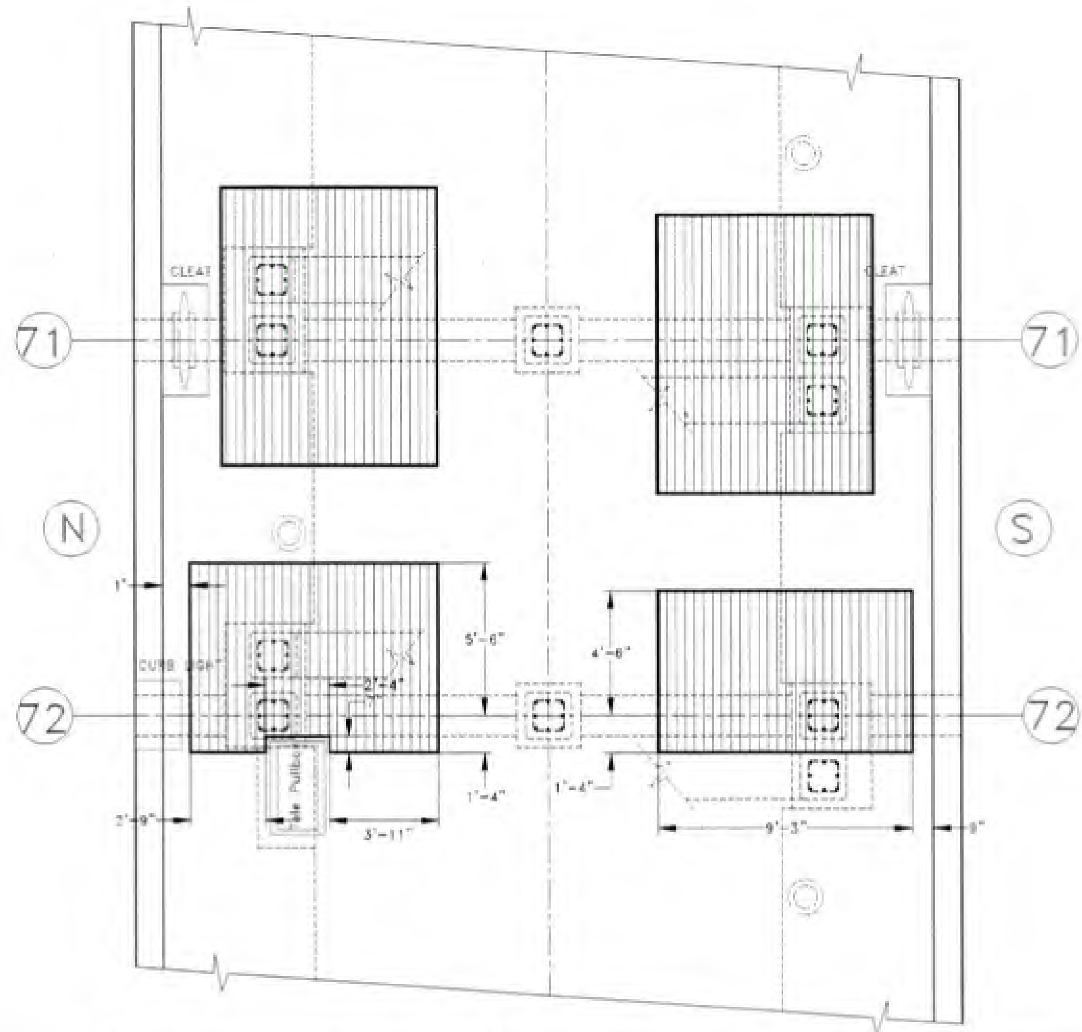
	3/8" DFI Diameter Carbon Reinforcing Rods @ 4" O.C	Measured Ultimate Tensile Strength : 220,000 psi @ .92 % strain (72.6 kips/in) Recommended Design Limit : 110,000 psi (36.3 kips/in)

Figure A5. Added Reinforcement over Bents 67 and 68.



3/8" DFI Diameter Carbon Reinforcing Rods @ 4" O.C	Measured Ultimate Tensile Strength : 220,000 psi @ .92% strain (72.6 kips/in) Recommended Design Limit : 110,000 psi (36.3 kips/in)

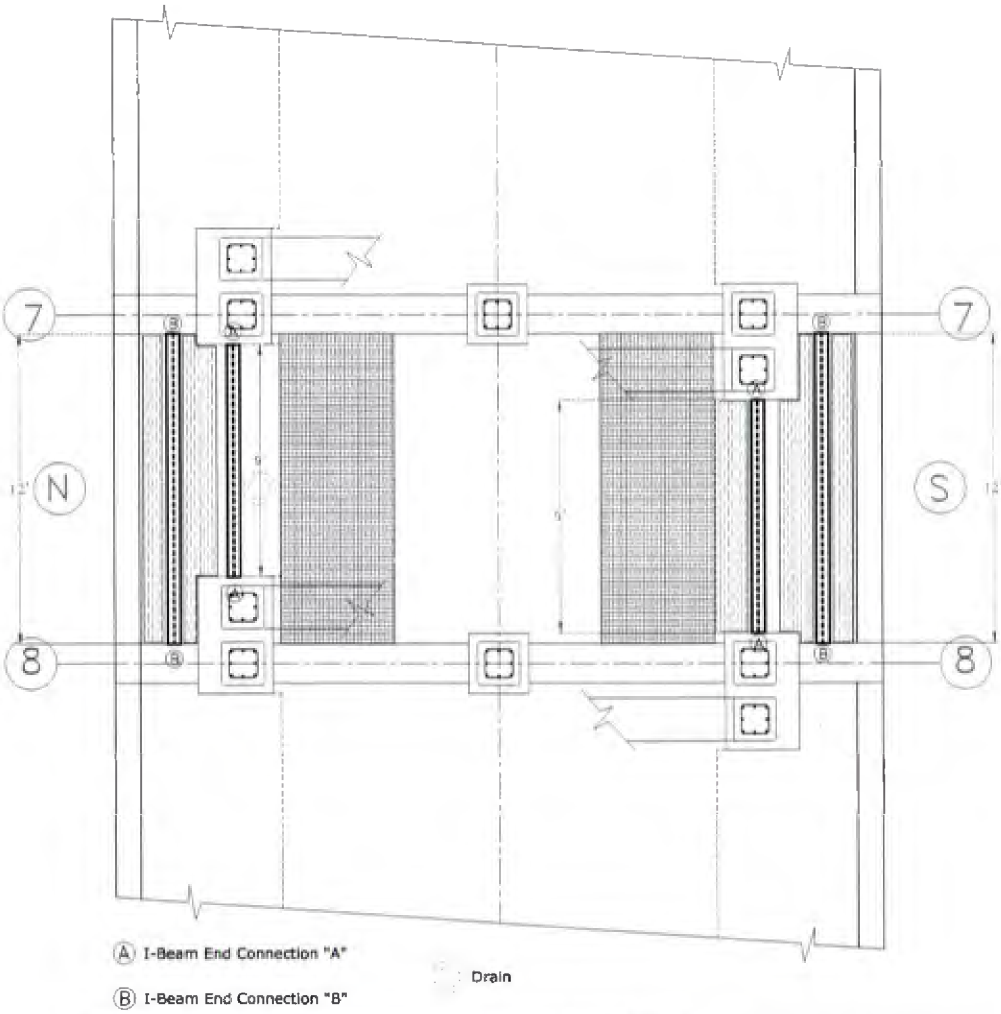
Figure A6. Added Reinforcement over Bents 69 and 70.



3/8" DFI Diameter Carbon Reinforcing Rods @ 4" O.C	Measured Ultimate Tensile Strength : 220,000 psi @ .92 % strain (72.6 kips/in) Recommended Design Limit : 110,000 psi (36.3 kips/in)

Figure A7. Added Reinforcement over Bents 71 and 72.

Reinforcement Added to Bottom Surface of Deck:

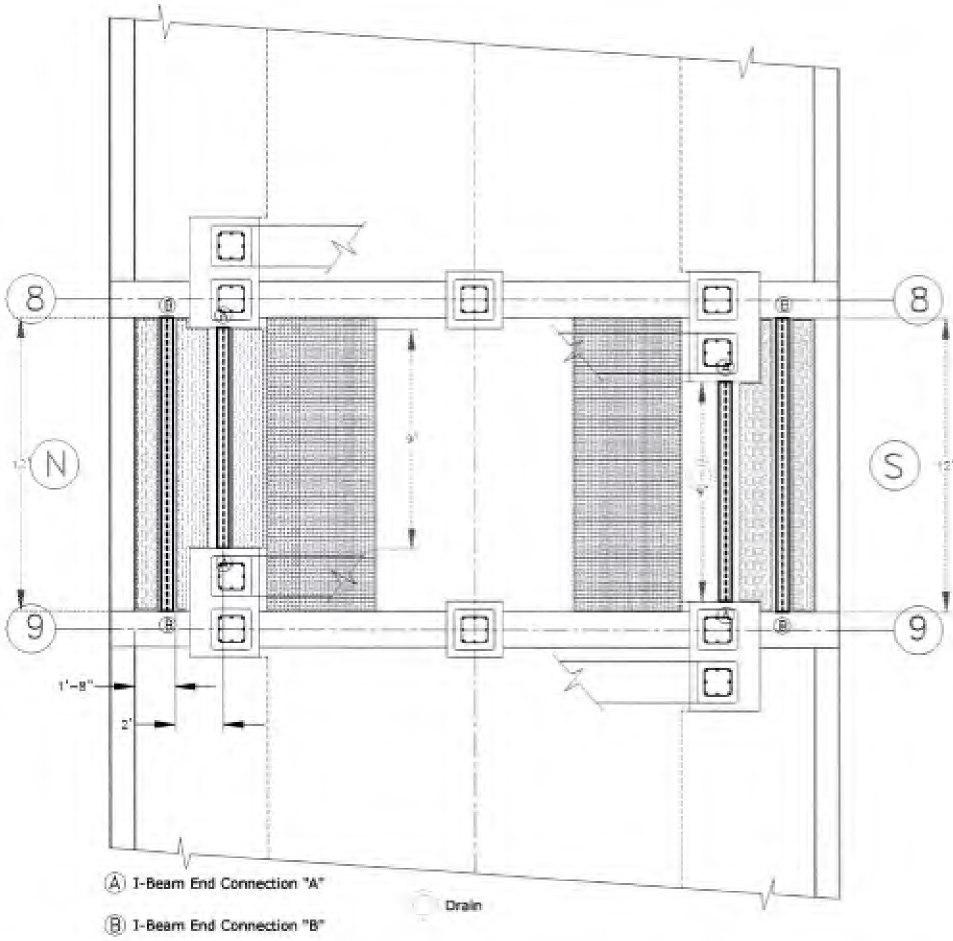


	Strongwell 12" Extreme I-Beam E-Fiberglass/Vinyl Ester Composite I=254.1 in^4 A=11.51 in^2	Measured Tensile Strength : 34,000 psi @ 1.4% strain Recommended Design Limit : 12,000 psi @ 0.05% strain Maximum Moment : 1269 in-kips Maximum Shear : 34.5 kips

	Carbon Composite System A Sika Carbodur 0.03 in^2/in (0.36 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

	Carbon Composite System B Fyfe Tyfo SCH 41 0.05 in^2/in (0.61 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Figure A8. Added Reinforcement between Bents 7 and 8.

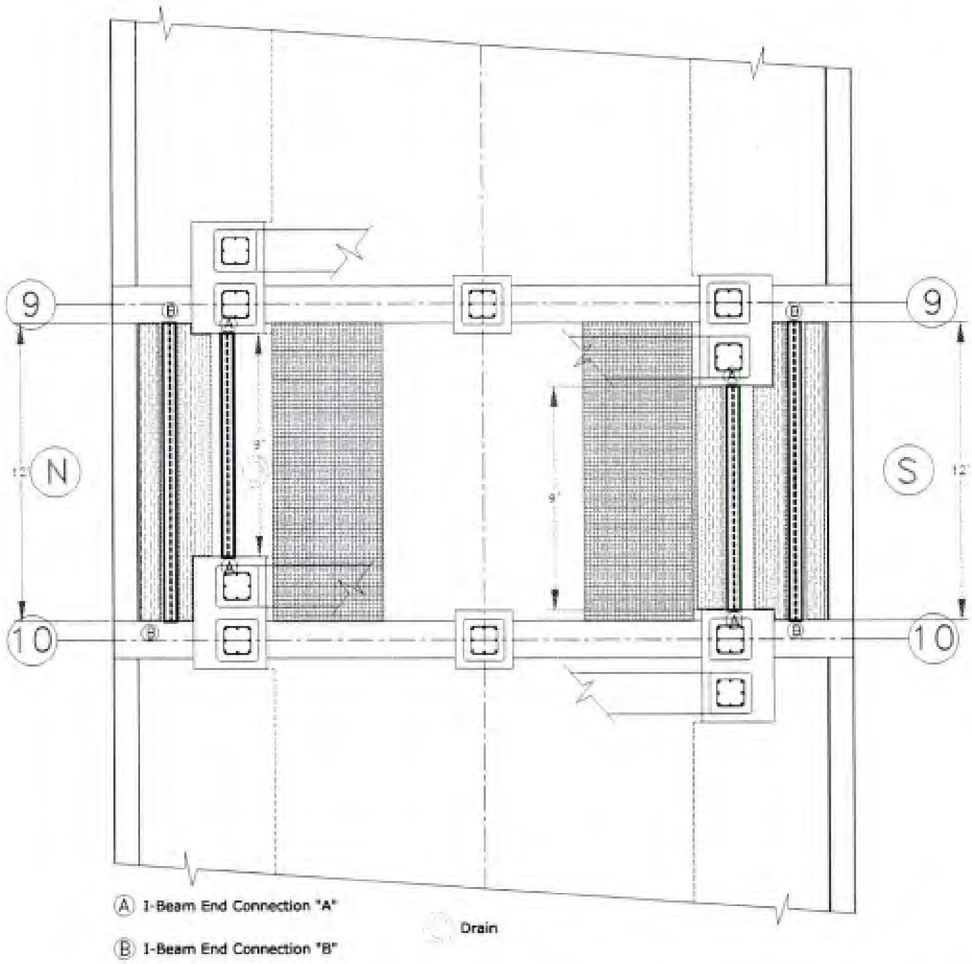


Strongwell 12" Extreme I-Beam E-Fiberglass/Vinyl Ester Composite $I=254.1\ in^4$ $A=11.51\ in^2$	Measured Tensile Strength : 34,000 psi @ 1.4% strain Recommended Design Limit : 12,000 psi @ 0.05% strain Maximum Moment : 1269 in-kips Maximum Shear : 34.5 kips

Carbon Composite System A Sika Carbodur 0.03 in^2/in (0.36 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Carbon Composite System B Fyfe Tyfo SCH 41 0.05 in^2/in (0.61 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Figure A9. Added Reinforcement between Bents 8 and 9.

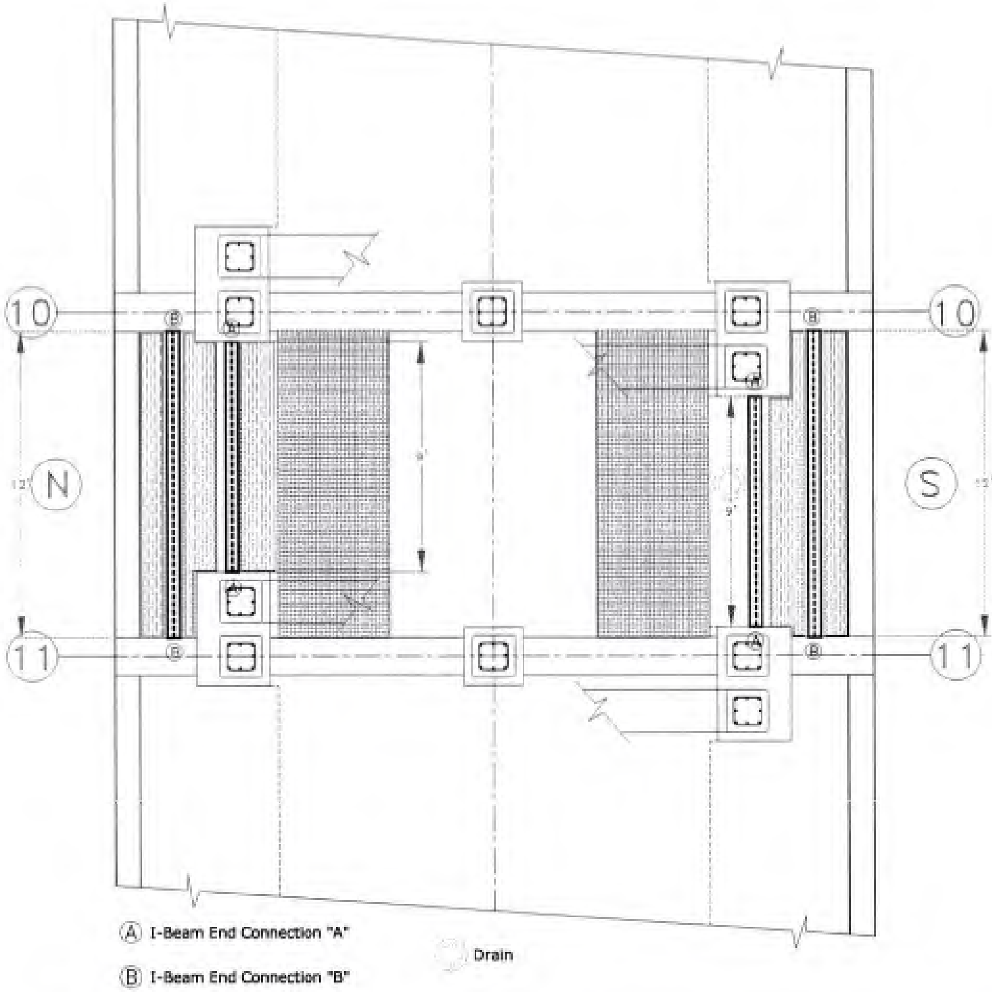


Strongwell 12" Extreme I-Beam E-Fiberglass/Vinyl Ester Composite $I=254.1\ in^4$ $A=11.51\ in^2$	Measured Tensile Strength : 34,000 psi @ 1.4% strain Recommended Design Limit : 12,000 psi @ 0.05% strain Maximum Moment : 1269 in-kips Maximum Shear : 34.5 kips

Carbon Composite System A Sika Carbodur $0.03\ in^2/in$ ($0.36\ in^2/ft$) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Carbon Composite System B Fyfe Tyfo SCH 41 $0.05\ in^2/in$ ($0.61\ in^2/ft$) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Figure A10. Added Reinforcement between Bents 9 and 10.

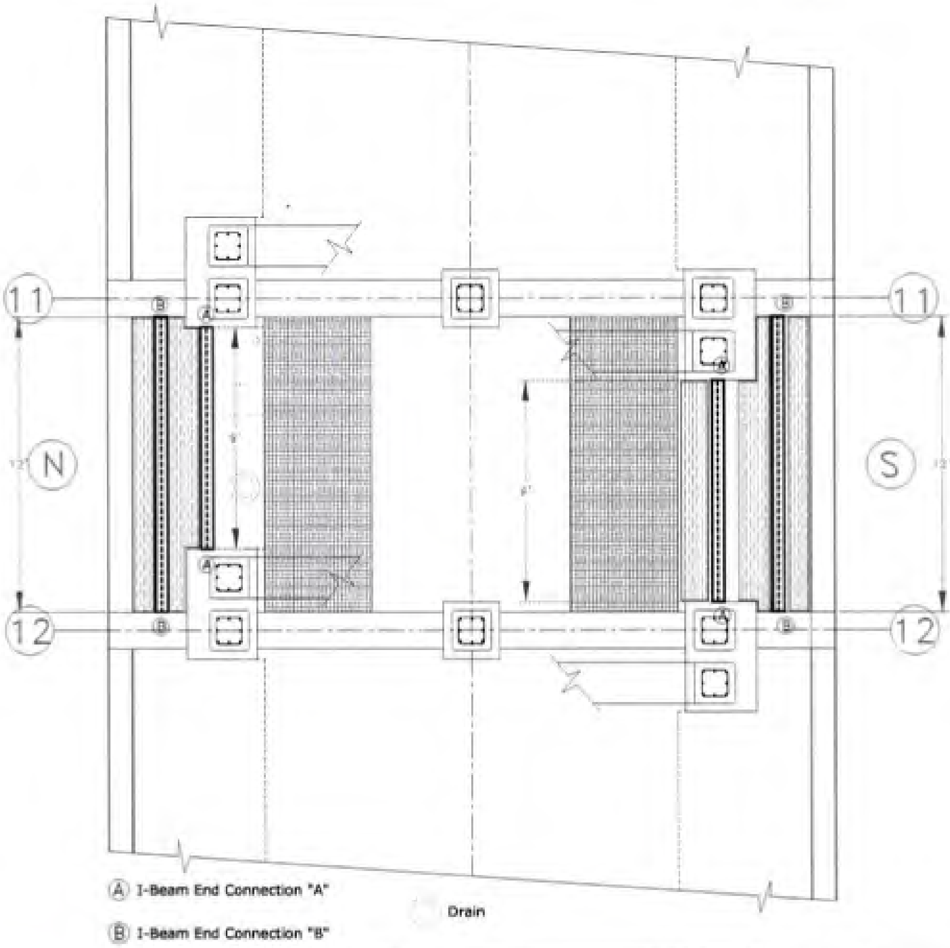


Strongwell 12" Extreme I-Beam E-Fiberglass/Vinyl Ester Composite I=254.1 in^4 A=11.51 in^2	Measured Tensile Strength : 34,000 psi @ 1.4% strain Recommended Design Limit : 12,000 psi @ 0.05% strain Maximum Moment : 1269 in-kips Maximum Shear : 34.5 kips

Carbon Composite System A Sika Carbodur 0.03 in^2/in (0.36 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Carbon Composite System B Fyfe Tyfo SCH 41 0.05 in^2/in (0.61 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Figure A11. Added Reinforcement between Bents 10 and 11.

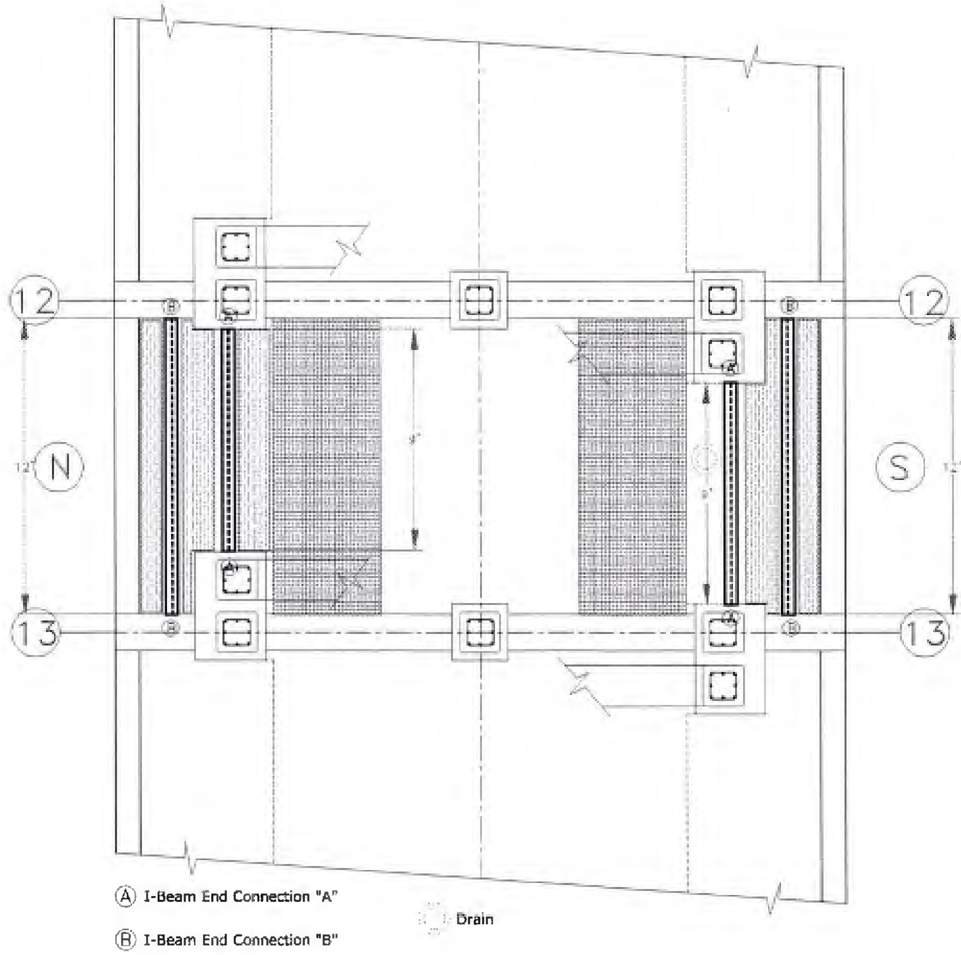


Strongwell 12" Extreme I-Beam E-Fiberglass/Vinyl Ester Composite $I=254.1\ in^4$ $A=11.51\ in^2$	Measured Tensile Strength : 34,000 psi @ 1.4% strain Recommended Design Limit : 12,000 psi @ 0.05% strain Maximum Moment : 1269 in-kips Maximum Shear : 34.5 kips

Carbon Composite System A Sika Carbodur 0.03 in^2/in (0.36 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Carbon Composite System B Fyfe Tyfo SCH 41 0.05 in^2/in (0.61 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Figure A12. Added Reinforcement between Bents 11 and 12.

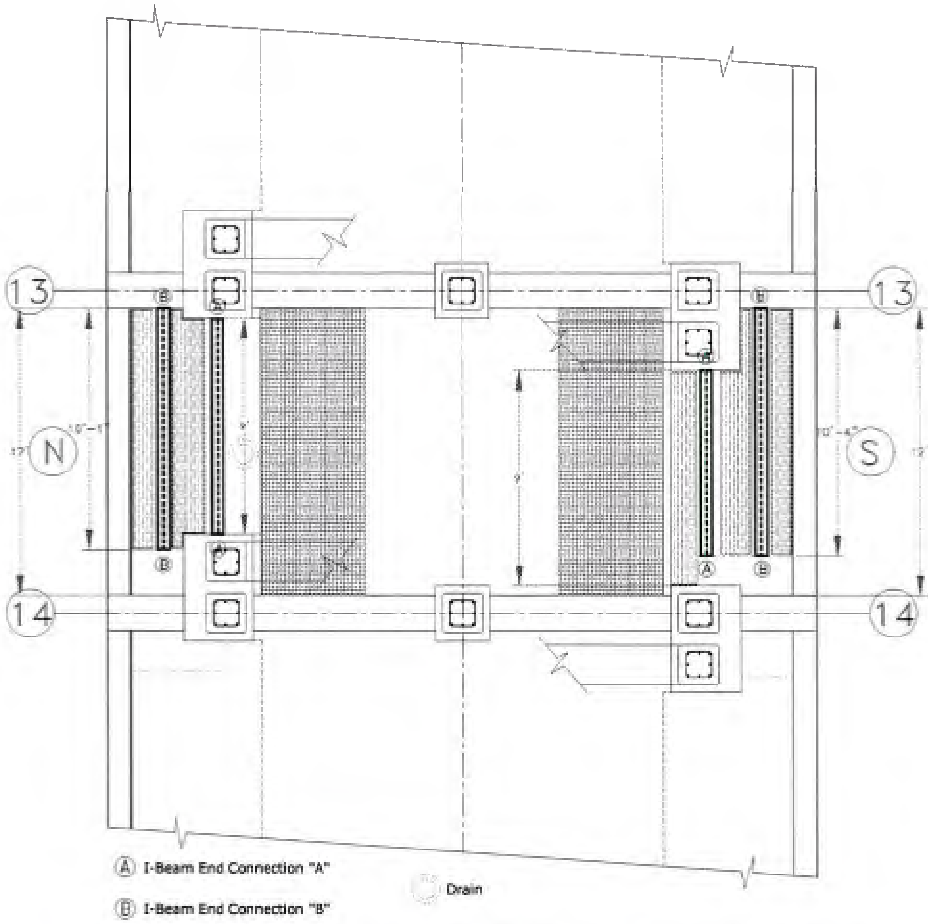


Strongwell 12" Extreme I-Beam E-Fiberglass/Vinyl Ester Composite I=254.1 in^4 A=11.51 in^2	Measured Tensile Strength : 34,000 psi @ 1.4% strain Recommended Design Limit : 12,000 psi @ 0.05% strain Maximum Moment : 1269 in-kips Maximum Shear : 34.5 kips

Carbon Composite System A Sika Carbodur 0.03 in^2/in (0.36 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Carbon Composite System B Fyfe Tyfo SCH 41 0.05 in^2/in (0.61 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Figure A13. Added Reinforcement between Bents 12 and 13.

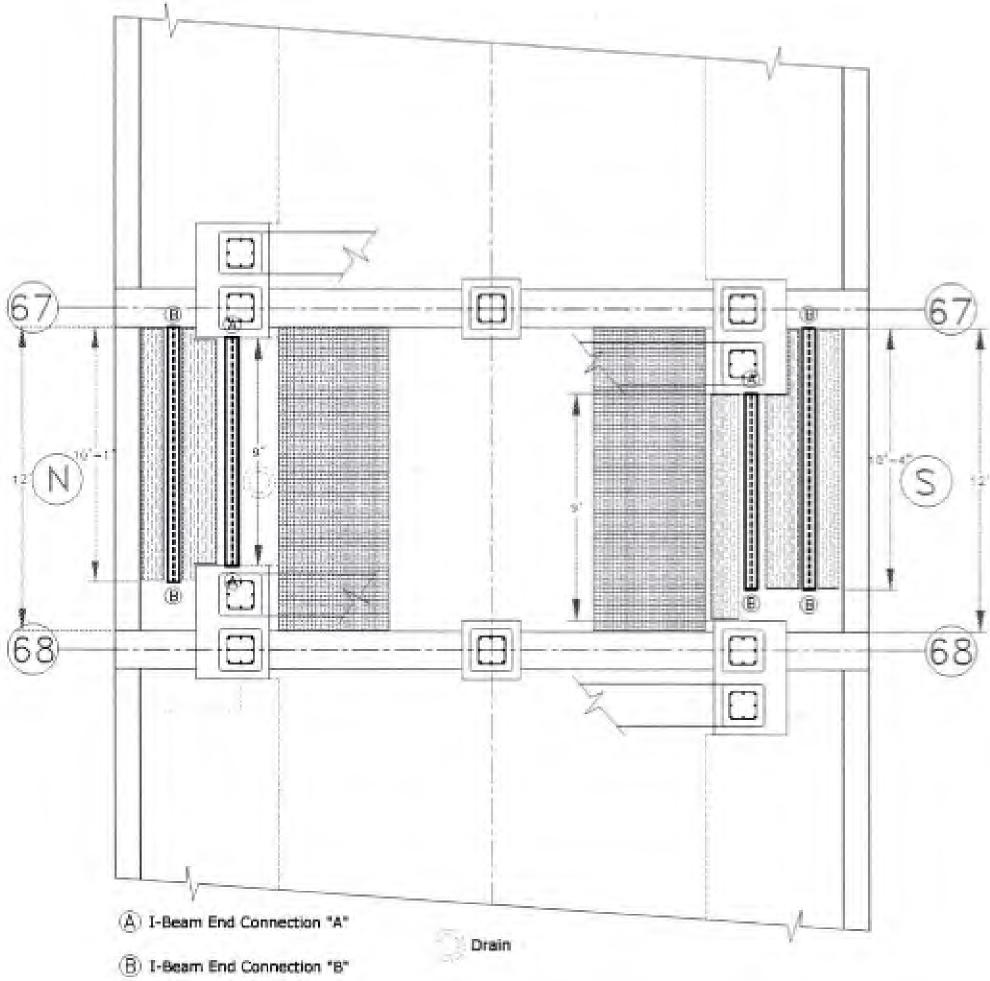


Strongwell 12" Extreme I-Beam E-Fiberglass/Vinyl Ester Composite $I=254.1\ in^4$ $A=11.51\ in^2$	Measured Tensile Strength : 34,000 psi @ 1.4% strain Recommended Design Limit : 12,000 psi @ 0.05% strain Maximum Moment : 1269 in-kips Maximum Shear : 34.5 kips

Carbon Composite System A Sika Carbodur 0.03 in^2/in (0.36 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Carbon Composite System B Fyfe Tyfo SCH 41 0.05 in^2/in (0.61 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Figure A14. Added Reinforcement between Bents 13 and 14.

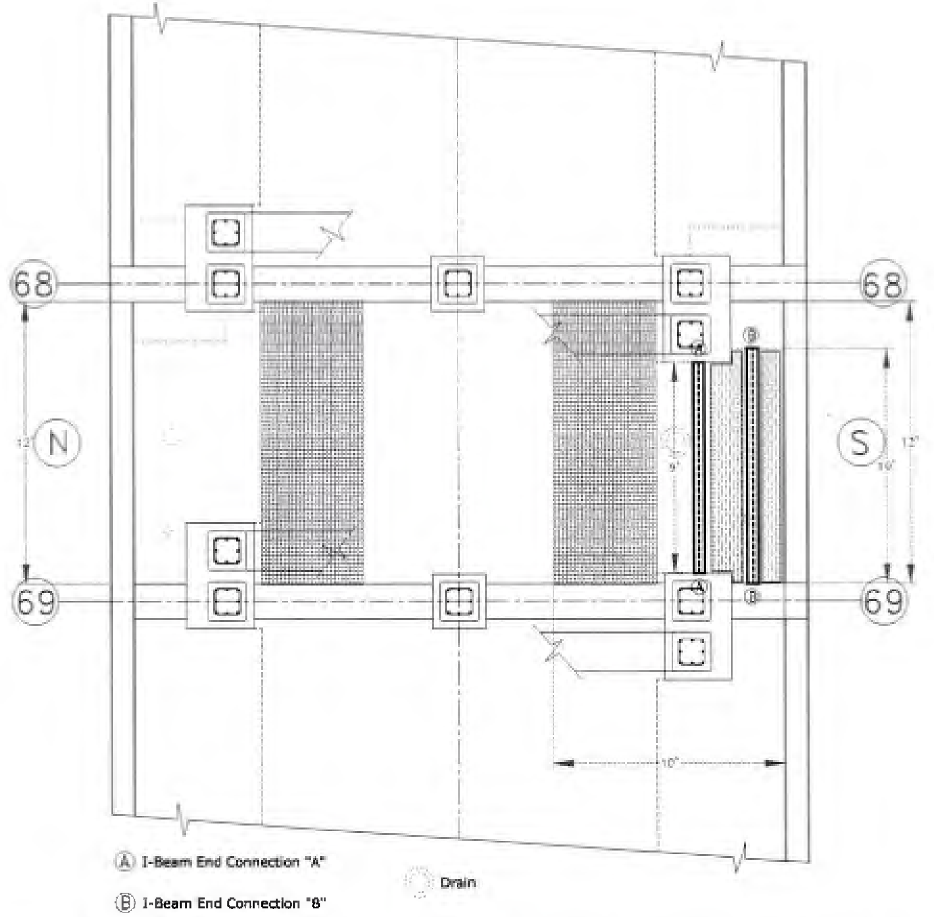


Strongwell 12" Extreme I-Beam E-Fiberglass/Vinyl Ester Composite $I=254.1\ in^4$ $A=11.51\ in^2$	Measured Tensile Strength : 34,000 psi @ 1.4% strain Recommended Design Limit : 12,000 psi @ 0.05% strain Maximum Moment : 1269 in-kips Maximum Shear : 34.5 kips

Carbon Composite System A Sika Carbodur 0.03 in^2/in (0.36 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Carbon Composite System B Fyfe Tyfo SCH 41 0.05 in^2/in (0.61 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Figure A15. Added Reinforcement between Bents 67 and 68.

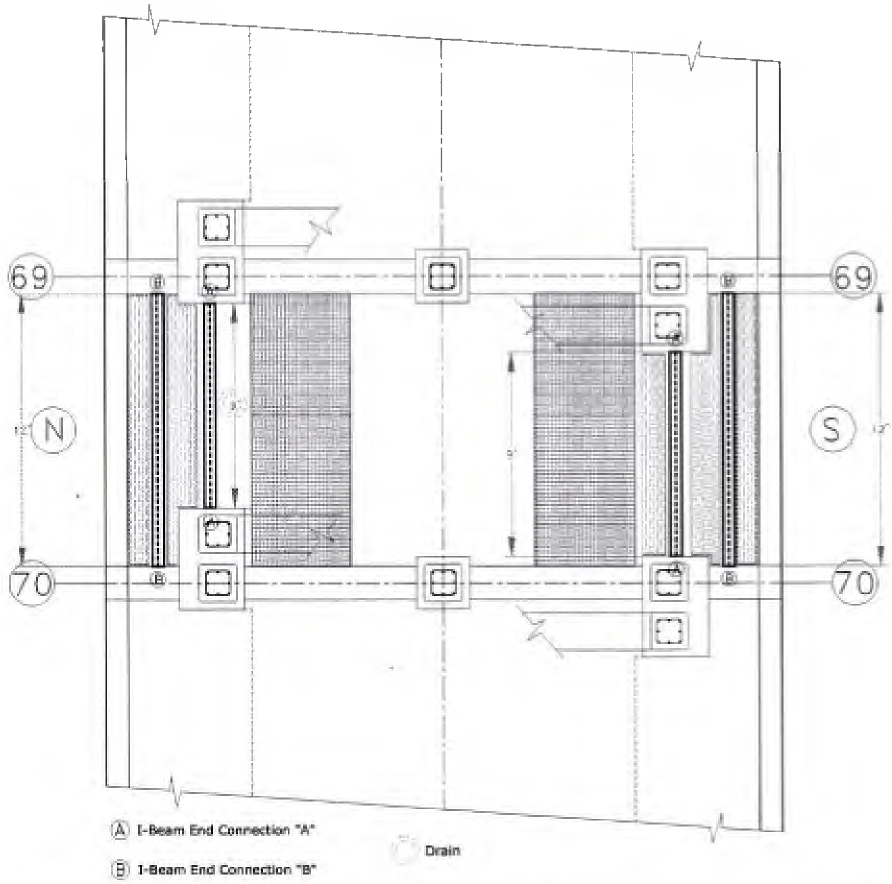


Strongwell 12" Extreme I-Beam E-Fiberglass/Vinyl Ester Composite $I=254.1\ in^4$ $A=11.51\ in^2$	Measured Tensile Strength : 34,000 psi @ 1.4% strain Recommended Design Limit : 12,000 psi @ 0.05% strain Maximum Moment : 1269 in-kips Maximum Shear : 34.5 kips

Carbon Composite System A Sika Carbodur 0.03 in^2/in (0.36 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Carbon Composite System B Fyfe Tyfo SCH 41 0.05 in^2/in (0.61 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Figure A16. Added Reinforcement between Bents 68 and 69.

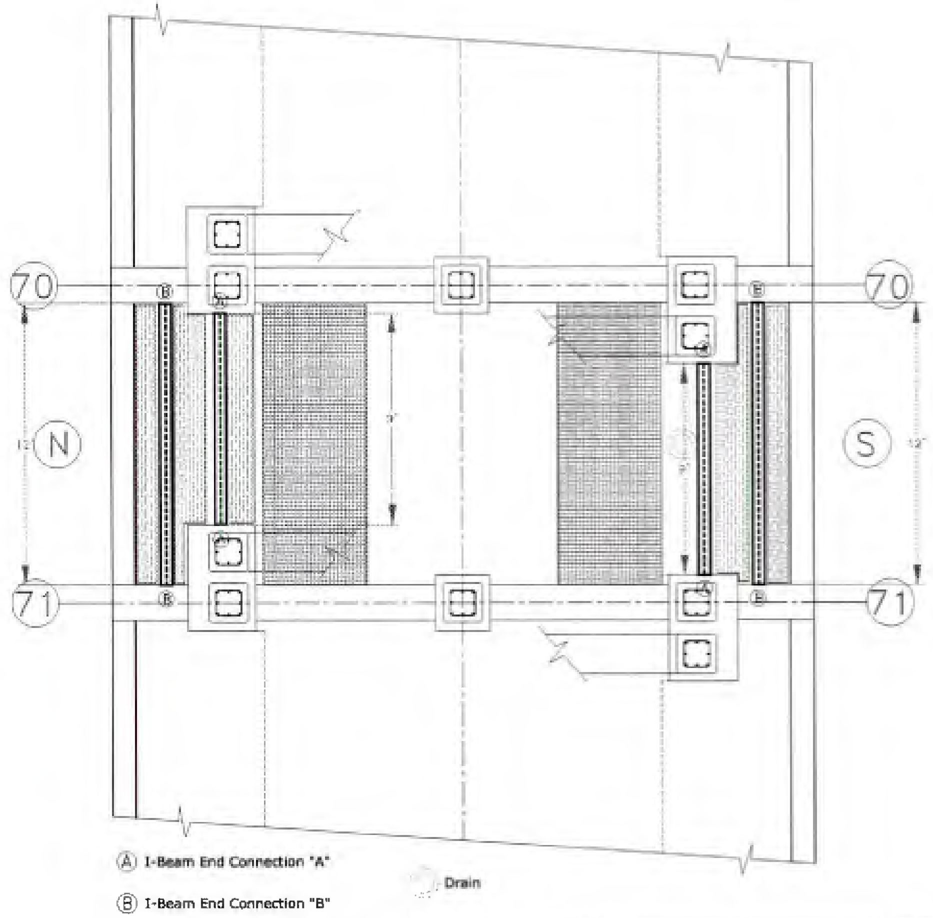


Strongwell 12" Extreme I-Beam	Measured Tensile Strength : 34,000 psi @ 1.4% strain
E-Fiberglass/Vinyl Ester Composite	Recommended Design Limit : 12,000 psi @ 0.05% strain
$I=254.1\ in^4$	Maximum Moment : 1269 in-kips
$A=11.51\ in^2$	Maximum Shear : 34.5 kips

Carbon Composite System A Sika Carbodur 0.03 in^2/in (0.36 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Carbon Composite System B Fyfe Tyfo SCH 41 0.05 in^2/in (0.61 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Figure A17. Added Reinforcement between Bents 69 and 70.

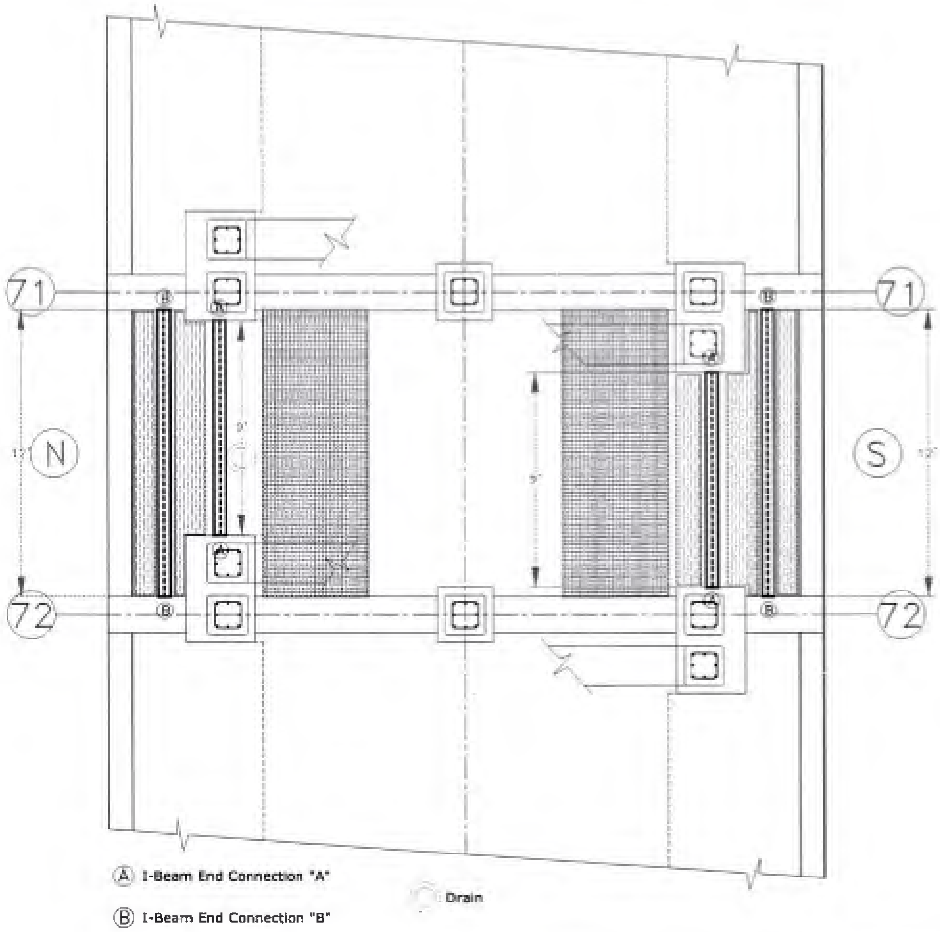


Strongwell 12" Extreme I-Beam E-Fiberglass/Vinyl Ester Composite $I=254.1\ in^4$ $A=11.51\ in^2$	Measured Tensile Strength : 34,000 psi @ 1.4% strain Recommended Design Limit : 12,000 psi @ 0.05% strain Maximum Moment : 1269 in-kips Maximum Shear : 34.5 kips

Carbon Composite System A Sika Carbodur 0.03 in^2/in (0.36 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Carbon Composite System B Fyfe Tyfo SCH 41 0.05 in^2/in (0.61 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Figure A18. Added Reinforcement between Bents 70 and 71.

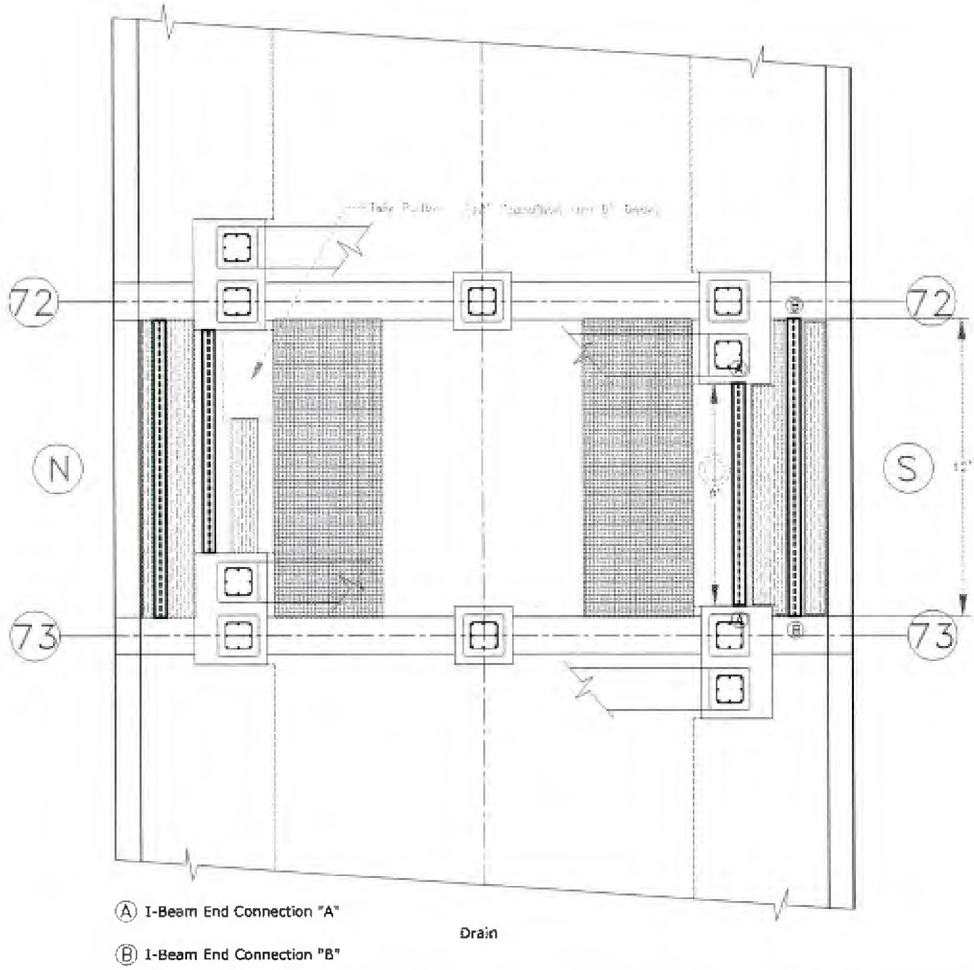


Strongwell 12" Extreme I-Beam E-Fiberglass/Vinyl Ester Composite I=254.1 in^4 A=11.51 in^2	Measured Tensile Strength : 34,000 psi @ 1.4% strain Recommended Design Limit : 12,000 psi @ 0.05% strain Maximum Moment : 1269 in-kips Maximum Shear : 34.5 kips

Carbon Composite System A Sika Carbodur 0.03 in^2/in (0.36 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Carbon Composite System B Fyfe Tyfo SCH 41 0.05 in^2/in (0.61 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Figure A19. Added Reinforcement between Bents 71 and 72.



Strongwell 12" Extreme I-Beam E-Fiberglass/Vinyl Ester Composite $I=254.1\ in^4$ $A=11.51\ in^2$	Measured Tensile Strength : 34,000 psi @ 1.4% strain Recommended Design Limit : 12,000 psi @ 0.05% strain Maximum Moment : 1269 in-kips Maximum Shear : 34.5 kips

Carbon Composite System A Sika Carbodur 0.03 in^2/in (0.36 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

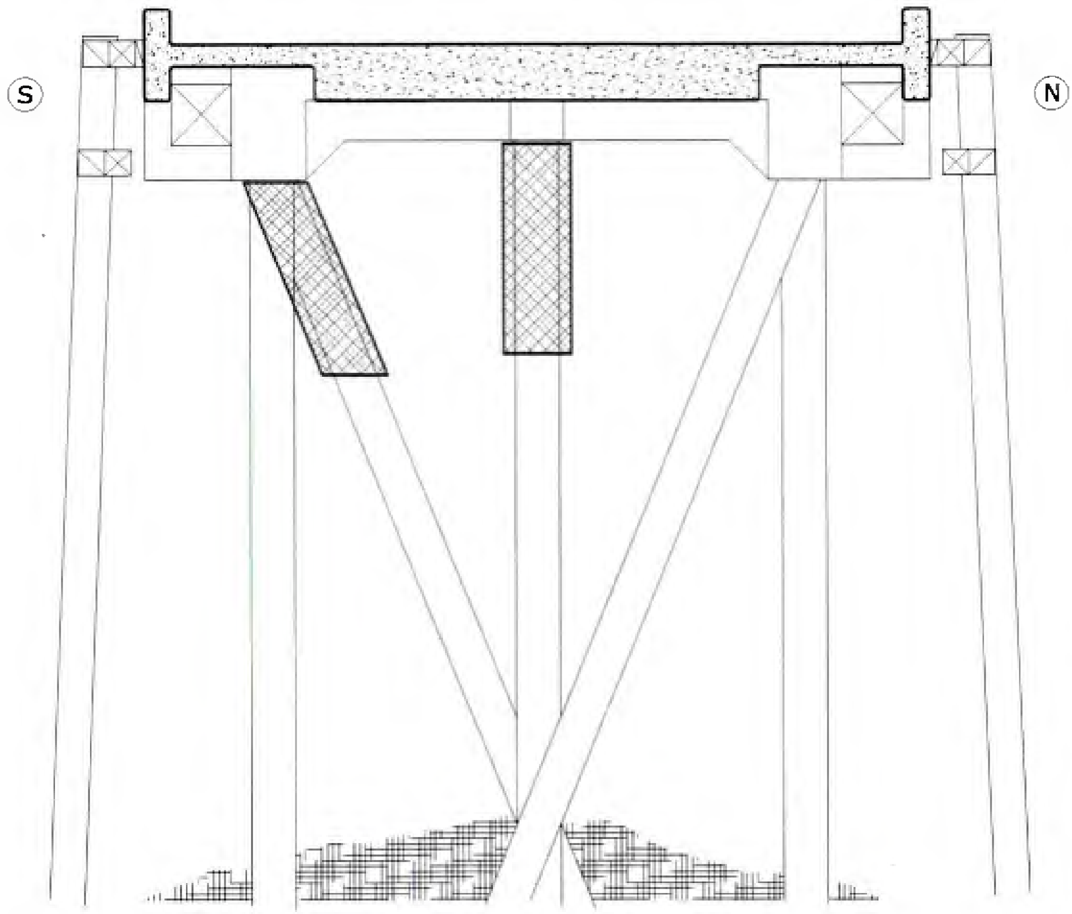
Carbon Composite System B Fyfe Tyfo SCH 41 0.05 in^2/in (0.61 in^2/ft) Carbon Fiber Content	Measured Ultimate Strength : 21 kips/in (252 kips/ft) @ 1.3% strain Recommended Design Limit : 7 kips/in (84 kips/ft) @ 0.4% strain

Figure A20. Added Reinforcement between Bents 72 and 73.

Pile Confinement Upgrade

Pile Confinement Reinforcement

Bent 8

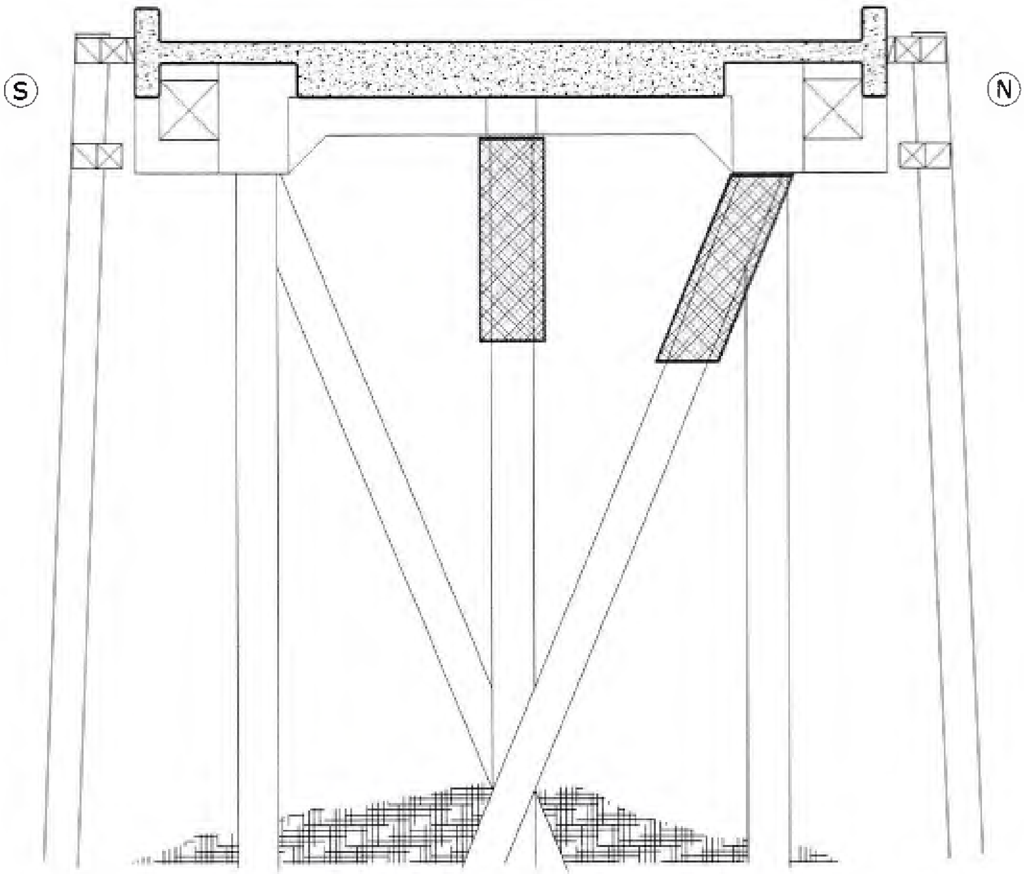


Pile Shell Hardcore Dupont Vinyl Ester - E-Glass Composite	Measured Circumferential Tensile Strength = 43 kips/in @ 2% Strain Recommended Design Limit @ .2 % Strain = 4.3 kips/in Measured Longitudinal Tensile Strength=10 kips/in Recommended Design Limit=1 kips/in Measured Lap Strength=17 kips/in

Figure A21. Bent 8 Pile Confinement.

Pile Confinement Reinforcement

Bent 9

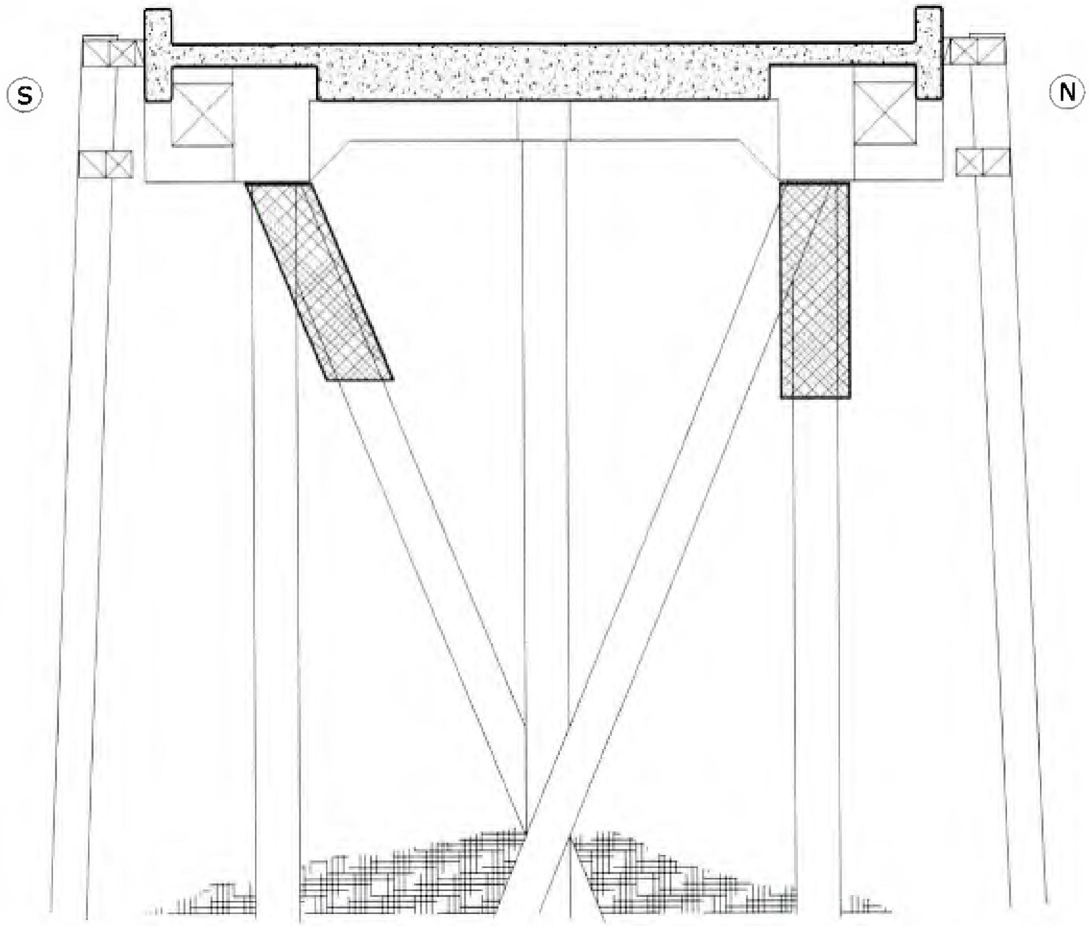


Pile Shell Hardcore Dupont Vinyl Ester - E-Glass Composite	Measured Circumferential Tensile Strength = 43 kips/in @ 2% Strain Recommended Design Limit @ .2 % Strain = 4.3 kips/in Measured Longitudinal Tensile Strength=10 kips/in Recommended Design Limit=1 kips/in Measured Lap Strength=17 kips/in

Figure A22. Bent 9 Pile Confinement.

Pile Confinement Reinforcement

Bent 10

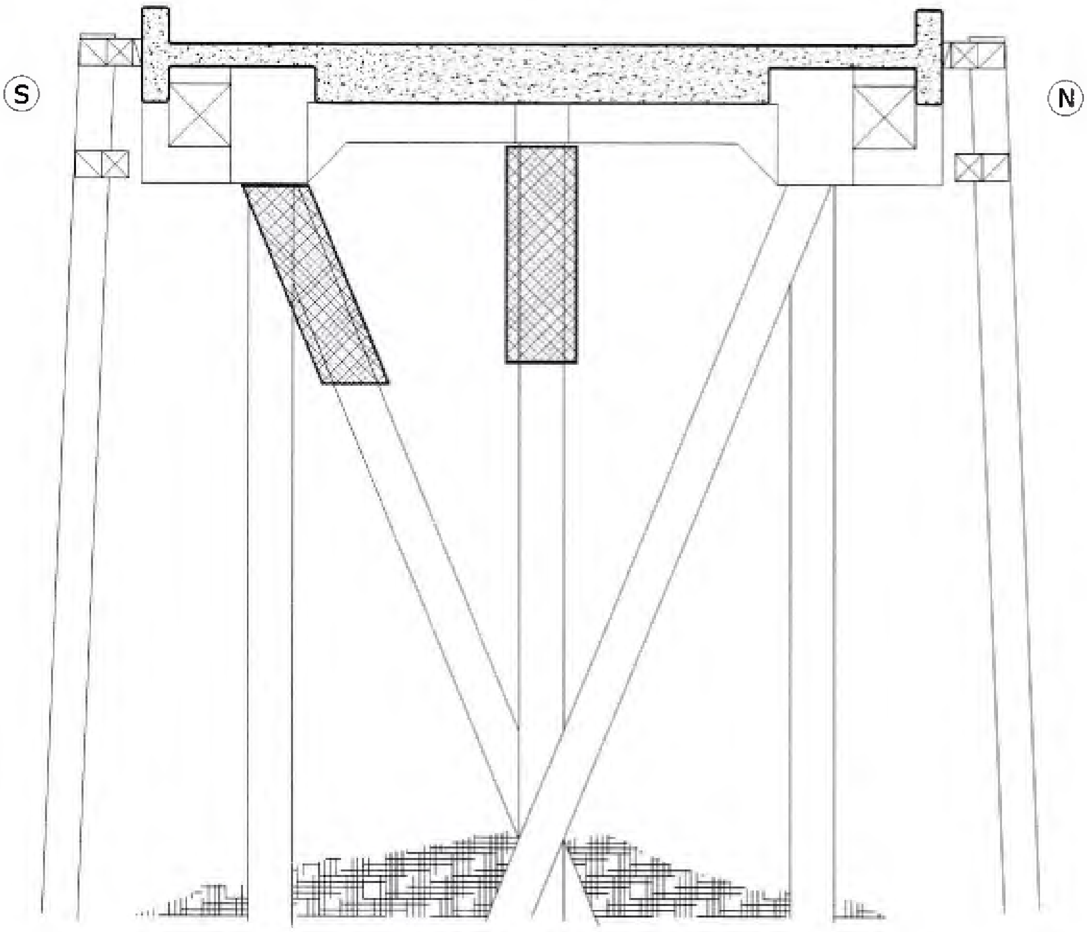


Pile Shell Hardcore Dupont Vinyl Ester - E-Glass Composite	Measured Circumferential Tensile Strength = 43 kips/in @ 2% Strain Recommended Design Limit @ .2 % Strain = 4.3 kips/in Measured Longitudinal Tensile Strength=10 kips/in Recommended Design Limit=1 kips/in Measured Lap Strength=17 kips/in

Figure A23. Bent 10 Pile Confinement.

Pile Confinement Reinforcement

Bent 11

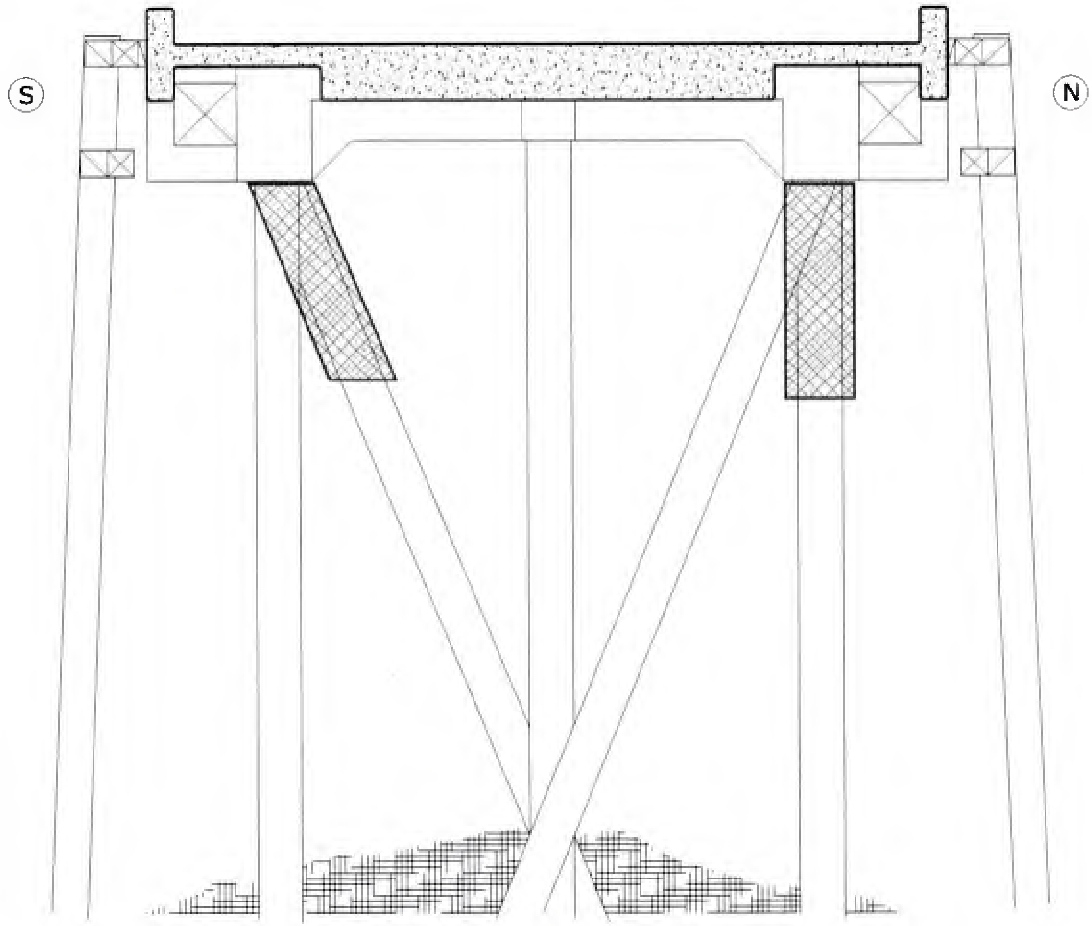


Pile Shell Hardcore Dupont Vinyl Ester - E-Glass Composite	Measured Circumferential Tensile Strength = 43 kips/in @ 2% Strain Recommended Design Limit @ .2 % Strain = 4.3 kips/in Measured Longitudinal Tensile Strength=10 kips/in Recommended Design Limit=1 kips/in Measured Lap Strength=17 kips/in

Figure A24. Bent 11 Pile Confinement.

Pile Confinement Reinforcement

Bent 12

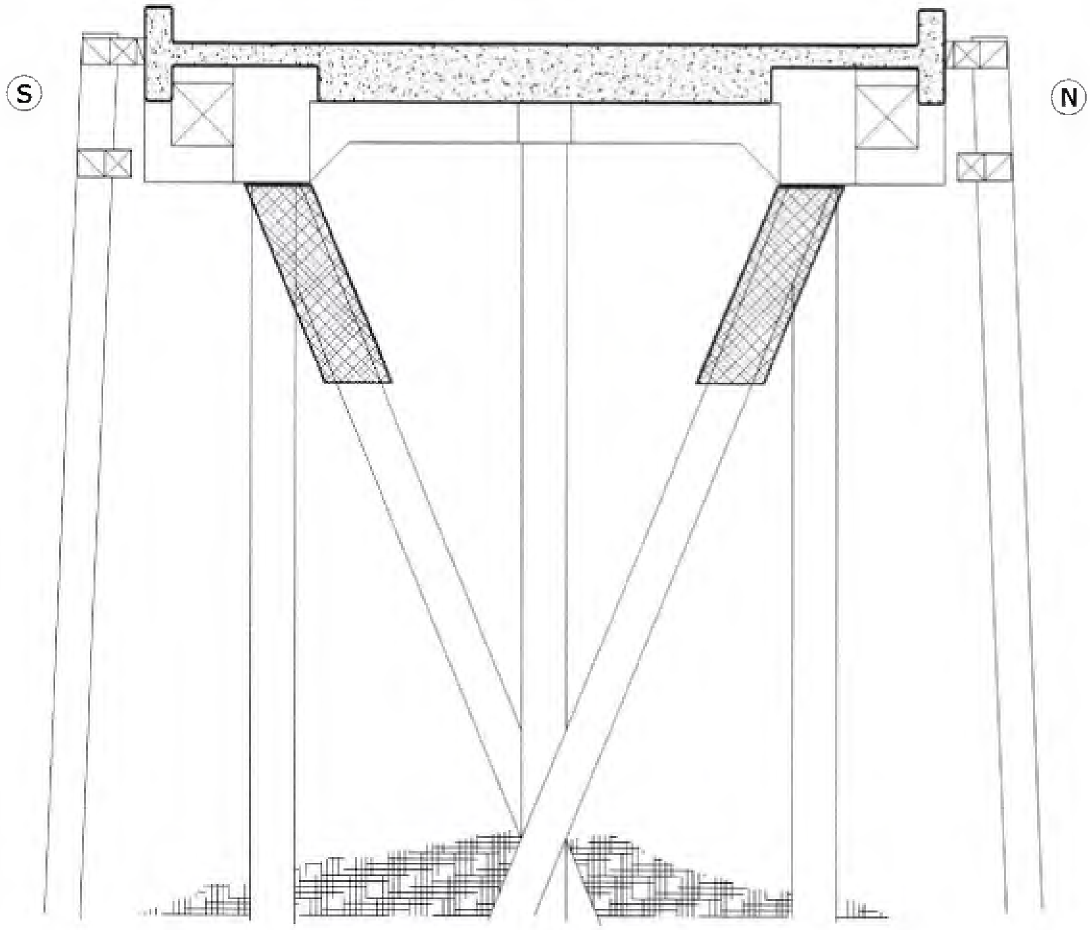


Pile Shell Hardcore Dupont Vinyl Ester - E-Glass Composite	Measured Circumferential Tensile Strength = 43 kips/in @ 2% Strain Recommended Design Limit @ .2 % Strain = 4.3 kips/in Measured Longitudinal Tensile Strength=10 kips/in Recommended Design Limit=1 kips/in Measured Lap Strength=17 kips/in

Figure A25. Bent 12 Pile Confinement.

Pile Confinement Reinforcement

Bent 13

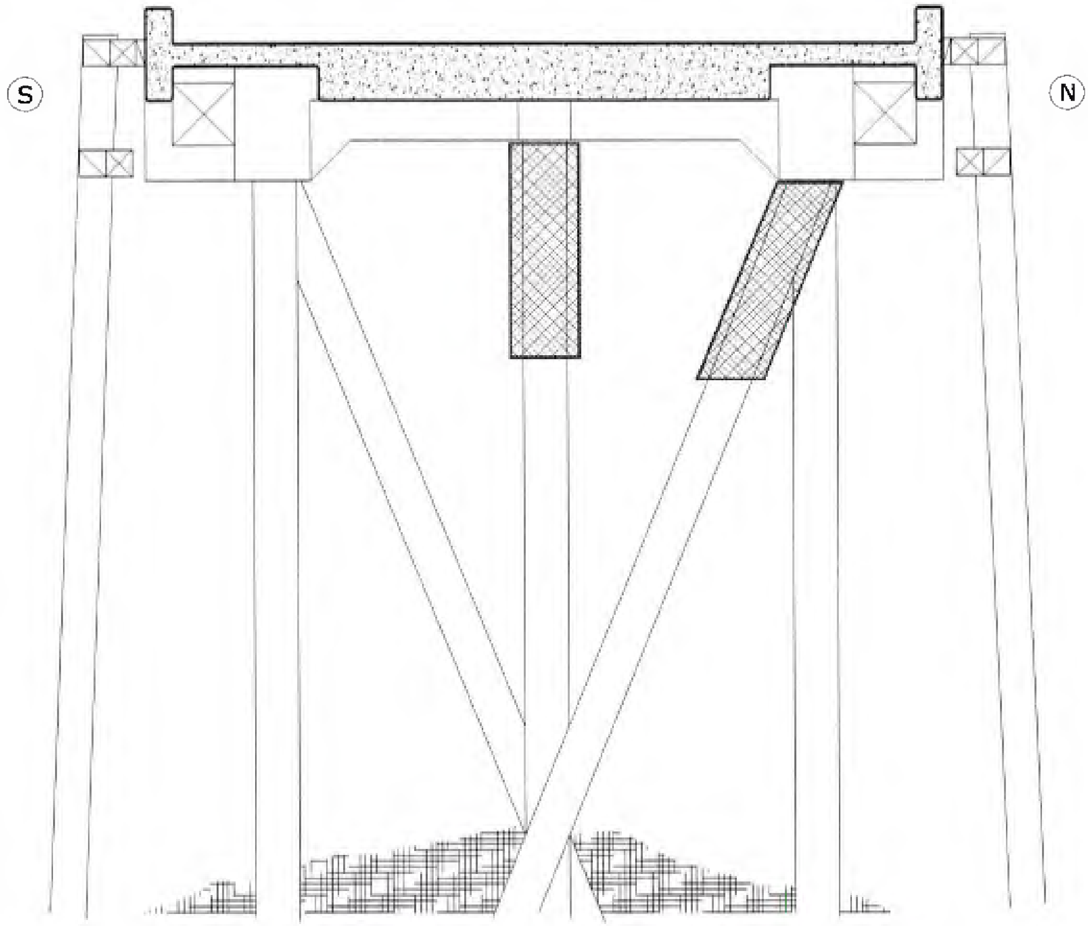


Pile Shell Hardcore Dupont Vinyl Ester - E-Glass Composite	Measured Circumferential Tensile Strength = 43 kips/in @ 2% Strain Recommended Design Limit @ .2 % Strain = 4.3 kips/in. Measured Longitudinal Tensile Strength=10 kips/in Recommended Design Limit=1 kips/in Measured Lap Strength=17 kips/in

Figure A26. Bent 13 Pile Confinement.

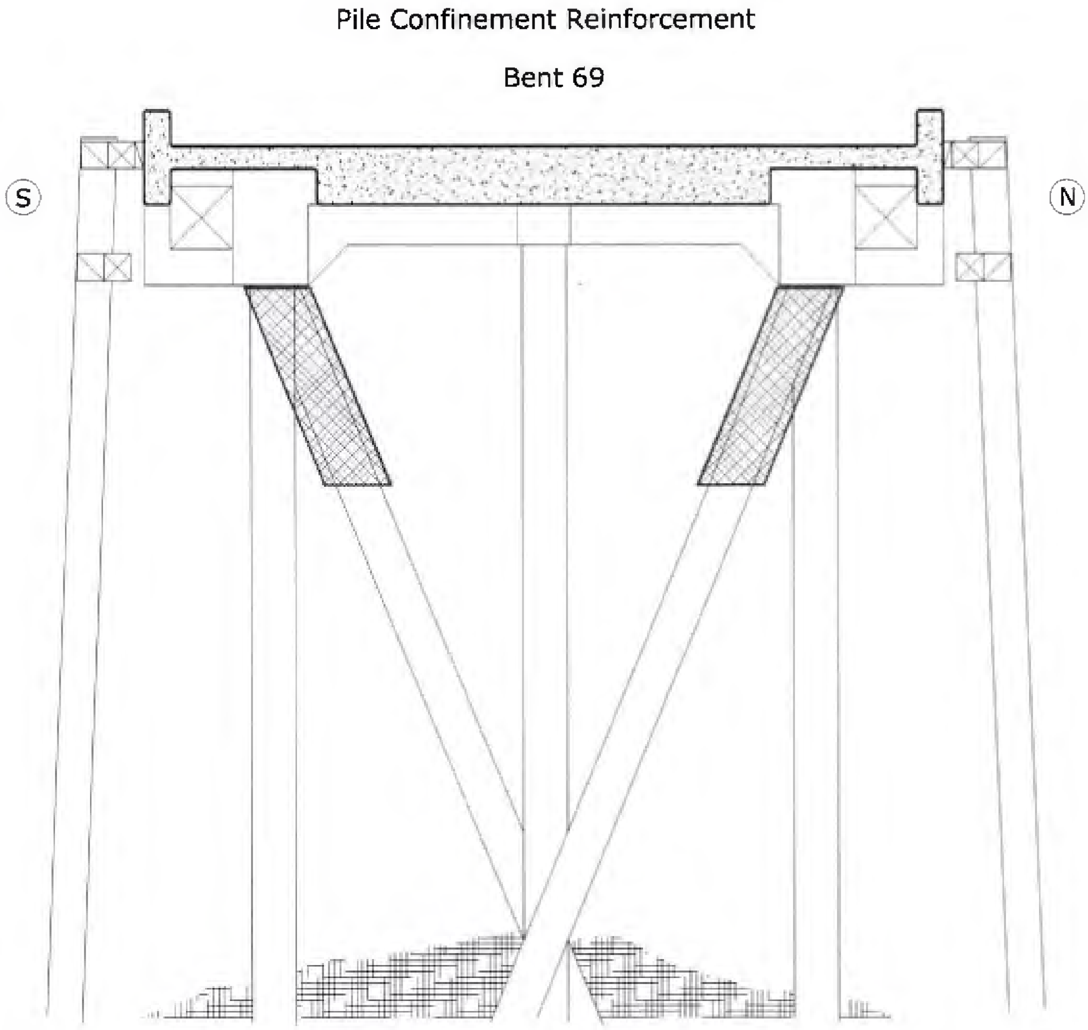
Pile Confinement Reinforcement

Bent 68



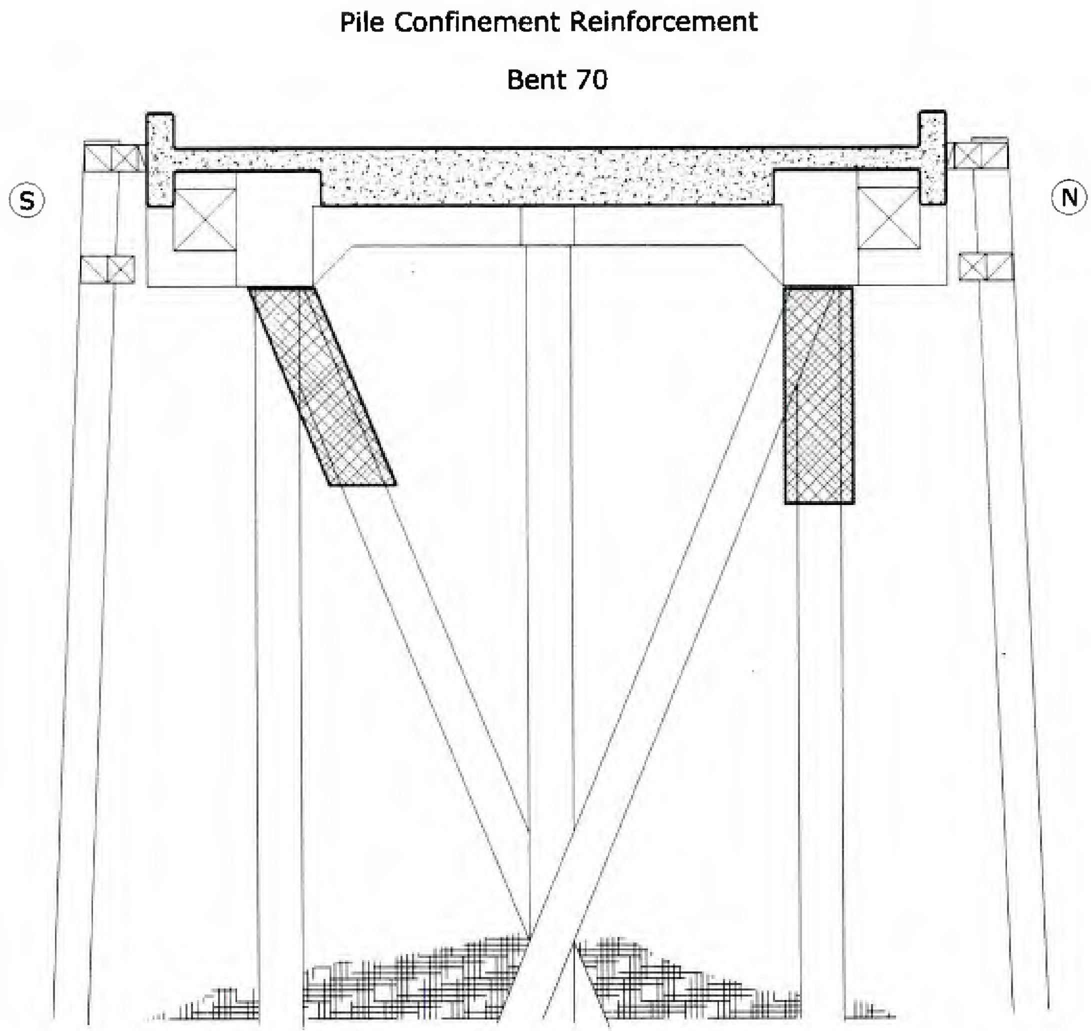
Pile Shell Hardcore Dupont Vinyl Ester - E-Glass Composite	Measured Circumferential Tensile Strength = 43 kips/in @ 2% Strain Recommended Design Limit @ .2 % Strain = 4.3 kips/in Measured Longitudinal Tensile Strength=10 kips/in Recommended Design Limit=1 kips/in Measured Lap Strength=17 kips/in

Figure A27. Bent 68 Pile Confinement.



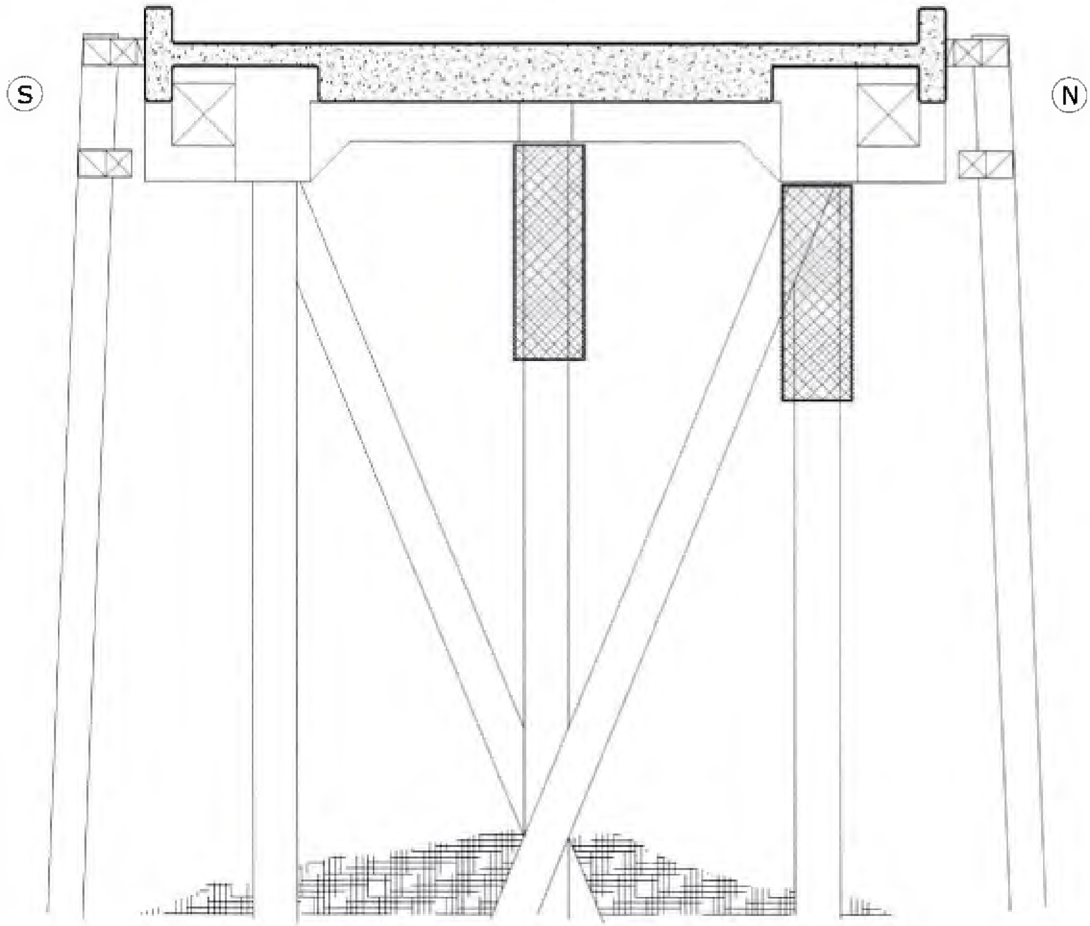
Pile Shell Hardcore Dupont Vinyl Ester - E-Glass Composite	Measured Circumferential Tensile Strength = 43 kips/in @ 2% Strain Recommended Design Limit @ .2 % Strain = 4.3 kips/in Measured Longitudinal Tensile Strength=10 kips/in Recommended Design Limit=1 kips/in Measured Lap Strength=17 kips/in

Figure A28. Bent 69 Pile Confinement.



Pile Shell Hardcore Dupont Vinyl Ester - E-Glass Composite	Measured Circumferential Tensile Strength = 43 kips/in @ 2% Strain Recommended Design Limit @ .2 % Strain = 4.3 kips/in Measured Longitudinal Tensile Strength=10 kips/in Recommended Design Limit=1 kips/in Measured Lap Strength=17 kips/in

Figure A29. Bent 70 Pile Confinement.



Pile Shell Hardcore Dupont Vinyl Ester - E-Glass Composite	Measured Circumferential Tensile Strength = 43 kips/in @ 2% Strain Recommended Design Limit @ .2 % Strain = 4.3 kips/in Measured Longitudinal Tensile Strength=10 kips/in Recommended Design Limit=1 kips/in Measured Lap Strength=17 kips/in

Figure A30. Bent 71 Pile Confinement.

APPENDIX B:

ANALYSES and DESIGN, ENVIRONMENTAL EFFECTS TESTS, CONCEPT TEST & EVALUATION, and QUALITY ASSURANCE TESTING:

Pier 12 NAVSTA San Diego

CONTENTS

INTRODUCTION

Carbon fiber reinforce plastic (CFRP) is corrosion resistant, lightweight, high strength and possesses outstanding fatigue properties. Due to its high strength and low weight, additional reinforcing can be added to existing steel reinforced concrete without adding excessive mass. It strengthens and stiffens deck slab cross section. It mitigates crack width and growth. CFRP restricts growth into the compression zone of the section, thereby increasing the shear strength of the concrete deck slab.

CFRP has no plastic deformation prior to rupture so plastic hinges do not form to redistribute loading. Epoxy polymers have different thermal coefficients than concrete. Epoxy thermal expansion coefficients are 7 times larger than concrete. Different thermal properties will increase stress between the concrete and adhesive interface. Cutting slots to embed CFRP rods will increase chances of initiating concrete cracks at the slot root parallel to the reinforcement.

Risks affecting safety. CFRP lacks a lengthy track record in civil structures that presents several risks. CFRP lacks reliable quality control for strength and geometric properties. It is subject to impact damage and precautions must be taken to protect it from exposure in industrial areas. The long-term effects of the waterfront environment such as salt water, high humidity, etc, on the concrete CFRP bond line are not well established. The fatigue strength and creep rates of the CFRP-concrete interface are not known. At high temperatures from solar radiation most cold setting epoxies experience a decline in shear modulus and a reduction in shear strength.

Risk mitigation program. To mitigate some of the risks associated with composites, NFESC has undertaken a comprehensive test and evaluation program. The testing served to characterize the materials for FEA modeling, to verify the materials for use on Pier 12, and to provide quality assurance of the materials and systems installed on Pier 12 as well as future pier upgrades. Testing included detailed laboratory testing of candidate materials as well as structural subsystems. The objective of these tests was to determine the bond limitations of the epoxy/rod embedded in the concrete, the adhesive quality of laminate bonding to concrete surfaces, and to quantify environmental effects including temperature extremes, thermal cycling, and salt-fog effects. Design development involved laboratory testing plus demonstrations at the Advanced Waterfront Technology Test Site (AWTTS) in Port Hueneme. Half-scale slabs were post strengthened and tested to failure at the AWTTS. These were to quantify the additional strength of the CFRP and test the integrity of the concrete/composite interface. The half scale slabs afforded us the opportunity to develop, evaluate, and modify the installation procedures and determine modes of failure. The AWTTS tests also provided the opportunity to verify derived equations for load response, resistance, and design.

Finite Element Analysis (FEA). The upgrade design program utilized detailed finite element modeling to characterize the structural behavior and verify proposed upgrade reinforcement designs. The models were modified to reflect the upgrade and were used to predict the load response of the as-built structure.

Calculation of Pier 12 Resistance

A conventionally steel reinforced concrete beam or slab is designed to be "under-reinforced" so the reinforcing steel yields followed by a concrete "failure" arbitrarily defined at strain level of 0.3 percent. This produces a "ductile" failure that is preceded by large cracks and deformation. The flexural resistance is calculated by simple equilibrium analysis while maintaining compatibility of strains. The resistance of a beam section post reinforced with a carbon rod or laminate (CFRP) is different because the CFRP is linear-elastic to failure and has no plastic reserve. Given an under-reinforced section, the design objective was to obtain the maximum deck slab bending resistance after the reinforcing steel yields and followed by laminate failure and concrete crushing. This assumes that there is sufficient concrete to offset all tensile forces (under reinforced), there is sufficient shear strength, and the laminate and reinforcing steel retains firm bonding with the concrete up to failure. Lower bending resistance results if the concrete fails first (over-reinforced), if shear failure occurs, or if anchorage (bond) is compromised.

The following assumptions apply to calculating the bending resistance of a cross section:

- Idealized stress-strain for concrete, steel and CFRP. Steel strain hardening is ignored.
- Concrete tensile force is ignored
- Strains are linearly distributed across the section in proportion to distance from the neutral axis (section planes remain plane)
- The position of the forces and neutral axis remain constant

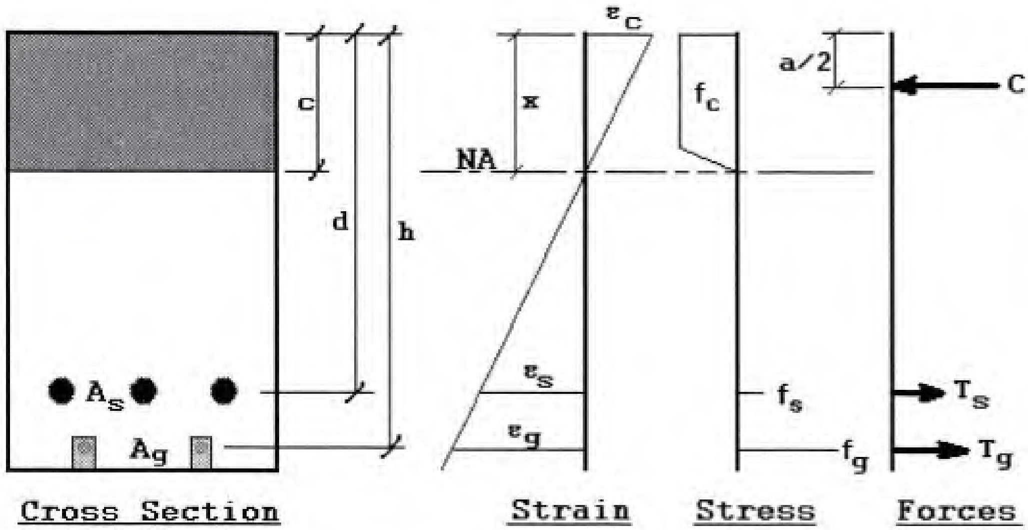


Figure B1. Post strengthened cross section using embedded composite bars. Assumed stress, strain, and internal forces for calculation of bending resistance.

Figure B1 shows the state of strain, stress, and force for calculating the bending resistance of a post strengthened reinforced-concrete section with embedded carbon bars. Similar methodology can be applied to bending resistance of external laminates and pultruded strips where the dimension, h, would be replaced by the full depth of the concrete section.

$$\Sigma F = 0$$
$$C = T_s + T_g$$

$$\Sigma M = 0$$
$$M_r = T_s(d-a/2) + T_g(h-a/2)$$

The strain relationships at maximum resistance are:

$$\varepsilon_c = -\varepsilon_g \,.\, x/(h-x)$$
$$\text{where } \varepsilon_c > 0.003$$

The value of a/2 is normally dependent on the concrete strength, f_c'. The average compressive stress in the concrete is **0.85 f_c'** (the standard ACI allowance). According to ACI the value of **a/2** is only slightly dependent of concrete strength if the steel has yielded at maximum resistance.

$$C = 0.85\, f_c'\, b\, a$$
$$T_s = A_s\, f_y$$
$$T_g = A_g f_g$$

So $\quad a/2 = 0.59(A_s f_y + A_g f_g) / f_c'b$

where b is the width of the section (or each foot width of a slab). So, changing the concrete strength of an under reinforced section has little effect on its flexural resistance.

$$M_r = A_s f_y (d - 0.59(A_s f_y + A_g f_g) / f_c'b) + A_g f_g (h - 0.59(A_s f_y + A_g f_g) / f_c'b)$$

The above relationship should be valid as long as the steel yields prior to a laminate failure and prior to the concrete strain reaching 0.003. This will be case when the following relationship holds:

$$A_s f_y + A_g f_g < 87{,}000 f_c'bd / (87{,}000 + f_y)$$

In designing an upgrade to post-strengthen a section, the steel stress should be limited to its yield value and the carbon laminate stress should be limited to less than half of its measured strength. This sets the value of the total tensile force of the internal couple which in turn sets the compression force. With the compression force the Whitney compression stress block can be determined and the resisting moment can be determined with the equation above. Setting the laminate stress also sets the laminate strain so a check of neutral axis location and concrete strain can be made by compatibility of strain requirements and since planes remain plane. The equations above have been organized in a EXCEL® spreadsheet program to design flexural members using CFRP (laminate, pultruded strips, and embedded rods). The spreadsheet was used to design the upgrade reinforcement for the Pier 12 deck. The original and upgrade flexural resistance for Pier 12 deck components are tabulated in **Table B1.**

In order to control crack width the strain in the laminate may be restricted more than the limits listed above. This is important when it is deemed necessary to protect existing steel reinforcing from corrosion. For example, given a carbon laminate with an ultimate strength of 300 ksi and a modulus of 20,000 ksi. The laminate strain for a stress limit of 150 ksi would be 0.0075 in/in. The average crack width will be almost 0.1 inches for average crack spacing of 12 inches (larger for greater spacing). The ACI code (Section 9.4) limits reinforcement design strength to 80,000 psi to control deformation. It would seem that similar restrictions may be necessary for CFRP reinforcement in future designs. Currently there are no specific design guidelines that provide for such large reductions in design stress of carbon laminates to reduce deformation and control cracks. NFESC recommends that the carbon fiber stress not be allowed to exceed one half ultimate.

Modes of Failure. The desired failure mode is rupture of the CFRP after the steel yields and before concrete strain reaches 0.003 in compression. If shear/flexure crack widths are excessive, differential displacement perpendicular to the CFRP at the crack will cause a shear failure of the laminate or the laminate will peel away from the concrete surface. This is another reason for restricting the stress/strain level of the carbon laminate design stress to less than 50 percent of its strength. Then flexural cracks can be spanned and do not influence flexural capacity. The most undesirable failure is delamination or failure of the adhesive between concrete and laminate. The latter can be avoided if the concrete surface is sound and precautions are taken to remove loose material and laitance. Concrete tensile strength is less than the adhesive. Thus, failure can occur in the concrete as a secondary fracture with a thin layer of concrete adhered to the laminate or adhesive.

Table B1. Flexural Capacities of Pier 12 Deck Sections

Deck Section	*Moment Location*	*Current Capacity in-kips/ft*	*Upgraded Capacity In-kips/ft*	*% Increase*
24" deck (top)	Pilecap	670	2905	333
24" deck (bottom)	Midspan	345	3005	770
8" deck (top)	Pilecap	[illegible]	[illegible]	740
8" deck (bottom)	Midspan	72	480	566

The punching shear strength of the 8-inch deck is more than 200,000 kips for a 24-inch outrigger pad.

Calculation of Pier 12 Load Response

Load response was determined using finite element analysis (FEA). Models of the original construction were validated using impact load response (NFESC SSR-2132-SHR). The basic FEA model is shown in **Figure B2**. The FEA program, STARDYNE, was used in the analysis. The model was discretized with orthogonal plate elements in the slab, beam elements for the piles, utility loops and pilecap beams, and 3-D elements for the pilecaps. Orthotropic plate elements modeled the two stiffness properties of the orthogonal reinforcing steel. Concrete strength and stiffness was varied to match the ILM response and was determined to be in excess of 3000 psi. Two models were validated. The first represented the pier in its original condition (pre upgrade) and the second represented the pier in the as-built upgrade configuration (**Figures B3 and B4**). The upgrade model included the stiffness effects (in the pier longitudinal direction) of the added carbon reinforcement and the fiberglass I-beams. **Table B2** contains the maximum flexural response to 100 kip outrigger loads in the upgraded slabs.

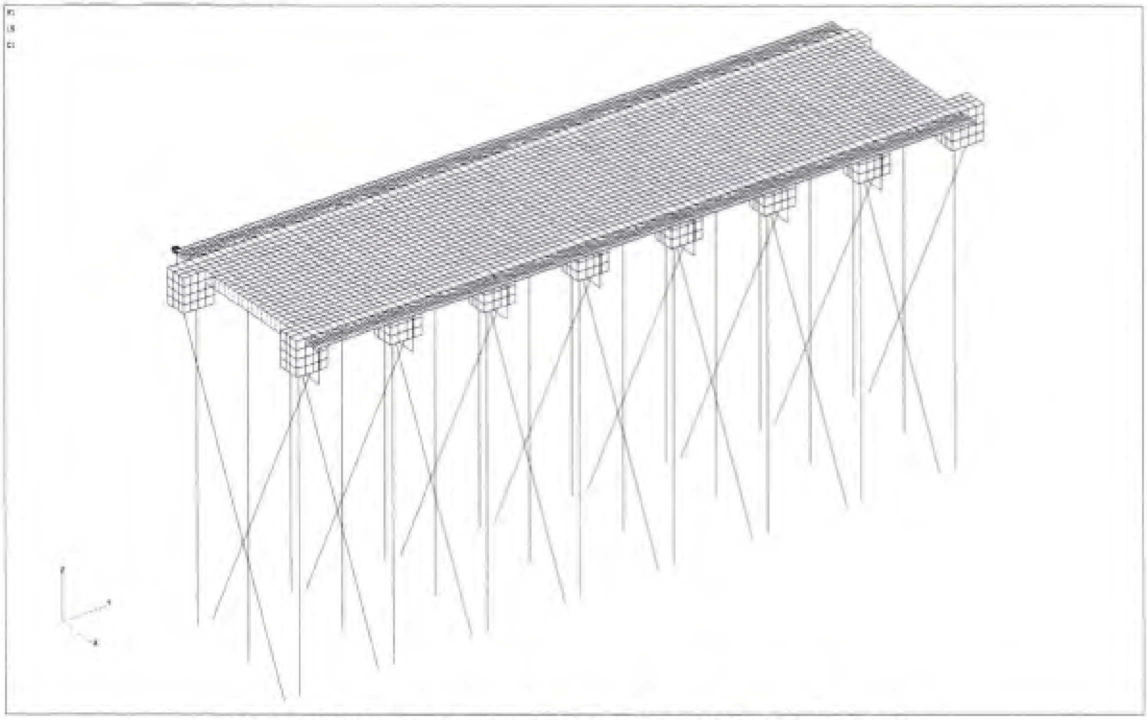


Figure B2. Finite element model of Pier 12 Section (7 spans).

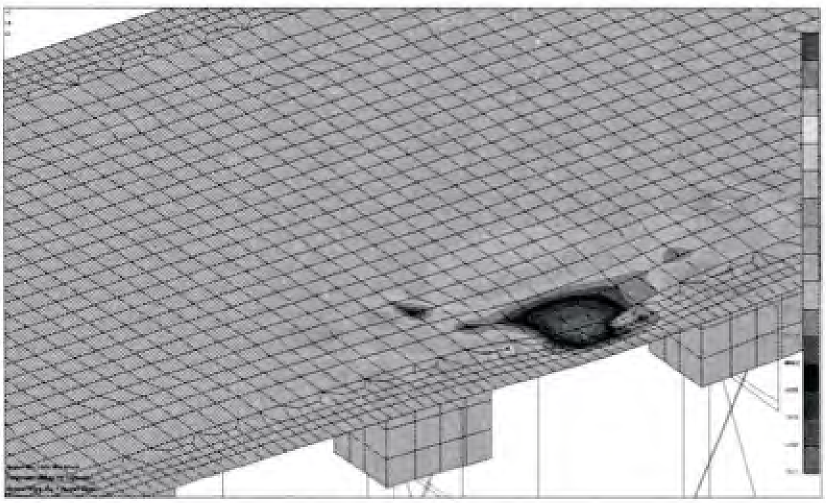


Figure B3. As-built deck (longitudinal) response to 100 kip outrigger at mid 8-inch section.

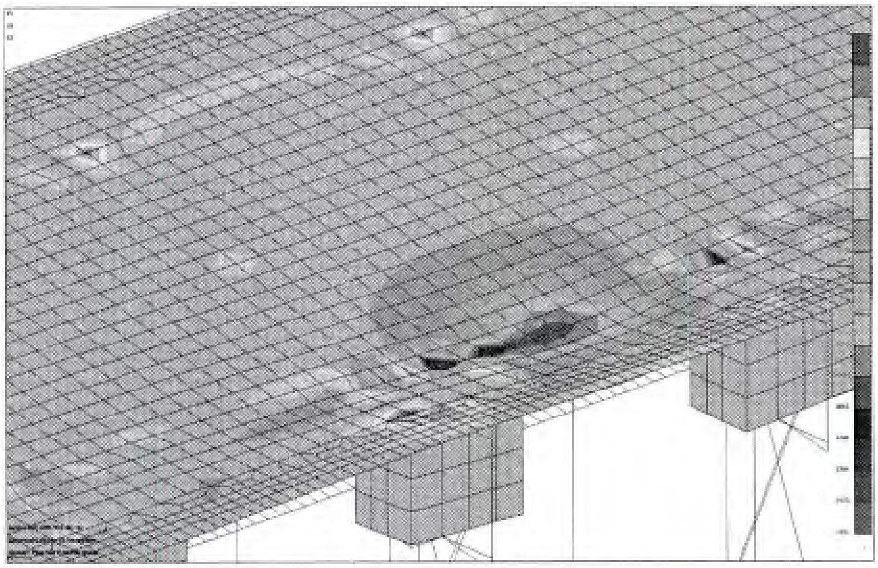


Figure B4. As-built deck response to 100-kip outrigger load at edge of 24-inch section.

Table B2. FEA Maximum Response of Pier 12 Deck to 100-kip Outrigger Load.

Deck Section	***Moment Location***	***Original Structure Moment In-kip/ft***	***Upgraded Structure Moment In-kip/ft***
24-inch Top	Pilecap	1465	1385
24-inch Bottom	Midspan	1500	1400
8-inch Top	Pilecap	122	120
8-inch Bottom	Midspan	265	240

ENVIRONMENTAL TEST PROGRAM

NFESC conducted numerous small beam tests to determine the bonding capacity of the carbon laminate and carbon rod to concrete. The beam tests were conducted in accordance with ASTM C-42 on beams that were 24 inches long (19 ½inches clear span) and 3-1/2 inches square cross section (**Figure B5**). The specimens were designed to fail by debonding the composite reinforcement from the concrete. The test program included tests at –50°F, tests on effects of freeze-thaw cycling, and tests on the effects of salt-fog environment at temperature of 95°F. The small beams were monitored during load to failure by deflection gages and strain gages placed on the reinforcement. Small beam test results are in **Table B3**. Typical load-defection responses to failure are shown in **Figures B5 through B7**. Typical strain responses are shown in **Figure B8**. Plots in **Figures B9 through B13** are graphic summaries of test results.

Long-term load tests (to one half the ultimate strength) continue without degradation on small beams that were placed in the tidal zone in Port Hueneme (more than two years exposure at this time). Other long-term tests include exposure of carbon/epoxy laminates to marine life in Pearl Harbor. There have been no ill effects to the laminate from marine life.

The salt-fog conditioning of the specimens were conducted in accordance ASTM B 117. The temperature was 95^0 F. The salt-fog solution was 95 parts water and 5 parts sodium chloride by weight. The pH of the salt water was between 6.5 - 7.2. The quantity of fog was measured as 1 - 2 ml of collected condensate per hour. Salt-fog exposure caused some damage to the concrete. The specimens accrued extensive concrete damage such as efflorescence and cracking after 1 year in the salt-fog chamber (**Figure B14**). The decline in load response shown in **Figures B9, B10, and B11** is due more to concrete degradation rather than loss in bond strength.

The specimens were subjected to 6 freeze/thaw cycles (between –30° and 130°) daily. Freeze-thaw was destructive to the concrete of the test specimens. The concrete was cast without air entrainment so there was concrete cracking, spalling, and general softening. At the end of the study (over 1000 cycles) a few load tests could not be conducted because the concrete was so badly deteriorated. At fewer cycles, the concrete cracked around the embedded bar (**Figure B15**). This was caused by the difference in thermal expansion of the epoxy and the concrete and was predicted by the FEA models (**Figure B16**). A slight decline in load-carrying capacity is shown in all results after 1000 freeze-thaw cycles (**Figures B12 and B13**). We do not recommend the application of external composite reinforcement to older structures in a freeze/thaw environment, because the laminates may trap moisture and hasten the concrete deterioration.

Table B3. Results of all small beam tests.

Beam #	Reinforcement Type	Reinforcement Dimensions	Slot Width	Test Type	environment period	environment cycles	Strain Gage	Thermo-Couple	Failure Load	Failure Mode
1	Embedded Rod	5mm - Deformed	3/8"	low temp		t	X	X	6839	anchorage/shear
2	Embedded Rod	5mm - Deformed	3/8"	Baseline			X		4738	anchorage/shear
3	Embedded Rod	5mm - Deformed	3/8"	Baseline			X		6222	anchorage/shear
4	Embedded Rod	5mm - Deformed	3/8"	low temp		t		X	8053	anchorage
5	Embedded Rod	5mm - Deformed	3/8"	low temp		t			7175	anchorage
6	Embedded Rod	5mm - Deformed	3/8"	Baseline					5569	anchorage
7	Embedded Rod	1/8 - Smooth	1/4"	low temp		t		X	2447	bond
8	Embedded Rod	1/8 - Smooth	1/4"	low temp		t		X	2029	
9	Embedded Rod	3mm - Deformed	1/4"	Baseline					4166	anchorage
10	Embedded Rod	3mm - Deformed	1/4"	low temp		t		X	4865	anchorage
11	Embedded Rod	1/8 - Smooth	1/4"	Baseline					2409	anchorage
12	Embedded Rod	1/8 - Smooth	1/4"	low temp		t			2304	anchorage
13	Embedded Rod	3mm - Smooth	1/4"	Baseline					2593	anchorage
14	Embedded Rod	3mm - Smooth	1/4"	low temp		t		X	1444	
15	Embedded Rod	1/8 - Smooth	1/4"	Baseline					2728	rod
16	Embedded Rod	1/8 - Smooth	1/4"	Baseline					2200	anchorage
17	Embedded Rod	1/8 - Smooth	1/4"	Baseline					2199	anchorage
18	Embedded Rod	1/8 - Smooth	1/4"	low temp		t		X	2069	anchorage
19	Pultruded Strip	5mm x 50mm		low temp		t			6568	shear/bond
20	Pultruded Strip	5mm x 50mm		low temp		t			6388	bond
21	Pultruded Strip	5mm x 50mm		low temp		t			5852	shear/bond
22	Pultruded Strip	5mm x 50mm		Baseline						
23	Pultruded Strip	5mm x 50mm		Salt/Fog	57 weeks	continuous			6017	bond
24	Pultruded Strip	5mm x 50mm		Salt/Fog	57 weeks	continuous			2785	bond
25	Pultruded Strip	5mm x 50mm		Salt/Fog	57 weeks	continuous			6364	bond
26	Pultruded Strip	5mm x 50mm		Baseline					6282	shear/bond
27	Pultruded Strip	5mm x 50mm		Baseline					6485	shear/bond
28	Pultruded Strip	5mm x 50mm		Baseline					6736	shear/bond
29	Embedded Rod	3mm - Deformed	3/8"	Salt/Fog	54 weeks	continuous			1654	shear/bond
30	Embedded Rod	3mm - Deformed	3/8"	Salt/Fog	54 weeks	continuous			5239	shear/bond
31	Embedded Rod	3mm - Deformed	3/8"	Salt/Fog	54 weeks	continuous			1300	shear/bond
32	Embedded Rod	3mm - Smooth	3/8"	Salt/Fog	54 weeks	continuous			642	bond
33	Embedded Rod	3mm - Smooth	3/8"	Salt/Fog	54 weeks	continuous			1023	shear/bond
34	Embedded Rod	3mm - Smooth	3/8"	Salt/Fog	54 weeks	continuous			1448	bond
35	Embedded Rod	3mm - Deformed	3/8"	Freeze/thaw	28 weeks	1180			3720	anchorage
36	Embedded Rod	3mm - Deformed	3/8"	Freeze/thaw	28 weeks	1180			4063	anchorage
37	Embedded Rod	3mm - Deformed	3/8"	Freeze/thaw	28 weeks	1180			4829	anchorage
38	Embedded Rod	3mm - Deformed	3/8"	Freeze/thaw	28 weeks	1180			4866	anchorage
39	Embedded Rod	3mm - Deformed	3/8"	Freeze/thaw	19 weeks	800			4493	
40	Embedded Rod	3mm - Deformed	3/8"	Baseline			X		4459	
41	Embedded Rod	3mm - Deformed	3/8"	Baseline			X		4718	
42	Embedded Rod	3mm - Deformed	3/8"	Baseline			X		5914	
43	Embedded Rod	3mm - Smooth	3/8"	Freeze/thaw	28 weeks	1180			2302	anchorage
44	Embedded Rod	3mm - Smooth	3/8"	Freeze/thaw	19 weeks	800			2673	
45	Embedded Rod	3mm - Smooth	3/8"	Freeze/thaw	19 weeks	800			2421	
46	Embedded Rod	3mm - Smooth	3/8"	Freeze/thaw	28 weeks	1180			2345	anchorage
47	Embedded Rod	3mm - Smooth	3/8"	Freeze/thaw	28 weeks	1180			1965	anchorage
48	Embedded Rod	3mm - Smooth	3/8"	Baseline			X		2041	
49	Embedded Rod	3mm - Smooth	3/8"	Baseline			X		2015	
50	Embedded Rod	3mm - Smooth	3/8"	Baseline			X		1908	
52	Embedded Rod	3mm - Deformed	3/8"	Baseline			X		4592	
1A	Tow Sheet	single ply x 2 in		Salt/Fog	54 weeks	continuous			3745	bond
2A	Tow Sheet	single ply x 2 in		Salt/Fog	54 weeks	continuous			2470	bond
3A	Tow Sheet	single ply x 2 in		Salt/Fog	35 weeks	continuous			3532	bond
4A	Tow Sheet	single ply x 2 in		Salt/Fog	35 weeks	continuous			2588	bond
5A	Tow Sheet	single ply x 2 in		Salt/Fog	35 weeks	continuous			3288	bond
6A	Tow Sheet	single ply x 2 in		Salt/Fog	54 weeks	continuous			2128	bond
7A	Tow Sheet	single ply x 2 in		Salt/Fog	54 weeks	continuous			1786	bond
8A	Tow Sheet	single ply x 2 in		Salt/Fog	18 weeks	continuous			2979	
9A	Tow Sheet	single ply x 2 in		Salt/Fog	18 weeks	continuous			2805	
10A	Tow Sheet	single ply x 2 in		Salt/Fog	18 weeks	continuous			2419	
1B	Tow Sheet	single ply x 2 in		Freeze/thaw	28 weeks	1180			800	concrete
2B	Tow Sheet	single ply x 2 in		Freeze/thaw	28 weeks	1180			2803	bond
3B	Tow Sheet	single ply x 2 in		Freeze/thaw	18 weeks	750			2404	
4B	Tow Sheet	single ply x 2 in		Freeze/thaw	18 weeks	750			2195	
5B	Tow Sheet	single ply x 2 in		Freeze/thaw	28 weeks	1180			1866	bond
6B	Tow Sheet	single ply x 2 in		Freeze/thaw	28 weeks	1180			280	concrete
7B	Tow Sheet	single ply x 2 in		Freeze/thaw	28 weeks	1180			957	concrete
8B	Tow Sheet	single ply x 2 in		Freeze/thaw	28 weeks	1180			1952	bond
9B	Tow Sheet	single ply x 2 in		Freeze/thaw	28 weeks	1180			2849	bond

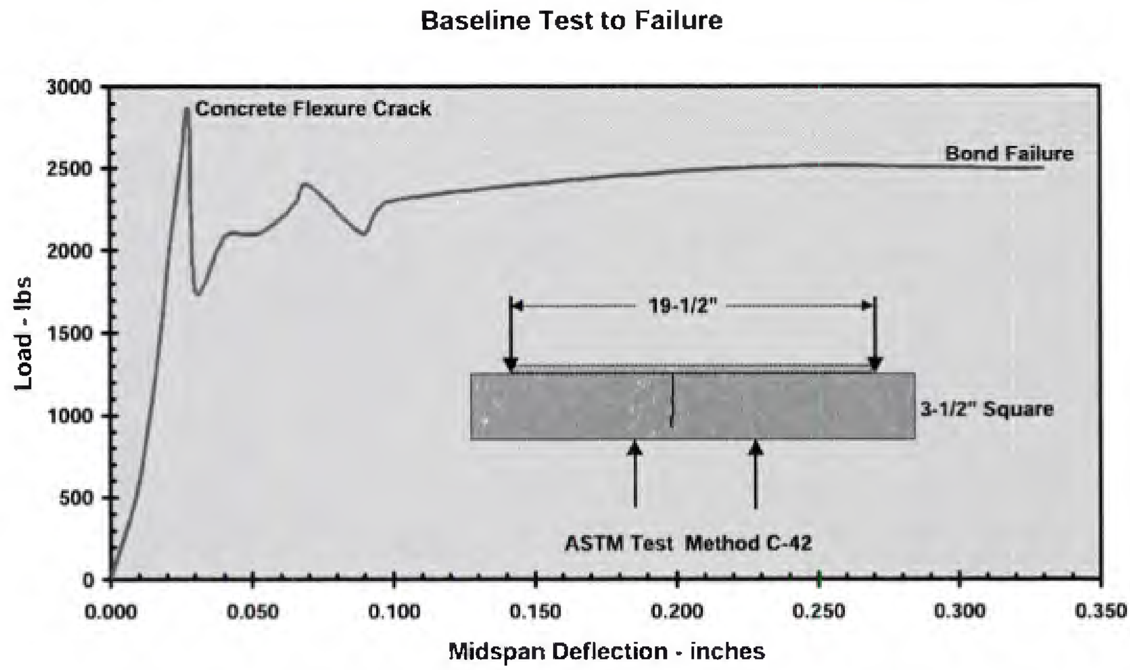


Figure B5. ASTM C-42 test method: Load to failure of small beam reinforced with tow sheet.

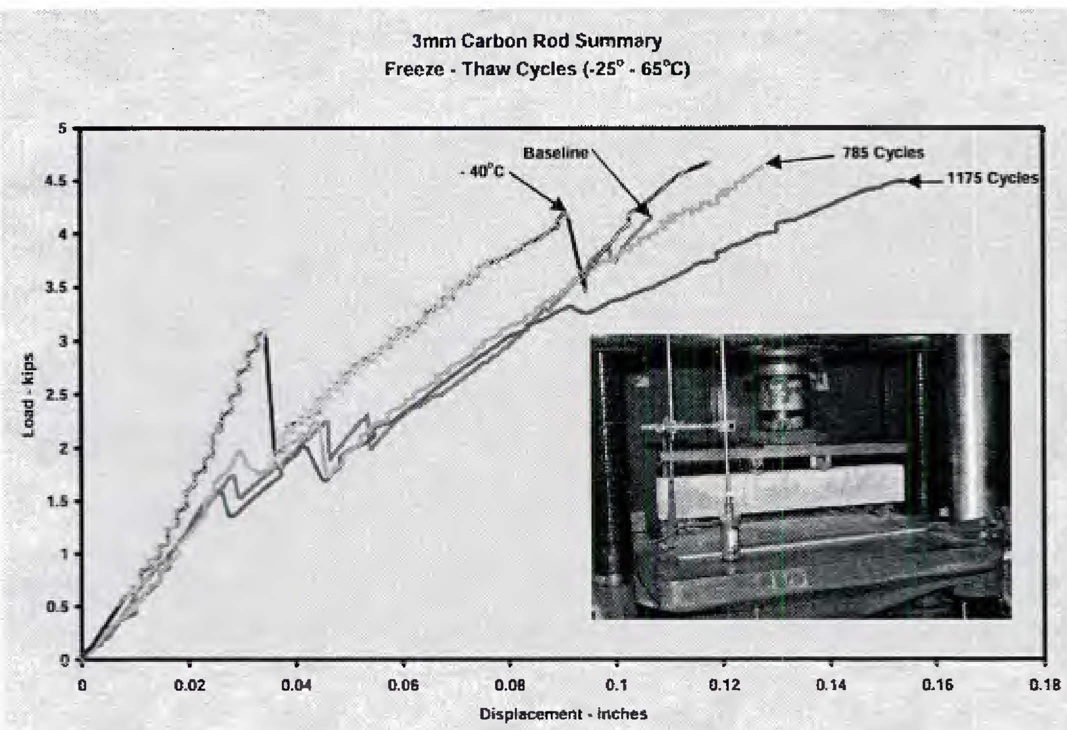


Figure B6. Example load-to-failure response of small test beams after freeze-thaw cycling.

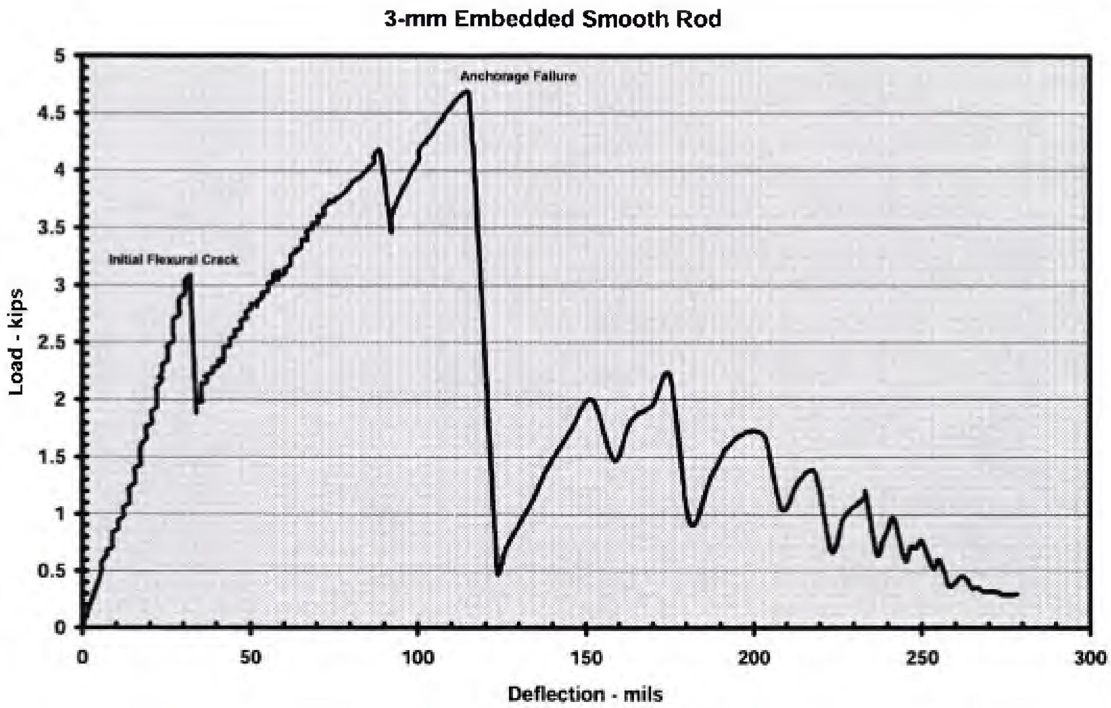


Figure B7. Load-deflection of beam up to bond failure.

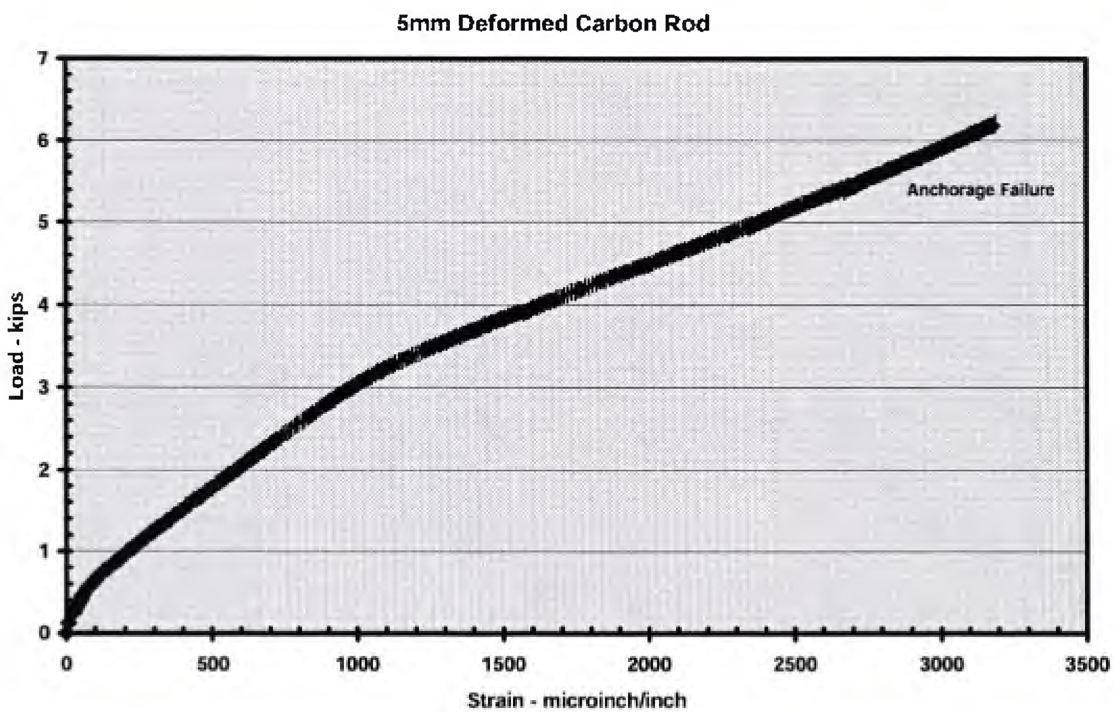


Figure B8. Load-strain response of small beam w/embedded rod.

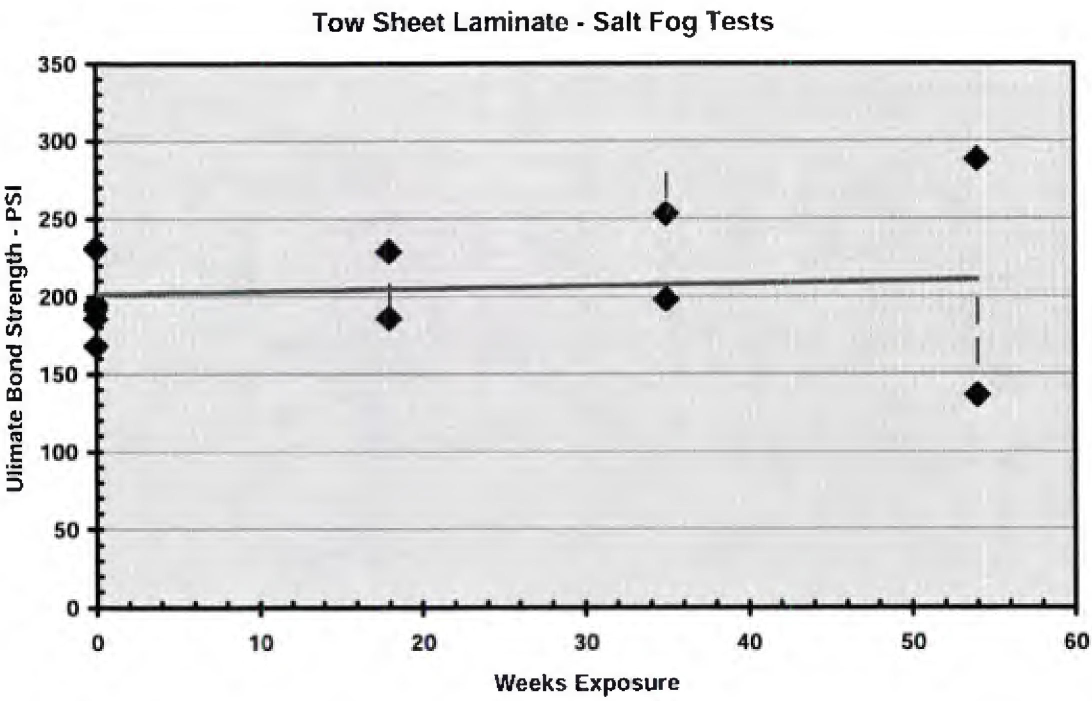


Figure B9. Tow sheet bond capacity as a function of exposure to salt-fog environment.

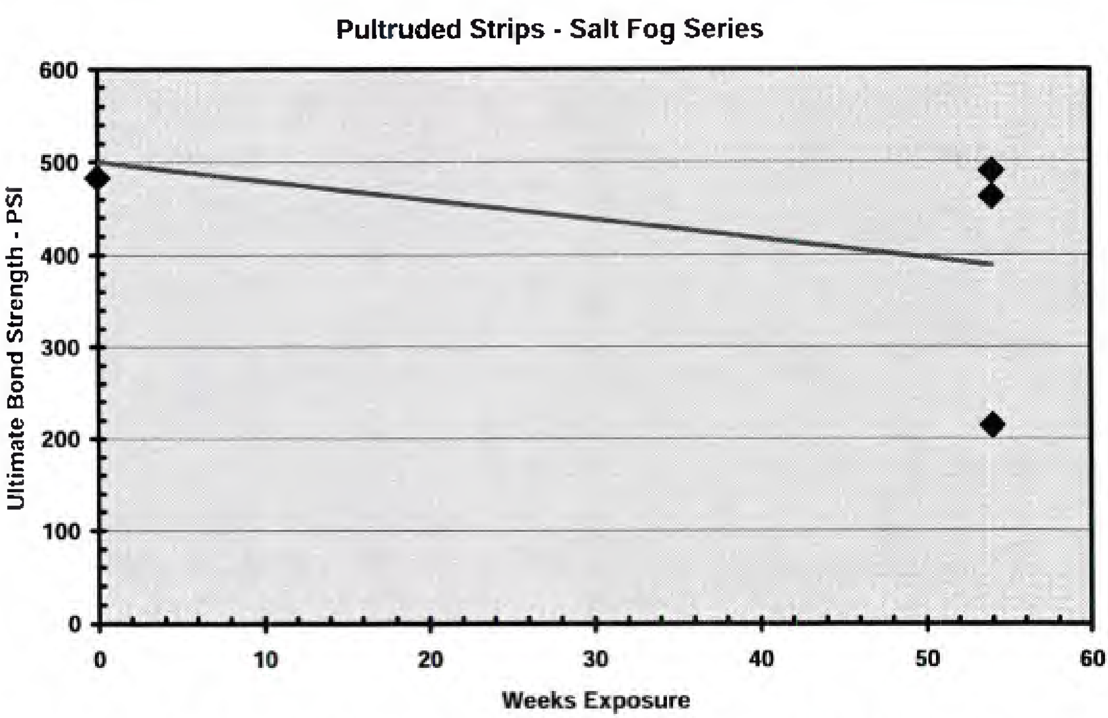


Figure B10. Pultruded strip bond capacity as a function of exposure to salt-fog environment.

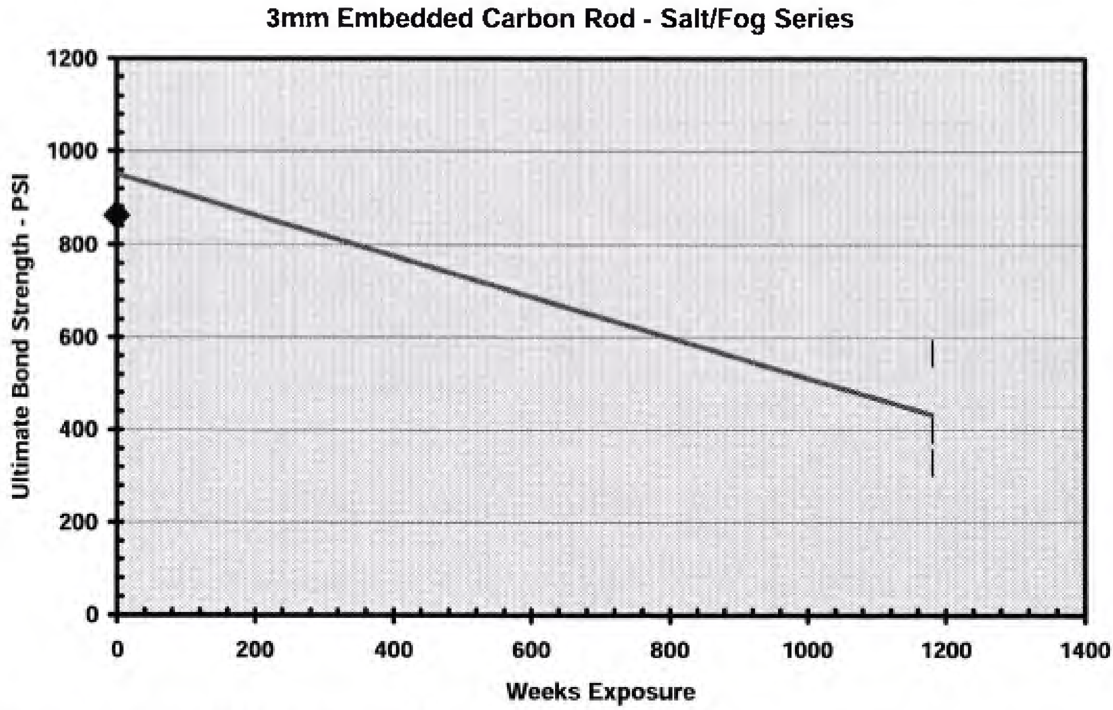


Figure B11. Embedded carbon rod bond capacity as a function of exposure in salt-fog chamber.

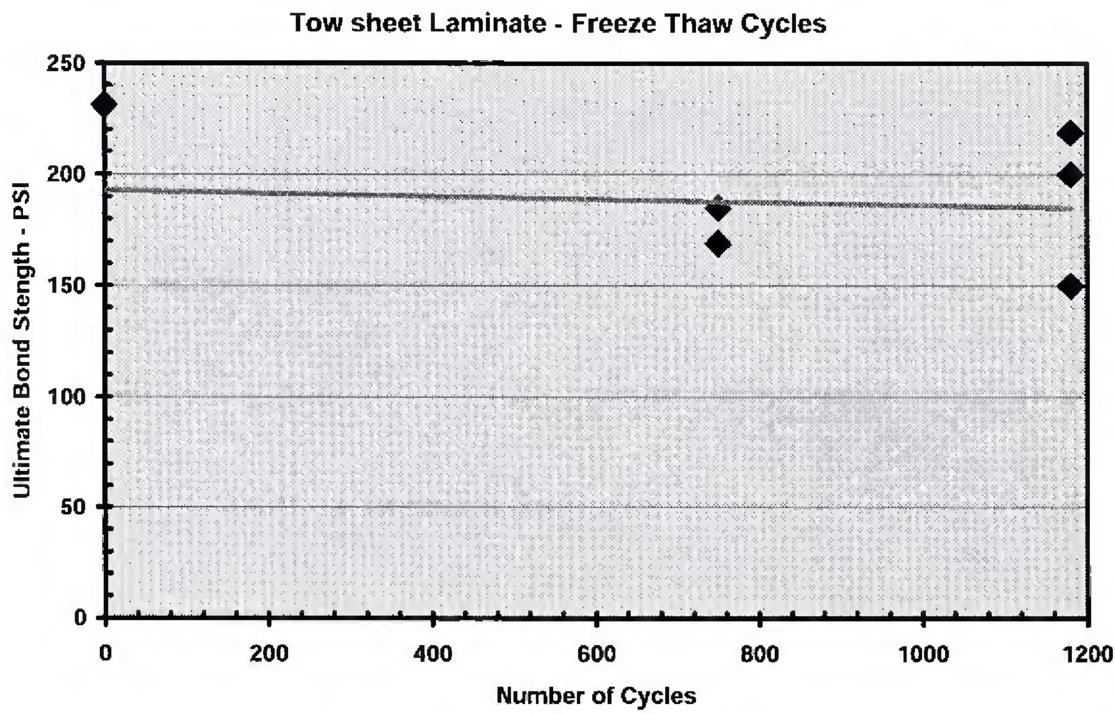


Figure B12. Tow sheet bond capacity as a function of freeze-thaw cycles.

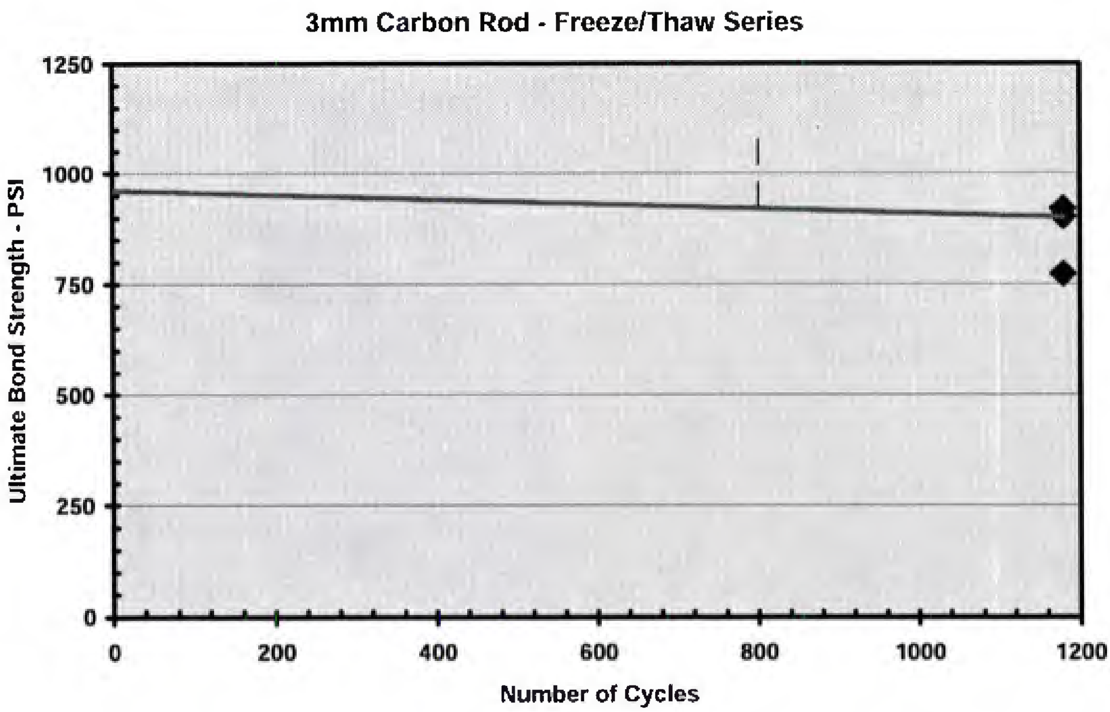


Figure B13. Embedded carbon rod bond capacity as a function of exposure to freeze-thaw cycles.

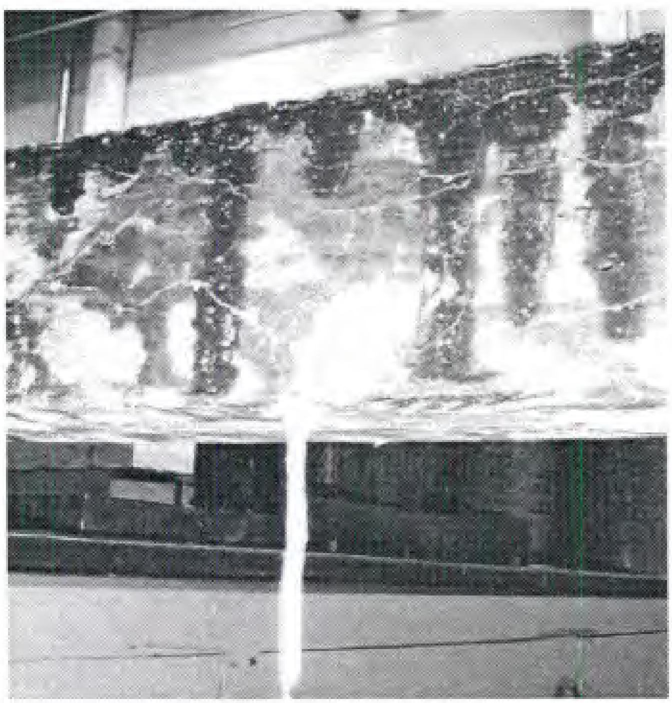


Figure B14. Small beam specimen after 1 year in salt-fog chamber.

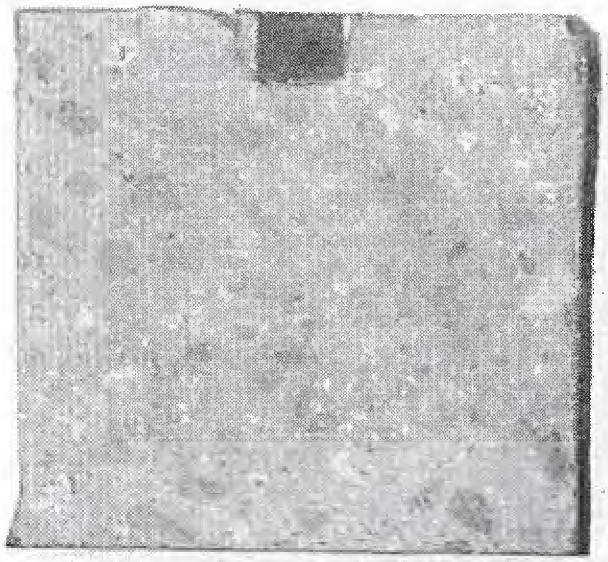


Figure B15. Cross section slice of beam specimen after 1000 thermal cycles.

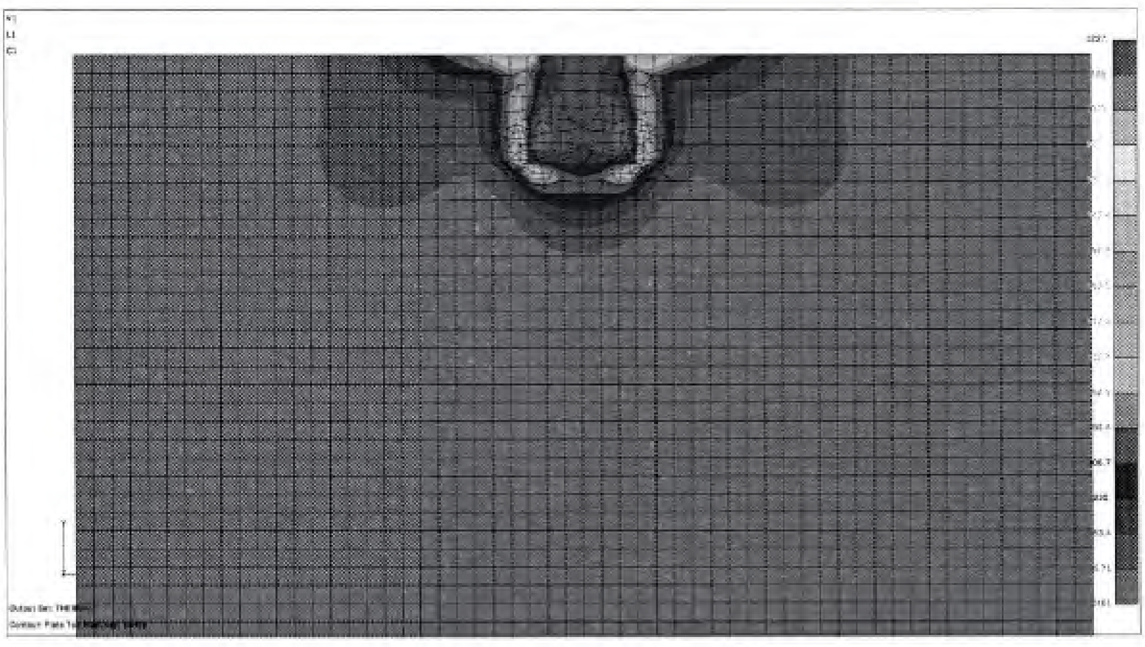


Figure B16. FEA predicted thermal-induced maximum shear in concrete adjacent to embedded rod.

Half-Scale STRUCTURAL CONCEPTS Program

NFESC conducted half scale tests of upgrade systems at the Advanced Waterfront technology Test Site (AWTTS) in Port Hueneme (**Figure B17**). The systems included hand lay-up laminated carbon uniaxial tow sheet, bonded pultruded carbon strips, and imbedded pultruded carbon rods (and strips). The purpose of these tests was to determine the suitability of applying the systems to waterfront upgrades, evaluate installation methodologies, quantify material performance, and to determine modes of failure. Two simple supported concrete slab types were tested: (1) 18 feet wide, 10 feet span, 9 inches thick, ordinary reinforced slab and (2) 6 feet wide, 10 feet span, 9 inches thick, prestressed, precast plank slab. The flexural resistance of these slabs are tabulated in **Table B4**. The slabs were patch-loaded at the center by a hydraulic ram onto a 12" x 12" steel plate working against a loading fame under the deck spanning between pilecaps (**Figure B18**). Steel reinforcement, concrete and composite reinforcement was monitored by strain gages. Deflection gages were also monitored at midspan and at the supports.

Figure B17. Slab tests being prepared on the AWTTS.

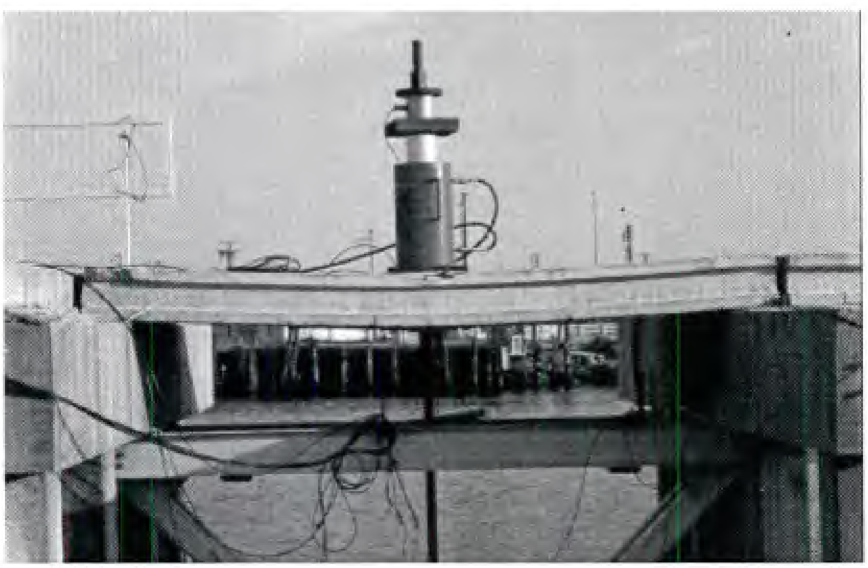


Figure B18. AWTTS load setup during test on precast plank slab.

Table B4. Flexural Capacities of Half Scale Slabs

Slab System	Slab Size feet	Composite Upgrade System	Strong Yield Moment in-kip/ft	Weak Yield Moment in-kip/ft	Strong Ultimate Moment in-kip/ft	Weak Ultimate Moment in-kip/ft	Percent Strength Increase
Ordinary Reinforced	10 x 18	Baseline	380	195	568	300	-
Ordinary Reinforced	10 x 18	Hand layup	449	230	770	489	35
Ordinary Reinforced	10 x 18	Embedded Rods	423	217	707	430	24
Precast Prestressed	10 x 6	Baseline			94		-
Precast Prestressed	10 x 6	Bonded Pultruded Strip			417		344
Precast Prestressed	10 x 6	Embedded Rods			343		265
Precast Prestressed	10 x 6	Embedded Pultruded Strips			379		303

Ordinary Reinforced Concrete. The baseline ordinary reinforced slab used No. 5 bars, ASTM A615 Grade 60. The concrete strength was 4000 psi. Tensile reinforcing was spaced 4 inches on center in the 10-ft (strong or longitudinal) direction and 8 inches on center in the 18-ft (weak or transverse) direction. Compression reinforcement was 6 inches on center in both directions. The calculated bending resistance in the strong direction of the baseline slab was:

The slabs were subjected to a 12" x 12" patch load in the center to failure. The Load-deflection response for the baseline ordinary reinforced slab is shown in **Figure B19**. The failure mode was punching shear; however, the tension reinforcement had yielded prior to ultimate load and exhibited strains as high 1.8 percent (**Figure B20**).

Prestressed Concrete Plank Slab. The precast plank slab was prestressed to 1000 psi. The concrete strength was 6000 psi. The plank was tested as non-reinforced section with the prestresssing steel located in the compression zone and the tension zone being unreinforced (upside down). The patch load was applied at the center of the slab. The failure mode of the prestressed plank slab was flexural rupture of the concrete on the tension face. The load-deflection response to failure is shown in **Figure B21** and the load strain response is shown in **Figures B22 and B23**.

Hand-Lay-up Laminated Uniaxial Carbon Fiber Sheets Added to Ordinary Reinforced Slab. The ordinary reinforced concrete slab was post strengthened by adding uniaxial carbon fiber tow sheets laminated with epoxy to the tension face. The laminate was applied overhead to simulate field application to strengthen the positive moment region of a Navy pier span. A single ply of carbon laminate was added to the weak direction (0.078 in^2/ft width) with a theoretical equivalent of 2.2 kips/inch width carbon strength and three plies (0.234 in^2/ft width) equivalent to 6.6 kips/inch was added to the strong direction. This increased the bending resistance by approximately 35 percent over the baseline slab.

Load responses of the laminate-upgraded slab are given in **Figures B24 and B25**. The slabs showed significant gains in strength and ductility over the baseline slabs. The failure mode was punching shear. The laminate remained intact up to the point of failure. The punching failure caused the laminate to peal away from the surface. The slab was not reusable after ultimate load.

Pultruded Carbon Rods Added to Ordinary Reinforced Slab. 1/8-inch diameter rods provided by Neptco were set in epoxy in 3/8-inch slots that were 1/2-inch deep and spaced at 4 inches on center. The rods were smooth with 67 percent carbon fiber volume and the remainder epoxy (bisphenol-A) resin. The slots were saw-cut in the slab. Rods were terminated 6 inches from the support and 30 inches from the free edges. Four rods were placed in each groove in the strong direction. Each groove in the weak direction contained one rod except for every fourth groove where two rods were placed. The rods were embedded in 1312F epoxy supplied by Madewell. The addition of carbon/carbon rods was equivalent to 4.3 kips/inch in the strong direction and 1.4 kips per inch in the weak direction (tensile strength of rods). The rods were 60 percent carbon fiber, so 0.086 in^2/ft width of carbon fiber was added in the strong direction and 0.028 in^2/ft was added in the weak direction. This increased the bending resistance by approximately 24 percent over the baseline slab.

Load response of the upgraded slab is given in **Figures B26 and B27**. The slabs showed significant gains in strength and ductility over the baseline slabs. The failure mode was punching shear. Prior to ultimate load some rods had begun to separate from the slab due to insufficient cover. There were no rod failures up to the prior to ultimate load. The punching failure caused the composite rods to separate from the concrete and to fail. The slab was not reusable after ultimate load.

Precast Plank Slab with Embedded Pultruded Strips. 1-inch wide, 1.2 mm thick pultruded Sika carbon laminate stripes were 2 inches on center in the long direction of the precast slab (0.28 in^2/ft width)(carbon area equal to 0.18 in^2/ft width). Sikadur 32 was used to encapsulate the composite rods. The addition of pultruded carbon composite rods was equivalent to 5.1 kips/inch in the strong direction. This increased the bending resistance by approximately 303 percent. Although smaller in area, the pultruded strips were stiffer than the pultruded rods, which resulted in a higher load resistance of the slab. The pultruded strips also presented a larger internal moment arm that also raised the load resistance slightly. The load response of the embedded strip upgraded plank slab is given in **Figures B28, B29 and B30**.

Of the three upgraded precast planks, the embedded pultruded strips upgrade offered the best flexural resistance. It failed by crushing the concrete in the compression zone adjacent to the patch load. Wide flexural cracks were formed prior to failure. However, there was no separation of the encapsulating epoxy from the concrete and there was no evidence of composite slippage in the epoxy. No strips failed during the load tests.

Precast Plank Slab with Embedded Rods. Reinforcing was embedded in the unreinforced face of the slab. 3/8-inch diameter, DFI, smooth carbon rods were embedded at 4 inches on center in the long direction (0.33 in^2/ft width)(carbon area equal to 0.20 in^2/ft width). Sikadur 32 was used to encapsulate the composite rods. The addition of carbon/carbon rods was equivalent to 5.5 kips/inch in the strong direction. This increased the bending resistance by approximately 265 percent over the baseline plank slab. Load response of the embedded rod upgraded plank slab is given in **Figures B31, B32, and B33.**

The test slab failed by crushing the concrete in the compression zone adjacent to the patch load. Wide flexural cracks were formed prior to failure and some separation of the encapsulating epoxy from the concrete occurred in the slots adjacent to a large flexural crack at midspan. There was also some evidence of rod slippage in the epoxy. No bars failed prior to ultimate load.

Precast Plank Slab with Bonded Pultruded Strips. 2-inch wide, 1.2 mm thick pultruded Sika carbon laminate stripes were bonded 4 inches on center in the long direction of the precast slab (0.28 in^2/ft width)(carbon area equal to 0.18 in^2/ft width). Sikadur 30 was the bonding epoxy adhesive. The addition of carbon/carbon rods was equivalent to 5.1 kips/inch in the strong direction. This increased the bending resistance by approximately 344 percent. The increase in bending resistance over the other precast plank upgrades is primarily due to the increase of the internal moment arm. The Load response of the prestressed plank with an embedded bar upgrade is shown in **Figures B34, B35, and B36.**

This configuration did not provide the largest flexural resistance as predicted. Some of the strips were not fully bonded to the concrete and were starting to separate prior to ultimate load. The slab failed by crushing the concrete in the compression zone adjacent to the patch load. Wide flexural cracks were formed prior to failure. No strips failed during the load tests.

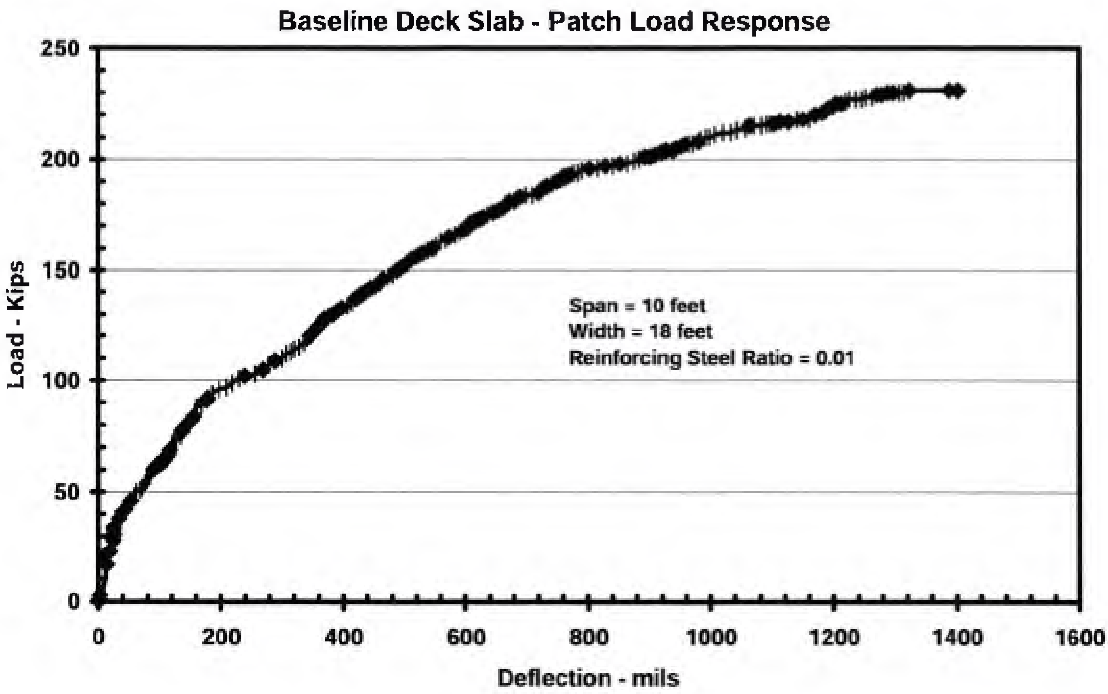


Figure B19. Load-deflection response to midspan patch load on half scale, ordinary reinforced slab.

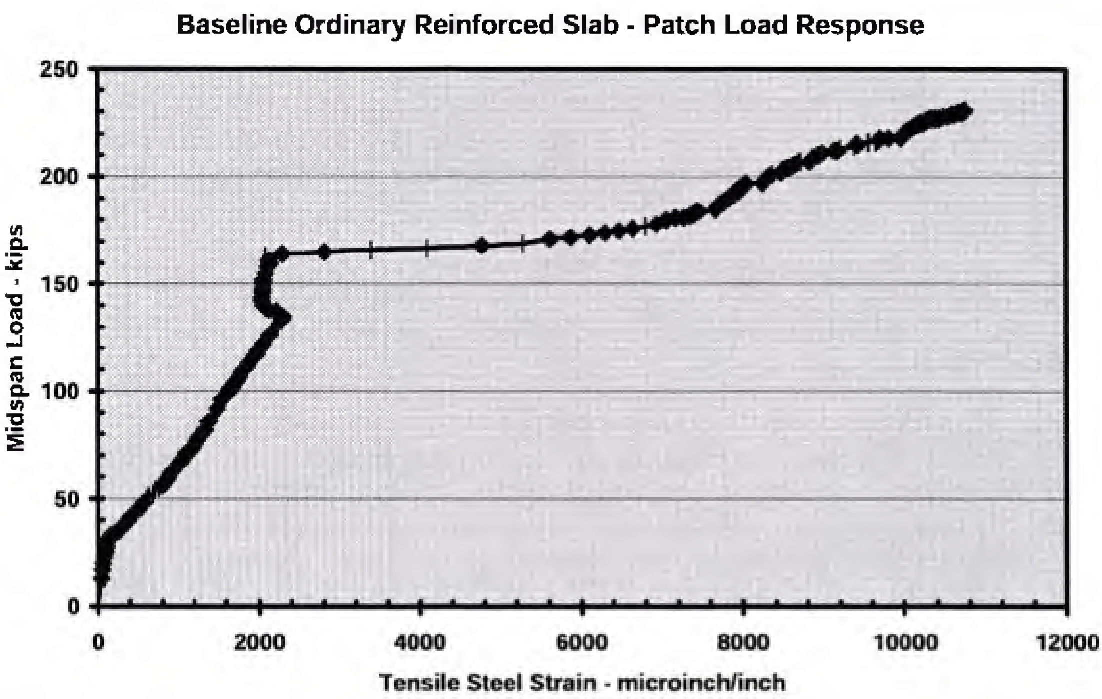


Figure B20. Load-Steel strain response to midspan patch load – baseline ordinary reinforced slab.

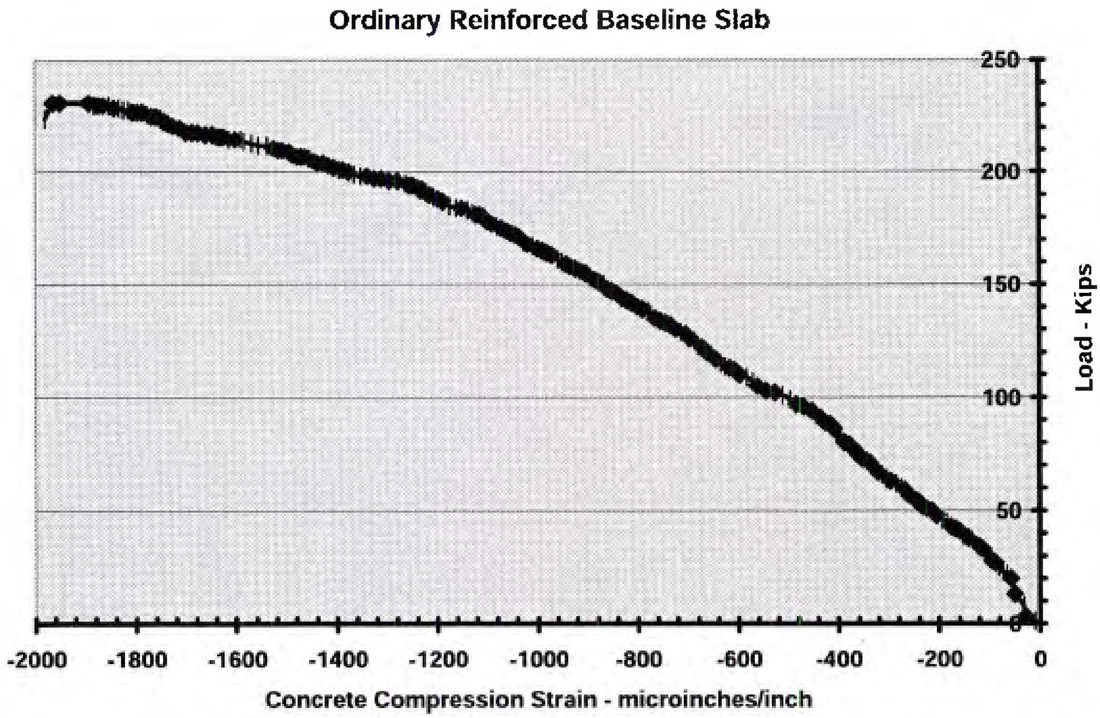


Figure B21. Midspan Load-Concrete Strain response of ordinary reinforced baseline slab.

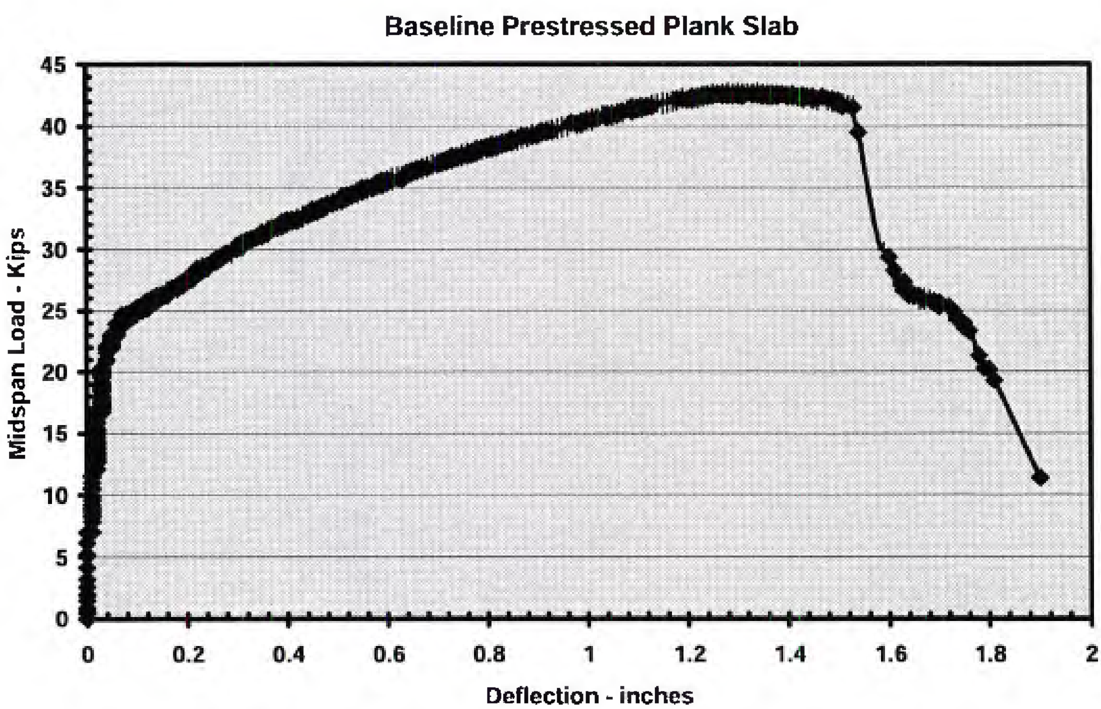


Figure B22. Midspan Load-Deflection response of baseline prestressed plank slab.

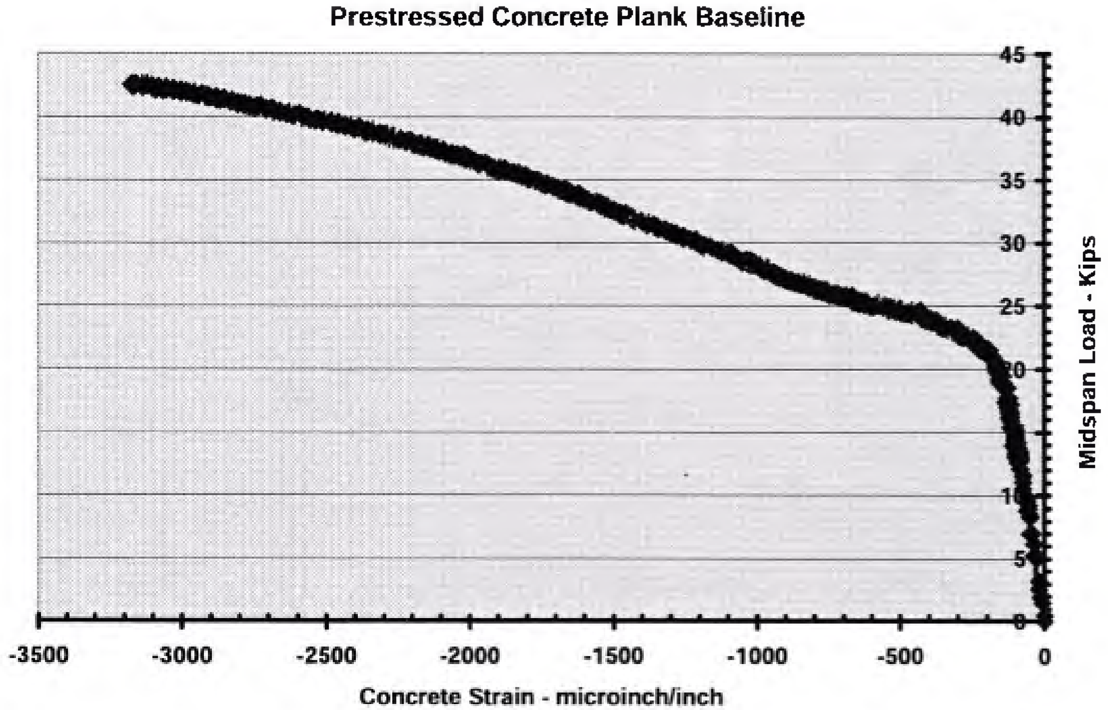


Figure B23. Midspan Load-Concrete Strain response of prestressed plank baseline slab

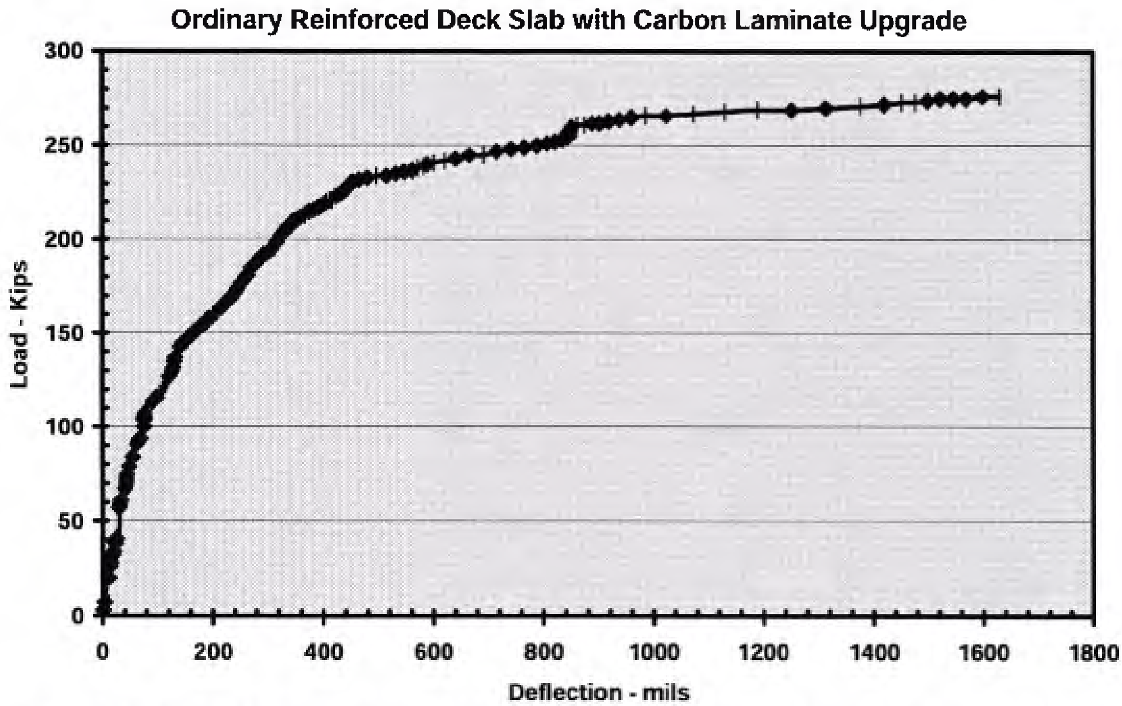


Figure B24. Load-Deflection response of carbon laminate upgraded ordinary reinforced slab.

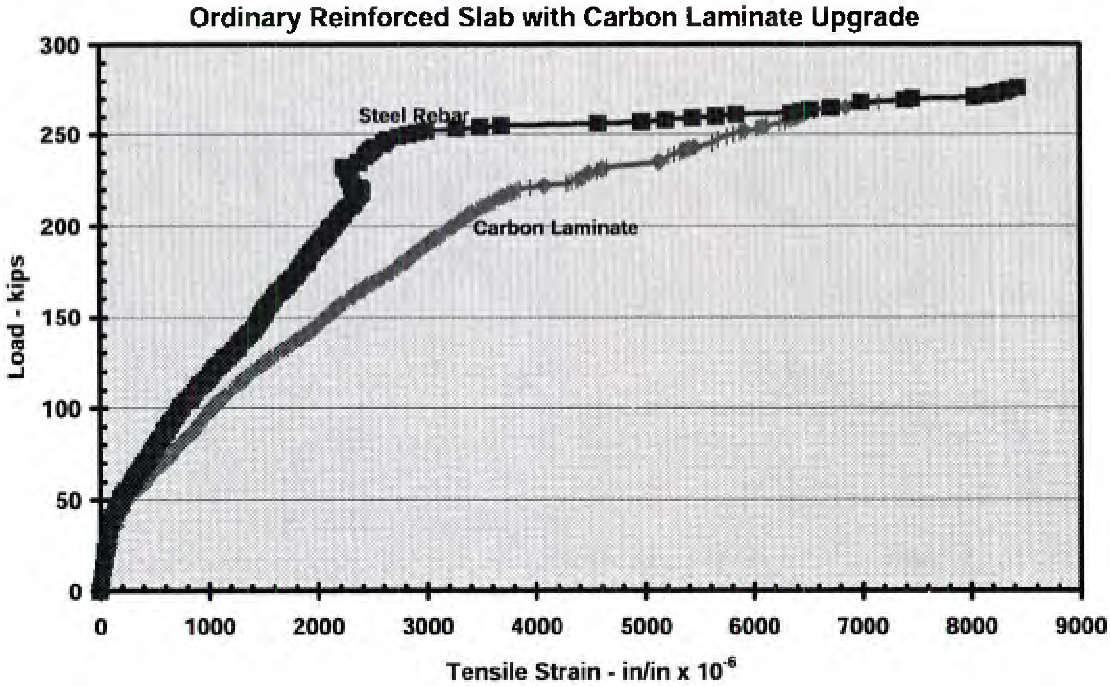


Figure B25. Load-Tensile Strain response to patch load on laminate upgraded ordinary reinforced slab.

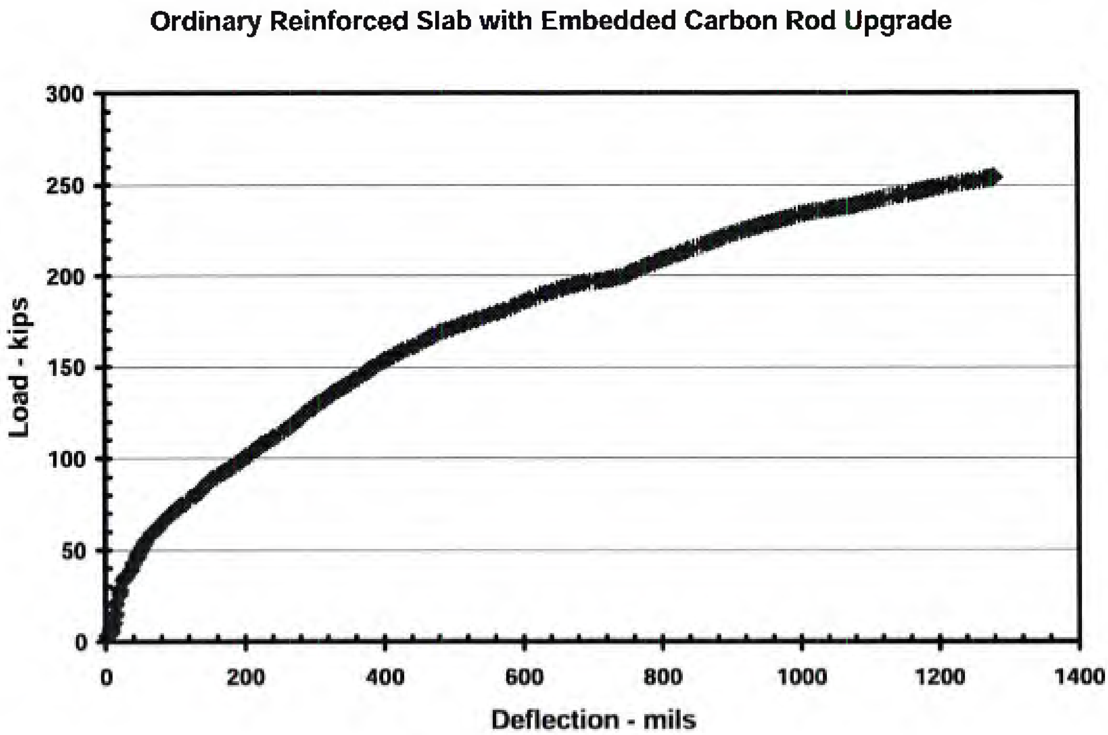


Figure B26. Load-Deflection response of ordinary reinforced slab with embedded carbon rod upgrade.

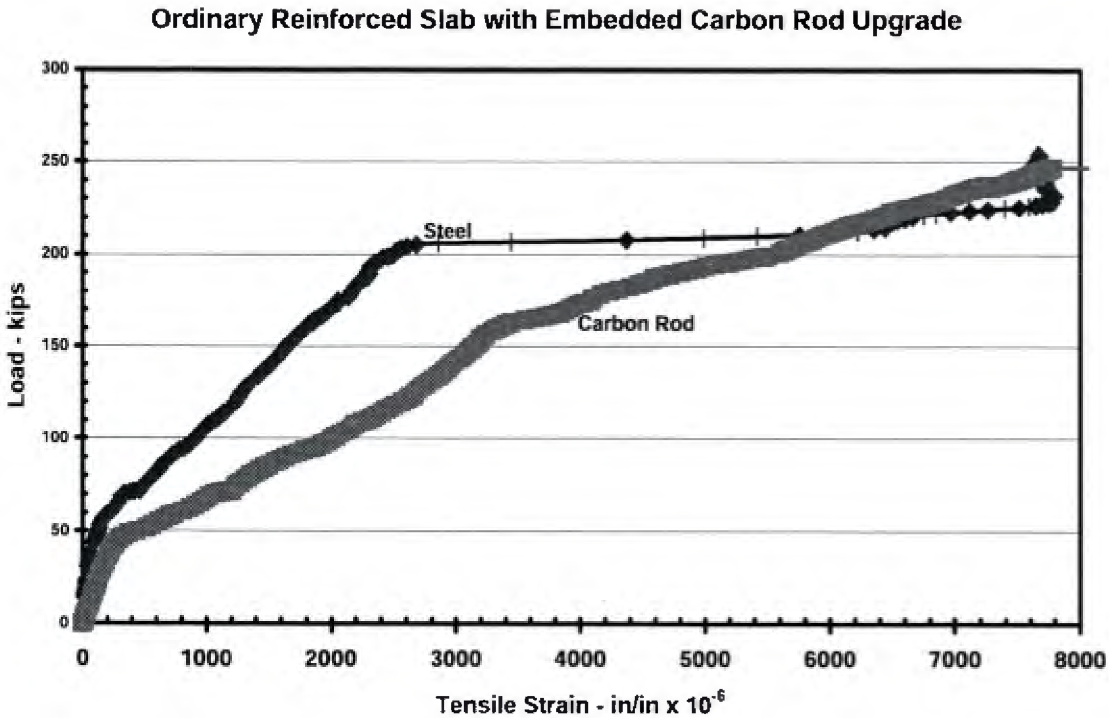


Figure B27. Load-Tensile Strain response of ordinary reinforced slab with carbon rod upgrade.

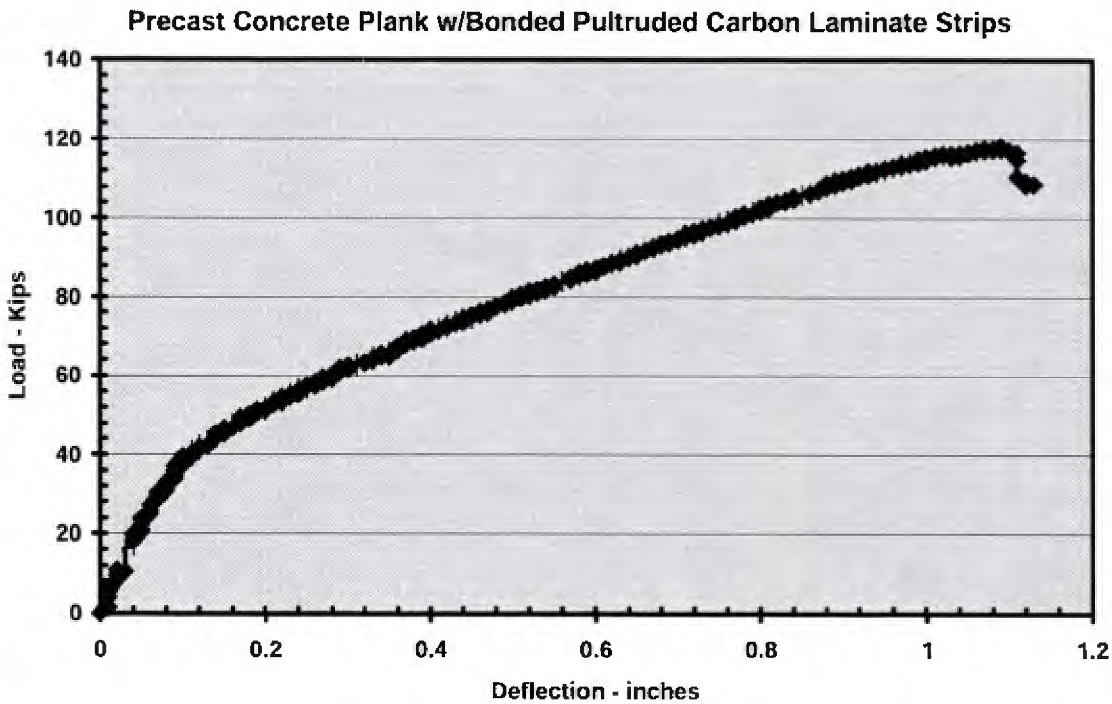


Figure B28. Load-Deflection response of prestressed plank upgraded with bonded pultruded carbon laminate.

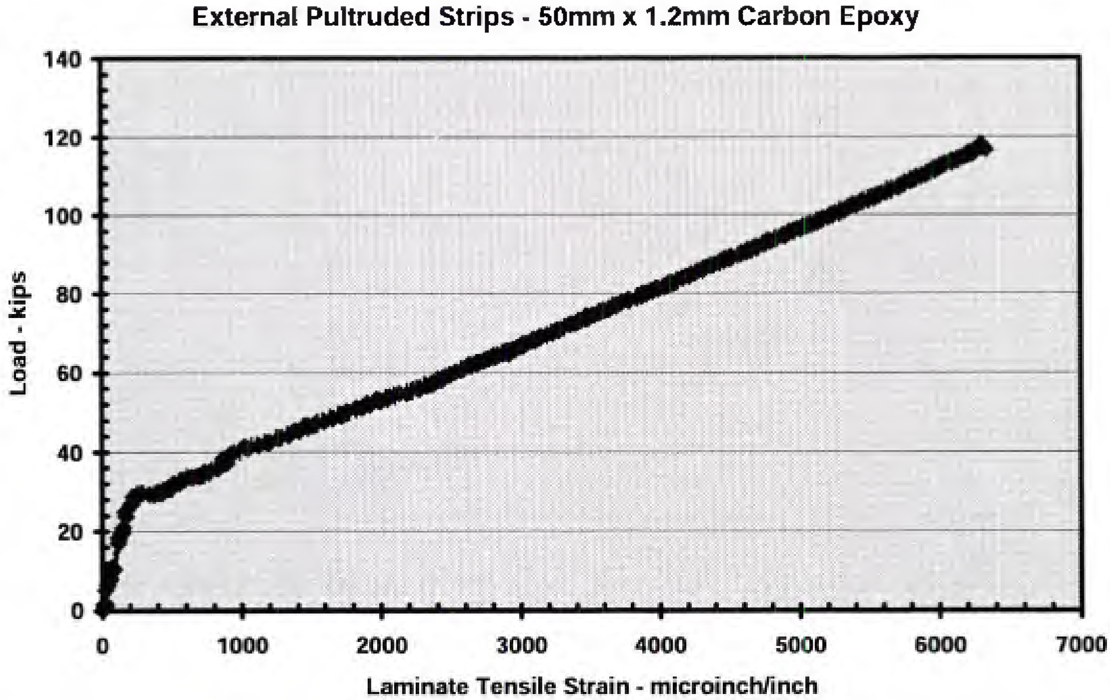


Figure B29. Load-Tensile Strain response of prestressed plank upgraded with bonded pultruded carbon laminate.

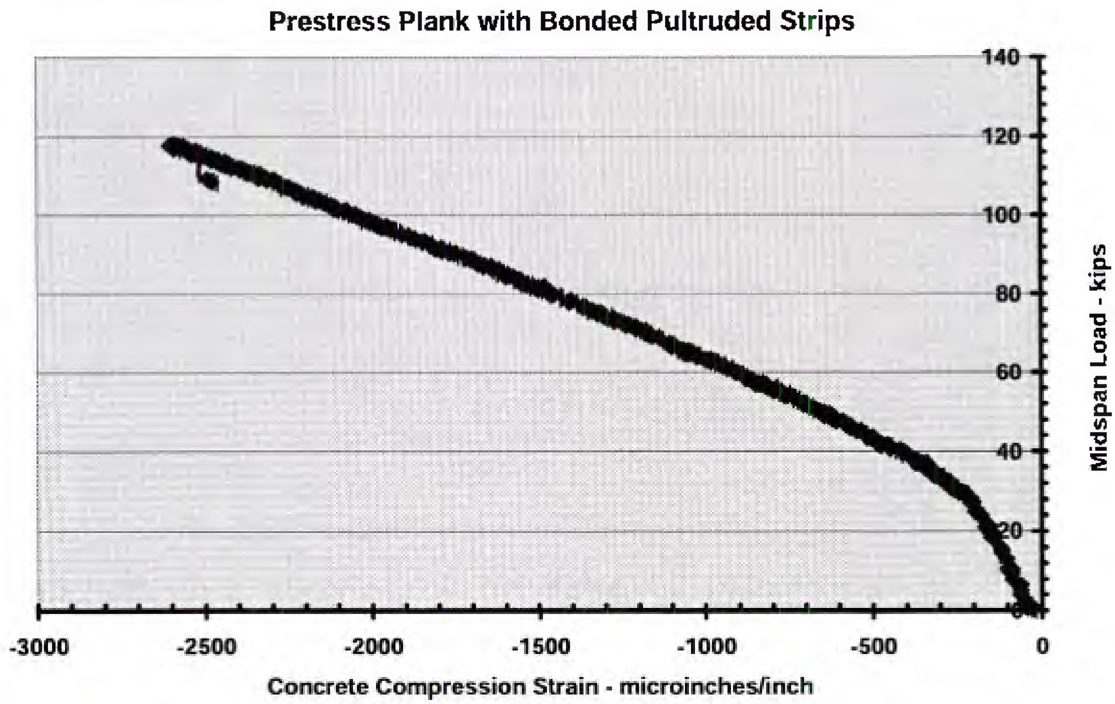


Figure B30. Load-Concrete Compression Strain response of prestressed plank upgraded with bonded pultruded carbon laminate.

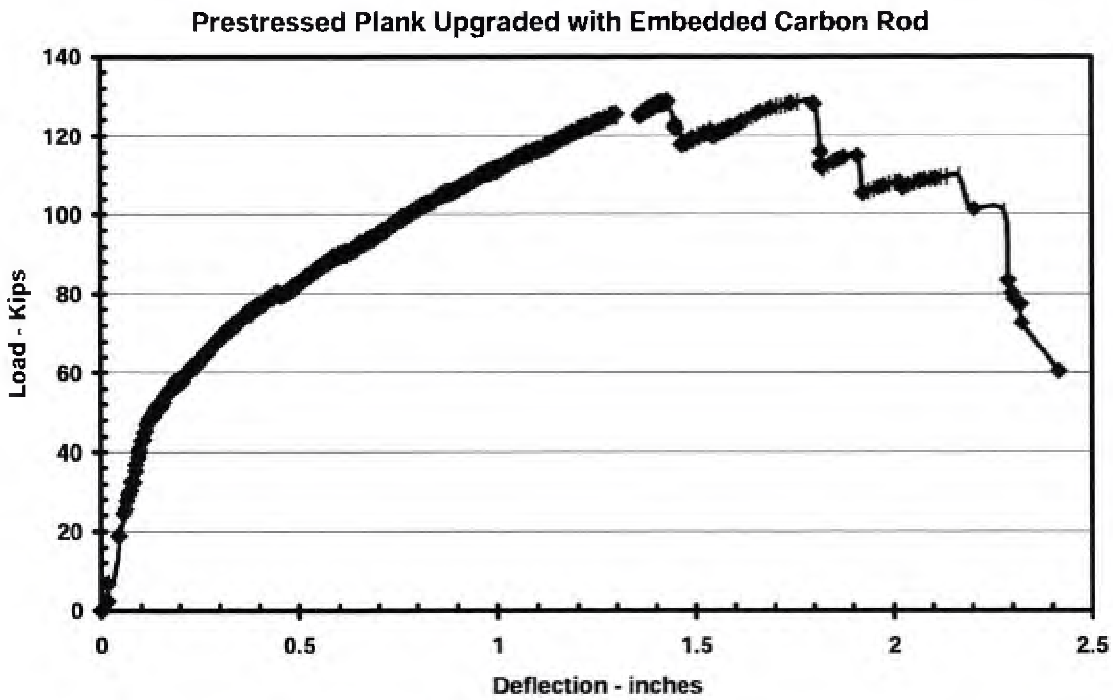


Figure B31. Load-Deflection response of prestressed slab with embedded carbon rod upgrade.

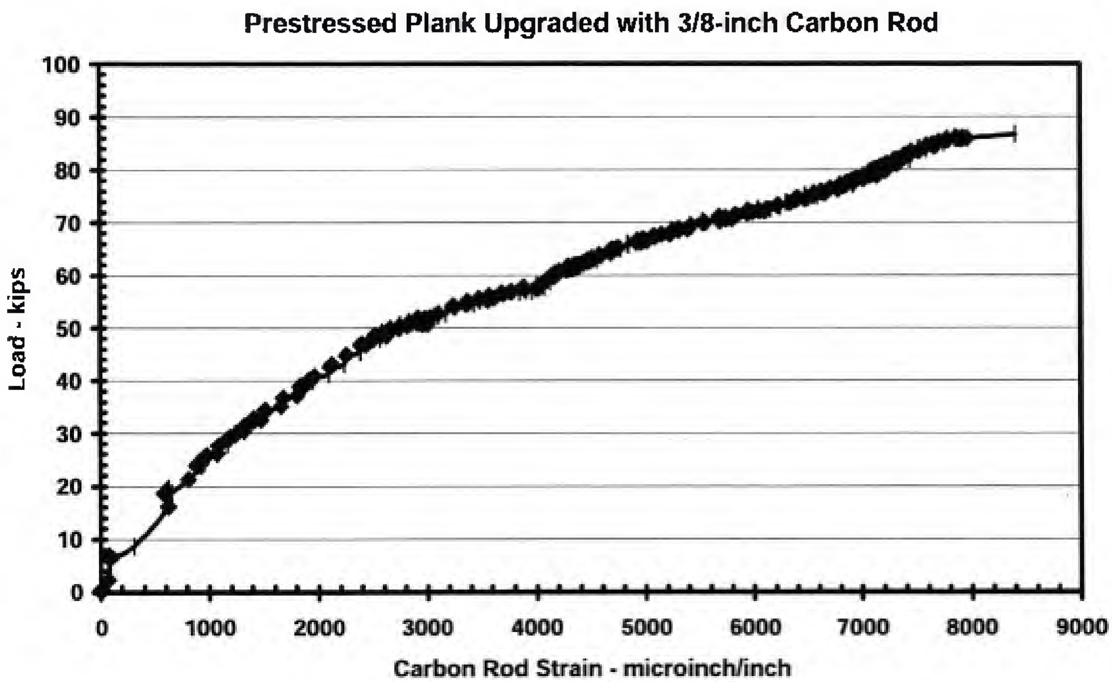


Figure B32. Tensile strain response of carbon rod embedded in prestressed plank.

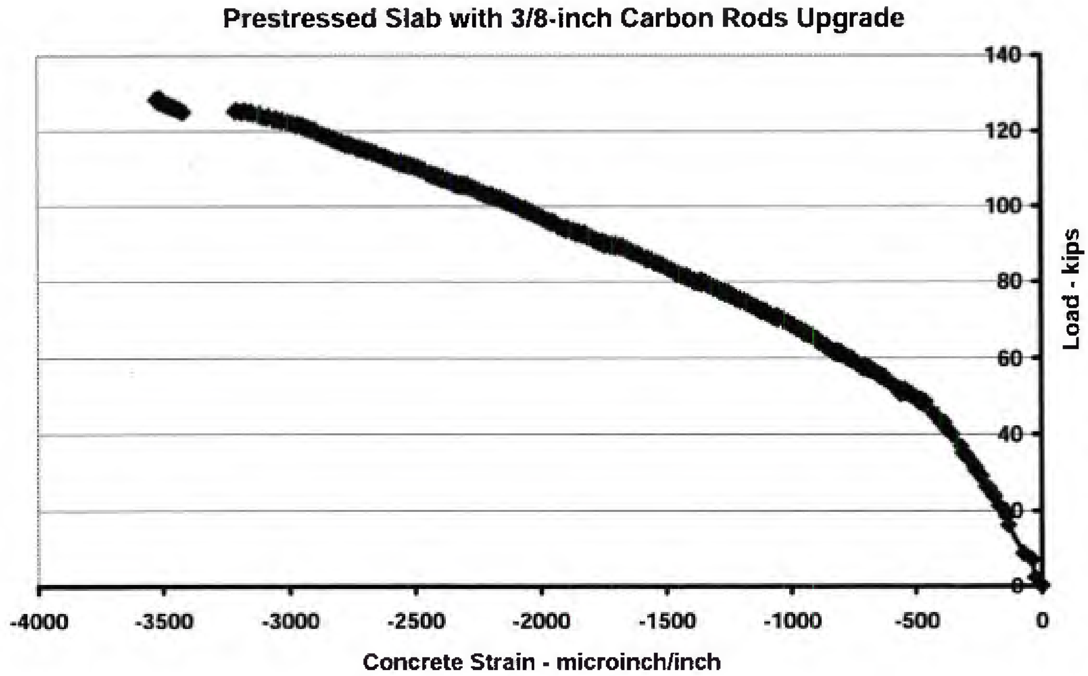


Figure B33. Load-Concrete Strain response of prestressed plank slab upgraded with embedded rods.

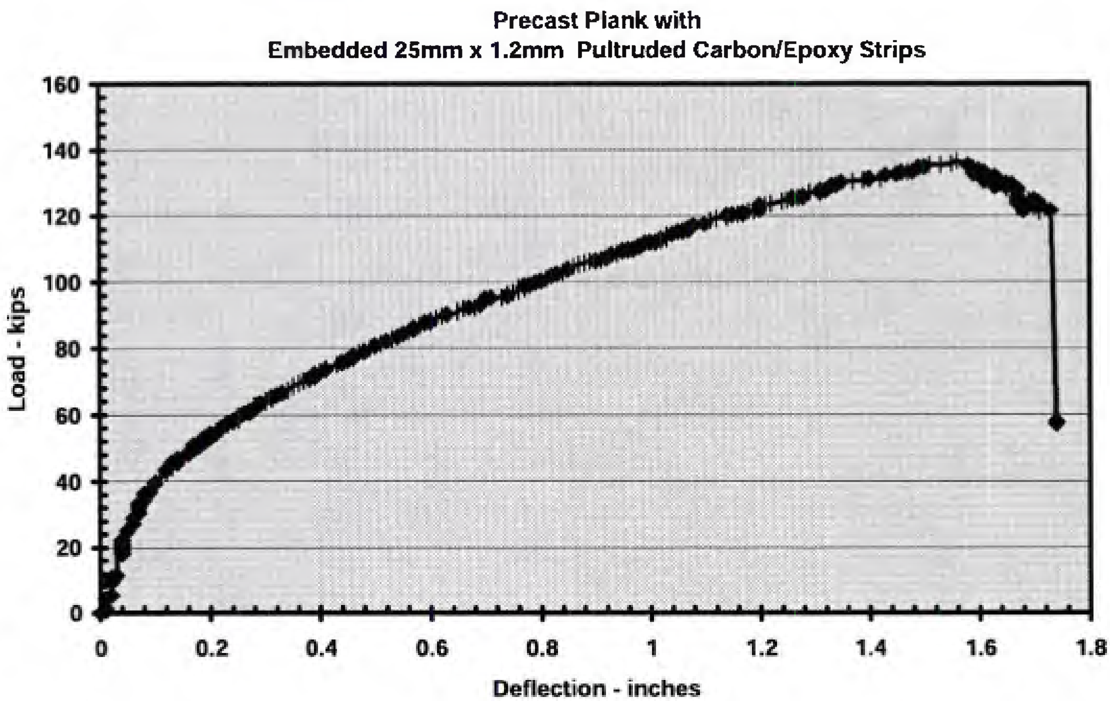


Figure B34. Load-Deflection response of precast plank upgraded with embedded carbon strips.

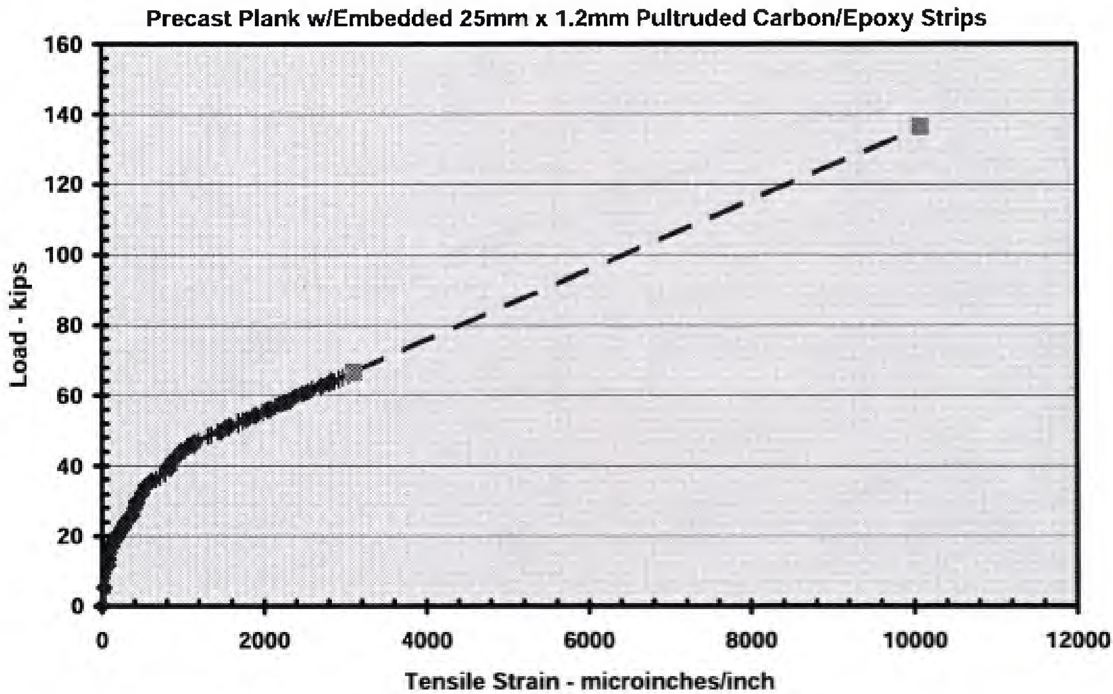


Figure B35. Load-Tensile Strain response of precast plank upgraded with embedded carbon strips.

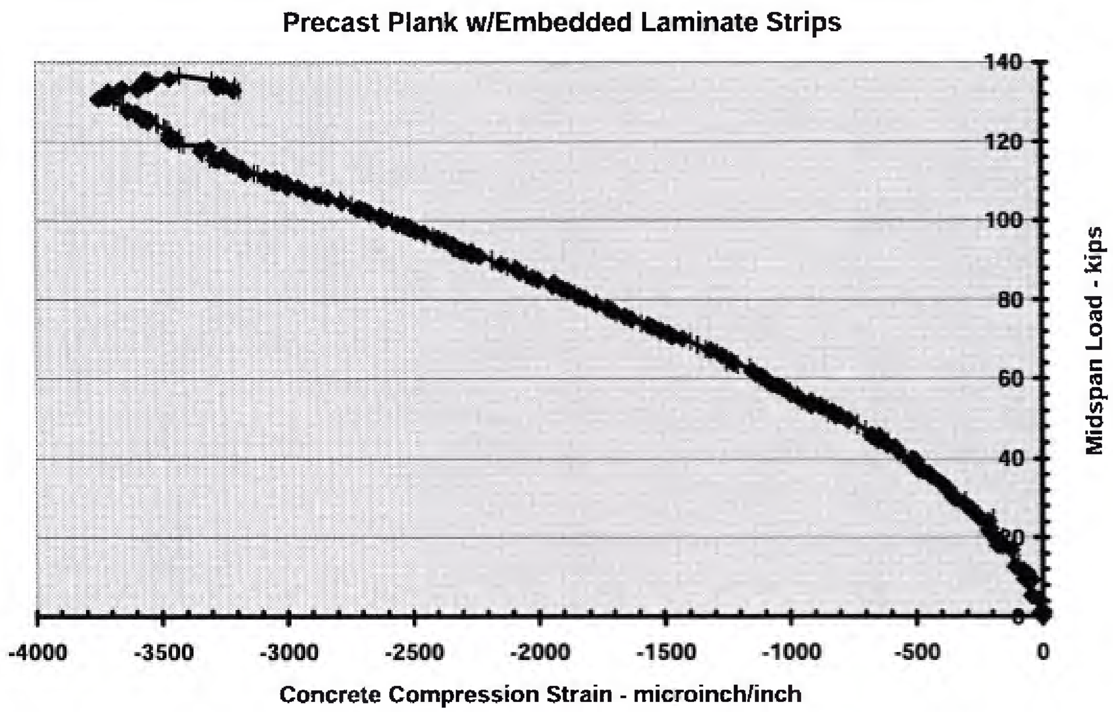


Figure B36. Load-Concrete Strain response of precast plank upgraded with embedded carbon strips.

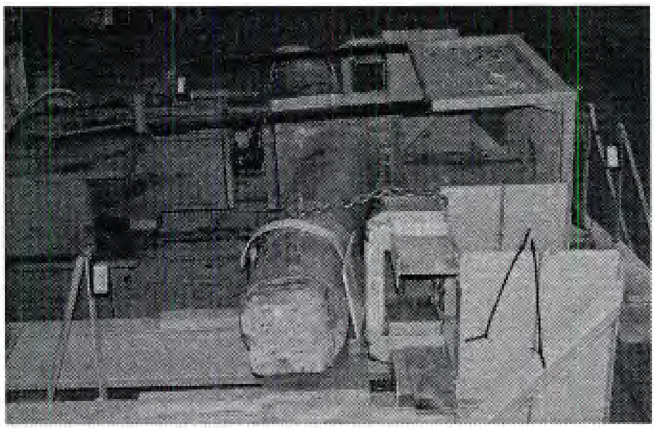


Figure B37. Testing Pile Confinement Specimen to failure.

Figure B38. Installing pile confinement upgrade demonstration on AWTTS pile.

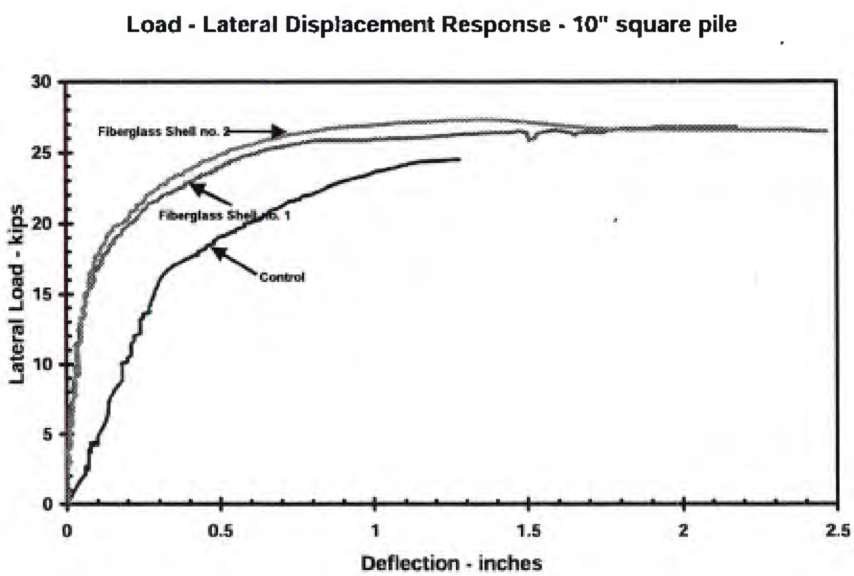


Figure B39. Lateral load-defection response of pile upgrade specimens.

Pile Confinement Concept Testing

NFESC evaluated concepts of confining existing piles by laboratory and AWTTS tests. The laboratory tests were conducted in NFESC's 400-kip hydraulic, lateral load test frame (**Figure B37**). The tests were conducted on two, fiberglass/vinyl ester composite shell configurations that were contributed by separate manufacturers for proof-of-concept evaluation. Each manufacturer prepared the specimens using NFESC prestressed pile sections that were in excess after the construction of the AWTTS. The specimens were prepared with a simulated pile cap. The pile sections were confined along a length 5 feet (m) from the pile cap. A shrink-resistant cementitious grout was pumped into the void between the shells and the rectangular shell section. The manufacturers also demonstrated installation on existing piles at the AWTTS (**Figure B38**). The laboratory specimens were tested to failure along with the existing pile configuration without the upgrade (control). **Figure B39** is a graphic of the lateral load-deflection response of the pile configurations. The piles were under reinforced for lateral, flexural resistance. In all cases including the baseline pile, the failures were caused by rupture of the prestressed steel reinforcing. The increase in stiffness and strength exhibited by the upgraded piles was due to the increase in cross-sectional area accompanied by an increase in internal moment arm of the tensile and compressive couples.

Quality Assurance Testing

NFESC implemented three procedures for maintaining control of the quality of materials and construction methodology. (1) We required submission of 1 percent of all materials to be submitted to NFESC engineers for test and evaluation. (2) The contractor was also responsible for testing on the site and by independent laboratories. (3) NFESC project engineers were on the site continuously during construction to monitor procedures and progress.

Material Coupon Testing

The contractors and installers were required to submit up to 1 percent of all materials to NFESC for laboratory testing. All composite materials were tested in the laboratory at NFESC. NFESC laboratory tests included coupon uniaxial tensile tests to failure, bond tests, and environmental testing. The tests quantified physical properties of the materials to verify manufacturer and installer claims as well as properties for finite element analyses. Since all of the composites were required to react in uniaxial tension, tension specimen coupons were prepared from each material system for load testing. The ends of the coupons were embedded in epoxy inside 1-inch diameter steel pipe (**Figure B40**) for gripping in a universal testing machine (**Figures B41 and B42**). We bonded wire and foil strain gages near the center of each specimen to measure the uniaxial strain response. We quantified the load - strain response, stiffness, and tensile strength of each composite used on Pier 12 (**Figure B43**). Results are provided in the report.

The contractor also submitted concrete repair material samples and laminate samples to independent laboratories for testing.

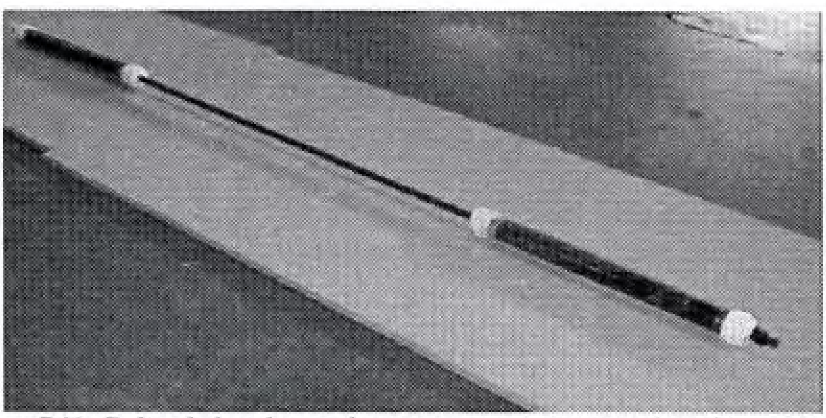


Figure B40. Pultruded carbon rod coupon test specimen anchored in steel pipe.

Figure B41. Coupon specimen in universal test machine.

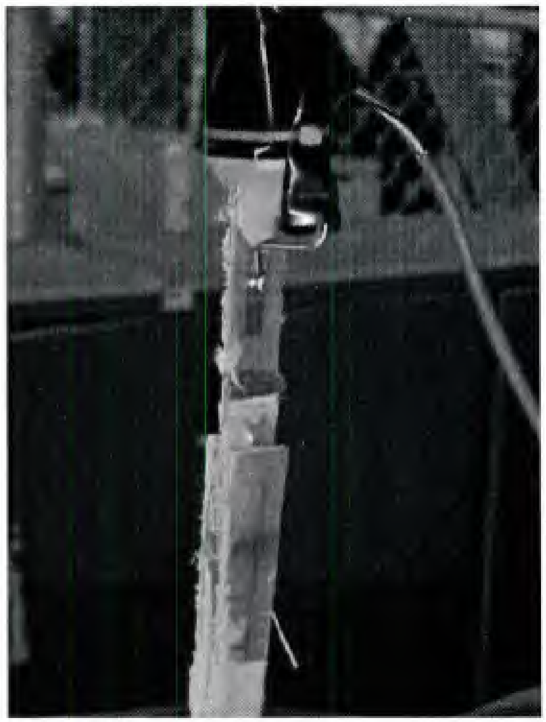


Figure B42. Double shear lap coupon specimen.

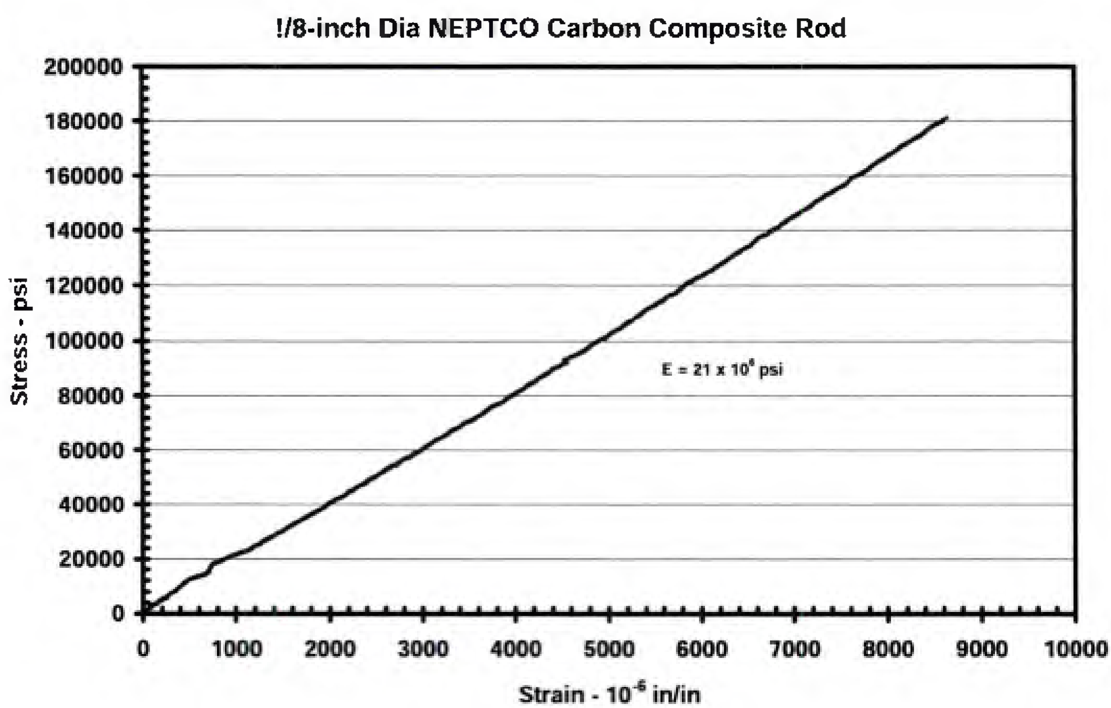


Figure B43. Stress-strain results from carbon rod coupon test.

Pultruded Rod-to-Epoxy Bond Tests

The Pier 12 Design required embedded reinforcement rods on the top deck surface. To quantify the bond between the encapsulate epoxy and the pultruded carbon laminate reinforcing rod, we conducted pull out bond tests using smooth rods embedded in the SIKA® epoxies. DFI Pultruded Composites, Inc provided the rods. We tested the rods with a slightly sanded surface to remove the "sheen". One end was embedded in 4 inches in epoxy (diameter 2 inches) while the other end of the rod was embedded in 12 inches of epoxy inside a 1-inch diameter pipe similar to the coupon specimens (**Figure B41**). The 12-inch length was clamped in a universal testing machine and the rod was pulled out of the 4-inch embedment. No clamping or confinement was placed on the 4-inch embedment. The bond was calculated as an average over the 4-inch embedment. The test parameters were epoxy encapsulate and the addition of sand to extend the epoxy volume. Adding sand also slightly decreased the coefficient of thermal expansion. We added 60 grit sand to the epoxy mixture (1 part sand to 2 parts epoxy by volume). The tests results are tabulated in **Table B5**. We determined that the added sand provided less variation in results but slightly reduced the bond strength and the wetability of the epoxy. It is reasonable to add 20 percent sand to the epoxy encapsulate and still have sufficient bond to the slightly roughened rods. The Sika® 32 epoxy had better bonding properties than the 35.

Table B5. Test Results of Pull Out Bond Specimens

Specimen	material	surface	Ultimate Load	Bond Strength	Average	std dev	Remarks
no 1	35	smooth	7308	1550.753			
no. 2	35	smooth	5390	1143.754			
no.1	35	smooth	4462	946.8335			
no.2	35	smooth	3506	743.9709	1096.328	250.9257	
no. 1	32	smooth	9730	2064.7			no failure
no.2	32	smooth	11820	2508.196			no failure
no.1	32	smooth	12226	2594.349			
no.2	32	smooth	7246	1537.596			
no.1	32	smooth	10200	2164.433			
no.2	32	smooth	5620	1192.56	2010.306	430.1516	
no.1	32	sanded	12736	2702.571			
no.2	32	sanded	8576	1819.822			
no.3	32	sanded	13098	2779.387			
no.4	32	sanded	13880	2945.327	2561.777	370.9775	
no 1	32 w/sand	sanded	10230	2170.799			
no.2	32 w/sand	sanded	10176	2159.34			
no.3	32 w/sand	sanded	10636	2256.952			
no.4	32 w/sand	sanded	10840	2300.241	2221.833	56.76332	

Figure B41. Epoxy bond specimen.

IN-SITU PULL-OFF BOND TESTS

All the techniques of adhering composite laminates to the concrete deck surface were required to demonstrate that the laminate-to-concrete interface was sound at the site. Testing included "pull-off" bond tests as required by the Specifications and thermographic (infrared) surveys of the laminate systems that were done of the contractor's own accord. The thermographic results were negative (a few insignificant flaws were found). The contractor conducted the "pull-off" tests on epoxy saturate systems, epoxy putty or grout systems, and other concrete bonding materials to demonstrate the bonding strength. These were conducted in accordance with ASTM D4541 and ISO 4624. The Specifications required laminate-to-concrete bond to be at least 300 psi (2 MPa) and develop the tensile capacity of the concrete.

To test the bond capacity the test surface was defined by making a circular incision through the laminate down to the concrete substrate using a 2-inch (50 mm) diameter core drill. The preferred failure is fracture of the concrete substrate, so the incision should score the concrete substrate. The circular incision ensured that the pulling force was applied only to the area inside the incision. A test disk (puck) was bonded to the laminate system at the measurement location using a standard, rapid set epoxy adhesive. After the adhesive is set the puck is then loaded perpendicular to the surface with a pull off tester (**Figure B43**). Failure occurred in the weakest plane between the puck and the concrete substrate. We recorded the failure stress as the maximum pull-off force divided by the area of circular incision. The results of the pull-off tests are in **TableB6**. The pull-off tests resulted in failure at an excess of 300 psi (2.1 Mpa). This insures interlaminar shear strength of more than 7.7 MPa that sufficient to develop the tensile strength of the external reinforcing before debonding.

Our pull off tests on Pier 12 and in the laboratory showed the penetrating sealer/primer epoxy (100 cps viscosity) significantly increased the tensile and impact strength of the first 1/8-inch (3 mm) layer of concrete. The increase in tensile strength is due to epoxy primer being absorbed into the microcracks of old concrete. This is significant for aging concrete marine structures on the Navy's waterfront. The pull-off failure was transferred deeper into the substrate where the concrete is more sound (**Figure B44**). NFESC considers application of a very low viscosity (100 cps) penetrating epoxy sealant/primer necessary on the porous concrete surfaces of the Navy's aged piers.

Outdoor NFESC application tests of the epoxy primer revealed the exposure to the sun and the inherent higher concrete surface temperatures caused evaporation and expanding air within the pores of the concrete. The expanding air outgases through the epoxy primer and causes bubbling and pinholing on the surface. After the primer seals the concrete, outgasing is not a problem for application of additional layers of epoxy. For the Pier 12 top deck applications, the primer was applied to the concrete in the afternoon after the concrete had reached maximum surface temperature.

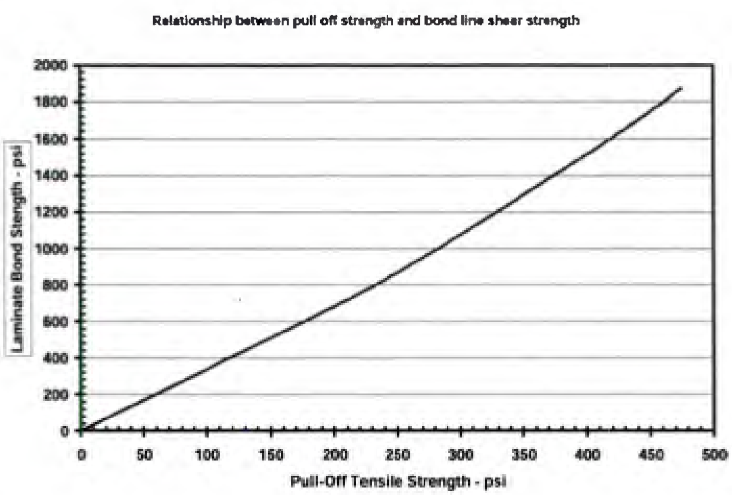


Figure B42. Relationship between laminate shear bond strength and direct tension bond.

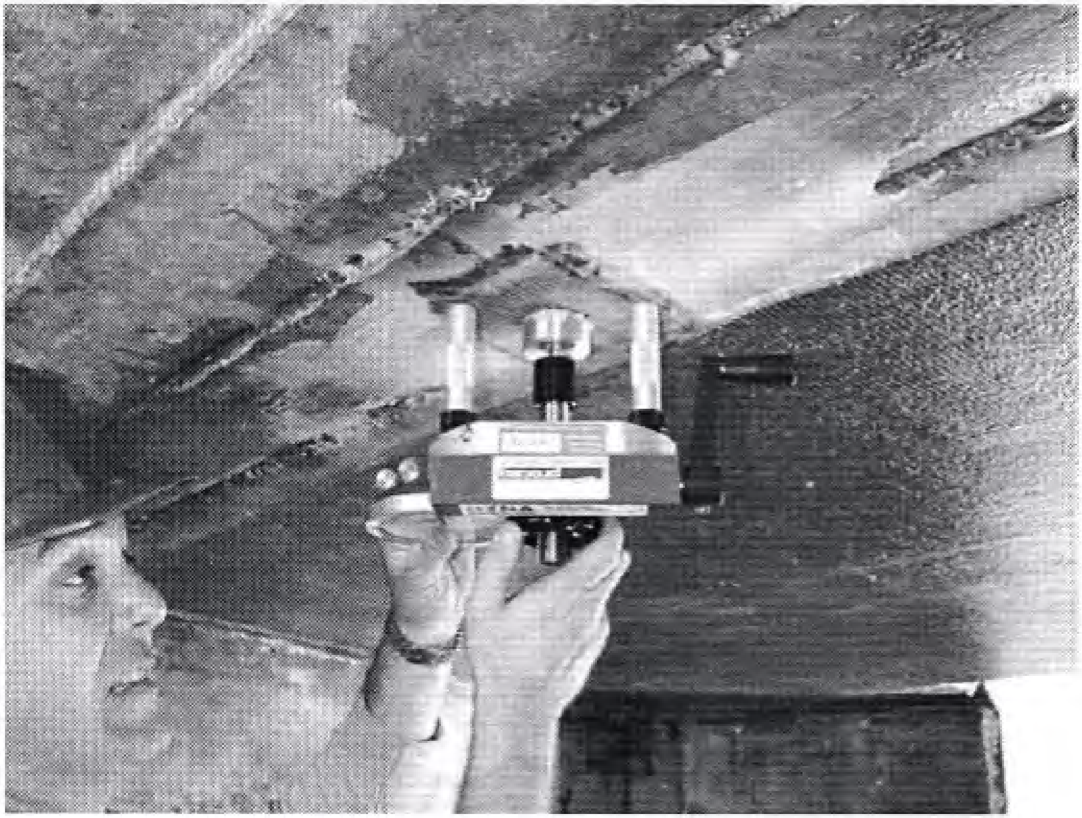


Figure B43. Bond testing with pull-off tester.

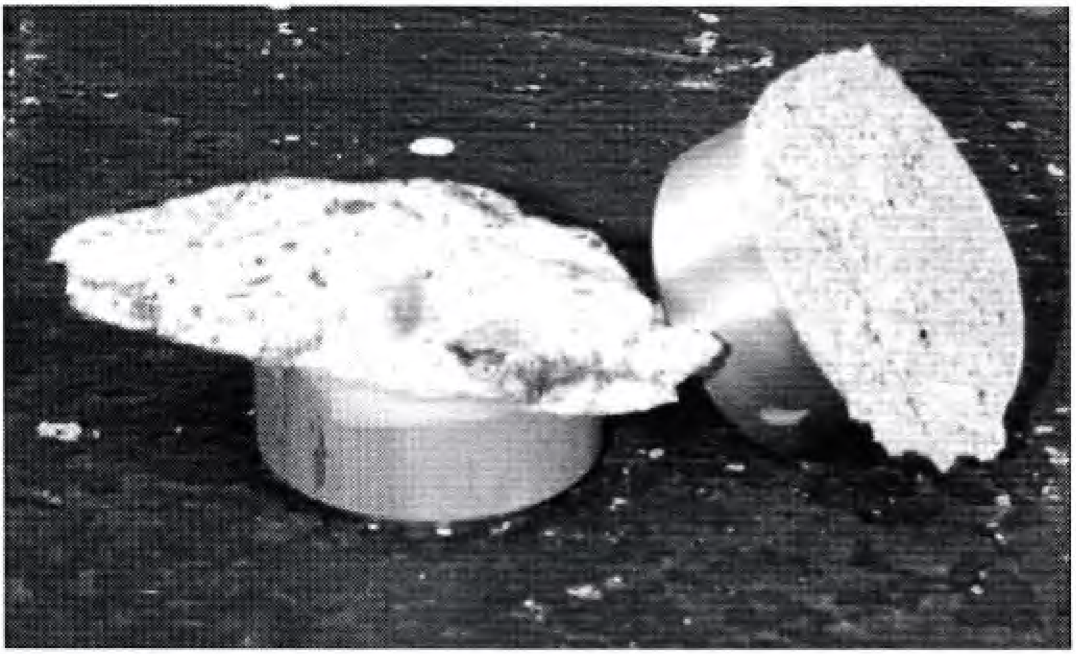


Figure B44. Pull-off test "pucks". Unprimed concrete surface on left puck and primed concrete surface on right puck.

Table B-6. Laminate Pull-off Test Results.

Composite	Location Bent	Bond Strength PSI	Comments
Sika Strip	10/11 S	500	Concrete tensile failure
Sika Strip	10/11 S	680	Puck adhesive failure
Sika Strip	10/11 S	625	Concrete tensile failure
Sika Strip	11/12 N	875	Composite bond failure
Sike Strip	11/12 N	640	Puck adhesive failure
Sika Strip	11/12 N	780	Concrete tensile failure
Fyfe Laminate	7/8 S	375	Laminate failure
Fyfe Laminate	7/8 N	325	Laminate failure
Fyfe Laminate	8/9 N	600	Concrete tensile failure
Fyfe Laminate	9/10 N	675	Laminate failure
Fyfe Laminate	11/12 S	680	Laminate debond
Fyfe Laminate	11/12 S	750	Laminate debond
I Beam	11/12 N	1150	Concrete tensile failure
I Beam	11/12 N	660	Concrete tensile failure
I Beam	10/11 S	310	Laminate debond
I Beam	10/11 S	350	Concrete tensile failure
I Beam	10/11 S	300	Concrete tensile failure
Sika Strip	67/68 N	675	Puck adhesive failure
Sika Strip	67/68 N	680	Laminate debond
Sika Strip	67/68 N	625	Puck adhesive failure
Sika Strip	68/69 S	525	Puck adhesive failure
Sika Strip	68/69 S	540	Puck adhesive failure
Sika Strip	68/69 S	725	Laminate debond
Fyfe Laminate	67/68 N	575	Laminate failure
Fyfe Laminate	67/68 N	585	Concrete tensile failure
Fyfe Laminate	67/68 N	375	Laminate debond
Fyfe Laminate	68/69 S	625	Laminate debond
Fyfe Laminate	68/69 S	500	Laminate debond
Fyfe Laminate	68/69 S	475	Laminate debond
I Beam	67/68 N	725	Concrete tensile failure
I Beam	67/68 N	775	Puck adhesive failure
I Beam	67/68 N	645	Concrete tensile failure
I Beam	68/69 S	785	Laminate debond
I Beam	68/69 S	775	Laminate debond
I Beam	68/69 S	640	Laminate debond

APPENDIX C:

SPECIFICATIONS FOR:

Concrete Repair and Crack Sealing in Preparation for Upgrade Reinforcement

CONCRETE REPAIR
PARTIAL DEPTH DECK REPAIRS

1. GENERAL

This specification covers the use of prepackaged cementitious concrete repair materials and procedures for making partial-depth repairs to Pier 12, Naval Station San Diego. California.

1.1 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to within the text by the basic designation only.

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO)

AASHTO T 2771989	Standard Method of Testing for Rapid Determination of the Chloride Permeability of Concrete

AMERICAN CONCRETE INSTITUTE

ACI 301 (1994)	Structural Concrete for Buildings

AMERICAN SOCIETY FOR TESTING AND MATERIAL (ASTM)

ASTM C 309 (1994)	Liquid Membrane-Forming Compounds for Curing Concrete
ASTM A 615	1993 Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
ASTM C 33	1993 Concrete Aggregates
ASTM C 109	1991 Standard Method for Compressive Strength of Hydraulic Cement Mortars
ASTM C 157	Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete
ASTM C 490	Standard Practice for Use of Apparatus for the Determination of Length Change of Hardened Cement Paste, Mortar, and Concrete
ASTM C 496	1990 Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
ASTM C 882	1991 Test Method for Bond Strength of Epoxy-Resin Systems Used with Concrete (modified for cementitious material)
ASTM C884	1987 Test Method for Thermal Compatibility Between Concrete and an Epoxy-Resin Overlay

1.2 DESCRIPTION OF WORK

Concrete repair work must be accomplished prior to application of composite upgrade materials. The concrete repair work consists of: A) Partial-depth repairs of deteriorated concrete on the top and bottom of the pier deck and, B) Sealing construction joints and cracks on the top of the deck to prevent water from wetting the underside of the deck. The repair work shall proceed by removing concrete from the areas identified by the contracting officer and NFESC using approved methods identified in the Contract Drawings and herein, cleaning the area by abrasive blasting, placing an approved bonding agent, placing an approved repair material, finishing and texturing, curing, and, finally, sealing joints and saw overcuts.

1.3 LOCATION

The Contracting Officer and NFESC will designate the locations and boundaries of each repair area with the Contractor. The Contractor will remove all unsound concrete and expose the rebar as necessary based on the repair criteria so that no visible corrosion is evident beyond normal "mill scale." Refer to Contract Drawings.

1.4 SUBMITTALS

Submit the following documentation and materials to the Contracting Officer prior to receiving authorization to proceed with concrete repair.

1.4.1 Manufacturer's Catalog Data/Instructions

a. Cementitious Repair Material
b. Curing Compounds

1.4.2 Laboratory Test Results and Verification

The contractor will submit to the Contracting Officer test results from an approved concrete laboratory showing that the repair material meets or exceeds the Navy's specifications on shrinkage and strength. Some laboratory tests shall be conducted on 35-day-old specimens. Therefore, it would be prudent for the Contractor to prepare for these tests well in advance to avoid delays in the concrete repair work.

1.4.3 Batch Samples

When requested by the NFESC, the Contractor shall provide batch samples of any materials that are used throughout the progression of the work. Batch samples will not exceed 1% of the total material used on this job.

1.5 DELIVERY, STORAGE, AND HANDLING

Inspect materials delivered to site for damage, unload and store with a minimum of handling. Deliver cementitious repair material components and aggregate materials in original sealed containers and store in dry covered areas at temperatures below 100°F.

1.6 WEATHER LIMITATIONS

Halt work when the weather conditions are inclement and detrimentally affect the quality of patching concrete. Windy conditions and rain will effect the concrete curing. Apply patching materials only when the atmospheric and surface temperature ranges are suitable for the specified material. Halt work if the temperature is below 40°F (4°C). Follow manufacturer's instructions for weather conditions and temperature ranges. Patches placed during adverse weather conditions may have to be removed and replaced.

1.7 EQUIPMENT

Use a container recommended by the manufacturer as the mixing vessel. Mixing vessels shall be reasonably clean and free of cured and partially cured repair materials and polymers. Use equipment specified by repair material manufacturer for field mixing, transporting and consolidation of cementitious repair materials.

1.8 QUALITY ASSURANCE

A Technical Representative of the manufacturer of the cementitious repair material being used shall be present during the start of repair work. The Technical Representative shall inspect and approve the surface preparation and observe the initial application. The Technical Representative may demonstrate and instruct the contractor on proper procedures.

A written report shall be submitted to the Contracting Officer outlining the Technical Representative's observations and suggestions, including but not limited to recommendations regarding mixing and placement procedures, and equipment used for mixing, placement, consolidation, and curing.

Mixing and handling instructions shall be available at the site at all times during the repair operations.

Throughout the progress of the work in this Specification, the Contractor will provide at least one (1) person certified by the material manufacturer who is thoroughly familiar with the specified requirements, completely trained and experienced with the necessary skills, and who will be present on the site and direct all work performed under this Specification.

In performing the work of this Specification, the Contractor will use an adequate number of skilled workmen to ensure the installation is in strict accordance with schedule, specification and the procedures recommended by the material manufacturer and the NFESC.

2. MATERIALS

2.1 MATERIAL SPECIFICATIONS

The materials used shall meet the requirements of the following specifications as well as other Contracting Officer and NFESC approved Proprietary repair materials:

AASHTO M-80 & M-6	Aggregate
AASHTO M-148	Curing compound
AASHTO M-194	Concrete admixtures

2.1.1 Cementitious Patch Material

The product shall be prepackaged by the manufacturer with premeasured, properly proportioned components. It shall be suitable for the hand-packed repair method (see Contract Drawings) and shall have the following properties:

a. Minimum pot life of 15 minutes at 75°F.

b. Bond strength per ASTM C 882 modified for cementitious material at 28 days: 1,200 psi minimum.

c. Maximum permeability of 1,000 coulombs per AASHTO T 277.

d. Drying shrinkage: Specimens shall be prepared per ASTM C 157 as modified to use molds per ASTM C 490 (3x3x11.25 inches) with a 10-inch gauge length. During the first 7 days the molded specimen shall be covered with a water-saturated rug or burlap. After the 7 day wet curing period the mold shall be removed and the specimen cured for an additional 28 days at 46 to 54% relative humidity at 70 to 76°F. The ultimate shrinkage to be reported is that value measured at the end of the 35th day. Allowable shrinkage shall not exceed 0.05%.

e. Minimum compressive strength per ASTM C 109 modified for cementitious material shall be 3,000 psi @ 3 days.

f. The water to cementitious material ratio shall not exceed 0.40.

2.1.2 Aggregate

If aggregate is added to the prebagged mixture, then all tests for acceptance criteria per Section 2.1.1 shall be conducted with the added aggregate. Aggregate added to the repair material, if allowed by the manufacturer, shall be 3/8-inch minus, clean, well graded, saturated surface dry material, having low absorption and high density, and conform to ASTM C 33. Aggregate must be approved for use by the Contracting Officer and the NFESC.

2.1.3 Reinforcing Bars

ACI 301 unless specified otherwise. ASTM A 615, Grade 60 bars.

2.1.4

Laboratory tests per Section 2.1.1 shall be submitted to the Contracting Officer for approval before concrete repair can proceed. Laboratory tests shall be conducted on 35-day-old specimens. Therefore, it would be prudent for the Contractor to prepare for these tests well in advance to avoid delays in the concrete repair work.

3. EQUIPMENT

3.1 GENERAL

The Contractor shall furnish and maintain such equipment as necessary to complete the work in accordance with the specifications.

3.2 CONCRETE SAW

The concrete saw shall be equipped with a diamond blade(s) or approved equal. The saw shall be capable of sawing concrete to the specified depth without damaging the surrounding concrete. Depth of cut shall be adjusted so as to avoid cutting the existing steel reinforcement.

3.3 CONCRETE REMOVAL EQUIPMENT

The Contractor shall provide equipment capable of removing the deteriorated concrete in the repair area to the depth required without damaging the sound concrete surrounding or below the repair. The Contractor shall provide the necessary means to assure that no concrete debris or slurry water enters the bay waters, refer to SITE DEMOLITION Specifications.

3.3.1 Pneumatic Jackhammers

Jackhammers heavier than 15 pounds (6.8 kg) shall not be permitted.

3.3.2 Abrasive Blasting or Mechanical Scarification

Abrasive blasting or mechanical scarification shall be capable of removing all contaminants and loose particles from the surface of the steel reinforcement and concrete in the repair area. The equipment shall be fitted with suitable traps, filters, drip pans, or other devices to prevent oil, fuel, grease, or other undesirable matter from being deposited on the cleaned surface and the Bay waters.

3.3.3 Brooms, Shovels

Stiff-bristled brushes shall be used to apply the bonding agent. Shovels may be used to place the repair materials, if appropriate.

3.4 FINISHING AND FLOATING EQUIPMENT AND STRAIGHTEDGES

The finishing and floating equipment shall be capable of consolidating and floating the concrete. A dense, homogenous repair must be produced and finished to the same surface slope as the existing concrete slab.

3.4.1 Pressure Hand Sprayer for Membrane-Curing Compounds

The pressure sprayer for membrane-curing compounds shall be capable of providing a uniform, even coating of the compound over the surface of the repair. Manually operated spray equipment may be used.

4. CONSTRUCTION METHOD

4.1 DETERMINATION OF REPAIR AREAS

The NFESC and the Contracting Officer shall determine areas to be repaired by using a hammer or other techniques to determine the extent of the unsound concrete. The NFESC shall mark the boundaries of the repair area. Large areas may use flowable repair materials while small areas and all areas below deck shall be repaired by the dry-pack method. Holes through the deck will be either filled with low shrinkage concrete.

4.2 PREPARATION OF REPAIR AREA

A hand-held 15-lb chipping hammer may be used. The Contracting Officer and NFESC must approve all other methods.

4.2.1 Concrete Removal

The deteriorated material in the repair area shall be removed using the methods specified in this section. A saw cut shall be made around the perimeter of the repair area to provide a vertical face at the edges and sufficient depth for the repair. The saw cut shall have a minimum depth of 1-inch (25 mm). Depth of cut shall be selected to preclude cutting reinforcing steel bars.

Concrete within the repair area shall be broken out to a minimum depth of 2 inches (50 mm) or until sound concrete is exposed.

Remove loose concrete from the designated areas. Inspect the cavity for remaining unsound concrete by tapping with a hammer or steel rod. In areas where tapping indicates unsound concrete, remove additional concrete. Make the entire cavity at least 2 inches (50 mm) deep. Where rebar is exposed remove all corrosion by abrasive blasting or mechanical means to a near white metal condition as per recommendations of patch material manufacturer, prior to installing patch material. Continue to "chase" all corroded steel reinforcement until no corrosion is visible beyond normal "mill scale." Prepare surfaces by abrasive blasting or mechanical scarification to achieve a uniformly rough surface.

4.2.2 Hand-Held Chipping Hammer

Concrete within the repair area shall be broken out to a minimum depth of 2 inches (50 mm) with pneumatic tools until sound concrete is exposed. The maximum size pneumatic hammer shall be 15 pounds (6.85 kg). Pneumatic hammers and chipping tools shall not be operated at an angle exceeding 45 degrees from the vertical. Such tools may be started in the vertical position but must be immediately tilted to a 45-degree operating angle. The removal shall start within the interior of the repair and work outward. Care shall be used to prevent fracture of the sound concrete below the repair area and the surrounding concrete. A minimum 1-inch (25-mm) vertical face (saw cut) on all sides shall be provided. However, adjustments shall be made to avoid cutting any steel rebar. All concrete chips/debris shall be contained and prevented from falling into the bay waters.

4.3 SURFACE PREPARATION

4.3.1 Concrete

Abrasive blast or mechanically scarify the exposed faces of the concrete to remove all loose particles, oil, dust, cement or slurry residue, paint, and other contaminants. Immediately prior to placing the concrete bonding agent, clean the exposed surfaces by compressed air blasting. All loose particles, oil, dust, cement or slurry residue, paint, and other contaminants shall be contained and prevented from falling into the bay water.

4.3.2 Steel Reinforcement

Reinforcing steel bars that has lost more than 25% cross sectional area must be repaired by welding a new segment of rebar of the same diameter to the existing rebar. Corroded or damaged rebar will be identified in the field by the contractor and verified for replacement by the Contracting Officer. The splice will cross the damaged length and the welds made at locations where the existing rebar is in excellent condition without loss of area. New reinforcing steel shall be ASTM A-615 grade 60 and welded in accordance with the Structural Welding Code – Reinforcing Steel (AWS D1.4). The welding surface shall be prepared by power cleaning as per SSPC-SP11. The weld will be a continuous 0.25-inch fillet that is at least 2-inches long. The contractor will remove any concrete that is damaged during the welding process. Abrasive blast or mechanically clean the steel to bright steel no more than 48 hours prior to application of concrete patch material. All loose particles, oil, dust, cement or slurry residue, paint, and other contaminants shall be contained and prevented from falling into the bay water.

4.4 APPLYING THE BONDING AGENT

Use a bonding agent recommended by the supplier of the repair material. It may consist of neat cement, cement-sand, or latex-cement slurry. Bonding agents must be approved by NFESC and the Contracting Officer. Apply the bonding agent to a clean surface saturated dry (SSD) concrete substrate and scrub it into the surfaces using a stiff-bristled brush. Bonding agents that contain epoxy will not be allowed.

4.5 PLACING THE REPAIR MATERIAL

Always place materials containing aggregate with a shovel to avoid segregation. Flowable materials may be placed by a bucket or other suitable means.

4.5.1 Proprietary Repair Materials

Place dry pack repair materials according to the method in the Contract Drawing. The application shall be in accordance with manufacturer's recommendations. Special attention shall be paid to pack the material below reinforcing bars and to working the material into the concrete substrate to achieve a sound bond. Use a hard wood dowel to ram the material tightly below and around reinforcing.

4.6 FINISHING REQUIREMENTS

Partial-depth repairs are usually small enough so that a stiff board resting on adjacent sound concrete can be used as a screed. Work the materials toward the perimeter of the patch to establish contact and enhance bonding to the existing slab. Make at least two passes with the screed to ensure a smooth repair surface that is level with the surface of the deck. Care should be taken to not "over-work" the surface. When practical, match the surface texture of the repair with that of the surrounding deck.

4.7 CURING

Spray apply two coats of a concrete curing compound (ASTM C 309) as soon as the concrete surface has set sufficiently to apply the curing agent without damage. Apply the curing compound at the rate of 150 ft^2/gal (3.7 m^2/L). In addition, repairs to the top deck shall also be moist cure for 7 days by covering with saturated pieces of wet rug or carpet.

4.8 SAW OVERCUTS

The saw cuts extending from the repair area into to the surrounding sound concrete must be filled with epoxy mortar or cement mortar.

4.9 OPENING TO PIER OPERATIONS

The concrete repairs may be opened to pier operations when a compressive strength of 3,000-psi (21 Mpa) has been achieved.

5. QUALITY CONTROL

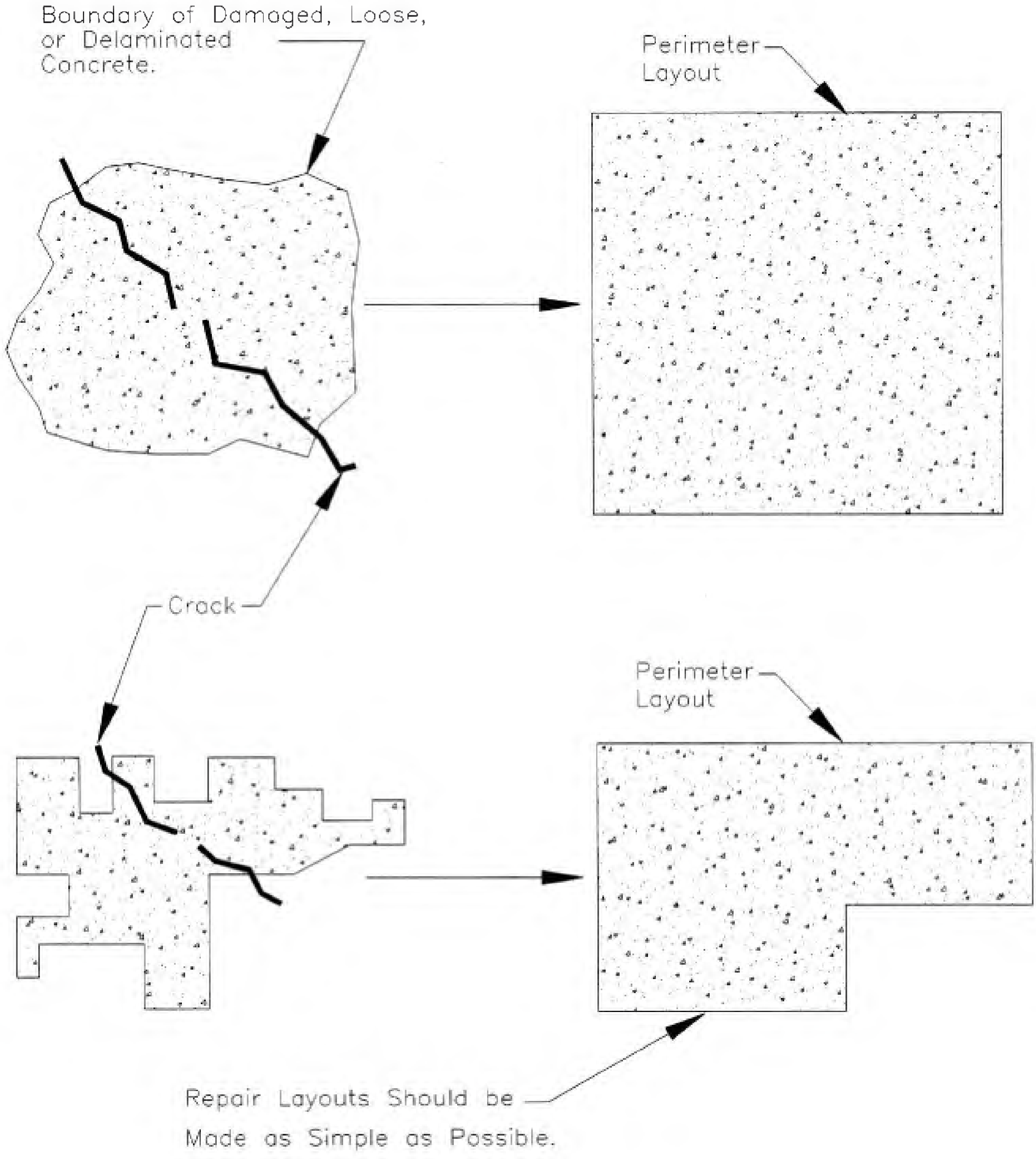
5.1 CONCRETE REPAIR MATERIALS

Material supplied to the job shall comply with Section 2.1.1 of this specification. Test reports shall be from an independent testing laboratory approved by the Contracting Officer.

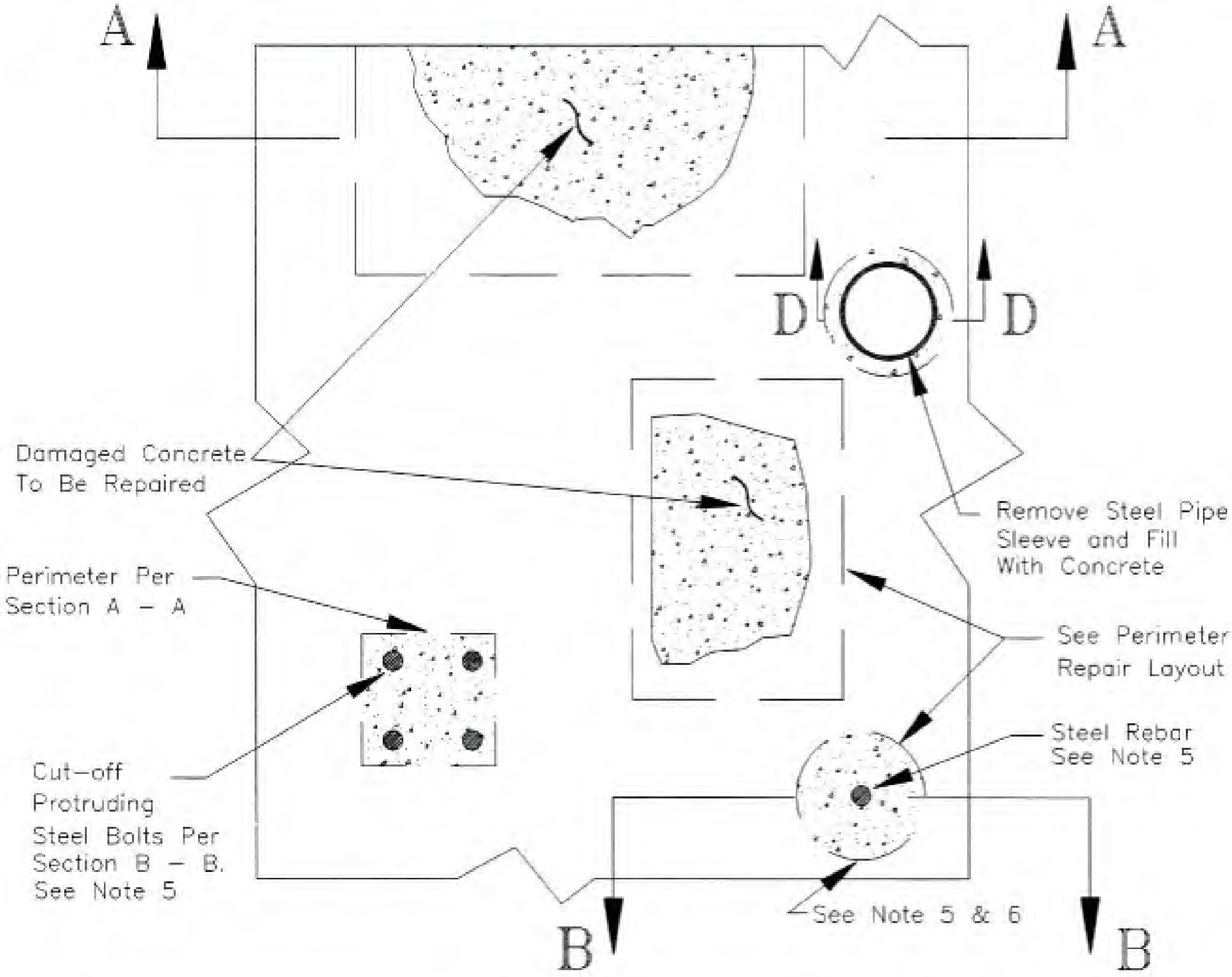
5.3 INSPECTION

The Contracting Officer and NFESC shall check each repaired area for cracks, spalls, popouts, and loss of bond between repaired area and surrounding concrete one week after the repair material was placed. Each repair area will be checked for voids by tapping with a hammer. In addition, the NFESC may take one, 1-inch diameter core, in each span to verify depth, bonding integrity, and material quality of the concrete repair. The contractor shall repair the cored site to the same level as required by this specification. Areas found to be defective will be removed and replaced by the Contractor to the satisfaction of the NFESC and the required performance and quality level of this specification.

Concrete Repair Contract Drawings

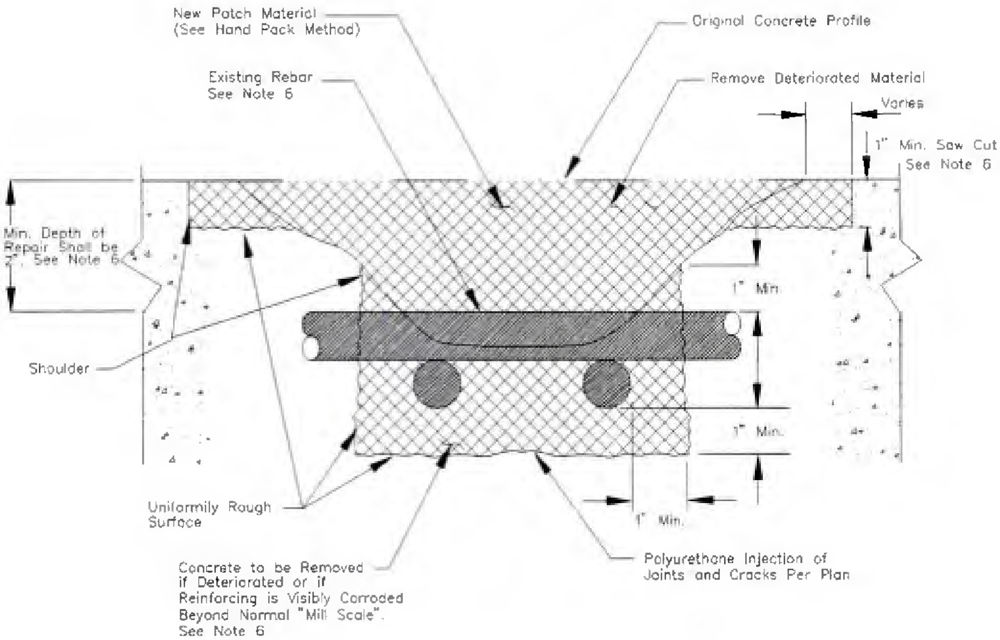


ILLUSTRATIVE REPAIR PERIMETER LAYOUTS

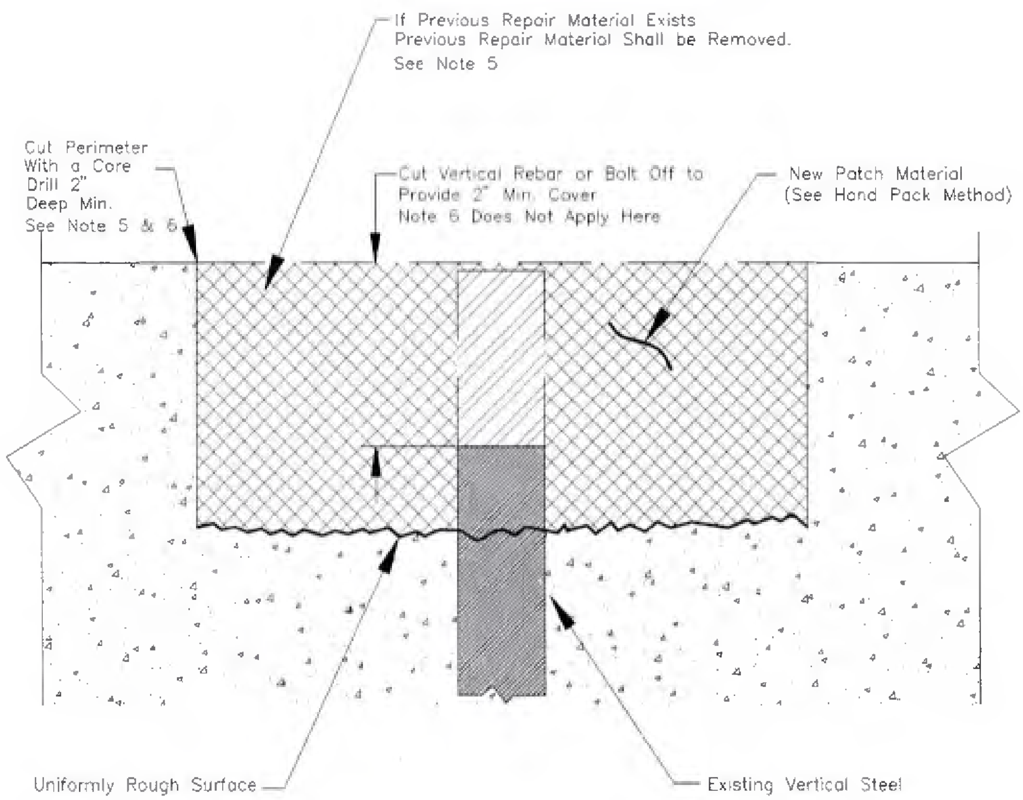


NOTE:
SAW CUT 1 INCH DEEP SHOULDER (SEE NOTE 6) AROUND THE PERIMETER OF REPAIR AREA AND REMOVE DETERIORATED CONCRETE ACCORDING TO THE SPECIFICATION TO A LEVEL OF SOUND CONCRETE AND CHASE THE REBAR TO THE POINT WHERE NO CORROSION IS VISIBLE BEYOND NORMAL MILL SCALE. DO NOT CUT THROUGH THE ENTIRE DECK THICKNESS. DO NOT FEATHER EDGES. IF REBAR IS EXPOSED, REMOVE MATERIAL TO A DEPTH AT LEAST 1 INCH BEYOND THE EXISTING REBAR. REBAR TO REMAIN INTACT. PREPARE CONCRETE AND REBAR SURFACE BY SAND BLASTING OR BY MECHANICAL MEANS SUCH AS SCARIFYING OR NEEDLE GUN (SEE SPECIFICATIONS). APPLY PATCH MATERIAL USING THE HAND PACK METHOD. FINISH SURFACE TO MATCH THE ORIGINAL PROFILE.

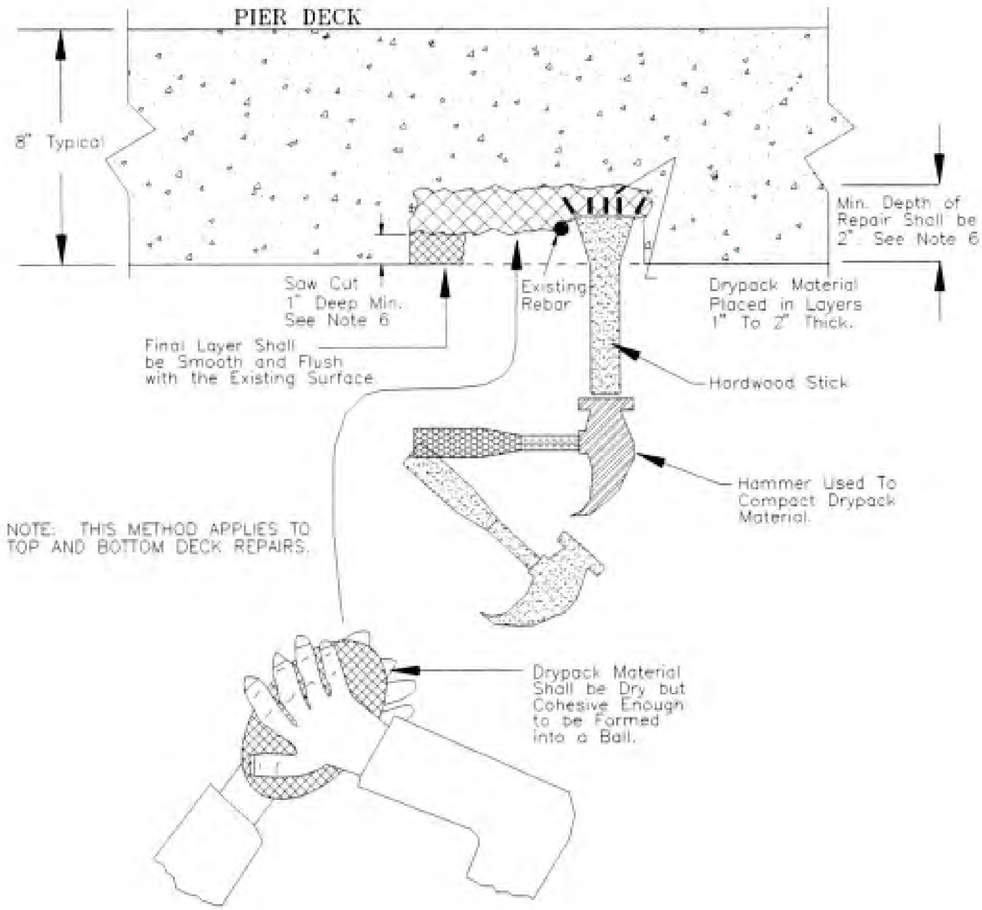
PARTIAL DEPTH REPAIRS



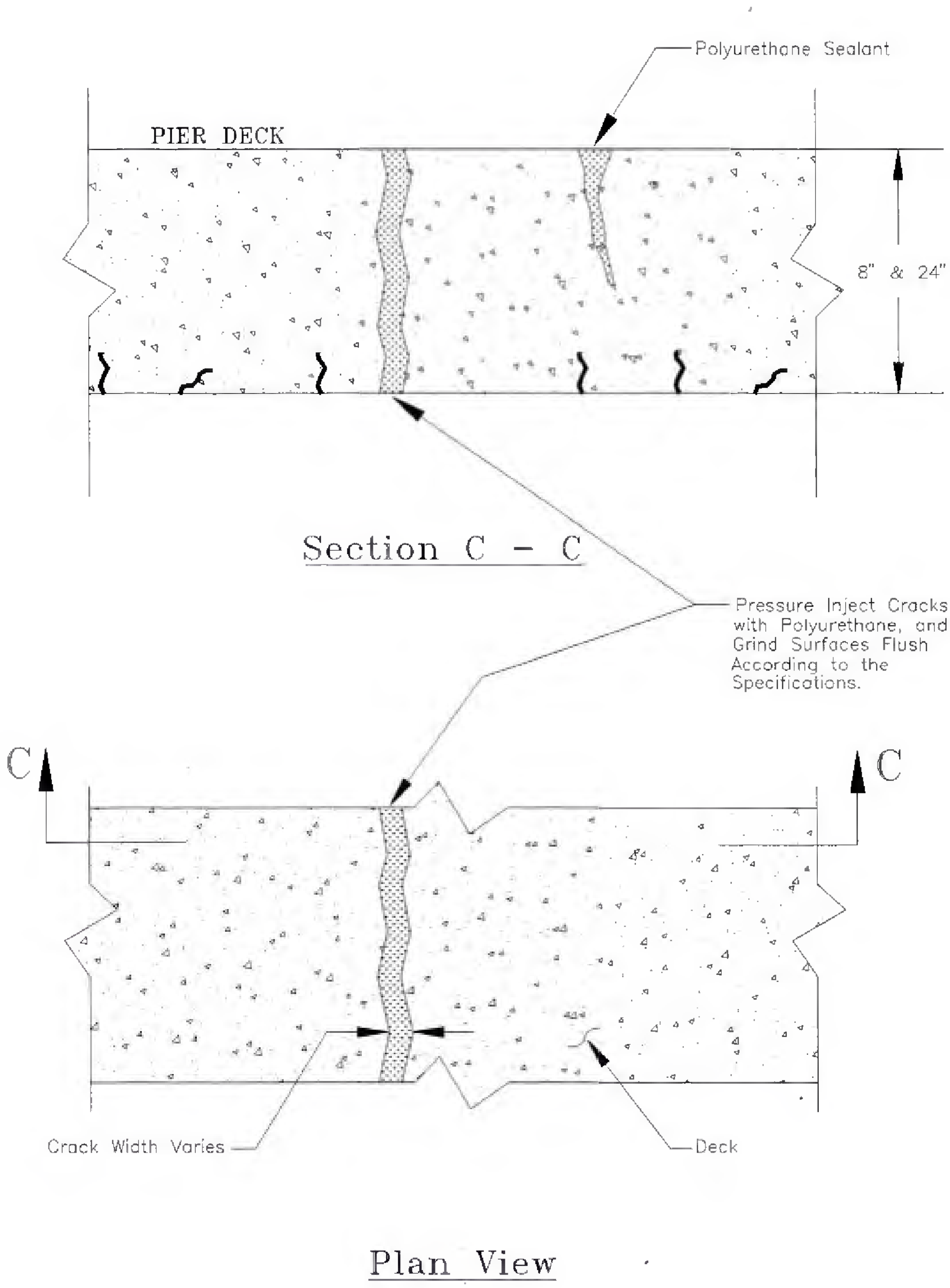
REPAIR SECTION A – A

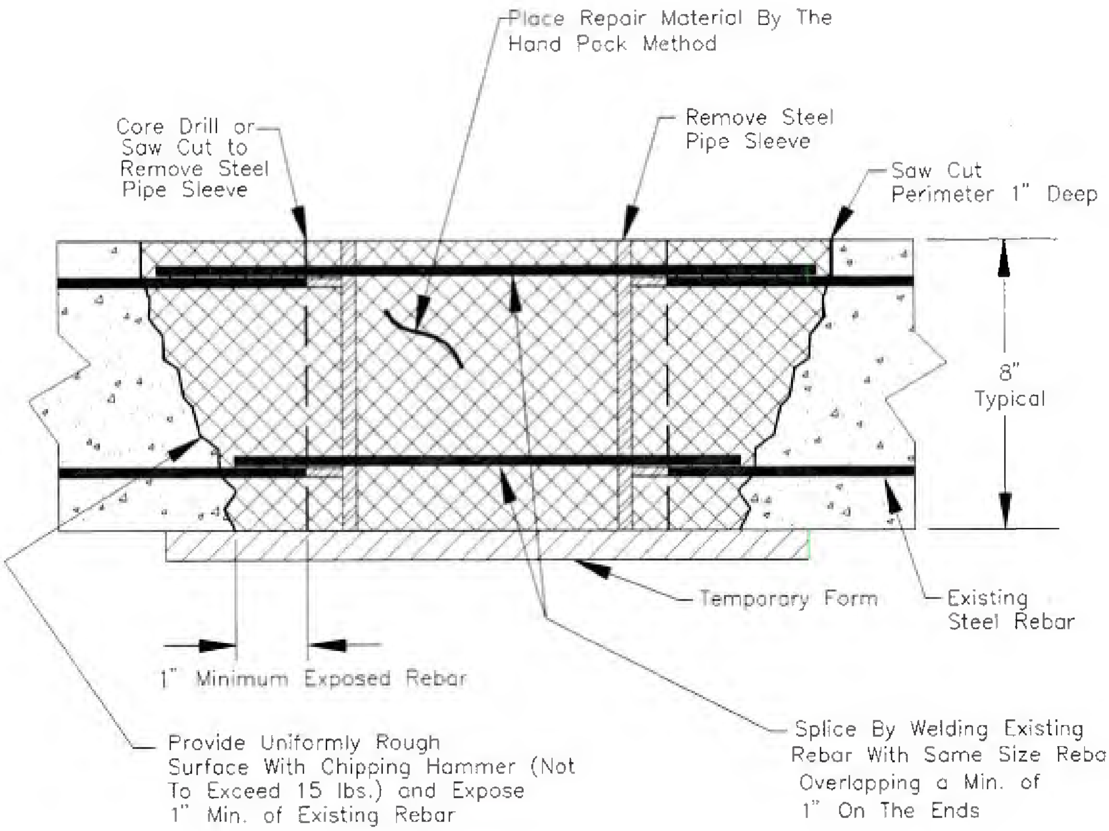


REPAIR SECTION B – B

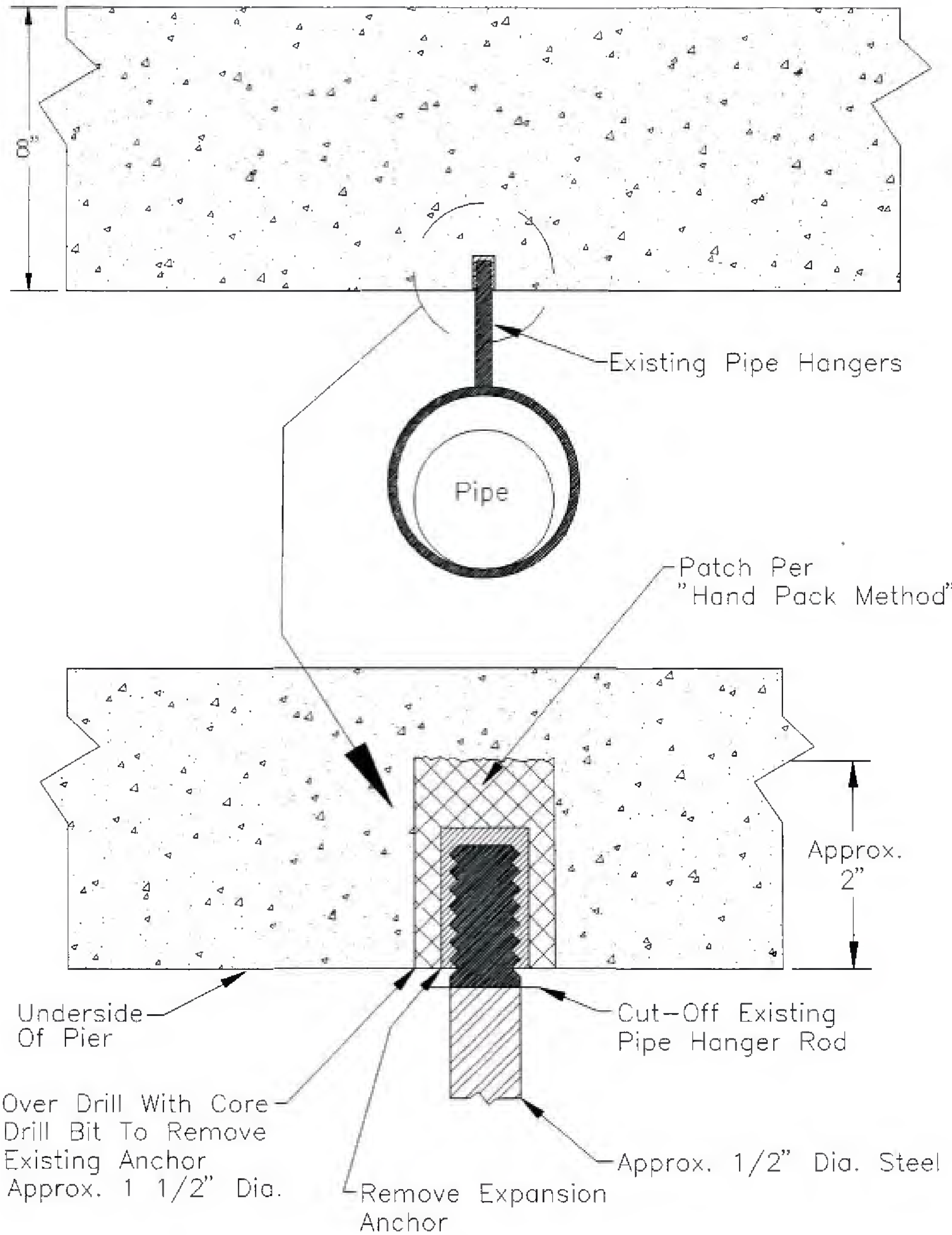


Hand Pack Method


Polyurethane Sealant
PIER DECK
8" & 24"
Section C - C
Pressure Inject Cracks with Polyurethane, and Grind Surfaces Flush According to the Specifications.
C
C
Crack Width Varies
Deck
Plan View



Section D – D
Full Depth Repairs



REMOVE PIPE HANGER ANCHOR
AND REPAIR CONCRETE

NOTES:

1. REPAIR DETAILS ARE ALL PARTIAL DEPTH AND APPLY TO THE TOP AND BOTTOM OF THE DECK. DO NOT CUT THROUGH ENTIRE DEPTH THICKNESS, EXCEPT FOR SECTION D-D

2. REPLACE ALL REBAR WHERE THE DIAMETER OF THE REBAR REMAINING AFTER CLEANING IS LESS THAN 80% OF THE ORIGINAL REBAR DIAMETER. SPLICE NEW REBAR TO EXISTING WITH A MECHANICAL COUPLING DEVICE. COUPLING DEVICE SHALL DEVELOP 130% (MIN.) OF THE YIELD STRENGTH OF THE REINFORCEMENT TO BE SPLICED.

3. ALL DEBRIS, SAW CUT CEMENT POWDER, AND SLURRY SHALL NOT BE ALLOWED TO ENTER THE BAY WATERS.

4. CEMENTITIOUS PATCH MATERIAL SHALL HAVE A MAXIMUM ALLOWABLE SHRINKAGE OF 0.05%. (SEE SPECIFICATIONS)

5. SEVERAL HOLES WERE CUT INTO THE DECK IN THE PAST AND PATCHED. TYPICALLY, A STEEL BAR WAS PLACED VERTICALLY IN THE CENTER OF THE REPAIR. THESE AREAS SHALL BE REPAIRED AS ILLUSTRATED IN SECTION B-B.

6. NEVER UNDER ANY CIRCUMSTANCES CUT OR DAMAGE THE EXISTING DECK REINFORCEMENT WITHOUT PRIOR APPROVAL FROM THE PROJECT ENGINEER. USE A REBAR LOCATOR (PACHOMETER) PRIOR TO CUTTING OR REMOVAL OF CONCRETE TO VERIFY DEPTH AND LOCATION OF REBAR. ADJUST DEPTH OF CUT ACCORDINGLY TO AVOID DAMAGING THE REBAR. EXCEPT FOR SECTION D-D.

CONCRETE REPAIR
CRACK SEALANTS

Cracks provide direct access for corrosive agents to attack steel reinforcement. Cracks in the top deck may also allow water to pond over the carbon fiber reinforcement applied to the underside of the deck. Cracks in the top deck will be filled to enhance the performance of the upgrade system (Figure E6).

The cracks in the pier are hairline to almost 1/4-inch wide and are likely to be "working" due to temperature changes, rebar corrosion, or operational loading. For hairline cracks up to 1/16-inch wide, pressure inject a water-activated polyurethane injection grout. For cracks greater than 1/16-inch wide, use a chemical-resistant polyurethane elastomer. Polyurethane crack sealants will minimize concrete water mitigation. Because polyurethane is relatively soft, it shall be injected after abrasive surface conditioning of the deck in preparation of adding the top surface upgrade reinforcing.

Crack Sealing Procedure Requirements

The basic **injection** procedure steps for use on both hairline cracks and cracks up to 1/16 inch in width are as follows:

1. Crack shall be free of dirt, debris, efflorescence, chipped concrete, and other obtrusive
2. Locate all rebar and conduit a minimum of 3 inches in width on both sides along the entire crack length.
3. Injection holes shall be drilled every 6 to 12 inches on alternating sides of the crack without disturbing or damaging existing rebar and conduit.
4. Install and secure mechanical packers into injection holes.
5. Flush crack through mechanical packers with a solution of 5 to 7 percent by volume muriatic acid followed by flushing with potable water.
6. Inject resin through mechanical packers at pressures exceeding 1,000 psi.
7. Remove mechanical packers.
8. Repair injection holes using non-shrink cementitious mortar.
9. Grind surface flush with surrounding concrete.

The basic expansion joint **sealing** procedure for use on cracks greater than 1/16 inch in width is as follows:

1. Locate all rebar and conduit a minimum of 3 inches on both sides of the crack along the entire crack length.
2. Route out crack to a depth-to-width ratio of 1:2 taking care to not disturb or damage existing rebar and conduit.
3. The crack shall be free of dirt, debris, efflorescence, chipped concrete, and other obtrusive materials.
4. Apply a continuous strip of polyethylene bond breaker tape to the inner base of the crack. Tape shall be flush with sides of crack.
5. Immediately following the day's high temperature and on dry concrete, trowel in two component polyurethane elastomer.
6. Trowel flush with adjacent concrete.

POLYURETHANE CRACK SEALANT

This specification applies to the requirements for repairing either static or dynamic concrete cracks with less than 25% movement that are **1/16 inch or wider**. This specification details both application procedures and material requirement.

PART 1 GENERAL

1.1 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to within the text by the basic designation only.

AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)

ASTM D 412	1997 Test Method for Vulcanized Rubber and Thermoplastic Rubbers and Thermoplastic Elastomers - Tension
ASTM D 638	1996 Test Method for Tensile Properties of Plastics
ASTM D 2240	1997 Test Method for Rubber Property - Durometer Hardness

CODE OF FEDERAL REGULATIONS (CFR)

29 CFR 1910.134	Respiratory Protection
29 CFR 1926.59	Hazard Communication
40 CFR 261	Identification and Listing of Hazardous Waste

1.2 SUBMITTALS

Submit the following documentation and materials to the Contracting Officer prior to receiving authorization to proceed with crack sealing.

1.2.1 Instructions

a. Two Component Self-leveling Polyurethane Sealant

Submit formulator's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include in the instructions detailed mixing and application procedures, quantity of material to be used per size of crack, total quantity of material to be used on job, minimum and maximum application temperatures, and curing procedures. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA \& 29 CFR 1926.59.

1.2.2 Field Test Reports

a. Tests and Inspections

1.2.3 Certificates

a. Two Component Self-leveling Polyurethane Sealant

Certify conformance to the requirements set forth in section 2.1.1.

1.2.4 Records

a. Installers Qualification.

Contractor shall have completed a minimum of five jobs within the past two years fabricating concrete cracks into expansion joints utilizing routing equipment, bondbreaker tape, and polyurethane sealants. The total area of linear feet repaired shall exceed 600 linear feet. Contractor shall submit documentation listing location of work, point of contact at job site, linear feet repaired, and type of urethane sealant used. The contractor shall have at least one qualified and experienced person (crack sealing with polyurethane) on the job site at all times during the crack sealing processes.

Throughout the progress of the work in this Specification, the Contractor will provide at least one (1) person who is thoroughly familiar with the specified requirements, completely trained and experienced with the necessary skills, and who will be present on the site and direct all work performed under this Specification.

In performing the work of this Specification, the Contractor will use an adequate number of skilled workmen to ensure the installation is in strict accordance with schedule, specification and the procedures recommended by the Naval Facilities Engineering Service Center (NFESC).

b. Disposal of Material.

All unused material, whether in its cured or uncured state, shall be remove from the job site by Contractor.

1.2.4 Batch Samples

When requested by the NFESC, the Contractor shall provide batch samples of any materials that are used throughout the progression of the work. Batch samples will not exceed 1% of the total material used on this job.

1.3 DELIVERY AND STORAGE

Ship urethanes and other materials in their original, sealed containers. Materials delivered to site shall be inspected for damage and container opening prior to use. Material delivered in dented, rusty, leaking, or previously opened containers and, in addition, material with an expired shelf life shall be returned to manufacturer. Material shall be unloaded and stored out of sun and weather, preferably in air conditioned spaces.

1.4 SAFETY

Ensure that employees are trained in the requirements of OSHA \&29 CFR 1926.59 and understand the information contained in the MSDS for their protection against toxic and hazardous chemical effects. Follow safety procedures as recommended by manufacturer. Procedures may include employing the use of impervious clothing, gloves, face shields, and other appropriate protective clothing necessary to prevent eye and skin contact with materials.

PART 2 PRODUCTS

2.1 TWO COMPONENT SELF-LEVELING POLYURETHANE SEALANT

A two component, self-leveling, high solids, polyurethane-based sealant which is chemically resistant to alkalis up to pH 13, organic acids, and most organic solvents. Sealant cures to a dense, flexible elastomer capable of resisting up to 25 percent crack movement. Sealant is thermosetting and cold applied.

2.1.1 Polyurethane Sealant

The polyurethane sealant shall be formulated to exhibit the following properties as listed in Table 1.

Table 1. Properties of Polyurethane Sealant

Solids (by volume)	≥95%
Elongation, % (ASTM D 638)	≥450%
Tensile Strength, psi (ASTM D 412)	≥175 psi
Hardness Shore A (ASTM D 2240)	25 - 50
Alkalis Resistance	Excellent
Organic Solvent Resistance	Good
Organic Acid Resistance	Good

PART 3 EXECUTION

3.1 CRACK PREPARATION

3.1.1 Locate Rebar and Conduit

All rebar and conduit located a minimum distance of three inches from each side of the crack shall be identified and mapped along the length of the crack. Mapping shall include depth of concrete cover and the exact location of rebar and/or conduit in relation to the crack.

3.1.2 Surface Cleaning

Crack shall be routed out to a 1:2 depth-to-width ratio with out disturbing or damaging existing rebar and conduit. All dirt, debris, efflorescence, chipped concrete, grease, oil, and other obtrusive material in each crack shall be removed both inside and a minimum of one half inch (1/2") in width on both sides of each crack to be repaired. Cleaning shall be accomplished by a combination of wire brushing, hand tool cleaning, power tool cleaning, compressed air, and aqueous based detergent cleaning. Cleaning utilizing organic solvents is prohibited.

3.1.3 Install Bondbreaker

Polyethylene bondbreaker tape shall be applied to the inner base of the routed out, cleaned, and dry crack. Tape shall be placed flush with inner walls of crack.

3.2 SEALANT APPLICATION

3.2.1 Mixing

Based on ambient temperature, relative humidity, and moisture content in concrete, consult sealant manufacturer and mix polyurethane sealant components in accordance to their recommendations.

3.2.2 Sealant Installation

Immediately following the day's high temperature and on dry concrete, pour into crack the polyurethane sealant over the polyethylene bondbreaker tape and finish by trowel. Resulting crack repair shall be flush with the surrounding concrete, exhibit complete crack depth penetration, and be free of surface irregularities, air voids, and discontinuities greater than 1/32 inch. An epoxy primer may be used to aid adhesion on either questionable concrete or concrete that contains excess moisture. Primer shall be chemically and mechanically compatible with polyurethane sealant.

3.2.3 Curing

Within forty-eight hours following application of sealant, sealant shall be tack free and ready for light traffic. If after forty-eight hours, the sealant is tacky or in any form of its uncured state, all uncured material shall be removed by Contractor. New sealant will be reapplied to the crack.

3.3 FINAL INSPECTION

Government shall inspect and verify repairs have been carry out in accordance to the guidelines set forth in sections 3.2.2 and 3.2.3. NFESC will take one, 1-inch diameter, core on each span to verify depth and quality of sealant penetration. The contractor will repair the cored site to the same level as required by this specification.

3.4 FINAL CLEANUP

Following completion of work, remove debris, equipment, and materials from the site. Remove temporary connections to Government furnished services. Restore existing facilities in and around the work areas to their original condition.

POLYURETHANE CRACK INJECTION

Note: This specification is intended to be applied to repairing concrete cracks up to one quarter inch (1/4") wide by the method of high pressure injection using polyurethane injection grout. This specification is designed to be used in repairing concrete cracks that either leak water or contribute to accelerated rebar corrosion.

PART 1 GENERAL

1.1 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to within the text by the basic designation only.

AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)

ASTM D 93 1994	Test Methods of Flash Point by Pensky- Martens Closed Tester
ASTM C 157 1993	Test Method for Length Change of Hardened Hydraulic-Cement, Mortar, and Concrete
ASTM C 273 1994	Test Method for Shear Properties of Sandwich Core Materials
ASTM D 638 1996	Test Method for Tensile Properties of Plastics
ASTM D 2196 1991	Test Method for Rheological Properties of Non-Newtonian Materials by Rotational (Brookfield) Viscometer
ASTM D 2939 1978	Test Methods for Emulsified Bitumens Used as Protective Coatings
ASTM D 3800 1979	Test Method for Density of High Modulus Fibers

CODE OF FEDERAL REGULATIONS (CFR)

29 CFR 1910.134	Respiratory Protection
29 CFR 1926.59	Hazard Communication
40 CFR 261	Identification and Listing of Hazardous Waste

1.2 SUBMITTALS

Submit the following documentation and materials to the Contracting Officer prior to receiving authorization to proceed with concrete crack injection.

1.2.1 Instructions

a. Water-Activated Polyurethane Injection Grout

Submit formulator's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include in the instructions detailed mixing and application procedures, quantity of material to be used per size of crack, total quantity of material to be used on job, minimum and maximum application temperatures, and curing procedures. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA \& 29 CFR 1926.59.

b. Mechanical Packers

Submit printed literature from manufacturer of mechanical packers to include brand name, catalog numbers, type, installation procedures, and recommended working pressures.

c. Cementitious Grout

Submit manufacturer's printed instructions to include brand name, catalog numbers, and type of cementitious grout. Include in the instructions detailed mixing and application procedures, total quantity of material to be used on job, minimum and maximum application temperatures, and curing procedures. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA \& 29 CFR 1926.59.

1.2.2 Field Test Reports

a. Tests and Inspections

1.2.3 Certificates

a. Water-Activated Polyurethane Injection Grout

For the polyurethane base resin, certify conformance to the requirements setforth in section 2.1.1. For the polyurethane accelerator, certify conformance to the requirements setforth in section 2.1.2.

b. Mechanical Packers

Certify mechanical packer's maximum working pressure exceeds 3500 psi.

c. Cementitious Grout

Certify conformance to the requirements set forth in section 2.3.

1.2.4 Records

a. Installers Qualification.

Contractor shall have completed a minimum of five jobs within the past two years pressure injecting Water-Activated Polyurethane Injection Grouts at a minimum pressure of 1000 psi into concrete cracks. The total area of linear feet repaired shall exceed 600 linear feet. Contractor shall submit documentation listing location of work, point of contact at job site, linear feet repaired, type of urethane system injected, amount of pressure used to inject system, and type of mechanical packer used.

Throughout the progress of the work in this Specification, the Contractor will provide at least one (1) person who is thoroughly familiar with the specified requirements, completely trained and experienced with the necessary skills, and who will be present on the site and direct all work performed under this Specification.

In performing the work of this Specification, the Contractor will use an adequate number of skilled workmen to ensure the installation is in strict accordance with schedule, specification and the procedures recommended by the Naval Facilities Engineering Service Center (NFESC).

b. Disposal of Material.

All unused material, whether in its cured or uncured state, shall be removed from the job site by Contractor.

1.2.5 Batch Samples

When requested by the NFESC, the Contractor shall provide batch samples of any materials that are used throughout the progression of the work. Batch samples will not exceed 1% of the total material used on this job.

1.3 DELIVERY AND STORAGE

Delivered to site shall be inspected for damage or damaged and opened containers prior to use. Material delivered in dented, rusty, or leaking containers and, in addition, material with an expired shelf life shall be returned to manufacturer. Material shall be unloaded and stored out of sun and weather, preferably in air-conditioned spaces.

1.4 SAFETY

Ensure that employees are trained in the requirements of OSHA \&29 CFR 1926.59 and understand the information contained in the MSDS 's for their protection against toxic and hazardous chemical effects. Follow safety procedures as recommended by manufacturer. Procedures may include employing the use of impervious clothing, gloves, face shields, and other appropriate protective clothing necessary to prevent eye and skin contact with materials.

PART 2 PRODUCTS

2.1 Water-Activated Polyurethane Injection Grout

A 100% solids, two component, hydrophobic grout primarily based on a combination of methylene-diphenyl-isocyanate polyurethane and polyol. The grout is designed to foam when the accelerator (a combination of tributylamine and polyurethane foam) has been added to the base resin and the resulting mixture contacts either moisture or water. The cured product is a closed cell rubber-like foam which remains flexible and is chemically resistant to a wide assortment of chemicals including organic solvents. This system will be used to fill concrete cracks up to 1/4 inch wide by the technique of high pressure injection using mechanical packers.

2.1.1 Polyurethane Base Resin

The polyurethane base resin shall be formulated to exhibit the following properties as listed in Table 1.

Table 1. Properties of Polyurethane Base Resin

Solids (ASTM D 2939)	100%

Density, g/ml (ASTM D 3800)	1.08 ± 0.02
Flash Point, C (ASTM D 93)	130 ± 5
Viscosity, cps (ASTM D 2196)	500 - 600
Expansion, %	1500 - 2500% (15 - 25 times)
Elongation, % (ASTM D 638)	>250
Tensile Strength, psi (ASTM D 638)	>70
Shear Strength, psi (ASTM C 273)	>15

2.1.2 Polyurethane Accelerator

The polyurethane accelerator shall be formulated to exhibit the following properties as listed in Table 2.

Table 2. Properties of Polyurethane Accelerator

Solids (ASTM D 2939)	100%
Density, g/ml (ASTM D 3800)	0.93 ± 0.02
Flash Point, C (ASTM D 93)	170 ± 5
Viscosity, cps (ASTM D 2196)	18 - 25

2.2 MECHANICAL PACKER

Injection port for use in the high pressure (1000 to 5000 psi) injection of polyurethane grouts into concrete cracks. The mechanical packer is made out of steel and contains a rubber expansion sleeve at the terminal end which is opposed by a zerk fitting. Packers are designed to snugly fit into a pre-drilled hole typically intersecting a crack at a 45 degree angle.

2.3 CEMENTITIOUS MORTAR

Prepacked cementitious repair material for use in the repair of concrete holes, voids, and defects. The mortar shall be free from all polymers, micro-silica and, in addition, shall not exceed 0.05% drying shrinkage at 28 days when tested in accordance to ASTM C 157. This material, when mixed and allowed to cure for 24 hours, shall attain a minimum compressive strength of 3,000 psi.

PART 3 EXECUTION

3.1 CRACK PREPARATION

3.1.1 Surface Cleaning

All dirt, debris, efflorescence, chipped concrete, and other obtrusive material in each crack shall be removed both inside and a minimum of one half inch (1/2") in width on both sides of each crack to be repaired. Cleaning shall be accomplished by a combination of wire brushing, hand tool cleaning, power tool cleaning, compressed air, and aqueous based detergent cleaning. Cleaning utilizing organic solvents is prohibited.

3.1.2 Locate Rebar and Conduit

All rebar and conduit will be located a minimum distance of three inches from each side of the crack along the entire crack's length shall be identified and mapped. Mapping shall include depth of concrete cover and the exact location of rebar and/or conduit in relation to crack.

3.1.3 Drill Injection Holes and Install Packers

Pattern location of injection holes based on rebar and conduit mapping. Injection holes shall not be drilled in such a manner as to intersect, touch, or damage existing rebar or conduit. Injection hole size shall be determined by the size of the selected mechanical packer. If a 1/2" mechanical packer is selected, then 1/2" injection holes shall be drilled. Injection holes shall be drilled approximately every six to twelve inches on alternating sides of the crack in such a manner as to intersect the crack at a 45 degree angle at a minimum of 2-inches below the surface. Place mechanical packers into the previously drilled holes with the rubber sleeve below the concrete surface and the zerk fitting above. Gently tap mechanical packer in place and secure by tightening with a wrench.

3.1.4 Deep Cleaning

Crack shall be cleaned with a solution of five to seven percent by volume muriatic acid. Acid solution shall be pumped through each mechanical packer along the entire length of the crack. It is recommended to disconnect zerk fitting not directly attached to the pressure line when pumping acid solution. This will create an exit for excess solution. Approximately ten minutes after muriatic acid injection, flush crack with potable water using the above procedure.

3.2 RESIN INJECTION

3.2.1 Mixing

Based on ambient temperature, relative humidity, and moisture content in concrete, consult manufacturer and mix polyurethane grout components in accordance to their recommendations. Standard mixing ratios vary between 50:1 (base resin: accelerator) to 10:1.

3.2.2 Crack Injection

Injection equipment and injection pressures shall be approved by polyurethane grout manufacturer prior to crack injection. Only compatible equipment and pressures recommended by polyurethane grout manufacturer shall be used. It is recommended to initiate polyurethane grout injection at point of highest resistance and work towards areas of lower resistance. Areas of high resistance are typically located at the narrowest portion of the crack. Zerk fittings not directly attached to the pressure line shall be disconnected to enable free port to port transfer of grout. Once port to port transfer has been attained, reconnect adjacent zerk fitting and continue pumping grout in this fashion until all mechanical packers have been injected with grout. If port to port transfer cannot be attained, additional injection ports may be required. Up to three grout injection per mechanical packer may be required to establish adequate penetration. Resulting crack repair shall be flush with the surrounding concrete, exhibit complete crack depth penetration, and be free of surface irregularities, air voids, and discontinuities greater than 1/32 inch. Concrete cores may be required to verify complete crack depth penetration

3.2.3 Curing

Within forty-eight hours following injection of grout, grout shall be fully cured to resemble a dense flexible foam. If after forty-eight hours, the grout is tacky or in any form of its uncured state, all uncured material shall be removed by Contractor and reapplied.

3.3 INJECTION HOLE REPAIR

3.3.1 Injection Hole Repair

Remove mechanical packers after sufficient time has been allowed for the grout to cure. Repair all injection holes using the cementitious mortar specified in section 2.3. Following manufacturer's recommendations for mixing, application, and curing of this material. Injection hole repair shall be flush with surrounding concrete and exhibit no shrinkage cracks greater than 1/64 inch after seven days.

3.4 FINAL INSPECTION

Government shall inspect and verify repairs have been carry out in accordance to the guidelines setforth in sections 3.2.2, 3.2.3, and 3.3.1.

NFESC will take one, 1-inch diameter, core on each span to verify depth and quality of sealant penetration. The contractor will repair the cored site to the same level as required by this specification.

3.5 FINAL CLEANUP

Following completion of work, remove debris, equipment, and materials from the site. Remove temporary connections to Government furnished water and electrical services. Restore existing facilities in and around the work areas to their original condition.

APPENDIX D:

Specifications for Upgrade Systems:

PULTRUDED CARBON/EPOXY COMPOSITE REINFORCING BAR SYSTEM

BONDED PULTRUDED UNIAXIAL CARBON/EPOXY LAMINATE STRIPS

HAND LAY-UP UNIAXIAL CARBON/EPOXY COMPOSITE LAMINATE

EPOXY BONDED, PULTRUDED FIBERGLASS REINFORCED POLYMER STRUCTURAL SECTIONS

E-FIBERGLASS COMPOSITE SHELL SYSTEM FOR CONFINEMENT STRENGTHENING OF REINFORCED CONCRETE PILES

FIBERGLASS COMPOSITE HANGERS FOR 14-INCH INSULATED STEAM LINE

PULTRUDED CARBON/EPOXY COMPOSITE REINFORCING BAR SYSTEM

This specification defines the material and procedural requirements for the preparation and installation of a pultruded carbon/epoxy composite reinforcing bar system for post strengthening of the reinforced concrete deck. The carbon fiber composite will be installed on the top surface of the deck across the transverse girders (pilecaps) in accordance with the Contract drawings.

1. GENERAL

1.1 Work to be Provided

The contractor shall complete the following work as shown on the contract drawings, details and as specified herein:

Provide access for confined spaces

Provide concrete surface preparation

Apply carbon/epoxy composite reinforcing bar system

Provide all required tests and inspections of the strengthened system

1.2 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to within the text by the basic designation only.

AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)

ASTM C 882	Standard Test Method for Bond Strength of Epoxy-Resin Systems Used with Concrete by Slant Shear
ASTM D 638	Standard Test Method for Tensile Properties of Plastics
ASTM D 695	Standard Test Method for Compressive Properties of Rigid Plastics
ASTM D 732	Standard Test Method for Shear Strength of Plastics by Punch Tool
ASTM D 790	Standard Test Method for Flexural Properties of Unreinforced and Reinforced Plastics and Electrical Insulating Materials
ASTM D 3171	Standard Test Method for Fiber Content of Resin-Matrix Composites by Matrix Digestion
ASTM D 3379	Standard Test Method for Tensile Strength and Young's Modulus for High-Modulus Single-Filament Materials
ASTM D 4258	Standard Practice for Surface Cleaning Concrete for Coating.
ASTM D 4541	Standard Test Method for Pull-Off Strength of Coatings Using Portable Adhesion Testers

AMERICAN CONCRETE INSTITUTE (ACI)

ACI 515.1R	A Guide to the Use of Waterproofing, Dampproofing, Protective, and Decorative Barrier Systems for Concrete.

NATIONAL ASSOCIATION OF CORROSION ENGINEERS (NACE)

NACE RP 0288	Inspection of Linings on Steel and Concrete

INTERNATIONAL CONCRETE REPAIR INSTITUTE

Guideline No. 03732	Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, and Polymer Overlays

STEEL STRUCTURES PAINTING COUNCIL (SSPC)

SSPC-PA Guide 3	A Guide to Safety in Paint Application.

1.3 SUBMITTALS

Submit the following in accordance with the section entitled "Submittal Procedures".

1.3.1 Instructions

a. Carbon/Epoxy Reinforcing Bars

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include with the instructions the quantity of material to be used per square foot and total quantity of material to be used on job. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

b. Epoxy Adhesive System

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include in the instructions detailed mixing and application procedures, quantity of material to be used per square foot, total quantity of material to be used on job, minimum and maximum application temperatures, and curing procedures. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

1.3.2 Certificates

a. Carbon/Epoxy Reinforcing Bars

For the fiber reinforcement, certify conformance to the requirements set forth in section 2.1.1. For the epoxy matrix material, certify conformance to the requirements set forth in section 2.1.2. For the composite bar, certify conformance to the requirements set forth in section 2.1.3.

b. Epoxy Adhesive System

For the primer/sealer certify conformance to the requirements set forth in section 2.2.1. For the Epoxy Adhesive, certify conformance to the requirements set forth in section 2.2.2. For UV protection certify conformance to the requirements set forth in section 2.2.3.

1.3.3 Records

a. Installers Qualifications

Throughout the progress of the work in this Specification, the Contractor will provide at least one (1) person who is thoroughly familiar with the specified requirements, completely trained and experienced with the necessary skills, and who will be present on the site and direct all work performed under this Specification.

In performing the work of this Specification, the Contractor will use an adequate number of skilled workmen to ensure the installation is in strict accordance with schedule, specifications and procedures recommended by the Naval Facilities Engineering Service Center (NFESC).

b. Disposal of Material

All polymer resins and adhesives shall be disposed of properly as indicated on MSDS's. All epoxy resins and adhesives shall be stored and transported as indicated on MSDS's. All materials (epoxies, concrete, grout, abrasive media, etc.) shall be contained at the site and prevented from entering the bay waters.

1.4 QUALITY ASSURANCE

Prior to commencement of any work, the Contractor shall be responsible for arranging and conducting a meeting between the CONTRACTING OFFICER, Contractor, and NFESC engineers to discuss the project requirements. The Contractor shall review the requirements of the Specification and overall project requirements. All aspects of the project including containment, environmental control, surface preparation, strengthening system application, quality assurance, schedule requirements, and safety shall be reviewed and discussed. The Contractor shall request clarification of any ambiguities, and advise the CONTRACTING OFFICER and NFESC of any potential conflicts and/or any technical requirements that appear improper or inappropriate.

The Contractor shall provide repair acceptance testing in accordance with Section 5 of this Specification.

The Contractor shall ensure the highest quality of workmanship at all times throughout the progression of the work. Only qualified applicators having prior experience in the specified surface preparation and epoxy coatings applications shall be assigned to perform the work described herein.

The Contractor shall provide full inspection of the surface preparation and composite systems applications to ensure the requirements of this Specification are fully complied with.

1.5 DELIVERY AND STORAGE

All materials shall be delivered in "new" condition only, packaged in their original, unopened containers bearing the manufacturer's name, product identification, batch number(s) and shelf life expiration date.

All materials shall be stored in a covered, well-ventilated area and protected from exposure to any detrimental conditions including: airborne contaminants dirt, dust, sunlight, extreme cold, heat, rainfall, sparks or flame.

When requested by the NFESC, the Contractor shall provide batch samples of any materials that are used throughout the progression of the work. The batch samples shall not exceed 1 percent of the total material of the job.

1.6 SAFETY

The processes described in this Specification involve potentially hazardous operations. The contractor shall take the necessary precautions in accordance with OSHA regulations, manufacturer's instructions, applicable material safety data sheets, and the Navy's site safety requirements to ensure the safety of all personnel that may be affected by the work described in this Specification.

The Contractor shall be responsible for providing any and all safety equipment required for confined spaces, the use of any products and/or equipment utilized during the progression of the work.

The Contractor shall establish and maintain safe working conditions throughout the progression of the work. The Contractor shall take immediate action to remedy any safety concerns and unsafe, or potentially unsafe working conditions.

The Contractor shall provide safe access to all work areas for inspection by the NFESC and/or their designated representatives.

2. PRODUCTS

2.1 CARBON/EPOXY REINFORCING BARS

The carbon/epoxy bars are the reinforcing elements for the negative moment regions over the transverse girders and the pile girders supporting the Pier 12 deck. The bars are pultruded carbon fiber reinforced epoxy.

2.1.1 High Tensile Carbon Fiber

Tensile Strength avg. (ksi), as per ASTM D 3379	500[1]
Tensile Strength min. (ksi), as per ASTM D 3379	450[1]
Tensile Modulus avg. (Mpsi), as per ASTM D 3379	33[1]
Tensile Modulus min. (Mpsi), as per ASTM D 3379	30[1]
Tensile Elongation avg. (percent), as per ASTM D 3379	1.5[1]
Tensile Elongation min. (percent), as per ASTM D 3379	1.1[1]

[1]at 72^{0}F and 50% relative humidity

2.1.2 Epoxy Matrix Material

Tensile Strength, avg. (psi), as per ASTM D 638	10,000[2]
Tensile Strength, min. (psi), as per ASTM D 638	9,000[2]
Tensile Modulus, avg. (ksi), as per ASTM D 638	450[2]
Tensile Modulus, min. (ksi), as per ASTM D 638	425[2]
Tensile Elongation, avg. (percent), as per ASTM D 638	5.0[2]
Tensile Elongation, min. (percent), as per ASTM D 638	3.5[2]

[2]Type 1 at 72^{0}F and at 50% relative humidity

2.1.3 Carbon Fiber Composite Bar

Percentage of fiber area to total area of bar, as per ASTM D 3171	60 minimum
Tensile Strength, Ultimate (ksi), as per ASTM D 3916	250[3] minimum

[3] Strength per unit of total area

Tensile Elongation, Ultimate (percent), as per ASTM D 3916	1.4

2.2 EPOXY ADHESIVE SYSTEM

The carbon bars are embedded in an epoxy resin in slots cut in the concrete surface as described in the project description (Part 2). The epoxy system shall include a primer/sealer for the concrete surface, the epoxy adhesive encasing the composite bar, and a UV protection layer. The system components must be chemically compatible so that individual properties are not compromised and so that solid bonding is developed. The system must be ultraviolet radiation (UV) resistant so that strengths are not degraded for ten years.

2.2.1 Epoxy Primer/Sealer

The epoxy primer/sealer seals the concrete surface while increasing its tensile strength. The primer/sealer is a 100% solids, two component epoxy with low viscosity and moisture tolerant. The Viscosity must be less than 100 cps to insure penetration into the concrete. The primer/sealer must not contain organic solvents.

Bond Strength (psi), as per ASTM C882 (hardened concrete to hardened concrete, 14 day moist cure)	2,700
Tensile Strength (psi), as per ASTM D 638	7,500

2.2.2 Epoxy Adhesive

The epoxy adhesive that encapsulates the carbon bar is a two part, 100 percent solids, high modulus, high-strength, moisture tolerant, highly filled epoxy. It must be chosen to be chemically and mechanically compatible with the carbon reinforcing bars and the concrete. The adhesive must develop a solid, permanent bond with the carbon composite and with the concrete.

Compressive Strength (ksi), as per ASTM D 695 @ 90^0F	8.2
Adhesive Strength to Concrete, as per ASTM D 4541	Exceed concrete strength and 300 psi
Tensile Modulus (ksi), as per ASTM D 638	320
Tensile Strength (ksi), as per ASTM D 638	5.1
Flexural Strength (ksi), as per ASTM D 790	7.4
Shear Strength (ksi), as per ASTM D 732	5.9
Adhesion Strength to Carbon Bar (ksi), as per ASTM C 882 Plastic Concrete to Hardened Concrete (14 day moisture cure)	2.4

2.2.3 UV Protection

The carbon reinforcing bar and the epoxy system must be protected from ultraviolet radiation (UV) degradation. A durable polymer layer must be added over the adhesive system with UV absorbers or stabilizers. It is preferable that additives be employed rather than resorting to a protective coating which can be damaged in the marine-industrial environment.

The polymer layer shall consist of a low modulus, medium viscosity, epoxy resin binder combined with sand to act as a UV inhibitor and as a abrasive protective coating for the top deck surface. The sand/epoxy layer should be applied over the epoxy encapsulate before the latter has polymerized.

Epoxy Resin Binder:

Compressive Strength (ksi), as per ASTM D 695 @ 90°F	6.2
Adhesive Strength to Concrete, as per ASTM D 4541	Exceed concrete strength and 300 psi
Tensile Modulus (ksi), as per ASTM D 638	540
Tensile Strength (ksi), as per ASTM D 638	1.1
Flexural Strength (ksi), as per ASTM D 790	2.6
Shear Strength (ksi), as per ASTM D 732	2.7
Adhesion Strength to Carbon Bar (ksi), as per ASTM C 882	2.2

Sand:

60 Grit oven dried silica sand free of organic impurities

Mix two (2) parts sand to one (1) part epoxy resin by volume

PART 3 EXECUTION

3.1 SURFACE PREPARATION

All concrete surfaces to be strengthened shall be thoroughly prepared to comply with the minimum requirement defined in this section.

Carbon bars can only be embedded after the concrete has been reconditioned and the cracks are filled. Examine carefully the concrete surface to be strengthened. Verify the areas of concrete surface and cracks have been properly repaired prior to preparation. Consult the CONTRACTING OFFICER AND NFESC for final advice and approval of additional areas to be repaired and methods of repair.

Surface preparation shall be performed to the extent to completely remove all laitance, loosely adhering concrete, and spalling.

Prior to initiating reinforcing procedures, the Contractor shall test the bond strength of the epoxy system by preparing a test patch to properly prepared concrete. The test patch area shall be prepared in accordance with the requirements of this Specification. After curing, the primer system's bond strength to concrete shall be determined in accordance to ASTM D 4541-93. Three pull-off tests will be performed where each pull-off will develop minimum bond strength to concrete fail the concrete substrate. All failures must be reported to the NFESC immediately.

The surface area to be reinforced will be primed with the two-part, penetrating epoxy sealer primer. After the primer has cured 24 hours, the reinforcing slots are laid out and slots are cut in the deck with a concrete saw or router. The diameter of the carbon bars and the number of bars per slot determine the depth and width of slots. Multiple bars or twisted strands can be embedded in each slot. The slots will allow at least one-sixteenth (1/16) of an inch between the bars and concrete, at least one-sixteenth of an inch between each bar in the slot, and at least three-eighths of an inch clear cover. Slots will be cut in the range of three-fourths of an inch deep and at one-half of an inch wide. The spacing of the slots will be four inches on center. Other slot sizes and slot spacing must be approved by the NFESC.

Perform abrasive blasting on concrete surfaces to be strengthened. Use materials and methods for project work as used to produce sample preparation acceptable to the CONTRACTING OFFICER AND NFESC. (See above paragraph).

Use abrasive of uniform size and angular shape so as to provide a uniformly etched profile.

Maintain control of concrete chips, dust and debris in each area of work. Clean up and remove such material at the completion of each day of blasting. All concrete chips, abrasive media, paint chips, dust, and other debris shall be contained and prevented from falling into the bay waters.

3.2 CARBON BAR INSTALLATION

3.2.1 General

The carbon bar reinforced system shall be installed in strict accordance with the Navy's written recommendation for procedures and equipment, in conjunction with the specific requirements defined herein.

It is possible that epoxy based materials used in the composite system may develop higher viscosity and/or slow curing and insufficient curing at low ambient temperature. Do not apply the specified system when substrate temperatures are lower than 50^0F (10^0C).

Presence of moisture may inhibit adhesion of the system to the concrete substrate. Provide necessary weather protection to protect surfaces from rain or cold.

Primer shall be mixed in accordance with the manufacturer's recommendations. Volume of primer to be prepared at one time must be such that it can be applied within its batch life. A fixed primer batch which has exceeded it batch life must not be used. The batch life may vary subject to ambient temperature or volume of the mixed primer batch and care must be taken accordingly. Do not mix any solvents with the epoxy primer.

Primer shall be applied by brush or roller. Alternatively, the primer may be spray applied with airless spray equipment, followed immediately by thorough backrolling to work the primer into the concrete surface. The primer shall be applied uniformly in sufficient quantity to fully penetrate the concrete and produce a non-porous film in the surface not to exceed one (1) dry mil in thickness after full penetration.

3.2.2 Installation of Carbon Reinforcing Bars

The slots are abrasively blasted to clean and remove all loose material. The slots are primed with two parts epoxy primer sealant and allowed to cure 24 hours.

The carbon bars are embedded in a two-part epoxy adhesive. Two parts epoxy may be combined with one part sand (60 grit) by volume. The slots are filled to within 1/8 to ¼ inch of the top surface with the epoxy/sand mixture and the bars are placed in the slots and pressed to the bottom of the slot. Before the epoxy/sand encapsulate mixture cures, the UV inhibitor layer can be applied directly to fill the slots completely. If the encapsulate is allowed to cure then the blush must be removed with abrasives and wiped clean with solvent prior to the application of the UV inhibitor. The UV inhibitor shall be placed so that there are no low spots, which would permit the pooling of water over the slots. The final layer shall be flush with the existing concrete surface.

The carbon bars shall be installed by hand. Bar splices and laps are not allowed. The process shall be executed to produce a uniform system that is completely free of voids and trapped air. Epoxy parts shall be mixed and applied in strict accordance with the manufacturer's recommendations. Do not mix any solvents with the epoxies.

Special care shall be taken to minimize the elapsed time between mixing and application of the epoxy encapsulate to ensure the carbon bars are installed at least fifteen minutes prior to any thickening or gelling. The work time limitations will vary according to temperature, and can be determined by information obtained from the manufacturer and practical experience with the product.

The installed reinforcing system shall be completely free of defects including air pockets, voids, and inclusions. After the installed system has been allowed to cure a minimum of 24 hours, the Contractor shall repair all defects as necessary to comply with the requirements of this Specification.

Maintain control of epoxy resin and carbon bar material in each area of work. Epoxy resin and carbon laminate material will be prevented from entering the bay waters. Clean up and remove unused and discarded excess material at the completion of each workday.

4. REWORK AND REPAIRS

Applied composite systems that are found to be defective or damaged will be replaced if deemed necessary by the NFESC to render the repairs in full compliance with the requirements of this Specification. Rework and repair procedures shall comply with all material and procedural requirements defined in this Specification. Defects shall be repaired in a manner that will restore the system to the designed level of quality. Repair procedures for conditions that are not specifically addressed in the Specification shall be approved by the NFESC on the case-by-case basis prior to initiating any repairs. All repairs shall be made to the satisfaction of the NFESC.

5. ACCEPTANCE TESTING

Soundings of the strengthened areas will be done to check for voids, bubbles and delaminations. All voids, bubbles, and delaminations shall be repaired by epoxy injection, replacement, or by some other means derived by the contractor and approved by the NFESC.

The direct pull-off test (ASTM D 4541-93 and ISO 4624) which measures the bond between the epoxy adhesive and existing concrete substrate in a composite system, shall be used for acceptance of the strengthening project. The NFESC will select a test area on each span for the epoxy adhesive. This area will be prepared in accordance with Part 3. A tension device will be attached to the concrete surface using the procedures outline in Part 3. During the test a tension device shall be loaded to failure. The device will record the force causing the failure, which, if divided by the core cross sectional area will result in tensile (bond) strength (psi). Upon failure of the core specimen, a visual examination of the failure plane location reveals whether the failure occurred at the bond line or within the substrate. Failure at the bond line at tensile stress below 300 psi will be unacceptable. The desired failure is within the concrete substrate.

One pull-off test shall be provided for every span. strengthened with the carbon bar system.

Bond strength test procedures shall be approved by the NFESC prior to testing.

Any deviation from this requirement will require a meeting among the Contracting Officer, the Contractor, and the NFESC to identify issues and remedies.

BONDED PULTRUDED UNIAXIAL CARBON/EPOXY LAMINATE STRIPS

This specification defines the material and procedural requirements for the preparation and installation of epoxy bonded carbon fiber reinforced polymer strips for post strengthening of the reinforced concrete deck of Pier 12, Naval Station, San Diego. The installation will be on the bottom surface of the deck on the spans between pile bents 6 through 14 and 67 through 73 in accordance with the Contract drawings.

1. GENERAL

1.1 WORK TO BE PROVIDED

The contractor shall complete the following work as shown on the Contract drawings, details and as specified herein:

Provide scaffolding, working platforms, and access for confined spaces

Provide concrete surface preparation

Apply epoxy bonded carbon fiber reinforced polymer strip system

Provide all required tests and inspections of the strengthened system

1.2 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to within the text by the basic designation only.

AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)

ASTM C 882	Standard Test Method for Bond Strength of Epoxy-Resin Systems Used with Concrete by Slant Shear
ASTM D 638	Standard Test Method for Tensile Properties of Plastics
ASTM D 695	Standard Test Method for Compressive Properties of Rigid Plastics
ASTM D3039	Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials
ASTM D 3171	Standard Test Method for Fiber Content of Resin-Matrix Composites by Matrix Digestion
ASTM D 3379	Standard Test Method for Tensile Strength and Young's Modulus for High-Modulus Single-Filament Materials
ASTM D 4258	Standard Practice for Surface Cleaning Concrete for Coating.
ASTM D 4541-93	Standard Test Method for Pull-Off Strength of

Coatings Using Portable Adhesion Testers

AMERICAN CONCRETE INSTITUTE (ACI)

ACI 515.1R	A Guide to the Use of Waterproofing, Dampproofing, Protective, and Decorative Barrier Systems for Concrete.

NATIONAL ASSOCIATION OF CORROSION ENGINEERS (NACE)

NACE RP 0288	Inspection of Linings on Steel and Concrete

INTERNATIONAL CONCRETE REPAIR INSTITUTE

Guideline No. 03732	Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, and Polymer Overlays

STEEL STRUCTURES PAINTING COUNCIL (SSPC)

SSPC-PA Guide 3	A Guide to Safety in Paint Application.

1.3 SUBMITTALS

Submit the following in accordance with the section entitled "Submittal Procedures".

1.3.1 Instructions

a. Carbon Fiber Reinforced Polymer (laminate) Strips

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include in the instructions the quantity of material to be used per square foot and the total quantity of material to be used on job. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

b. Epoxy Adhesive System

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include in the instructions detailed mixing and application procedures, quantity of material to be used per square foot, total quantity of material to be used on job, minimum and maximum application temperatures, and curing procedures. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

1.3.2 Certificates

a. Carbon Fiber Reinforced Polymer (laminate) Strips

For the fiber reinforcement, certify conformance to the requirements set forth in section 2.1.1. For the Epoxy Matrix, certify conformance to the requirements set forth in section 2.1.2. For the Carbon Fiber Reinforced Composite, certify conformance to the requirements set forth in section 2.1.3.

b. Epoxy Adhesive System

For the Epoxy Primer/Sealer, certify conformance to the requirements set forth in section 2.2.1. For the Epoxy Putty Surfacer/Void Filler, certify conformance to the requirements set forth in section 2.2.2. For the Epoxy Adhesive, certify conformance to the requirements set forth in section 2.2.3.

1.3.3 Records

a. Installers Qualifications

Throughout the progress of the work in this Specification, the Contractor will provide at least one (1) person who is thoroughly familiar with the specified requirements, completely trained and experienced with the necessary skills, and who will be present on the site and direct all work performed under this Specification.

In performing the work of this Specification, the Contractor will use an adequate number of skilled workmen to ensure the installation is in strict accordance with schedule, specification and as recommended by the NFESC.

b. Disposal of Material

All epoxy resins and adhesives shall be disposed of properly as indicated on MSDS's. All epoxy resins and adhesives shall be stored and transported as indicated on MSDS's. Spent abrasive media shall be contained and disposed of properly as required by local authorities. All materials (epoxies, concrete, grout, abrasive media, etc.) shall be contained at the site and prevented from entering the bay waters.

1.4 QUALITY ASSURANCE

Prior to commencement of any work, the Contractor shall be responsible for arranging and conducting a meeting between the CONTRACTING OFFICER, Contractor, and NFESC to discuss the project requirements. The Contractor shall review the requirements of the Specification and overall project requirements. All aspects of the project including containment, environmental control, surface preparation, strengthening system application, quality assurance, schedule requirements, and safety shall be reviewed and discussed. The Contractor shall request clarification of any ambiguities, and advise the CONTRACTING OFFICER AND NFESC of any potential conflicts and/or any technical requirements that appear improper or inappropriate.

The Contractor shall provide acceptance testing in accordance with Section 5 of this Specification.

The Contractor shall ensure the highest quality of workmanship at all times throughout the progression of the work. Only qualified applicators having prior experience in the specified surface preparation and epoxy coatings applications shall be assigned to perform the work described herein.

The Contractor shall provide full inspection of the surface preparation and composite systems applications to ensure the requirements of this Specification are fully complied with.

1.5 DELIVERY AND STORAGE

All materials shall be delivered in "new" condition only, packaged in their original, unopened containers bearing the manufacturer's name, product identification, batch number(s) and shelf life expiration date.

All materials shall be stored in a covered, well-ventilated area and protected from exposure to any detrimental conditions including: airborne contaminants dirt, dust, sunlight, extreme cold, heat, rainfall, sparks or flame.

When requested by the NFESC, the Contractor shall provide batch samples of any materials that are used throughout the progression of the work. Batch samples will not exceed 1% of the total material used on this job.

1.6 SAFETY

The processes described in this Specification involve potentially hazardous operations. The contractor shall take the necessary precautions in accordance with OSHA regulations, manufacturer's instructions, applicable material safety data sheets, and the Navy's site safety requirements to ensure the safety of all personnel that may be affected by the work described in this Specification.

The Contractor shall be responsible for providing any and all safety equipment required for confined spaces, the use of any products and/or equipment utilized during the progression of the work.

The Contractor shall establish and maintain safe working conditions throughout the progression of the work. The Contractor shall take immediate action to remedy any safety concerns and unsafe, or potentially unsafe working conditions.

The Contractor shall provide safe access to all work areas for inspection by the NFESC and/or their designated representatives.

The Contractor shall conform with the guidelines set forth by SSPC-PA Guide 3, "A Guide to Safety in Paint Application."

2. PRODUCTS

The bonded carbon laminate strip reinforcing system consists of pultruded uniaxial carbon fiber strips, epoxy primer/sealant, epoxy putty surfacer/void filler, and epoxy adhesive. These components must be chemically and mechanically compatible and develop sufficient bonding so that individual component strengths and properties are not compromised. The system must be weather and salt resistant so that properties are not degraded for 10 years.

2.1 CARBON FIBER REINFORCED POLYMER (LAMINATE) STRIPS

The uniaxial carbon strips will externally reinforce the concrete deck slab. The strip is pultruded high tensile carbon fiber in an epoxy matrix.

2.1.1 Fiber Reinforcement

High Tensile Carbon Fiber

Tensile Strength avg. (ksi), as per ASTM D 3379	500[1]
Tensile Strength min. (ksi), as per ASTM D 3379	450[1]
Tensile Modulus avg. (Mpsi), as per ASTM D 3379	33[1]
Tensile Modulus min. (Mpsi), as per ASTM D 3379	30[1]
Tensile Elongation avg. (percent), as per ASTM D 3379	1.5[1]
Tensile Elongation min. (percent), as per ASTM D 3379	1.1[1]

[1]at 72^{0}F and 50% relative humidity

2.1.2 Epoxy Matrix

Tensile Strength, avg. (psi), as per ASTM D 638	10,000[3]
Tensile Strength, min. (psi), as per ASTM D 638	9,000[3]

Tensile Modulus, avg. (ksi), as per ASTM D 638	450[3]
Tensile Modulus, min. (ksi), as per ASTM D 638	425[3]
Tensile Elongation, avg. (percent), as per ASTM D 638	5.0[3]
Tensile Elongation, min. (percent), as per ASTM D 638	3.5[3]

[3]Type 1 at 72^0F and at 50% relative humidity

2.1.3 Carbon Fiber Reinforced Composite

Volumetric Content of Fibers (percent), as per ASTM D 3171	60
Laminate Tensile Strength, Ultimate (ksi), as per ASTM D 3039	300[2]

[2] Strength per unit area of the strip

Tensile Modulus (ksi), as per ASTM D 3039	22,000
Tensile Elongation, Ultimate (percent), as per ASTM D 3039	1.4
Total Carbon Fiber Design Area (in^2/in width)	0.033

2.2. EPOXY ADHESIVE SYSTEM

The epoxy system shall include: a primer/sealer, a polymer concrete repair and void filler, and a strip adhesive.

2.2.1 Epoxy Primer/Sealer

The epoxy primer/sealer seals the concrete surface while increasing its tensile strength. The primer/sealer is a 100% solids, two component epoxy with low viscosity and moisture tolerant. The Viscosity must be less than 100 cps to insure penetration into the concrete. The primer/sealer must not contain organic solvents.

Bond Strength (psi), as per ASTM C882 (hardened concrete to hardened concrete, 14 day moist cure)	2,700
Tensile Strength (psi), as per ASTM D 638	7,500

2.2.2 Epoxy Putty Surfacer/Void Filler

The epoxy putty surfacer and void filler provides a void-free concrete surface for application of the carbon/epoxy strips. The surfacer/void filler should be a high-modulus, high-strength 100% solids thixotropic adhesive epoxy paste.

Compressive Strength (ksi), as per ASTM D 695	8.6
Adhesive Strength to Concrete, as per ASTM D 4541	exceed concrete strength and 300 psi
Tensile Modulus (ksi), as per ASTM D 638	650
Tensile strength (ksi), as per ASTM D 638	3.6

Shear strength (ksi), as per ASTM D 732 3.6

2.2.3 Adhesive

The epoxy adhesive bonds the unidirectional carbon fiber strips to the concrete. The adhesive shall be a highly filled, 100% solids, high viscosity, two component epoxy that will develop the strength of the carbon strip without separating from the concrete deck.

Two part, highly filled epoxy paste.

Compressive Strength (ksi), as per ASTM D 695	8.6
Adhesive Strength to Concrete, as per ASTM D 4541	exceed concrete strength and 300 psi
Tensile Modulus (ksi), as per ASTM D 638	650
Tensile Strength (ksi), as per ASTM D 638	3.6
Flexural Strength (ksi), as per ASTM D 790	6.8
Shear Strength (ksi), as per ASTM D 732	3.6

3. EXECUTION

3.1 SURFACE PREPARATION

All concrete surfaces to be strengthened shall be thoroughly prepared to comply with the minimum requirement defined in this section.

Examine carefully the concrete surface to be strengthened. Verify the areas of concrete surface and cracks have been properly repaired prior to preparation. Consult the Contracting Officer and NFESC for final advice and approval of additional areas to be repaired and methods of repair.

Surface preparation of the base concrete shall be performed to the extent to completely remove all weathered concrete, laitance, loosely adhering concrete, and spalling. All bugholes and subsurface voids shall be opened. Concrete surface irregularities, fins, and/or sharp angles may result in separation and delamination of carbon laminate from the concrete. Concrete surface irregularities must be removed and smoothed to less than 40 mils. Concrete angles must be rounded to no less than a radius of 3/8-inch. Methods that bruise the concrete shall not be allowed.

Difference of adjacent concrete surface levels must be no greater than 40 mils when prepared for laminate application. Concrete surface protrusions must be flattened. Small depressions of concrete surfaces must be filled with epoxy putty or mortar.

Presence of moisture may inhibit adhesion of primer and resin. Wet surfaces must be thoroughly dried before applying primer and resin. Do not apply primer or resin when rainfall or dew is present. Do not apply resin or primer when the humidity is greater than 85 percent. Do not expose primer, resin, or epoxy putty to rainfall, dew, or humidity in excess of 85 percent.

Prior to initiating surface preparation procedures, the Contractor shall first prepare a representative sample area. The sample area shall be prepared in accordance with the requirements of this Specification, and shall be used as a mutually agreed upon reference standard depicting a satisfactorily prepared surface.

Perform abrasive blasting on concrete surfaces to be strengthened. Use materials and methods for project work as used to produce sample preparation acceptable to the Contracting Officer and NFESC. (See above paragraph).

Face shields, goggles and gloves will be worn by workers removing concrete.

Provide a uniformly etched profile. The concrete surface prior to the application of the epoxy primer is equivalent to CSP 3 as defined by the International Concrete Repair Institute.

Maintain control of abrasive media, concrete chips, dust, and debris in each area of work. Clean up and remove such material at the completion of each day of blasting. All concrete chips, abrasive media, paint chips, dust, and other debris shall be contained and prevented from falling into the bay waters. Any material that falls into the bay must be removed by the contractor.

3.2 CARBON FIBER STRIP INSTALLATION

3.2.1 General

Carbon fiber reinforced polymer strips shall be installed in strict accordance with the manufacturer's written recommendation for procedures and equipment, in conjunction with the specific requirements defined herein. In the event of the conflict between the requirements of the Specification and the manufacturer's recommendations, this Specification shall take precedence.

It is possible that epoxy based materials used in the composite system may develop higher viscosity and/or slow curing and insufficient curing at low ambient temperature. Do not apply the specified system when substrate temperatures are lower than 50°F (10°C).

Presence of moisture may inhibit adhesion of the system to the concrete substrate. Provide necessary weather protection to protect surfaces from rain or cold.

All surfaces to receive the carbon fiber strips shall be primed with the specified penetrating primer prior to application of any subsequent coatings.

Primer shall be mixed in accordance with the manufacturer's recommendations. Volume of primer to be prepared at one time must be such that it can be applied within its batch life. A fixed primer batch which has exceeded it batch life must not be used. The batch life may vary subject to ambient temperature or volume of the mixed primer batch and care must be taken accordingly. Uncured primer and primer components shall not be exposed to moisture. The priming procedure will not commence during rainfall or in the case of predicted rainfall. Do not mix any solvents with the epoxy primer.

Primer shall be applied by brush or roller. Alternatively, the primer may be spray applied with airless spray equipment, followed immediately by thorough backrolling to work the primer into the concrete surface. The primer shall be applied uniformly in sufficient quantity to fully penetrate the concrete and produce a non-porous film in the surface not to exceed one (1) dry mil in thickness after full penetration. If necessary, a second coat shall be applied after the first coat has completely penetrated into the concrete. Volume of primer to be applied may vary depending on the coarseness of the concrete surface, but a minimum of 0.46 lb/yd^2 and a maximum of 0.74 lb/yd^2 shall be applied.

Maintain control of epoxy resins in each area of work. Primer resins will be prevented from entering the bay waters. Clean up and remove such material at the completion of each workday.

3.2.2 Application of Void Filler

Void filler mortar shall be applied to surfaces that have been primed with the specified penetrating primer. All bugholes and subsurface voids shall be filled.

Void filler shall be trowelled tight against the surface with opposing passes. Void filler shall be applied in strict accordance with the manufacturer's recommendations. Do not mix any solvents with the epoxy void filler.

3.2.3 Application of Carbon Reinforced Laminate Strips

The laminate bonding surface is thoroughly cleaned until all epoxy dust and other loose material is removed.

After the void filler mortar has cured the concrete surface is abrasive blasted to roughen the primed concrete surface where the strips will be applied. The roughened surface will be equivalent to CSP3 roughness. The surface must be vacuumed to remove all dust and debris prior to applying the carbon strips.

The highly filled epoxy adhesive is applied to the surface for bonding the reinforcing strips. The adhesive is applied "∩"-shaped across the strip width so that the center peak height is 1/20 of the width of the laminate strip. The extra adhesive is squeezed out when the laminate is pressed to the concrete surface.

The laminates must be pressed to the to the concrete with a pressure of 10 psi until the adhesive cures. Pressure may be applied using vacuum bags or by mechanical means.

The strips shall then be completely completed bonded to the concrete surface without bubbles, delaminations or voids.

The process shall be carefully planned to allow sufficient working time for placing the carbon laminate strip to produce a uniform system that is completely free of voids and trapped air. The adhesive layer shall be applied in strict accordance with the manufacturer's recommendations. Do not mix any solvents with the epoxy adhesives.

Strip overlapping is not allowed.

Special care shall be taken to minimize the elapsed time between mixing and application of the adhesive to ensure the material is applied at least fifteen minutes prior to any gelling. The work time limitations will vary according to temperature, and can be determined by information obtained from the manufacturer and practical experience with the product.

The installed carbon laminate strip system shall be completely free of defects including air pockets, voids, sags, and inclusions. After the installed system has been allowed to cure a minimum of 24 hours, the Contractor shall repair all defects as necessary to comply with the requirements of this Specification.

Maintain control of epoxy resins, putty and laminate material in each area of work. Epoxy resin, putty, and laminate will be prevented from entering the bay waters. Clean up and remove unused and discarded excess material at the completion of each workday.

4. REWORK AND REPAIRS

Applied composite laminate strips that are found to be defective or damaged will be replaced if deemed necessary by the NFESC to render the repairs in full compliance with the requirements of this Specification. Rework and repair procedures shall comply with all material and procedural requirements defined in this Specification. Defects shall be repaired in a manner that will restore the system to the designed level of quality. Repair procedures for conditions that are not specifically addressed in the Specification shall be approved by the NFESC on the case-by-case basis prior to initiating any repairs. All repairs shall be made to the satisfaction of the NFESC.

5. ACCEPTANCE TESTING

Soundings of the strengthened areas will be done to check for voids, bubbles and delaminations. All voids, bubbles and delaminations shall be repaired by epoxy injection or replacement.

The direct pull-off test (ASTM D 4541-93) which measures the bond between the laminate and existing concrete substrate in a composite system, shall be used for acceptance of the strengthening project. A core shall be drilled through the carbon laminate strip ¼ - ½in. into the concrete substrate, providing an isolated test location for attachment of the pull-off tester. During the test a tension device shall be loaded to failure. The device will record the force causing the failure, which, if divided by the core cross sectional area will result in tensile strength (psi). Upon failure of the core specimen, a visual examination of the failure plane location reveals whether the failure occurred at the bond line or within the substrate. Failure of the concrete is the only acceptable failure.

One pull-off test shall be provided for every 200 S.F. of area strengthened with the carbon fiber system or once for every deck span.

Bond strength test procedures shall be approved by the NFESC prior to testing.

Any deviation from this requirement will require a meeting between Contractor and NFESC to identify issues and remedies.

The test areas of the composite system shall be repaired to the satisfaction of the NFESC in accordance with Part 4.

HAND LAY-UP UNIAXIAL CARBON/EPOXY COMPOSITE LAMINATE

This specification defines the material and procedural requirements for the preparation and installation of an epoxy bonded carbon fiber composite system for post strengthening of the reinforced concrete deck of Pier 12, Naval Station, San Diego. The carbon fiber composite will be applied to the bottom surface of the spans between pile bents 6 through 14 and 67 through 73 in accordance with the Contract drawings. This specification applies to hand lay up systems and in situ (wet) impregnation systems.

1. GENERAL

1.1 WORK TO BE PROVIDED

The contractor shall complete the following work as shown on the contract drawings, details and as specified herein:

Provide scaffolding, working platforms, and access beneath Pier 12.

Provide concrete surface preparation

Apply epoxy bonded carbon fiber overlay system

Provide all required tests and inspections of the strengthened system

1.2 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to within the text by the basic designation only.

AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)

ASTM C 882	Standard Test Method for Bond Strength of Epoxy-Resin Systems Used with Concrete by Slant Shear
ASTM D 638	Standard Test Method for Tensile Properties of Plastics
ASTM D 695	Standard Test Method for Compressive Properties of Rigid Plastics
ASTM D3039	Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials
ASTM D 3171	Standard Test Method for Fiber Content of Resin-Matrix Composites by Matrix Digestion
ASTM D 3379	Standard Test Method for Tensile Strength and Young's Modulus for High-Modulus Single-Filament Materials
ASTM D 4258	Standard Practice for Surface Cleaning Concrete for Coating.
ASTM D 4541	Standard Test Method for Pull-Off Strength of

Coatings Using Portable Adhesion Testers

AMERICAN CONCRETE INSTITUTE (ACI)

ACI 515.1R	A Guide to the Use of Waterproofing, Dampproofing, Protective, and Decorative Barrier Systems for Concrete.

NATIONAL ASSOCIATION OF CORROSION ENGINEERS (NACE)

NACE RP 0288	Inspection of Linings on Steel and Concrete

INTERNATIONAL CONCRETE REPAIR INSTITUTE

Guideline No. 03732	Selecting and Specifying Concrete Surface Preparation for Sealers, Coatings, and Polymer Overlays

STEEL STRUCTURES PAINTING COUNCIL (SSPC)

SSPC-PA Guide 3	A Guide to Safety in Paint Application.

1.3 SUBMITTALS

Submit the following in accordance with the section entitled "Submittal Procedures".

1.3.1 Instructions

a. Uniaxial Carbon Fiber Sheet

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include with instructions the quantity of material to be used on the job. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

b. Epoxy System

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include with instructions the total quantity of material to be used on the job. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

1.3.2 Certificates

a. Uniaxial Carbon Fiber Sheet

For the fiber reinforcement, certify conformance to the requirements set forth in section 2.1.

b. Epoxy Adhesive System

For the Epoxy Primer/Sealer certify conformance to the requirements set forth in section 2.2. For the Epoxy Putty Surfacer/Void Filler, certify conformance to the requirements set forth in section 2.3. For the Epoxy Saturant, certify conformance to the requirements set forth in section 2.4.

1.3.3 Records

a. Installers Qualifications

Throughout the progress of the work in this Specification, the Contractor will provide at least one (1) person who is thoroughly familiar with the specified requirements, completely trained and experienced with the necessary skills, and who will be present on the site and direct all work performed under this Specification.

In performing the work of this Specification, the Contractor will use an adequate number of skilled workmen to ensure the installation is in strict accordance with schedule, specification and the procedures recommended by the NFESC.

b. Disposal of Material

All epoxy resins and adhesives shall be disposed of properly as indicated on MSDS's. All epoxy resins and adhesives shall be stored and transported as indicated on MSDS's. Spent shotblasting media shall be contained and disposed of properly as required by local authorities. All materials (epoxies, concrete, grout, abrasive media, etc.) shall be contained at the site and prevented from entering the bay waters.

1.4 QUALITY ASSURANCE

Prior to commencement of any work, the Contractor shall be responsible for arranging and conducting a meeting between the Contracting Officer, Contractor, and NFESC engineers to discuss the project requirements. The Contractor shall review the requirements of the Specification and overall project requirements. All aspects of the project including containment, environmental control, surface preparation, strengthening system application, quality assurance, schedule requirements, and safety shall be reviewed and discussed. The Contractor shall request clarification of any ambiguities, and advise the Contracting Officer and NFESC of any potential conflicts and/or any technical requirements that appear improper or inappropriate.

The Contractor shall provide repair acceptance testing in accordance with Section 5 of this Specification.

The Contractor shall ensure the highest quality of workmanship at all times throughout the progression of the work. Only qualified applicators having prior experience in the specified surface preparation and epoxy coatings applications shall be assigned to perform the work described herein.

The Contractor shall provide full inspection of the surface preparation and composite systems applications to ensure the requirements of this Specification are fully complied with.

1.5 DELIVERY AND STORAGE

All materials shall be delivered in "new" condition only, packaged in their original, unopened containers bearing the manufacturer's name, product identification, batch number(s) and shelf life expiration date.

All materials shall be stored in a covered, well-ventilated area and protected from exposure to any detrimental conditions including: airborne contaminants dirt, dust, sunlight, extreme cold, heat, rainfall, sparks or flame.

When requested by the NFESC, the Contractor shall provide batch samples of any materials that are used throughout the progression of the work. Batch samples will not exceed 1% of the total material used on this job.

1.6 SAFETY

The processes described in this Specification involve potentially hazardous operations. The contractor shall take the necessary precautions in accordance with OSHA regulations, manufacturer's instructions, applicable material safety data sheets, and the Navy's site safety requirements to ensure the safety of all personnel that may be affected by the work described in this Specification.

The Contractor shall be responsible for providing any and all safety equipment required for confined spaces, the use of any products and/or equipment utilized during the progression of the work.

The Contractor shall establish and maintain safe working conditions throughout the progression of the work. The Contractor shall take immediate action to remedy any safety concerns and unsafe, or potentially unsafe working conditions.

The Contractor shall provide safe access to all work areas for inspection by the NFESC and/or their designated representatives.

The Contractor shall conform with the guidelines set forth by the SSPC-PA Guide 3, "A Guide to Safety in Paint Application."

2. PRODUCTS

The epoxy bonded carbon fiber overlay reinforcing system consists of uniaxial carbon fiber sheets, epoxy matrix, epoxy primer/sealant, and epoxy putty surfacer/void filler. These components must be chemically and mechanically compatible and develop sufficient bonding so that individual component strengths and properties are not compromised. The system must be ultraviolet (UV), weather, and salt resistant so that properties are not degraded for 10 years. It is preferable that UV stabilizers or absorbers are added to the epoxy matrix rather than relying on a protective coating.

2.1 Uniaxial Carbon Fiber Sheet

Uniaxial carbon fibers will provide the strength of the laminate reinforcing. The carbon fibers will be high tensile carbon fiber.

Tensile Strength avg. (ksi), as per ASTM D 3379	500[1]
Tensile Strength min. (ksi), as per ASTM D 3379	450[1]
Tensile Strength of Tow Sheet (kips per inch width)	3.3[1]
Tensile Modulus avg. (Mpsi), as per ASTM D 3379	33[1]
Tensile Modulus min. (Mpsi), as per ASTM D 3379	30[1]
Tensile Elongation, avg. (percent), as per ASTM D 3039	1.5[1]
Tensile Elongation, min. (percent), as per ASTM D 3039	1.1[1]
Total Carbon Fiber Design Area (in^2/in width)	0.033

2.2 Epoxy Primer/Sealer

The epoxy primer/sealer seals the concrete surface while increasing its tensile strength. The primer/sealer is a 100% solids, two component epoxy with low viscosity and moisture tolerant. The Viscosity must be less than 100 cps to insure penetration into the concrete. The primer/sealer must not contain organic solvents.

Bond Strength (psi), as per ASTM C882 (hardened concrete to hardened concrete, 14 day moist cure)	2,700
Tensile Strength (psi), as per ASTM D 638	7,500

2.3 Epoxy Putty Surfacer/Void Filler

The epoxy putty surfacer and void filler provides a void-free concrete surface for application of the carbon laminate. The surfacer/void filler should be a 100% solids thixotropic epoxy.

Compressive Strength (ksi), as per ASTM D 695	10
Adhesive Strength to Concrete, as per ASTM D 4541	exceed concrete strength and 300 psi
Tensile Strength (ksi) as per ASTM D 638	3.6
Tensile Modulus (ksi), as per ASTM D 638	650
Shear Strength (ksi) as per ASTM D 732	3.6

2.4 Epoxy Saturant

The epoxy saturant is the matrix that completely coats, envelops and binds the unidirectional carbon fiber sheets to the concrete. The saturant shall be a 100% solids, two component epoxy.

Compressive Strength (ksi), as per ASTM D 695	8.6
Adhesive Strength to Concrete, as per ASTM D 4541	exceed concrete strength and 300 psi
Tensile Modulus (ksi), as per ASTM D 638	650
Tensile Strength (ksi), as per ASTM D 638	3.6
Flexural Strength (ksi), as per ASTM D 790	6.8
Shear Strength (ksi), as per ASTM D 732	3.6

3. EXECUTION

3.1 CONCRETE REPAIRS

Restoration of the concrete cross section is addressed in "Specifications for Concrete Repairs" Restoration includes chipping and removal of damaged concrete, replacing steel reinforcing when necessary, replacement concrete, and repairing cracks.

3.2 SURFACE PREPARATION

All concrete surfaces to be strengthened shall be thoroughly prepared to comply with the minimum requirement defined in this section.

Examine carefully the concrete surface to be strengthened. Verify the areas of concrete surface and cracks have been properly repaired prior to preparation. Consult the Contracting Officer and NFESC for final advice and approval of additional areas to be repaired and methods of repair.

Surface preparation of the base concrete shall be performed to the extent to completely remove all weathered concrete, laitance, loosely adhering concrete, and spalling. All bugholes and subsurface voids shall be opened. Concrete surface irregularities, fins, and/or sharp angles may result in separation and delamination of carbon laminate from the concrete. Concrete surface irregularities must be removed and smoothed to less than 40 mils.

Concrete angles must be rounded to no less than a radius of 3/8-inch. Methods that bruise the concrete shall not be allowed.

Difference of adjacent concrete surface levels must be no greater than 40 mils when prepared for laminate application. Concrete surface protrusions must be flattened. Small depressions of concrete surfaces must be filled with epoxy putty or mortar.

Presence of moisture may inhibit adhesion of primer and resin. Wet surfaces must be thoroughly dried before applying primer and resin. Do not apply primer or resin when rainfall or dew is present. Do not apply resin or primer when the humidity is greater than 85 percent. Do not expose primer, resin, or epoxy putty to rainfall, dew, or humidity in excess of 85 percent.

Prior to initiating surface preparation procedures, the Contractor shall first prepare a representative sample area. The sample area shall be prepared in accordance with the requirements of this Specification, and shall be used as a mutually agreed upon reference standard depicting a satisfactorily prepared surface.

Perform abrasive blasting on concrete surfaces to be strengthened. Use materials and methods for project work as used to produce sample preparation acceptable to the Contracting Officer and NFESC. (See above paragraph).

Face shields, goggles and gloves will be worn by workers removing concrete.

Provide a uniformly etched profile. The concrete surface prior to the application of the epoxy primer is equivalent to CSP 3 as defined by the International Concrete Repair Institute.

Maintain control of abrasive media, concrete chips, dust, and debris in each area of work. Clean up and remove such material at the completion of each day of blasting. All concrete chips, abrasive media, paint chips, dust, and other debris shall be contained and prevented from falling into the bay waters. Any material that falls into the bay must be removed by the contractor.

3.3 CARBON FIBER SYSTEM INSTALLATION

3.3.1General

Carbon fiber system shall be installed in strict accordance with the manufacturer's written recommendation for procedures and equipment, in conjunction with the specific requirements defined herein. In the event of the conflict between the requirements of the Specification and the manufacturer's recommendations, this Specification shall take precedence.

It is possible that epoxy based materials used in the composite system may develop higher viscosity and/or slow curing and insufficient curing at low ambient temperature. Do not apply the specified system when substrate temperatures are lower than 50°F (10°C).

Presence of moisture may inhibit adhesion of the system to the concrete substrate. Provide necessary weather protection to protect surfaces from rain or cold. Installation in rain is strictly prohibited.

All surfaces to receive the carbon fiber fabric shall be primed with the specified penetrating primer prior to application of any subsequent coatings.

Primer shall be mixed in accordance with the manufacturer's recommendations. Do not mix any solvents with the epoxy primer. Volume of primer to be prepared at one time must be such that it can be applied within its batch life. A fixed primer batch which has exceeded it batch life must not be used. The batch life may vary subject to ambient temperature or volume of the mixed primer batch and care must be taken accordingly. No primer coat shall be applied if ambient temperature is lower than 50°F.

Primer must be thoroughly mixed with hardener at the manufacturer's specified ratio in a mixing pot until it is uniform. At least two minutes of agitation is required by means of an electric hand mixer.

Primer shall be applied by brush or roller. Alternatively, the primer may be spray applied with airless spray equipment, followed immediately by thorough backrolling to work the primer into the concrete surface. The primer shall be applied uniformly in sufficient quantity to fully penetrate the concrete and produce a non-porous film in the surface not to exceed one (1) dry mil in thickness after full penetration. If necessary, a second coat shall be applied after the first coat has penetrated into the concrete. Volume of primer to be applied may vary depending on the coarseness of the concrete surface but a minimum of 0.46 lbs/yd^2 and a maximum of 0.74 lbs/yd^2 shall be applied.

Applied primer coat must be cured overnight or until it is tack free before subsequent applications can be made.

Surface irregularities caused by primer coating must be ground smooth using a disc grinder. Minor surface defects that remain may be corrected using epoxy filler/surfacer.

Maintain control of primer resins in each area of work. Epoxy resin will be prevented from entering the bay waters. Clean up and remove unused and discarded excess material at the completion of each workday.

3.3.2 Application of Filler/Surfacer

Filler/surfacers shall be applied to surfaces that have been primed with the specified penetrating primer. All bugholes and subsurface voids shall be filled.

Filler/surfacer shall be troweled tight against the surface with opposing passes, in both to and from and left and right directions (four (4) way method). Filler/surfacer shall be applied in strict accordance with the manufacturer's recommendations. Do not mix any solvents with the epoxy surfacer/void filler. Although not required, it is desirable that the epoxy surfacer cover the entire concrete surface in the amount of 0.5 lbs/yd^2 (16 ft^2/gal).

Maintain control of epoxy putty filler material in each area of work. Epoxy putty will be prevented from entering the bay waters. Clean up and remove unused and discarded excess material at the completion of each workday.

3.3.3. Application of Carbon Fiber Sheet

Carbon fiber sheets will not be applied when the temperature is less than 50°F or if surface moisture is present or anticipated.

The carbon fiber sheets must be cut beforehand into prescribed sizes using scissors or cutter. The number of sheets cut shall be limited to the number to be installed that day.

The primer and filler/surfacer must be thoroughly cured before applying the carbon sheets. The primer surface must be roughened to an equivalent to a CSP 3 finish if it was applying more than a week in advance.

A saturant coating shall be roller applied to concrete surface or to the carbon sheet. The carbon fiber sheet shall then be installed by hand lay-up method. The sheet will be properly aligned and set into surface saturant. The sheet shall then be completely saturated with a second application of a roller-applied saturant. The process shall be carefully planned to allow sufficient working time for the rolling of the carbon fiber sheet and saturant to produce a uniform system that is completely free of voids and trapped air and to be completed within the time limits of the saturant pot life. Saturant coat shall be applied in strict accordance with the manufacturer's recommendations.

Special care shall be taken to minimize the elapsed time between mixing and application of the saturant to ensure the material is applied to the sheet at least fifteen minutes prior to any thickening or gelling. The work time limitations will vary according to temperature, and can be determined by information obtained from the manufacturer and practical experience with the product. Do not mix any of the solvents with the epoxy saturant.

The laminate is hand laid and cured without the aid of external heating or vacuum bags. The first layer can be applied directly to the uncured putty after coating the fiber sheet or the concrete surface with epoxy saturate. Successive plies are added between layers of epoxy saturate. Excess saturate and bubbles are brushed, squeeged, rolled, and otherwise "worked out" of each layer.

No Holes will be cut in the composite to circumvent obstacles such as pipes, hangers and drain holes. The carbon sheets must be split to circumvent these obstacles. No overlapping is required perpendicular to fiber direction.

The installed carbon fiber system shall be completely free of film defects including air pockets, voids, unsaturated areas, sags, and inclusions. After the installed system has been allowed to cure a minimum of 24 hours, the Contractor shall repair all defects as necessary to comply with the requirements of this Specification.

Maintain control of epoxy resin saturant and carbon fiber material in each area of work. Epoxy resin and laminate will be prevented from entering the bay waters. Clean up and remove unused and discarded excess material at the completion of each workday.

4. REWORK AND REPAIRS

Applied composite systems that are found to be defective or damaged will be replaced if deemed necessary by the NFESC to render the repairs in full compliance with the requirements of this Specification. Rework and repair procedures shall comply with all material and procedural requirements defined in this Specification. Defects shall be repaired in a manner that will restore the system to the designed level of quality. Repair procedures for conditions that are not specifically addressed in the Specification shall be approved by the NFESC on the case-by-case basis prior to initiating any repairs. All repairs shall be made to the satisfaction of the NFESC.

PART 5 ACCEPTANCE TESTING

Soundings of the strengthened areas will be done to check for voids, bubbles and delaminations. All voids, bubbles and delaminations shall be repaired by epoxy injection or replacement.

The direct pull-off test (ASTM D 4541-93) which measures the bond between the laminate and existing concrete substrate in a composite system, shall be used for acceptance of the strengthening project. A core shall be drilled through the carbon laminate strip ¼ - ½in. into the concrete substrate, providing an isolated test location for attachment of the pull-off tester. During the test a tension device shall be loaded to failure. The device will record the force causing the failure, which, if divided by the core cross sectional area will result in tensile strength (psi). Upon failure of the core specimen, a visual examination of the failure plane location reveals whether the failure occurred at the bond line or within the substrate. Failure of the concrete substrate is the only acceptable type of failure.

One pull-off test shall be provided for every 200 S.F. of area strengthened with the carbon fiber system.

Bond strength test procedures shall be approved by the NFESC prior to testing.

Any deviation from this requirement will require a meeting between Contractor and NFESC to identify issues and remedies.

The test areas of the composite system shall be repaired to the satisfaction of the NFESC in accordance with Part 4.

EPOXY BONDED, PULTRUDED FIBERGLASS REINFORCED POLYMER STRUCTURAL SECTIONS

This specification defines the material and procedural requirements for the preparation and installation of epoxy bonded, pultruded fiberglass reinforced polymer structural sections for post strengthening of the reinforced concrete deck of Pier 12, Naval Station, San Diego. The structural sections are 12-inch deep I-sections with 6-inch wide flanges and ½inch thick. The installation will be on the bottom surface of the 8-inch deck on the spans between pile bents 7 through 14 and 67 through 72 in accordance with the Contract drawings. The sections will be bonded to the bottom of the deck and mechanically attached to the pilecaps.

PART 1 GENERAL

1.1 WORK TO BE PROVIDED

The contractor shall complete the following work as shown on the Contract drawings, details and as specified herein:

- Provide scaffolding, working platforms, and access under the deck of Pier 12
- Provide concrete surface preparation
- Install fiberglass reinforced structural sections to the bottom deck surface

Provide all required tests and inspections of the strengthened system.

1.2 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to within the text by the basic designation only.

AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)

ASTM C 882	Standard Test Method for Bond Strength of Epoxy-Resin Systems Used with Concrete by Slant Shear
ASTM D 570	Standard Test Method for Water Absorption of Plastics
ASTM D 635	Standard Test Method for Rate of Burn and/or Extent and Time of Burning of Self-Supporting Plastics in a Horizontal Position
ASTM D 638	Standard Test Method for Tensile Properties of Plastics
ASTM D 695	Standard Test Method for Compressive Properties of Rigid Plastics
ASTM D 732	Standard Test Method for Shear Strength of Plastics by Punch Tool
ASTM D 790	Standard Test Method for Flexural Properties of Unreinforced and Reinforced Plastics and Electrical Insulating Materials

ASTM D 2583	Standard Test Method for Indentation Hardness of Rigid Plastics by Means of a Barcol Impressor
ASTM D 3171	Standard Test Method for Fiber Content of Resin-Matrix Composites by Matrix Digestion
ASTM D 3379	Standard Test Method for Tensile Strength and Young's Modulus for High-Modulus Single-Filament Materials
ASTM 3647	Standard Practice for Classifying Reinforced Plastic Pultruded Shapes According to Composition
ASTM D 3917	Standard Specifications for Dimensional Tolerances Of Thermosetting Glass Reinforced Plastic Pultruded Shapes
ASTM D 3918	Standard Definitions of Terms Relating to Reinforced Plastic Pultruded Products.
ASTM D 4258	Standard Practice for Surface Cleaning Concrete for Coating.
ASTM E 84	Standard Test Method for Surface Burning Characteristics of Building Materials

AMERICAN CONCRETE INSTITUTE (ACI)

ACI 515.1R	A Guide to the Use of Waterproofing, Dampproofing, Protective and Decorative Barrier Systems for Concrete.

NATIONAL ASSOCIATION OF CORROSION ENGINEERS (NACE)

NACE RP 0288	Inspection of Linings on Steel and Concrete

STEEL STRUCTURES PAINTING COUNCIL (SSPC)

SSPC-PA Guide 3	A Guide to Safety in Paint Application.

1.3 SUBMITTALS

Submit the following in accordance with the section entitled "Submittal Procedures".

1.3.1 Instructions

a. Pultruded Fiberglass Reinforced Polymer Structural Section

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include with instructions the quantity of material to be used on the job. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

b. Epoxy Adhesive Systems

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include in the instructions detailed mixing and application procedures, quantity of material to be used per square foot, total quantity of material to be used on job, minimum and maximum application temperatures, and curing procedures. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

b. Stainless Steel Hardware

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include in the instructions application procedures, total quantity of material to be used on job, and minimum and maximum application temperatures. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

1.3.2 Certificates

a. Epoxy Bonded, Pultruded Fiberglass Reinforced Polymer Structural Section

For the fiber reinforcement, certify conformance to the requirements set forth in section 2.1.1. For the polymer matrix, certify conformance to the requirements set forth in section 2.1.2. For the structural section, certify conformance to the requirements set forth in section 2.1.3.

a. Epoxy Adhesive System

For the epoxy adhesive system, certify conformance to the requirements set forth in section 2.2.

b. Stainless Steel Hardware

For the stainless steel hardware employed in the end connections of the structural sections, certify conformance to the requirements set forth in section 2.3.

1.3.3 Records

a. Installers Qualifications

Throughout the progress of the work in this Specification, the Contractor will provide at least one (1) person who is thoroughly familiar with the specified requirements, completely trained and experienced with the necessary skills, and who will be present on the site and direct all work performed under this Specification.

In performing the work of this Specification, the Contractor will use an adequate number of skilled workmen to ensure the installation is in strict accordance with schedule, specification and the procedures recommended by the NFESC.

a. Disposal of Material

All epoxy resins and adhesives shall be disposed of properly as indicated on MSDS's. All epoxy resins and adhesives shall be stored and transported as indicated on MSDS's. Spent sandblasting media shall be contained and disposed of properly as required by local authorities. All materials (epoxies, concrete, grout, sand, etc.) shall be contained at the site and prevented from entering the bay waters.

1.4 QUALITY ASSURANCE

Prior to commencement of any work, the Contractor shall be responsible for arranging and conducting a meeting between the ROICC, Contractor, and NFESC technical engineers to discuss the project requirements. The Contractor shall review the requirements of the Specification and overall project requirements. All aspects of the project including containment, environmental control, surface preparation, strengthening system application, quality assurance, schedule requirements, and safety shall be reviewed and discussed. The Contractor shall request clarification of any ambiguities, and advise the ROICC and NFESC of any potential conflicts and/or any technical requirements that appear improper or inappropriate.

The Contractor shall provide acceptance testing in accordance with Section 5 of this Specification.

The Contractor shall ensure the highest quality of workmanship at all times throughout the progression of the work. Only qualified applicators having prior experience in the specified concrete surface preparation and epoxy adhesives applications to concrete and vinyl esters shall be assigned to perform the work described herein.

The Contractor will submit to full inspection conducted by the ROICC and NFESC of the surface preparation and structural section systems applications to ensure the requirements of this Specification are fully complied with.

1.5 DELIVERY AND STORAGE

All materials shall be delivered in "new" condition only, packaged in their original, unopened containers bearing the manufacturer's name, product identification, batch number(s) and shelf life expiration date.

All materials shall be stored in a covered, well-ventilated area and protected from exposure to any detrimental conditions including: airborne contaminants dirt, dust, sunlight, extreme cold, heat, rainfall, sparks or flame.

When requested by the NFESC, the Contractor shall provide batch samples of any materials that are used throughout the progression of the work. Batch samples will not exceed 1% of the total material used on this job.

1.6 SAFETY

The processes described in this Specification involve potentially hazardous operations. The contractor shall take the necessary precautions in accordance with OSHA regulations, manufacturer's instructions, applicable material safety data sheets, and the Navy's site safety requirements to ensure the safety of all personnel that may be affected by the work described in this Specification.

The Contractor shall be responsible for providing any and all safety equipment required for confined spaces, the use of any products and/or equipment utilized during the progression of the work.

The Contractor shall establish and maintain safe working conditions throughout the progression of the work. The Contractor shall take immediate action to remedy any safety concerns and unsafe, or potentially unsafe working conditions.

The Contractor shall provide safe access to all work areas for inspection by the NFESC and/or their designated representatives.

The Contractor shall conform with the guidelines set forth by SSPC-PA Guide 3, "A Guide to Safety in Paint Application."

PART 2 PRODUCTS

2.1 PULTRUDED FIBERGLASS REINFORCED POLYMER STRUCTURAL SECTIONS

The structural section will be nominally a 12 inch deep I-section with 6-inch flange width. The strong moment of inertia will be at least 254 in^4 and the weak moment of inertia will be 18 in^4. The section will weigh no more than 8.6 lbs per foot. The section will be made of E-glass in a vinyl ester matrix:

Additives such as alumina trihydrate and/or antimony trioxide will be added to the vinyl ester resin for flame retardancy. The structural sections will have a flame-retardant rating of "self extinguishing" per ASTM D-635 or a maximum 25 flame spread per ASTM E-84-80. Ultraviolet Radiation (UV) resistance will be provided by adding a UV inhibitor or absorber, a resin rich surface and a synthetic veil. The structural section will have a 15-mil thick gel coat protection consisting of resin rich outer layer over the exterior surface of the structural section. A Nexus® veil or an equivalent will protect the section surface. UV resistance will also be provided by adding a UV inhibitor or absorber.

The structural sections will be pultruded so that all installed lengths will have minimum dimensional variation as per ASTM D 3917-80. Each delivered length of structural section will be a continuous pultruded piece. Manufactured joints will not be allowed in delivered lengths.

2.1.1 E-Fiberglass

E-fiberglass is the reinforcing of the pultruded structural shapes.

Tensile strength (ksi), as per ASTM D 3379	500
Tensile modulus (ksi), as per ASTM D 3379	10,500

2.1.2. Polymer Matrix

The polymer matrix for the structural sections will be vinyl ester with the flame retardant and a UV absorber or inhibitor.

Vinyl ester

Tensile strength (psi), as per ASTM D 638	11,800
Tensile modulus (psi x 10^5), as per ASTM D 638	4.9
% elongation, as per ASTM D 638	4.5
Flexural strength, as per ASTM D 790	19,400
Flexural modulus (psi x 10^5), as per ASTM D 790	4.5
Barcol hardness, as per ASTM D 2583	35 minimum

2.1.3 Structural Section Properties

Volumetric content of fiberglass fibers (%), as per ASTM D 3171	45 minimum
Longitudinal tensile strength, ultimate (ksi), as per ASTM D 638	37.5
Longitudinal tensile modulus (ksi), as per ASTM D 638	3,000
Transverse tensile strength (ksi), as per ASTM D 638	10
Longitudinal flexural strength (ksi), as per ASTM D 790	37.5

Transverse flexural strength (ksi), as per ASTM D 790	10
Shear strength (ksi), as per ASTM D 732	7
Water absorption, as per ASTM D 570	0.5

2.2 Epoxy Adhesive System

The adhesive system shall include: a primer/sealer, a polymer concrete repair and void filler, a composite-to-concrete adhesive, and a composite to composite adhesive.

2.2.1 Epoxy Primer/Sealer

The epoxy primer/sealer seals the concrete surface while increasing its tensile strength. The primer/sealer is a 100% solids, two component epoxy equivalent to Madewell 927 Primer/Sealer. The viscosity must be less than 1500 cps to insure penetration into concrete. The primer/sealer must not contain organic solvents.

2.2.2 Epoxy Putty Surfacer/Void Filler

The epoxy putty surfacer and void filler provides a void-free concrete surface for the application of the carbon/epoxy strips. The surfacer/void filler should be a 100% solids epoxy equivalent to Madewell 1312P Epoxy Putty.

Compressive strength (ksi), as per ASTM D 695	10
Adhesive strength to concrete (psi), as per ASTM 4541	300 minimum and concrete failure
Tensile strength (ksi), as per ASTM D 638	3.6
Tensile modulus (ksi), as per ASTM D 638	650
Shear Strength (ksi), as per ASTM D 732	3.6

2.2.3 Composite-to-Concrete Epoxy Adhesive

The epoxy adhesive bonds the pultruded structural sections to the concrete deck. The contractor must select a highly filled, 100 percent solids, epoxy adhesive that will bond to concrete and to vinyl ester composite sections. The adhesive shall be a highly filed, 100% solids, high viscosity, two component epoxy that will develop the strength of the structural section without separating from the concrete deck. The adhesive must be viscous to form a fit between the dissimilar surface of the concrete and pultruded composite and to hold in place as it cures. The tensile strength and shear strength must be greater than that of the concrete substrate. This will be demonstrated in preliminary tests for the NFESC.

Compressive strength (ksi), as per ASTM D 695	8.6
Adhesive strength to concrete (psi), as per ASTM D 4541	300 minimum and concrete failure
Tensile modulus (ksi), as per ASTM D 638	650
Tensile strength (ksi), as per ASTM D 638	3.6

Flexural strength (ksi), as per ASTM D 790	6.8
Shear Strength (ksi), as per ASTM D 732	3.6

2.2.4 Composite-to-Composite Adhesive

The two-part epoxy is used in the end connections for attaching the composite structural angle sections to the composite structural I-sections. The adhesive must develop the interlaminar shear strength of the vinyl ester composite.

Transverse tensile strength (psi), as per ASTM D 638	1,500
Ultimate Elongation as per ASTM D638	20%
Lap shear strength (psi), as per ASTM D 732	1,000
T_g as per ASTM D4065	160°
Hardness as per ASTM D2240	60

2.3 END CONNECTION HARDWARE

The structural sections attached each end of the I-sections to the pilecaps or bollard/cleat platforms as shown on the contract drawings. Seats that are anchored to the pile caps will provide vertical support for each structural section. The structural angle sections must meet the same material requirements as the fiberglass composite structural I-sections (2.1.1, 2.1.2, and 2.1.3). The structural angle sections are attached to the pile caps or bollard/cleat platforms with stainless steel anchor bolts that are embedded in epoxy into sound concrete. The structural angles are attached to the structural I-section using a combination composite-to-composite epoxy adhesive and stainless steel bolts, nuts and washers.

2.3.1 Stainless Steel hardware

Alloy group, as per ASTM F 593	#2
Type, as per ASTM F 593	316
Material composition, as per ASTM F 593, UNS S31600	cold worked
Minimum yield strength (ksi), as per ASTM F 593	60
Tensile strength (ksi), as per ASTM F 593	100 to 150
Elongation (percent), as per ASTM F 593	20

2.3.2 Embedment Epoxy

The two-part epoxy for embedding the stainless steel anchor bolts of the end connections must comply with ASTM C 881-90 Type IV Grade 3.

PART 3 EXECUTION

3.1 SURFACE PREPARATION

All concrete surfaces to be strengthened shall be thoroughly prepared to comply with the minimum requirement defined in this section.

Examine carefully the concrete surface to be strengthened. Verify the concrete and cracks have been properly repaired prior to preparation of the surface for attaching the structural sections. Consult the contracting officer and NFESC for final advice and approval of additional areas to be repaired and methods of repair.

Prior to initiating surface preparation procedures, the Contractor shall first prepare a representative sample area for NFESC engineers to conduct adhesive tests in accordance with ASTM D 4541-93. The adhesive tests will verify that the Contractor is capable preparing the concrete surface, applying primer, and applying adhesive to fully develop the concrete strength. The sample area shall be prepared in accordance with the requirements of this Specification, and shall be used as a mutually agreed upon reference standard depicting a satisfactorily prepared surface.

Surface preparation shall be performed to the extent of completely removing all laitance, loosely adhering concrete, and spalling. All bugholes and subsurface voids shall be opened. All surface discontinuities and fins must be removed by chipping or grinding. The areas where the structural sections are attached must be made completely flat.

After removing all surface discontinuities, the contractor will perform abrasive blasting on all concrete surfaces that will have structural sections attached. Use materials and methods for project work as used to produce sample preparation acceptable to the contracting officer and NFESC (See above paragraph). Use abrasive of uniform size and angular shape so as to provide a uniformly etched profile. The roughness of the etched surface shall be equivalent to 150 grit.

Maintain control of abrasive, concrete chips, dust and debris in each area of work. No abrasive material or concrete material shall allowed to enter the Bay waters. Clean up and remove such material at the completion of each day of blasting.

3.2 FIBERGLASS COMPOSITE STRUCTURAL SECTION INSTALLATION

3.2.1General

Fiberglass composite structural sections shall be installed in strict accordance with the manufacturer's written recommendations for procedures and equipment as well as in conjunction with the specific requirements defined herein. In the event of the conflict between the requirements of this Specification and the manufacturer's recommendations, this Specification shall take precedence.

All concrete surfaces to receive the structural sections shall be primed with the specified penetrating primer prior to application of subsequent materials. The areas to be primed will extend two inches beyond the perimeter of each structural section flange print.

Primer shall be mixed in accordance with the manufacturer's recommendations. Do not mix any solvents with the epoxy primer. Volume of primer to be prepared at one time must be such that it can be applied within its batch life. A fixed primer batch that has exceeded its batch life must not be used. The batch life may vary subject to ambient temperature or volume of the mixed primer batch and care must be taken accordingly. Uncured primer and primer components shall not be exposed to moisture. The priming procedure will not commence during rainfall or in the case of predicted rainfall.

Primer shall be applied by brush or roller. Alternatively, the primer may be spray applied with airless spray equipment, followed immediately by thorough backrolling to work the primer into the concrete surface. The primer shall be applied uniformly in sufficient quantity to fully penetrate the concrete and produce a non-porous film in the concrete surface not to exceed one (1) dry mil in thickness after full penetration. If necessary, a second coat shall be applied after the first coat has completely penetrated into the concrete. Volume of primer to be applied may vary depending on the coarseness of the concrete surface but a minimum of 0.46 lbs/yd^2 and a maximum of 0.74 lbs/yd^2 shall be applied.

It is possible that epoxy based materials used in adhesive system may develop higher viscosity and/or slow curing and insufficient curing at low ambient temperature. Do not apply the specified system when substrate temperatures are lower than 50°F (10°C).

Presence of moisture may inhibit adhesion of the system to the concrete substrate. Provide necessary weather protection to protect surfaces from rain or cold.

Maintain control of epoxy resins in each area of work. Primer resins will be prevented from entering the Bay waters. Clean up and remove such material at the completion of each workday.

3.2.2 APPLICATION OF VOID FILLER

Void filler epoxy mortar shall be applied to surfaces that will have I-section flanges attached and that have been primed with the specified penetrating primer. All bugholes and subsurface voids shall be filled. Void filler shall be trowelled tight against the surface with opposing passes. Void filler shall be applied in strict accordance with the manufacturer's recommendations. Do not mix any solvents with the epoxy void filler.

3.2.3 INSTALLATION OF STRUCTURAL SECTIONS

Each structural section length will be continuous - splices will not be allowed. Structural sections are located as shown in the contract drawings of each span. Each length will be field-cut custom fitted to each location with a maximum of 1/16-inch space between the end of the sections and the concrete pile cap where the sections will be anchored.

The bonding surface of the structural section will be the top surface of one flange. The bonding surface must be abraded for bonding to the epoxy adhesive. The abrasion must equivalent to 150 grit. The laminate bonding surface is then thoroughly cleaned with a solvent until all polymer dust and other loose material is removed.

After the void filler mortar has cured the concrete surface is abrasive blasted (e.g. sand blasted) to abrade the primed concrete surface where the structural sections will be applied. The roughened surface will be equivalent to 150 grit roughness. The surface must be vacuumed to remove all dust and foreign material prior to applying adhesive.

The highly filled epoxy adhesive is applied to the surface of the structural section top flange for bonding to the primed concrete surface. The adhesive is applied in a "∩"-shape across the flange width so that the center peak height is 1/20 of the width of the flange. The extra adhesive is squeezed out when the structural section is pressed to the concrete surface.

The structural section (flange) must be pressed to the concrete surface with a minimum pressure of 10 psi until the adhesive cures. Pressure may be applied using mechanical means such as jacks. The applied pressure must be maintained until the adhesive cures.

The structural sections shall then be completely bonded to the concrete surface without bubbles, delaminations or voids. The process shall be carefully planned to allow sufficient working time for placing the structural section to produce a uniform system that is completely free of voids and trapped air. The adhesive layer shall be applied in strict accordance with the manufacturer's recommendations.

No splices or joints are allowed over the length of each structural section. Joints are defined as field splices or prefabricated in the pultrusion process of manufacturing the structural section.

Special care shall be taken to minimize the elapsed time between mixing and application of the adhesive to ensure the material is applied at least fifteen minutes prior to any gelling. The work time limitations will vary according to temperature, and can be determined by information obtained from the manufacturer and practical experience with the product.

Maintain control of epoxy resins in each area of work. Epoxy resin will be prevented from entering the Bay waters. Clean up and remove unused and discarded excess material at the completion of each workday. Do not mix any solvents with the epoxy adhesives.

The installed system shall be completely free of defects including air pockets, voids, sags, and inclusions. The maximum allowable single defect is 1 inch in diameter and the maximum total defective adhesive area will be no more than 5 percent of the total area. After the installed system has been allowed to cure a minimum of 24 hours, NFESC will inspect the adhesive joint for voids, bubbles, and delaminations. The inspection will be conducted before the end connections are installed. Contractor shall repair all defects as necessary to comply with the requirements of this Specification.

3.2.4 INSTALLATION OF END ANCHORS FOR THE STRUCTURAL SECTIONS

The end anchors are required to resist the total shear capacity of the 12-inch section or 50,000 lbs. End anchor details are shown in contract drawings. The installer may present alternatives to end anchors for NFESC approval.

The end anchors consist of pultruded E-fiberglass vinyl ester structural angle sections. The dimensions of the angles listed on the contract drawings 6 inches by 6 inches by ½inch. The material requirements for the structural angles are the same as those for the structural I-sections. The angle lengths may be precut. The angles are attached to the pilecaps or to bollard/cleat platforms with epoxy embedded stainless steel anchor bolts. The stainless steel hardware much meet the requirements of ASTM F593 and F594 stainless steel – no material substitutions are allowed.

Holes are drilled in the concrete pile caps or bollard/cleat platforms in accordance with contract drawings. These may be positioned before installation of the structural I-sections. The anchor bolts must match with holes cut in the angle sections. The holes may be precut or field cut for greater flexibility.

The angles are attached to the structural I-sections by a combination of epoxy adhesive and stainless steel bolts (ASTM F593 and F594). The material requirements for the angle/I-section connections are provided on the contract drawings.

Applied bolt torque is specified on the contract drawings. Torque on the anchor bolts can not be applied until the embedment epoxy is cured. The bolts should be used to hold the end connections in place as the adhesives cure.

PART 4 REPAIRS

Applied structural sections that are found to be defective or damaged will be replaced if deemed necessary by the NFESC to render the repairs in full compliance with the requirements of this Specification. Repair procedures shall comply with all material and procedural requirements defined in this Specification. Defects shall be repaired in a manner that will restore the system to the designed level of quality. Repair procedures for conditions that are not specifically addressed in the Specification shall be approved by the NFESC on the case-by-case basis prior to initiating any repairs. All repairs shall be made to the satisfaction of the NFESC.

PART 5 ACCEPTANCE TESTING

Soundings of the strengthened areas will be done to check for voids, bubbles and delaminations. All voids, bubbles, and delaminations shall be repaired by epoxy injection or replacement.

The direct pull-off test (ASTM D 4541-93) which measures the bond between the adhesive and existing concrete substrate in a composite system, shall be used for acceptance of the strengthening project. NFESC will select an isolated test location for attachment of the pull-off tester. During the test a tension device shall be loaded to failure. The device will record the force causing the failure, which, if divided by the core cross sectional area will result in tensile strength (psi). Upon failure of the core specimen, a visual examination of the failure plane location reveals whether the failure occurred at the bond line or within the substrate. Failure at the bond line at tensile stress below 300 psi will be unacceptable. The most desirable failure is within the concrete substrate.

One pull-off test shall be provided for every span strengthened with the fiberglass composite structural sections.

Bond strength test procedures shall be approved by the NFESC prior to testing.

Any deviation from this requirement will require a meeting between Contractor, the contracting officer and NFESC to identify issues and remedies.

The test areas of the composite system shall be repaired to the satisfaction of the NFESC in accordance with Part 4.

E-FIBERGLASS COMPOSITE SHELL SYSTEM FOR CONFINEMENT STRENGTHENING OF REINFORCED CONCRETE PILES

This specification defines the material and procedural requirements for the preparation and installation of an E-glass fiber composite shell system for confinement strengthening of reinforced concrete piles supporting Pier 12, Naval Station, San Diego. The selected piles are supporting pilecaps 7 through 13 and 67 through 72. The existing pile cross sections are 20 inches square. The strengthened cross section will be nominally 28.5 inches in diameter. The length of the strengthened sections extends 8 feet from the pile cap.

PART 1 GENERAL

1.1 WORK TO BE PROVIDED

The contractor shall complete the following work as shown on the contract drawings, details and as specified herein:

Provide scaffolding, working platforms, and access below the deck of Pier 12

Provide concrete surface preparation of selected piles

Install the preformed circular composite shell system on selected piles

Provide all required tests and inspections of the strengthened system

1.2 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to within the text by the basic designation only.

AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)

ASTM D 635	Standard Test Method for Rate of Burn and/or Extent and Time of Burning of Self-Supporting Plastics in a Horizontal Position
ASTM D 638	Standard Test Method for Tensile Properties of Plastics
ASTM D 695	Standard Test Method for Compressive Properties of Rigid Plastics
ASTM D 1002	Standard Test Method for Apparent Shear Strength of Single-Lap-Joint Adhesively Bonded Metal Specimens by Tension Loading (Metal-to-Metal)
ASTM D 2240	Standard Test Method for Rubber Property Durometer Hardness
ASTM D 2583	Standard Test Method for Indentation Hardness of Rigid Plastics by Means of a Barcol Impressor
ASTM D 3039	Standard Test Method for Tensile Properties of Polymer Matrix Composite Materials Composites by Matrix Digestion
ASTM D 3171	Standard Test Method for Fiber Content of Resin-Matrix Composites by Matrix Digestion

ASTM D 4065	Standard Practice for Determining and Reporting Dynamic Mechanical Properties of Plastics
ASTM D 4258	Standard Practice for Surface Cleaning Concrete for Coating.
ASTM E 84	Standard Test Method for Surface Burning Characteristics of Building Materials

AMERICAN CONCRETE INSTITUTE (ACI)

ACI 515.1R	A Guide to the Use of Waterproofing, Dampproofing, Protective, and Decorative Barrier Systems for Concrete.

NATIONAL ASSOCIATION OF CORROSION ENGINEERS (NACE)

NACE RP 0288	Inspection of Linings on Steel and Concrete

STEEL STRUCTURES PAINTING COUNCIL (SSPC)

SSPC-PA Guide 3	A Guide to Safety in Paint Application.

1.3 SUBMITTALS

Submit the following in accordance with the section entitled "Submittal Procedures".

1.3.1 Instructions

a. E-glass Fiber Composite Shell System

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include with the instructions application procedures, quantity of material to be used per square foot of pile area, and total quantity of material to be used on job. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

b. Adhesive

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include in the instructions detailed mixing and application procedures, quantity of material to be used per pile, total quantity of material to be used on job, minimum and maximum application temperatures, and curing procedures. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

c. Grout

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include in the instructions detailed mixing and application procedures, quantity of material to be used per pile, total quantity of material to be used on job, minimum and maximum application temperatures, and curing procedures. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

1.3.2 Certificates

a. E-glass Fiber Composite Shell System

For the fiber reinforcement, certify conformance to the requirements set forth in section 2.1.1. For the composite matrix system, certify conformance to the requirements set forth in section 2.1.2. For the shell composite, certify conformance to the requirements set forth in section 2.1.3. For the epoxy adhesive, certify conformance to the requirements set forth in section 2.1.4.

a. Adhesive

For the Adhesive, certify conformance to the requirements set forth in section 2.2.

b. Grout

For the Grout, certify conformance to the requirements set forth in section 2.2.

1.3.3 Records

a. Installers Qualifications

Throughout the progress of the work in this Specification, the Contractor will provide at least one (1) person who is thoroughly familiar with the specified requirements, completely trained and experienced with the necessary skills, and who will be present on the site and direct all work performed under this Specification.

In performing the work of this Specification, the Contractor will use an adequate number of skilled workmen to ensure the installation is in strict accordance with schedule, specification and the procedures recommended by the NFESC.

b. Disposal of Material

All polymer resins and adhesives shall be disposed of properly as indicated on MSDS's. All epoxy resins and adhesives shall be stored and transported as indicated on MSDS's. Spent sandblasting media shall be contained and disposed of properly as required by local authorities. All materials (epoxies, concrete, grout, sand, etc.) shall be contained at the site and prevented from entering the bay waters.

1.4 QUALITY ASSURANCE

Prior to commencement of any work, the Contractor shall be responsible for arranging and conducting a meeting between the CONTRACTING OFFICER, Contractor, and NFESC engineers to discuss the project requirements. The Contractor shall review the requirements of the Specification and overall project requirements. All aspects of the project including containment, environmental control, surface preparation, strengthening system application, quality assurance, schedule requirements, and safety shall be reviewed and discussed. The Contractor shall request clarification of any ambiguities, and advise the CONTRACTING OFFICER and NFESC of any potential conflicts and/or any technical requirements that appear improper or inappropriate.

The Contractor shall provide repair acceptance testing in accordance with Section 5 of this Specification.

The Contractor shall ensure the highest quality of workmanship at all times throughout the progression of the work. Only qualified applicators having prior experience in the specified surface preparation and polymer coatings applications shall be assigned to perform the work described herein.

The Contractor shall provide full inspection of the surface preparation and composite systems applications to ensure the requirements of this Specification are fully complied with.

1.5 DELIVERY AND STORAGE

All materials shall be delivered in "new" condition only, packaged in their original, unopened containers bearing the manufacturer's name, product identification, batch number(s) and shelf life expiration date.

All materials shall be stored in a covered, well-ventilated area and protected from exposure to any detrimental conditions including: airborne contaminants dirt, dust, sunlight, extreme cold, heat, rainfall, sparks or flame.

When requested by the NFESC, the Contractor shall provide batch samples of any materials that are used throughout the progression of the work. The batch samples shall not exceed 1 percent of the total material of the job.

1.6 SAFETY

The processes described in this Specification involve potentially hazardous operations. The contractor shall take the necessary precautions in accordance with OSHA regulations, manufacturer's instructions, applicable material safety data sheets, and the Navy's site safety requirements to ensure the safety of all personnel that may be affected by the work described in this Specification.

The Contractor shall be responsible for providing any and all safety equipment required for confined spaces, the use of any products and/or equipment utilized during the progression of the work.

The Contractor shall establish and maintain safe working conditions throughout the progression of the work. The Contractor shall take immediate action to remedy any safety concerns and unsafe, or potentially unsafe, working conditions.

The Contractor shall provide safe access to all work areas for inspection by the NFESC and/or their designated representatives.

The Contractor shall conform with the guidelines set forth by SSPC-PA Guide 3, "A Guide to Safety in Paint Application."

PART 2 PRODUCTS

2.1 E-Fiberglass Composite Shell System

The fiberglass composite shell must provide lateral confinement of the reinforced concrete. The preformed shell will consist of E-glass fibers in a vinyl ester matrix with flame-retardant and ultraviolet radiation (UV) inhibitors or absorbers. A fiberglass or polymeric veil (Nexus® veil or an equivalent) should be added to the shell construction to provide a resin rich surface to the shell. The shell will have a 3-mil thick resin rich outer layer protection.

2.1.1 E-Fiberglass

Fiber area weight density (areal weights)(5% deviation)

90^0 continuous strand roving, as per ASTM D 3171	45.0 oz/yd^2
Mat, as per ASTM D 3171	4.50 oz/yd^2
Tensile strength (ksi) as per ASTM D 3039	65
Tensile modulus (ksi) as per ASTM D 3039	3,250
Tensile elongation, ultimate, percent, as per ASTM D 3039	1.5

2.1.2 Matrix Resin

The composite matrix will be a halogenated fire retardant vinyl ester with antimony trioxide additive and UV stabilizer or inhibitor. The structural sections will have a flame retardant rating of "self extinguishing" per ASTM D-635 or a maximum 25 flame spread per ASTM E-84-80. The structural section will have a 15-mil gel coat protection consisting of resin rich outer layer over the exterior surface of the structural section. The section surface will be protected by a Nexus® veil or an equivalent.

Tensile Strength (psi), as per ASTM D 638	11,000
Tensile modulus (ksi), as per ASTM D 638	490
Compressive strength (psi), as per ASTM D 695	16,000
Compression modulus (ksi), as per ASTM D 695	3,500
Barcol hardness, as per ASTM D 2583	35 minimum
Fire rating as per ASTM E-84	25 minimum

2.1.3 Shell composite

The composite shell shall utilize fillers and a surface veil plus a 15-mil gel coat to minimize the effects of exposure to UV. The contractor must demonstrate to the satisfaction of the NFESC that the system will retain more than 85 percent of the following strength values when exposed to 1 ppm ozone, 12.5 pH, and UV at 140°F. Laminate shell shall possess Class I fire-retardance.

Ultimate tensile strength (psi), as per ASTM D 3039	10,000
Circumferential tensile strength of the shell (kips per inch)	4
Longitudinal tensile strength of the shell (kips per inch)	1
Tensile Modulus (ksi), as per ASTM D 3039	5,000
Ultimate tensile strain (%), as per ASTM D 638	2.5%
Fiber volume fraction (%), as per ASTM D 3171	55%
Glass transition temperature (^{0}F), as per ASTM D	150
Surface flammability, as per ASTM E-84	25 max.
Rate of combustion, as per ASTM D 635 (Flammability - self extinguishing)	0.16 in max.
Flame resistance, as per FTMS 406-20 2023, ignition/burn	75/75 seconds

2.3 ADHESIVE

Joints and connections of the confinement shell will be joined with a two-part epoxy adhesive that will develop the interlaminar shear strength of the shell matrix.

Minimum lap shear strength (psi), as per ASTM D 1002	800
Minimum ultimate tensile strength (psi), as per ASTM D 638	1,500
Elongation (%), as per ASTM D 638	20

Glass transition temperature (^{0}F), as per ASTM D4065	160
Shore hardness, as per ASTM D2240)	60

2.3 GROUT

The grout must be pumped into the space between the shell and the existing pile exterior surface. The grout must have a shrinkage ratio less than 0.05 percent. The NFESC recommends the following grout mixture. The contractor may submit an alternative mixture or prepackaged grout for NFESC engineer approval.

Max Aggregate size: 1inch

Cement: 6.5 sacks Type II plus fly ash

Quantities per cubic yard:	Cement	560 lbs	
	Fly Ash	98 lbs	
	1-inch media	1680 lbs	
	Sand	1280 lbs	
	Water	274.9 lbs	33 gallons
	Admixtures		
	DARACEM 100®	79 lbs	
	DARAVAIR®	4.94 lbs	

Air percentage: 3.00 percent
Design slump: 8 inch maximum

Compressive strength (psi) 4,000

PART 3 EXECUTION

3.1 SURFACE PREPARATION

All pile concrete surfaces to be confined will be prepared to comply with the minimum requirement defined in this section.

Prior to initiating surface preparation procedures, the Contractor shall first prepare a representative sample pile. The sample pile shall be prepared in accordance with the requirements of this Specification, and shall be used as a mutually agreed upon reference standard depicting a satisfactorily prepared surface.

Surface preparation shall be performed to the extent of completely removing all sea life, loosely adhering concrete and shotcrete, and spalling.

Maintain control of concrete chips and debris in each area of work. Clean up and remove such material at the completion of each day of concrete removal and blasting.

3.2 INSTALLATION OF PREFORMED SHELL

3.2.1 General

The fiberglass reinforced composite shell system shall be installed in strict accordance with the manufacturer's written recommendation for procedures and equipment, in conjunction with the specific requirements defined herein. In the event of the conflict between the requirements of the Specification and the manufacturer's recommendations, this Specification shall take precedence.

It is possible that epoxy based materials used in the adhesive system may develop higher viscosity and/or slow curing and insufficient curing at low ambient temperature. Do not apply the specified adhesive system when substrate temperatures are lower than 50^0F (10^0C).

The two-part epoxy adhesive shall be mixed in accordance with the manufacturer's recommendations. Volume of adhesive to be prepared at one time must be such that it can be applied within its batch life. A fixed adhesive batch which has exceeded it batch life must not be used. Solvents and thinners are not allowed to thin adhesives used in this project. The batch life may vary subject to ambient temperature or volume of the mixed adhesive batch and care must be taken accordingly.

Preparation of laminate surface for adhesive. Smooth, resin-rich outer surfaces should be abraded by sanding or light sand blasting to remove mold release agents if present. Surfaces should be cleaned and decontaminated with a solvent such as methyl alcohol.

The bonded surfaces are clamped and/or fastened until the adhesive is cured. The adhesive should be allowed to cure for 24 hours before grout is pumped into the cavity between the circular shell and the square pile.

The adhesive joints of the installed composite shell shall be completely free of film defects including air pockets, voids, unsaturated areas, sags, and inclusions. After the installed system has been allowed to cure a minimum of 24 hours, the Contractor shall repair all defects as necessary to comply with the requirements of this Specification.

Maintain control of epoxy resins, putty and laminate material in each area of work. Epoxy resin, putty, and laminate will be prevented from entering the bay waters. Clean up and remove unused and discarded excess material at the completion of each workday.

3.2.2 FILLING SHELL WITH GROUT

After the adhesive has cured the composite shell will be centered on the axis of the pile and anchored in place. All openings will be sealed.

Grout shall be pumped into the shell in strict accordance with the shell manufacturer's recommendations.

PART 4 REPAIRS

Composite shell systems that are found to be defective or damaged will be replaced if deemed necessary by the NFESC to render the repairs in full compliance with the requirements of this Specification. Repair procedures shall comply with all material and procedural requirements defined in this Specification. Defects shall be repaired in a manner that will restore the system to the designed level of quality. In effect, if the shell is defective, a new shell of design level properties will be installed over the defective shell and cemented in place with the adhesive in compliance with the specifications above. Repair procedures for conditions that are not specifically addressed in the Specification shall be approved by the NFESC on the case-by-case basis prior to initiating any repairs. All repairs and touch up shall be made to the satisfaction of the NFESC.

PART 5 ACCEPTANCE TESTING

Soundings of the strengthened areas will be done to check for voids, bubbles and delaminations. All voids, bubbles and delaminations shall be repaired by epoxy injection, replacement, or by some other means derived by the contractor and approved by the NFESC.

Any deviation from this requirement will require a meeting among the Contracting Officer, the Contractor, and the NFESC to identify issues and remedies.

FIBERGLASS COMPOSITE HANGERS FOR 14-INCH INSULATED STEAM LINE

This specification defines the material and procedural requirements for the preparation and installation of pipe hangers for the 14-inch insulated steam line of Pier 12, Naval Station, San Diego. The composite hangers will be anchored to the bottom surface of the deck.

The 14-inch outside diameter steam pipeline near the bottom of the utility loop crosses under the deck from the north side of Pier 12 to the south at midspan between pile bents 9 and 10. The steam pipeline consists of an inner steel carrier pipe surrounded by an insulating fiberglass conduit. As it crosses under the deck it passes within ½ inch of the 24-inch deck section. The contractor will disconnect this line in the spans near the upgrades between pile bents 9 and 10. The line will be lowered to rest on the utility loop to provide clearance to complete the repair and upgrade. The steam line can only be moved after a NAVSTA-approved steam outage is issued and the steam has been turned off. The contractor will place a non-abrasive, energy absorbent rubber-like material between the steam line and the utility loop and lower the line onto it. The contractor will remove the hangers that were supporting the steam line and the hanger holes will be repaired in accordance with contract specifications for concrete repair. After the repair and upgrade is completed, the contractor will re-hang the steam pipe in a method that will not damage the fiberglass conduit or the steel carrier pipe and place the line in a position that will clear the added upgrades. The steam line must be hung to slope towards an existing steam trap in the line on the north side of the span between pile bents 9 and 10. The steam line will not slope toward a point where there is not trap installed. Fiberglass reinforced vinyl ester composite pipe hangers will be used in accordance with contract specifications. The contractor will repair any damage that occurs to the steam line using the same material as existing.

PART 1 GENERAL

1.1 WORK TO BE PROVIDED

The contractor shall complete the following work as shown on the contract drawings, details and as specified herein:

1. Provide working platform and access beneath Pier 12
2. Obtain Naval Station approved steam outage permit to turn off the steam source
3. Disconnect steam pipe line from the deck in the vicinity of the crossing of the between pilecaps 9 and 10
4. Dispose the existing pipe hangers and repair the pipe hanger holes according to concrete repair specifications
5. Lower the pipeline temporarily onto a rubberlike, nonabrasive support placed at the base of the utility loop
6. After repairs and upgrade re-hang the pipeline from the deck using the same number of hangers at similar locations as originally supporting the pipeline
7. Repair any damage to the steam line that occurred during the upgrade installation and pipeline relocation.

1.2 REFERENCES

The publications listed below form a part of this specification to the extent referenced. The publications are referred to within the text by the basic designation only.

AMERICAN SOCIETY FOR TESTING AND MATERIALS (ASTM)

ASTM B 565	Standard Test Method for Shear Testing of Aluminum and Aluminum-Alloy Rivets and Cold-Heading Wire and Rods

ASTM C 881	Standard Specification for Epoxy-Resin-Based Bonding Systems for Concrete
ASTM D 635	Standard Test Method for Rate of Burning and/or Extent and Time of Burning of Self-Supporting Plastics in a Horizontal Position
ASTM D 790	Standard Test Method for Flexural Properties of Unreinforced and Reinforced Plastics and Electrical Insulating Materials
ASTM E 84	Standard Test Method for Surface Burning Characteristics of Burning Materials
ASTM F 593	Standard Specification for Stainless Steel Bolts, Hex Cap Screws, and Studs
ASTM F 594	Standard Specification for Stainless Steel Nuts

AMERICAN CONCRETE INSTITUTE (ACI)

ACI 515.1R	A Guide to the Use of Waterproofing, Dampproofing, Protective, and Decorative Barrier Systems for Concrete.

AMERICAN NATIONAL STANDARDS INSTITUTE (ANSI)

ANSI B18.22.1	Stainless Steel Washer
ANSI B31.3	Static Loading

MILITARY SPECIFICATION (MIL)

MIL-R-7575	Molded Plastics

1.3 SUBMITTALS

Submit the following in accordance with the section entitled "Submittal Procedures".

1.3.1 Instructions

a. Fiberglass Composite Hangers

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include in the instructions application procedures, total quantity of material to be used on job, and minimum and maximum application temperatures. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

b. Anchor Rods

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include in the instructions application procedures, total quantity of material to be used on job, and minimum and maximum application temperatures. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

c. Stainless Steel Nuts and Washers

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include in the instructions application procedures, total quantity of material to be used on job, and minimum and maximum application temperatures. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

d. Epoxy

Submit supplier's printed instructions to include brand name, catalog numbers, and names of manufacturers. Include in the instructions detailed mixing and application procedures, total quantity of material to be used on job, and minimum and maximum application temperatures. Include copies of Material Safety Data Sheets (MSDS) for all materials to be used at the job site in accordance with OSHA\& 29 CFR 1926.59.

1.3.2 Certificates

a. Fiberglass Composite Hangers

For the Fiberglass Composite Hangers, certify conformance to the requirements set forth in section 2.1.1.

b. Anchor Rod

For the Anchor Rod, certify conformance to the requirements set forth in section 2.1.2.

c. Stainless Steel Nut and Washer

For the Stainless Steel Nut and Washer, certify conformance to the requirements set forth in section 2.1.3.

d. Epoxy

For the Epoxy, certify conformance to the requirements set forth in section 2.1.4.

1.3.3 Records

a. Installers Qualifications

Throughout the progress of the work in this Specification, the Contractor will provide at least one (1) person who is thoroughly familiar with the specified requirements, completely trained and experienced with the necessary skills, and who will be present on the site and direct all work performed under this Specification.

In performing the work of this Specification, the Contractor will use an adequate number of skilled workmen to ensure the installation is in strict accordance with schedule, specification and as recommended by the NFESC.

b. Disposal of Material

All epoxy resins and adhesives shall be disposed of properly as indicated on MSDS's. All epoxy resins and adhesives shall be stored and transported as indicated on MSDS's. Spent sandblasting media shall be contained and disposed of properly as required by local authorities. All materials (epoxies, concrete, grout, sand, etc.) shall be contained at the site and prevented from entering the bay waters.

1.4 QUALITY ASSURANCE

Prior to commencement of any work, the Contractor shall be responsible for arranging and conducting a meeting between the CONTRACTING OFFICER, Contractor, and NFESC engineers to discuss the project requirements. The Contractor shall review the requirements of this Specification and overall project requirements. All aspects of the project including containment, environmental control, surface preparation, strengthening system application, quality assurance, schedule requirements, and safety shall be reviewed and discussed. The Contractor shall request clarification of any ambiguities, and advise the CONTRACTING OFFICER and NFESC of any potential conflicts and/or any technical requirements that appear improper or inappropriate.

The Contractor shall provide repair acceptance testing in accordance with Section 5 of this Specification.

The Contractor shall ensure the highest quality of workmanship at all times throughout the progression of the work. The contractor shall submit to full inspection of the completed system by NFESC to ensure the requirements of this specification are fully complied with.

1.5 DELIVERY AND STORAGE

All materials shall be delivered in "new" condition only, packaged in their original, unopened containers bearing the manufacturer's name, product identification, batch number(s) and shelf life expiration date.

All materials shall be stored in a covered, well-ventilated area and protected from exposure to any detrimental conditions including: airborne contaminants dirt, dust, sunlight, extreme cold, heat, rainfall, sparks or flame.

When requested by the NFESC, the Contractor shall provide batch samples of any materials that are used throughout the progression of the work. The batch samples shall not exceed 1 percent of the total material of the job.

1.6 SAFETY

The processes described in this Specification involve potentially hazardous operations. The contractor shall take the necessary precautions in accordance with OSHA regulations, manufacturer's instructions, applicable material safety data sheets, and the Navy's site safety requirements to ensure the safety of all personnel that may be affected by the work described in this Specification.

The Contractor shall be responsible for providing any and all safety equipment required for confined spaces, the use of any products and/or equipment utilized during the progression of the work.

The Contractor shall establish and maintain safe working conditions throughout the progression of the work. The Contractor shall take immediate action to remedy any safety concerns and unsafe, or potentially unsafe working conditions.

The Contractor shall provide safe access to all work areas for inspection by the NFESC and/or their designated representatives.

Protective gear such as clothing, masks, goggles, and rubber gloves must be used without fail during installation of anchors for the pipe hangers and while setting them in epoxy. Care must be taken to prevent inhalation of resin fumes.

PART 2 PRODUCTS

2.1 FIBERGLASS COMPOSITE HANGERS

2.1.1 Hangers

Fiberglass pipe hangers will be clevis type, or other acceptable composite, hangers to fit 14-inch outside diameter pipe. They shall be contact molded according to MIL-R-7575, Grade 1, with vinyl ester resin with a class I flame spread rating (25 or less) according to ASTM E-84 and rated self extinguishing according to ASTM D-635. The fiberglass reinforcing will be bi-directional, E-glass cloth. The glass to resin volume ratio will be two to one. The composite will have a synthetic surface veil for corrosion resistance and UV protection. The laminate will have an ultimate strength of 37,000 psi and a modulus of 2,500,000 psi. The assembled hanger must have a capacity of 1200 lbs. The clevis hanger will have a thickness of 0.24 inches minimum and a width of a least 5.5 inches. The transverse, fiberglass-reinforced vinyl ester, threaded rod through the clevis will be 1/2-inch diameter.

2.1.2 Anchor Rod

Fiberglass composite hangers will be employed to hang the steam pipe after the deck slab has been upgraded. The hangers will be anchored in the concrete at similar locations to the current hangers using either ¾-inch diameter stainless steel threaded rods (ASTM F593) OR 1-inch diameter pultruded, fiberglass-reinforced vinyl ester threaded rod. Fiberglass-reinforced vinyl ester threaded rods will have a flexural strength of 55,000 psi, a flexural modulus of 2,500,000 psi per ASTM D-790. The shear strength will be 10,000 psi minimum per ASTM B-565 and have a self extinguishing fire rating per ASTM D-635. The anchor rods will be embedded in the concrete a minimum of 4.5 inches

2.1.3 Stainless Steel Nut and Washer

The stainless steel nuts shall conform to ASTM F 594 and the stainless steel washers to ANSI B18.22.1.

2.1.4 Epoxy

The anchor rod into the bottom of the deck will be imbedded in Epoxy per ASTM C661-90 Type IV Grade 3.

PART 3 EXECUTION

The anchor bolts for the pipe hangers will be embedded in epoxy in sound concrete. The location of the hangers will be in similar positions as those that were removed to lower the pipeline. Hanger locations will be adjusted to avoid disrupting the continuity of external reinforcing added as strength upgrade. Similar or greater numbers of hangers will be deployed as were removed. The holes for the anchor bolts will be drilled 1/8-inch oversize than the bolt diameter. Face shields, goggles, and gloves will be worn by workers drilling holes in the concrete.

The drilled hole will be thoroughly cleaned of all loose material before embedding epoxy and bolt. Presence of moisture will inhibit the adhesion of the epoxy. The hole must be completely dry before applying the epoxy. Do not apply the epoxy when the humidity is greater than 80 percent. The epoxy will be cured a minimum of 24 hours before placing the hanger and mounting the steam pipeline. Each anchor bolt will be proof tested to ½design capacity by NFESC engineers before the pipe hangers are positioned.

The pipeline level will be adjusted to insure a straight slope to the nearest water trap, which is located on the north side of Pier 12 at he crossing between pilecaps 9 and 10.

PART 4 REPAIRS

Applied composite systems that are found to be defective or damaged will be replaced if deemed necessary by the NFESC to render the repairs in full compliance with the requirements of this Specification. Repair procedures shall comply with all material and procedural requirements defined in this Specification. Defects shall be repaired in a manner that will restore the system to the designed level of quality. Repair procedures for conditions that are not specifically addressed in the Specification shall be approved by the NFESC on the case-by-case basis prior to initiating any repairs. All repairs shall be made to the satisfaction of the NFESC.

3. Malvar, L. J., "Durability of Composites in Concrete," First International Conference on Durability of Composites for Construction, CDCC'98, Sherbrooke, Quebec, Canada, August 1998, pp.361-372

CDCC'98 - First International Conference on
Durability of Composites for Construction
Sherbrooke (Québec) Canada
August 1998

DURABILITY OF COMPOSITES IN REINFORCED CONCRETE

L.J. Malvar
Naval Facilities Engineering Service Center
Port Hueneme, CA 93043-4370

ABSTRACT

Many fiber reinforced plastics (FRP's) have excellent corrosion resistance properties and can be engineered to have mechanical properties comparable to steel. These characteristics have promoted their use in many structural applications all over the world. Although the short-term mechanical properties of these materials are usually well documented, long-term durability issues still remain. Some of these issues are hereby summarized, mostly for the case of reinforced concrete applications.

Experimental observations indicate that all fiber reinforced plastics have long-term strengths that are only a fraction of the short-term strength. For glass, aramid and carbon FRP's, the fraction is about 30%, 50% and 80%, respectively. In addition glass and aramid FRP's will degrade if in direct contact with concrete, in the presence of moisture, and when subjected to UV radiation. These poor durability characteristics place significant restrictions on the working stress allowables for design.

INTRODUCTION

Excellent mechanical and corrosion resistant characteristics have promoted the use of fiber reinforced plastics (FRP's) in many structural applications all over the world (Malvar, 1996; Nanni, 1993). Although the short-term mechanical properties of these materials are usually well documented, long-term durability issues still remain. Some of these issues are hereby summarized, mostly for the case of reinforced concrete applications.

BACKGROUND

Recent structural applications of FRP materials include:

(a) reinforcing bars for concrete structures, typically with E-glass fibers embedded in a vinylester or epoxy matrix. For these bars, strengths are around 80 to 100 ksi (552 to 690 MPa), but the modulus of elasticity is only around 6 Msi or 41 GPa (versus 29 Msi or 200 GPa for steel bars) (Malvar, 1995). Aramid, carbon, vinylon, as well as hybrid rebars can also be obtained

(b) prestressing strands (Iyer, 1993, 1994; Sen et al, 1993a, 1993b, 1996c; Malvar, 1996)

(c) fiberglass sheets to retrofit columns (Fyfe, 1995; Priestley et al., 1992)

(d) carbon sheets to retrofit columns, beams and slabs (Sultan, 1995; Inaba, 1996)

(e) gratings and railings (Warren et al., 1995), structural shapes

(f) cables (Burgoyne, 1993)

Current FRP composites for structural applications use mainly three types of continuous fibers: carbon, aramid and glass. These are often designed as CFRP, AFRP and GFRP composites, respectively. E-glass fibers are the cheapest (around \$0.80 per lb.) and most commonly used. They have a tensile modulus of 10.5 Msi (72 MPa) and a tensile strength around 500 ksi (3.45 GPa) at a strain of 4.8%, although in composite applications ultimate strains rarely reach 3.4% (2.5% is more common), with the corresponding strength reduction. S-glass fibers have more desirable properties, e.g. about a 20% higher strength and modulus of elasticity, but may cost up to 8 times more. Aramid fibers have slightly higher tensile moduli E (between 83 and 186 GPa, i.e. between 12 and 27 Msi, with 1 Msi = 10^3 ksi = 10^6 psi) and similar tensile elongations (between 2 and 4%). Carbon fibers, however, offer a wide variety of moduli, from 25 to 120 Msi (172 to 827 GPa). The most common (such as AS4 and T300) have a modulus around 33 Msi (227 GPa) and a strength around 550 ksi (3.8 GPa) (Malvar, 1996). Other fibers have been used, such as Vinylon (or polyvinyl alcohol, PVA) and Polyester, but in limited applications. With respect to fiber content, total contents from 45% to 68% by volume (i.e. 60% to 80% by weight) have been reported for pultruded composites.

Epoxies and polyesters are often used as the composite matrix for their fiber protection properties. Polyester resins are not very resistant to alkalis and are typically avoided for uses in concrete. Vinylester resins are resistant to a wide range of acids (sulfuric, hydrochloric, hydrofluoric, phosphoric, nitric) as well as to chloride salts and chlorine making them ideal for marine environments. For use in concrete, BPA fumarates have shown the best resistance to strong basic solutions (ASM, 1987). Epoxies can be even more resistant but are somewhat more expensive.

LONG-TERM STRENGTH

Most composites exhibit a long-term static strength that is significantly lower than the short-term strength. This long-term static strength is observed by exposing the material to sustained stress for a long period of time and without any specific adverse environmental exposure (tests typically run in air, indoors, and at ambient temperature). This failure due to the degradation of the material properties with time is also referred to as creep rupture. This loss of strength can be accelerated in adverse environments, such as in the presence of water, or strong acidic or alkaline solutions.

For Polystal E-glass tendons, the long-term static strength at 10,000 hours (about 1 year) has been reported to be 70% of the short-term static strength (Wolff and Miesseler, 1989; Taerwe, 1993). Sultan et al. (1995) report remaining strengths after 10 to 15 years of 40% for hand laid-up fiberglass, and 55% for filament wound composites. Slattery (1994) reports that long-term tests on Glass/Epoxy composites showed failure of about half of the samples tested at a sustained stress of only 50% of ultimate, after about 7 years. Some of the samples ruptured at levels as low as 33% of ultimate. Active E-glass composite wraps applied as confinement for some of CALTRANS circular highway columns and pressurized via grout slurry failed after 3 years under sustained stresses around 32% of the manufacturer's reported strength (Hawkins et al., 1996). If the most conservative estimates from these tests are used, it appears that the long-term static strength of glass composites can be as low as about 32% of the short-term strength.

For Kevlar fibers, the 100-year sustained strength is around 60% of short-term strength (Taerwe, 1993; Horn et al., 1977). PARAFIL ropes using the high modulus Aramid fibers limit the long-term (100 year) tensile strength to 50% of the short-term strength (Burgoyne, 1993). This matches tests on Kevlar/Epoxy composites which show a sustained strength of 60% after about 7 years (Slattery, 1994). If the most conservative estimate from these tests is used, it appears that the long-term static strength of aramid composites can be as low as 50% of the short-term strength.

Test data on carbon fibers shows very few failures after several years and a sustained stress of 80% of the short-term ultimate (Slattery, 1994).

DESIGN ALLOWABLES VERSUS LONG-TERM STRENGTH

Working stress design is often recommended for concrete members reinforced with commercially available FRP rebars. Most of these use E-glass fibers in a vinylester matrix. The recommended allowable working stress is often 30% to 40% of the ultimate tensile strength, that is around 30 ksi or 200 MPa (since most E-glass rebar manufacturers claim short-term ultimate strengths from around 80 to 100 ksi, or 550 to 690 MPa) (Malvar, 1995, 1996). Assuming that no alkali degradation will occur during the structure's life, the recommended working stress is approximately equal to the long-term strength for E-glass composites, as indicated previously. Hence the long-term design factor of safety is 1. In contrast, if working stress design is used for a grade 60 bar, the allowable stress is limited to 24 ksi or 165 MPa. The long-term ultimate strength for a typical grade 60 bar (also assuming no corrosion) is about 109 ksi or 750 MPa (Malvar, 1998). Hence for a steel bar the long-term factor of safety is 4.5 for working stress design. To get a similar safety factor for an E-glass bar, its allowable stress should rather be around 6% to 10% of the measured short-term strength, i.e., not more than 10 ksi (69 MPa) for a typical 100 ksi (690 MPa) nominal strength pultruded E-glass bar. At least one manufacturer recommends limiting sustained stresses to 25% of the tensile strength, and another recommends limiting the allowable working stress to 25%. In the design of fiberglass pipes, the recommended axial tensile design stress is often 25% of ultimate (Composites Institute, 1992). It should, however, be noted that some conservatism could be built into the measurement of the short-term strength of the rebar if standard tensile tests (e.g. ASTM D3916) are used (Malvar and Bish, 1995).

To adopt a design allowable stress level, it appears that perhaps a distinction should be made between sustained and transient loads. In reinforced concrete structures, however, the dead load is often a large portion of the total load applied to the structure. Hence a simple limit on the allowable working stress, both for reinforced and poststressed applications may be sufficient.

In summary, it appears that, in the absence of adverse environmental factors, and until further research is conducted, the allowable working stress for glass FRP rebar should be limited to 25% of the ultimate tensile strength or 25 ksi (172 MPa), whichever is lower. This would provide some safety with respect to the observed long-term strengths of about 32% of the short-term ultimate. For glass rebars in concrete, this would apply *only* if either (1) direct contact with the concrete is prevented, or (2) a pH-neutral concrete is used, or (3) a glass fiber that is really resistant to alkali attack is developed (otherwise the allowable working stress should be lowered).

ALKALI RESISTANCE OF CARBON FIBERS

With regard to environmental interaction, carbon fibers are not affected by moisture, atmosphere, solvents, bases and weak acids (ASM, 1987). In terms of their specific resistance to alkalis, Judd (1971) showed that carbon was resistant to alkaline solutions at all concentrations and all temperatures up to boiling. Carbon tows immersed for 257 days in a very basic 50% w/v sodium hydroxide solution showed variations in strength and elastic modulus only around 15%. Carbon strands soaked for 9 months in a pH 13 solution (at 60°C) showed no variation in strength nor modulus (Santoh et al., 1993a, 1993b). Beams prestressed with carbon strands and subjected to wet/dry cycles in an alkaline solution showed no degradation of flexural strength after 9 months (Arockiasamy, 1995). In summary, carbon fibers have the potential to withstand direct interaction with concrete for long periods of time.

ALKALI RESISTANCE OF ARAMID FIBERS

Para-aramid fibers (such as Kevlar) are fairly resistant to many solvents and chemicals, but are affected by strong acids and bases (ASM, 1987). Kevlar 29 exposed to a 10% sodium hydroxide solution for 1000 hours loses 74% of its strength. Higher modulus Kevlar 49 is much more resistant but still shows some strength loss in an alkaline environment (3% loss in a 40% sodium hydroxide solution after 100 hours (DuPont, 1992). Due to the protection provided by the resins, aramid composite bars show smaller losses (from 2% to 10%) due to exposure to sodium hydroxide solutions (Noritake et al., 1993; Tamura, 1993). The estimated 100-year sustained strength of an aramid rod (Arapree) decreased from 60% in air to 50% of the short-term strength in an alkaline environment (Horn et al., 1977; Gerritse et al., 1992, 1995). Separate tests on the same aramid bar showed sustained to short-term strength ratios of 75%, 70%, 60% and 50% for exposures to 20°C air and 20°C, 40°C and 60°C alkaline environments, respectively (at 10,000 hours) (Scheibe and Rostasy, 1995).

In summary, aramids will show strength decay when in contact with concrete. Higher modulus aramids exhibit better alkali resistance and should be the ones used in such applications.

ALKALI RESISTANCE OF GLASS FIBERS

Glass fibers are chemically vulnerable to many acids and bases and will deteriorate if in direct contact with concrete. In composites, it is generally expected that the matrix will provide the needed chemical protection for the fibers. However this may not always be the case.

Concrete environment

As mentioned earlier, the phenomenon of creep rupture, or static fracture, is the gradual reduction of the tensile strength of glass under stress. This problem is accelerated significantly in the presence of water, acids and alkalis, and may result in sudden cracking of the fibers. This was emphasized in a paper summarizing the findings from a symposium on durability of glass fiber reinforced concrete: glass fiber composites can exhibit "spontaneous loss over time of much of the flexural and tensile strengths of the composite to little more than that of the matrix" in case of exposure to wet conditions in an alkaline environment (Diamond, 1985).

The resins used in the composite must provide a two-fold protection in a concrete environment: the matrix toughness must be high enough to prevent the development of matrix microcracks, and diffusion through the matrix must be minimal.

Fujii et al. (1993) tested E-glass composites with a relatively brittle polyester matrix. These composites showed significant matrix microcracking when loaded to only 40% of their short-term ultimate strength. The matrix microcracks effectively would allow for direct contact between fibers and concrete. This microcracking resulted in a significant loss of tensile strength (more than 50% in 720 hours) when the composite was immersed in an acidic solution. The same tests under the same conditions but with a tough vinylester resin resulted in no matrix cracking and no strength loss. Although currently used resins exhibit elongations at failure of up to 4%, tougher resins with elongations of 10% or more can be obtained.

Sen et al. (1993a, 1993b) tested 12 beams pretensioned with two 3/8-inch, seven rod fiberglass strands. The strands were made of S-2 glass fibers in a Shell Epon 910 epoxy resin, and were stressed at about 50% of short-term ultimate. Five beams were precracked, and a total of nine were exposed to simulated tidal cycles in a 15% sodium chloride solution. Three of the five precracked beams failed at a load lower than the cracking load, indicating a total loss of the fiberglass strands after less than 9 months of exposure. One of the uncracked beams failed without the application of any external load (exposure time 18 months). Scanning electron microscope examination of the strands showed the fiber deterioration for the exposed specimens. This total loss of the strands is a concern, particularly since S-2 glass fibers are more resistant than E-glass fibers to alkalis and the matrix used is also one of the most resistant to basic environments. Scanning electron microscope examination of the strands showed that no matrix cracking was apparent. Damage was attributed to diffusion of the hydroxil ions through the matrix, indicating that, in some cases, even an uncracked matrix of the best type available may not be sufficient to protect the fibers.

Tests by Dolan et al. (1997) also show long-term strengths (at 5500 hours and for GFRP tendons embedded in concrete) of about 55% of short-term ultimate.

Alkaline solutions

Alkaline solutions are often used to evaluate the fiber protection provided by the matrix. This evaluation may be somewhat conservative, as shown in the first case below, but it emphasizes the importance of this problem with GFRP.

The Federal Highway Administration sponsored an evaluation of GFRP gratings for use in concrete bridge decks (Anderson et al., 1994). Tests were conducted in an alkaline environment on two sets of gratings using E-glass fibers, one set with a polyester matrix, another with a vinylester matrix. Polyester resins are less resistant to alkali attack and showed very rapid deterioration when compared to the vinylesters. Three-point bending tests on the grating samples after 160 days showed strength reductions of up to about 80% for polyester, and up to 25% for vinylester. Tests of the gratings embedded in small concrete beam specimens showed significant degradation for both polyester and vinylester gratings, with 30% to 40% loss of ultimate strength after 168-day exposures.

Tests by Katsuki et al. (1995) for GFRP rods in a NaOH solution (1 mol/liter at 40°C) showed strength losses of 70% after 120 days. Hou and Martine (1996) tested GFRP bars in a basic solution with pH of about 13. They reported losses in flexural strengths of 7% to about 30% for the vinylester bars (depending on the sizing) after 90 days. They also tested polyester bars that showed significantly higher degradation.

Tests at Iowa State University used accelerated aging techniques to determine the long-term strength of GFRP composites (Porter et al., 1996a, 1996b). The accelerated aging procedure involved exposing specimens to an alkaline solution at high temperature (up to 140°F) for 2 to 3 months, simulating about 50 years of exposure to real weather. Tensile tests on 3 rebar types indicated remaining strengths of 34%, 52%, and 71% of the measured short-term strengths.

Tests by Sen et al. (1996a) for the Navy and U.S. Army showed that rebars in concrete exposed to a 13.5 pH solution would lose 70% of their strength after 9 months under long-term stresses equal to 10% of the static strength. They state that "it would be unwise to use [current E-glass rebar] as reinforcement for structural concrete members."

OTHER ENVIRONMENTAL EFFECTS

Effect of water

Carbon fibers are not affected by water. However, the matrix is usually affected, and consequently so are the composite properties. For unidirectional carbon composites this usually translates into a reduction of the compressive and shear strengths, but a small effect on the tensile strength (Ciriscioli, 1988; Sen et al., 1996c). Graphite composites used as bonded external reinforcement in beams and subjected to 100 freeze-thaw and wet-dry cycles showed little effect on the composite itself, but some loss of the composite-to-concrete adhesion (Chajes et al., 1994; Karbhari and Engineer, 1996).

Para-aramid fibers (such as Kevlar) absorb and are affected by water, mostly at higher temperatures (Dolan, 1993; Horn et al., 1977). Saturated aramid composites have been reported to lose 35% of their flexural strength at room temperature (Allred, 1984), and up to 55% if stressed and under wet/dry and thermal cycles (Sen et al., 1996b).

Glass fibers are also affected by moisture. Typically, losses in tensile and flexural strengths of 10% or more may be expected after a few months of exposure (Springer et al., 1981; Novinson et al., 1998; Faza et al. 1994; Pantuso et al., 1998), although some studies indicate that the losses may be negligible (Rahman et al., 1996).

UV effects

Aramids are most vulnerable to ultraviolet (UV) attack. A thin Kevlar 29 fabric exposed to Florida sun for 5 weeks lost 49% of its strength [48]. However, a thick (1/2 inch) rope lost 31% of its strength after 24 months due to the protection of the inner fibers by the outer ones (DuPont, 1992). Resins, in general, will be affected by UV unless adequate protection is provided by additives or coatings. In turn, the composite properties would also be affected, mostly in compression, shear, and transverse tension.

DRAFT CODES IN CANADA, JAPAN AND THE U.S.

Canada, Japan, and the U.S. have issued draft codes for the design of reinforced and prestressed concrete structures (Canadian, 1996; CHBDC, 1996; Japanese, 1995a, 1995b; Sonobe et al., 1997; Gilstrap et al., 1997; ACI 440, 1996; ACI 440, 1997).

The Canadian Standards Association (1996) is in the process of amending the Canadian Highway Bridge Design Code to include Chapter 16: "Structures with Fibre Reinforcement". In this chapter, composite applications are limited by fiber type. These limitations are necessary due to the potentials problems described earlier.

In the Canadian Code, the use of GFRP was proposed to be restricted as follows:

- GFRP rod can be the sole primary tensile reinforcement only for barrier walls and for the interior panels of deck slabs of slab-on-girder bridges
- If GFRP is used as primary tensile (or shear) reinforcement, then CFRP, AFRP or steel must be used to withstand the unfactored dead loads
- GFRP tendons shall be used only when not in direct contact with concrete
- GFRC (glass fiber reinforced concrete) shall not be used
- Stresses in GFRP tendons at transfer are limited to 48% of short-term ultimate

The use of AFRP and CFRP was restricted as follows:

- Stresses in AFRP tendons at transfer are limited to 35% of short-term ultimate
- Stresses in CFRP tendons at transfer are limited to 60% of short-term ultimate

In addition, all FRP should be protected against UV rays.

PROPOSED ALLOWABLES FOR DESIGN WITH FRP BARS

To arrive to an allowable design stress the Canadian draft code proposes limiting the factored loads to ϕF times the ultimate strength for FRP rebars. Using the most conservative data presented here, F should be 0.32, 0.5 and 0.8 for GFRP, AFRP and CFRP, respectively (Table 1, Proposed F). For ratios of dead load to live load greater than 2, the Canadian draft code suggests values of F of 0.7, 0.5 and 0.9 (for mostly dead load, these values were shown as 0.6, 0.45, and 0.75 in a draft). It is estimated that the value for GFRP may be too liberal, given the failure studies previously mentioned. For tendons, the Japanese code suggests a factor β_3 of 0.7 for all three FRP types.

The Canadian draft code also proposes a resistance factor ϕ of 0.9. This resistance factor is a function of the variability of the rebar strength. This seems also liberal since (1) tests on some of these rebars have shown coefficients of variation in strength in excess of 20% (Malvar, 1995), and (2) the product ϕF provides a very low safety factor when compared to working stress design with steel rebars. The Japanese draft code recommends ϕ factors of 0.77 for glass, 0.87 otherwise. Hence it is proposed that ϕ could be taken around 0.8 (Table 1).

The Eurocode draft (Gilstrap et al., 1996) uses safety factors of 3.3, 2.0, and 1.67 for GFRP, AFRP and GFRP, respectively. An equivalent ϕF factor could be taken as the inverse of these safety factors.

Hence, it is proposed that the ϕF ratio of allowable stress to ultimate strength may be chosen as 0.25, 0.40 and 0.64 for GFRP, AFRP and CFRP, respectively (Table 1). Note that (a) these values are close to the most conservative of all code values, and (b) these values should be further reduced for direct exposure to concrete, water, and U.V.

For tendons, the stresses at transfer, which would be long-term sustained stresses, could be limited to similar values. Further limits at jacking and ultimate should be determined, which is beyond the scope of this work.

Table 1. Proposed Allowable to Ultimate Stress Ratio ϕF for FRP Rebar Design.

FACTOR	SOURCE	GFRP*	AFRP*	CFRP
F	Canadian draft, $R \geq 2$	0.7	0.5	0.9
	Canadian draft, $R \to \infty$	0.6	0.45	0.75
	Japanese draft, tendons, β_3	0.7	0.7	0.7
	Proposed	0.32	0.5	0.8
ϕ	Canadian draft	0.9	0.9	0.9
	Japanese draft	0.77	0.87	0.87
	Proposed	0.80	0.80	0.80
ϕF	Canadian draft, $R \geq 2$	0.63	0.45	0.81
	Canadian draft, $R \to \infty$	0.54	0.40	0.68
	Japanese draft	0.54	0.61	0.61
	Eurocode 1/(safety factor)	0.30	0.50	0.60
	Proposed	0.25*	0.40*	0.64

* These values should be further reduced for direct exposure to concrete, water, and U.V.

CONCLUSIONS

Durability issues of some currently available FRP products limit their use for some structural applications. Long-term strength in the absence of detrimental environmental factors have been measured which are well below the short-term strengths. The presence of water, acids, alkalis, and UV, can further reduce the long-term strengths significantly. The following recommendations are only suggested interim guidelines until formal design criteria is developed. Further limitations may be necessary for allowable stresses and usage.

The use of GFRP composites should be very restricted when in direct contact with concrete, at least until further durability guarantees are provided. It is further recommended that:

- GFRP tendons should not be used in direct contact with concrete;
- GFRP allowable working stresses for bars (and permissible stresses at transfer for tendons) should be limited to 25% of the measured ultimate tensile strength;
- GFRP bars and tendons should not be used as the sole primary reinforcement, except in applications where the reinforcement is not subjected to sustained load;
- GFRP reinforcements should be permitted for secondary reinforcement, such as column transverse reinforcement and passive column wraps;

The use of aramid composites should be allowed with restrictions, such as:

- AFRP allowable working stresses for bars (and permissible stresses at transfer for tendons) should be limited to 35 % of the measured ultimate tensile strength;
- AFRP are not recommended for waterfront applications;
- High modulus AFRP reinforcements are better suited for exposure to concrete.

The use of carbon composites should be allowed with some restrictions, such as:

- CFRP allowable working stresses for bars (and permissible stresses at transfer for tendons) should be limited to 60 % of the measured ultimate tensile strength.

In addition, all FRP reinforcements should be protected against moisture and UV, and the use of pH-neutral concretes is strongly recommended to prevent alkali attack.

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Port Hueneme, California 93043-4370

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EFFECT OF MOISTURE AND TEMPERATURE OF CEMENT MORTAR SURFACES ON QUALITY OF ADHESIVE BOND

An Investigation Conducted By

Narayan R. Joshi, Ph.D.
Department of Civil Engineering
Prairie View A & M University
Prairie View, TX 77446

September 1998

EXECUTIVE SUMMARY

Carbon fiber reinforced plastics (CFRPs) are receiving greater utilization in many structural applications due to their excellent mechanical and corrosion resistance characteristics. In addition, they can be designed and fabricated to get adequate strength and stiffness by changing the angle and the sequence of their laminations. The composite materials are increasingly used in industries of aerospace, automobile, ship, leisure and sports. Applications in strengthening existing reinforced concrete beams, piles, and decks using carbon fiber sheets are also being pursued. Carbon fiber sheets are utilized to strengthen waterfront concrete structures such as wharves and piers. The sheets are bonded to the under-side of reinforced concrete pier using structural adhesives. Structural bonding implies the use of adhesives in an engineering application. A bonded joint should be capable of withstanding the stresses to be transmitted under different environmental conditions and over many years of service life. In this study three structural adhesives, i.e. epoxy resin systems, were chosen to establish adhesive bonds to the hydraulic cement mortar cubes. The assemblies were subjected to the combinations of three different temperatures and four different relative humidities in an environmental control chamber. Effects of moisture and temperature on the quality of bonds were investigated using pull-off tests. Results are reported describing different types of failures observed.

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1. INTRODUCTION

Excellent mechanical and corrosion resistant characteristics have promoted the use of fiber reinforced plastics (FRP's) in many structural applications all over the world. Although the short-term mechanical properties of these materials are usually well documented, long-term durability issues still remain to be researched. The Naval Facilities Engineering Service Center (NFESC) is studying use of fiber reinforced plastics (FRP) to upgrade existing reinforced concrete piers and wharves. Carbon fiber reinforced plastic (CFRP) sheets are bonded to the under-side of reinforced concrete pier decks to increase their structural capacity, in particular to resist large patch loads from mobile crane outrigger floats. Laboratory tests of beam, slab, and pile scale models were conducted to quantify upgrades of moment capacity, shear strength, deflection, and ductility and to determine effects on failure modes. The composite plates and composite fabrics epoxy-bonded to the concrete structure for the purpose of external reinforcement showed great improvement in the performance of the host structure [1]. However, there is not enough information regarding environmental effects on the quality and durability of the bond between the reinforcement and the host structure.

2. ADHESIVE BONDS

At present two methods are used to reinforce the host concrete structure by using the carbon fiber reinforced plastics (CFRP). In one method the pre-impregnated plates are bonded to the concrete surface using a structural adhesive such as epoxy resins. In this case the structural adhesive may or may not be different from the polymeric matrix of the composite plate. In the second method, the dry fibrous reinforcement is impregnated with a resin system during placement itself. The composite is thus formed at the same time as it bonds to concrete. The resin thus serves the dual purpose of impregnating and bonding the fibers together, and bonding the composite to the concrete surface. This second method is therefore called a wet lay-up.

Structural bonding implies the use of adhesives in an engineering application. It is expected that the proposed bonded joint will be capable of withstanding the stresses to be transmitted. These stresses will be applied either in tension or shear, or in combination. The adhesive performance at various temperatures has also been studied using double lap joints. It was observed in the case of the adhesive Araldite that there is in general loss of room temperature strength for any gain in high temperature performance [2]. Many adhesive systems such as alkyd, epoxy ester, chlorinated rubber and styrene-butadiene paints showed loss of adhesion in 100% relative humidity (RH) conditions at room temperature. At lower RH values the loss of adhesion effect was less marked [3]. In case of epoxy adhesives it was observed in earlier research investigations that both with polar and non-polar surfaces the properties of the adhesive joint are determined by an interaction of surface properties, adhesive properties and process variables. Porosity plays the significant role in case of porous materials like cement concrete or mortar. Flaws at the weak interfacial layer can cause poor adhesion [4]. The adhesive bond line thickness also affects the tensile strength. In general it was observed that the thicker the bond line, the lower the tensile strength [5]. The aspect ratio of the adhesive film is defined as the geometric ratio of its bonded area to force-free lateral surface area. Thin adhesive films have this ratio greater than 100. A thin adhesive film cannot contract laterally as a result of the geometric restriction. It has a lower Poisson's ratio than a thick film. The tensile bond strength depends on the aspect ratio [6].

3. SCOPE OF THE PRESENT STUDY

It is proposed to limit the present project to the study of the environmental effects (effects of different temperatures and humidities) on the quality of bonds to the hydraulic cement concrete or mortar surface. Out of many available resin systems such as unsaturated polyesters, vinylesters, phenolic resins and epoxies, only epoxies were chosen for the project. This is so because polyester resins are not very resistant to alkalies and are typically avoided for uses in concrete [7]. Vinylester resins are resistant to a wide range of acids as well as to chloride salts making them ideal for marine environments. However, the volume shrinkage of vinylester resins upon cure is significantly higher than for epoxy resins [8]. Phenolic resins have not been used to a wide extent in fiber reinforced composites. They are highly aromatic materials and have very short distances between crosslinkable points. Thus, their cured networks are relatively brittle. Moreover many of them have large volumes of volatiles upon cure [9]. On the other hand, epoxies have many desirable properties as structural adhesives. They cure with only a fraction of the shrinkage of vinyl-type adhesives such as polyesters and acrylics. Consequently, less strain is built into the bond line, and the bond is stronger. They exhibit low creep. The epoxies in the unmodified state cure without releasing water or other condensation byproducts. They are resistant to moisture. Moisture does not affect epoxy but will migrate through the joint and deteriorate the substrate.

Epoxy adhesives are particularly compatible with Portland cement concrete or mortar because of the insensitivity to the alkali and moisture contents of this structural material [10]. It was observed that when epoxy bonded joints are subjected to moisture or water immersion, the failures usually occur at the interface. This indicates the importance of proper surface preparation of the adherends. However, it opens the area of research where there is either no opportunity or limited opportunity to prepare the surface of one of the adherends, say, that of the concrete structure. It was therefore decided to undertake an investigation of the effects of dry and moisturized mortar surfaces on the quality of the adhesive bond. It was proposed to vary both temperature and moisture content of cement mortar cube surfaces by conditioning them in an environmental control chamber.

4. METHODS OF TESTING OF ADHESIVE BONDS

A wide range of approaches is available for estimating the strength of concrete or mortar. These include destructive methods such as cube or cylinder compression to failure testing, nondestructive methods and partially destructive tests. In the last category is included a test called the pull-off test developed originally by A. E. Long [11]. Figure 1 shows schematically the simple arrangement of the pull-off test. It involves bonding a circular steel or aluminum probe to the concrete surface using an epoxy resin adhesive which is stronger than concrete in tension. An increasing tensile load is applied to the disc by means of a portable hydraulic jack. Since the tensile strength of the epoxy resin is greater than that of the concrete, the latter will fail. From the peak load applied to pull off the disc from the concrete, a nominal (engineering) tensile strength of the surface is calculated dividing the force (load) by the area of the disc [12]. The ASTM Standard D4541-93 describes a similar partially destructive test called 'Pull-off test for Adhesion' [13]. It is designed not for the estimation of the tensile strength of the substrate but for the estimation of adhesive strength of a coating to the substrate. In this test an aluminum dolly in place of a steel disc is glued to a coating of the substrate. An instrument called Elcometer 106 Adhesion Tester is then used to pull off the glued dolly normal to the glued surface [14]. In the list of physical/chemical properties of coatings, the bond strength is typically referred to in terms such as 'greater than 200 psi' etc..

Figure 2 shows schematically different types of expected failures of an assembly. Failures are classified as either substrate, adhesive, cohesive or mixed. In the adhesive failure, the adhesive layer separates from the substrate. In the cohesive failure, the adhesive layer breaks into two portions, one remaining attached to the substrate (mortar cube in this case) and the other attached to the dolly. We expected higher number of substrate failures and negligible cases of cohesive failures in this work. In the case of a mixed failure, the percentages of the substrate and adhesive failures will be estimated by careful visual observation using a magnifying glass [15]. This new version of the original 'Pull-off Test' thus can be used for different purposes. It can be used to measure tensile strength of a cement concrete or mortar as in the original version, or the adhesion strength of a coating on a concrete surface, or the adhesive strength of the glue used to attach the aluminum dolly to the plain surface of a concrete. The test could also be used to study the effect of contaminated concrete surface on the adhesion of a coating to the surface [16]. Contaminants at the concrete surface affect osmotic pressure and adhesion. Different versions of the pull-off test are useful for in-situ testing of concrete specimens of big size. For small size specimens of cement concrete or mortar cubes of 2 inches in dimension, a tensile testing machine like Instron or MTS, could be used to get data more accurately than that could be read from the Elcometer instrument primarily designed for paint and coating testing.

5. EXPERIMENTAL SETUP

It was decided to use the "Pull-off Test" for the purpose of studying the effects of the moisture on the quality of adhesive bonds to cement mortar surfaces. The test specimens prepared were the assemblies of mortar cubes, epoxy resin and sand blasted aluminum dollies as shown in Figure 3. Three different temperatures (70°F, 85°F and 100°F) and four different relative humidities at each temperature (50% RH, 65% RH, 80% RH and 95% RH) were chosen for the study. About 108 mortar (Portland cement + sand + water) cubes of dimension 2 x 2 x 2 in were cast using plastic molds (impermeable formwork of polypropylene plastic material). The sand to cement ratio of the mortar was 3:1 and the water to cement ratio 0.5. The cubes were kept moist for 28 days. After that period they were kept in the laboratory at ambient temperature and humidity. On one of the smooth faces of each mortar cube an aluminum dolly was attached using the structural epoxy adhesive. A dolly resembles an hourglass in shape, about half an inch tall, and three quarter inch in diameter. Figure 3 shows the dimensions of the cement mortar specimens used in the experimental work.

The mortar cube surfaces were wire brushed and then dusted off with a soft brush making them free of laitance and contamination. The surface abrasion of the dollies helped establish the proper mechanical anchoring to the mortar surface through a thin film (less than 1/16 in thick) of an epoxy resin. It was decided to use three different epoxy resins - Sikadur 30, Sikadur 32 Hi-Mod, and Madewell 1312F to glue the aluminum dollies to the brushed surfaces of the cubes. All three epoxy resins are two-component systems. They were mixed and stirred according to specifications recommended by their manufacturers. These are listed in the Appendix. For each cube, two dollies were attached one on the surface coated with a thin layer of a primer and other on a surface without primer. The primer Sikadur 55 SLV was used with resins Sikadur 30 and 32. The primer Madewell 927 was used with the resin Madewell 1312F. Their specifications are also included in the Appendix. A primer works as a penetrant as well as sealer. It was expected to fill micropores in concrete or mortar and not air bubbles or blow holes. Only a thin layer (1 mil or 2 mils) of primer is supposed to perform its expected function. The thickness of a dolly and a cube pair were measured separately using vernier caliper. Their masses were also measured.

Dollies were mounted on the surfaces of the mortar cubes using thin layers of epoxy resins and were held in position for a couple of minutes with a constant pressure. Three samples for each epoxy system were prepared. So for a chosen pair of temperature and relativity humidity, 9 samples, three each for three epoxies, were prepared each carrying two dollies, one on a surface with primer and another on a surface without primer. For four different relative humidities at the same temperature, a total of 36 samples was conditioned. For three different temperatures, the total added to 108 samples. The tensile tests performed were similar to the ASTM Standard D4541-93-Pull-Off Strength of Coatings. Instead of using a portable Elcometer 106 Adhesion Tester, the pull-off tests were carried out by using a computerized Instron Testing Machine. A special fixture was prepared to align the mortar cube with a dolly along the vertical axis of the tensile loading machine so that the applied load was normal to the cube surface. Misalignment was avoided because it can change a tensile test into a peel-off test.

Each set of 9 cubes was cured for 7 days in the environmental control chamber set at the chosen values of temperature and relative humidity. The procedure was repeated for 12 sets. After 7 days, assemblies were taken out of the chamber. The total thickness of each assembly was measured using the vernier caliper. By subtracting the individual thickness of the dolly and the cube from the thickness of the assembly, the thickness of the epoxy layer was computed. The excessive epoxy layers around the base of the attached dollies were removed using the testing apparatus sharp hollow cylindrical scraper before the pull-off tests. The assemblies were pulled off using the special fixture (Figure 4) attached to the Instron testing machine Model 4206. The elongation rate for all 216 tests was fixed at 0.01 inch per minute. A 1000 lb capacity load cell of was used. The load range of 500 lb (on the Y-axis) and the elongation range of 0.5 inch (on the X-axis) were set for all tests.

6. EXPERIMENTAL DATA

A computer equipped with a data acquisition system acquired load and elongation values during each test and plotted them on the screen. For each test, peak tensile load to pull-off the dolly was recorded. After the break of the assembly, the dolly with the pulled-off portion of substrate was weighed again to calculate only the detached mass of the substrate. The combined thickness of the dolly and the mortar chunk at the point of the deepest extent was measured to calculate the maximum depth of the mortar chunk. Viewing through the magnifying glass, the percentages of the mixed failures were estimated. Thus for each test, the mass of the mortar cube, the peak load to pull off the dolly, the adhesive film thickness, the pulled-off mass of mortar layer, maximum depth of the detached mortar layer and types of failures that occurred were recorded.

For the chosen pair of the temperature and relative humidity, six tests were run for each of three epoxy resins. This group of 18 tests is called Set #1. For three different temperatures and four different RH values, a total of 216 tests were performed grouped among twelve sets. In a given set, for each epoxy there were three tests for dollies attached to surfaces with no primer and another three to surfaces with primer. On each cube two dollies were attached, one on the surface as it was and the other on the surface carrying a thin coating of the primer. The twelve sets thus constituted the initial direct data from physical measurements and testing. Later on, the average peak load was calculated from three tests on the surfaces with no primer. The procedure was repeated for three tests on surfaces with primer. The data from twelve sets from 1 to 12 are presented in Tables 1 to 12.

The following abbreviations were used in the test names.

1) In Set #1, the nine tests are named N11 to N19. In Set #8, the tests were named N81 to N89 and so on. Thus the first test was N11 and the last was N129 in Set #12, adding to 108.

2) The suffix 'P' indicates a test on a surface with primer. These tests are named N11P to N19P. Thus the first test was N11P and the last was N129P in Set #12, adding to 108.

3) The tests in which the pulled-off mortar mass was found to be equal to, or greater than 1.5 grams are identified with the star (*) at the end of their names.

4) The tests in which the mounted dollies slid down from the original location but stayed glued on the surface are identified with the suffix 'SL".

5) When the dollies did not stick at all to the surface or were pulled off with a peak load less than 25 pounds, the suffix 'X' was attached to their test names. The value of zero was entered for the peak load. The peak load of such test was excluded from the average.

6) The suffix 'R" was attached to those tests which were repeated.

7. REGROUPING AND PLOTTING OF DATA

In order to study the performance of the three epoxy resins separately, at different combinations of temperatures and relative humidities, new tables were constructed for each one of them. Thus Table 13 describes the performance of the adhesive Sika 30 at a temperature of 70°F, and at different relative humidities. Table 14 does the same at 85°F and Table 15 at 100°F.

From Table 13, three new data tables were constructed. In Table 13A, tests with primer are separated from those without primer. In Table 13B, the average of three peak loads is calculated for a single relative humidity (RH%) value. The procedure is repeated for adhesive film thickness, pulled off mass of mortar layer, and its thickness. Masses of individual cubes were also entered into tables. Their averages were not performed since they carry no significance for this study. In Table 13C, only average values of peak loads were listed. Thus two average peak load values (one with primer and other without primer) corresponded to a single pair of temperature and relative humidity values. These are used in plotting of experimental data for the purpose of comparing performance of the three different epoxy systems.

The procedure was repeated for the adhesive Sika 32 in Table 16 (with 16A, 16B and 16C), Table 17 (with 17A, 17B, and 17C) and Table 18 (with 18A, 18B, and 18C). One more repetition is performed for the adhesive Madewell 1312F in Table 19 (with 19A, 19B, and 19C), Table20 (with 20A, 20B, and 20C), and Table 21 (with 21A, 21B, and 21C).

The regrouped data of the epoxy Sika 30 from Tables 13C, 14C and 15C were plotted in Figures 5, 6, and 7. The regrouped data of the epoxy Sika 32 from Tables 16C, 17C and 18C were plotted in Figures 8, 9, and 10. The regrouped data of the epoxy Madewell 1312F from Tables 19C, 20C, and 21C were plotted in Figures 11, 12 and 13.

Data on types of failures from the last columns of Set #1 to Set #12 were regrouped in Table 22 to study their distributions at different temperatures and relative humidities irrespective of the epoxy resins used. The regrouped data from Table 22 were plotted as three different bar graphs, one for each temperature, in Figure 14 for 70°F, in Figure 15 for 85°F and in Figure 16 for 100°F.

8. EXPERIMENTAL RESULTS

8.1. Adhesive Sika 30

Figures 5, 6 and 7 show the behavior of the adhesive Sika 30 at three different temperatures. On each plot, there are two curves of the average tensile peak loads (ATPL), one for no primer and the second for with primer. Each point is the average of three tests. Curves are drawn through four data points at four different relative humidities.

8.1.1. Without primer

The maximum ATPL is 254.83 lbs at 70°F and 50% RH. The minimum ATPL is 121.8 lbs at 85°F and 50% RH. In general, the ATPL have decreased in going from lower temperature to the higher temperature.

8.1.2. With primer

The maximum ATPL is 284.53 lbs at 70°F and 50% RH. The minimum ATPL is 129.16 lbs at 100°F and 50% RH. In general, the ATPL have decreased in going from lower temperature to the higher temperature.

Both the maximum and minimum ATPL have values higher in tests with primer than those in tests without primer. However the curve of ATPL with primer is not above the curve without primer at all temperatures and relative humidities as expected.

8.2. Adhesive Sika 32

Figures 8, 9, and 10 show behavior of the adhesive Sika 32 at three different temperatures. On each plot there are two curves of ATPL, one for tests with primer and the other for tests without primer. Each point is the average of three tests. Curves are drawn through four data points at four different relative humidities.

8.2.1. Without primer

The maximum ATPL is 381.4 lbs at 85°F and 80% RH. The minimum ATPL is 122.7 lbs at 100°F and 95% RH. In general, the ATPL have values higher at 85°F than those at the lower and the higher temperatures.

8.2.2. With primer

The maximum ATPL is 380.8 lbs at 70°F and 50% RH. The minimum ATPL is 222 lbs at 100°F and 95% RH. In general the ATPL have increased in going from the lower temperature to the higher temperature.

Both the maximum and minimum ATPL have values higher in tests with primer than those in tests without primer. The curve of ATPL with primer is most of the time at a higher load level than the curve without primer, as expected.

8.3. Adhesive Madewell 1312F

Figures 11, 12, and 13 show behavior of the adhesive Madewell 1312F at three different temperatures. Each plot shows two curves of ATPL, one for tests with primer and other for tests without primer. Each point is the average of three tests. Curves are drawn through four data points at four different relative humidities.

8.3.1. Without primer

The maximum ATPL is 360.7 lbs at 100°F and 50% RH. The minimum ATPL is 118.5 lbs at 100°F and 95% RH. In general, the ATPL have increased in going from the lower temperature to the higher temperature.

8.3.2. With primer

The maximum ATPL is 436.3 lbs at 100°F and 50% RH. The minimum ATPL is 235.6 lbs at 70°F and 65% RH. In general, the ATPL have increased in going from the lower temperature to the higher temperature.

Both the maximum and the minimum ATPL have values were higher in tests with primer than those in tests without primer. The curve of ATPL with primer is most of the time at a higher load level than the curve without primer, as expected.

8.4. The Range of Average Tensile Peak Loads

Among all three epoxy systems (resins with primers), the highest ATPL is 436.3 lbs for Madewell 1312F with primer at 100°F and 50% RH.

Among all three epoxy systems, the lowest ATPL is 118.5 lbs for Madewell1312F without primer at 100°F and 95% RH.

Table A: Maxima and Minima in ATPL for Three Epoxy Resin Systems

EPOXY RESIN	PRIMER	ATPL (LBS)	NATURE	TEMPERATURE (°F)	RH%
Sika30	No	254.8	Max	70	50
Sika30	Yes	284.5	Max	70	50
Sika30	No	121.8	Min	85	50
Sika30	Yes	129.2	Min	100	50
Sika32	No	381.4	Max	85	80
Sika32	Yes	380.8	Max	70	50
Sika32	No	122.7	Min	100	95
Sika32	Yes	222.0	Min	100	95
Madewell	No	360.7	Max	100	50
Madewell	Yes	436.3	Max(highest)	100	50
Madewell	No	118.5	Min(lowest)	100	95
Madewell	Yes	235.6	Min	70	65

Out of 6 maxima of ATPL, 5 occurred at 50% RH, only one occurred at 80% RH and none occurred at the highest value 95% of RH used in the tests. Out of 6 minima of ATPL, 3 occurred at 95% RH, 2 occurred at 50% RH and only 1 occurred at 65% RH.

For the effect of temperature on ATPL, 3 maxima occurred at 70°F, 2 occurred at 100°F and only 1 occurred at 85°F. In the case of ATPL minima, 4 occurred at 100°F, 1 occurred at 85°F and 1 occurred at 70°F.

The diameter of a dolly was 0.787 inch (20 mm) and its area was 0.487 square inch. The greatest ATPL of 436.3 pounds is equivalent to 896 psi. The lowest ATPL of 118.5 pounds is equivalent to 243 psi.

The compression tests were run on three mortar cubes from the same stock from which 108 cubes were taken for the project. The average compressive strength was found to be 5000 psi. At this point it was found necessary to investigate the number of types of failures at different combinations of temperatures and relative humidities. This data is presented in Table 22.

8.5. Types of Failures

In the assembly of the layered structure of aluminum dolly, adhesive film and the substrate of the hydraulic cement mortar cube, the weakest link was expected to be the substrate. Not in a single case was there any separation between the aluminum dolly and the adhesive used.

Out of 216 tensile pull-off tests, the 100% substrate failures occurred in 127 tests, equivalent to 59 percent. The 100% substrate failure means that the whole area of the pulled off dolly was covered with substrate. In the case of 80% substrate failures, the remainder 20% would be adhesive failure or interface separation.

In addition:

- between 99% to 80% substrate failures occurred in 50 tests equivalent to 23 percent;
- between 79% to 60% substrate failures occurred in 13 tests equivalent to 6 percent;
- between 59% to 1% substrate failures occurred in 22 tests equivalent to 10 percent;
- one hundred percent adhesive failures occurred in 3 tests equivalent to 1.4 percent;
- only one cohesive failure occurred equivalent to 0.5 percent.

The highest number (18) of 100% substrate failures occurred at 70°F and 50% RH. The lowest number (3) of 100% substrate failures occurred at 100°F and 95% RH.

For all epoxy resin systems used and at all RH settings the highest number (51) of 100% substrate failures occurred at 70°F. That number was followed by the smaller number (44) at the increased temperature of 85°F, and finally by the smallest number (32) at the highest temperature of 100°F. The data from Table 22 was plotted separately in Figures 14, 15, and 16 as three bar graphs for three different temperatures.

8.6. Strong Bonds

In certain tests, although the type of failure was 100% substrate, the mass of detached substrate was a thin layer attached to the adhesive film below. In a few other tests, big chunks of substrate were pulled off by the dolly. The tests with the pulled-off mass of substrate greater than or equal to 1.5 grams were identified by the star (*) sign in Tables 1 through 12. They are identified as strong bonds. The 35 strong tests are listed below.

Table B: Tests with Detached Substrate Mass Equal to or Greater than 1.5 grams.

TEST	RESIN	PRIMER	PEAK LOAD (LBS)	MORTAR MASS (GR)	TEMP (°F)	RH (%)	TYPE OF FAILURE
N13	Sika30	No	275.2	2.639	70	50	100%S
N12P	Sika30	Yes	297.7	2.595	70	50	100%S
N13P	Sika30	Yes	332.9	3.087	70	50	100%S
N16	Sika32	No	452.6	3.530	70	50	100%S
N14P	Sika32	Yes	390.9	2.727	70	50	100%S
N18	MW	No	315.2	1.746	70	50	100%S
N18P	MW	Yes	329.4	2.097	70	50	100%S
N21P	Sika30	Yes	235	1.882	70	65	100%S
N26P	Sika32	Yes	436	1.814	70	65	100%S
N28P	MW	Yes	167.3	1.85	70	65	100%S
N29P	MW	Yes	233.6	2.103	70	65	100%S
N45	Sika32	No	242.5	1.887	70	95	100%S
N46	Sika32	No	200.7	1.998	70	95	100%S
N48P	MW	Yes	369	1.924	70	95	100%S
N55P	Sika32	Yes	252.8	1.686	85	50	100%S
N56P	Sika32	Yes	483.8	1.677	85	50	100%S
N61	Sika30	No	237.6	1.686	85	65	100%S
N66	Sika32	No	281.3	2.027	85	65	100%S
N65P	Sika32	Yes	303.2	1.697	85	65	98%S
N67	MW	No	277.9	2.105	85	65	100%S
N67P	MW	Yes	256	1.731	85	65	100%S
N75	Sika32	No	340.1	3.947	85	80	100%S
N75P	Sika32	Yes	439.2	2.135	85	80	95%S
N76P	Sika32	Yes	373.7	2.640	85	80	100%S
N78	MW	No	320.8	1.557	85	80	100%S
N84P	Sika32	Yes	318	2.227	85	95	100%S
N85P	Sika32	Yes	280.9	1.514	85	95	100%S
N86P	Sika32	Yes	270.6	1.697	85	95	100%S
N89P	MW	Yes	370.6	2.720	85	95	100%S
N94	Sika32	No	322.5	1.715	100	50	100%S
N95P	Sika32	Yes	319.5	2.075	100	50	100%S
N105	Sika32	No	276.9	3.110	100	65	100%S
N108	MW	No	320.4	1.565	100	65	100%S
N121	Sika30	No	176.5	1.628	100	95	100%S
N127P	MW	Yes	209	1.608	100	95	100%S

Out of 35 strong tests, 6 tests belonged to Sika 30, 18 belonged to Sika 32 and 11 belonged to Madewell 1312F. Tests with primer were 21, and without primer 14. The distribution at different temperatures was 15 tests at 85°F, 14 at 70°F and only 6 at 100°F. The distribution at different relative humidities was 11 tests each at 50% and 65% RH, 4 tests at 80% RH and 9 tests at 95% RH.

8.7. Weak Bonds

The peak pull-off load was less than 25 lbs in 6 tests out of 216 tests. The samples with improper adhesion, either due to the experimental error or due to other causes, were only 2.8 percent of the whole set. These were excluded from the average of the peak pull-off load. Out of 6 tests, 3 tests were on surfaces without primer and other 3 were on surfaces with primers. They are listed below.

Table C: List of Tests with Peak Pull-Off Load less than 25 lbs.

TEST	ADHESIVE	PRIMER	TEMPERATURE (°F)	RH (%)
N51	Sika30	No	85	50
N123	Sika30	No	100	95
N127	Madewell1312F	No	100	95
N32P	Sika30	Yes	70	80
N57P	Madewell1312F	Yes	85	50
N124P	Sika32	Yes	100	95

All three epoxies are involved. All three temperatures are involved. In the three cases with primers on the surfaces, thicker coats could be the cause of improper adhesion. In the three cases with RH 95% at 100°F, the moist surfaces could be cause of improper adhesion. Since they were excluded in calculating the average peak loads, their influence on the tests results is nil.

8.8. Effects of Adhesive Bond Thickness on Pull-Off Strength

Attention was given to apply a thin uniform layer of the adhesive to the sand blasted surfaces of the dollies first, and then set them on the mortar surfaces with primer and without primer in pre-selected positions. They were held in positions by moderate pressure. All three adhesives were in liquid paste form and hence got squeezed out from under the dollies. There was no control, therefore, on the thickness of the adhesive film between the dolly and the mortar surface. The adhesive film thickness were computed and are documented in Tables 1 to 12. It was observed that the resin Sika 30 formed films thicker than Sika 32 that in turn formed films thicker than Madewell 1312F. But the adhesive film thickness was also affected by the microscopic variations in the morphology of the individual mortar surfaces. No efforts, therefore, were made to establish the relationship between adhesive film thickness and the pull-off strength, although it was observed that the thicker films lead to the lower values of pull-off strengths in tests that could be reasonably compared with each other.

9. DISCUSSION OF RESULTS

Nine figures, from Figure 5 to Figure 13, are the plots of average tensile peak load (ATPL) versus relative humidity (RH) at 3 different temperatures for 3 epoxy resin systems. Among the resin systems, the curves of Sika 32 show ATPL, in general, at values higher than those of Sika 30, as expected. Between Sika 32 and Sika 30, the general trend can be explained by the difference in properties, as described by the manufacturer. The 7day cure tensile strength (ASTM D-638) of Sika 30 is 3600 psi, while that of Sika 32 is 5100 psi. Thus performance of Sika 32 was expected to be better than that of Sika 30. It was observed accordingly in this work. The data on the 7-day cure tensile strength of Madewell 1312F was not available.

The nine plots of Figures 5 to 13 show the improvement in the values of ATPL with the application of primer. It is interesting to note that the effect of primer becomes more dominant at higher temperatures for all resin systems, at all relative humidities. Primers may prevent outgasing from the substrate at higher temperatures, and thus improve the performance of the adhesive bond.

The bar graphs of Figures 14, 15, and 16 show trends in substrate and adhesive failures. The number of substrate failures decreased and that of adhesive failures increased in the environment of higher temperatures and relative humidities.

Among the strong tests, the number of those with primer is 1.5 times the number of those without primer. The number of strong tests decreased in the environment of higher temperatures and relative humidities.

Although the general trends observed in the averaged experimental data are consistent with properties of adhesives described by their respective manufacturers, deviations from the general trends were also observed in the individual data points. This may be due to certain chemical reactions (such as cross-linking) being accelerated at higher temperatures and certain other reactions being prevented from happening at a high relative humidity environment.

10. CONCLUSIONS

The total number of 100% adhesive failures was only 3 out of 216 tests. Out of 3, two occurred at 100°F and 95% RH, only one occurred at 85°F and 50% RH and none occurred at 70°F. Thus only 1.4% bonds turned out to be bad bonds.

It can be concluded, therefore, that all three epoxy resin systems performed their intended function of establishing a bond to the hydraulic cement mortar surfaces in the temperature range of 70°F to 100°F and in the RH% of 50 to 95. This happened 213 times out of 216, which is equivalent to 98.6% of the time.

The total number of mix failures (1% adhesive to 99% adhesive or equivalently 99% substrate to 1% substrate) was 85 out of 216 tests equivalent to 39 percent. It can be said therefore, that in 39% cases, bonds varied from good to fair to weak. The cases of 6 weak bonds are included in the mix failure category because they pulled out some substrate, albeit a small piece.

The total number of 100% substrate failures (better bonds) was 127 out of 216 equivalent to 59%. These cases do not tell us about the state of the bond unless we compare them on the basis of the quantity of mortar they pulled off. Out of these are selected 35 tests that pulled a mortar mass greater than 1.5 grams. These are the best bonds. The adhesive bond performance is summarized in the Table D.

Table D: Summary of Quality of Bonds

SERIAL NO.	BOND QUALITY	QUANTITY
1	Best	35
2	Good	92
3	Fair to weak	85
4	No bond	3
5	Cohesive failure	1
Total		216

11. RECOMMENDATIONS

In the research work reported here, there was no way to know the condition of the adhesive film formed under the dolly in contact with the mortar cube surface before subjecting it to a destructive test like the pull-off test. Sometimes a kissing bond is formed with intermittent contact with the substrate rather than a uniform bond. Sometimes the film becomes thicker on the edges. These non-uniformities in thickness and contact as well as gaps in the adhesive film can be detected prior to destructive testing by means of nondestructive techniques such as ultrasonics. This process will bring control on the formation of adhesive films. It will lead to the understanding of their true performance without being colored by the effects of the film defects.

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Table 1. Set #1 – Temperature 70°F, Relative Humidity 50%.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Adhesive Sika 30 without Primer				
N11	280.312	168.3	0.012	0.573	0.058	100%S
N12	279.058	321	0.008	0.716	0.104	100%S
N13*	275.824	275.2	0.012	2.639	0.199	100%S
		Adhesive Sika 30 with Primer Sika 55				
N11P	280.312	223	0.020	0.594	0.077	100%S
N12P*	279.058	297.7	0.010	2.595	0.179	100%S
N13P*	275.824	332.9	0.007	3.087	0.213	100%S
		Adhesive Sika 32 without Primer				
N14	270.399	372.1	0.002	1.008	0.084	100%S
N15	282.337	278.1	0.002	1.379	0.151	100%S
N16*	291.342	452.6	0.004	3.530	0.192	100%S
		Adhesive Sika 32 with Primer Sika 55				
N14P*	270.399	390.9	0.001	2.727	0.194	100%S
N15P	282.337	340.3	0.002	0.532	0.06	100%S
N16P	291.342	411.4	0.025	1.651	0.139	100%S
		Adhesive Madewell 1312F without Primer				
N17	281.962	212.6	0.005	0.609	0.104	100%S
N18*	291.696	315.2	0.001	1.746	0.159	100%S
N19	281.738	243.2	0.005	0.891	0.146	100%S
		Adhesive Madewell 1312F with Primer Madewell 927				
N17P	281.962	244.2	0.001	1.937	0.159	100%S
N18P*	291.696	329.4	0.008	2.097	0.183	100%S
N19P	281.738	183.4	0.002	0.982	0.119	100%S

Table 2. Set #2 – Temperature 70°F, Relative Humidity 65%.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Mass of Mortar Layer pulled off grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Adhesive Sika 30 without Primer				
N21	274.124	108.2	0.011	0.618	0.149	100%S
N22	264.988	280.9	0.005	0.771	0.234	90%S+10%A
N23	288.71	276.2	0.014	0.550	0.051	95%S+5%A
		Adhesive Sika 30 with Primer Sika 55				
N21P*	274.124	235	0.011	1.882	0.144	100%S
N22P	264.988	113.4	0.013	1.033	0.129	95%S+5%A
N23P	288.71	187.4	0.009	0.430	0.050	10%S+90%A
		Adhesive Sika 32 without Primer				
N24	287.947	276.2	0.010	0.638	0.100	100%S
N25	285.454	403.1	0.003	0.405	0.034	100%S
N26	286.765	359.9	0.008	0.636	0.044	100%S
		Adhesive Sika 32 with Primer Sika 55				
N24P	287.947	301.5	0.008	0.705	0.059	100%S
N25P	285.454	293.6	0.008	0.447	0.054	95%S+5%A
N26P*	286.765	436	0.004	1.814	0.165	100%S
		Adhesive Madewell 1312F without Primer				
N27-SL	286.433	149.4	0.005	0.497	0.091	90%S+10%A
N28	265.607	161.3	0.012	1.011	0.164	100%S
N29	276.982	267.2	0.006	1.114	0.168	100%S
		Adhesive Madewell 1312F with Primer Madewell 927				
N27P	286.433	306	0.006	1.448	0.129	100%S
N28P*	265.607	167.3	0.005	1.85	0.161	100%S
N29P*	276.982	233.6	0.007	2.103	0.159	100%S

Table 3. Set #3 – Temperature 70°F, Relative Humidity 80%.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Adhesive Sika 30 without Primer				
N31	279.708	223.4	0.010	0.320	0.097	20%A+80%S
N32	284.493	187	0.009	1.385	0.154	98%S+20%A
N33	275.726	167.9	0.003	1.119	0.124	95%S+5%A
		Adhesive Sika 30 with Primer Sika 55				
N31P	279.708	91.11	0.008	0.155	0.019	10%S+90%A
N32P	284.493	23	0.009	0.461	0.080	66%S+34%A
N33P	275.726	66.47	0.008	0.223	0.052	20%S+80%A
		Adhesive Sika 32 without Primer				
N34	286.264	268.7	0.002	1.121	0.105	100%S
N35	283.979	287.4	0.01	1.060	0.108	100%S
N36	285.642	221.2	0.003	1.409	0.130	100%S
		Adhesive Sika 32 with Primer Sika 55				
N34P	286.264	280.9	0.006	0.876	0.094	100%S
N35P	283.979	268.3	0.004	0.866	0.099	100%S
N36P	285.642	390.6	0.003	0.345	0.055	100%S
		Adhesive Madewell 1312F without Primer				
N37	280.439	184.3	0.005	0.731	0.088	100%S
N38	288.469	136	0.002	0.374	0.064	98%S+2%A
N39	270.004	128.5	0.002	0.845	0.151	95%S+5%A
		Adhesive Madewell 1312F with Primer Madewell 927				
N37P	280.439	261.2	0.004	0.876	0.109	100%S
N38P	288.469	298.5	0.004	0.809	0.084	100%S
N39P	270.004	194.2	0.004	1.221	0.117	100%S

Table 4. Set #4 – Temperature 70°F, Relative Humidity 95%.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Adhesive Sika 30 without Primer				
N41	285.319	336.8	0.014	0.282	0.052	15%S+85%A
N42	276.686	88.19	0.006	0.315	0.045	10%S+90%A
N43	290.384	143.2	0.019	0.975	0.109	95%S+5%A
		Adhesive Sika 30 with Primer Sika 55				
N41P	285.319	144.1	0.015	0.481	0.059	90%S+10%A
N42P	276.686	134.9	0.004	0.285	0.062	50%S+50%A
N43P	290.384	156.3	0.008	0.825	0.114	40%S+60%A
		Adhesive Sika 32 without Primer				
N44	286.217	355.8	0.004	0.521	0.053	100%S
N45*	279.680	242.5	0.002	1.887	0.149	100%S
N46*	273.337	200.7	0.003	1.998	0.219	100%S
		Adhesive Sika 32 with Primer Sika 55				
N44P	286.217	306.2	0.003	1.082	0.122	100%S
N45P	279.680	314.6	0.002	0.846	0.094	100%S
N46P	273.337	297.7	0.004	1.405	0.178	100%S
		Adhesive Madewell 1312F without Primer				
N47	278.662	200.7	0.009	0.892	0.118	96%S+4%A
N48	289.921	315.4	0.009	0.838	0.089	100%S
N49	281.559	229.8	0.005	1.422	0.169	100%S
		Adhesive Madewell 1312F with Primer Madewell 927				
N47P	278.662	304.2	0.005	1.272	0.132	100%S
N48P*	289.921	369	0.004	1.924	0.139	100%S
N49P	281.559	264.2	0.004	1.189	0.096	100%S

Table 5. Set #5 – Temperature 85°F, Relative Humidity 50%.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Adhesive Sika 30 without Primer				
N51X	280.141	0	0.016	0.866	0.118	85%S+15%A
N52	281.79	161.1	0.010	0.976	0.074	80%S+20%A
N53	276.78	82.5	0.030	0.978	0.059	70%S+30%A
		Adhesive Sika 30 with Primer Sika 55				
N51P	280.141	130.4	0.035	1.330	0.102	98%S+2%A
N52P	281.79	91.6	0.012	0.942	0.093	75%S+25%A
N53P	276.78	181.6	0.019	0.879	0.069	80%S+20%A
		Adhesive Sika 32 without Primer				
N54	281.776	332.8	0.013	0.725	0.042	100%S
N55	279.558	273.8	0.017	0.837	0.064	100%S
N56	288.795	317.7	0.003	1.147	0.112	100%S
		Adhesive Sika 32 with Primer Sika 55				
N54P	281.776	389	0.003	0.913	0.079	100%S
N55P*	279.558	252.8	0.005	1.686	0.099	100%S
N56P*	288.795	483.8	0.005	1.677	0.124	100%S
		Adhesive Madewell 1312F without Primer				
N57	287.163	406.7	0.004	0.561	0.062	98%S+2%A
N58	290.875	291.5	0.005	0.525	0.067	100%S
N59	288.471	170	0.005	0.396	0.043	100%S
		Adhesive Madewell 1312F with Primer Madewell 927				
N57PX	287.163	0	0.006	0.085	0.090	100%A
N58P	290.875	458.3	0.004	0.633	0.075	98%S+2%A
N59P	288.471	330.3	0.003	1.270	0.123	100%S

Table 6. Set #6 – Temperature 85°F, Relative Humidity 65%.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Adhesive Sika 30 without Primer				
N61*	292.429	237.6	0.007	1.686	0.173	100%S
N62	288.232	180.2	0.008	1.135	0.126	90%S+10%A
N63	282.199	280.9	0.016	1.157	0.141	100%S
		Adhesive Sika 30 with Primer Sika 55				
N61P	292.429	98.5	0.013	0.220	0.055	20%S+80%A
N62P	288.232	178.7	0.021	0.881	0.114	90%S+10%A
N63P	282.199	198.9	0.009	0.287	0.141	60%S+40%A
		Adhesive Sika 32 without Primer				
N64	291.673	394.6	0.002	1.032	0.114	100%S
N65	289.440	232.8	0.003	1.301	0.104	100%S
N66*	288.040	281.3	0.004	2.027	0.159	100%S
		Adhesive Sika 32 with Primer Sika 55				
N64P	291.673	312.6	0.006	0.907	0.083	90%S+10%A
N65P*	289.440	303.2	0.009	1.697	0.129	98%S+2%A
N66P	288.040	319.9	0.004	1.380	0.147	100%S
		Adhesive Madewell 1312F without Primer				
N67*	289.809	277.9	0.011	2.105	0.207	100%S
N68 SL	286.662	253.7	0.009	0.411	0.061	75%S+25%A
N69	292.428	223.2	0.004	0.922	0.124	100%S
		Adhesive Madewell 1312F with Primer Madewell 927				
N67P*	289.809	256	0.002	1.731	0.144	100%S
N68P	286.662	259.1	0.008	0.747	0.083	100%S
N69P	292.428	329.1	0.007	0.908	0.094	100%S

Table 7. Set #7 – Temperature 85°F, Relative Humidity 80%.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Adhesive Sika 30 without Primer				
N71	290.506	297.9	0.026	1.287	0.112	100%S
N72	289.865	133.7	0.011	0.535	0.097	80%S+20%A
N73	293.065	180	0.005	0.926	0.109	98%S+2%A
		Adhesive Sika 30 with Primer Sika 55				
N71P	290.506	91.4	0.008	0.245	0.056	20%S+80%A
N72P	289.865	57.72	0.017	0.420	0.069	25%S+75%A
N73P	293.065	299.9	0.029	0.814	0.097	100%S
		Adhesive Sika 32 without Primer				
N74	281.978	409.5	0.005	0.473	0.045	100%S
N75*	292.584	340.1	0.007	3.947	0.239	100%S
N76	286.958	394.5	0.005	0.451	0.046	95%S+5%A
		Adhesive Sika 32 with Primer Sika 55				
N74P	281.978	405.1	0.006	0.508	0.084	90%S+10%A
N75P*	292.584	439.2	0.009	2.135	0.171	95%S+5%A
N76P*	286.958	373.7	0.005	2.640	0.220	100%S
		Adhesive Madewell 1312F without Primer				
N77	285.693	381.5	0.005	0.798	0.107	100%S
N78*	292.739	320.8	0.003	1.557	0.112	100%S
N79	283.421	375.4	0.007	0.670	0.094	100%S
		Adhesive Madewell 1312F with Primer Madewell 927				
N77P	285.693	438.3	0.008	0.471	0.059	100%S
N78P	292.739	258.3	0.003	0.843	0.100	100%S
N79P	283.421	393.6	0.003	0.519	0.065	100%S

Table 8. Set #8 – Temperature 85°F, Relative Humidity 95%.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Adhesive Sika 30 without Primer				
N81	284.219	116.9	0.020	0.633	0.089	90%S+10%A
N82	285.349	271.3	0.024	0.585	0.050	100%S
N83	282.322	167.300	0.014	1.202	0.144	98%S+2%A
		Adhesive Sika 30 with Primer Sika 55				
N81P	284.219	101.2	0.015	0.727	0.119	90%S+10%A
N82P	285.349	301.7	0.018	0.435	0.049	100%S
N83P	282.322	287.4	0.013	0.670	0.112	90%S+10%A
		Adhesive Sika 32 without Primer				
N84	291.642	330.1	0.007	1.240	0.1	100%S
N85	294.170	335.4	0.003	1.395	0.089	100%S
N86	288.356	326.3	0.003	1.512	0.114	100%S
		Adhesive Sika 32 with Primer Sika 55				
N84P*	291.642	318	0.003	2.227	0.189	100%S
N85P*	294.170	280.9	0.005	1.514	0.114	100%S
N86P*	288.356	270.6	0.003	1.697	0.171	100%S
		Adhesive Madewell 1312F without Primer				
N87 SL	290.119	231.5	0.005	1.105	0.140	100%S
N88	292.453	231.8	0.007	0.815	0.104	95%S+5%A
N89 SL	291.940	307.4	0.003	1.070	0.127	100%S
		Adhesive Madewell 1312F with Primer Madewell 927				
N87P	290.119	332.1	0.004	1.410	0.126	100%S
N88P	292.453	337.6	0.009	1.000	0.114	100%S
N89P*	291.940	370.6	0.004	2.720	0.199	100%S

Table 9. Set #9 – Temperature 100°F, Relative Humidity 50%.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Adhesive Sika 30 without Primer				
N91	287.858	147.2	0.008	0.349	0.061	60%S+40%A
N92	289.315	212.3	0.012	0.618	0.060	100%S
N93	282.628	134.5	0.012	0.555	0.189	80%S+20%A
		Adhesive Sika 30 with Primer Sika 55				
N91P	287.858	65.48	0.014	0.302	0.057	30%S+70%A
N92P	289.315	117.2	0.010	0.700	0.076	95%S+5%A
N93P	282.628	204.8	0.011	0.530	0.055	95%S+5%A
		Adhesive Sika 32 without Primer				
N94*	280.561	322.5	0.004	1.715	0.161	100%S
N95	283.215	300.9	0.002	0.445	0.034	100%S
N96	279.572	302	0.004	0.423	0.049	100%S
		Adhesive Sika 32 with Primer Sika 55				
N94P	280.561	341.6	0.005	0.440	0.040	100%S
N95P*	283.215	319.5	0.004	2.075	0.214	100%S
N96P	279.572	348.7	0.003	0.489	0.066	100%S
		Adhesive Madewell 1312F without Primer				
N97R	280.561	352.0	0.005	0.497	0.070	98%S+2%A
N98R	283.215	277.0	0.004	0.509	0.085	100S
N99R	279.572	453.0	0.002	0.468	0.060	100%
		Adhesive Madewell 1312F with Primer Madewell 927				
N97PR	286.211	428.200	0.003	0.969	0.135	100%
N98PR	281.546	468.000	0.009	0.642	0.095	100%
N99PR	290.306	412.800	0.003	0.236	0.057	98%S+2%A

Table 10. Set #10 – Temperature 100°F, Relative Humidity 65%.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Adhesive Sika 30 without Primer				
N101	307.138	129.4	0.010	0.395	0.055	70%S+30%A
N102	293.312	164.1	0.012	0.590	0.071	95%S+5%A
N103	304.477	213.7	0.009	1.163	0.122	100%S
		Adhesive Sika 30 with Primer Sika 55				
N101P	307.138	102	0.015	0.385	0.066	70%S+30%A
N102P	293.312	93.93	0.012	0.597	0.073	90%S+10%A
N103P	304.477	311.8	0.011	0.900	0.114	95%S+5%A
		Adhesive Sika 32 without Primer				
N104	301.486	403.9	0.003	0.725	0.090	100%S
N105*	289.653	276.9	0.004	3.110	0.245	100%S
N106	298.328	329.1	0.010	0.982	0.114	98%S+2%A
		Adhesive Sika 32 with Primer Sika 55				
N104P	301.486	382	0.005	0.732	0.085	100%S
N105P	289.653	313.6	0.005	0.750	0.081	100%S
N106P	298.328	305.8	0.006	0.565	0.054	98%S+2%A
		Adhesive Madewell 1312F without Primer				
N107	287.550	243.8	0.003	0.33	0.069	95%S+5%A
N108*	285.745	320.4	0.004	1.565	0.168	100%S
N109	287.488	342.7	0.003	0.552	0.072	100%S
		Adhesive Madewell 1312F with Primer Madewell 927				
N107P	287.550	292.2	0.03	0.530	0.067	95%S+5%A
N108P	285.745	312.5	0.002	0.270	0.090	40%S+60%A
N109P	287.488	327.9	0.004	0.619	0.075	100%S

Table 11. Set #11 – Temperature 100°F, Relative Humidity 80%.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Adhesive Sika 30 without Primer				
N111	288.235	269	0.018	1.100	0.116	100%S
N112	288.365	144.9	0.012	0.435	0.073	70%S+30%A
N113	297.109	113	0.018	0.635	0.124	60%S+40%A
		Adhesive Sika 30 with Primer Sika 55				
N111P	288.235	223.8	0.008	0.733	0.091	60%S+40%A
N112P	288.365	189.5	0.011	0.250	0.043	5%S+95%A
N113P	297.109	309	0.012	0.625	0.074	97%S+3%A
		Adhesive Sika 32 without Primer				
N114	292.419	315.2	0.006	1.230	0.163	100%S
N115	297.120	294.9	0.004	0.250	0.034	100%S(thin lyr)
N116	305.989	259.3	0.009	0.252	0.018	100%S(thin lyr)
		Adhesive Sika 32 with Primer Sika 55				
N114P	292.419	288.2	0.008	0.325	0.031	50%S+50%A
N115P	297.120	331.5	0.008	1.468	0.149	100%S
N116P	305.989	445.9	0.006	0.402	0.073	100%S(thin lyr)
		Adhesive Madewell 1312F without Primer				
N117	306.328	429.4	0.004	0.345	0.087	75%S+25%A
N118 SL	309.997	265.1	0.005	0.785	0.126	100%S
N119 SL	287.058	194.5	0.003	0.955	0.124	100%S
		Adhesive Madewell 1312F with Primer Madewell 927				
N117P	306.328	287.1	0.007	1.117	0.133	100%S
N118P	309.997	371.9	0.008	0.255	0.030	100%S(thin lyr)
N119P	287.058	350.3	0.003	0.630	0.074	98%S+2%A

Table 12. Set #12 – Temperature 100°F, Relative Humidity 95%.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Adhesive Sika 30 without Primer				
N121*	302.726	176.5	0.020	1.628	0.177	100%S
N122	300.294	157.9	0.021	0.268	0.067	10%S+90%A
N123X	305.358	0	0.012	0.020	0.013	100%S
		Adhesive Sika 30 with Primer Sika 55				
N121P	302.726	220.4	0.028	0.538	0.108	90%S+10%A
N122P	300.294	250.1	0.016	0.604	0.080	98%S+2%A
N123P	305.358	279.9	0.016	0.335	0.059	70%S+30%A
		Adhesive Sika 32 without Primer				
N124 SL	307.822	63.03	0.02	0.091	0.018	1%S+99%A
N125 SL	289.640	48.97	0.017	0.040	0.012	100%C
N126	305.451	122.7	0.003	0.086	0.038	5%S+95%A
		Adhesive Sika 32 with Primer Sika 55				
N124PX	307.822	0	0.012	0.100	0.020	100%A
N125P	289.640	86.07	0.027	0.141	0.045	1%S+99%A
N126P	305.451	357.9	0.004	0.350	0.065	90%S+10%A
		Adhesive Madewell 1312F without Primer				
N127X	290.985	0	0.003	0.002	0.004	100%A
N128 SL	323.841	111.8	0.011	0.098	0.040	5%S+95%A
N129 SL	298.946	125.2	0.010	0.068	0.068	2%S+98%A
		Adhesive Madewell 1312F with Primer Madewell 927				
N127P*	290.985	209	0.003	1.608	0.273	100%S
N128P	323.841	364.2	0.009	0.295	0.081	20%S+80%A
N129P	298.946	359.3	0.017	0.053	0.053	50%S+50%A

Table 13. Adhesive Sika 30 at Temperature 70ºF.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
RH=50 %		Without Primer				
N11	280.312	168.3	0.012	0.573	0.058	100%S
N12	279.058	321	0.008	0.716	0.104	100%S
N13*	275.824	275.2	0.012	2.639	0.199	100%S
		With Primer Sika 55				
N11P	280.312	223	0.020	0.594	0.077	100%S
N12P*	279.058	297.7	0.010	2.595	0.179	100%S
N13P*	275.824	332.9	0.007	3.087	0.213	100%S
RH=65 %		Without Primer				
N21	274.124	108.2	0.011	0.618	0.149	100%S
N22	264.988	280.9	0.005	0.771	0.234	90%S+10%A
N23	288.71	276.2	0.014	0.550	0.051	95%S+5%A
		With Primer Sika 55				
N21P*	274.124	235	0.011	1.882	0.144	100%S
N22P	264.988	113.4	0.013	1.033	0.129	95%S+5%A
N23P	288.71	187.4	0.009	0.430	0.050	10%S+90%A
RH=80 %		Without Primer				
N31	279.708	223.4	0.010	0.320	0.097	20%A+80%S
N32	284.493	187	0.009	1.385	0.154	98%S+20%A
N33	275.726	167.9	0.003	1.119	0.124	95%S+5%A
		With Primer Sika 55				
N31P	279.708	91.11	0.008	0.155	0.019	10%S+90%A
N32PX	284.493	0	0.009	0.461	0.080	66%S+34%A
N33P	275.726	166.47	0.008	0.223	0.052	20%S+80%A
RH=95 %		Without Primer				
N41	285.319	336.8	0.014	0.282	0.052	15%S+85%A
N42	276.686	88.19	0.006	0.315	0.045	10%S+90%A
N43	290.384	143.2	0.019	0.975	0.109	95%S+5%A
		With Primer Sika 55				
N41P	285.319	144.1	0.015	0.481	0.059	90%S+10%A
N42P	276.686	134.9	0.004	0.285	0.062	50%S+50%A
N43P	290.384	156.3	0.008	0.825	0.114	40%S+60%A

Table 13A. Adhesive Sika 30 at Temperature 70°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N11	280.312	168.3	0.012	0.573	0.058	100%S
N12	279.058	321	0.008	0.716	0.104	100%S
N13*	275.824	275.2	0.012	2.639	0.199	100%S
RH=65%						
N21	274.124	108.2	0.011	0.618	0.149	100%S
N22	264.988	280.9	0.005	0.771	0.234	90%S+10%A
N23	288.71	276.2	0.014	0.550	0.051	95%S+5%A
RH=80%						
N31	279.708	223.4	0.010	0.320	0.097	20%A+80%S
N32	284.493	187	0.009	1.385	0.154	98%S+20%A
N33	275.726	167.9	0.003	1.119	0.124	95%S+5%A
RH=95%						
N41	285.319	336.8	0.014	0.282	0.052	15%S+85%A
N42	276.686	88.19	0.006	0.315	0.045	10%S+90%A
N43	290.384	143.2	0.019	0.975	0.109	95%S+5%A
		With Primer Sika 55				
RH=50%						
N11P	280.312	223	0.020	0.594	0.077	100%S
N12P*	279.058	297.7	0.010	2.595	0.179	100%S
N13P*	275.824	332.9	0.007	3.087	0.213	100%S
RH=65%						
N21P*	274.124	235	0.011	1.882	0.144	100%S
N22P	264.988	113.4	0.013	1.033	0.129	95%S+5%A
N23P	288.71	187.4	0.009	0.430	0.050	10%S+90%A
RH=80%						
N31P	279.708	91.11	0.008	0.155	0.019	10%S+90%A
N32PX	284.493	0	0.009	0.461	0.080	66%S+34%A
N33P	275.726	166.47	0.008	0.223	0.052	20%S+80%A
RH=95%						
N41P	285.319	144.1	0.015	0.481	0.059	90%S+10%A
N42P	276.686	134.9	0.004	0.285	0.062	50%S+50%A
N43P	290.384	156.3	0.008	0.825	0.114	40%S+60%A

Table 13B. Adhesive Sika 30 at Temperature 70°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N11	280.312	168.3	0.012	0.573	0.058	100%S
N12	279.058	321	0.008	0.716	0.104	100%S
N13*	275.824	275.2	0.012	2.639	0.199	100%S
Average		**254.83**	**0.011**	**1.309**	**0.120**	
RH=65%						
N21	274.124	108.2	0.011	0.618	0.149	100%S
N22	264.988	280.9	0.005	0.771	0.234	90%S+10%A
N23	288.71	276.2	0.014	0.550	0.051	95%S+5%A
Average		**221.77**	**0.010**	**0.646**	**0.145**	
RH=80%						
N31	279.708	223.4	0.010	0.320	0.097	20%A+80%S
N32	284.493	187	0.009	1.385	0.154	98%S+20%A
N33	275.726	167.9	0.003	1.119	0.124	95%S+5%A
Average		**192.77**	**0.007**	**0.941**	**0.125**	
RH=95%						
N41	285.319	336.8	0.014	0.282	0.052	15%S+85%A
N42	276.686	88.19	0.006	0.315	0.045	10%S+90%A
N43	290.384	143.2	0.019	0.975	0.109	95%S+5%A
Average		**189.40**	**0.013**	**0.524**	**0.069**	
		With Primer Sika 55				
RH=50%						
N11P	280.312	223	0.020	0.594	0.077	100%S
N12P*	279.058	297.7	0.010	2.595	0.179	100%S
N13P*	275.824	332.9	0.007	3.087	0.213	100%S
Average		**284.53**	**0.012**	**2.092**	**0.156**	
RH=65%						
N21P*	274.124	235	0.011	1.882	0.144	100%S
N22P	264.988	113.4	0.013	1.033	0.129	95%S+5%A
N23P	288.71	187.4	0.009	0.430	0.050	10%S+90%A
Average		**178.6**	**0.011**	**1.115**	**0.108**	
RH=80%						
N31P	279.708	91.11	0.008	0.155	0.019	10%S+90%A
N32PX	284.493	0	0.009	0.461	0.080	66%S+34%A
N33P	275.726	166.47	0.008	0.223	0.052	20%S+80%A
Average		**128.79**	**0.008**	**0.280**	**0.050**	
RH=95%						
N41P	285.319	144.1	0.015	0.481	0.059	90%S+10%A
N42P	276.686	134.9	0.004	0.285	0.062	50%S+50%A
N43P	290.384	156.3	0.008	0.825	0.114	40%S+60%A
Average		**145.1**	**0.009**	**0.530**	**0.078**	

Table 13C. Adhesive Sika 30 at Temperature 70°F.

RH%	Average Peak Load to pull off dolly pounds	Average Adhesive Film Thickness inch	Average Pulled off Mass of Mortar Layer grams	Average Max Height of Mortar Layer inch
	Without Primer			
50	254.83	0.011	1.309	0.12
65	221.77	0.01	0.646	0.145
80	192.77	0.007	0.941	0.125
95	189.4	0.013	0.524	0.069
	With Primer Sika 55			
50	284.53	0.012	2.092	0.156
65	178.6	0.011	1.115	0.108
80	128.79	0.008	0.28	0.05
95	145.1	0.009	0.53	0.078

Data rearranged for plotting.

RH%	No Primer	With Primer
50	254.83	284.53
65	221.77	178.6
80	192.77	128.79
95	189.4	145.1

Table 14. Adhesive Sika 30 at Temperature 85°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive
RH=50%		Without Primer				
N51X	280.141	0	0.016	0.866	0.118	85%S+15%A
N52	281.79	161.1	0.010	0.976	0.074	80%S+20%A
N53	276.78	82.5	0.030	0.978	0.059	70%S+30%A
		With Primer Sika 55				
N51P	280.141	130.4	0.035	1.330	0.102	98%S+2%A
N52P	281.79	91.6	0.012	0.942	0.093	75%S+25%A
N53P	276.78	181.6	0.019	0.879	0.069	80%S+20%A
RH=65%		Without Primer				
N61*	292.429	237.6	0.007	1.686	0.173	100%S
N62	288.232	180.2	0.008	1.135	0.126	90%S+10%A
N63	282.199	280.9	0.016	1.157	0.141	100%S
		With Primer Sika 55				
N61P	292.429	98.5	0.013	0.220	0.055	20%S+80%A
N62P	288.232	178.7	0.021	0.881	0.114	90%S+10%A
N63P	282.199	198.9	0.009	0.287	0.141	60%S+40%A
RH=80%		Without Primer				
N71	290.506	297.9	0.026	1.287	0.112	100%S
N72	289.865	133.7	0.011	0.535	0.097	80%S+20%A
N73	293.065	180	0.005	0.926	0.109	98%S+2%A
		With Primer Sika 55				
N71P	290.506	91.4	0.008	0.245	0.056	20%S+80%A
N72P	289.865	57.72	0.017	0.420	0.069	25%S+75%A
N73P	293.065	299.9	0.029	0.814	0.097	100%S
RH=95%		Without Primer				
N81	284.219	116.9	0.020	0.633	0.089	90%S+10%A
N82	285.349	271.3	0.024	0.585	0.050	100%S
N83	282.322	167.300	0.014	1.202	0.144	98%S+2%A
		With Primer Sika 55				
N81P	284.219	101.2	0.015	0.727	0.119	90%S+10%A
N82P	285.349	301.7	0.018	0.435	0.049	100%S
N83P	282.322	287.4	0.013	0.670	0.112	90%S+10%A

Table 14A. Adhesive Sika 30 at Temperature 85°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive
RH=50%		Without Primer				
N51X	280.141	0	0.016	0.866	0.118	85%S+15%A
N52	281.79	161.1	0.010	0.976	0.074	80%S+20%A
N53	276.78	82.5	0.030	0.978	0.059	70%S+30%A
RH=65%						
N61*	292.429	237.6	0.007	1.686	0.173	100%S
N62	288.232	180.2	0.008	1.135	0.126	90%S+10%A
N63	282.199	280.9	0.016	1.157	0.141	100%S
RH=80%						
N71	290.506	297.9	0.026	1.287	0.112	100%S
N72	289.865	133.7	0.011	0.535	0.097	80%S+20%A
N73	293.065	180	0.005	0.926	0.109	98%S+2%A
RH=95%						
N81	284.219	116.9	0.020	0.633	0.089	90%S+10%A
N82	285.349	271.3	0.024	0.585	0.050	100%S
N83	282.322	167.300	0.014	1.202	0.144	98%S+2%A
		With Primer Sika 55				
RH=50%						
N51P	280.141	130.4	0.035	1.330	0.102	98%S+2%A
N52P	281.79	91.6	0.012	0.942	0.093	75%S+25%A
N53P	276.78	181.6	0.019	0.879	0.069	80%S+20%A
RH=65%						
N61P	292.429	98.5	0.013	0.220	0.055	20%S+80%A
N62P	288.232	178.7	0.021	0.881	0.114	90%S+10%A
N63P	282.199	198.9	0.009	0.287	0.141	60%S+40%A
RH=80%						
N71P	290.506	91.4	0.008	0.245	0.056	20%S+80%A
N72P	289.865	57.72	0.017	0.420	0.069	25%S+75%A
N73P	293.065	299.9	0.029	0.814	0.097	100%S
RH=95%						
N81P	284.219	101.2	0.015	0.727	0.119	90%S+10%A
N82P	285.349	301.7	0.018	0.435	0.049	100%S
N83P	282.322	287.4	0.013	0.670	0.112	90%S+10%A

Table 14B. Adhesive Sika 30 at Temperature 85°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive
		Without Primer				
RH=50%						
N51X	280.141	0	0.016	0.866	0.118	85%S+15%A
N52	281.79	161.1	0.010	0.976	0.074	80%S+20%A
N53	276.78	82.5	0.030	0.978	0.059	70%S+30%A
Average		**121.8**	**0.019**	**0.94**	**0.084**	
RH=65%						
N61*	292.429	237.6	0.007	1.686	0.173	100%S
N62	288.232	180.2	0.008	1.135	0.126	90%S+10%A
N63	282.199	280.9	0.016	1.157	0.141	100%S
Average		**232.9**	**0.010**	**1.326**	**0.147**	
RH=80%						
N71	290.506	297.9	0.026	1.287	0.112	100%S
N72	289.865	133.7	0.011	0.535	0.097	80%S+20%A
N73	293.065	180	0.005	0.926	0.109	98%S+2%A
Average		**203.867**	**0.014**	**0.916**	**0.106**	
RH=95%						
N81	284.219	116.9	0.020	0.633	0.089	90%S+10%A
N82	285.349	271.3	0.024	0.585	0.050	100%S
N83	282.322	167.300	0.014	1.202	0.144	98%S+2%A
Average		**185.167**	**0.019**	**0.807**	**0.094**	
		With Primer Sika 55				
RH=50%						
N51P	280.141	130.4	0.035	1.330	0.102	98%S+2%A
N52P	281.79	91.6	0.012	0.942	0.093	75%S+25%A
N53P	276.78	181.6	0.019	0.879	0.069	80%S+20%A
Average		**134.533**	**0.022**	**1.050**	**0.088**	
RH=65%						
N61P	292.429	98.5	0.013	0.220	0.055	20%S+80%A
N62P	288.232	178.7	0.021	0.881	0.114	90%S+10%A
N63P	282.199	198.9	0.009	0.287	0.141	60%S+40%A
Average		**158.7**	**0.014**	**0.463**	**0.103**	
RH=80%						
N71P	290.506	91.4	0.008	0.245	0.056	20%S+80%A
N72P	289.865	57.72	0.017	0.420	0.069	25%S+75%A
N73P	293.065	299.9	0.029	0.814	0.097	100%S
Average		**149.673**	**0.018**	**0.493**	**0.074**	
RH=95%						
N81P	284.219	101.2	0.015	0.727	0.119	90%S+10%A
N82P	285.349	301.7	0.018	0.435	0.049	100%S
N83P	282.322	287.4	0.013	0.670	0.112	90%S+10%A
Average		**230.1**	**0.015**	**0.611**	**0.093**	

Table 14C. Adhesive Sika 30 at Temperature 85°F.

TEST	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch
RH%	Without Primer			
50	121.8	0.019	0.94	0.084
65	232.9	0.01	1.326	0.147
80	203.867	0.014	0.916	0.106
95	185.167	0.019	0.807	0.094
	With Primer Sika55			
50	134.533	0.022	1.05	0.088
65	158.7	0.014	0.463	0.103
80	149.673	0.018	0.493	0.074
95	230.1	0.015	0.611	0.093

Data rearranged for plotting

RH%	No Primer	With Primer
50	121.8	134.533
65	232.9	158.7
80	203.867	149.673
95	185.167	230.1

Table 15. Adhesive Sika 30 at Temperature 100°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
RH=50%		Without Primer				
N91	287.858	147.2	0.008	0.349	0.061	60%S+40%A
N92	289.315	212.3	0.012	0.618	0.060	100%S
N93	282.628	134.5	0.012	0.555	0.189	80%S+20%A
		With Primer Sika 55				
N91P	287.858	65.48	0.014	0.302	0.057	30%S+70%A
N92P	289.315	117.2	0.010	0.700	0.076	95%S+5%A
N93P	282.628	204.8	0.011	0.530	0.055	95%S+5%A
RH=65%		Without Primer				
N101	307.138	129.4	0.01	0.395	0.055	70%S+30%A
N102	293.312	164.1	0.012	0.59	0.071	95%S+5%A
N103	304.477	213.7	0.009	1.163	0.122	100%S
		With Primer Sika 55				
N101P	307.138	102	0.015	0.385	0.066	70%S+30%A
N102P	293.312	93.93	0.012	0.597	0.073	90%S+10%A
N103P	304.477	311.8	0.011	0.9	0.114	95%S+5%A
RH=80%		Without Primer				
N111	288.235	269	0.018	1.1	0.116	100%S
N112	288.365	144.9	0.012	0.435	0.073	70%S+30%A
N113	297.109	113	0.018	0.635	0.124	60%S+40%A
		With Primer Sika 55				
N111P	288.235	223.8	0.008	0.733	0.091	60%S+40%A
N112P	288.365	189.5	0.011	0.25	0.043	5%S+95%A
N113P	297.109	309	0.012	0.625	0.074	97%S+3%A
RH=95%		Without Primer				
N121*	302.726	176.5	0.02	1.628	0.177	100%S
N122	300.294	157.9	0.021	0.268	0.067	10%S+90%A
N123X	305.358	0	0.012	0.02	0.013	100%S
		With Primer Sika 55				
N121P	302.726	220.4	0.028	0.538	0.108	90%S+10%A
N122P	300.294	250.1	0.016	0.604	0.08	98%S+2%A
N123P	305.358	279.9	0.016	0.335	0.059	70%S+30%A

Table 15A. Adhesive Sika 30 at Temperature 100°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N91	287.858	147.2	0.008	0.349	0.061	60%S+40%A
N92	289.315	212.3	0.012	0.618	0.060	100%S
N93	282.628	134.5	0.012	0.555	0.189	80%S+20%A
RH=65%						
N101	307.138	129.4	0.01	0.395	0.055	70%S+30%A
N102	293.312	164.1	0.012	0.59	0.071	95%S+5%A
N103	304.477	213.7	0.009	1.163	0.122	100%S
RH=80%						
N111	288.235	269	0.018	1.1	0.116	100%S
N112	288.365	144.9	0.012	0.435	0.073	70%S+30%A
N113	297.109	113	0.018	0.635	0.124	60%S+40%A
RH=95%						
N121*	302.726	176.5	0.02	1.628	0.177	100%S
N122	300.294	157.9	0.021	0.268	0.067	10%S+90%A
N123X	305.358	0	0.012	0.02	0.013	100%S
		With Primer Sika 55				
RH=50%						
N91P	287.858	65.48	0.014	0.302	0.057	30%S+70%A
N92P	289.315	117.2	0.010	0.700	0.076	95%S+5%A
N93P	282.628	204.8	0.011	0.530	0.055	95%S+5%A
RH=65%						
N101P	307.138	102	0.015	0.385	0.066	70%S+30%A
N102P	293.312	93.93	0.012	0.597	0.073	90%S+10%A
N103P	304.477	311.8	0.011	0.9	0.114	95%S+5%A
RH=80%						
N111P	288.235	223.8	0.008	0.733	0.091	60%S+40%A
N112P	288.365	189.5	0.011	0.25	0.043	5%S+95%A
N113P	297.109	309	0.012	0.625	0.074	97%S+3%A
RH=95%						
N121P	302.726	220.4	0.028	0.538	0.108	90%S+10%A
N122P	300.294	250.1	0.016	0.604	0.08	98%S+2%A
N123P	305.358	279.9	0.016	0.335	0.059	70%S+30%A

Table 15B. Adhesive Sika 30 at Temperature 100°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N91	287.858	147.2	0.008	0.349	0.061	60%S+40%A
N92	289.315	212.3	0.012	0.618	0.060	100%S
N93	282.628	134.5	0.012	0.555	0.189	80%S+20%A
Average		**164.7**	**0.011**	**0.507**	**0.103**	
RH=65%						
N101	307.138	129.4	0.01	0.395	0.055	70%S+30%A
N102	293.312	164.1	0.012	0.59	0.071	95%S+5%A
N103	304.477	213.7	0.009	1.163	0.122	100%S
Average		**169.1**	**0.010**	**0.716**	**0.083**	
RH=80%						
N111	288.235	269	0.018	1.1	0.116	100%S
N112	288.365	144.9	0.012	0.435	0.073	70%S+30%A
N113	297.109	113	0.018	0.635	0.124	60%S+40%A
Average		**175.6**	**0.016**	**0.723**	**0.104**	
RH=95%						
N121*	302.726	176.5	0.02	1.628	0.177	100%S
N122	300.294	157.9	0.021	0.268	0.067	10%S+90%A
N123X	305.358	0	0.012	0.02	0.013	100%S
Average		**167.2**	**0.018**	**0.639**	**0.086**	
		With Primer Sika 55				
RH=50%						
N91P	287.858	65.48	0.014	0.302	0.057	30%S+70%A
N92P	289.315	117.2	0.010	0.700	0.076	95%S+5%A
N93P	282.628	204.8	0.011	0.530	0.055	95%S+5%A
Average		**129.16**	**0.012**	**0.511**	**0.063**	
RH=65%						
N101P	307.138	102	0.015	0.385	0.066	70%S+30%A
N102P	293.312	93.93	0.012	0.597	0.073	90%S+10%A
N103P	304.477	311.8	0.011	0.9	0.114	95%S+5%A
Average		**169.2**	**0.013**	**0.627**	**0.084**	
RH=80%						
N111P	288.235	223.8	0.008	0.733	0.091	60%S+40%A
N112P	288.365	189.5	0.011	0.25	0.043	5%S+95%A
N113P	297.109	309	0.012	0.625	0.074	97%S+3%A
Average		**240.8**	**0.010**	**0.536**	**0.069**	
RH=95%						
N121P	302.726	220.4	0.028	0.538	0.108	90%S+10%A
N122P	300.294	250.1	0.016	0.604	0.08	98%S+2%A
N123P	305.358	279.9	0.016	0.335	0.059	70%S+30%A
Average		**250.1**	**0.020**	**0.492**	**0.082**	

Table 15C. Adhesive Sika 30 at Temperature 100°F.

TEST	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch
	Without Primer			
RH%				
50	164.7	0.011	0.507	0.103
65	169.1	0.010	0.716	0.083
80	175.6	0.016	0.723	0.104
95	167.2	0.018	0.639	0.086
	With Primer Sika 55			
RH%				
50	129.16	0.012	0.511	0.063
65	169.2	0.013	0.627	0.084
80	240.8	0.010	0.536	0.069
95	250.1	0.020	0.492	0.082

Data rearranged for plotting

RH%	No Primer	With Primer
50	164.7	129.16
65	169.1	169.2
80	175.6	240.8
95	167.2	250.1

Table 16. Adhesive Sika 32 at Temperature 70°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
RH=50%		Without Primer				
N14	270.399	372.1	0.002	1.008	0.084	100%S
N15	282.337	278.1	0.002	1.379	0.151	100%S
N16*	291.342	452.6	0.004	3.530	0.192	100%S
		With Primer				
N14P*	270.399	390.9	0.001	2.727	0.194	100%S
N15P	282.337	340.3	0.002	0.532	0.06	100%S
N16P	291.342	411.4	0.025	1.651	0.139	100%S
RH=65%		Without Primer				
N24	287.947	276.2	0.01	0.638	0.1	100%S
N25	285.454	403.1	0.003	0.405	0.034	100%S
N26	286.765	359.9	0.008	0.636	0.044	100%S
		With Primer Sika 55				
N24P	287.947	301.5	0.008	0.705	0.059	100%S
N25P	285.454	293.6	0.008	0.447	0.054	95%S+5%A
N26P*	286.765	436	0.004	1.814	0.165	100%S
RH=80%		Without Primer				
N34	286.264	268.7	0.002	1.121	0.105	100%S
N35	283.979	287.4	0.01	1.06	0.108	100%S
N36	285.642	221.2	0.003	1.409	0.13	100%S
		With Primer Sika 55				
N34P	286.264	280.9	0.006	0.876	0.094	100%S
N35P	283.979	268.3	0.004	0.866	0.099	100%S
N36P	285.642	390.6	0.003	0.345	0.055	100%S
RH=95%		Without Primer				
N44	286.217	355.8	0.004	0.521	0.053	100%S
N45*	279.68	242.5	0.002	1.887	0.149	100%S
N46*	273.337	200.7	0.003	1.998	0.219	100%S
		With Primer Sika 55				
N44P	286.217	306.2	0.003	1.082	0.122	100%S
N45P	279.68	314.6	0.002	0.846	0.094	100%S
N46P	273.337	297.7	0.004	1.405	0.178	100%S

Table 16A. Adhesive Sika 32 at Temperature 70°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N14	270.399	372.1	0.002	1.008	0.084	100%S
N15	282.337	278.1	0.002	1.379	0.151	100%S
N16*	291.342	452.6	0.004	3.530	0.192	100%S
RH=65%						
N24	287.947	276.2	0.010	0.638	0.100	100%S
N25	285.454	403.1	0.003	0.405	0.034	100%S
N26	286.765	359.9	0.008	0.636	0.044	100%S
RH=80%						
N34	286.264	268.7	0.002	1.121	0.105	100%S
N35	283.979	287.4	0.010	1.060	0.108	100%S
N36	285.642	221.2	0.003	1.409	0.130	100%S
RH=95%						
N44	286.217	355.8	0.004	0.521	0.053	100%S
N45*	279.68	242.5	0.002	1.887	0.149	100%S
N46*	273.337	200.7	0.003	1.998	0.219	100%S
		With Primer Sika 55				
RH=50%						
N14P*	270.399	390.9	0.001	2.727	0.194	100%S
N15P	282.337	340.3	0.002	0.532	0.06	100%S
N16P	291.342	411.4	0.025	1.651	0.139	100%S
RH-65%						
N24P	287.947	301.5	0.008	0.705	0.059	100%S
N25P	285.454	293.6	0.008	0.447	0.054	95%S+5%A
N26P*	286.765	436	0.004	1.814	0.165	100%S
RH=80%						
N34P	286.264	280.9	0.006	0.876	0.094	100%S
N35P	283.979	268.3	0.004	0.866	0.099	100%S
N36P	285.642	390.6	0.003	0.345	0.055	100%S
RH=95%						
N44P	286.217	306.2	0.003	1.082	0.122	100%S
N45P	279.68	314.6	0.002	0.846	0.094	100%S
N46P	273.337	297.7	0.004	1.405	0.178	100%S

Table 16B. Adhesive Sika 32 at Temperature 70°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N14	270.399	372.1	0.002	1.008	0.084	100%S
N15	282.337	278.1	0.002	1.379	0.151	100%S
N16*	291.342	452.6	0.004	3.530	0.192	100%S
Average		**367.6**	**0.003**	**1.972**	**0.142**	
RH=65%						
N24	287.947	276.2	0.010	0.638	0.100	100%S
N25	285.454	403.1	0.003	0.405	0.034	100%S
N26	286.765	359.9	0.008	0.636	0.044	100%S
Average		**346.4**	**0.007**	**0.560**	**0.059**	
RH=80%						
N34	286.264	268.7	0.002	1.121	0.105	100%S
N35	283.979	287.4	0.010	1.060	0.108	100%S
N36	285.642	221.2	0.003	1.409	0.130	100%S
Average		**259.1**	**0.005**	**1.197**	**0.114**	
RH=95%						
N44	286.217	355.8	0.004	0.521	0.053	100%S
N45*	279.68	242.5	0.002	1.887	0.149	100%S
N46*	273.337	200.7	0.003	1.998	0.219	100%S
Average		**266.3**	**0.003**	**1.469**	**0.140**	
		With Primer Sika 55				
RH=50%						
N14P*	270.399	390.9	0.001	2.727	0.194	100%S
N15P	282.337	340.3	0.002	0.532	0.06	100%S
N16P	291.342	411.4	0.025	1.651	0.139	100%S
Average		**380.9**	**0.009**	**1.637**	**0.131**	
RH-65%						
N24P	287.947	301.5	0.008	0.705	0.059	100%S
N25P	285.454	293.6	0.008	0.447	0.054	95%S+5%A
N26P*	286.765	436	0.004	1.814	0.165	100%S
Average		**343.7**	**0.007**	**0.989**	**0.093**	
RH=80%						
N34P	286.264	280.9	0.006	0.876	0.094	100%S
N35P	283.979	268.3	0.004	0.866	0.099	100%S
N36P	285.642	390.6	0.003	0.345	0.055	100%S
Average		**313.3**	**0.004**	**0.696**	**0.083**	
RH=95%						
N44P	286.217	306.2	0.003	1.082	0.122	100%S
N45P	279.68	314.6	0.002	0.846	0.094	100%S
N46P	273.337	297.7	0.004	1.405	0.178	100%S
Average		**306.2**	**0.003**	**1.111**	**0.131**	

Table 16C. Adhesive Sika 32 at Temperature 70°F.

TEST	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch
	Without Primer			
RH%				
50	367.6	0.003	1.972	0.142
65	346.4	0.007	0.56	0.059
80	259.1	0.005	1.197	0.114
95	266.3	0.003	1.469	0.14
	With Primer Sika 55			
RH%				
50	380.9	0.009	1.637	0.131
65	343.7	0.007	0.989	0.093
80	313.3	0.004	0.696	0.083
95	306.2	0.003	1.111	0.131

Data rearranged for plotting

RH%	No Primer	With Primer
50	367.6	380.9
65	346.4	343.7
80	259.1	313.3
95	266.3	306.2

Table 17. Adhesive Sika 32 at Temperature 85°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
RH=50%		Without Primer				
N54	281.776	332.8	0.013	0.725	0.042	100%S
N55	279.558	273.8	0.017	0.837	0.064	100%S
N56	288.795	317.7	0.003	1.147	0.112	100%S
		With Primer Sika 55				
N54P	281.776	389	0.003	0.913	0.079	100%S
N55P*	279.558	252.8	0.005	1.686	0.099	100%S
N56P*	288.795	483.8	0.005	1.677	0.124	100%S
RH=65%		Without Primer				
N64	291.673	394.6	0.002	1.032	0.114	100%S
N65	289.44	232.8	0.003	1.301	0.104	100%S
N66*	288.04	281.3	0.004	2.027	0.159	100%S
		With Primer Sika 55				
N64P	291.673	312.6	0.006	0.907	0.083	90%S+10%A
N65P*	289.44	303.2	0.009	1.697	0.129	98%S+2%A
N66P	288.04	319.9	0.004	1.380	0.147	100%S
RH=80%		Without Primer				
N74	281.978	409.5	0.005	0.473	0.045	100%S
N75*	292.584	340.1	0.007	3.947	0.239	100%S
N76	286.958	394.5	0.005	0.451	0.046	95%S+5%A
		With Primer Sika 55				
N74P	281.978	405.1	0.006	0.508	0.084	90%S+10%A
N75P*	292.584	439.2	0.009	2.135	0.171	95%S+5%A
N76P*	286.958	373.7	0.005	2.640	0.22	100%S
RH=95%		Without Primer				
N84	291.642	330.1	0.007	1.240	0.100	100%S
N85	294.17	335.4	0.003	1.395	0.089	100%S
N86	288.356	326.3	0.003	1.512	0.114	100%S
		With Primer Sika 55				
N84P*	291.642	318	0.003	2.227	0.189	100%S
N85P*	294.17	280.9	0.005	1.514	0.114	100%S
N86P*	288.356	270.6	0.003	1.697	0.171	100%S

Table 17A. Adhesive Sika 32 at Temperature 85°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N54	281.776	332.8	0.013	0.725	0.042	100%S
N55	279.558	273.8	0.017	0.837	0.064	100%S
N56	288.795	317.7	0.003	1.147	0.112	100%S
RH=65%						
N64	291.673	394.6	0.002	1.032	0.114	100%S
N65	289.44	232.8	0.003	1.301	0.104	100%S
N66*	288.04	281.3	0.004	2.027	0.159	100%S
RH=80%						
N74	281.978	409.5	0.005	0.473	0.045	100%S
N75*	292.584	340.1	0.007	3.947	0.239	100%S
N76	286.958	394.5	0.005	0.451	0.046	95%S+5%A
RH=95%						
N84	291.642	330.1	0.007	1.240	0.100	100%S
N85	294.17	335.4	0.003	1.395	0.089	100%S
N86	288.356	326.3	0.003	1.512	0.114	100%S
		With Primer Sika 55				
RH=50%						
N84P*	291.642	318	0.003	2.227	0.189	100%S
N85P*	294.17	280.9	0.005	1.514	0.114	100%S
N86P*	288.356	270.6	0.003	1.697	0.171	100%S
RH=65%						
N54P	281.776	389	0.003	0.913	0.079	100%S
N55P*	279.558	252.8	0.005	1.686	0.099	100%S
N56P*	288.795	483.8	0.005	1.677	0.124	100%S
RH=80%						
N64P	291.673	312.6	0.006	0.907	0.083	90%S+10%A
N65P*	289.44	303.2	0.009	1.697	0.129	98%S+2%A
N66P	288.04	319.9	0.004	1.380	0.147	100%S
RH=95%						
N74P	281.978	405.1	0.006	0.508	0.084	90%S+10%A
N75P*	292.584	439.2	0.009	2.135	0.171	95%S+5%A
N76P*	286.958	373.7	0.005	2.640	0.22	100%S

Table 17B. Adhesive Sika 32 at Temperature 85°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N54	281.776	332.8	0.013	0.725	0.042	100%S
N55	279.558	273.8	0.017	0.837	0.064	100%S
N56	288.795	317.7	0.003	1.147	0.112	100%S
Average		**308.1**	**0.011**	**0.903**	**0.073**	
RH=65%						
N64	291.673	394.6	0.002	1.032	0.114	100%S
N65	289.44	232.8	0.003	1.301	0.104	100%S
N66*	288.04	281.3	0.004	2.027	0.159	100%S
Average		**302.9**	**0.003**	**1.453**	**0.126**	
RH=80%						
N74	281.978	409.5	0.005	0.473	0.045	100%S
N75*	292.584	340.1	0.007	3.947	0.239	100%S
N76	286.958	394.5	0.005	0.451	0.046	95%S+5%A
Average		**381.4**	**0.006**	**1.624**	**0.110**	
RH=95%						
N84	291.642	330.1	0.007	1.240	0.100	100%S
N85	294.17	335.4	0.003	1.395	0.089	100%S
N86	288.356	326.3	0.003	1.512	0.114	100%S
Average		**330.6**	**0.004**	**1.382**	**0.101**	
		With Primer Sika 55				
RH=50%						
N84P*	291.642	318	0.003	2.227	0.189	100%S
N85P*	294.17	280.9	0.005	1.514	0.114	100%S
N86P*	288.356	270.6	0.003	1.697	0.171	100%S
Average		**289.8**	**0.004**	**1.813**	**0.158**	
RH=65%						
N54P	281.776	389	0.003	0.913	0.079	100%S
N55P*	279.558	252.8	0.005	1.686	0.099	100%S
N56P*	288.795	483.8	0.005	1.677	0.124	100%S
Average		**375.2**	**0.004**	**1.425**	**0.101**	
RH=80%						
N64P	291.673	312.6	0.006	0.907	0.083	90%S+10%A
N65P*	289.44	303.2	0.009	1.697	0.129	98%S+2%A
N66P	288.04	319.9	0.004	1.380	0.147	100%S
Average		**311.9**	**0.006**	**1.328**	**0.120**	
RH=95%						
N74P	281.978	405.1	0.006	0.508	0.084	90%S+10%A
N75P*	292.584	439.2	0.009	2.135	0.171	95%S+5%A
N76P*	286.958	373.7	0.005	2.640	0.220	100%S
Average		**406.0**	**0.007**	**1.761**	**0.158**	

Table 17C. Adhesive Sika 32 at Temperature 85°F.

TEST	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch
	Without Primer			
RH%				
50	308.1	0.011	0.903	0.073
65	302.9	0.003	1.453	0.126
80	381.4	0.006	1.624	0.110
95	330.6	0.004	1.382	0.101
	With Primer Sika 55			
RH%				
50	289.8	0.004	1.813	0.158
65	375.2	0.004	1.425	0.101
80	311.9	0.006	1.328	0.120
95	406	0.007	1.761	0.158

Data rearranged for plotting

RH%	No Primer	With Primer
50	308.1	289.8
65	302.9	375.2
80	381.4	311.9
95	330.6	406

Table 18. Adhesive Sika 32 at Temperature 100°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
RH=50%		Without Primer				
N94*	280.561	322.5	0.004	1.715	0.161	100%S
N95	283.215	300.9	0.002	0.445	0.034	100%S
N96	279.572	302	0.004	0.423	0.049	100%S
		With Primer Sika 55				
N94P	280.561	341.6	0.005	0.440	0.040	100%S
N95P*	283.215	319.5	0.004	2.075	0.214	100%S
N96P	279.572	348.7	0.003	0.489	0.066	100%S
RH=65%		Without Primer				
N104	301.486	403.9	0.003	0.725	0.09	100%S
N105*	289.653	276.9	0.004	3.11	0.245	100%S
N106	298.328	329.1	0.010	0.982	0.114	98%S+2%A
		With Primer Sika 55				
N104P	301.486	382	0.005	0.732	0.085	100%S
N105P	289.653	313.6	0.005	0.750	0.081	100%S
N106P	298.328	305.8	0.006	0.565	0.054	98%S+2%A
RH=80%		Without Primer				
N114	292.419	315.2	0.006	1.230	0.163	100%S
N115	297.12	294.9	0.004	0.250	0.034	100%S(thin lyr)
N116	305.989	259.3	0.009	0.252	0.018	100%S(thin lyr)
		With Primer Sika 55				
N114P	292.419	288.2	0.008	0.325	0.031	50%S+50%A
N115P	297.12	331.5	0.008	1.468	0.149	100%S
N116P	305.989	445.9	0.006	0.402	0.073	100%S(thin lyr)
RH=95%		Without Primer				
N124 SL	307.822	63.03	0.020	0.091	0.018	1%S+99%A
N125 SL	289.64	48.97	0.017	0.04	0.012	100%C
N126	305.451	122.7	0.003	0.086	0.038	5%S+95%A
		With Primer Sika 55				
N124PX	307.822	0	0.012	0.100	0.020	100%A
N125P	289.64	86.07	0.027	0.141	0.045	1%S+99%A
N126P	305.451	357.9	0.004	0.35	0.065	90%S+10%A

Table 18A. Adhesive Sika 32 at Temperature 100°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N94*	280.561	322.5	0.004	1.715	0.161	100%S
N95	283.215	300.9	0.002	0.445	0.034	100%S
N96	279.572	302	0.004	0.423	0.049	100%S
RH=65%						
N104	301.486	403.9	0.003	0.725	0.09	100%S
N105*	289.653	276.9	0.004	3.11	0.245	100%S
N106	298.328	329.1	0.010	0.982	0.114	98%S+2%A
RH=80%						
N114	292.419	315.2	0.006	1.230	0.163	100%S
N115	297.12	294.9	0.004	0.250	0.034	100%S(thin lyr)
N116	305.989	259.3	0.009	0.252	0.018	100%S(thin lyr)
RH=95%						
N124 SL	307.822	63.03	0.020	0.091	0.018	1%S+99%A
N125 SL	289.64	48.97	0.017	0.04	0.012	100%C
N126	305.451	122.7	0.003	0.086	0.038	5%S+95%A
		With Primer Sika 55				
RH=50%						
N94P	280.561	341.6	0.005	0.440	0.040	100%S
N95P*	283.215	319.5	0.004	2.075	0.214	100%S
N96P	279.572	348.7	0.003	0.489	0.066	100%S
RH=65%						
N104P	301.486	382	0.005	0.732	0.085	100%S
N105P	289.653	313.6	0.005	0.750	0.081	100%S
N106P	298.328	305.8	0.006	0.565	0.054	98%S+2%A
RH=80%						
N114P	292.419	288.2	0.008	0.325	0.031	50%S+50%A
N115P	297.12	331.5	0.008	1.468	0.149	100%S
N116P	305.989	445.9	0.006	0.402	0.073	100%S(thin lyr)
RH=95%						
N124PX	307.822	0	0.012	0.100	0.020	100%A
N125P	289.64	86.07	0.027	0.141	0.045	1%S+99%A
N126P	305.451	357.9	0.004	0.35	0.065	90%S+10%A

Table 18B. Adhesive Sika 32 at Temperature 100°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N94*	280.561	322.5	0.004	1.715	0.161	100%S
N95	283.215	300.9	0.002	0.445	0.034	100%S
N96	279.572	302	0.004	0.423	0.049	100%S
Average		**308.5**	**0.003**	**0.861**	**0.081**	
RH=65%						
N104	301.486	403.9	0.003	0.725	0.09	100%S
N105*	289.653	276.9	0.004	3.11	0.245	100%S
N106	298.328	329.1	0.010	0.982	0.114	98%S+2%A
Average		**336.6**	**0.006**	**1.606**	**0.150**	
RH=80%						
N114	292.419	315.2	0.006	1.230	0.163	100%S
N115	297.12	294.9	0.004	0.250	0.034	100%S(thin lyr)
N116	305.989	259.3	0.009	0.252	0.018	100%S(thin lyr)
Average		**289.8**	**0.006**	**0.577**	**0.072**	
RH=95%						
N124 SL	307.822	63.03X	0.020	0.091	0.018	1%S+99%A
N125 SL	289.64	48.97X	0.017	0.04	0.012	100%C
N126	305.451	122.7	0.003	0.086	0.038	5%S+95%A
Average		**122.7**	**0.013**	**0.072**	**0.023**	
		With Primer Sika 55				
RH=50%						
N94P	280.561	341.6	0.005	0.440	0.040	100%S
N95P*	283.215	319.5	0.004	2.075	0.214	100%S
N96P	279.572	348.7	0.003	0.489	0.066	100%S
Average		**336.6**	**0.004**	**1.001**	**0.107**	
RH=65%						
N104P	301.486	382	0.005	0.732	0.085	100%S
N105P	289.653	313.6	0.005	0.750	0.081	100%S
N106P	298.328	305.8	0.006	0.565	0.054	98%S+2%A
Average		**333.8**	**0.005**	**0.682**	**0.073**	
RH=80%						
N114P	292.419	288.2	0.008	0.325	0.031	50%S+50%A
N115P	297.12	331.5	0.008	1.468	0.149	100%S
N116P	305.989	445.9	0.006	0.402	0.073	100%S(thin lyr)
Average		**355.2**	**0.007**	**0.732**	**0.084**	
RH=95%						
N124PX	307.822	0	0.012	0.100	0.020	100%A
N125P	289.64	86.07	0.027	0.141	0.045	1%S+99%A
N126P	305.451	357.9	0.004	0.35	0.065	90%S+10%A
Average		**222.0**	**0.014**	**0.197**	**0.043**	

Table 18C. Adhesive Sika 32 at Temperature 100°F.

TEST	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch
	Without Primer			
RH%				
50	308.5	0.003	0.861	0.081
65	336.6	0.006	1.606	0.15
80	289.8	0.006	0.577	0.072
95	122.7	0.013	0.072	0.023
	With Primer Sika 55			
RH%				
50	336.6	0.004	1.001	0.107
65	333.8	0.005	0.682	0.073
80	355.2	0.007	0.732	0.084
95	222	0.014	0.197	0.043

Data rearranged for plotting

RH%	No Primer	With Primer
50	308.5	336.6
65	336.6	333.8
80	289.8	355.2
95	122.7	222

Table 19. Adhesive Madewell 1312 at Temperature 70°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
RH=50%		Without Primer				
N17	281.962	212.6	0.005	0.609	0.104	100%S
N18*	291.696	315.2	0.001	1.746	0.159	100%S
N19	281.738	243.2	0.005	0.891	0.146	100%S
		With Primer Madewell 927				
N17P	281.962	244.2	0.001	1.937	0.159	100%S
N18P*	291.696	329.4	0.008	2.097	0.183	100%S
N19P	281.738	183.4	0.002	0.982	0.119	100%S
RH=65%		Without Primer				
N27-SL	286.433	149.4	0.005	0.497	0.091	90%S+10%A
N28	265.607	161.3	0.012	1.011	0.164	100%S
N29	276.982	267.2	0.006	1.114	0.168	100%S
		With Primer Madewell 927				
N27P	286.433	306	0.006	1.448	0.129	100%S
N28P*	265.607	167.3	0.005	1.85	0.161	100%S
N29P*	276.982	233.6	0.007	2.103	0.159	100%S
RH=80%		Without Primer				
N37	280.439	184.3	0.005	0.731	0.088	100%S
N38	288.469	136	0.002	0.374	0.064	98%S+2%A
N39	270.004	128.5	0.002	0.845	0.151	95%S+5%A
		With Primer Madewell 927				
N37P	280.439	261.2	0.004	0.876	0.109	100%S
N38P	288.469	298.5	0.004	0.809	0.084	100%S
N39P	270.004	194.2	0.004	1.221	0.117	100%S
RH=95%		Without Primer				
N47	278.662	200.7	0.009	0.892	0.118	96%S+4%A
N48	289.921	315.4	0.009	0.838	0.089	100%S
N49	281.559	229.8	0.005	1.422	0.169	100%S
		With Primer Madewell 927				
N47P	278.662	304.2	0.005	1.272	0.132	100%S
N48P*	289.921	369	0.004	1.924	0.139	100%S
N49P	281.559	264.2	0.004	1.189	0.096	100%S

Table 19A. Adhesive Madewell 1312 at Temperature 70°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N17	281.962	212.6	0.005	0.609	0.104	100%S
N18*	291.696	315.2	0.001	1.746	0.159	100%S
N19	281.738	243.2	0.005	0.891	0.146	100%S
RH=65%						
N27-SL	286.433	149.4	0.005	0.497	0.091	90%S+10%A
N28	265.607	161.3	0.012	1.011	0.164	100%S
N29	276.982	267.2	0.006	1.114	0.168	100%S
RH=80%						
N37	280.439	184.3	0.005	0.731	0.088	100%S
N38	288.469	136	0.002	0.374	0.064	98%S+2%A
N39	270.004	128.5	0.002	0.845	0.151	95%S+5%A
RH=95%						
N47	278.662	200.7	0.009	0.892	0.118	96%S+4%A
N48	289.921	315.4	0.009	0.838	0.089	100%S
N49	281.559	229.8	0.005	1.422	0.169	100%S
		With Primer Madewell 927				
RH=50%						
N17P	281.962	244.2	0.001	1.937	0.159	100%S
N18P*	291.696	329.4	0.008	2.097	0.183	100%S
N19P	281.738	183.4	0.002	0.982	0.119	100%S
RH=65%						
N27P	286.433	306	0.006	1.448	0.129	100%S
N28P*	265.607	167.3	0.005	1.85	0.161	100%S
N29P*	276.982	233.6	0.007	2.103	0.159	100%S
RH=80%						
N37P	280.439	261.2	0.004	0.876	0.109	100%S
N38P	288.469	298.5	0.004	0.809	0.084	100%S
N39P	270.004	194.2	0.004	1.221	0.117	100%S
RH=95%						
N47P	278.662	304.2	0.005	1.272	0.132	100%S
N48P*	289.921	369	0.004	1.924	0.139	100%S
N49P	281.559	264.2	0.004	1.189	0.096	100%S

Table 19B. Adhesive Madewell 1312 at Temperature 70°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N17	281.962	212.6	0.005	0.609	0.104	100%S
N18*	291.696	315.2	0.001	1.746	0.159	100%S
N19	281.738	243.2	0.005	0.891	0.146	100%S
Average		**257.0**	**0.004**	**1.082**	**0.136**	
RH=65%						
N27-SL	286.433	149.4	0.005	0.497	0.091	90%S+10%A
N28	265.607	161.3	0.012	1.011	0.164	100%S
N29	276.982	267.2	0.006	1.114	0.168	100%S
Average		**192.6**	**0.008**	**0.874**	**0.141**	
RH=80%						
N37	280.439	184.3	0.005	0.731	0.088	100%S
N38	288.469	136	0.002	0.374	0.064	98%S+2%A
N39	270.004	128.5	0.002	0.845	0.151	95%S+5%A
Average		**149.6**	**0.003**	**0.650**	**0.101**	
RH=95%						
N47	278.662	200.7	0.009	0.892	0.118	96%S+4%A
N48	289.921	315.4	0.009	0.838	0.089	100%S
N49	281.559	229.8	0.005	1.422	0.169	100%S
Average		**248.6**	**0.008**	**1.051**	**0.125**	
		With Primer Madewell 927				
RH=50%						
N17P	281.962	244.2	0.001	1.937	0.159	100%S
N18P*	291.696	329.4	0.008	2.097	0.183	100%S
N19P	281.738	183.4	0.002	0.982	0.119	100%S
Average		**252.3**	**0.004**	**1.672**	**0.154**	
RH=65%						
N27P	286.433	306	0.006	1.448	0.129	100%S
N28P*	265.607	167.3	0.005	1.85	0.161	100%S
N29P*	276.982	233.6	0.007	2.103	0.159	100%S
Average		**235.6**	**0.006**	**1.800**	**0.150**	
RH=80%						
N37P	280.439	261.2	0.004	0.876	0.109	100%S
N38P	288.469	298.5	0.004	0.809	0.084	100%S
N39P	270.004	194.2	0.004	1.221	0.117	100%S
Average		**251.3**	**0.004**	**0.969**	**0.103**	
RH=95%						
N47P	278.662	304.2	0.005	1.272	0.132	100%S
N48P*	289.921	369	0.004	1.924	0.139	100%S
N49P	281.559	264.2	0.004	1.189	0.096	100%S
Average		**312.5**	**0.004**	**1.462**	**0.122**	

Table 19C. Adhesive Madewell 1312 at Temperature 70°F.

TEST	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch
	Without Primer			
RH%				
50	257	0.004	1.082	0.136
65	192.6	0.008	0.874	0.141
80	149.6	0.003	0.65	0.101
95	248.6	0.008	1.051	0.125
	With Primer Madewell 927			
RH%				
50	252.3	0.004	1.672	0.154
65	235.6	0.006	1.8	0.15
80	251.3	0.004	0.969	0.103
95	312.5	0.004	1.462	0.122

Data rearranged for plotting

RH%	No Primer	With Primer
50	257	252.3
65	192.6	235.6
80	149.6	251.3
95	248.6	312.5

Table 20. Adhesive Madewell 1312 at Temperature 85°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
RH=50%		Without Primer				
N57	287.163	406.7	0.004	0.561	0.062	98%S+2%A
N58	290.875	291.5	0.005	0.525	0.067	100%S
N59	288.471	170	0.005	0.396	0.043	100%S
		With Primer Madewell 927				
N57PX	287.163	0	0.006	0.085	0.090	100%A
N58P	290.875	458.3	0.004	0.633	0.075	98%S+2%A
N59P	288.471	330.3	0.003	1.270	0.123	100%S
RH=65%		Without Primer				
N67*	289.809	277.9	0.011	2.105	0.207	100%S
N68 SL	286.662	253.7	0.009	0.411	0.061	75%S+25%A
N69	292.428	223.2	0.004	0.922	0.124	100%S
		With Primer Madewell 927				
N67P*	289.809	256	0.002	1.731	0.144	100%S
N68P	286.662	259.1	0.008	0.747	0.083	100%S
N69P	292.428	329.1	0.007	0.908	0.094	100%S
RH=80%		Without Primer				
N77	285.693	381.5	0.005	0.798	0.107	100%S
N78*	292.739	320.8	0.003	1.557	0.112	100%S
N79	283.421	375.4	0.007	0.67	0.094	100%S
		With Primer Madewell 927				
N77P	285.693	438.3	0.008	0.471	0.059	100%S
N78P	292.739	258.3	0.003	0.843	0.1	100%S
N79P	283.421	393.6	0.003	0.519	0.065	100%S
RH=95%		Without Primer				
N87 SL	290.119	231.5	0.005	1.105	0.14	100%S
N88	292.453	231.8	0.007	0.815	0.104	95%S+5%A
N89 SL	291.94	307.4	0.003	1.07	0.127	100%S
		With Primer Madewell 927				
N87P	290.119	332.1	0.004	1.41	0.126	100%S
N88P	292.453	337.6	0.009	1	0.114	100%S
N89P*	291.94	370.6	0.004	2.72	0.199	100%S

Table 20A. Adhesive Madewell 1312 at Temperature 85°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N57	287.163	406.7	0.004	0.561	0.062	98%S+2%A
N58	290.875	291.5	0.005	0.525	0.067	100%S
N59	288.471	170	0.005	0.396	0.043	100%S
RH=65%						
N67*	289.809	277.9	0.011	2.105	0.207	100%S
N68 SL	286.662	253.7	0.009	0.411	0.061	75%S+25%A
N69	292.428	223.2	0.004	0.922	0.124	100%S
RH=80%						
N77	285.693	381.5	0.005	0.798	0.107	100%S
N78*	292.739	320.8	0.003	1.557	0.112	100%S
N79	283.421	375.4	0.007	0.670	0.094	100%S
RH=95%						
N87 SL	290.119	231.5	0.005	1.105	0.140	100%S
N88	292.453	231.8	0.007	0.815	0.104	95%S+5%A
N89 SL	291.94	307.4	0.003	1.070	0.127	100%S
		With Primer Madewell 927				
RH=50%						
N87P	290.119	332.1	0.004	1.410	0.126	100%S
N88P	292.453	337.6	0.009	1.000	0.114	100%S
N89P*	291.94	370.6	0.004	2.720	0.199	100%S
RH=65%						
N57PX	287.163	0	0.006	0.085	0.090	100%A
N58P	290.875	458.3	0.004	0.633	0.075	98%S+2%A
N59P	288.471	330.3	0.003	1.270	0.123	100%S
RH=80%						
N67P*	289.809	256	0.002	1.731	0.144	100%S
N68P	286.662	259.1	0.008	0.747	0.083	100%S
N69P	292.428	329.1	0.007	0.908	0.094	100%S
RH=95%						
N77P	285.693	438.3	0.008	0.471	0.059	100%S
N78P	292.739	258.3	0.003	0.843	0.100	100%S
N79P	283.421	393.6	0.003	0.519	0.065	100%S

Table 20B. Adhesive Madewell 1312 at Temperature 85°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N57	287.163	406.7	0.004	0.561	0.062	98%S+2%A
N58	290.875	291.5	0.005	0.525	0.067	100%S
N59	288.471	170	0.005	0.396	0.043	100%S
Average		**289.4**	**0.005**	**0.494**	**0.057**	
RH=65%						
N67*	289.809	277.9	0.011	2.105	0.207	100%S
N68 SL	286.662	253.7	0.009	0.411	0.061	75%S+25%A
N69	292.428	223.2	0.004	0.922	0.124	100%S
Average		**251.6**	**0.008**	**1.146**	**0.131**	
RH=80%						
N77	285.693	381.5	0.005	0.798	0.107	100%S
N78*	292.739	320.8	0.003	1.557	0.112	100%S
N79	283.421	375.4	0.007	0.670	0.094	100%S
Average		**359.2**	**0.005**	**1.008**	**0.104**	
RH=95%						
N87 SL	290.119	231.5	0.005	1.105	0.140	100%S
N88	292.453	231.8	0.007	0.815	0.104	95%S+5%A
N89 SL	291.94	307.4	0.003	1.070	0.127	100%S
Average		**256.9**	**0.005**	**0.997**	**0.124**	
		With Primer Madewell 927				
RH=50%						
N87P	290.119	332.1	0.004	1.410	0.126	100%S
N88P	292.453	337.6	0.009	1.000	0.114	100%S
N89P*	291.94	370.6	0.004	2.720	0.199	100%S
Average		**346.8**	**0.006**	**1.710**	**0.146**	
RH=65%						
N57PX	287.163	0	0.006	0.085	0.090	100%A
N58P	290.875	458.3	0.004	0.633	0.075	98%S+2%A
N59P	288.471	330.3	0.003	1.270	0.123	100%S
Average		**394.3**	**0.004**	**0.663**	**0.096**	
RH=80%						
N67P*	289.809	256	0.002	1.731	0.144	100%S
N68P	286.662	259.1	0.008	0.747	0.083	100%S
N69P	292.428	329.1	0.007	0.908	0.094	100%S
Average		**281.4**	**0.006**	**1.129**	**0.107**	
RH=95%						
N77P	285.693	438.3	0.008	0.471	0.059	100%S
N78P	292.739	258.3	0.003	0.843	0.100	100%S
N79P	283.421	393.6	0.003	0.519	0.065	100%S
Average		**363.4**	**0.005**	**0.611**	**0.075**	

Table 20C. Adhesive Madewell 1312 at Temperature 85°F.

TEST	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch
	Without Primer			
RH%				
50	289.4	0.005	0.494	0.057
65	251.6	0.008	1.146	0.131
80	359.2	0.005	1.008	0.104
95	256.9	0.005	0.997	0.124
	With Primer Madewell 927			
RH%				
50	346.8	0.006	1.71	0.146
65	394.3	0.004	0.663	0.096
80	281.4	0.006	1.129	0.107
95	363.4	0.005	0.611	0.075

Data rearranged for plotting

RH%	No Primer	With Primer
50	289.4	346.8
65	251.6	394.3
80	359.2	281.4
95	256.9	363.4

Table 21. Adhesive Madewell 1312 at Temperature 100°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
RH=50%		Without Primer				
N97R	280.561	352.000	0.005	0.497	0.070	98%S+2%A
N98R	283.215	277.000	0.004	0.509	0.085	100%S
N99R	279.572	453.000	0.002	0.468	0.060	100%S
		With Primer Madewell 927				
N97PR	286.211	428.200	0.003	0.969	0.135	100%S
N98PR	281.546	468.000	0.009	0.642	0.095	100%S
N99PR	290.306	412.800	0.003	0.236	0.057	98%S+2%A
RH=65%		Without Primer				
N107	287.55	243.8	0.003	0.330	0.069	95%S+5%A
N108*	285.745	320.4	0.004	1.565	0.168	100%S
N109	287.488	342.7	0.003	0.552	0.072	100%S
		With Primer Madewell 927				
N107P	287.55	292.2	0.03	0.530	0.067	95%S+5%A
N108P	285.745	312.5	0.002	0.270	0.090	40%S+60%A
N109P	287.488	327.9	0.004	0.619	0.075	100%S
RH=80%		Without Primer				
N117	306.328	429.4	0.004	0.345	0.087	75%S+25%A
N118 SL	309.997	265.1	0.005	0.785	0.126	100%S
N119 SL	287.058	194.5	0.003	0.955	0.124	100%S
		With Primer Madewell 927				
N117P	306.328	287.1	0.007	1.117	0.133	100%S
N118P	309.997	371.9	0.008	0.255	0.030	100%S(thin lyr)
N119P	287.058	350.3	0.003	0.630	0.074	98%S+2%A
Rh=95%		Without Primer				
N127X	290.985	0	0.003	0.002	0.004	100%A
N128 SL	323.841	111.8	0.011	0.098	0.040	5%S+95%A
N129 SL	298.946	125.2	0.01	0.068	0.068	2%S+98%A
		With Primer Madewell 927				
N127P*	290.985	209	0.003	1.608	0.273	100%S
N128P	323.841	364.2	0.009	0.295	0.081	20%S+80%A
N129P	298.946	359.3	0.017	0.053	0.053	50%S+50%A

Table 21A. Adhesive Madewell 1312 at Temperature 100°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N97R	280.561	352.0	0.005	0.497	0.070	98%S+2%A
N98R	283.215	277.0	0.004	0.509	0.085	100%S
N99R	279.572	453.0	0.002	0.468	0.060	100%S
RH=65%						
N107	287.55	243.8	0.003	0.330	0.069	95%S+5%A
N108*	285.745	320.4	0.004	1.565	0.168	100%S
N109	287.488	342.7	0.003	0.552	0.072	100%S
RH=80%						
N117	306.328	429.4	0.004	0.345	0.087	75%S+25%A
N118 SL	309.997	265.1	0.005	0.785	0.126	100%S
N119 SL	287.058	194.5	0.003	0.955	0.124	100%S
RH=95%						
N127X	290.985	0	0.003	0.002	0.004	100%A
N128 SL	323.841	111.8	0.011	0.098	0.040	5%S+95%A
N129 SL	298.946	125.2	0.01	0.068	0.068	2%S+98%A
		With Primer Madewell 927				
RH=50%						
N97PR	286.211	428.2	0.003	0.969	0.135	100%S
N98PR	281.546	468.0	0.009	0.642	0.095	100%S
N99PR	290.306	412.8	0.003	0.236	0.057	98%S+2%A
RH=65%						
N107P	287.55	292.2	0.03	0.530	0.067	95%S+5%A
N108P	285.745	312.5	0.002	0.270	0.090	40%S+60%A
N109P	287.488	327.9	0.004	0.619	0.075	100%S
RH=80%						
N117P	306.328	287.1	0.007	1.117	0.133	100%S
N118P	309.997	371.9	0.008	0.255	0.030	100%S(thin lyr)
N119P	287.058	350.3	0.003	0.630	0.074	98%S+2%A
RH=95%						
N127P*	290.985	209	0.003	1.608	0.273	100%S
N128P	323.841	364.2	0.009	0.295	0.081	20%S+80%A
N129P	298.946	359.3	0.017	0.053	0.053	50%S+50%A

Table 21B. Adhesive Madewell 1312 at Temperature 100°F.

TEST	Mass of Mortar Cube grams	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch	Type of Failure S=substrate A=adhesive C=cohesive
		Without Primer				
RH=50%						
N97R	280.561	352.0	0.005	0.497	0.070	98%S+2%A
N98R	283.215	277.0	0.004	0.509	0.085	100%S
N99R	279.572	453.0	0.002	0.468	0.060	100%S
Average		**360.7**	**0.004**	**0.491**	**0.072**	
RH=65%						
N107	287.55	243.8	0.003	0.330	0.069	95%S+5%A
N108*	285.745	320.4	0.004	1.565	0.168	100%S
N109	287.488	342.7	0.003	0.552	0.072	100%S
Average		**302.3**	**0.003**	**0.816**	**0.103**	
RH=80%						
N117	306.328	429.4	0.004	0.345	0.087	75%S+25%A
N118 SL	309.997	265.1	0.005	0.785	0.126	100%S
N119 SL	287.058	194.5	0.003	0.955	0.124	100%S
Average		**296.3**	**0.004**	**0.695**	**0.112**	
RH=95%						
N127X	290.985	0	0.003	0.002	0.004	100%A
N128 SL	323.841	111.8	0.011	0.098	0.040	5%S+95%A
N129 SL	298.946	125.2	0.01	0.068	0.068	2%S+98%A
Average		**118.5**	**0.008**	**0.056**	**0.037**	
		With Primer Madewell 927				
RH=50%						
N97PR	286.211	428.2	0.003	0.969	0.135	100%S
N98PR	281.546	468.0	0.009	0.642	0.095	100%S
N99PR	290.306	412.8	0.003	0.236	0.057	98%S+2%A
Average		**436.3**	**0.005**	**0.616**	**0.096**	
RH=65%						
N107P	287.55	292.2	0.03	0.530	0.067	95%S+5%A
N108P	285.745	312.5	0.002	0.270	0.090	40%S+60%A
N109P	287.488	327.9	0.004	0.619	0.075	100%S
Average		**310.9**	**0.012**	**0.473**	**0.077**	
RH=80%						
N117P	306.328	287.1	0.007	1.117	0.133	100%S
N118P	309.997	371.9	0.008	0.255	0.030	100%S(thin lyr)
N119P	287.058	350.3	0.003	0.630	0.074	98%S+2%A
Average		**336.4**	**0.006**	**0.667**	**0.079**	
RH=95%						
N127P*	290.985	209	0.003	1.608	0.273	100%S
N128P	323.841	364.2	0.009	0.295	0.081	20%S+80%A
N129P	298.946	359.3	0.017	0.053	0.053	50%S+50%A
Average		**310.8**	**0.01**	**0.652**	**0.136**	

Table 21C. Adhesive Madewell 1312 at Temperature 100°F.

TEST	Peak Load to pull off dolly pounds	Adhesive Film Thickness inch	Pulled off Mass of Mortar Layer grams	Max Height of Mortar Layer inch
	Without Primer			
RH%				
50	360.7	0.004	0.491	0.072
65	302.3	0.003	0.816	0.103
80	296.3	0.004	0.695	0.112
95	118.5	0.008	0.056	0.037
	With Primer Madewell 927			
RH%				
50	436.3	0.005	0.616	0.096
65	310.9	0.012	0.473	0.077
80	336.4	0.006	0.667	0.079
95	310.8	0.010	0.652	0.136

Data rearranged for plotting

RH%	No Primer	With Primer
50	360.7	436.3
65	302.3	310.9
80	296.3	336.4
95	118.5	310.8

Table 22. Distribution of Types of Failures.

RH%	Substrate 100%	Substrate 99%-80%	Substrate 79%-60%	Substrate 59%-1%	Adhesive 100%	Cohesive 100%	Total
			TEMPERATURE = 70 DEG F				
50(Set1)	18	0	0	0	0	0	18
65(Set2)	12	5	0	1	0	0	18
80(Set3)	10	5	1	2	0	0	18
95(Set4)	11	3	0	4	0	0	18
Total A	51	13	1	7	0	0	72
			TEMPERATURE = 85 DEG F				
50(Set5)	9	6	2		1	0	18
65(Set6)	11	4	2	1	0	0	18
80(Set7)	11	5	0	2	0	0	18
95(Set8)	13	5	0	0	0	0	18
Total B	44	20	4	3	1	0	72
			TEMPERATURE = 100 DEG F				
50(Set9)	11	5	1	1	0	0	18
65(Set10)	8	7	2	1	0	0	18
80(Set11)	10	2	4	2	0	0	18
95(Set12)	3	3	1	8	2	1	18
Total C	32	17	8	12	2	1	72
A+B+C	127	50	13	22	3	1	216

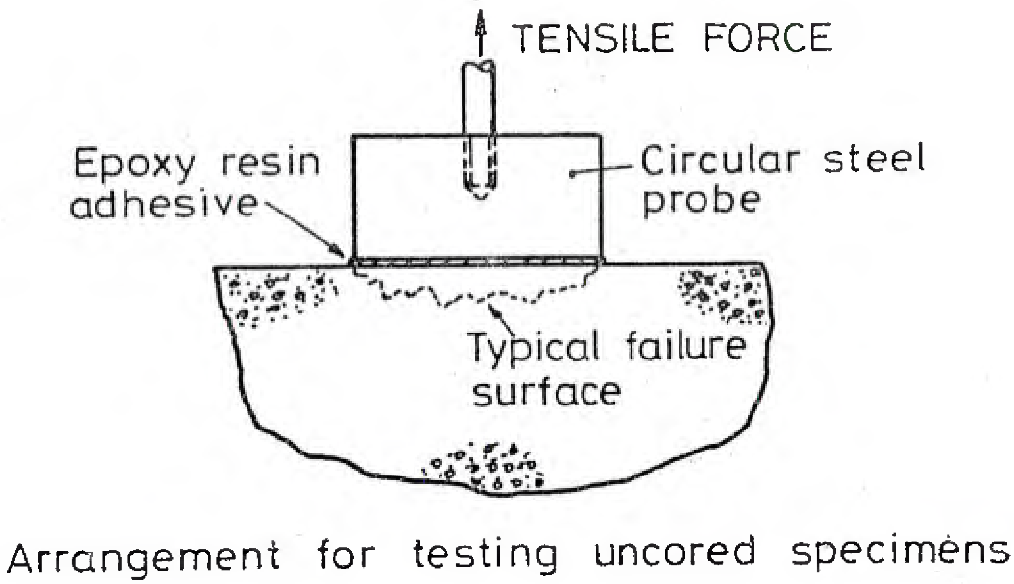


Figure 1: Original Version of Pull-Off Test (after A. E. Long)

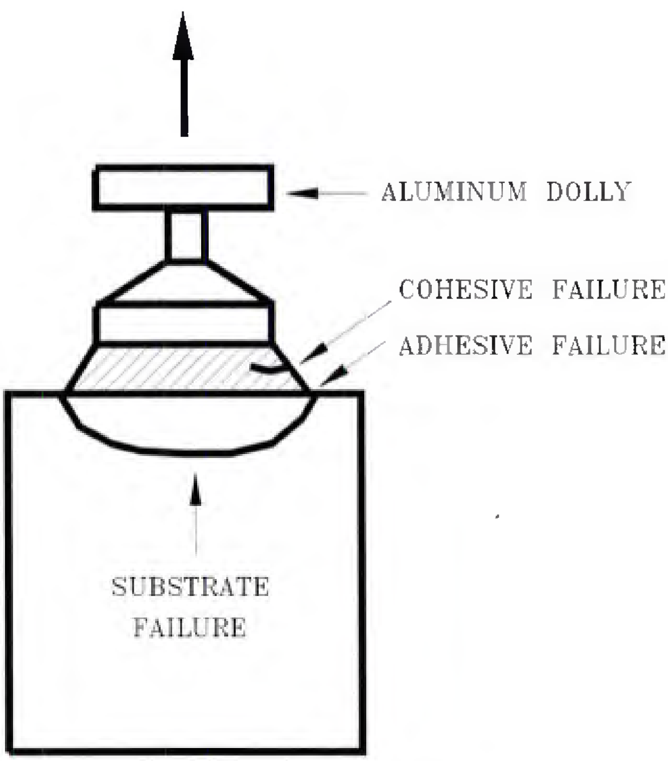


Figure 2: Schematic of Different Types of Failures

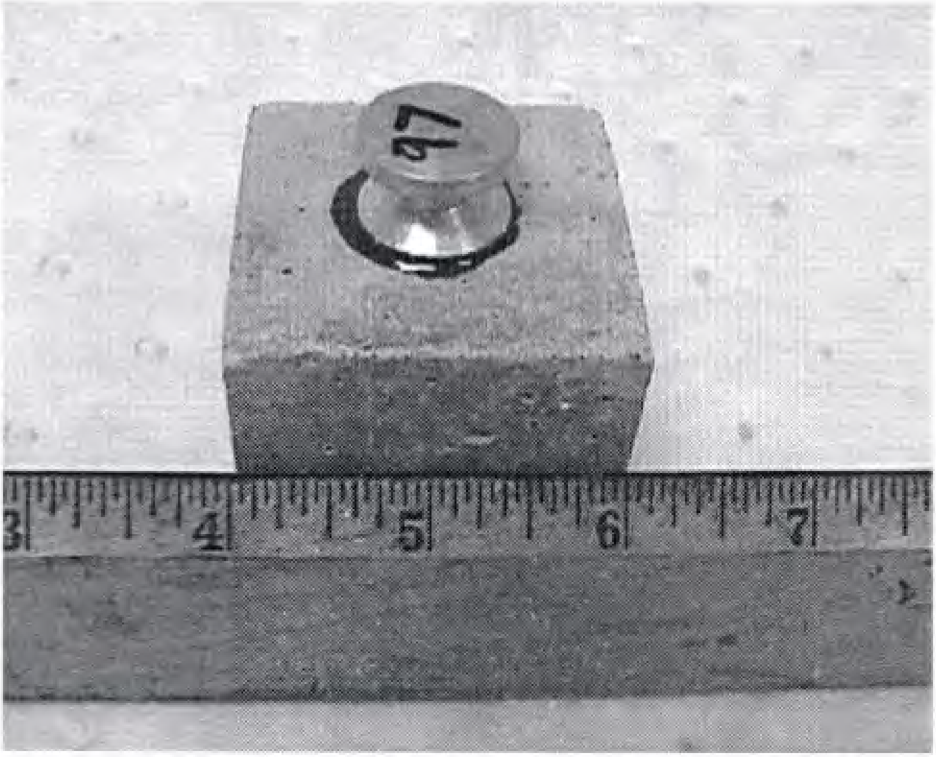


Figure 3: Photo of Dolly attached to Cement Mortar Cube showing Dimensions

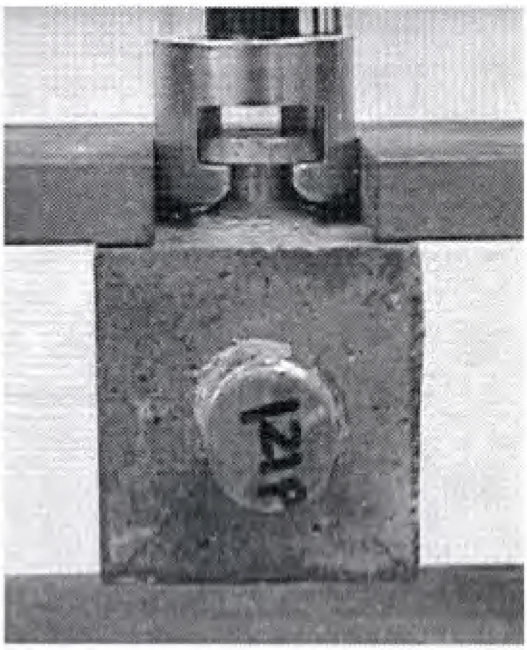


Figure 4: Special Fixture for Pull-Off Test with Instron Testing Machine

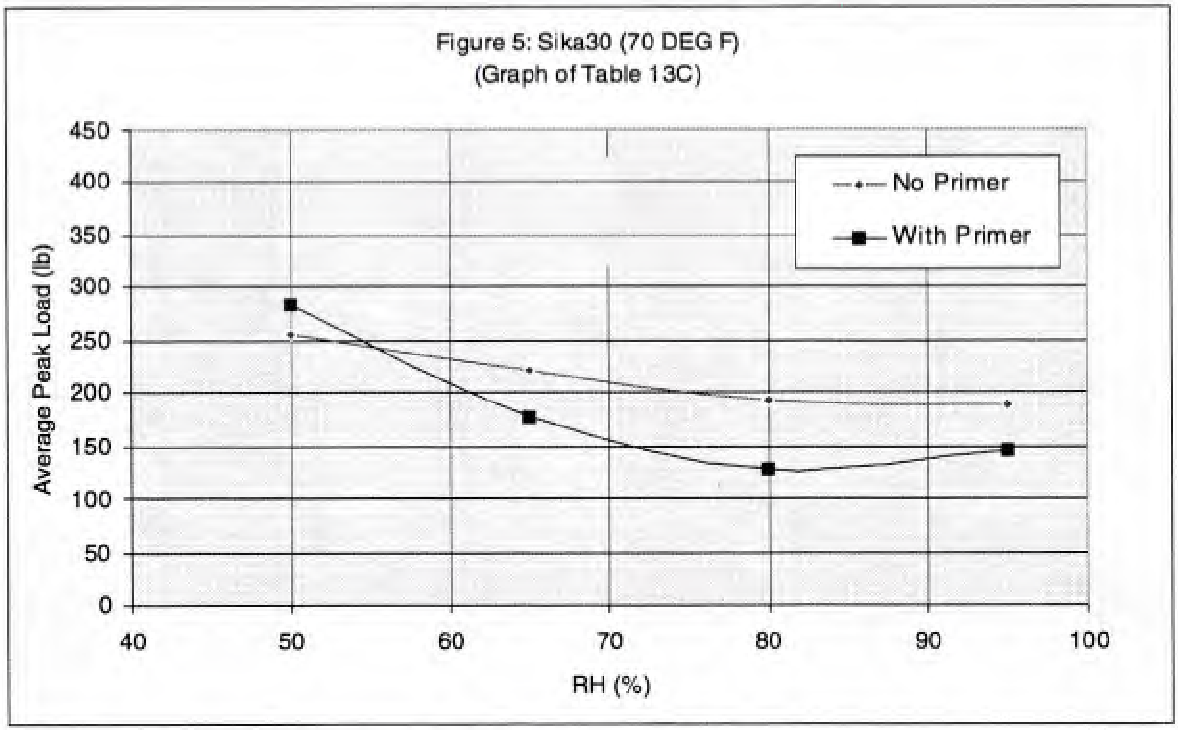


Figure 5: Sika 30 at 70°F

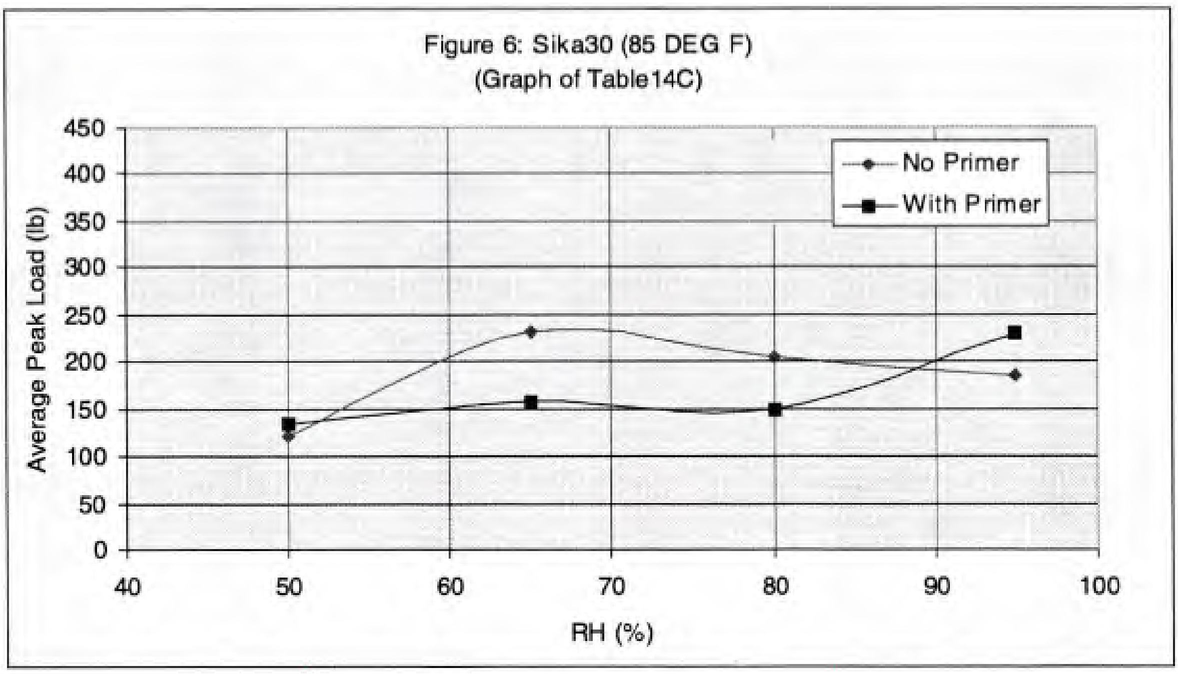


Figure 6: Sika 30 at 85°F

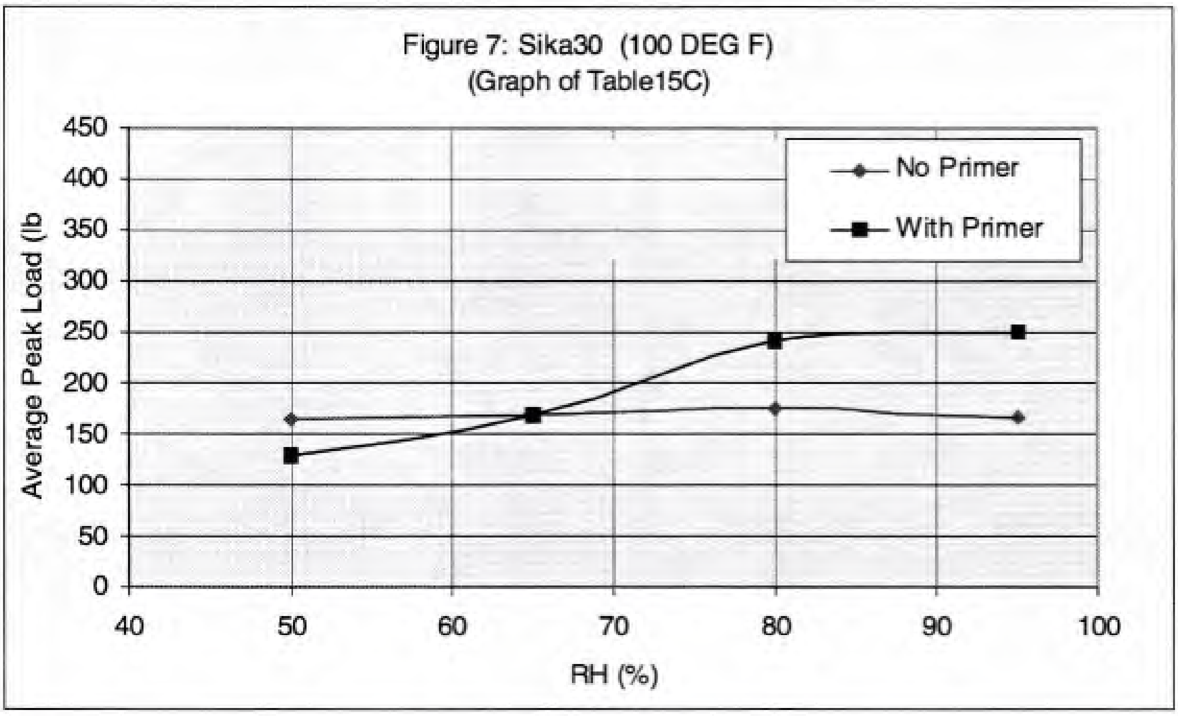


Figure 7: Sika 30 at 100°F

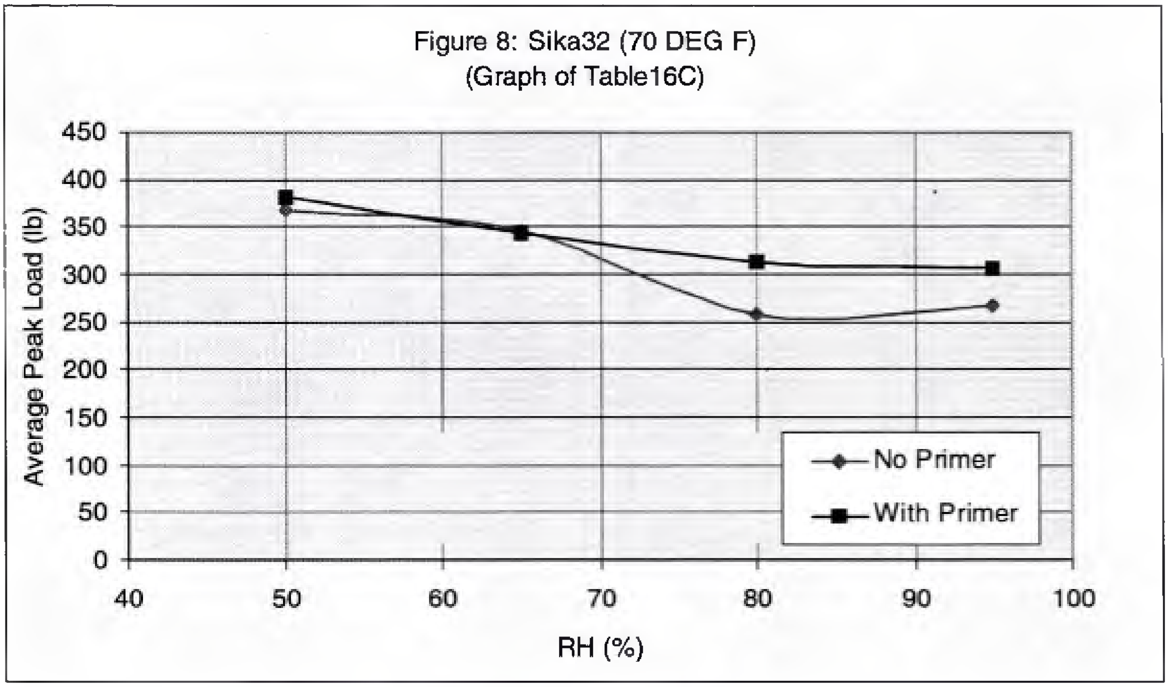


Figure 8: Sika 32 at 70°F

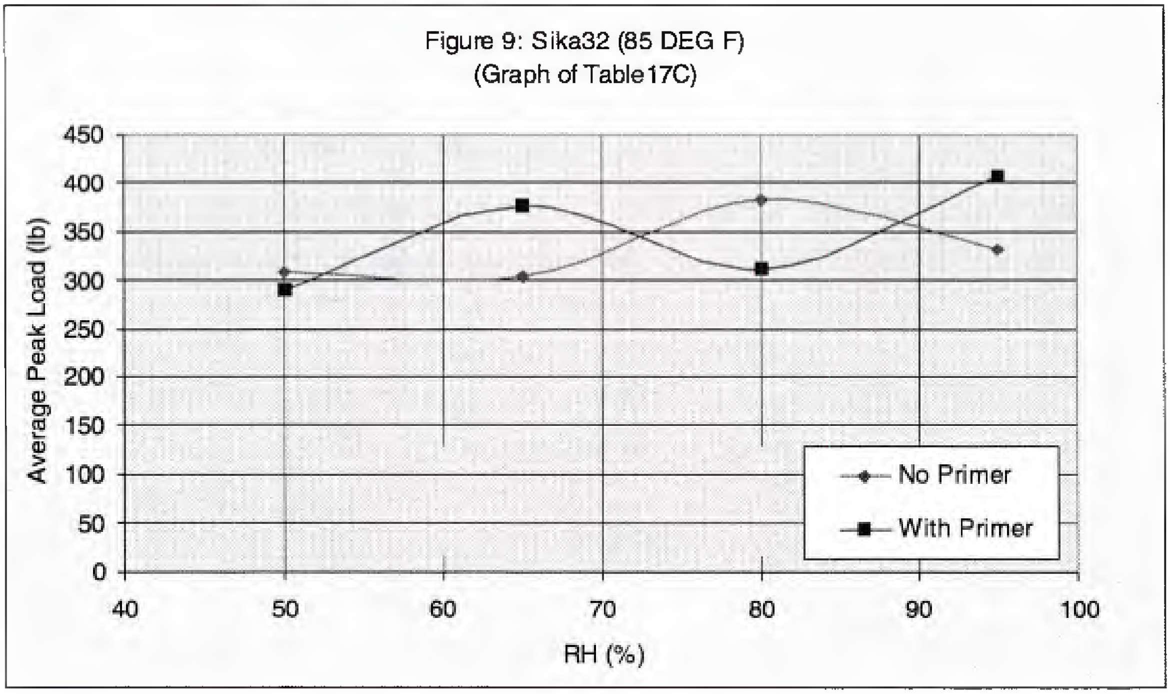


Figure 9: Sika 32 at 85°F

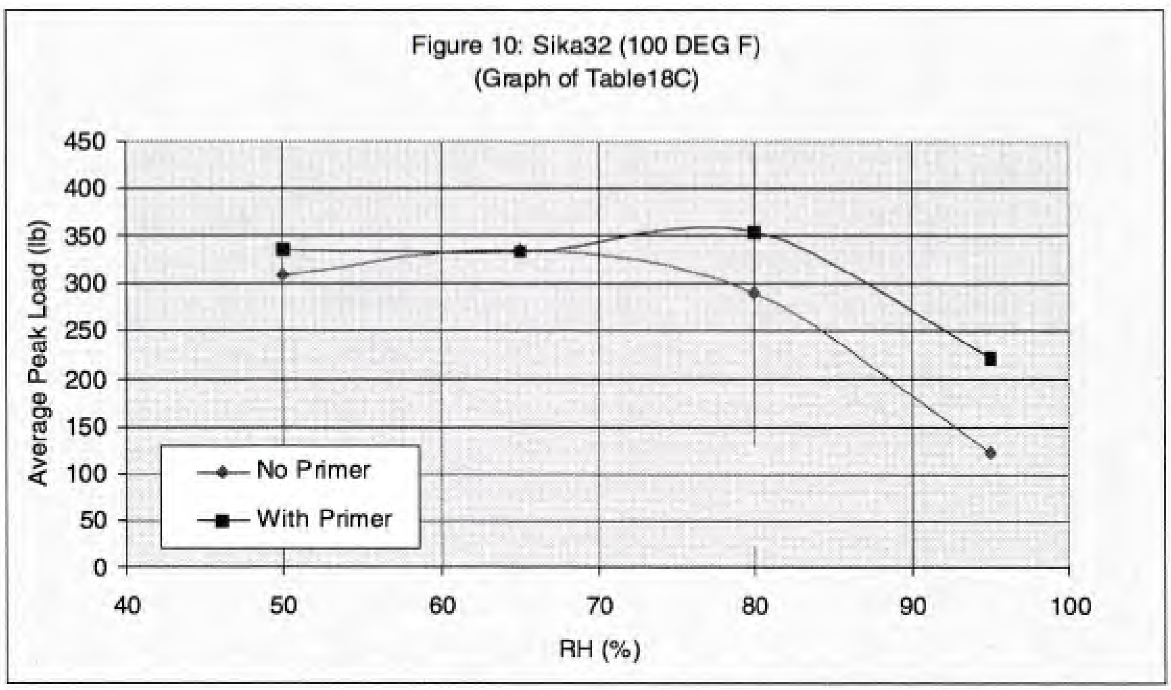


Figure 10: Sika 32 at 100°F

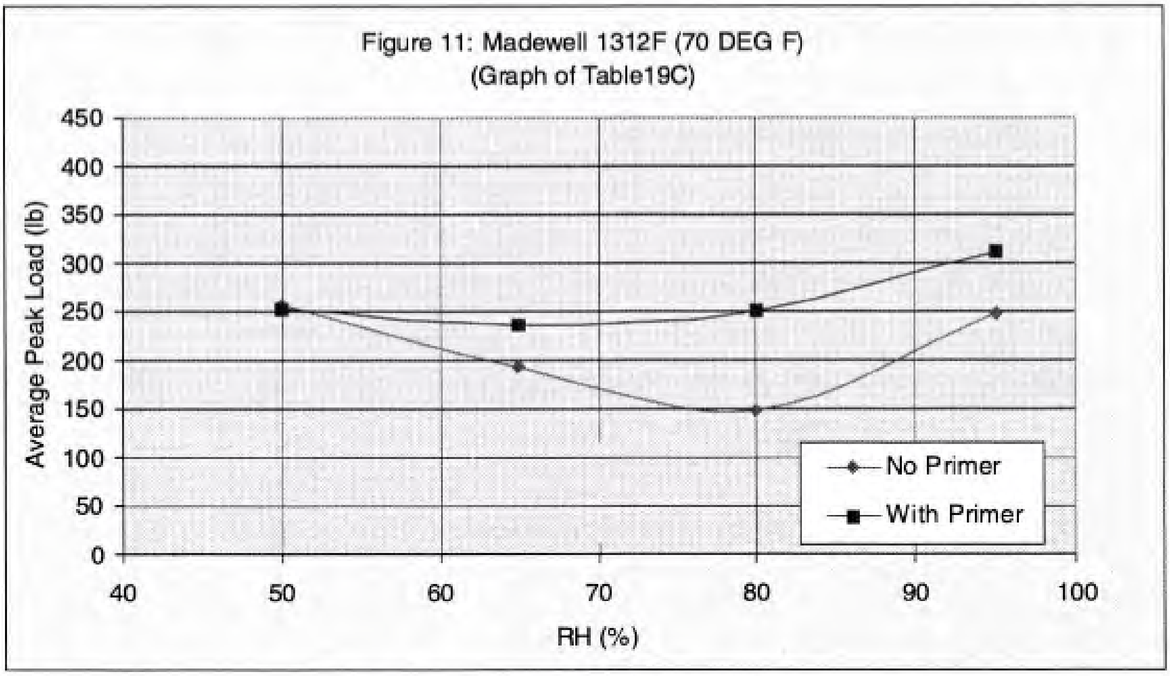


Figure 11: Madewell 1312 at 70ºF

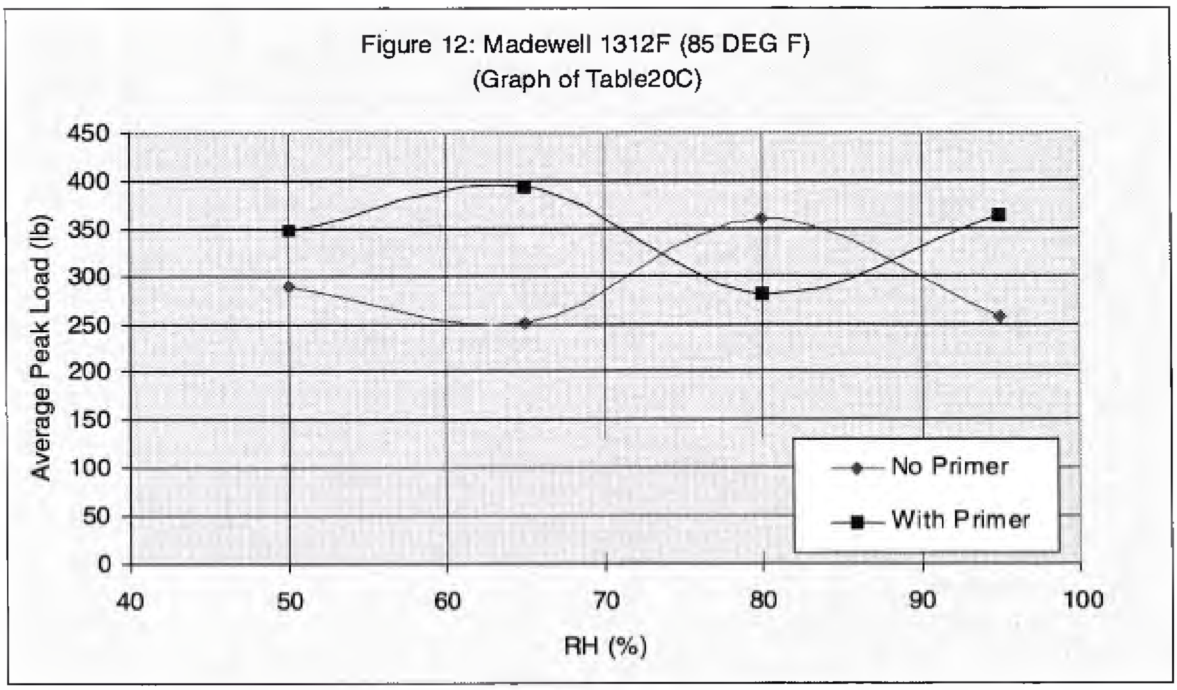


Figure 12: Madewell 1312 at 85ºF

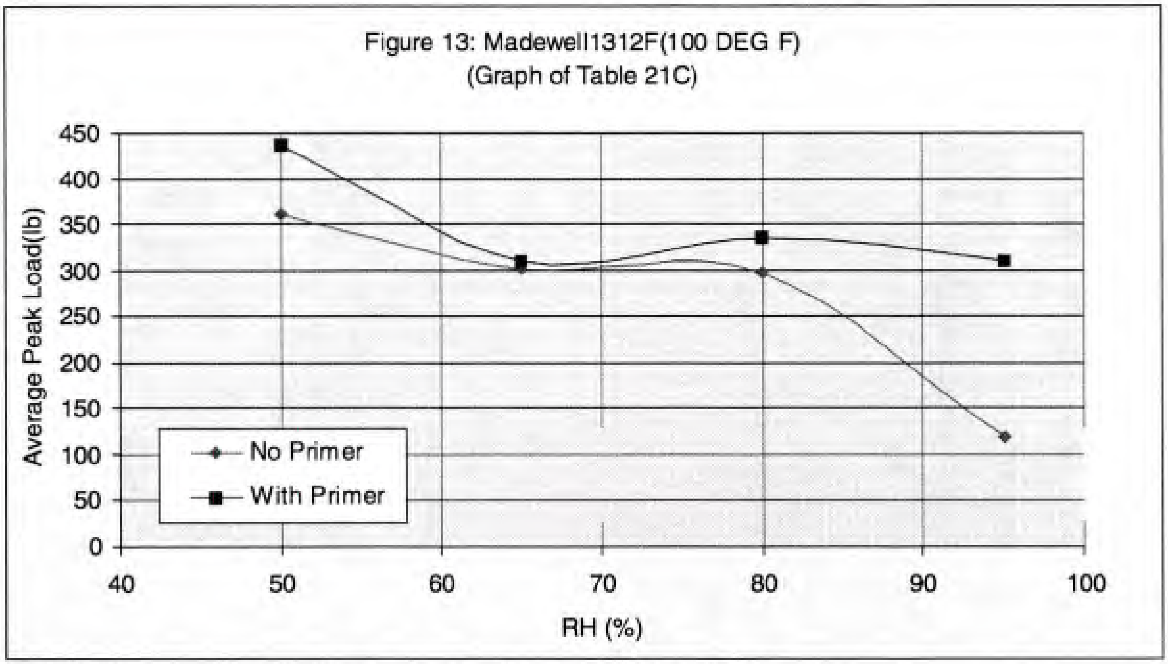


Figure 13: Madewell 1312 at 100°F

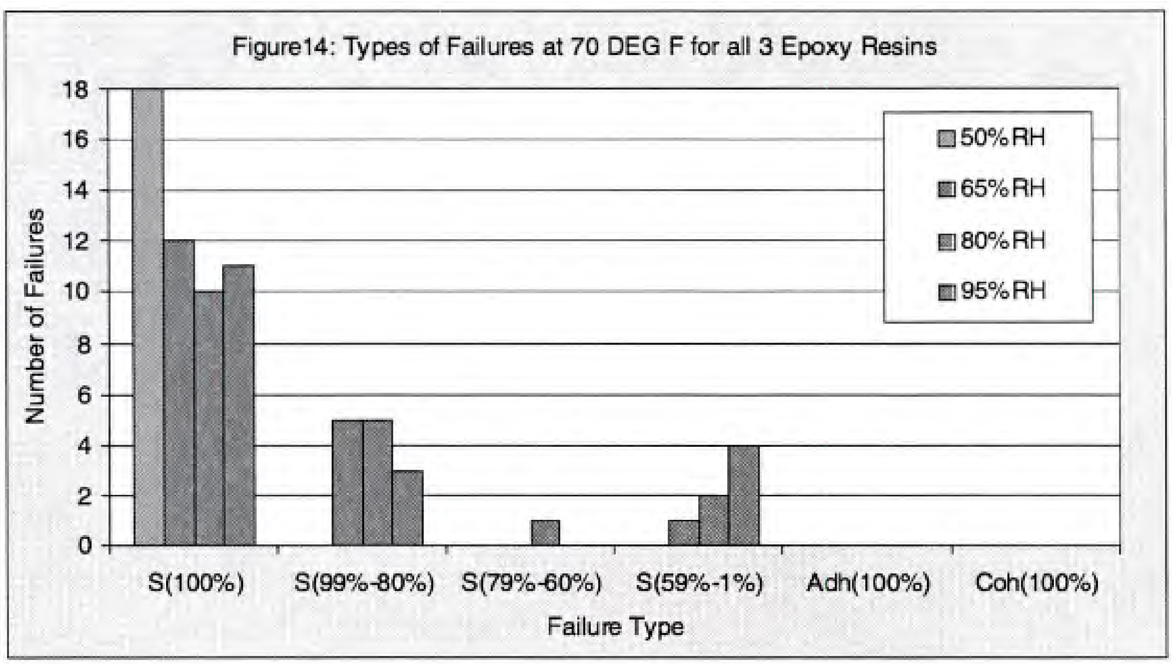


Figure 14: Failure Types at 70°F

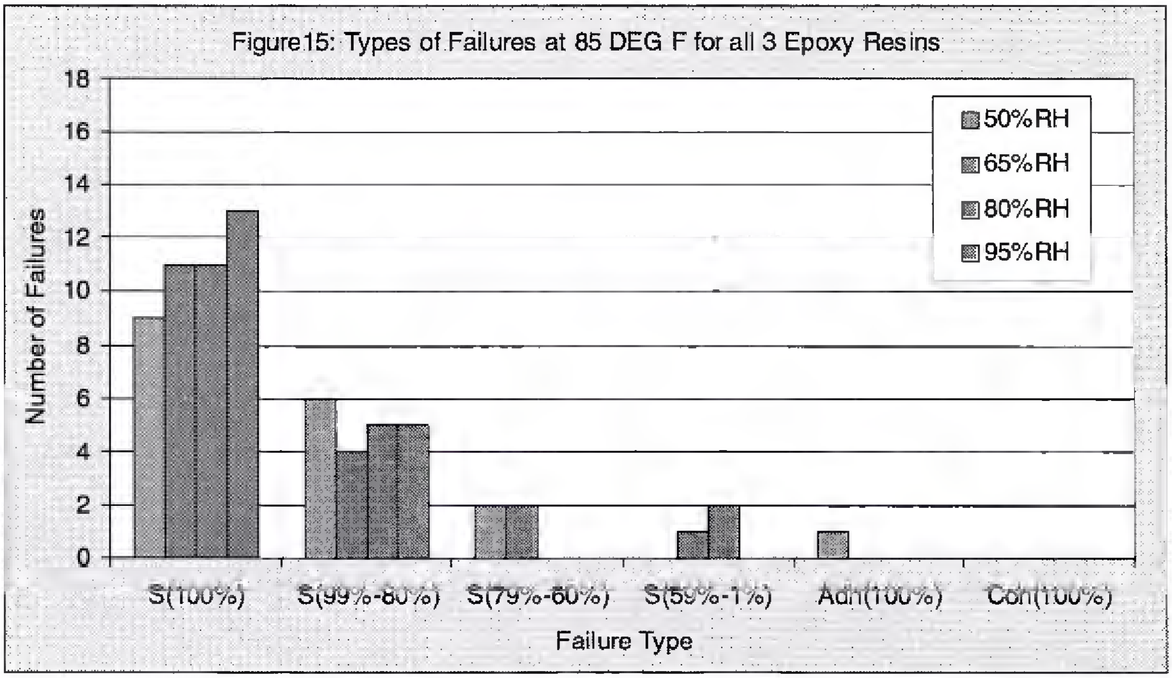


Figure 15: Failure Types at 85°F

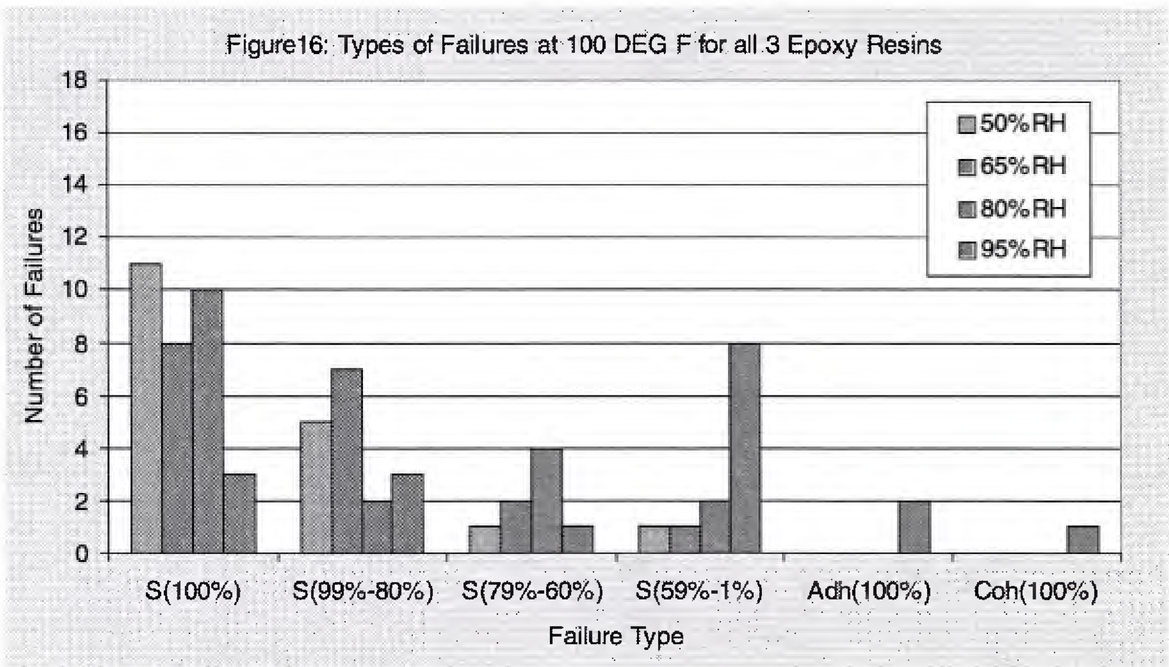


Figure 16: Failure Types at 100°F

APPENDIX A

This appendix includes data for the various adhesives and primers used. Data were obtained and summarized from the manufacturer's specifications.

Table A1. Properties of Sikadur 30

TYPICAL DATA FOR SIKADUR 30 *(Material and curing conditions @ 73F and 50% R.H.)* *Sikadur 30 is a 2-component, 100% solids, moisture-tolerant structural epoxy paste adhesive*			
SHELF LIFE	2 years in original, unopened containers.		
STORAGE CONDITIONS	Store dry at 40-95F. Condition material to 65-85F before using.		
COLOR	Light gray.		
MIXING RATIO	Component 'A': Component 'B' = 3:1 by volume.		
CONSISTENCY	Non-sag paste.		
POT LIFE	Approximately 70 minutes @ 73F. (1 qt.)		
TENSILE PROPERTIES (ASTM D-638)			
7 day Tensile Strength	3,600 psi (24.8 MPa)		
Elongation at Break	1%		
Modulus of Elasticity	6.5×10^5 psi		
FLEXURAL PROPERTIES (ASTM D-790)			
14 day Flexural Strength (Modulus of Rupture)	6,800 psi (46.8 MPa)		
Tangent Modulus of Elasticity In Bending	1.7×10^6 psi		
SHEAR STRENGTH (ASTM D-790) 14 day Shear Strength	3,600 psi (24.8 MPa)		
BOND STRENGTH (ASTM C-882): Hardened Concrete to Hardened Concrete			
2 day (moist cure) Bond Strength	2,700 psi (18.6 MPa)		
2 day (dry cure) Bond Strength	3,200 psi (22.0 MPa)		
14 day (moist cure) Bond Strength	3,100 psi (21.3 MPa)		
Hardened Concrete to Steel			
2 day (moist cure) Bond Strength	2,600 psi (17.9 MPa)		
2 day (moist cure) Bond Strength	3,000 psi (20.6 MPa)		
14 day (moist cure) Bond Strength	2,600 psi (17.9 MPa)		
DEFLECTION TEMPERATURE (ASTM D-648)			
7 day Deflection Temperature (fiber stress loading = 264 psi)	118F (47C)		
WATER ABSORPTION (ASTM D-570)			
24 hours Total Water Absorption	0.03%		
COMPRESSIVE PROPERTIES (ASTM D-695)			
Compressive Strength, psi	**40F**	**73F**	**90F**
4 hour	-		5,500 (37.9 MPa)
8 hour	-	3,500 (24.1 MPa)	6,700 (46.2 MPa)
16 hour	-	6,700 (46.2 MPa)	7,400 (51.0 MPa)
1 day	750 (5.1 MPa)	7.800 (53.7 MPa)	7,800 (53.7 MPa)
3 day	6,800 (46.8 MPa)	8,300 (57.2 MPa)	8,300 (57.2 MPa)
7 day	8,000 (55.1 MPa)	8,600 (59.3 MPa)	8,600 (59.3 MPa)
14 day	8,500 (58.6 MPa)	8,600 (59.3 MPa)	8,900 (61.3 MPa)
28 day	8,500 (58.6 MPa)	8,600 (59.3 MPa)	9,000 (62.0 MPa)
MODULUS OF ELASTICITY, PSI 7 day	3.9×10^5 psi		

Table A2. Properties of Sikadur 32 Hi-Mod

TYPICAL DATA FOR SIKADUR 32 HI-MOD
(Material and curing conditions @ 73F and 50% R.H.)
Sikadur 32 is a 2-component, 100% solids, moisture-tolerant structural epoxy paste adhesive

SHELF LIFE	2 years in original, unopened containers.
STORAGE CONDITIONS	Store dry at 40-95F. Condition material to 65-75F before using.
COLOR	Concrete gray.
MIXING RATIO	Component 'A': Component 'B' = 1:1 by volume.
VISCOSITY	Approximately 2,800 cps.
POT LIFE	Approximately 30 minutes. (60 gram mass)

CONTACT TIME

40F	73F	90F
14-16 hr	3.5-4 hr	1.5-2 hr

COMPRESSIVE MODULUS, PSI: 7 day 2.0×10^5 psi (1379.3 MPa)

TENSILE PROPERTIES (ASTM D-638)

7 day	Tensile Strength	5,100 psi (35.1 MPa)
	Elongation at Break	1.8%
14 day	Modulus of Elasticity	3.2×10^5 psi (2206.9 MPa)

FLEXURAL PROPERTIES (ASTM D-790)

14 day	Flexural Strength (Modulus of Rupture)	7,400 psi (51 MPa)
	Tangent Modulus of Elasticity In Bending	4.7×10^5 psi

SHEAR STRENGTH (ASTM D-790) 14 day Shear Strength 5,900 psi (40.6 MPa)

WATER ABSORPTION (ASTM D-570) 24 hours Total Water Absorption 0.79%

DEFLECTION TEMPERATURE (ASTM D-648)

7 day	Deflection Temperature (fiber stress loading = 264 psi)	121F

BOND STRENGTH (ASTM C-882): Hardened Concrete to Hardened Concrete

2 day	(moist cure)	Plastic Concrete to Hardened Concrete	1,700 psi
14 day	(moist cure)	Plastic Concrete to Hardened Concrete	2,400 psi
		Plastic Concrete to Steel	1,900 psi

COMPRESSIVE PROPERTIES (ASTM D-695)

Compressive Strength, psi	**40F**	**73F**	**90F**
8 hour	-	-	100 (.18 MPa)
16 hour	-	2,400 (16.5 MPa)	4,500 (31 MPa)
1 day	-	4,600 (31.7 MPa)	6,400 (44.1 MPa
3 day	800 (5.5 MPa)	8,100 (55.8 MPa)	8,200 (56.5 MPa)
7 day	8,100 (55.9 MPa)	10,300 (71 MPa)	8,200 (56.5 MPa)
14 day	8,100 (55.9 MPa)	10,300 (71 MPa)	8,200 (56.5 MPa)
28 day	8,800 (60.7 MPa)	10,300 (10,300 MPa)	8,200 (56.5 MPa)

Table A3. Properties of Sikadur 55 SLV

TYPICAL DATA FOR SIKADUR 55 SLV
(Material and curing conditions @ 73F (23C) and 50% R.H.)
Sikadur 55 is a 2-component, 100% solids, moisture-tolerant epoxy crack healer/penetrating sealer. It is a super low viscosity hi-strength adhesive formulated specifically for grouting dry and damp cracks.

SHELF LIFE	2 years in original, unopened containers.			
STORAGE CONDITIONS	Store dry at 40-95F (4-35C). Condition material to 65-75F before using.			
COLOR	Clear, amber			
MIXING RATIO	Component 'A': Component 'B' = 2:5:1 by volume.			
VISCOSITY	Approximately 95 cps			
POT LIFE	Approximately 25 minutes			
TACK FREE TIME		**40F**	**60F**	**73F**
		-	16 hours	6 hours

TENSILE PROPERTIES (ASTM D-638)

		40F	60F	73F
7 day	Tensile Strength	-	5,000psi (34.4 MPa)	7,500psi (51.7 MPa)
	Elongation at Break	-	1.6%	2.3%

BOND STRENGTH (ASTM C-882)

Hardened Concrete to Hardened Concrete		
2 day	(moist cure)	1,400 psi (9.6 MPa)
14 day	(moist cure)	2,700 psi (18.6 MPa)
Hardened Concrete to Steel		
2 day	(moist cure)	1,800 psi (12.4 MPa)
14 day	(moist cure)	2,000 psi (13.8 MPa)

FLEXURAL PROPERTIES (ASTM D-790)

7 day	Flexural Strength	9,500 psi (65.5 MPa)
	Tangent Modulus of Elasticity	4.8×10^5 psi

SHEAR STRENGTH (ASTM D-790)	14 day	Shear Strength	7,600 psi (52.4 MPa)
DEFLECTION TEMPERATURE (ASTM D-648)	7 day		120F (49C)

WATER ABSORPTION (ASTM D-570)

7 day Total Water Absorption (24 hour Immersion) 0.61%

COMPRESSIVE PROPERTIES (ASTM D-695)

Compressive Strength, psi	**40F**	**73F**	**90F**
1 day	-	250 (1.7 MPa)	5,150 (35.5 MPa)
3 day	1,200 (8.2 MPa)	11,600 (80 MPa)	12,900 (88.9 MPa)
7 day	7,900 (54.4 MPa)	13,700 (94.4 MPa)	14,800 (102 MPa)
14 day	12,600 (86.8 MPa)	14,000 (96.5 MPa)	15,300 (105.5 MPa)
28 day	13,000 (89.6 MPa)	14,000 (96.5 MPa)	15,800 (108.9 MPa)

MODULUS OF ELASTICITY, PSI 7 day $3.7 \text{ X } 10^5$ psi

Table A4. Properties of Madewell 1312F

TYPICAL DATA FOR MADEWELL 1312F *Madewell 1312F is a 2-component, 100% solids, epoxy saturant (resin) specifically designed for use with glass, carbon or other synthetic fiber reinforcement systems for protection and/or reinforcement of concrete, steel, wood or composite structures.*			
SHELF LIFE			
STORAGE CONDITIONS			
COLOR	Transparent blue		
MIXING RATIO	Component 'A': Component 'B' = : by volume.		
VISCOSITY	Approximately cps		
POT LIFE	Approximately 45 minutes at 100F, longer at lower temperatures		
TACK FREE TIME	**40F**	**60F**	**73F**
	-		
TENSILE PROPERTIES (ASTM D-638)	**40F**	**60F**	**73F**
7 day Tensile Strength	-		
Elongation at Break	-		
BOND STRENGTH (ASTM C-882)			
Hardened concrete to hardened			
2 day (moise cure)			
14 day (moist cure)			
Hardened Concrete to Steel			
2 day (moist cure)			
14 day (moist cure)			
FLEXURAL PROPERTIES (ASTM D-790)			
7 day Flexural Strength			
Tangent Modulus of Elasticity			
SHEAR STRENGTH (ASTM D-790)			
DEFLECTION TEMPERATURE (ASTM D-648)	7 day		
WATER ABSORPTION (ASTM D-570)			
7 day Total Water Absorption (24 hour Immersion)			
COMPRESSIVE PROPERTIES (ASTM D-695)			
Compressive Strength, psi	**40F**	**73F**	**90F**
1 day	-		
3 day	-		
7 day	-		
14 day	-		
28 day	-		
MODULUS OF ELASTICITY, PSI	7 day		

Table A5. Properties of Madewell 927

TYPICAL DATA FOR MADEWELL 927 *Madewell 927 is a 2-component, 100% solids, penetrating epoxy primer and sealer for porous substrates such as wood, concrete and other cementitious surfaces. It has very low viscosity, deep penetrating action, and can be applied to damp surfaces.*				
SHELF LIFE	1 year minimum			
STORAGE CONDITIONS				
COLOR	Semi-transparent amber			
MIXING RATIO	Component 'A': Component 'B' = : by volume.			
VISCOSITY	Approximately cps			
POT LIFE				
TACK FREE TIME	**40F**	**60F**	**70F**	
	-		24 hours	
TENSILE PROPERTIES (ASTM D-638)				
	40F	**60F**	**73F**	
7 day Tensile Strength	-			
Elongation at Break	-			
BOND STRENGTH (ASTM C-882)				
Hardened concrete to hardened				
2 day (moise cure)				
14 day (moist cure)				
Hardened Concrete to Steel				
2 day (moist cure)				
14 day (moist cure)				
FLEXURAL PROPERTIES (ASTM D-790)				
7 day Flexural Strength				
Tangent Modulus of Elasticity				
SHEAR STRENGTH (ASTM D-790)				
DEFLECTION TEMPERATURE (ASTM D-648)	7 day			
WATER ABSORPTION (ASTM D-570)				
7 day Total Water Absorption (24 hour Immersion)				
COMPRESSIVE PROPERTIES (ASTM D-695)				
Compressive Strength, psi	**40F**		**73F**	**90F**
1 day	-			
3 day	-			
7 day	-			
14 day	-			
28 day	-			
MODULUS OF ELASTICITY, PSI	7 day			

5. Beran, J.A., "Correlation of Surface Chloride Concentration of a Pile Exposed To The Marine Environment To the Adhesiveness of a Commercially Available Epoxy," Contract Report CR-98.19-SHR, Naval Facilities Engineering Service Center, September 1998.

NAVAL FACILITIES ENGINEERING SERVICE CENTER
Port Hueneme, California 93043-4370

Contract Report
CR-98.19-SHR

CORRELATION OF SURFACE CHLORIDE CONCENTRATION OF A PILE EXPOSED TO THE MARINE ENVIRONMENT TO THE ADHESIVENESS OF A COMMERCIALLY AVAILABLE EPOXY

An Investigation Conducted By

J. A. Beran, Ph.D.
Department of Chemistry
Texas A&M University
Kingsville, TX 78363

September 1998

EXECUTIVE SUMMARY

The U.S. Navy is studying the use of carbon fiber reinforced plastic (CFRP) for retrofitting the strengths of piers and piles. The CFRP is adhered to the piers and piles with a commercial adhesive. Several of those investigations have been (and are being) conducted at the Navy Facilities Engineering Service Center (NFESC) at Port Hueneme, California. One area of investigation relates to the effectiveness of the epoxies under different environmental exposures of the concrete. Most other investigations have been conducted with "virgin" concrete samples of pier and pile.

A pile that has been exposed to marine conditions for approximately 4 years was removed from the harbor at Port Hueneme. The pile was transferred to the NFESC service yard at which point several parameters for the adhesiveness of an epoxy to the concrete pile surface were investigated.

The effect of chloride levels, with and without hydroblasting surface preparation, on the adhesiveness of an epoxy applied to the surface of the pile, with and without the application of primer, was investigated. The laboratory analysis of chloride levels on the surface of the concrete pile and the pull-off forces of the adhesive were the principal parameters upon which the research was focused.

In general it was found that adhesion would somewhat decrease with chloride content. The use of hydroblasting and pretreatment with a primer increased the adhesion in all cases, and both are recommended.

Keywords: chloride, epoxy, carbon fibers, piles, reinforced concrete

TABLE OF CONTENTS

1. INTRODUCTION

This investigation covers the effect of chloride concentrations at the surface of two piles on the adhesiveness of two epoxies (Sikadur 30 and 32) commonly used for adhering carbon fiber reinforced plastic to concrete. The fifteen-foot concrete piles had been exposed to the marine environment at Port Hueneme, CA for approximately 4 years (Figure 1). The vertical position in the harbor permitted the piles to be exposed to the atmosphere above the average high tide level to below the average low tide level in the water. They were removed from the harbor and relocated to the service yard of the NFESC.

Figure 1. Piles in the harbor

Several parameters were studied to determine the adhesiveness of the epoxy as a function of the changes in chloride concentrations from top to bottom, mostly along pile #1. The chloride and adhesion tests were conducted at approximately one-foot intervals from top (identified as the pile end that was out of the water) to bottom, along the pile. The pile extended from about 7 ¾ feet *above* the average high tide level to about 1 foot *below* the average low tide level for May 1998 (Figure 2).

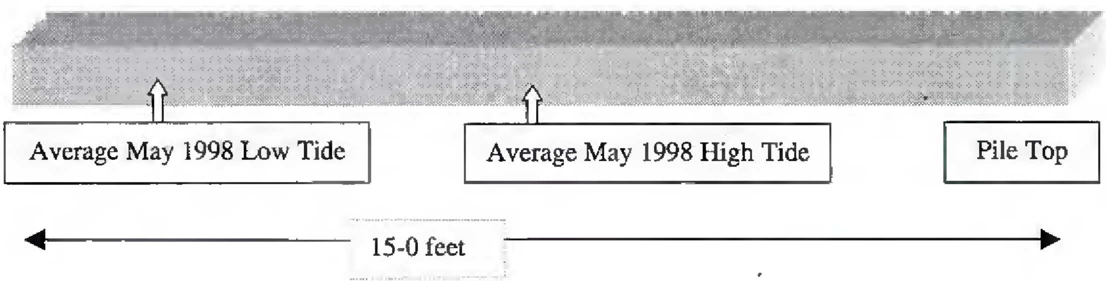


Figure 2. Pile Schematic.

2. PILE PREPARATION

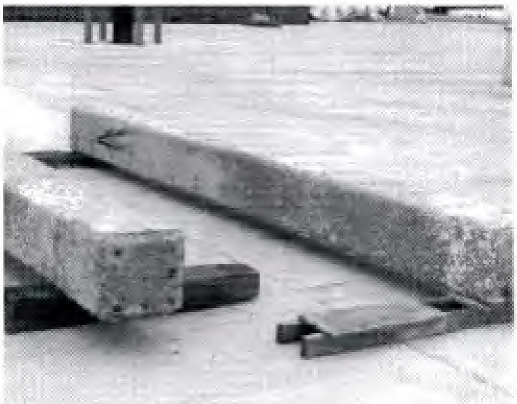
Two vertically positioned piles located in the Port Hueneme harbor were removed and transferred to the NFESC service area. The piles were distinguished by numbers #1 and #2 with faces identified as A, B, C, and D. Face A of both piles is the top-side-up face of the concrete forms, faces B, C, and D are all form faces (sides and bottom, smoother and less pitted). The crustacean material from Pile #1, Faces A and B and Pile #2, Faces A and D were removed by hand scrapping. The faces were then hydroblasted with a power sprayer at 200 psi. All data of this report were obtained from Pile #1.

From the tidal chart of May, 1998, the average high tide line was about 7 ¾feet from the "top" of the pile and the average low tide line was about 13 ft from the "top" of the pile.

The pile was marked at approximately one-foot intervals, starting at the ½foot mark at the top of the pile.

2.1 Application of Test Dollies

Carbon fiber reinforced plastic (CFRP) circles were formed to the size of the aluminum dollies. CFRP circles and dollies were then adhered to Pile #1 (Figure 3).



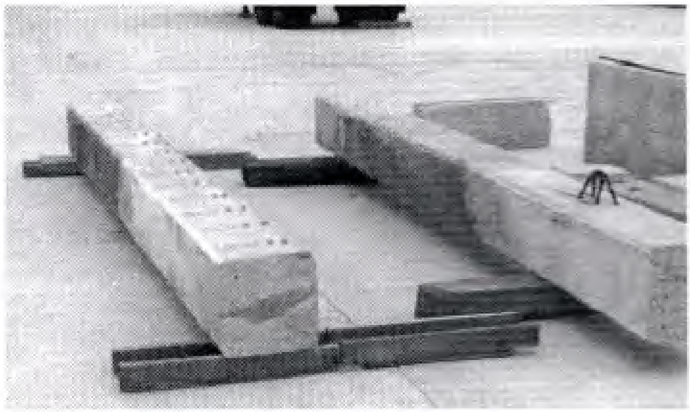


Figure 3. Piles at NFESC showing adhered dollies.

Sikadur 32 epoxy was used to adhere two aluminum dollies with CFRP circles at *each* of the marked positions of Pile #1, Face A. Sikadur 30 epoxy was used to adhere two aluminum dollies with CFRP circles at *each* of the marked positions of Pile #1, Face B. Finally, Sikadur 30 was also used to adhere two aluminum dollies with CFRP circles at *alternate* (with an additional application at the 7 ½foot position) marked positions of Pile #1, Face D. Temperature and relative humidity conditions were recorded.

2.2 Procedure for Testing the Adhesiveness of Epoxy

The epoxy (adhered to the dolly and the CFRP circle) was allowed to cure for at least seven (7) days.

A 500 psi elcometer (Figure 4) was used to remove the dollies and the force (or tensile stress) required was recorded. Since two dollies were available at each marked position along the pile, the average of the two tensile strength values was used for a subsequent analysis of the data. The diameter of a dolly was 0.787 inch (20 mm) and its area was 0.487 square inch.

Figure 4. Elcometer

3. CHLORIDE ANALYSIS

3.1 Concrete Sample Extraction

An impact drill was used to extract surface samples from Pile #1, Faces B and D at the marked positions, collecting about 20 grams of concrete sample at each location. Samples were less than one-quarter inch in depth, as shown at right in Figure 5 photos.

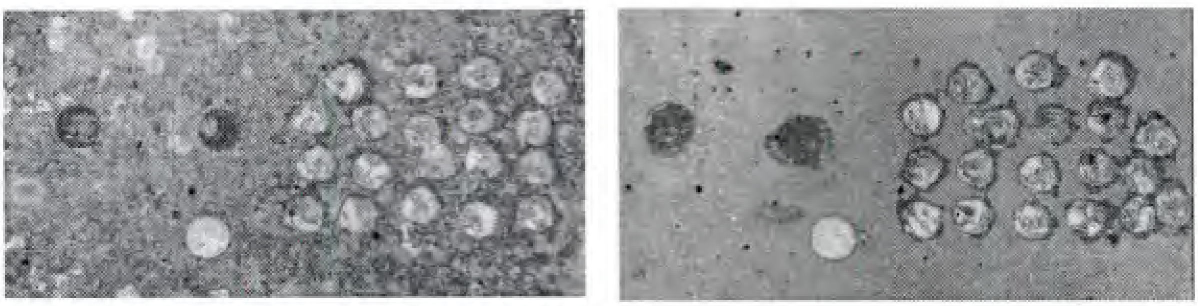


Figure 5. Pile #1, Faces B and D.

Approximately 10 grams of the very finely divided (powdered) part of the sample was subsequently used for the chloride analysis.

3.2 Chloride Measurement

The chloride analyses were performed using a chloride ion selective electrode. The meter used for the analysis was standardized using standardized solutions of chloride ion, 10 PPM, 100 PPM and 1000 PPM. All checks/tests indicated that the meter was performing according to the manufacturer's calibration procedures; additionally, excellent linearity with the standardized solutions was obtained.

3.3 Experimental Procedure for Preparing Concrete Samples

The experimental procedure for preparing the concrete samples from the marked positions of the pile for chloride analysis was (in brief) [1]:

- A quantitative mass (about 10 grams) of very finely divided concrete sample was measured.
- Sample was mixed with 50.0 mL of deionized (DI) water, heated to boiling for 5 minutes (covered), and allowed to set for 24 hours.
- The sample solution was filtered; the concrete sample was washed with 50.0 mL of DI water, acidified with 1:1 nitric acid, and momentarily heated to boiling.
- The sample solution was cooled to ambient temperature, adjusted to an approximate pH of 5 with a potassium hydroxide solution, and diluted to volume with DI water in a 100 mL volumetric flask.
- A 2 mL aliquot of ion strength adjuster (ISA) solution was pipetted into the 100 mL volumetric flask before the chloride concentration was determined.

3.4 Experimental Procedure for Chloride Analysis of Concrete Samples

The measurement of the chloride concentrations of the surface samples at the marked positions on the pile was as follows (Figure 6) [1]:

- The chloride meter was calibrated with standard solutions of 10 ppm (parts per million by mass), 100 ppm, and 1000 ppm chloride concentrations before and after each set of analyses. A standardization curve of the data (millivolt vs. ppm chloride) was constructed.
- Millivolt readings for each sample solution were determined (10 minutes were allowed for each measurement to reach stability). The standardization curve was used to determine the chloride concentrations (at the ppm levels) in the sample solution.

Figure 6. Chloride analysis setup

- Calculations: from the known volume of the sample solution, the mass of chloride in the sample solution was calculated. From the measured mass of the original sample, the ppm chloride of the surface sample at each of the marked positions along the pile was calculated.

4. ADHESIVE FORCES ON SURFACE OF PILE #1

4.1 Adhesion of Sikadur 30 Epoxy on Hydroblasted Surface.

Figure 7 shows the actual pull-off forces of the dollies placed at 1 foot intervals along Pile #1, Face B. The "connected" data points represents the average pull-off force at each sample site.

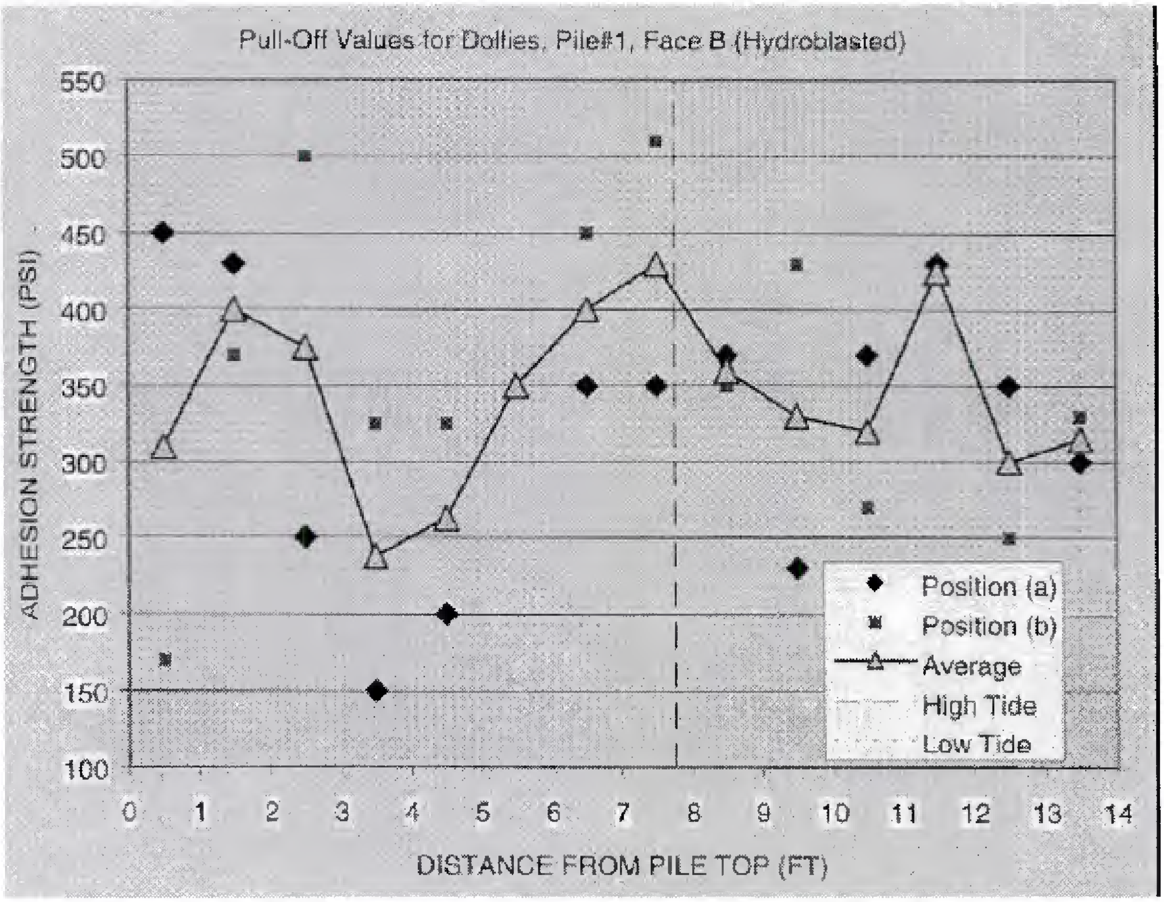


Figure 7. Pull-off stress required to remove the adhered dollies along the pile with a hydroblasted surface

The average "tidal zone" for the month of May, 1998 was from about 7 ¾ft to 13 ½ft along the pile. From the data of Figure 7, there appears to be a somewhat higher adhesiveness just above the average high tide level (at 7½feet from the pile top), and then again at a much larger distance from the high tide level (at 18 inches from the pile top) . The latter positioning could be consider a region that lies above the "splash zone" of the water. Within the tidal zone and lower, there seems to be little variation in adhesiveness of the epoxy.

4.2 Adhesion of Sikadur 30 Epoxy on Nonhydroblasted Surface

Figure 8 shows the actual pull-off forces of the dollies placed at 1 foot intervals along Pile #1, Face D. The "triangular" data points represent the average pull-off force at each sample site.

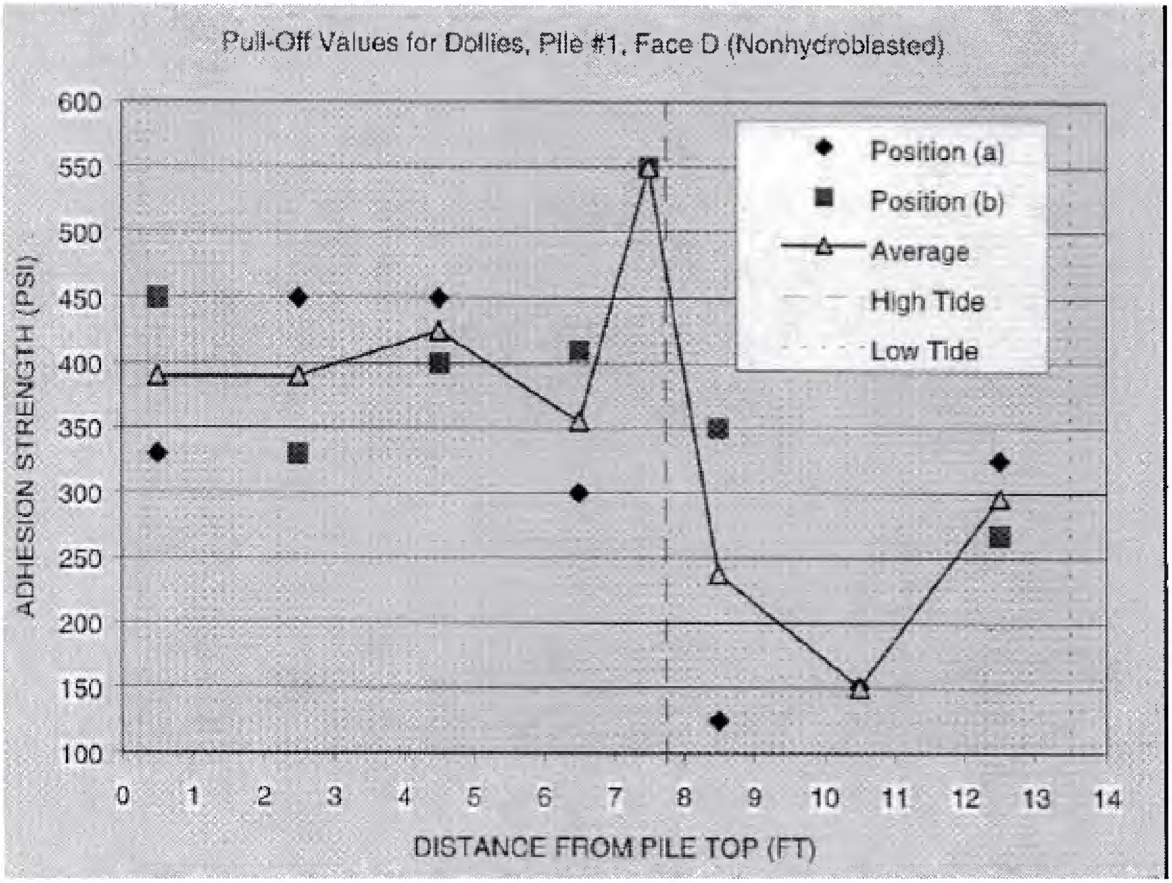


Figure 8. Pull-off stress required to remove the adhered dollies along the pile with a non-hydroblasted surface

With fewer data points than on the hydroblasted surface (Figure 7), the adhesiveness of the epoxy on a non-hydroblasted surface (Figure 8) seems to somewhat correlate with the data of the hydroblasted surface, with a higher adhesiveness just above the average high tide level (at 7½feet from the pile top).

4.3 Adhesion of Sikadur 30 Epoxy on Hydroblasted and Nonhydroblasted Surface

Figure 9 compares the pull-off forces for the dollies adhered to a hydroblasted surface (Face B) with those of a nonhydroblasted surface (Face D), using the Sikadur 30 epoxy.

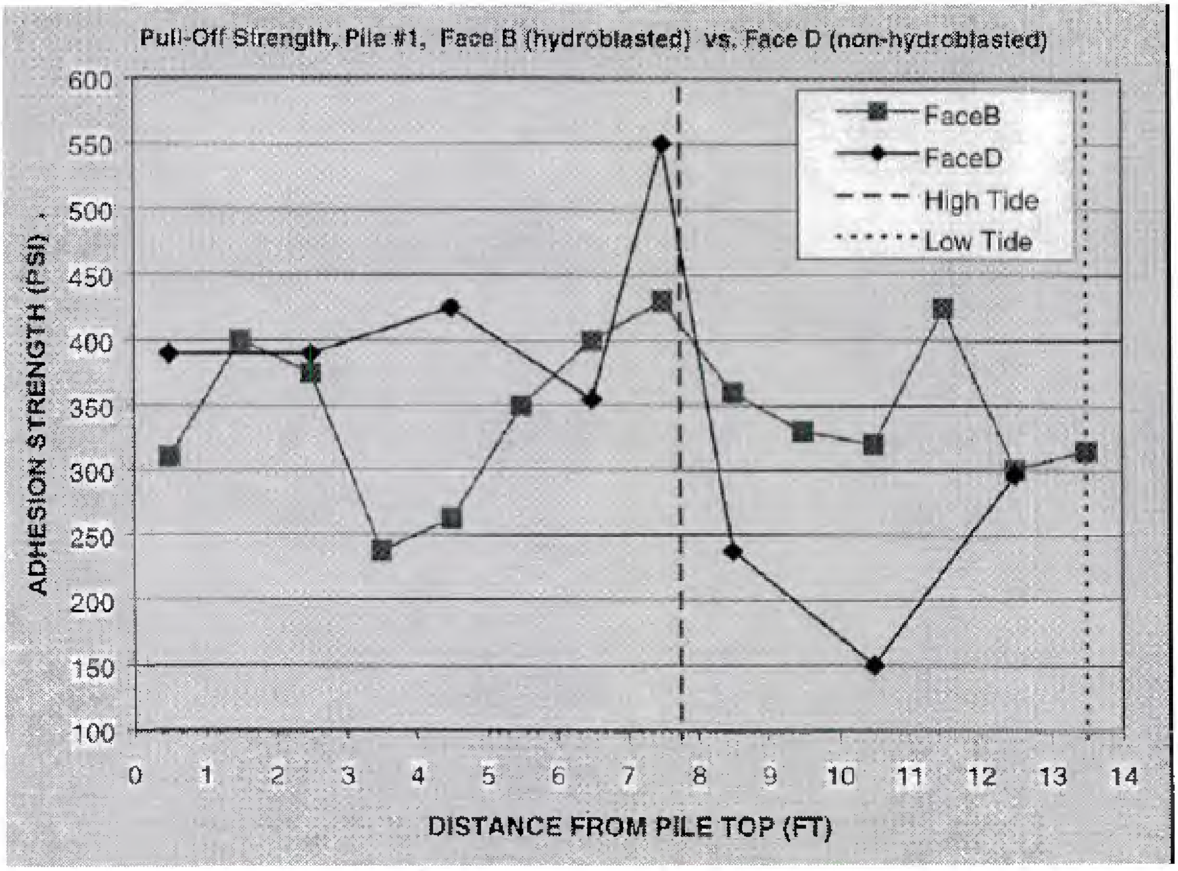


Figure 9. Comparison of the Adhesiveness of Sikadur 30 Epoxy on a Hydroblasted Surface (Face B) and a Nonhydroblasted Surface (Face D)

Figure 9 summarizes the data of Figures 7 and 8. The correlation of the adhesiveness of Sikadur 30 between the two surfaces with different preparations, while not quantitative, does appear to exist. It also appears that hydroblasting resulted in minimum adhesion values around 240 psi, in excess of the minimum 150 psi for nonhydroblasting. The improvement is more obvious in the intertidal zone.

4.4 Adhesion of Sikadur 30 Epoxy on Hydroblasted Surface with a Pretreatment of Sikadur 55 Primer

Figure 10 shows the actual and average pull-off forces of the dollies placed at 1-foot intervals along Pile #1, Face B in which the surface had not only been hydroblasted but also pretreated with Sikadur 55 primer. The "connected" triangular data points represent the average of the two dollies at that location along the pile surface.

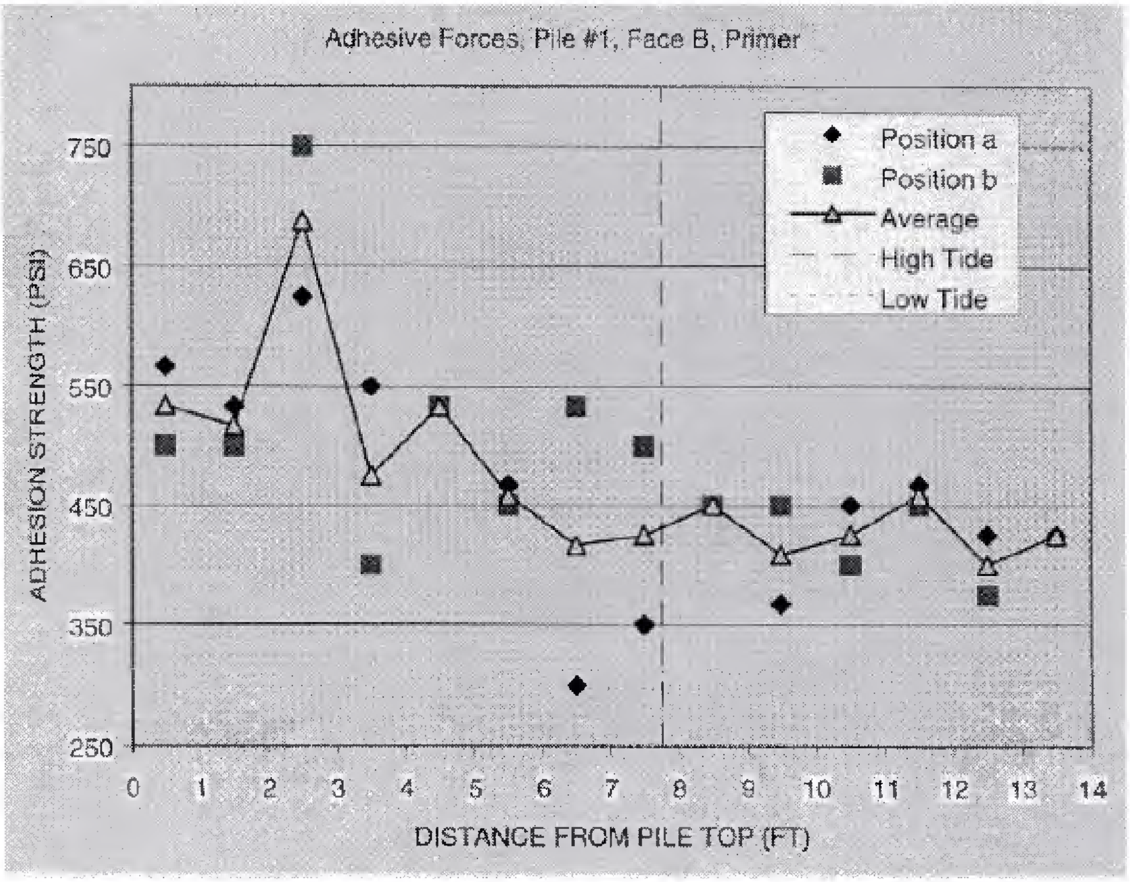


Figure 10. Adhesion strength with Sikadur 30 and Sikadur 55 Primer, Pile #1, Face B.

The plotted data of Figure 10 indicates a greater adhesiveness of the Sikadur 30 epoxy above the tidal zone than below the tidal zone. This trend was not as apparent on the surfaces in which the Sikadur 55 primer was not used (both Faces B and D, Figure 9). The data appears to indicate that the application of Sikadur 55 results in a higher adhesiveness above the high tide level.

4.5 Adhesion of Sikadur 30 Epoxy on a Hydroblasted Surface, With and Without the Application of Sikadur 55 Primer

Figure 11 is the plotted data of the adhesiveness of Sikadur 30 with and without the application of Sikadur 55 primer to Pile #1, Face B (hydroblasted).

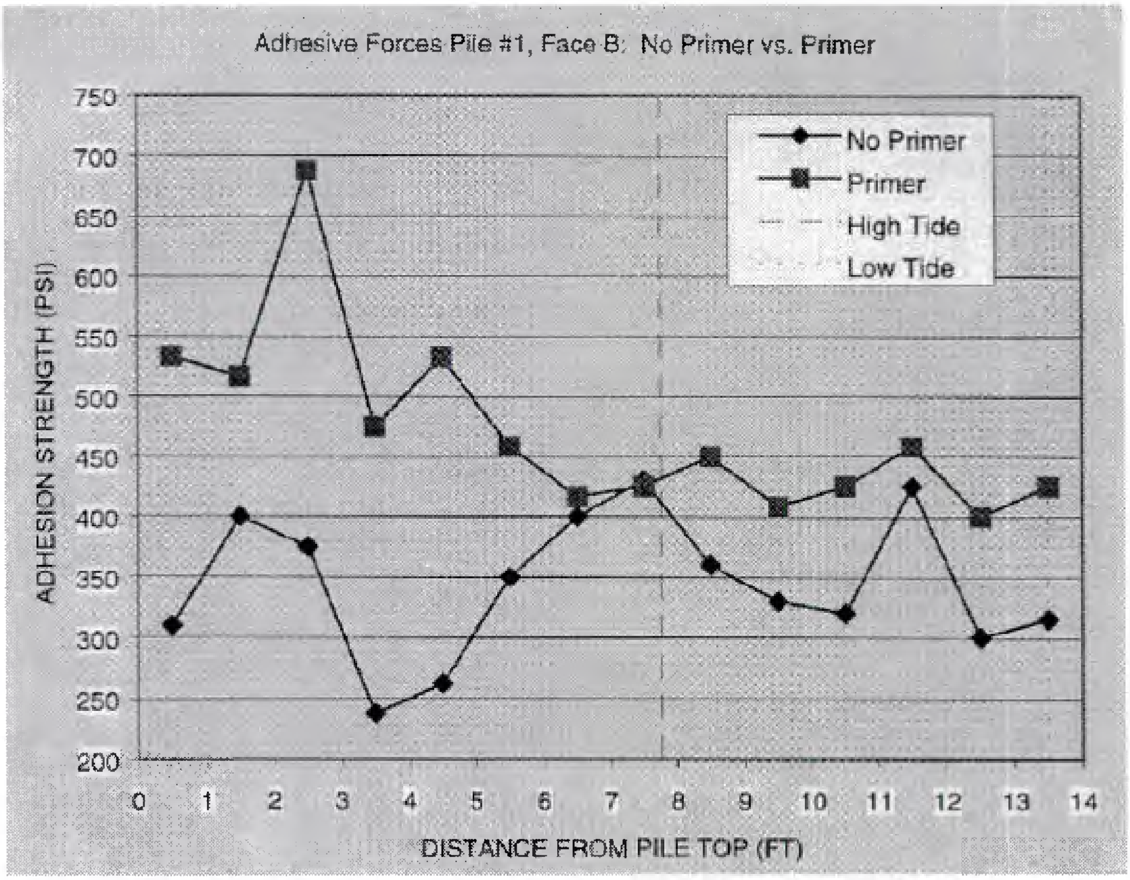


Figure 11. A Comparison of the Adhesiveness of Sikadur 30 With and Without the Application of Sikadur 55 Primer

The plotted data of Figure 11 indicates that along the entire length of the pile, the application of Sikadur 55 primer enhances the adhesiveness of the Sikadur 30 epoxy. The differentiation is most apparent above the high tide line (above the tidal zone).

4.6 Adhesion of Sikadur 32 Epoxy on Hydroblasted/Non-Form Surface

Figure 12 shows the actual pull-off forces of the dollies placed at 1 foot intervals along Pile #1, Face A. The "triangular" data points (connected) represent the average pull-off force at each sample site.

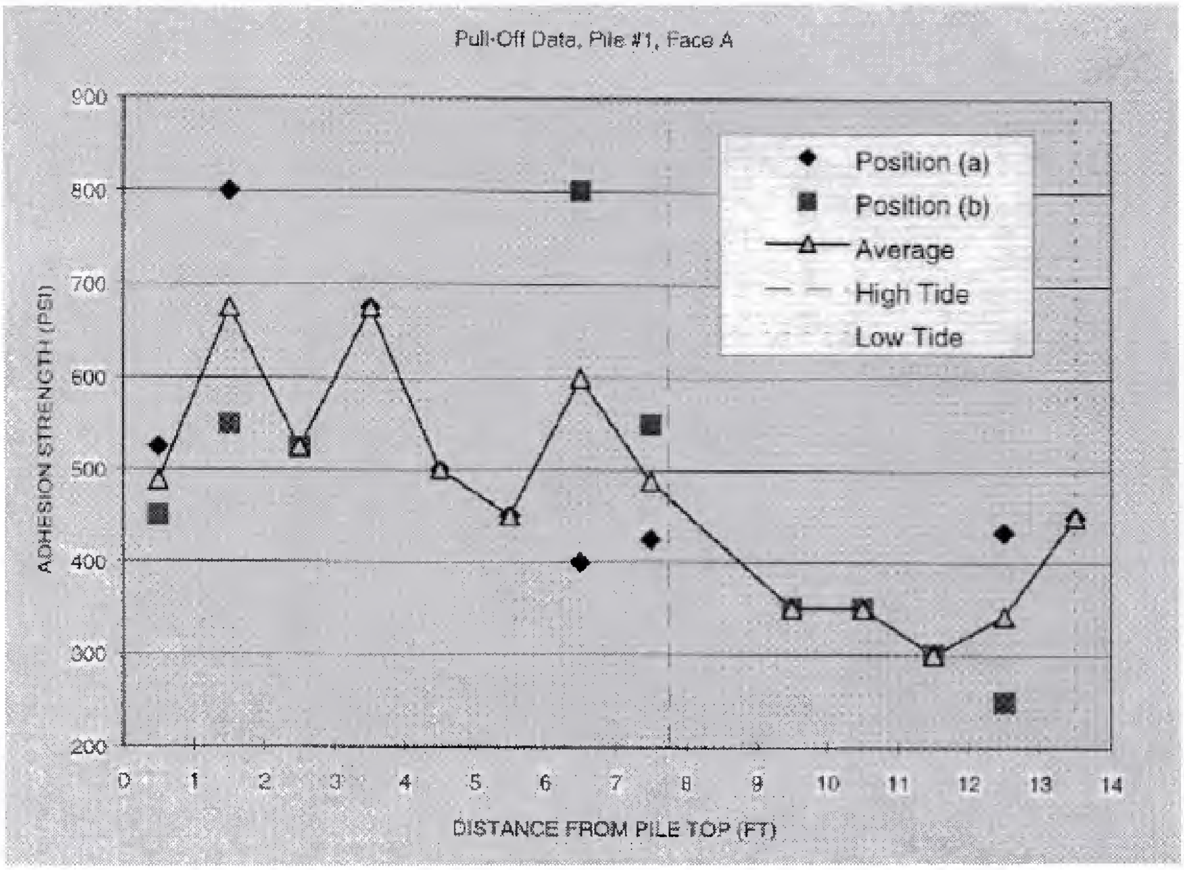


Figure 12. Pull-Off Forces Required to Remove the Adhered Dollies
Along the Pile with a Hydroblasted Surface

Figure 12 shows a similar trend to that of Figure 9. While the epoxies are not the same (Sikadur 32 vs. Sikadur 30 respectively), the adhesiveness appears to be greater for the concrete that is generally out of the water (average high tide line is 7 ¾ft) than the region generally in the water.

5 CHLORIDE CONCENTRATIONS ON THE PILE SURFACE

5.1 Surface Chloride Concentrations on Hydroblasted Surface

Figure 13 shows the chloride concentrations of the surface samples of Pile #1, Face B. Alternate sample analyses at the marked sample sites on the pile were conducted on separate days.

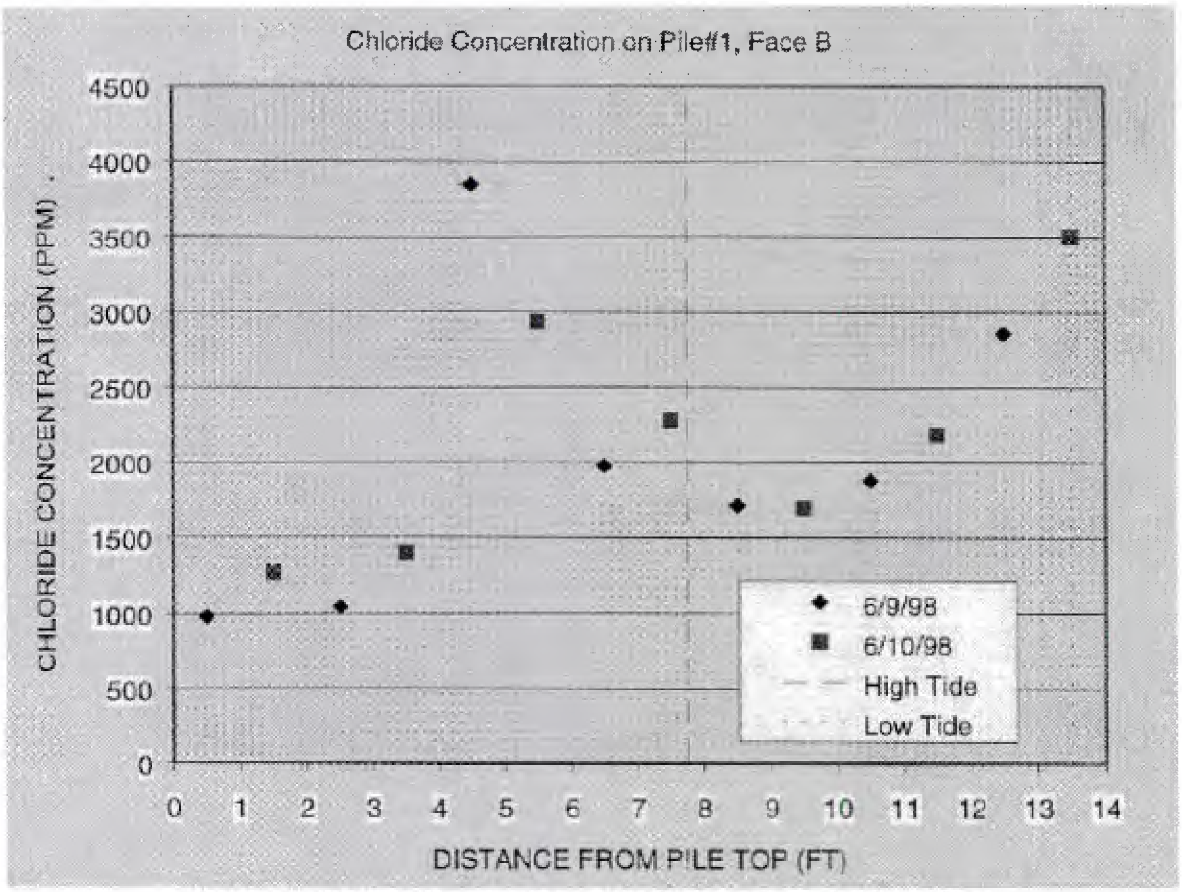


Figure 13. Chloride Concentrations at the Surface of a Hydroblasted Face

The average "tidal zone" for the month of May, 1998 was from about 7 ¾ft to 13 ½ft along the pile. The data reflect what might be expected: above the average high tide (what might be considered the splash zone along the pile, a region that is not continually washed by tidal action) the chloride levels are comparatively high and decreasing with distance away from the high tide level; within the tidal zone (where a continuous washing of the pile surface occurs) there is a lesser variation in the surface chloride concentrations; and near or below the average low tide (where chloride levels remain constant to the water), the chloride levels increase slightly.

5.2 Surface Chloride Concentrations on Nonhydroblasted Surface

Figure 14 show the chloride concentrations of the surface samples on Pile #1, Face D.

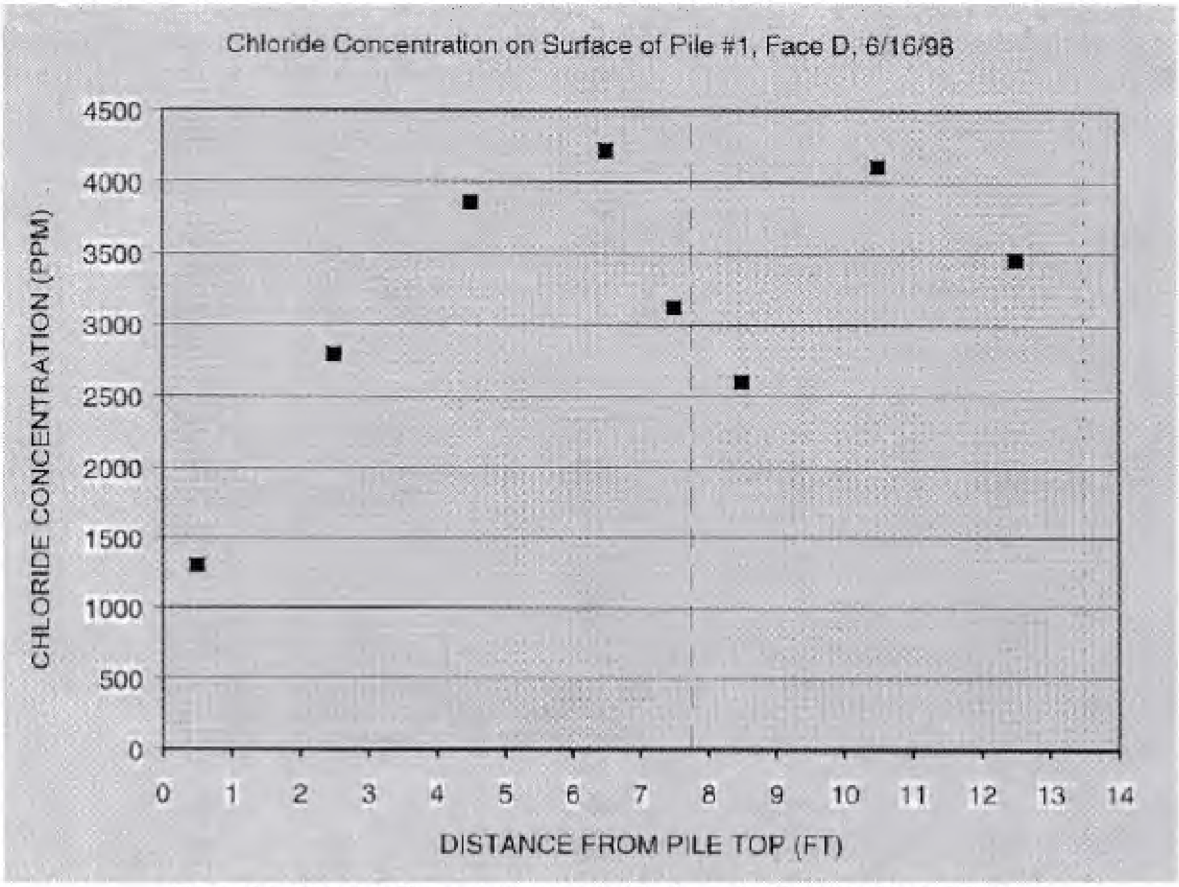


Figure 14. Chloride Concentrations at the Surface of a Nonhydroblasted Face

The average "tidal zone" for the month of May, 1998 was from about 7 ¾ft to 13 ½ft along the pile. Again, with fewer data points in Figure 14 than Figure 13, the same correlation of surface chloride concentrations above, in, and below the tidal zone appears. Figure 15, below, summarizes the two sets of data.

5.3 Surface Chloride Concentrations on Hydroblasted and Nonhydroblasted Face

Figure 15 compares the surface chloride concentrations of a hydroblasted surface (Face B) and a nonhydroblasted surface (Face D).

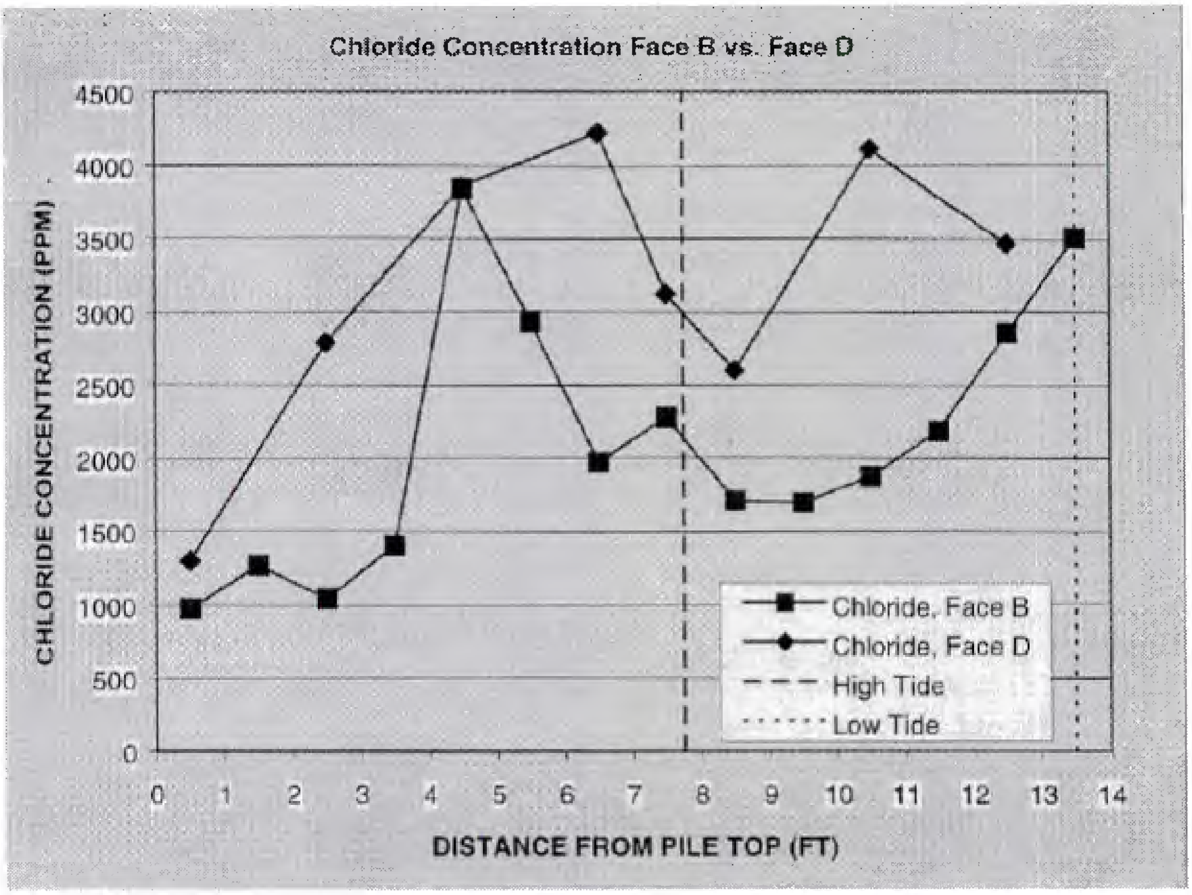


Figure 15. A Comparison of the Surface Chloride Concentrations of a Hydroblasted Surface (Face B) and the Surface Chloride Concentrations of a Nonhydroblasted Surface (Face D)

Figure 15 is a summary of Figures 13 and 14. The chloride levels are slightly higher on the nonhydroblasted surface (which may be expected); the surface chloride levels are higher just above the tidal zone, and near/below the low tide line.

As the distance increases from the high tide line toward the top of the pile, the chloride levels decrease. This trend is expected because of the lower concentration of the salt water spray from the tidal action of the harbor environment.

6. THE ROLE OF CHLORIDE ON THE ADHESIVENESS OF SIKADUR 30 EPOXY

6.1 The Correlation of Surface Chloride Concentrations of a Hydroblasted Surface to the Adhesiveness of the Sikadur 30 Epoxy

Figure 16 shows how the chloride concentrations along the surface of Pile#1, Face B (a hydroblasted surface) relates to the adhesivness of the Sikadur 30 epoxy.

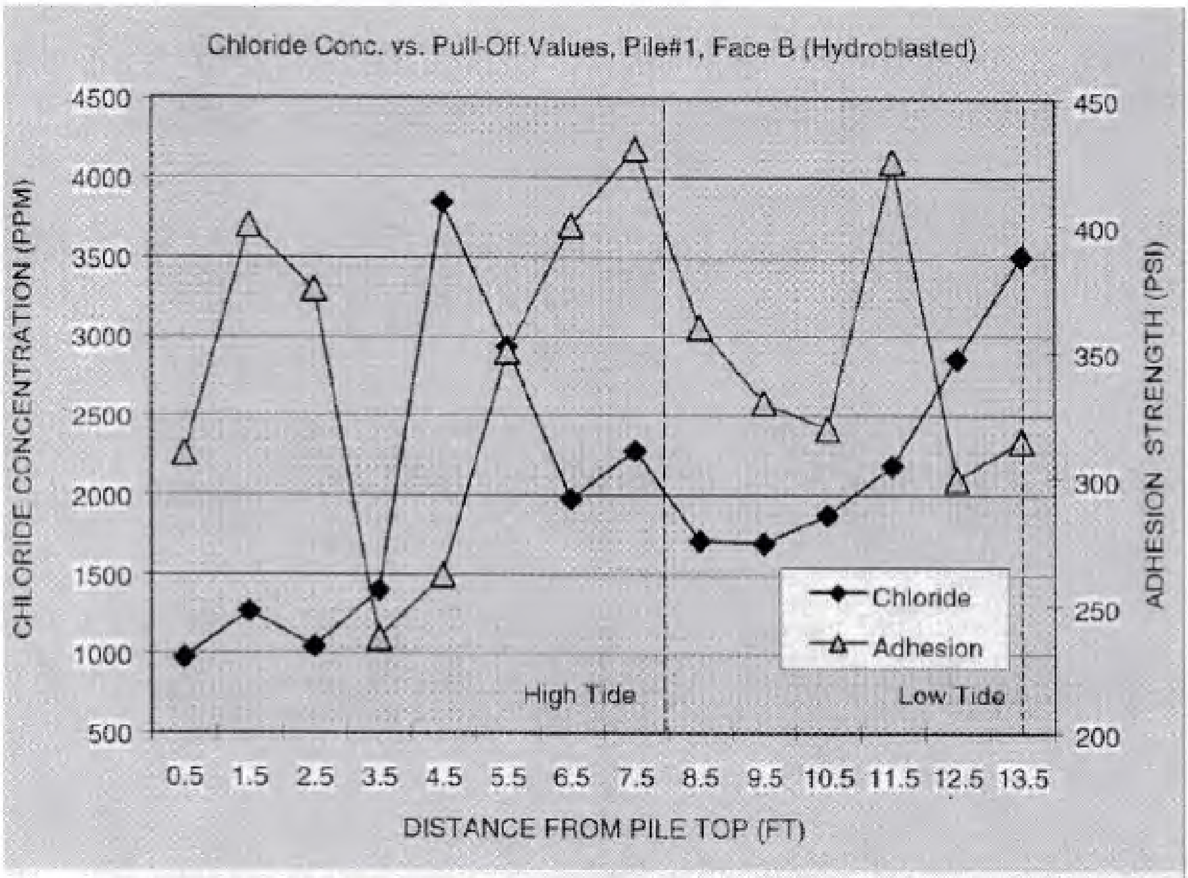


Figure 16. Chloride Concentrations along Pile #1, Face B (hydroblasted) vs. Adhesivness of the Sikadur 30 Epoxy

Figure 16 appears to produce little indication of a correlation between increasing surface chloride concentrations to the adhesiveness of the Sikadur 30 epoxy. Figure 17 appears to have more meaning.

6.2 The Inverse Correlation of Surface Chloride Concentrations of a Hydroblasted Surface to the Adhesiveness of the Sikadur 30 Epoxy

Figure 17 shows how the chloride concentrations along the surface of Pile#1, Face B (a hydroblasted surface) relates to the *inverse* of the adhesivness of the Sikadur 30 epoxy.

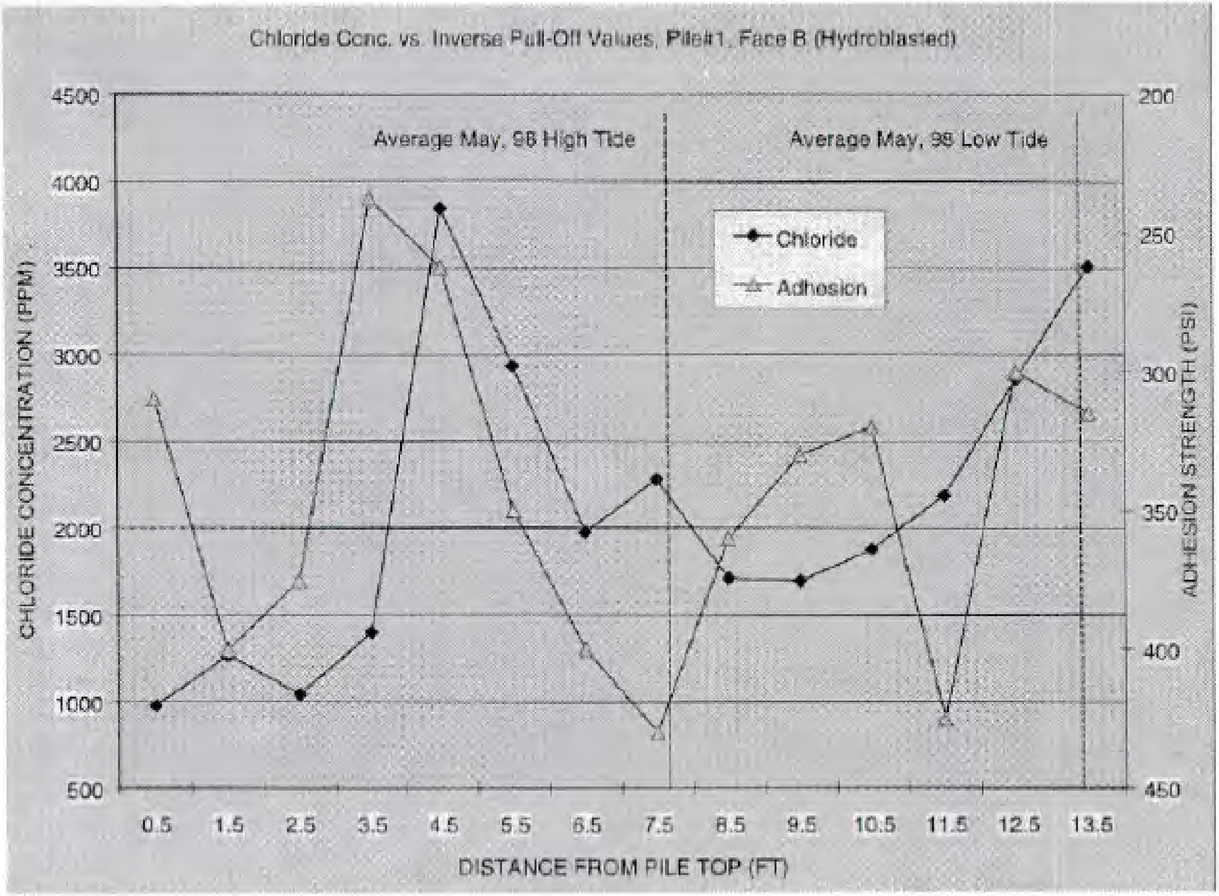


Figure 17. Chloride Concentrations along Pile #1, Face B (a hydroblasted surface) vs. the Inverse Adhesivness of the Epoxy

Figure 17 is an inverse correlation of chloride levels and adhesiveness. The plotted data shows that high surface chloride concentrations results in a decreased adhesiveness in the Sikadur 30 epoxy. The correlation is mostly apparent in the region of the pile above the average high tide, the region of the pile generally consider out of the water.

At the top of the pile,where the surface chloride levels are lower, the adhesiveness generally increases again.

6.3 Correlation of Surface Chloride Concentrations of a Hydroblasted Face to the Adhesiveness of the Sikadur 30 Epoxy With Sikadur 55 Primer

Figure 18 shows the plotted data of the surface chloride concentrations and the adhesiveness of Sikadur 30 with the application of Sikadur 55 primer.

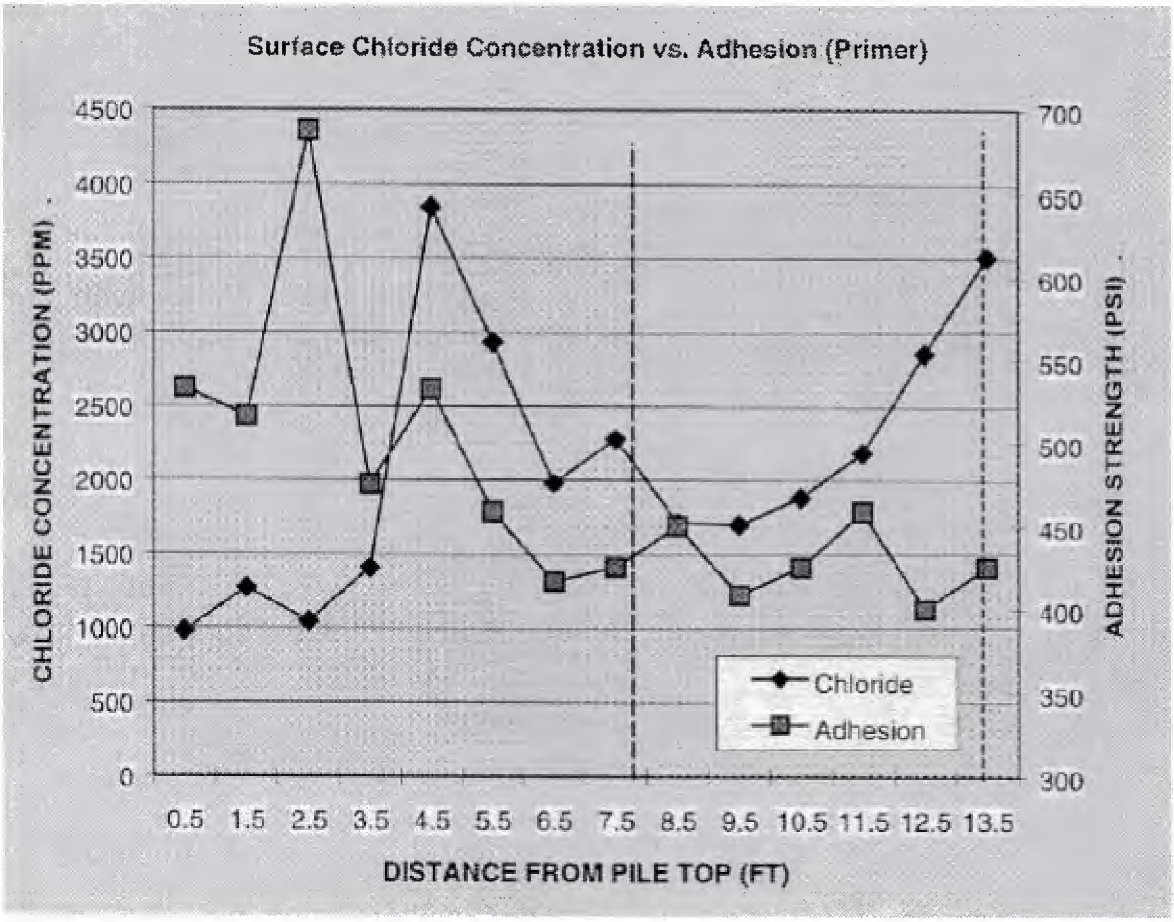


Figure 18. A Correlation of the Surface Chloride Concentrations and the Adhesiveness of Sikadur 30 With the Application of Sikadur 55 Primer Along Pile #1, Face B.

The plotted data of Figure 18 does not indicate much of a correlation between the chloride levels and the adhesiveness of the Sikadur 30 epoxy. It may be hypothesized that primer has a more significant effect on the adhesiveness of the Sikadur 30 epoxy than does the surface chloride concentrations. Additionally the concrete above the tidal zone may a more dense/compact (higher compressive strength) than within or below the tidal zone. See Figures 27 and 28.

6.4 The *Inverse* Correlation of Surface Chloride Concentrations of a Hydroblasted Surface to the Adhesiveness of the Sikadur 30 Epoxy With the Application of Sekadur 55 Primer

Figure 19 shows the effect of the surface chloride concentrations on the adhesiveness of the Sikadur 30 epoxy with the pre-application of Sikadur 55 primer.

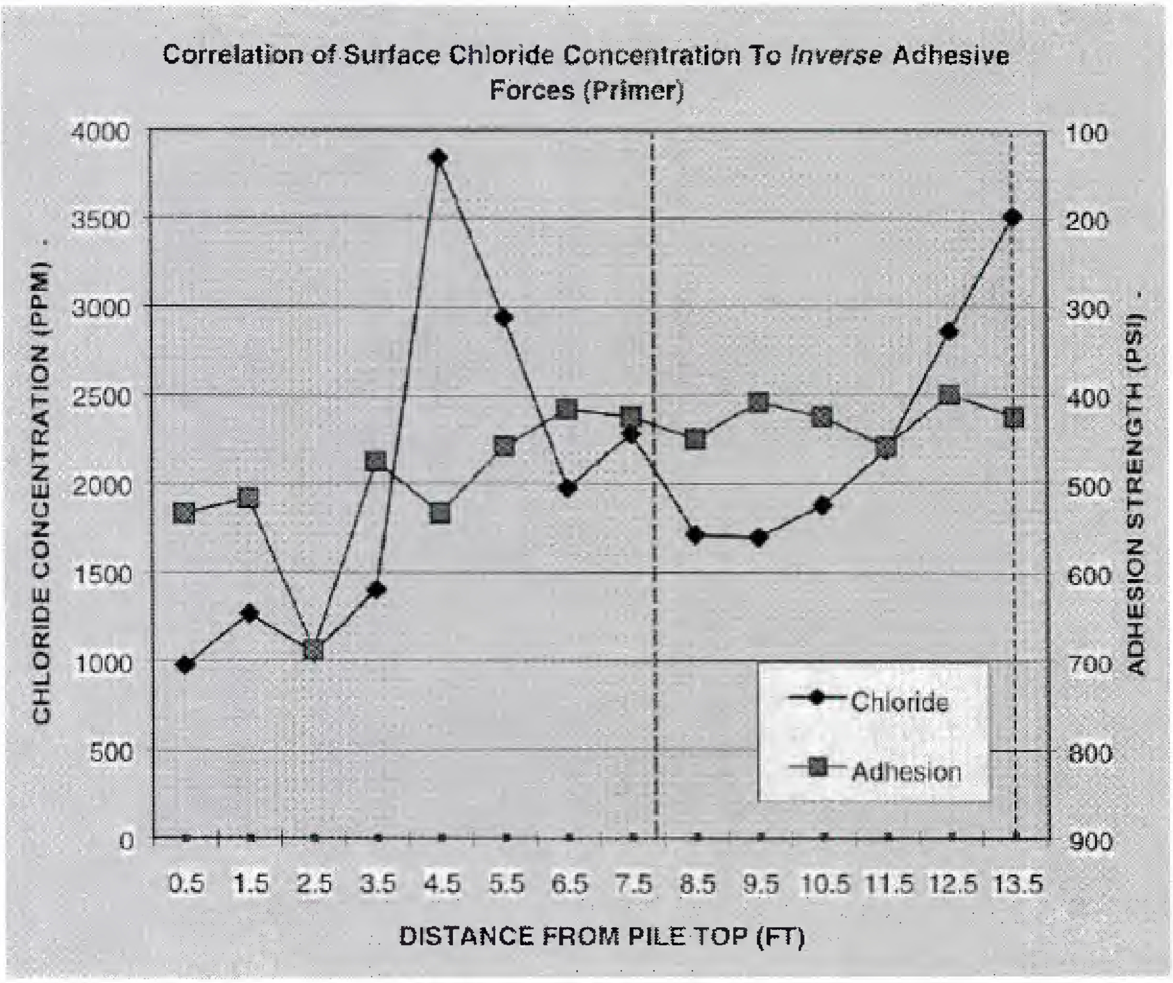


Figure 19. Inverse Correlation of the Surface Chloride Concentrations and the Adhesiveness of Sikadur 30 With the Application of Sikadur 55 Primer, Pile #1, Face B.

The plotted data of Figure 19 does not show the strong inverse correlation that was observed for the data in which the Sikadur 55 primer was not used (see Figures 17 and 23). Again, the presence of the Sikadur 55 primer may override some of the effects of the surface chloride concentrations. However, at the top of the pile, the highest adhesion values are indeed obtained for the lowest chloride concentrations.

6.5 The Correlation of Surface Chloride Concentration to Adhesiveness of Sikadur 30 With and Without the Application of Sikadur 55 Primer

Figure 20 summarizes the effect of surface chloride concentrations of the adhesiveness of Sikadur 30 with and without the application of Sikadur 55 primer on Pile #1, Face B (hydroblasted).

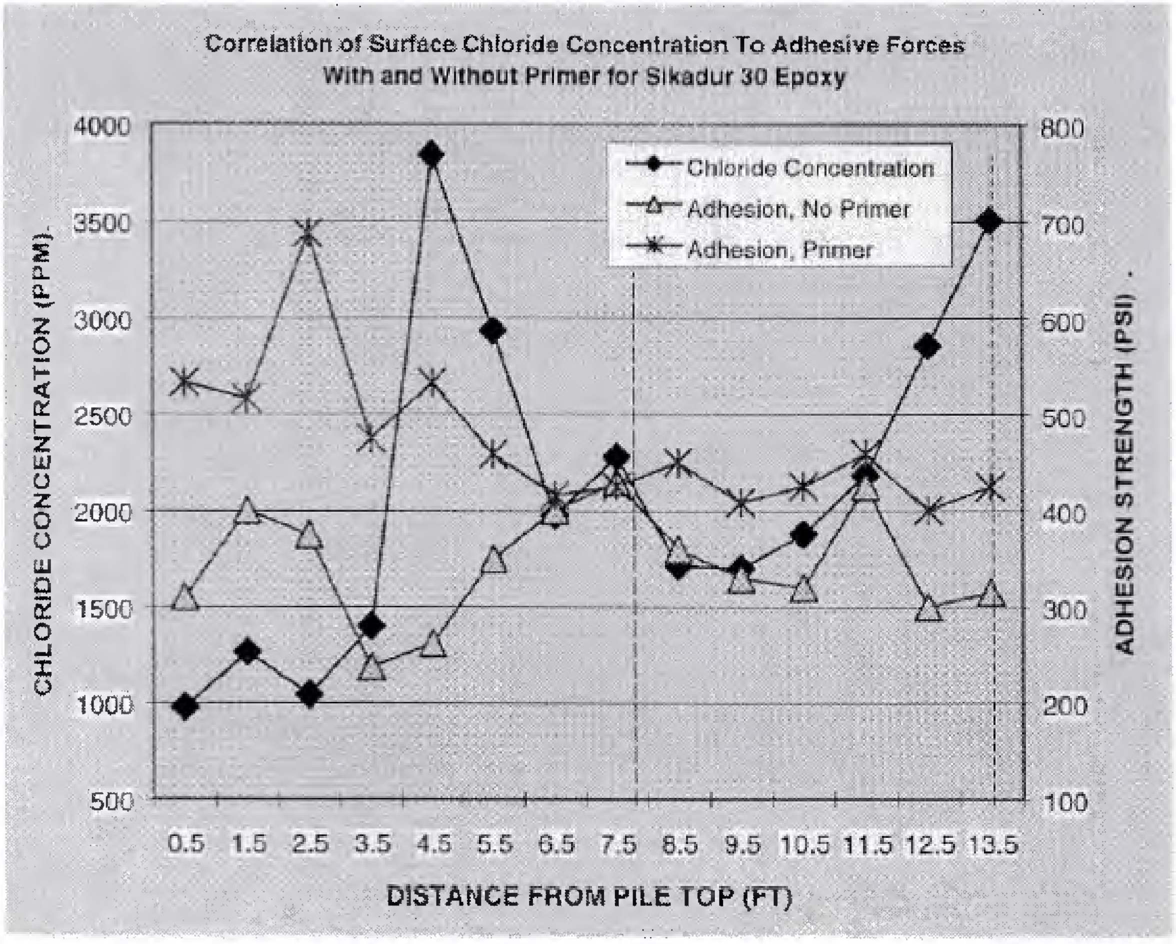


Figure 20. Summary of Surface Chloride Concentrations and Adhesiveness of Sikadur 30 Epoxy to Pile #1, Face B, With and Without Sikadur 55 Primer

The plotted data of Figure 20 summarizes the data of Figures 16 and 18. The surface chloride concentrations would seem to inversely correlate with the adhesiveness of the Sikadur 30 epoxy with the Sikadur 55 primer, but not so without the primer.

6.6 The *Inverse* Correlation of Surface Chloride Concentration to Adhesivness of Sikadur 30 With and Without the Application of Sikadur Primer

Figure 21 assumes the same correlation that appeared in Figure 20, except of the inverse correlation. A strong evidence of the inverse correlation was apparent in earlier plotted data.

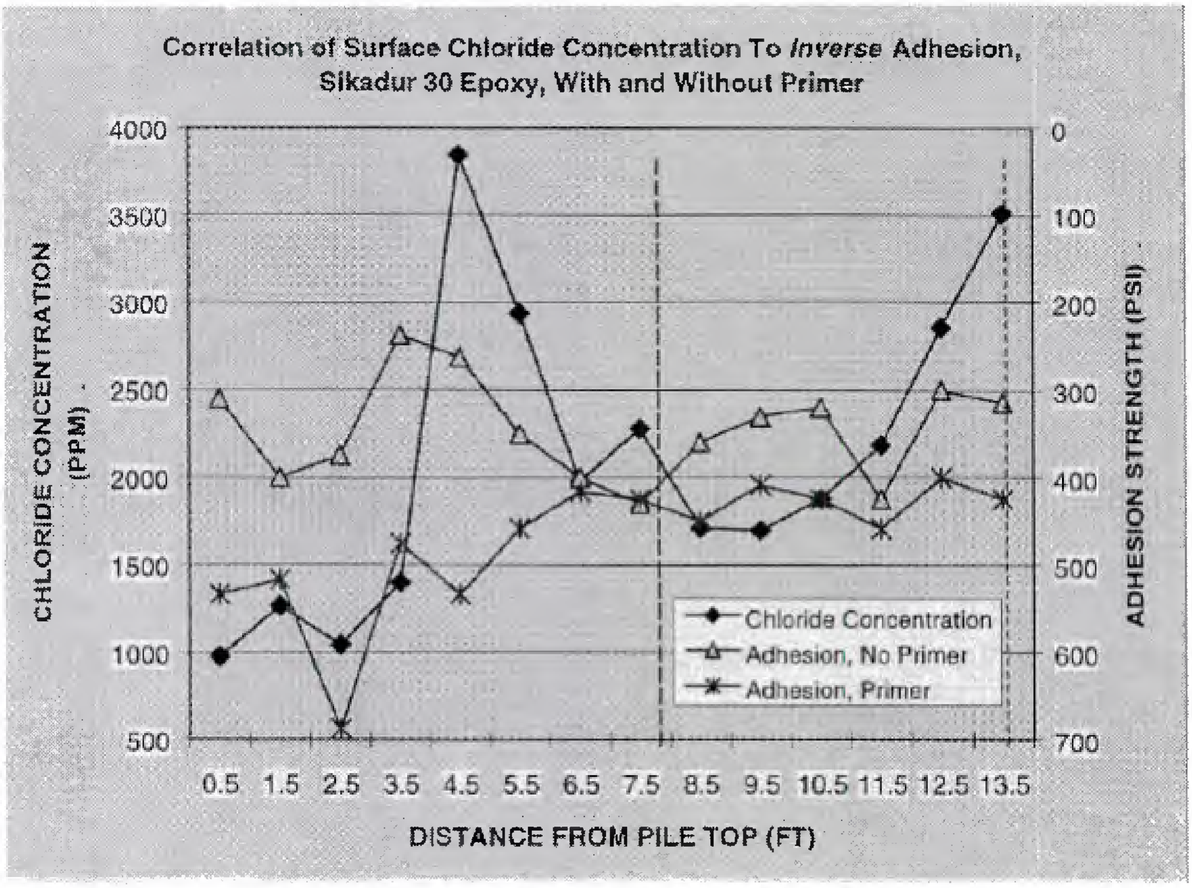


Figure 21. A Summary of the Data Relating the Surface Chloride Concentrations to the Inverse Adhesiveness of Sikadur 30 Epoxy to Pile #1, Face B, With and Without the Application of Sikadur 55 Primer

The plotted data of Figure 21 summarizes the data of Figures 17 and 19. The surface chloride concentrations correlate with the *inverse* of the adhesiveness of the Sikadur 30 epoxy with or without the Sikadur 55 primer.

6.7 The Correlation of the Surface Chloride Concentration of a Nonhydroblasted Surface to the Adhesiveness of Sikadur 30 Epoxy

Figure 22 shows how the chloride concentrations along the surface of Pile#1, Face D (a nonhydroblasted surface) relates to the adhesivness of the Sikadur 30 epoxy.

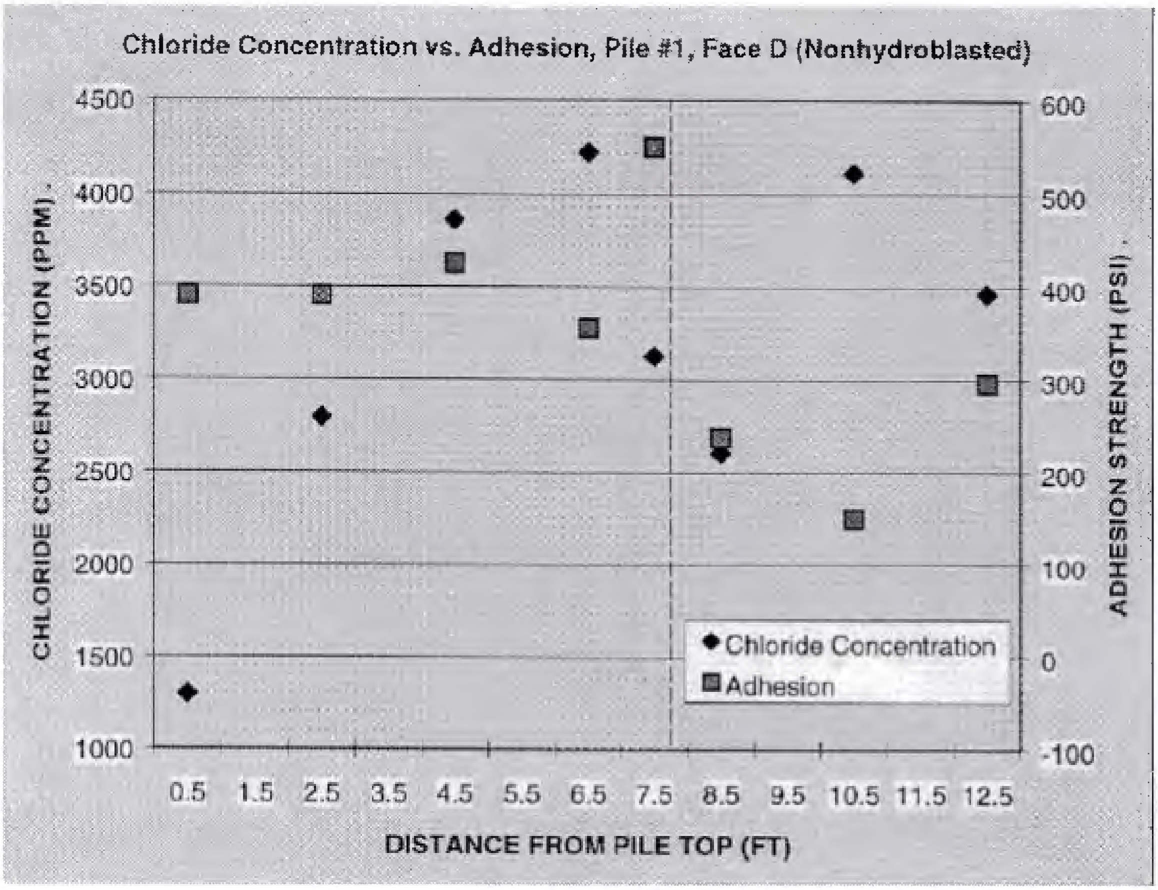


Figure 22. Chloride Concentrations along Pile #1, Face D (nonhydroblasted) vs. Adhesivness of the Epoxy

The correlation of surface chloride concentrations to adhesiveness for the nonhydroblasted surface (Figure 22) is analogous to that for the hydroblasted surface (compare to Figure 16). Where the surface chloride levels are higher, the adhesiveness decreases, especially in the tidal zone and below. Above the tidal zone (top of the pile) the "pull-off" values appear to taper off and be less influenced by the chloride content; this is again borne out in Figure 23.

6.8 The *Inverse* Correlation of the Surface Chloride Concentration of a Nonhydroblasted Surface to the Adhesiveness of Sikadur 30 Epoxy

Figure 23 shows how the chloride concentrations along the surface of Pile#1, Face D (a nonhydroblasted surface) relates to *inverse* of the adhesivness of the Sikadur 30 epoxy.

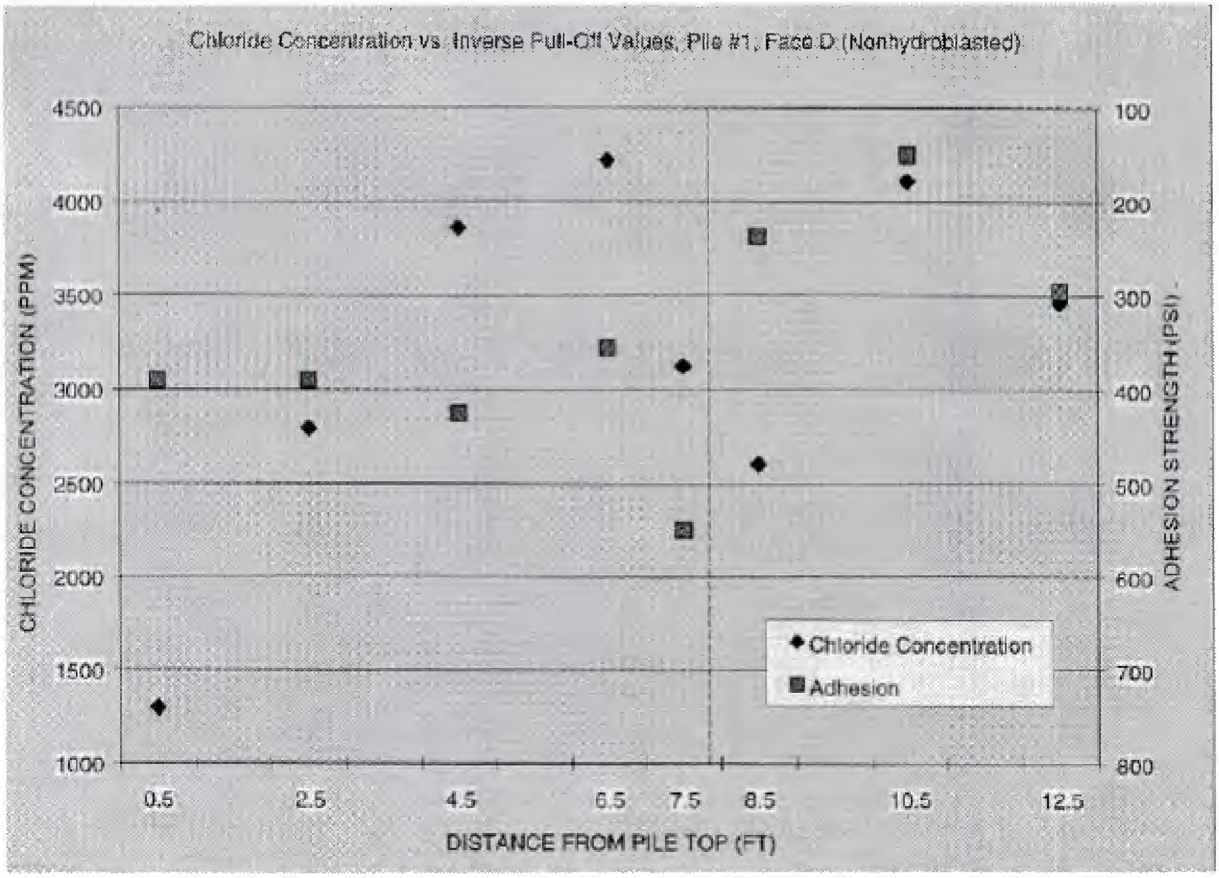


Figure 23. Chloride Concentrations along Pile #1, Face D (nonhydroblasted) vs. the Inverse Adhesivness of the Epoxy

Figure 23 again (as in Figure 22) indicates a slight inverse correlation (mostly in the tidal zone and below) between adhesion and chloride concentration. This inverse correlation was also evident on the hydroblasted surface (compare to Figure 17).

7. EFFECT OF CHLORIDE ON THE ADHESIVENESS OF SIKADUR 32 EPOXY

Most data recorded and analyzed earlier in this report was based upon the adhesiveness of the Sikadur 30 epoxy. The Sikadur 32 epoxy is a less viscous adhesive and is generally not used for adhering carbon fiber strips, as was the focus of this study. However since the dollies had been applied, they were separated from the concrete surface using the elcometer.

7.1 The Correlation of Surface Chloride Concentrations of a Hydroblasted Surface to the Adhesiveness of Sikadur 32 Epoxy

Figure 24 shows the correlation of the surface chloride concentrations on Face B and the adhesiveness of the Sikadur 32 epoxy on Face A. Both of these surfaces are hydroblasted. The data should be viewed with some reservation in that Face B is a form surface and Face A is a top-of-the-form surface. The exposure to the marine environment of the two faces, however, should be the same.

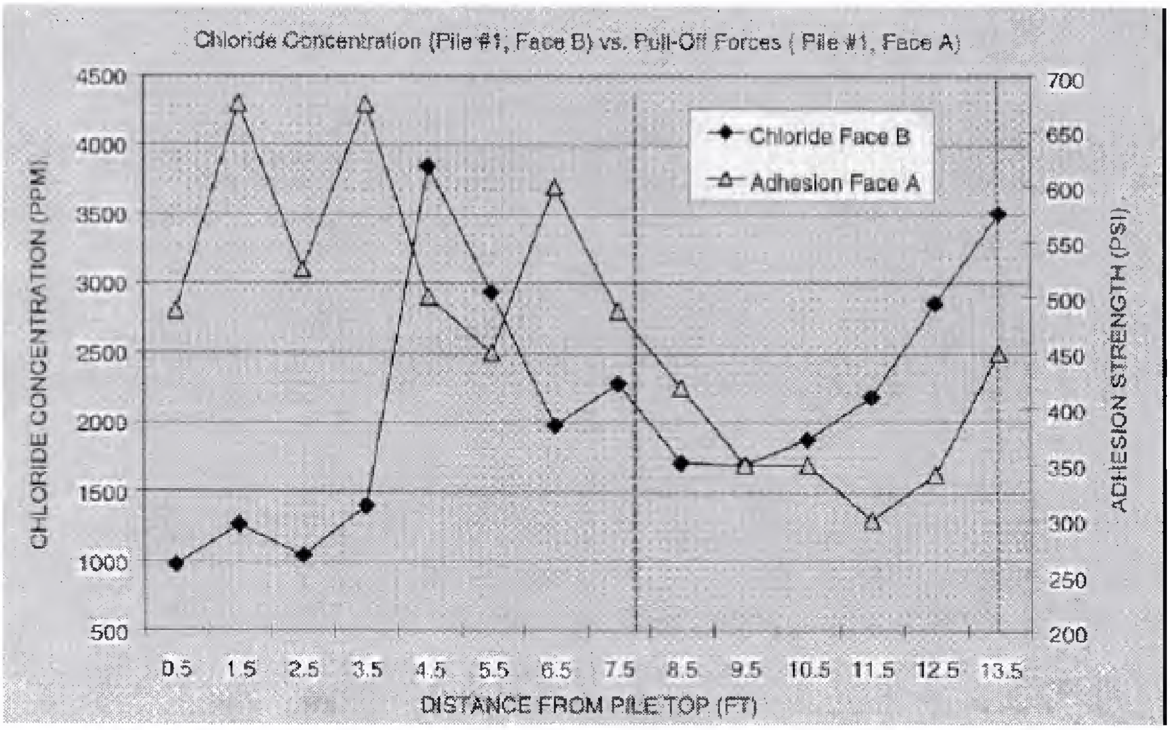


Figure 24. A Correlation of the Surface Chloride Concentrations (Face B) vs. the Adhesiveness of the Sikadur 32 Epoxy (Face A) Along Pile #1.

The plotted data of Figure 24 appears rather sporadic, although some correlation of chloride levels to adhesiveness of Sikadur 32 in the tidal zone and below is apparent. The plotted data of Figure 25 provides evidence of "some" correlation above the tidal zone.

7.2 The *Inverse* Correlation of Surface Chloride Concentrations of a Hydroblasted Surface to the Adhesiveness of Sikadur 32 Epoxy

Figure 25 shows the correlation of the surface chloride concentrations on Face B and the *inverse* adhesiveness (pull-off forces) of the Sikadur 32 epoxy on Face A.

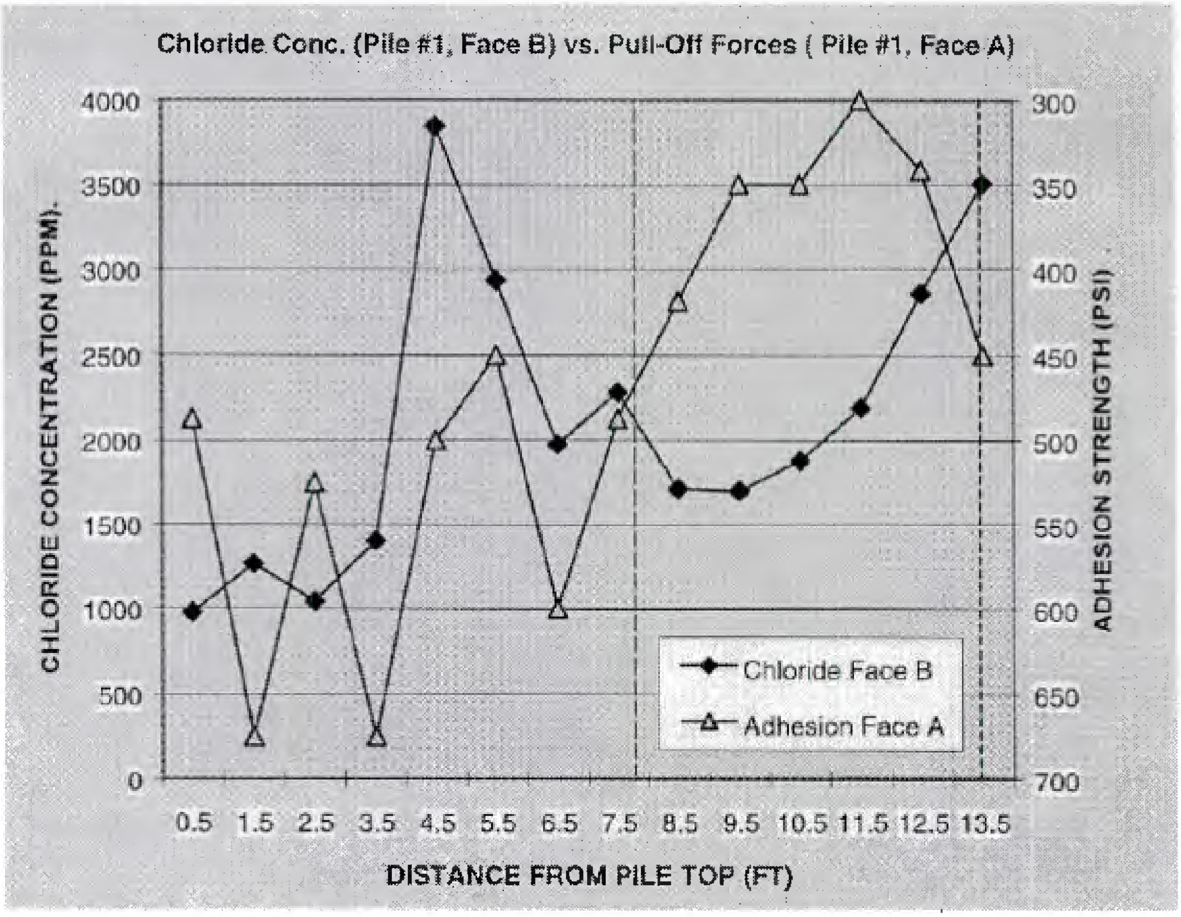


Figure 25. A Correlation of the Surface Chloride Concentrations (Face B) vs. the Inverse Adhesiveness of the Sikadur 32 Epoxy (Face A) Along Pile #1

The plotted data of Figure 25 again (as in the plotted data of Figure 17 and, questionably, Figure 23) indicates an inverse correlation of chloride levels to adhesiveness. This is especially apparent above the tidal zone: the higher the surface chloride concentration along the pile above the tidal zone, the lower is the adhesive strength of the epoxy.

8. EFFECT OF COMPRESSIVE STRENGTH OF HYDROBLASTED/ NON-FORM SURFACE

The surface of Pile #1, Face B was additionally analyzed using a rebound hammer [2], a test to determine the concrete compressive strength at the concrete surface.

Figure 26. Rebound hammer.

8.1 Correlation of Chloride Concentrations to Compressive Strength

Figure 27 correlates the surface chloride concentrations to the compressive strength of the pile surface along the hydroblasted Face B.

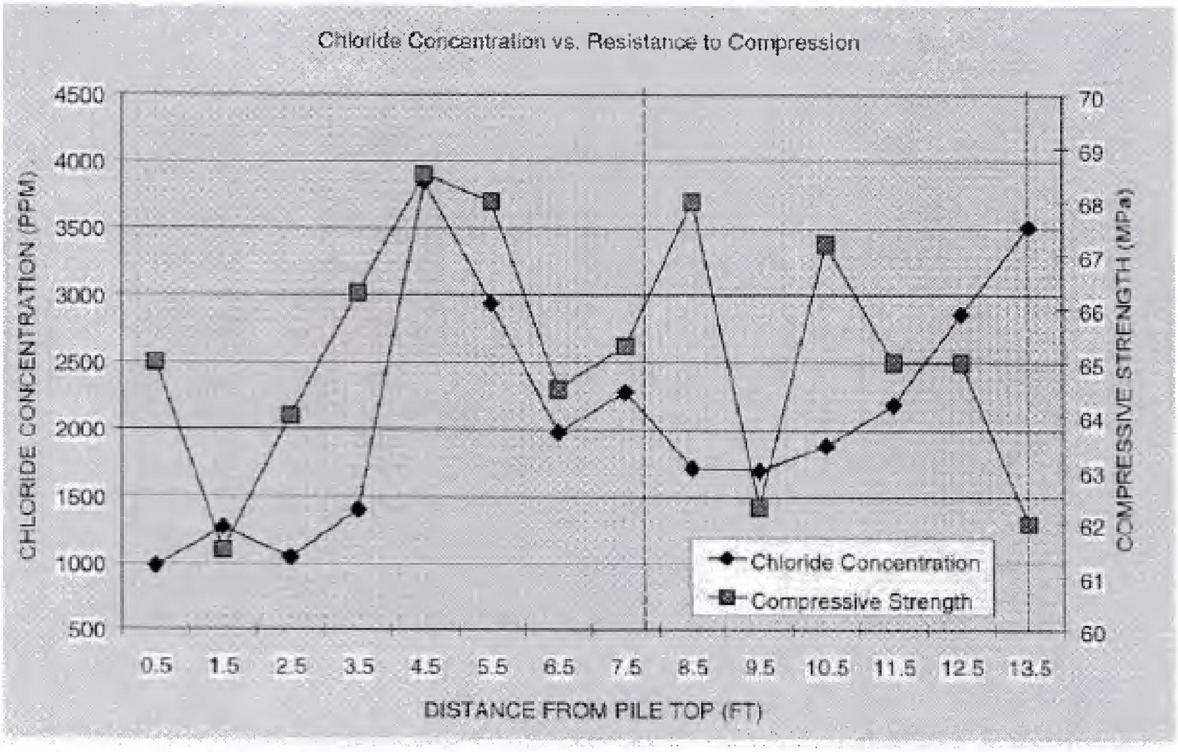


Figure 27. Correlation of Surface Chloride Concentrations and Compression Resistance Along Pile #1, Face B, a Hydroblasted Surface

The concrete compressive strength measured with a rebound hammer along the pile represents the density or compactness of the surface. Figure 27 indicates that, above the tidal zone, the concrete appears to be slightly denser where the surface chloride levels are high. Within the tidal zone and below, there appears to be little correlation.

8.2 The *Inverse* Correlation of the Adhesiveness of Sikadur 30 Epoxy to Concrete Compressive Strength at the Surface

Figure 28 compares the compressive strength to the adhesiveness of the Sikadur 30 epoxy along the pile surface.

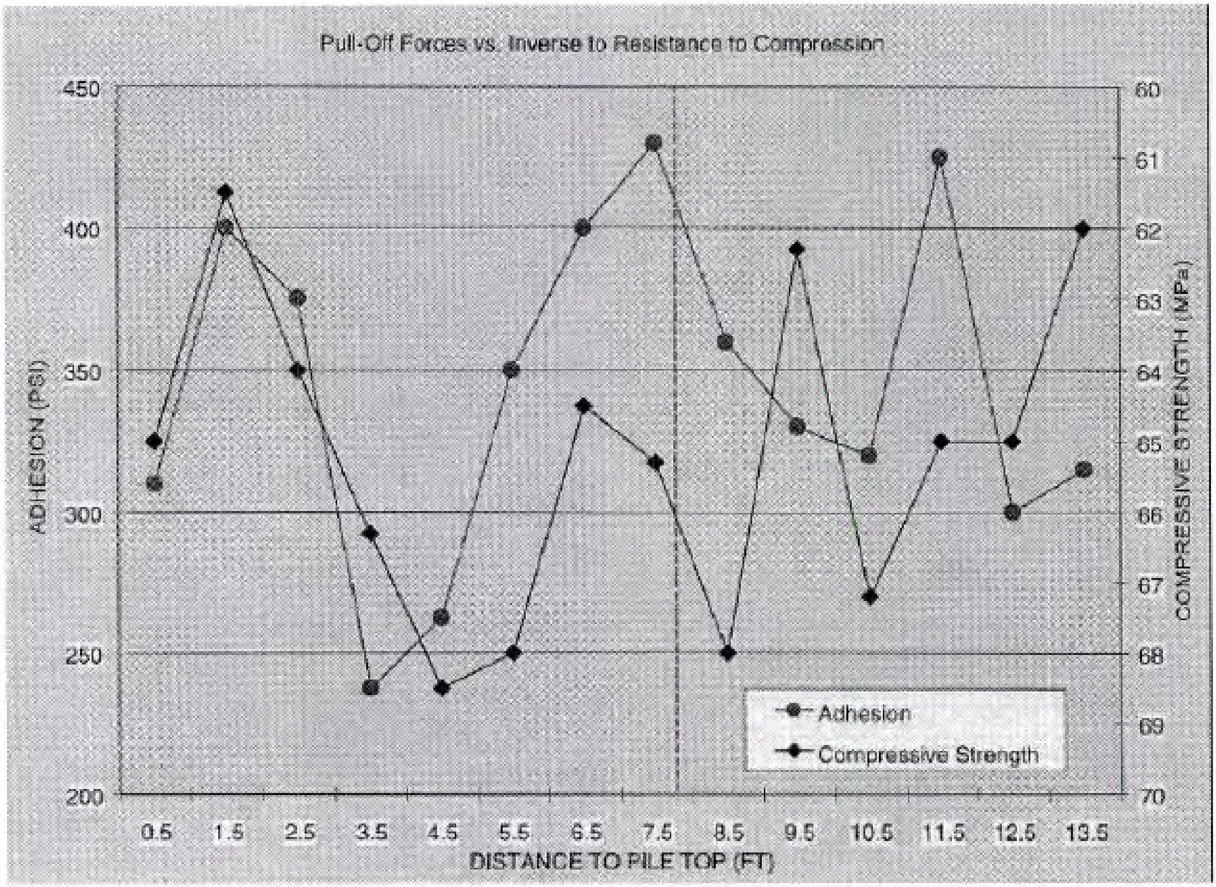


Figure 28. A Correlation of the Adhesiveness of Sikadur 30 Epoxy with the Compression Resistance of the Surface of Pile #1, Face B, a Hydroblasted Surface

The plotted data of Figure 28 is an inverse correlation of adhesiveness of the Sikadur 30 epoxy to the compression resistance of the concrete. This inverse correlation is apparent for the region of the pile above the tidal zone, a correlation that was also evident in Figure 27, that of the surface chloride concentrations. This inverse correlation is surprising but it may be actually due to the higher chloride content rather than the higher strengths, as shown in Figure 27.

9. CONCLUSIONS

The research for this project focused on the effect of surface chloride concentrations on the adhesion of two commercial epoxys, Sikadur 30 and 32, on a concrete pile that had been exposed to the marine environment for approximately 4 years. Additionally, various surface preparations of the pile were also considered: the surface parameters included hydroblasting (vs. nonhydroblasting) the surface, and the use of a primer coating (Sikadur 55) before the application of the epoxy.

The surface chloride concentrations were measured analytically with a chloride ion selective electrode. A general trend in the chloride levels along the pile was apparent (Figures 13 and 14).

The adhesive forces of the epoxy were measured with an elcometer. As might be expected the specific data points were rather erratic, but with the duplication of data points at selected locations along the pile, a general trend was also evident (Figures 7-11). A visual review of the dollies removed from the pile showed that separation occurred as a result of *total* concrete failure on about 50% of the dollies, located primarily in the region above the tidal zone. About 25% of the dollies showed mostly concrete failure (between 60% and 90% concrete failure), and about 25% showed mostly epoxy-concrete (adhesive) failure. No failure occurred within the relatively thick Sika CFRP strip.

Conclusive results for the effect of surface chloride concentrations on the adhesiveness of Sikadur 30 of a hydroblasted surface *without* the application of a primer indicate that as the surface chloride concentrations increase the adhesion of the epoxy decreases. This trend is especially true for the section of the concrete that is above the average tidal zone (Figure 17).

Hydroblasting by itself seems to provide some degree of improvement in the adhesion (Figure 9). Hydroblasting can remove a significant amount of chlorides (Figure 15), which in turn results in increased adhesion (Figure 17).

With the application of the primer, the trend appears to be similar, but less pronounced (Figure 18). This latter trend indicates that the surface chloride concentration may not be the major factor in this analysis, but rather the greater adhesiveness of the epoxy with the application of the primer on a more dense/compact concrete surface (Figures 27 and 28). One very evident factor in the adhesion of the epoxy is that the adhesive forces at all points along the pile were higher when the primer was applied (Figure 11). Hence both hydroblasting and the use of primer are recommended.

A correlation of the surface chloride concentrations on the adhesion of the Sikadur 30 epoxy on a nonhydroblasted surface is less conclusive. First, fewer data points were obtained. If any trend can be extracted from the data, an inverse correlation between the surface chloride concentrations and the adhesion of the epoxy is more apparent (compare Figures 22 and 23). This would be in agreement with the data interpretations from the hydroblasted surface. No "primer" data were obtained from the nonhydroblasted surface of the pile.

An additional factor that could affect adhesion was the compressive strength of the concrete, as measured at the surface using a rebound hammer. The surface chloride concentrations increased with the compressive strength of the concrete, especially above the tidal zone (Figure 27). However, where the surface compressive strength increased, the adhesive forces of the epoxy decreased (Figure 28). The latter observation may only indicate that the surface chloride concentrations, not the strength of the concrete at the surface, play the major role in affecting the adhesiveness of the epoxy. While a set of data for the surface chloride concentrations and the adhesion of the Sikadur 32 epoxy were obtained (Figures 24 and 25), any conclusions based upon that data should be carefully construed.

10. ACKNOWLEDGMENTS

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A LIMITED MARINE DURABILITY ANALYSIS OF CFRP ADHERED TO CONCRETE

G.WARREN, D. BURKE, S. HARWELL, C. INABA, AND D. HOY
Naval Facilities Engineering Service Center
Port Hueneme, CA, USA

Abstract
It has been determined that selected Navy piers and wharves be post-strengthened (upgraded) because of the switch from rail-mounted cranes to portable truck-mounted cranes at the waterfront. It is planned that a combination of carbon fiber reinforced plastic (CFRP) pultruded rods and sheets will be applied externally to the structures. This paper presents a marine durability analysis of these external reinforcements conducted in the laboratory.

Unreinforced concrete beams, 90mm x 90mm x 600mm (3.5 x 3.5 x 24 inches), were prepared. In one group of beams, CFRP rods were embedded in epoxy resin in a longitudinal slot cut in the concrete. In the second group, CFRP sheets were adhered to the surface of the beams with epoxy. Representative samples from both groups were exposed to laboratory cyclic thermal loading and salt fog conditions. The test specimens were visually examined for cracks in the concrete and for disbondment of the CFRP sheet. Results are presented.
Keywords: Repair, upgrade, durability, carbon fibers, concrete, reinforcement

1 Introduction

This paper presents a durability analysis and visual observations of the deteriorations of carbon fiber reinforced plastic (CFRP) rods and sheets adhered to plain concrete test specimens before and after laboratory exposure to cyclic temperature and salt fog conditions.

Patch loads from outrigger pads of large mobile cranes usually exceed the capacity of older Navy piers which were designed for rail-mounted cranes. Selected Navy piers will be structurally upgraded. The proposed concrete strengthening strategies [11, a topic of ongoing research at the Naval Facilities Engineering Service Center (NFESC), consist of applying high strength plastic composites, such as pultruded CFRP rods and sheets adhered to the concrete with epoxy. Reference 1 proposes that CFRP rods be embedded in the topside of the pier deck providing negative reinforcement over the pile bents and CFRP sheets be adhered to the deck underside to provide supplemental flexural reinforcement at critical locations.

The successful long-term performance of the resulting composite structure will depend on its ability to transfer stresses, and resist cracking and disbondment in the marine environment. Before these methods and materials are applied, their ability to resist environmental conditions and loading conditions must be better understood.

Unreinforced concrete beams were cast and externally reinforced with CFRP. Twelve groups of three specimens each were exposed to different and varied environmental conditions, and then visually examined for deterioration before being loaded to failure in third point flexural loading tests per ASTM C78-94. The specimens were designed to fail under load by disbondment of the CFRP rod from the epoxy adhesive.

2 Material performance requirements

CFRP epoxy adhesive, and concrete all have substantially different physical properties such as coefficient of thermal expansion, stiffness, ductility, and strength. Changes in temperature, moisture, and loading will cause differential expansion and contractions of these materials. As a result, stresses will develop at the bond interface and within the materials. If these stresses exceed the strength of the materials, cracking or disbondment will occur. If the concrete substrate or CFRP upgrade fails, the enhanced structural capacity of the upgraded Navy structure may be insufficient to meet the new working load requirements. Therefore, laboratory testing is needed to quantify deterioration, if any, due to extreme thermal cycling and environmental conditions.

The most critical requirements for durability are to provide a sound substrate, an adequate initial bond between the external reinforcement and the existing concrete, and the long-term survival of the bond. The bond strength should ensure that the composite system behaves monolithically under load. The long-term performance of the composite upgrade may dictate the design strength and determine its stability under operational loads and environmental conditions. As long as the concrete substrate remains sound, the critical area of the upgraded structure is likely to be at the adhesive-concrete interfaces.

3 Durability analysis

Navy piers and wharves under consideration for structural upgrading are over 50 years old and suffer from moderate to severe corrosion of the steel reinforcement, resulting in cracking, delamination, and spalling of the concrete. The underside of a Navy pier deck typically contains higher levels of chloride ion contamination at the depth of rebar than the top deck, consequently the underside has greater amounts and severity of deterioration. It is also common that many reinforced concrete elements supporting the deck have been previously repaired with conventional cementitious materials. For Navy piers, ongoing corrosion-related deterioration typically requires repairs 3 to 12 years after the patch repairs have been made. A large percentage of these new repairs are found to be immediately adjacent to the previous repair. In general, the rate of corrosion activity is greater for piers in warm water locations and for concrete in the intertidal zone.

The CFRP strengthening systems under consideration supplement reinforcement provided by the rebar. Prior to the application of the CFRP, the deteriorated concrete must be repaired to provide a sound and durable substrate [2]. Conventional cementitious patches do not arrest the rebar corrosion process in the remaining structure. In many cases, the repair may accelerate the rate of rebar corrosion [3]. The installation of a cathodic protection system on the entire structure affected by chloride related rebar corrosion is one method which has the potential to stop the corrosion of the embedded steel reinforcement. If the corrosion process is NOT completely arrested, then the continued expansion of the corroding rebar may result in delamination of the concrete substrate which may undermine any CFRP system attached to the surfaces. Data predict that subsequent corrosion damage will progress to a degree that will require new repairs after 10 to 12 years [4].

Application of the CFRP sheets will affect the moisture, chloride, and oxygen concentrations at the depth of the rebar. For example, existing cracks through the pier deck will permit water to transport soluble salts and free lime to the impermeable epoxy interface. The resulting ponding of these corrosive agents will increase their concentrations at the depth of the steel reinforcement. Conversely, CFRP sheets may restrict the ingress of moisture and oxygen to the depth of the rebar. As a result, the steel rebar will be subjected to differential environmental conditions which are likely to increase corrosion potentials in the steel reinforcement, resulting in accelerated corrosion activities and possibly the initiation of new corrosion sites. For these reasons, it may be advisable to apply the CFRP sheets as narrow strips with exposed concrete between each strip in an attempt to maintain environmental equilibrium at the rebar. The proposed strengthening should be applied only after the corrosion problem has been determined and the limitations/consequences of conventional repairs are fully understood by all concerned with the structural upgrade.

In-situ loading of the completed upgraded structure occurs from many sources including: abrasion, environmental attack, temperature and moisture changes, as well as static and moving loads. Failure of the proposed upgrade system will occur when it no longer meets the demands of this general loading. Durable performance depends primarily on the ability and effectiveness of the adhesive to transfer stresses to the carbon fiber. This in turn depends on the adhesive-concrete and adhesive-CFRP bonding, interface shear stresses, and adhesive strength. Tests reported by Saadatmanesh and Ehsani [5] show that not all epoxy adhesives behave satisfactorily with the concrete-adhesive-CFRP systems~ however, adequate resin adhesives are available.

The proposed upgrades are presented in Figures 1 and 2. Both are three-phase composite systems consisting of:

1. Existing concrete substrate
2. Adhesive interface (transition zone) between substrate and the CFRP
3. Carbon fiber reinforced plastic rods or sheets

These systems have a plane of weakness at the interface. In application, this layer will be subject to moisture and temperature related strains and stresses. If the bond is not sufficient to resist the stresses, then local disbondment of the CFRP could occur. The system could then "break-up" as a result of static and moving loads and environmental forces.

Epoxy adhesion to concrete is affected by the quality of the concrete and its surface preparation, including profile, contamination, and moisture. Therefore, the performance of the bond can be improved by quality workmanship. In addition, epoxy adhesive materials are strongly influenced by environmental conditions during the curing phase and during service. In-situ environmental conditions may range from –23°C (-10°F) up to 49°C (120°F) and from very dry to high humidity. Epoxies, being organic in nature, have a coefficient of thermal expansion several times higher than inorganic materials such as concrete. Therefore, when the epoxy adhesive is subjected to temperature changes, it undergoes greater volumetric changes than the substrate, creating stresses at the bond interface. The cumulative effect of these stresses may cause disbondment due to (1) adhesive failure at the interface or (2) shear failure within the concrete substrate. Therefore, thermal and moisture compatibility of the composite upgrade system may determine its long term performance. Sprinkel [6] reported that the temperature changes bridge decks are typically subjected to are sufficient to cause deterioration and eventual failure of polymer-concrete overlays. Although a polymer-concrete overlay is not being considered in this effort, the proposed CFRP laminate system (Figure 2) is sufficiently similar to warrant concern because the epoxy adhesive has an even greater coefficient of thermal expansion than polymer-concrete and both are three-phase systems. Sprinkel reported that failures for polymer overlays can occur in the following manner:

1. *The shearing of Portland cement concrete below the bond line.* Shearing of the concrete below the bond line causes the overlay to delaminate with concrete remaining bonded to its underside. Failure is most likely to occur when the shear strength of the base concrete is low and the bond is good and the tensile strength of the overlay is high. Failure will likely occur after a few cycles of temperature change and result in the delamination of the polymer-concrete overlay.
2. *The delamination of the bond between the polymer-concrete overlay and the base concrete.* Delamination of the bond between the polymer-concrete overlay and base concrete causes the overlay to delaminate with no concrete remaining on the underside. Failure is likely to occur when the surface preparation prior to the installation of the overlay is poor or when the shear strength of the base concrete and the tensile strength of the overlay are high. Where the initial bond is poor, failure is likely to occur after a few cycles of temperature change. Where the initial bond is good, a significant number of thermal cycles may be required to complete the failure.

Research at the University of Delaware [7] reported that, "In the presence of chlorides, both wetting and drying and freezing and thawing, led to reduced ultimate beam strengths." Therefore, further research is necessary to quantify the reduction in ultimate bending strength and to identify failure mechanisms.

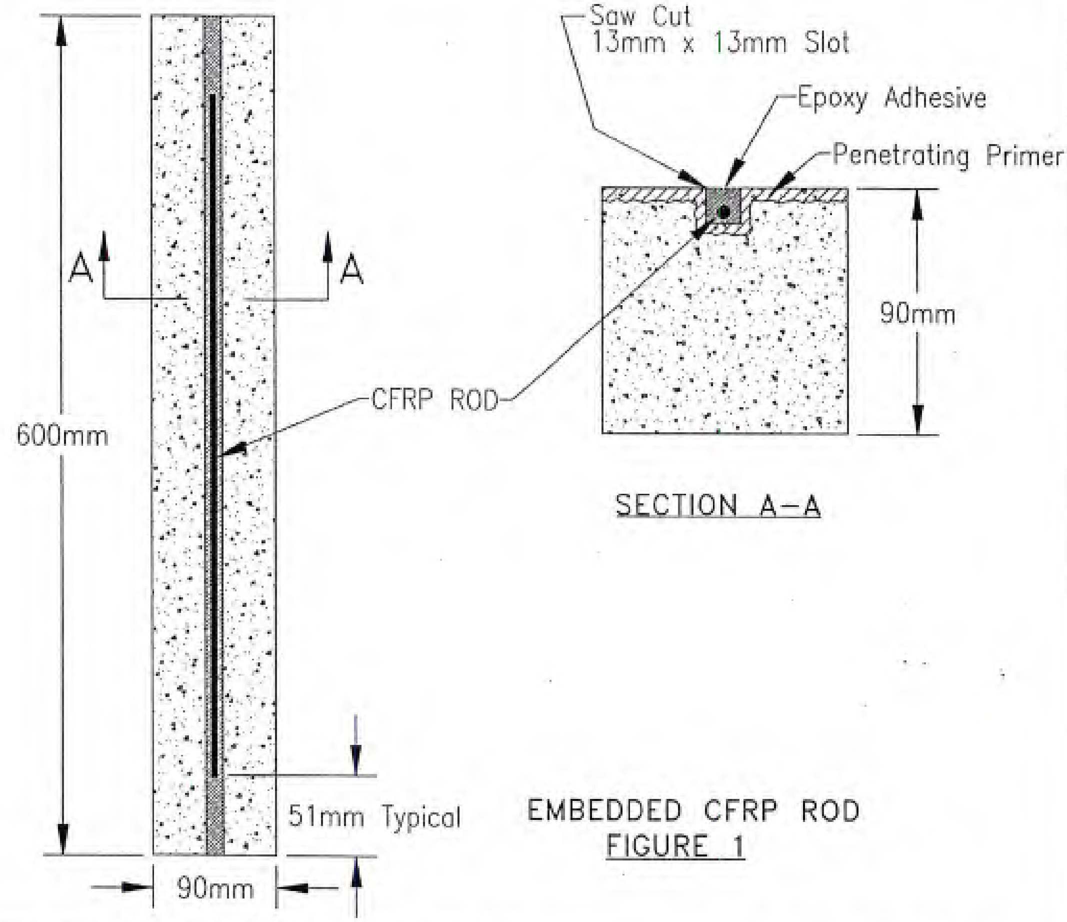
4 Estimated performance life

The estimated performance life of the upgrade systems will be governed by the quality of the existing concrete, the workmanship installing the upgrade, the electrochemical corrosion activity of the chloride-contaminated reinforced concrete substrate, and the durability of the bond at the CFRP concrete interface subjected to operational loading and environmental conditions.

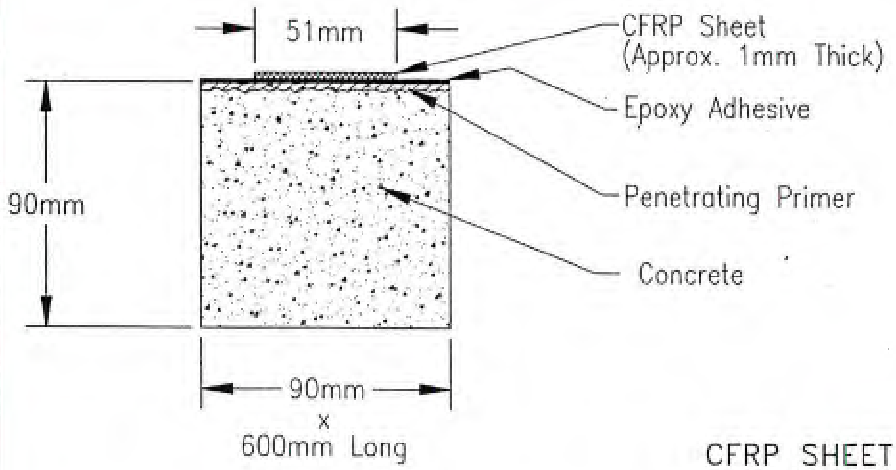
Based on the durability analysis and limited by the need for additional research, it is predicted that the ultimate life of CFRP sheets adhered to the underside of a chloride-contaminated marine structure may be limited by the ongoing corrosion of the steel reinforcement. Corrosion of the rebar is likely to cause a plane of delamination between the rebar and the externally applied CFRP. This mechanism could result in the failure of the concrete substrate in 3 to 12 years.

The predicted performance of the embedded CFRP rods in the top deck is likely to be longer lived than the underdeck application of CFRP sheets for two reasons: (1) corrosion of the rebar in the top deck of a Navy pier is typically progressing at a slower rate compared to rebar corrosion in the underdeck elements, and (2) the surface area of adhesive concrete interfaces is substantially less.

For the CFRP upgrading methods proposed, environmental factors such as moisture and temperature in combination with operational loading may limit the life performance of the upgraded structure. Identification of failure modes that may supersede failure by delamination of the concrete substrate were investigated in the laboratory testing.



EMBEDDED CFRP ROD
FIGURE 1



CFRP SHEET
FIGURE 2

THREE–PHASE STRENGTHENING SYSTEMS

5 Preparation of concrete test specimens

In the preparation of the concrete test specimens, ready mix concrete was ordered from a local supplier. The mix design, which represented the minimum concrete quality of Navy piers, was 24 MPa (3,500 psi) at 28 days with a maximum aggregate size of 10mm (3/8 inch), and an air content of 7.5 percent on delivery. Three kilograms per cubic metre (5 pounds per cubic yard) of NaCI was added to the mix to approximate the chloride contamination of Navy structures. A set retarder was used to off-set the set acceleration effects of the sodium chloride. The mix design is as follows:

Sand	703 kg (1,550 lb)
10mm (3/8-inch aggregate	480 kg (1,058 lb
Cement	295 kg (650 lb)
Water	159 kg (350 lb)
Polyheed 300R	770 ml (26 fluid ounces)
Micro Air	295 ml (10 fluid ounces)

The wet unit weight was 84.2 kg/cu.m. (142 lb/cu.ft) using ASTM Standard C 138. The slump was 51mm (2 inches) following ASTM Standard C 143. Initial air content after the addition of the NaCI was 4.9 percent and the last material taken from the mixer was 4.4 percent air using ASTM Standard C231.

All of the specimen molds were filled from a single batch and vibrated on the vibrator table and moved immediately to the NFESC moist curing room where they were finished. One hundred twenty-five specimens 90mm x 90mm x 600mm (3.5 x 3.5 x 24 inches) long were made. The specimens were covered with wet burlap and stripped the following day and returned to the moist curing room to cure.

6 Application of the CFRP rods and sheets

After 28 days of moist cure, slots were cut into the surface of some of the concrete beams with a diamond saw. An epoxy penetrating primer was applied prior to filling with the epoxy resin to enhance adhesion and to mitigate or minimize cracks from forming at the interface. All of the CFRP materials tested are composed of approximately 65 percent carbon fiber and 35 percent resin. Two brands of CFRP rods were tested: Brand X was 3mm in diameter with a smooth surface and Brand Y was 3mm and 5mm in diameter with indented surfaces. The epoxy used for adhesion of the rods and sheets was the same for each and was recommended by the CFRP reinforcement manufacturer. Figure 1 illustrates the completed upgrade for the CFRP rods per the following procedure:

1. Clean the slot with a brush and blow out loose material with compressed air.
2. Prime the slot with epoxy primer.
3. Seal the ends of the slot.
4. Fill the slot approximately 1/4 full with epoxy resin.
5. Place the CFRP rod and instrumentation (if applicable).

6. Pour epoxy resin to the top edge of the slot completely covering the CFRP rod.

Additional test specimens were upgraded with CFRP sheets to the surface of the concrete. The pultruded flat sheets are 51mm x 1 mm thick. One manufacturer of CFRP sheets was tested, Brand Z. Figure 2 illustrates test beams upgraded using CFRP sheets.

7 Exposure of test specimens

The CFRP must transfer the internal stresses to the concrete without loss of bond. Although many different loading conditions contribute to these combined internal stresses; thermal cycling is one that induces severe stresses and can be readily and repetitively applied to the test specimens. In addition, test specimens were exposed to 46°C (115°F) salt fog conditions in the laboratory to simulate extreme waterfront exposure conditions.

Each group contains three specimens (listed in Table I). One group was exposed to one cycle to –23°C (-10°F). Other groups were exposed to cyclic temperature in the NFESC Envorotronics chamber, model #ET64-2-5, -23°C to +60°C (-10°F to 140°F) every 4 hours. In addition, two groups of specimens were exposed to continuous salt fog conditions per ASTM B 117-94 Standard Practice for Operating Salt Spray (Fog) Testing Apparatus at 46°C (11 5°F) for 1 year.

8 Test results

When cyclic exposure was completed, all beam specimens were visually inspected for loss of adhesive bond and cracking. Some specimens were loaded to failure using the NFESC 54,432-kg (120-kip) Baldwin machine per ASTM C76-94. Table 1 and Figures 3, 4, and 5 contain the results of these investigations.

Table 1. Visual Deterioration

Product	CFRP Upgrade Method	Control A18°C	Thermo Cycles and Temperature			Salt Fog @ 46°C 115°F		
			One Cycle -23° to 18°C	-23° to 60°C (-10° to 140°F)				
				762 Cycles	1188 Cycles	124 Days	250 Days	370 Days
X	3mm Rod	none	none	none	(a)	(b)	(b)	(b)
Y	3mm Rod	none	none	none	(a)	none	(c)	(c)
Y	5mm Rod	none	none	none	(a)	(b)	(b)	(b)
Z	1x51mm Sheet	none	none	none	none	none	none	none

(a) All specimens cracked longitudinally adjacent to the epoxy groove (See Figure 3).
(b) Not Tested
(c) To be determined

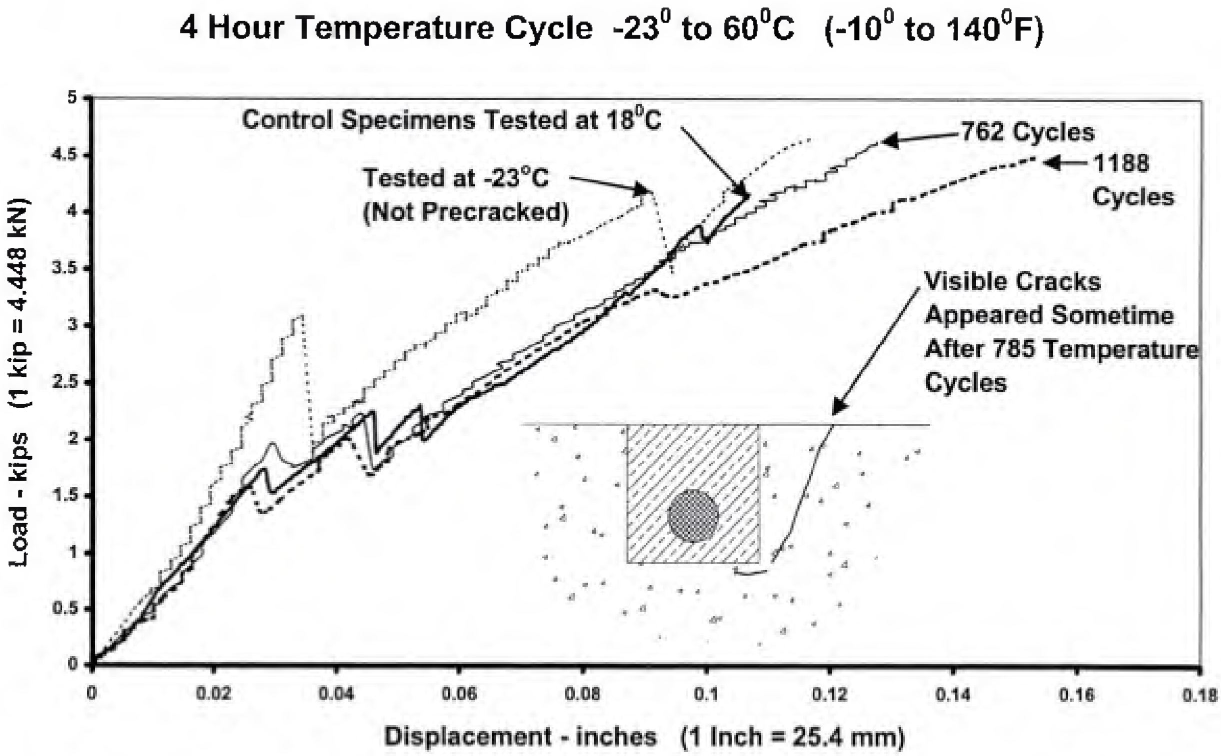


Figure 3. 3mm Smooth Carbon Rod Summary

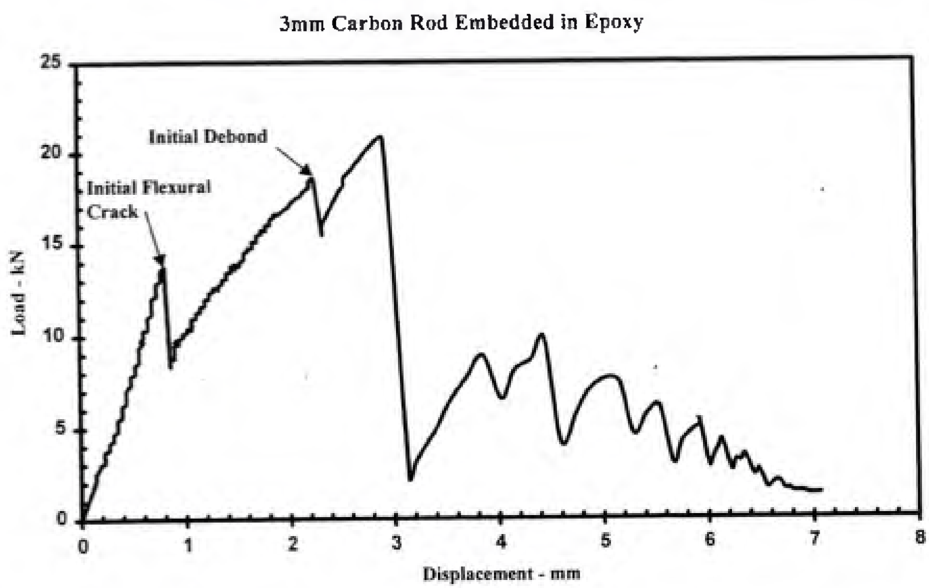


Figure 4. Load displacement for beam specimen that reinforcement debonded from epoxy.

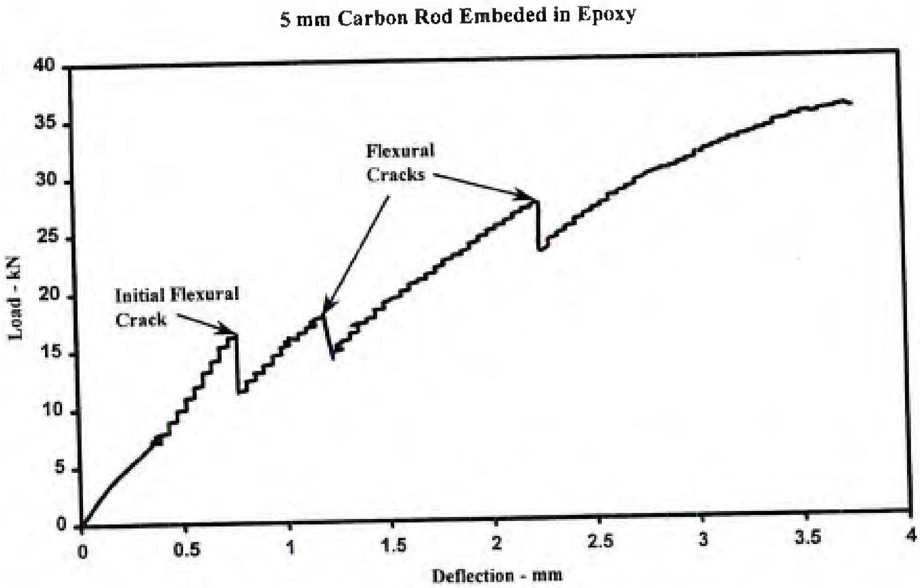


Figure 5. Load-displacement for beam specimen that substrate (concrete) failed.

9 Discussion

Exposure of the test specimens to a differential temperature of 83°C (150°F) for 762 thermal cycles did not result in visible cracking. However, after 1188 cycles, all of the specimens upgraded with CFRP rods contained visible cracks in the concrete adjacent to the groove filled with epoxy. No observations were made between 762 and 1188 cycles. Exposure to continuous 46"C (1 15"F) salt fog conditions did not result in any visible deterioration of the epoxy to adhere the CFRP strips or rods to the concrete.

10 Field demonstration

It is planned to demonstrate the technology by upgrading three Navy piers and wharves. For example, selected structural members and deck areas on Pier 12 at the Naval Station San Diego will be upgraded by embedding CFRP rods on the pier deck topside and sheets to the underside. Dynamic load tests have been conducted to provide a structural baseline for the demonstration projects.

11 Conclusions

A marine durability analysis concluded that the estimated performance life of CFRP pultruded rods and sheets adhered to the concrete surfaces of a Navy pier will be governed by the quality of the existing concrete, the workmanship of the upgrade, the electrochemical corrosion activity of the chloride-contaminated reinforced concrete substrate, and the durability of the bond at the CFRP-concrete interface when subjected to operational loading and environmental conditions.

Failure of the concrete substrate caused by corrosion of the steel reinforcement is one failure mechanism that may limit the life performance of the upgrade. Because of the high number of thermal cycles required to produce visible cracks in laboratory specimens, the proposed use of CFRP reinforcement should be adequate in a temperate climate.

12 References

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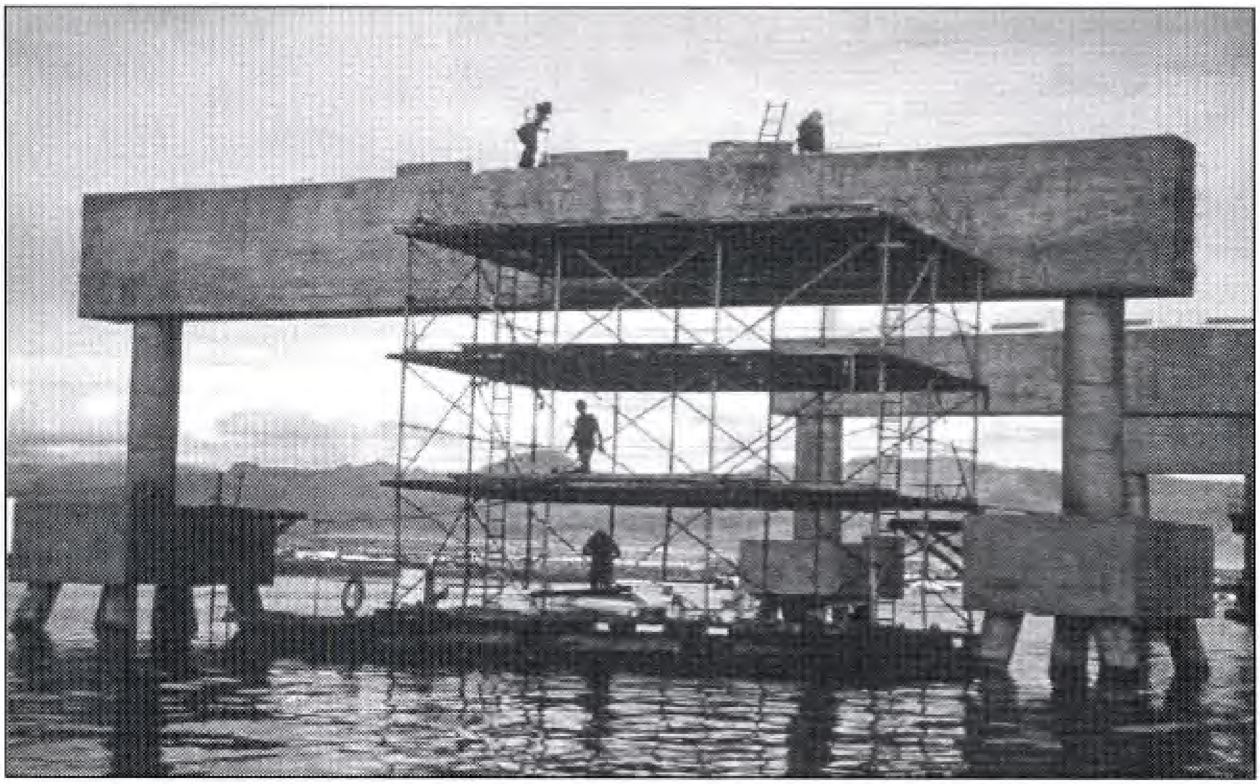
1998 NACE Western Area
SSPC & Joint Military
Corrosion Conference
October 13-23
Honolulu, Hawaii

Use of New Generation Epoxy-Coated Rebar In the Admiral Clarey Bridge

Douglas F. Burke, NFESC
Melvyn T. Tsutahara, PACNAVFACENGCOM

INTRODUCTION

The design and construction of the Admiral Clarey Bridge exemplifies the use of durable reinforced concrete in a marine environment. Planners, designers, and builders must pay great attention to the many critical factors that ultimately contribute to the durability of the reinforced concrete. This paper provides a brief summary of some of the important concrete material issues related to performance with particular emphasis on supplemental corrosion protection using new standards for prefabricated epoxy-coated steel rebar.



Construction of the Admiral Clarey Bridge, Pearl Harbor, Hawaii

THE ADMIRAL CLAREY BRIDGE

The Admiral Clarey Bridge connecting Ford Island to the Pearl Harbor Hawaii Naval Complex was dedicated April 15, 1998. The 4,700-foot long bridge is one of six reinforced concrete floating bridges in the world. The 650-foot moveable span is the longest in the world.

The request for proposals (RFP) for the design/build contract was developed with the assistance of a number of people and organizations. For the bridge concept, the RFP relied heavily on studies by the Pacific Division Naval Facilities Engineering Command's (PACNAVFACENGCOM) Planning Department. These included various Ford Island Access studies accomplished in 1987/88 and the Final Environmental Impact Study in 1990.

The RFP provided specific design criteria, which were developed mostly by PACNAVFACENGCOM's Design Division engineers with assistance from the following:

- Naval Facilities Engineering Command, John Headland, Coastal Engineering.
- Washington State Department of Transportation, Myint Lwin, floating pontoon section and concrete.
- Federal Highway Administration, Raymond McCormick, highway/bridge.
- Naval Facilities Engineering Service Center (NFESC), Douglas Burke, prefabricated fusion-bonded pipeline-type epoxy-coated reinforcement.

Because rebar corrosion occurs much faster in a tropical environment, such as Hawaii, it was particularly important that emphasis be placed on the design of the concrete materials to provide long term durability. To maximize concrete durability, these design decisions were made:

- The use of 5 percent silica fume was recommended by Mr. Lwin based on his experience and success in using silica fume on the most recently constructed Washington State floating bridges.
- The use of a maximum allowable water-to-cement ratio (w/c) of 0.38 was based on waterfront engineering practices using locally available Hawaiian concrete materials.
- The use of a zero tension under service load criterion was based on the State of Hawaii Department of Transportation requirement for all bridges in Hawaii.
- The use of prefabricated fusion-bonded epoxy-coated rebar was a difficult decision to specify since the Navy's new standard was still under development by NFESC and the increased cost was uncertain. Ultimately, 4,600,000 pounds of epoxy-coated mild reinforcing steel was used to construct the bridge.

COST/BENEFIT

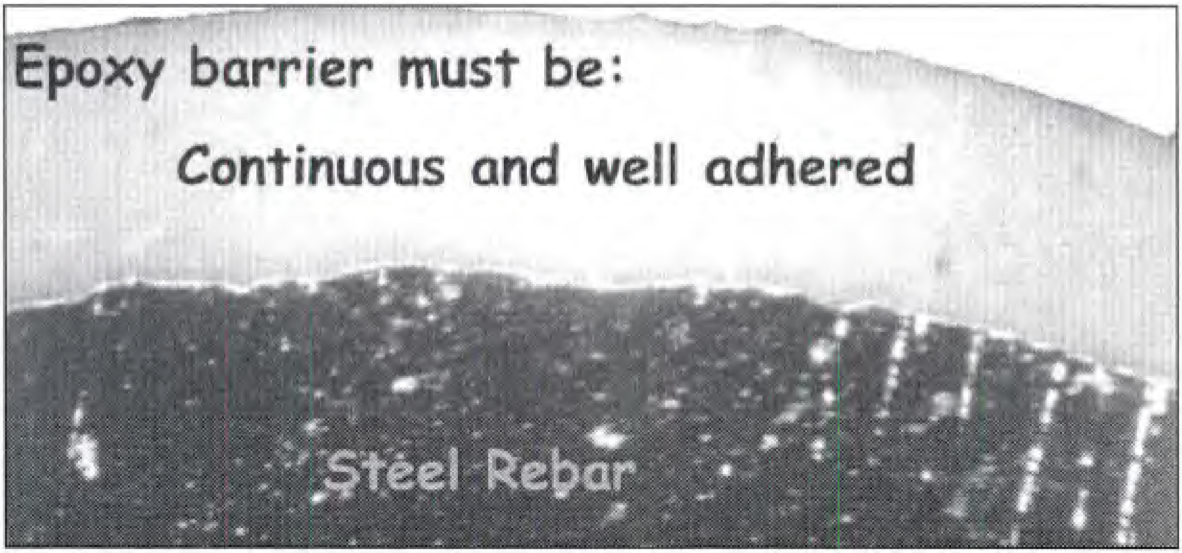
For the Admiral Clarey Bridge a cost comparison of using plain steel rebar versus prefabricated epoxy-coated rebar was done. The predicted costs were $1.20/pound for plain rebar compared to $1.60/pound for coated rebar, installed. Therefore the additional cost amounted to ($0.40) x (4,600,000) = $1,840,000. Since the cost of the total project was $86 million, the premium to use this technology was 2.1 percent. Use of high quality concrete materials and workmanship with proper concrete cover should provide a 50-year service life. Field performance evaluations and accelerated laboratory tests of coated rebar indicate that the technology will provide a substantial increase in life performance. Rebar life extension on the order of 20 to 40 years is a rational expectation.

EPOXY-COATED REBAR DEVELOPMENT

The decision to use epoxy-coated rebar in new Navy construction was based on extensive evaluations that began in 1984, when the Office of Navy Research tasked the Naval Civil Engineering Laboratory to conduct long-term field evaluations. Test specimens were suspended in a marine intertidal zone for 76 months at Key West, Florida to rank the relative performance of popular corrosion control methods. Damage-free epoxy-coated rebar performed best. Results from this study were presented by the Concrete Reinforcing Steel Institute in their Research Series 2 report of July 1994.

Despite the good performance in the Navy's long-term field tests, the Florida Department of Transportation and other agencies had found moderate to severe corrosion much earlier than expected on some marine structures using epoxy-coated rebar. By 1994, much controversy surrounded the use and performance of epoxy-coated steel reinforcing bars produced and placed in accordance with current specifications. Consequently, the Navy Criteria Office funded the NFESC to identify the failure mechanisms in current practices and to develop a new standard in cooperation with industry experts. This effort resulted in an Interim User's Guide for Prefabricated Epoxy-Coated Rebar for Oceans and Other Severe Environments (PROSE). The document included two new Navy Facilities Guide Specifications (NFGS), 03201 and 03202, and recommendations for a quality control program. The Navy Criteria Office identified candidate construction projects to incorporate the new generation of epoxy-coated rebar. Two Navy submarine piers were constructed, one in Pearl Harbor, Hawaii, and the other in New London, Connecticut. NFESC monitored the construction of each project and evaluated the cost and constructablity. Both projects proved highly successful and the differential costs were about 2 percent higher for each with respect to the overall construction cost. The toughness of the new epoxy powder formulation developed by 3M proved exceptionally good, requiring very few repairs after shipping, storage, and placement. The bridge also included small sections of epoxy-coated rebar coated with epoxy powder formulated by Akzo Nobel and Herbert's-O'Brien, which appeared to be equally durable.

The American Society of Testing and Materials (ASTM) used the Navy's draft specifications as a basis for the development of ASTM A 934/A 934M published in July 1995, "Standard Specification for Epoxy-Coated Prefabricated Steel Reinforcing Bars." Mr. D. Burke of NFESC is the current chairman of the ASTM Subcommittee A01.05 task group for development and revision of coated reinforcement standards. In February 1998, the NAVFAC Criteria Office published, for the first time, a definitive guide for Marine Concrete, NFGS 03311. Included is a requirement to use prefabricated epoxy-coated reinforcing steel according to the new ASTM Standard.



Magnified View of Epoxy-Coated Rebar

IMPORTANT FEATURES FOR ENHANCED PERFORMANCE

There are many important features of the new technology for prefabricated epoxy-coated rebar contained in the ASTM Standard that contribute to improved performance. Some of these are:

- All of the rebar is prefabricated to final size and shape prior to coating. This avoids stress cracks in the coating and loss of coating adhesion in the bend areas during post fabrication, which has been a typical site for corrosion.

- Since the coating no longer needs to be flexible, new epoxy powder formulations can be used. These formulations are more durable and resistant to the intrusion of corrosive elements.

- Extensive quality control tests must be performed on every batch of coated rebar, including cathodic disbondment tests for coating adhesion. This requirement greatly reduces problems with underfilm corrosion.

- All visible defects in the coating must be repaired prior to concrete placement. This minimizes the number of locations in the barrier coating that might otherwise become corrosion sites.

In addition to the recommendations contained in the ASTM standard, NFESC strongly recommends that:

- Coated rebar is not mixed with plain rebar in the structure. This avoids the possibility of creating a large corrosion cell if there is electrical continuity between the coated and uncoated steel.
- Coated rebar should not be used in structures that are subject to large impact loads and in areas where the steel is severely congested (e.g., 50 percent or more of the cross section is steel). Because of the lack of adhesion of the cement paste to the epoxy coating, the concrete that covers the reinforcement may disbond when subject to impact loads, which was reported when a reinforced concrete component was accidentally dropped.
- Designers should not specify the use of coated rebar that exceeds number 11 (2-3/4" diameter) until definitive data is available that addresses the effect on bond and anchorage.

CORROSION ACTIVITY

The purpose of providing supplemental corrosion protection, such as an epoxy coating, is to reduce the rate of rebar corrosion, thus increasing the time before corrosion related repairs are necessary. This is accomplished in two important ways:

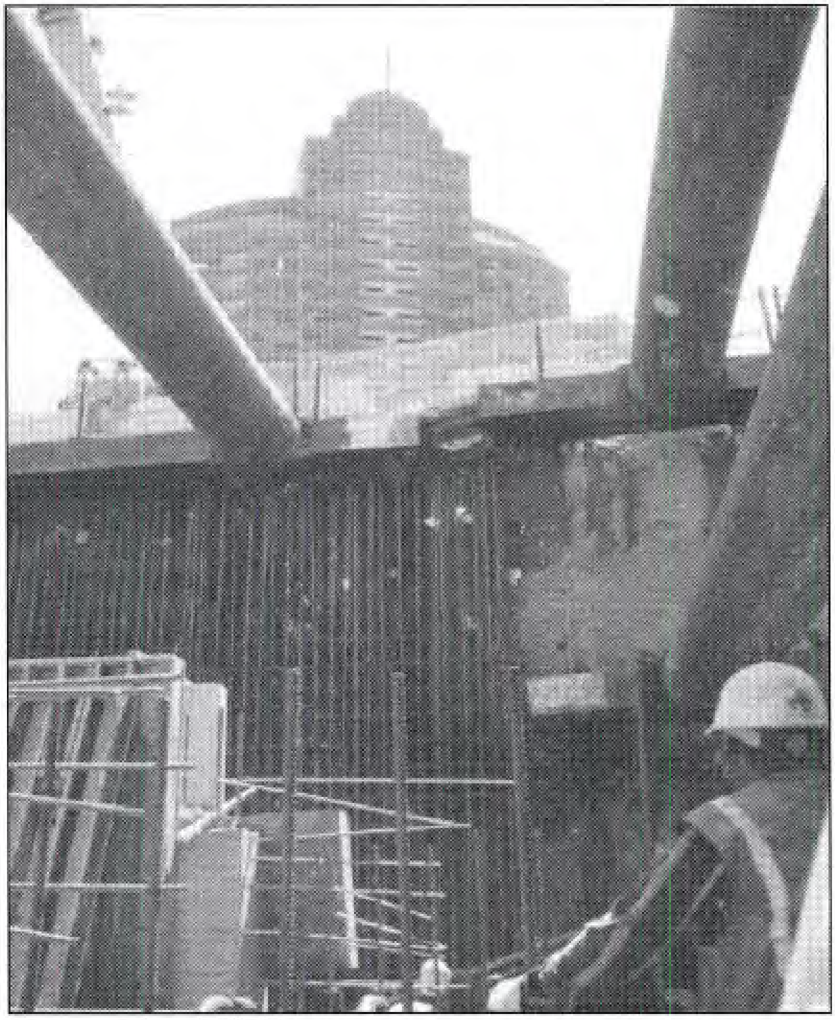
- If the quality of the concrete is compromised in any manner that results in cracks, increased concrete permeability, or reduced concrete cover, then chloride, oxygen, and water will find their way to the rebar sooner than expected. An excellent barrier coating on the steel will extend the time before corrosion will take place.
- Eventually corrosive elements will reach the rebar regardless of the concrete materials used and the quality of the workmanship. When the chloride contamination reaches the threshold level necessary for the initiation of steel corrosion, the presence of a highly impermeable well-adhered barrier coating with a minimum number of defects will retard the potential for corrosion activity in the steel reinforcement.

CONCLUSION

Concrete durability in a marine environment requires strict attention to many important aspects of planning, materials, design, and workmanship. Life performance of marine structures can be enhanced by the use of prefabricated epoxy-coated steel reinforcing bars with good coating adhesion and no visible damage to the coating. Construction of the Admiral Clarey Bridge exemplifies the use of these design and construction principles.

OTHER PROJECTS USING EPOXY-COATED REBAR

Several projects within the Navy and in the private sector have used the new standards for prefabricated epoxy-coated rebar, such as, the Muni-Metro Turn Back in San Francisco, California and the Long Beach Aquarium in Long Beach, California. In June 1997, the technology was reviewed and adopted by the California Department of Transportation (CALTRANS) for reinforced concrete structures in contact with sea and brackish water.



Construction of Muni-Metro Turn Back, San Francisco, California

ADDITIONAL INFORMATION

Additional information about the design and construction of the Admiral Clarey Bridge is contained in an excellent and comprehensive article by Michael Abrahams and Gary Wilson featured in the PCI Journal July/August 1998 issue. For more information about the use of prefabricated epoxy-coated reinforcement, please contact Douglas Burke at 805-982-1055 or burkedf@nfesc.navy.mil.

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NAVAL FACILITIES ENGINEERING SERVICE CENTER
Port Hueneme, California 93043-4370

Special Project
SP-2005-SHR

LIMITED FLEXURAL TESTS OF PLASTIC COMPOSITE PILE CONFIGURATIONS

by

George Warren

August 1996

Sponsored by
Naval Facilities Engineering Command

Executive Summary

The Naval Facilities Engineering Service Center (NFESC) teamed with the Composites Institute of the Society of Plastics Industry (CI/SPI), the U.S. Army Construction Engineering Research Laboratory (USACERL), and Rutgers University on a Construction Productivity Advancement Research (CPAR) Project to develop, test and demonstrate high performance, polymer composite structural pilings, fender pilings, and sheet piling for marine/waterfront civil engineering construction. NFESC conducted flexural load tests on candidate pile configurations and determine flexural stiffness. The test program covered reinforced, recycled plastic and fiberglass-reinforced composite plastic fender and bearing piles. Twelve pile configurations were submitted by seven manufacturers. This report provides test results and should not be construed as either an endorsement or condemnation of specific product or products.

Although the test results are mixed, reinforced recycled plastic and concrete-filled composite shells can be attractive structural alternatives to timber for fender and bearing piles.

The pile concept of placing concrete inside a composite shell is worthy of further investigation by the Navy. Concrete is relatively cheap and superior for resisting compressive forces. Concrete will prevent localized buckling and pinching of the composite shell while increasing bending stiffness. A properly designed plastic composite shell is external reinforcement that serves as a stay-in-place form which will provide protection for, and enhance confinement performance of, the concrete.

Quality control and assurance will be important issues in plastic and composite piles as it is in concrete construction. Dimensional variation (thickness, out of roundness, position of rebar, etc.) and material consistency are most significant. Procurement of recycled polyethylene should require quality assurance that cracking would be grounds for pile replacement by the manufacturer.

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LIMITED FLEXURAL TESTS OF PLASTIC COMPOSITE PILE CONFIGURATIONS

Naval Facilities Engineering Service Center

G Warren

INTRODUCTION

The U.S. Navy has been proactive in the application of plastic composites to waterfront structures. The use of composite piles is a positive step in the direction the Navy is taking towards a more environmentally friendly and maintenance free waterfront infrastructure. The Naval Facilities Engineering Service Center (NFESC) has teamed with the Composites Institute of the Society of Plastics Industry (CI/SPI), the U.S. Army Construction Engineering Research Laboratory (USACERL), and Rutgers University on a Construction Productivity Advancement Research (CPAR) Project to develop, test and demonstrate high performance, polymer composite structural pilings, fender pilings, and sheet piling for marine/waterfront civil engineering construction. NFESC's role was to conduct flexural load tests on candidate pile configurations and determine flexural stiffness. Limited data was collected on a narrow range of pile properties. This report provides test results and should not be construed as either an endorsement or condemnation by the Navy of specific product or products.

SCOPE

The test program covered reinforced recycled plastic and fiberglass-reinforced composite plastic fender and bearing piles. Twelve pile configurations were submitted by seven manufacturers.

CONFIGURATIONS DESCRIPTIONS

Table 1 lists descriptions of the pile test configurations. There were six configurations that employed recycled polyethylene reinforced with steel or fiberglass-reinforced plastic (FGRP) rods. Configurations AF (13-inch (33 cm) diameter), CF (13-inch (33 cm) diameter), and Configuration DB (16-inch (41 cm) diameter) were reinforced with GFRP rebars. Steel reinforcement in Configuration BF, EF and GF included ASTM Grade 60 reinforcing bars, ASTM A53 Grade B structural steel pipe, and 8,000-psi-(54 MPa)-concrete-filled steel pipe. The steel reinforcing cage of the former was welded and included spiral confinement steel. Steel or fiberglass rebars were arranged in a circular configuration concentric with the pile cross section. Configurations AF and DB used a high density polyethylene shell containing the reinforcement and surrounding a core of low density polyethylene. Configurations BF, CF, EF, and GF contained a mixture of high and low density recycled polyethylene.

There were six configurations employing fiberglass composite cylindrical shells. Configuration FB used an epoxy matrix, Configurations HB, KF, and LF used vinylester, while Configuration IB used polyester. Configuration FB, a filament wound fiberglass-reinforced epoxy composite bearing pile specimen, was filled with a lightweight concrete (5,000 psi (34 MPa) measured strength (by coring) and 2.2×10^6 psi (14,700 MPa) Young's Modulus), Configuration LF was filled with regular concrete (estimated at 2,900 psi (17 MPa)), while the others were hollow. Configuration IB was tapered to mimic a power or light standard pole with flexural stiffness decreasing from the base to the top while all others were right circular cylinders. The three bearing pile specimens of Configuration HB, using longitudinal and wound fibers, included two that were spliced at midspan. Fender pile specimens Configuration KF were fabricated using SCRIMP (Seemann Composite Resin Infusion Molding Process) with longitudinal and diagonal directional fibers (3 plies). Concrete-filled Configuration LF was also fabricated using SCRIMP.

Table 1. Plastic and Composite Pile Configurations. (1 inch = 2.54 cm.)

Specimen	*Nominal Size*	*Description*	*Reinforcement*	*Test Setup*	*Notes*
AF	13 inch diameter	Recycled Polyethylene	8 - GFRP Rods 1.25" dia, concentric	A	1
BF	13 inch diameter	Recycled Polyethylene	8 - 1" dia Steel Rebars and spiral cage	A	2
CF	13 inch diameter	Recycled Polyethylene	8 - GFRP Rods 1" dia, concentric	B	
DB	16-inch diameter	Recycled Polyethylene	16 GFRP Rods 1.25" dia, concentric	B	1
EF	13-inch diameter	Recycled Polyethylene with steel pipe	6-inch dia Sch 40 steel pipe	A	
GF	13-inch diameter	Recycled Polyethylene with concrete-filled steel pipe	6-inch dia Sch. 40 steel pipe encasing concrete	A	
FB	10-inch diameter	Epoxy Composite, concrete-filled shell	E Glass fibers	C	
HB	15-inch diameter	Vinylester composite hollow shell	E Glass fibers	B	3
IB	Tapered, 11 to 15 inch diameter	Polyester composite hollow tapered shell	E Glass fibers	A	
JB	12 inch X 12 inch	Vinylester composite grid section	E glass fibers	B	
KF	13-inch diameter	Vinylester composite hollow shell	E glass fibers	B	4
LF	13-inch diameter	Vinylester composite concrete-filled shell	E glass fibers	B	5

Notes:

1. Low density polyethylene core surrounded by fiberglass-rod-reinforced high density polyethylene.
2. Welded steel reinforcing cage.
3. One continuous specimen and two specimens with splices at midspan
4. SCRIMP process. Two cylindrical shell configurations: 1) 3-ply constant thickness and 2) 3-ply thickness with built-up, 8-ft (2.44 m) length at load point.
5. 3-ply constant thickness Config KF filled with concrete.

LOADING ARRANGEMENT

The test arrangement was not designed to mimic any specific field application but to provide a common basis for comparing the flexural stiffness of different cross sectional configurations and identify bending characteristics. Fender piles in Naval service are subjected to sharp concentrated loads applied through "camels" (or by similar means), which are rigid floating cylindrical members aligned perpendicular to the axis of the piles to distribute ship berthing and mooring forces among the fenders (Figure 1). On the other hand, bearing piles supporting pier decks are not subjected to concentrated loads (except for debris and ice). Stiffness of bearing piles is of interest for evaluating handling and driving limitations.

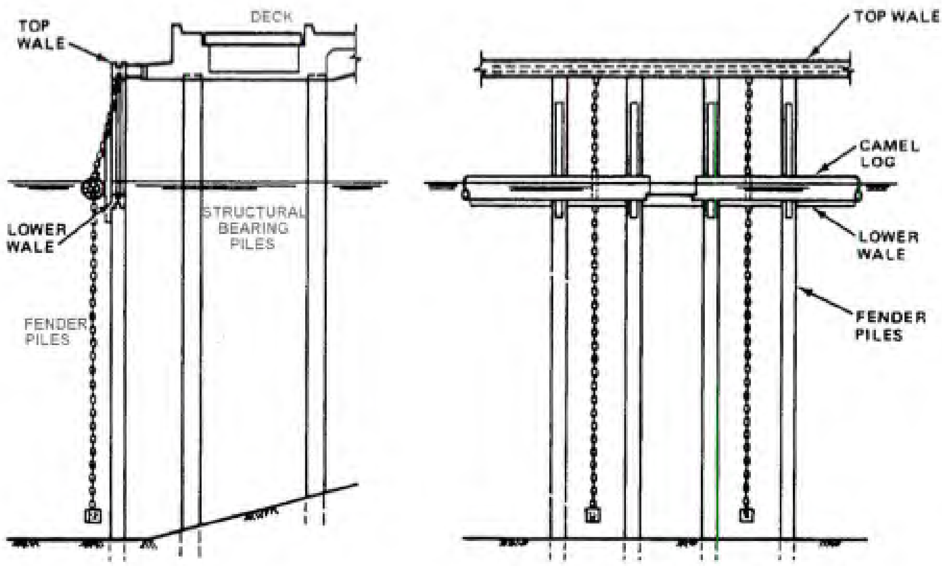


Figure 1. Conventional fender and structural piles.

Pile specimens were submitted in various lengths and the test spans consisted of 22 feet (6.7 m), 30.5 feet (9.3 m), and 32 feet (9.75 m) in a four-point loading arrangement (Figures 2, 3 and 4). The loading configuration provided spans of constant shear and a span of constant moment and zero shear. Strain gages and deflections gages were attached at locations shown. Load was applied at a deflection rate of approximately 2 inches (5 cm) per minute by horizontal hydraulic rams (Figure 5). The load was applied through a spreader beam to 3-inch (8 cm) wide oak saddles separated 6 inches (15 cm) or 24 inches (61 cm) (Figure 6). The oak saddles were lined with ½-inch (1.3 cm) neoprene and fitted to the cross section of each pile. The reaction points consisted of a wood reaction pad, a 1-inch (2.5 cm) thick steel plate, and a 3-inch (8 cm) diameter steel roller. Since the load was applied horizontally, the pile weights were supported on roller bearing pads at the quarter point of their lengths. The static, horizontal resistance of these roller supports was less than 50 lbs (220 N).

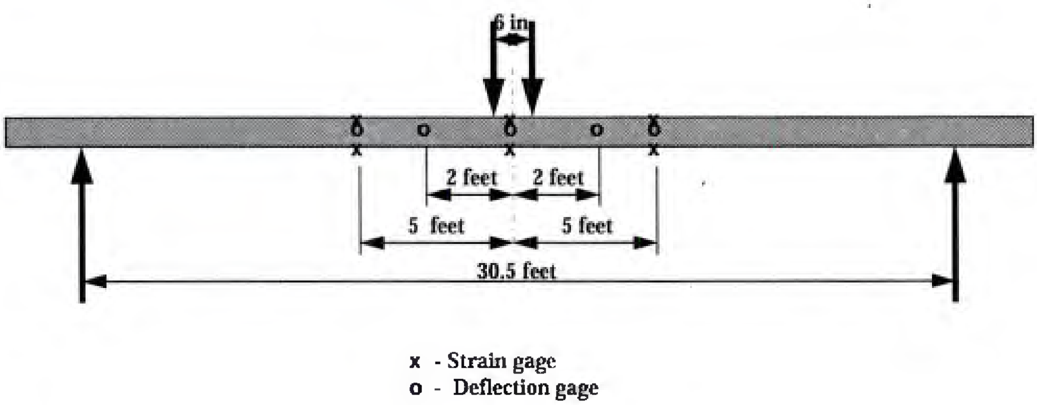


Figure 2. Test setup A. (1 inch = 2.54 cm, 1 foot = 0.305 m).

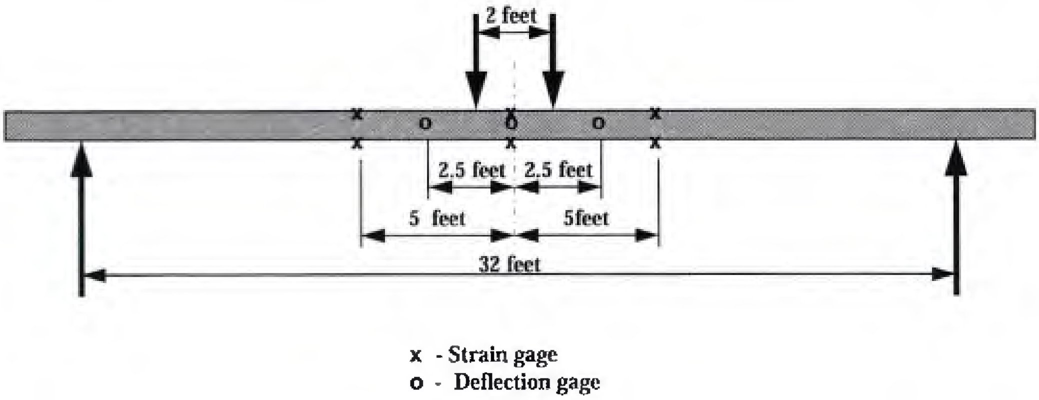


Figure 3. Test setup B. (1 foot = 0.305 m).

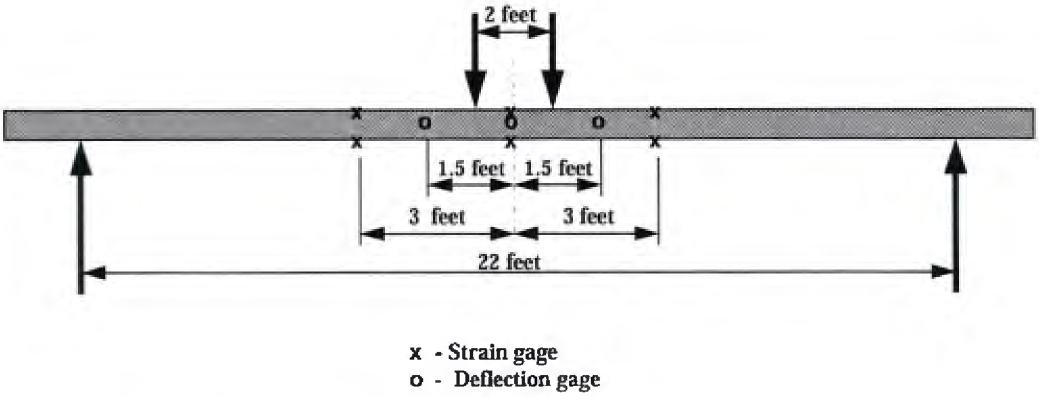


Figure 4. Test setup C. (1 foot = 0.305 m).

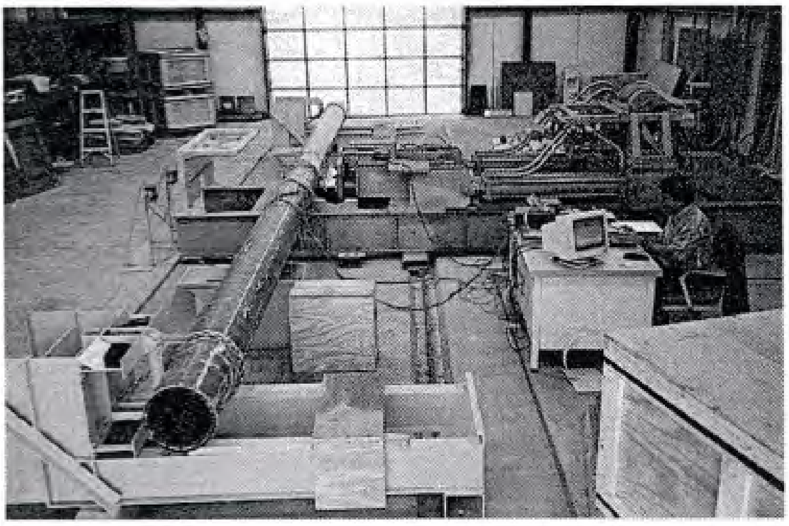


Figure 5. Load test in progress.

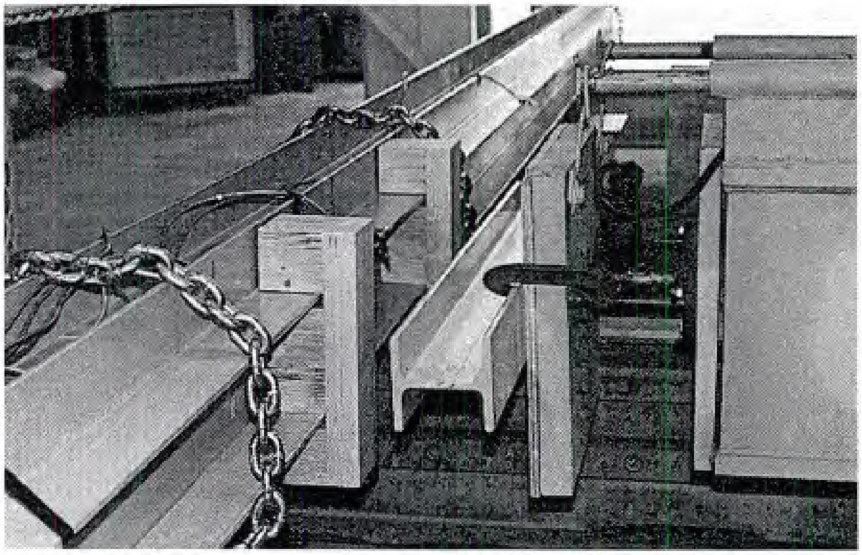


Figure 6. Spreader beam and oak saddles at load point on grid section pile specimen.

Figure 7 is a schematic of the instrumentation. Strain gages were mounted on outer surfaces in the plane of loading to acquire maximum bending strain. The strain gages are Micro-Measurement, 350 ohm, 1 inch (2.5 cm) length, foil gages which were adhered to the recycled polyethylene with Dexter Corporation Epoxi-Patch 907 Blue base coat and Micro-Measurement's M-Bond 200 adhesive. Hardman's extra fast setting epoxy 04001 was used as a base coat on all fiberglass composites with M-Bond adhesive. The deflection gages are Houston Scientific rotary pot transducers. The Load cell is a 50-kip (220 kN), BLH strain gage type. The data was acquired with a Campbell 21X data logger sampling at 1 second intervals and downloaded to a PC for processing using Microsoft Excel.

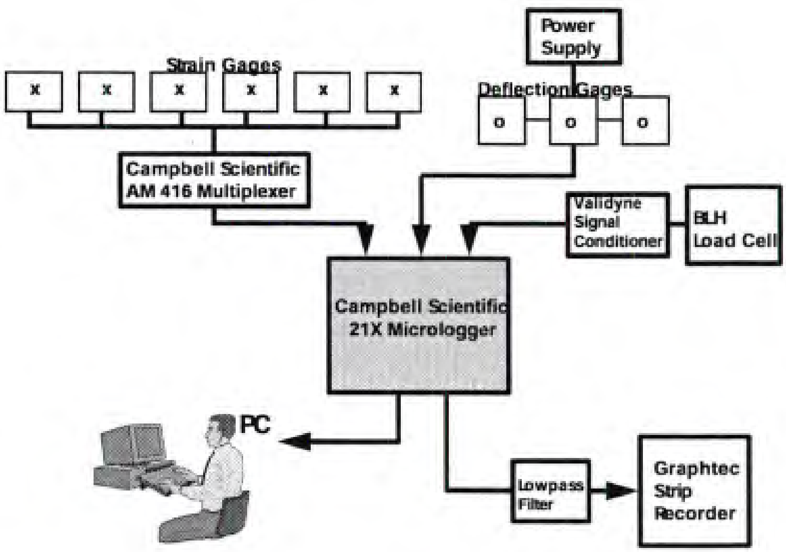


Figure 7. Test instrumentation schematic.

Test load limits were governed by specimen strength, deflection limits of the hydraulic rams, and cross sectional geometry. Fender pile configurations were deflected to the displacement limits of the loading rams (36 inches), or to rupture of the reinforcement, or until localized shell buckling occurred. Tests of the bearing pile configurations were terminated prior to 20 inches (51 cm) of load-induced deformation. Further, loading of hollow composite shells and grid section bearing pile specimens were limited to smaller deformations governed by shell/plate thickness and diameter/depth. Diameter/sectional changes at load points were kept to less than 0.05 inches (0.13 cm). Local shell or plate buckling would have occurred at concentrated load points long before attaining material compressive or tensile limits.

Load cycling was applied to determine the variation in the load-deflection response from cycle to cycle such as hysterisis and permanent deflection or creep. Strain monitoring assured that linear limits of the reinforcing materials was not exceeded during the load cycling.

STRUCTURAL PARAMETER IDENTIFICATION

Strain was the primary structural parameter measured with deflection as a backup check. Strain provides an excellent means of determining flexural stiffness, EI. Strain measurements are easily referenced to zero and are independent of movement and deformation unrelated to flexural stiffness such as rigid body movement, support settlement/deformation, or localized section-out-of-roundness caused by supports and load points. Deflections were used to provide a less refined check on linearity and flexural stiffness.

For the loading configurations shown in Figures 2, 3, and 4, the relationship among applied load, **P**, moment, **M**, deflection, Δ, strain, ε, and curvature, ϕ, are as follows:

$$M_{max} = P\,a / 2$$

$$\phi = (\varepsilon_t - \varepsilon_c) / d = M / EI$$

$$EI = M\,d / (\varepsilon_t - \varepsilon_c)$$

$$\phi_{max} = P\,a / 2EI$$

$$\begin{aligned}\Delta_{max} &= P\,a\,(3L^2 - 4a^2) / 48\,EI \\ &= M_{max}\,(3L^2 - 4a^2) / 24\,EI \\ &= \phi_{max}\,(3L^2 - 4a^2) / 24\end{aligned}$$

$$\phi_{max} = 24\,\Delta_{max} / (3L^2 - 4a^2)$$

Where **L** is the total flexural span, **a** is the shear span, and $(\varepsilon_t - \varepsilon_c)$ is the numerical difference of cross sectional, outermost, tensile and compressive strains for a given moment, **M**. **EI** is the flexural stiffness of the cross section and can be determined from loading geometry, experimental values of strain, $(\varepsilon_t - \varepsilon_c)$, and the diametric distance between the strain points (diameter or depth), **d**. The moment-curvature relationship for a structural member in the linear range is independent of loading and boundary conditions. Thus, the experimental values for structural stiffness, **EI**, can be used to determine load-deflection and elastic energy absorption response of other loading and boundary conditions.

TEST RESULTS

Load-Deflection and moment-curvature plots of all specimens are in the Appendix. The plotted deflections are those at the center span point. Plotted curvatures are taken at midspan and other strain gage locations. Table 2 contains flexural stiffness, EI, (averages of all strain gages) as well as terminal loads strains, and deflections. Conditions for terminating the tests, where applicable, are also listed in Table 2. The most stiff sections were the reinforced recycled plastic configurations and concrete-filled composite shells. The stiffness of timber, prestressed concrete, and steel fender pile sections are also listed in Table 2 for comparison.

Table 2. Flexural Load Test Results.

(1.0 lb-in^2 = 0.235 x 10^6 N-m^2, 1 inch = 2.54 cm, 1 kip = 4.445 kN)

Specimen	Test No.	Measured EI x 10^9 lb-in^2	Test Termination Strain x 10^{-6}	Deflect inches	Load[1] kips	Failure or Reason For Test Termination	Residual[2] Deflection inches
AF	1	0.88	19,000	33	23	Rebar debond	7
	2	0.95	17,000	35	23	Rebar rupture [3]	36 [4]
BF	1	2.55	18,000	27	22	Rebar rupture	36 [4]
	2	2.45	19,000	36	22	deflection limit [3]	23
CF		0.84	23,000	30	9	Rebar debond	10
DB		2.85	8,000	18	33	Not Applicable	< 1
EF		1.14	20,000	32	11	Pipe buckle	19
GF		1.32	20,000	30	13	not available	16
FB	1	0.30	20,000	17	15	Not Applicable [3]	2
	2	0.32	20,000	19	12	Specimen rupture	36 [4]
	3	0.28	20,000	17	11	Specimen rupture	36 [4]
HB	1	2.65	5,600	12	25	Not Applicable	< 1
	2	2.50	5,000	13	25	Splice rupture [3]	36 [4]
	3	2.38	7,000	13	25	Splice rupture	36 [4]
IB		1.0 - 2.0[5]	1,000	2	2	Not Applicable	0
JB	1	0.73	2,650	6	4	Not Applicable	0
	2	0.70	3,450	8	4.5	Not Applicable [3]	< 1
KF	1	1.04 & 1.38[6]	4,600	11	10	Local buckling [7]	< 1
	2	1.12	7,600	11	9	Local buckling [7]	< 1
	3	0.99 & 1.43[6]	4,800	11	11	Local buckling [7]	< 1
LF		1.7	21,000	32	32	Composite rupture	36[4]
Timber[8]		1.6					
Concrete[9]		4.9					
6-in Pipe[10]		0.84					
8-in Pipe[11]		2.17					

Notes:

1. Total Force applied by hydraulic ram.
2. Permanent deflection remaining after load is removed.
3. Load cycling
4. Exceeded deflection limits after loss of load carrying capacity
5. Variable due to taper
6. Values for nominal section and built-up section
7. Crippled to 10% of original capacity
8. 12-inch (30.5 cm) diameter creasoted timber (Reference 1)
9. 18-Inch (46 cm) square prestressed concrete section (Reference 2)
10. Schedule 40 (0.71 cm wall thickness) steel pipe section
11. Schedule 40 (0.82 cm wall thickness) steel pipe section

Figure 8 is moment-curvature plots of three configurations (AF, EF, and FB) with a prestressed concrete fender pile and a 12-inch (30 cm) diameter timber pile. The latter two are from tests conducted by the Naval Civil Engineering Laboratory (References 1 and 2). The timber curve represents five tests conducted on creasoted and water soaked Douglas Fir. A 65-ft (19.8 m) length timber pile has 40 kip-inch (4,500 N-m) energy absorption capacity. The concrete fender pile configuration is an 18- by 18-inch (46- by 46-cm), 8,000 psi (54 MPa) section reinforced with 20, 270 ksi (1800 MPa), ½inch (1.3 cm) strands confined by no. 3 (1 cm), grade 60 ties with 3-inch (8 cm) pitch. It has an ultimate energy capacity of 700 kip-in (79,000 N-m) for 65-ft lengths. Figure 9 is a moment curvature plots of bearing pile configurations for comparing stiffness characteristics. Abbreviated curves for Configurations IB and JB are not to be interpreted as indications of low strength because tests were terminated after the stiffness was defined. An 8-inch (20 cm) diameter, schedule 40 (0.82 cm wall thickness), steel pipe is also included.

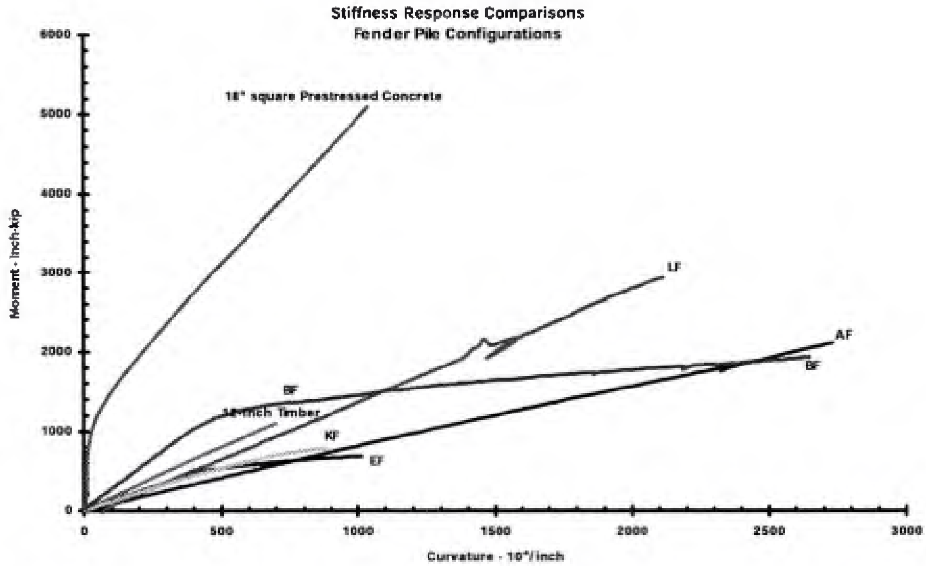


Figure 8. Moment-curvature comparison among fender pile configurations.
(1 inch-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

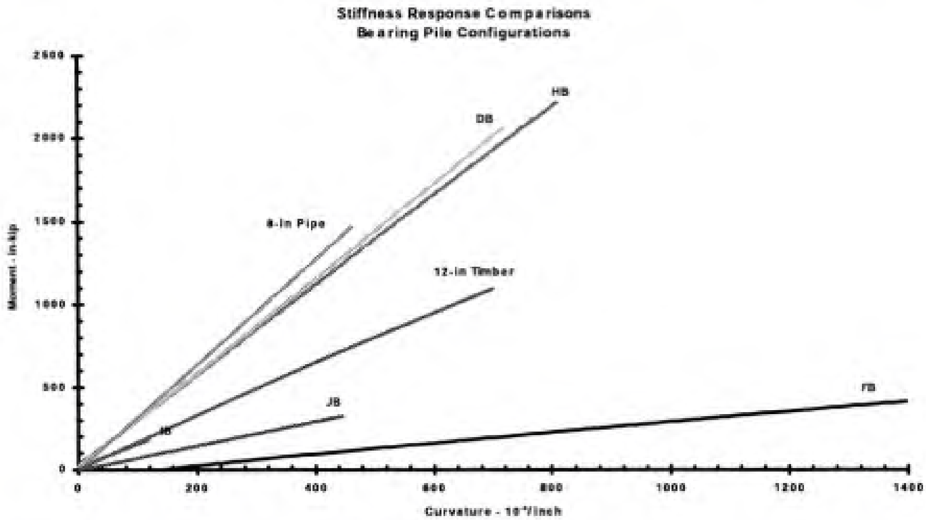


Figure 9. Stiffness comparison among bearing pile configurations.
(1 in-kip = 113 N-m, 10^{-6}/in = 0.4 x 10^{-6}/cm)

Recycled Plastic

Properties of recycled plastic varies from manufacturer to manufacturer. There are no military specifications for material properties. Material properties are difficult to control for recycled plastics. Manufacturers are devoting a lot of research and effort to the task of maintaining uniformity in their products. Those with more experience seem to have tighter control on consistency and quality.

We observed that the position of reinforcing rods in the cross section varied along the length. The rebar was exposed in Configuration GF. This produced some variation in bending stiffness, EI, along the length.

We observed transverse cracking through the cross section of many recycled polyethylene pile specimens (all except configurations AF and DB). These cracks occurred as the piles lay in waiting for testing and were not present when they arrived at the laboratory. They were not subjected to, and were not a result of, mechanical loading, chemical attack or temperature extremes. Over a period of 6 months they continued to widen. At least one crack grew in excess of 1 inch wide. Manufacturers are sensitive to the occurrence of these cracks, offer warranties on the materials, and will replace the piles prior to installation if cracks are discovered. Manufacturers also continue to research the cause of plastic cracking as they improve their quality control procedures. The likelihood of occurrence of these cracks and the impact on pile performance needs further investigation by the Navy.

Unlike concrete in reinforced concrete sections, recycled plastic is not the primary contributor to the structural stiffness of the reinforced plastic sections. For example, 6 inch (15 cm) diameter, Schedule 40 (0.71 cm wall thickness) steel pile has an EI of 0.84 x 10^9 lb-in^2 (2.4 x 10^6 N-m^2) compared to 1.14 x 10^9 lb-in^2 (3.27 x 10^6 N-m^2) for Configuration EF. The reinforcing alone in Configuration BF (EI = 2.5 x 10^9 lb-in^2) (7.2 x 10^6 N-m^2) has a stiffness of 2.05 x 10^9 lb-in^2 (5.9 x 10^6 N-m^2). On the other hand, a 13-inch (33 cm) diameter section of plain, 5,000 psi (34 MPa)concrete has a stiffness of 5.6 x 10^9 lb-in^2 (16.1 x 10^6 N-m^2) and lightweight, 5,000 psi (34 MPa) concrete has a stiffness of 3.2 x 10^9 lb-in^2 (9.2 x 10^6 N-m^2) (57 percent of regular concrete).

The sections reinforced with steel rebar and those with fiberglass rods exhibited very different responses. The fiberglass sections (Configurations AF and DB) exhibited a linear behavior until the tests were terminated while the steel reinforcement of configuration BF yielded at approximately 15.5 kips (69 kN) applied lateral load.

Similar to concrete members, polyethylene plastic piling displays unrecoverable deformation after the load is removed. This occurred to some degree at all load levels, to all the recycled polyethylene plastic configurations, and was not recovered 24 hours after load removal. Determining the exact cause of this characteristic was not part of this program but we suspect that the polyethylene was permanently deformed while the steel or fiberglass reinforcement was slipping through the plastic. Reinforcing steel with lugs for anchorage experienced less residual deflection when the steel strain did not exceed yield strain. After 500 cycles at loads between 6 and 15 kips (27 and 67 kN), Configuration BF experienced a 2-1/2 inch (6.4 cm) permanent set while 200 cycles at 10 kips (45 kN) produced a 2-inch (5 cm) set with Configuration AF. The permanent set which incurred below 15 kips (67 kN) did not significantly effect the ultimate capacity or the stiffness of either configuration. Permanent set was more profound after the steel yielded.

There is significant slippage of the reinforcement rods in polyethylene as the maximum bending strain nears 1 percent. For example, the fiberglass rebar was slipping through the plastic of Configuration CF at a tensile strain near 0.006. As further load-deflection is applied the plastic of the compression zone near the load point wrinkled and buckled, then split longitudinally and separated from the rebar. This worsened permanent set.

Composite shells

Out of roundness and composite thickness variation has a significant effect on the buckling characteristics of composite shells and contributes to stiffness variation. For example, the shell thickness of Configuration HB was approximately ¾inch +/- 1/8 inch (1.9 cm +/- 0.3 cm) and the diameter of Configuration KF was ½inch (1.2 cm) out of round.

The concrete-filled fiberglass composite shells Configuration FB and LF responded nonlinearly. Cyclic loading exhibited hysterisis and permanent set. Performance was dominated by the concrete. The hollow shell Configuration KF had a stiffness of about 1 x 10^9 lb-inch^2 (2.9 x 10^6 N-m^2) while filling the shell with concrete (Configuration LF) had an increase of 70 percent in stiffness, changed the failure mode from buckling to fiberglass tension rupture, and increased the strength almost three-fold.

The modulus of 5,000 psi (34 MPa) lightweight concrete is approximately 2.2 x 10^6 psi (15 x 10^3 MPa) and EI for an uncracked, 9.5-inch (24 cm) diameter cylinder is about 0.88 x 10^9 lb-in^2 (2.5 x 10^6 N-m^2). This compares with the measured value of 0.30 x 10^9 lb-in^2 (0.86 x 10^6 N-m^2) for the composite section. Analysis of strain data indicates that the neutral axis of the section was positioned about 3 inches (8 cm) from the outermost compression fiber. This means that the concrete was cracked more than 7 inches (18 cm) across the section after applying a moment of 150 inch-kips (17,000 N-m). A similar cracked section occurred with Configuration LF but normal weight concrete and a larger diameter provided significantly more stiffness. To lower stiffness further, the concrete shrank away from the composite shell during the concrete curing period (Figure 10) and was not acting fully integral with the composite.

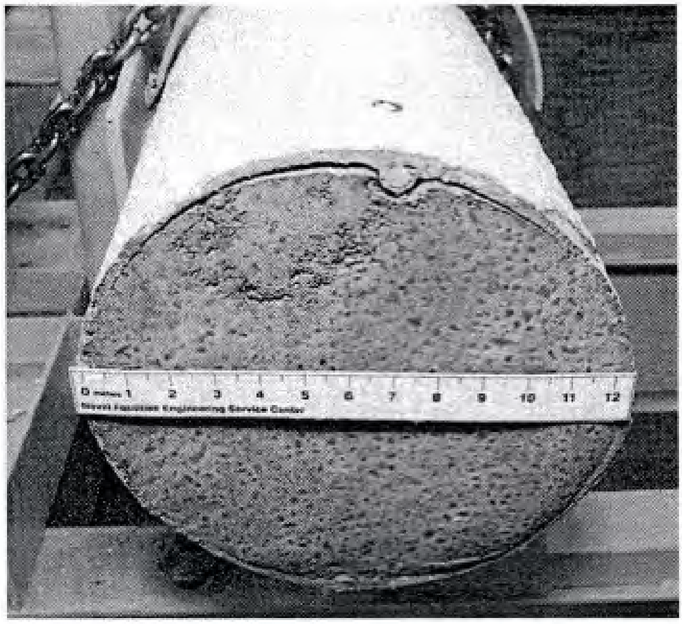


Figure 10. Shrinkage of curing concrete causes separation from external composite shell.

FAILURES

The objective of this study did not include failure analysis. However, failures occurred in some fender pile specimens prior to attaining deflection limits or in some bearing pile specimens while attempting to identify nonlinear characteristics.

Composite Shells

The splices employed on the hollow fiberglass composite piles were considerably weaker in bending than the continuous configuration. Failure occurred with the cylindrical section separating from the splice material (Figure 11).

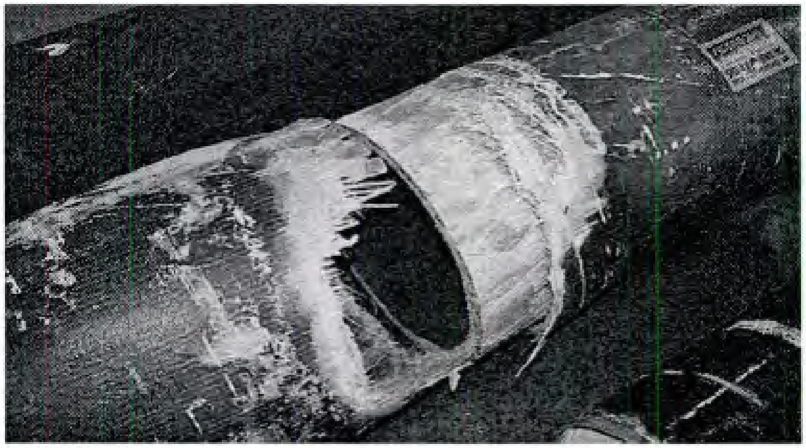


Figure 11. Splice failure of Configuration FB.

Thin shelled cylindrical hollow composites failed by buckling in the compression zone (Figure 12). The concrete-filled, epoxy composite of fiberglass Configuration FB failed by fracturing into two pieces while concrete-filled vinylester SCRIMP Configuration LF failed by rupturing the fiberglass. The breaks are shown in Figures 13 and 14, respectively.

The shell employed in Configuration FB was a pipe design to resist internal pressure where longitudinal strength requirements are only half that necessary to resist circumferential forces. In axial/flexural members longitudinal reinforcement resists bending while the circumferential strength offsets the applied shear and confines the concrete in compression. Concrete confinement enhances axial and bending resistance by increasing its compression capacity. However, in the axial and bending ranges of a Naval pile, confinement strength should not be twice the longitudinal reinforcement strength. Consequently, configuration FB's rupture of composite shell and fracture of the concrete in bending occurred long before the confinement of the concrete could contribute to the flexural strength. A more efficient design, Configuration LF, had a unidirectional ply of E-glass longitudinally and 1 ply each at 45° to the longitudinal. While this did not deform as much as Configuration FB, it carried higher loads and was much stiffer.

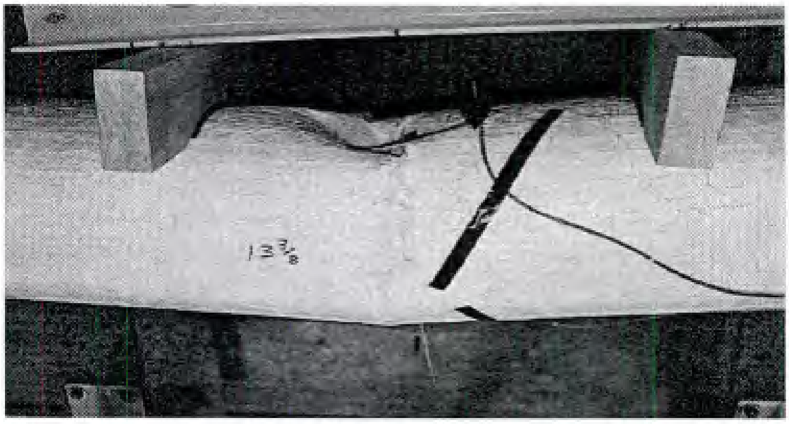


Figure 12. Localized shell buckling of Configuration KF.

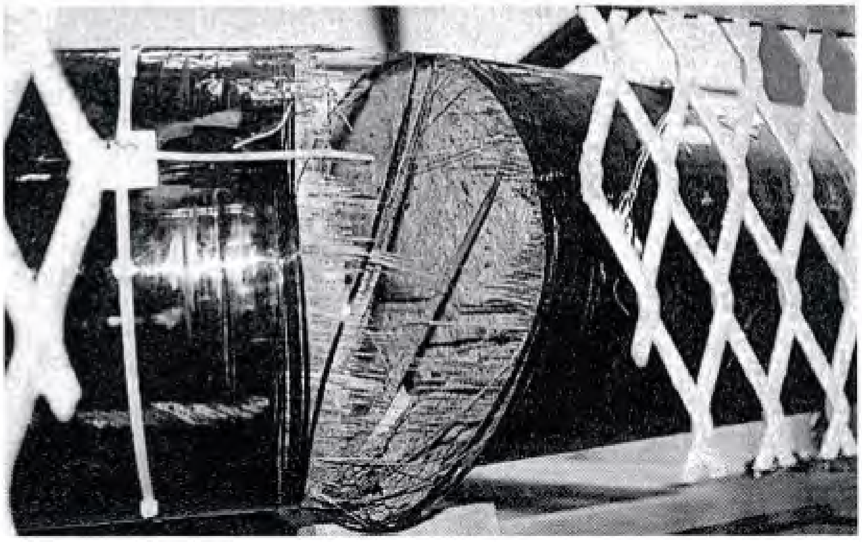


Figure 13. Failure of configuration FB.

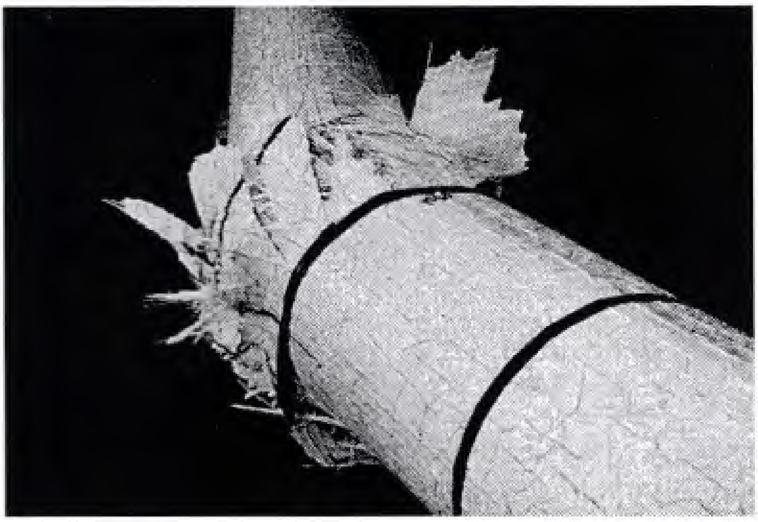


Figure 14. Bending failure of Configuration LF.

Recycled Plastic

Steel reinforcement: In the first specimen the steel fractured at a deflection of 18 inches (46 cm) while the second specimen was able to sustain over 33 inches (84 cm) of deflection without fracture. The steel fracture occurred at the weld attaching the spiral steel to the principal reinforcement. The steel rebars failed in sequence which systematically reduced the resistance of the pile and ruptured the polyethylene on the tension face (Figure 15). In the compression zone the plastic wrinkled, split, and buckled away from the reinforcement. All recycled plastic configurations showed significant damage at the load point at deflections above 30 inches (76 cm). This damage included splitting of plastic and separation from the reinforcement.

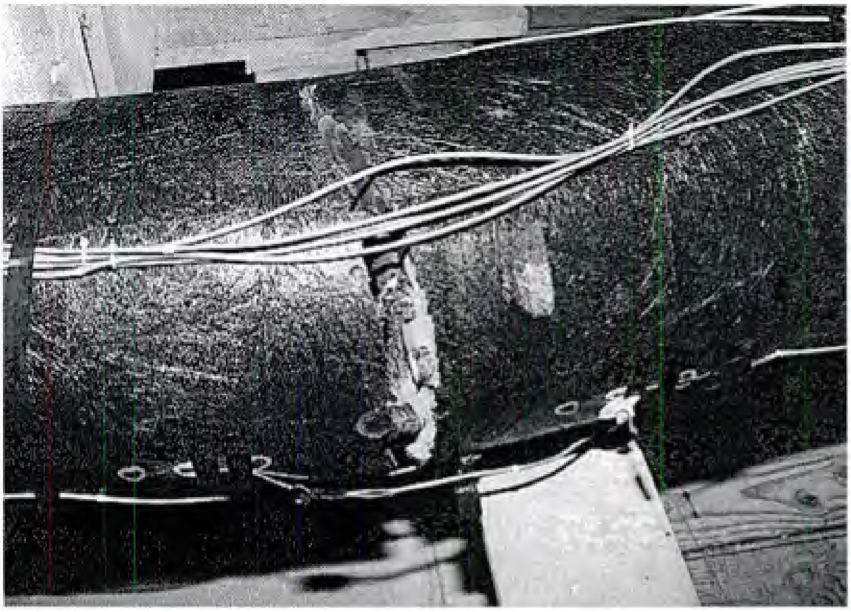


Figure 15. Failure of tension steel in Configuration BF.

CONCLUSIONS

Although the test results are mixed, reinforced recycled plastic and concrete-filled composite shells can be attractive structural alternatives to timber for fender and bearing piles.

Hollow shells are excellent structural elements. They provide an excellent ratio of strength and stiffness to weight. They should also be easy to repair by addition of cylindrical shells to the outer surface of damaged areas. However, shells will buckle suddenly in compression/bending and lose all load carrying capacity. Further, they do not perform as well under concentrated loads required of fender piles. They will also be subject to collapse due to pinching at the load (the camel) and reaction (the deck level) points as these points merge as the tide rises.

The pile concept of placing concrete inside a composite shell is worthy of further investigation by the Navy. Concrete increased bending stiffness by 70 percent in a 13-inch (33 cm) diameter section. Concrete is relatively cheap and superior for resisting compressive forces and will prevent localized buckling and pinching of the composite shell. A properly designed plastic composite shell is external reinforcement that serves as a stay-in-place form which will provide protection for, and enhance confinement performance of, the concrete. Thus, concrete can be placed in the shell at the fabricators or it can be poured at site after the pile is driven. The pile can be easily upgraded and repaired by attaching additional composite shells to the exterior surface. To insure composite action the concrete must adhere to the shell interior. As the concrete cures it will shrink away and separate from external reinforcing (Figure 10) which will degrade it's flexural performance. Adhesion can be enhanced by a roughened or wrinkled surface such as that produced by SCRIMP processing. Unfavorably, if fiberglass fibers are used they must be protected from the alkalinity of curing concrete probably by enriching the plastic matrix at the concrete interface. Further, concrete weighs between 100 and 140 lbs/ft^3 (4.1 and 5.7 kg/m^3 which will make a finished pile weight up to 150 lbs/ft^3 (6.1 kg/m^3). A systematic design effort must be brought to bear to select and balance longitudinal, circumferential, and diagonal fiber strength (number of plies) with the (confined) compressive strength of the concrete to resist required bending and axial forces. A more efficient design using the volume of fibers in Configuration FB would be to proportion the shell with longitudinal strength being thrice that of the circumferential (3 plies longitudinal to 1 ply circumferential).

Quality control and assurance will be important issues in plastic and composite piles as it is in concrete construction. Dimensional variation (thickness, out of roundness, position of rebar, etc.) and material consistency are most significant. Procurement of recycled polyethylene should require quality assurance that cracking would be grounds for pile replacement by the manufacturer. A one year warranty period is not enough time for these cracks to be discovered particularly if they occur below the waterline. The warranty period should be five years.

REFERENCES

1. Civil Engineering Laboratory Technical Note TN-1535, "Mechanical Properties of Preservative Treated Marine Piles: Results of Limited Full Scale Testing," by M.L. Eaton et. al., Nov. 1978. Port Hueneme, CA.
2. Naval Civil Engineering Laboratory Technical Report TR-927, "Laterally Loaded Partially Prestressed Concrete Piles," by G. Warren, Sept 1989. Port Hueneme, CA

APPENDIX - TEST PLOTS

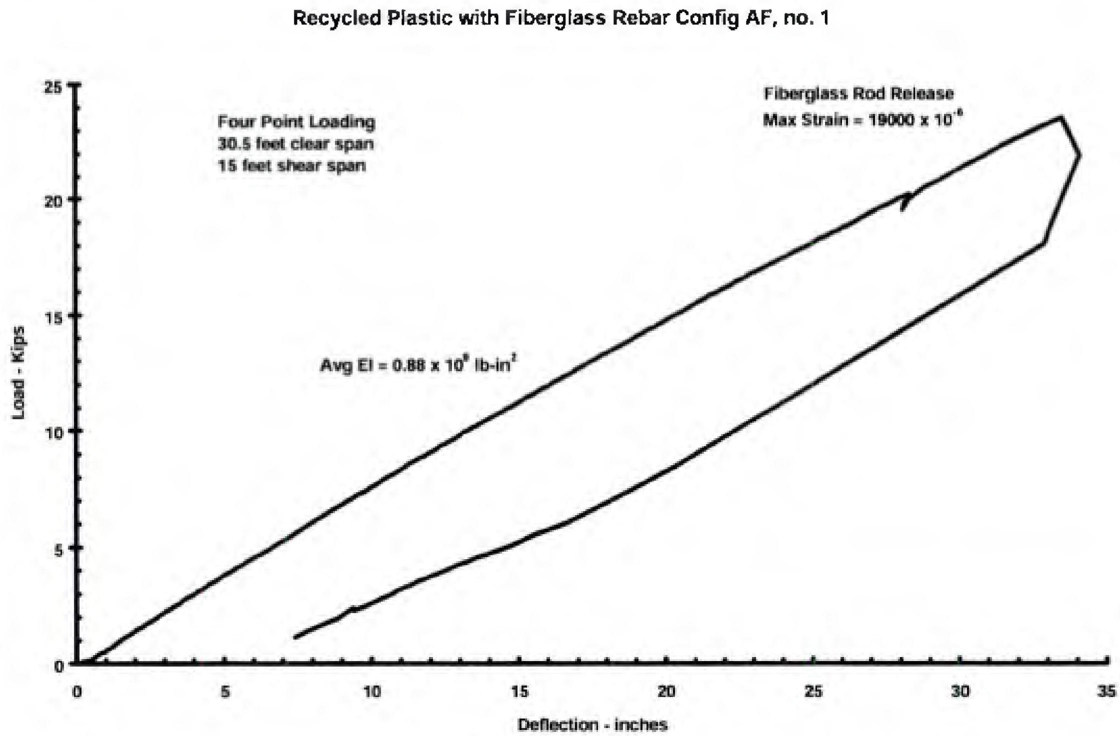


Figure A1. Load-deflection response of Configuration AF, test setup A. (1 kip = 445 N, 1 inch = 2.54 cm)

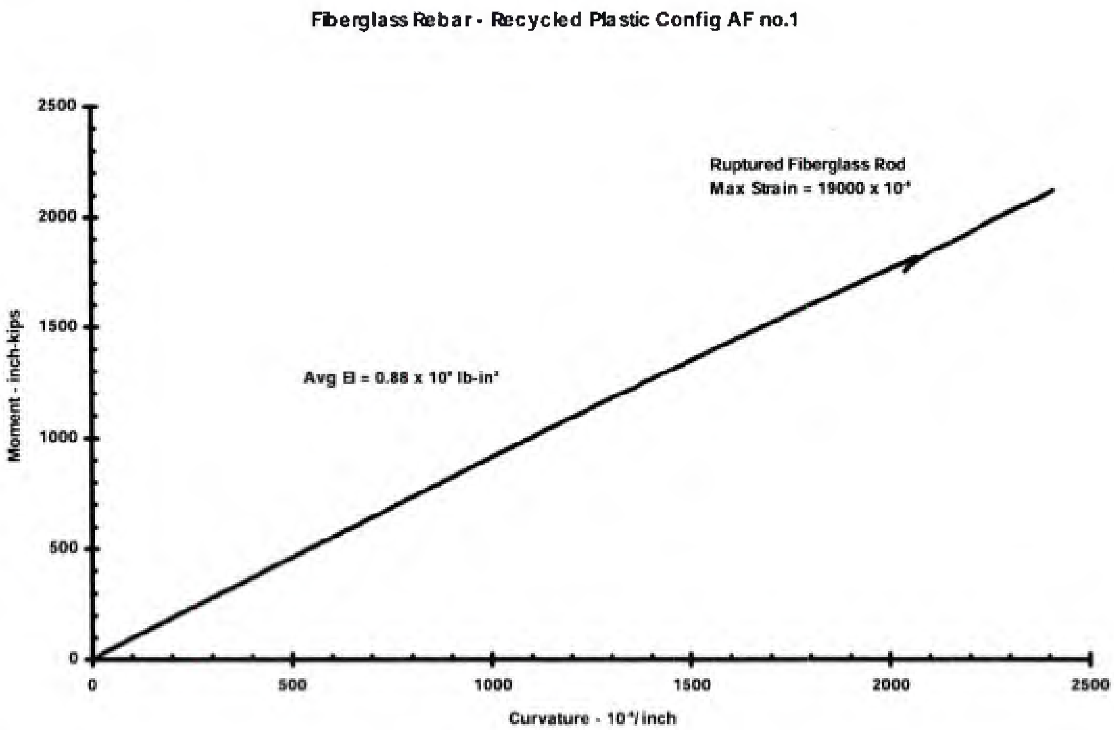


Figure A2. Moment - Curvature response of Configuration AF, specimen number 1. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

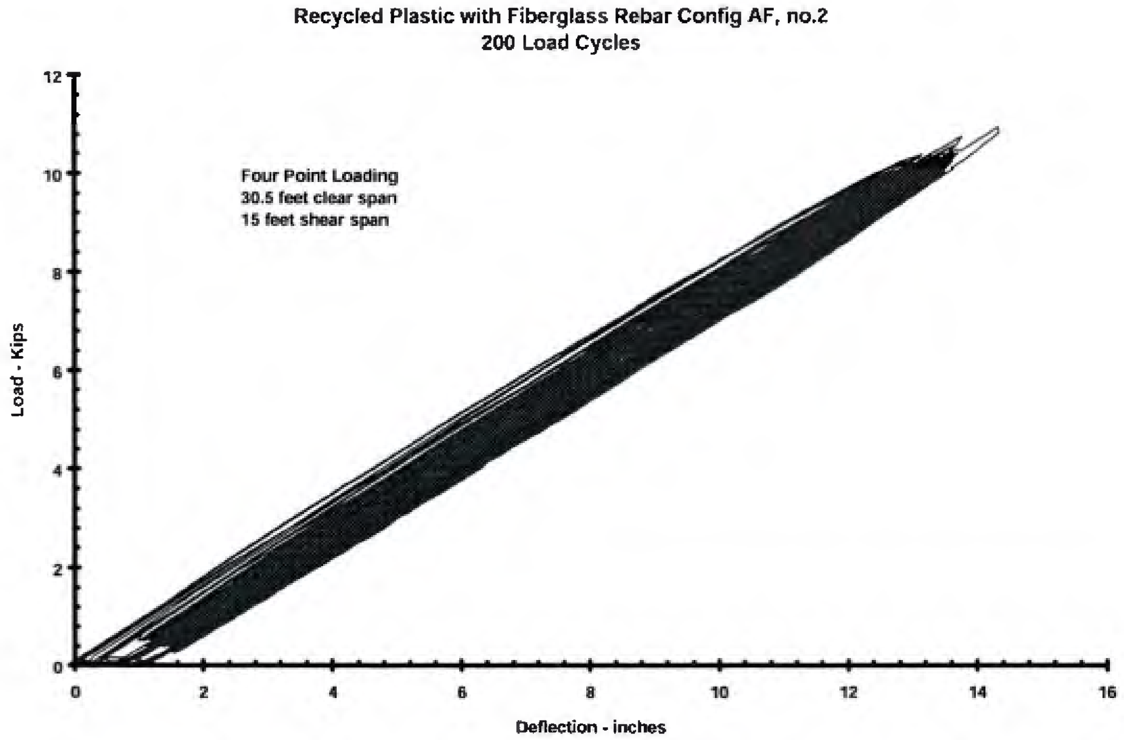


Figure A3. Cyclic load-deflection response of Configuration AF, specimen number 2. (1 in-kip = 445 N, 1 inch = 2.54 cm)

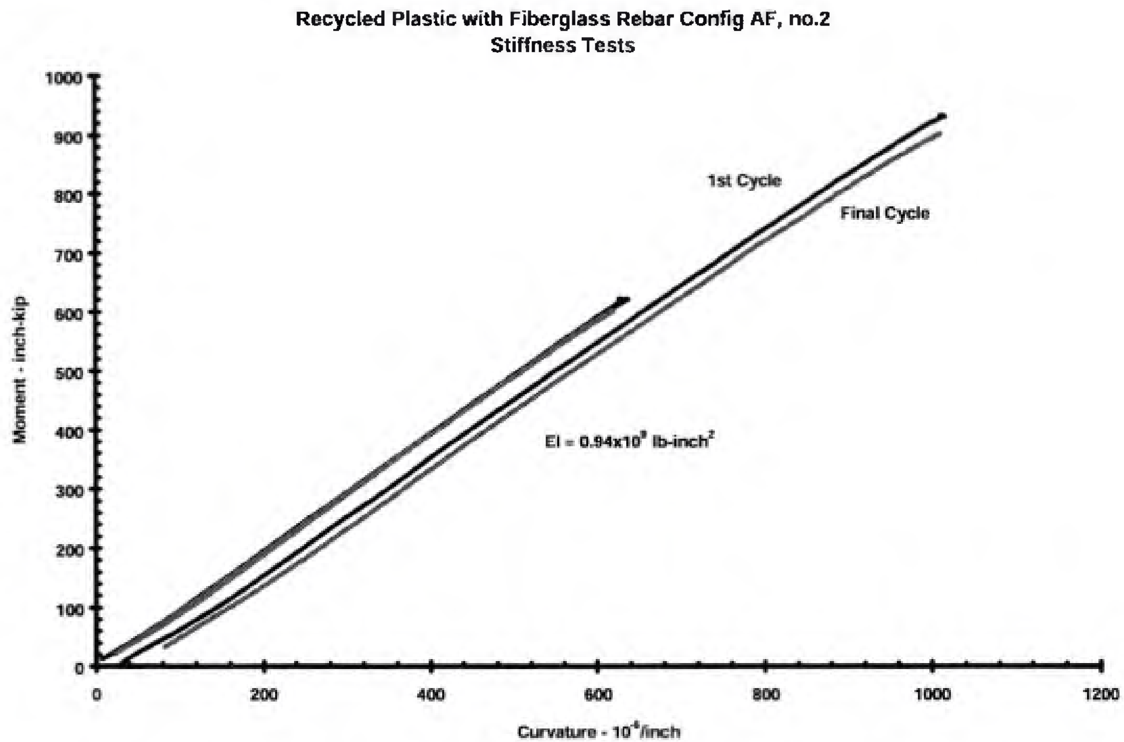


Figure A4. Moment-curvature response of Configuration AF, specimen number 2. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

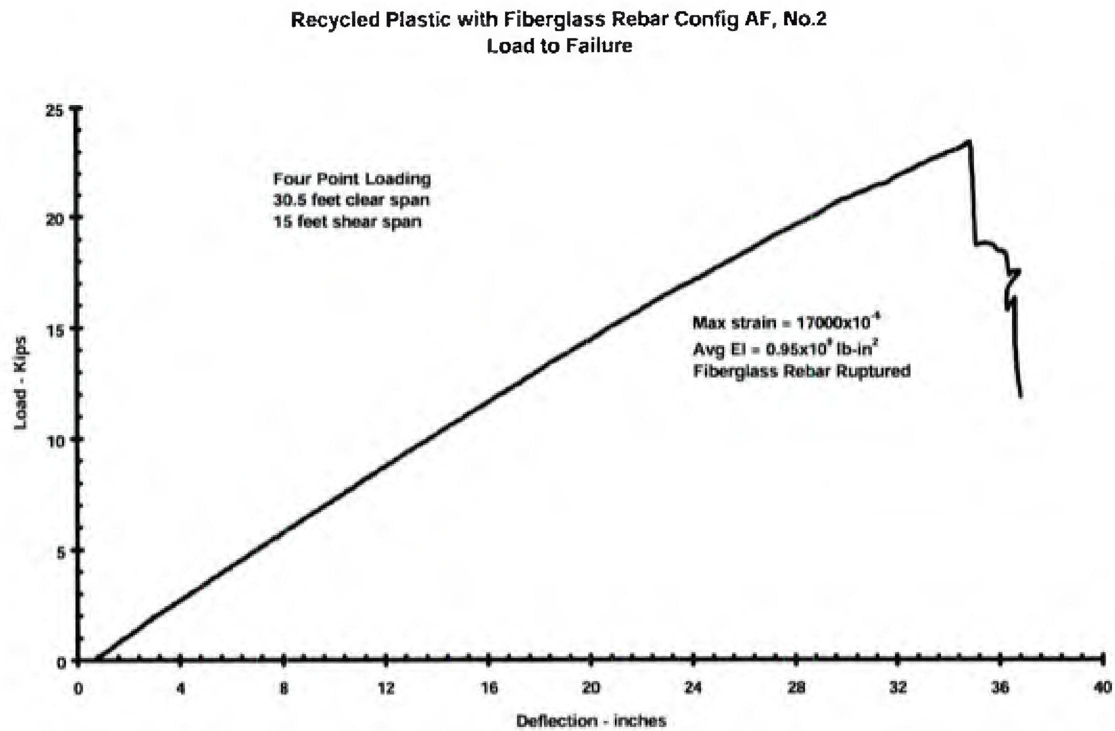


Figure A5. Load-deflection to failure of Configuration AF, specimen number 2. (1 in-kip = 445 N, 1 inch = 2.54 cm)

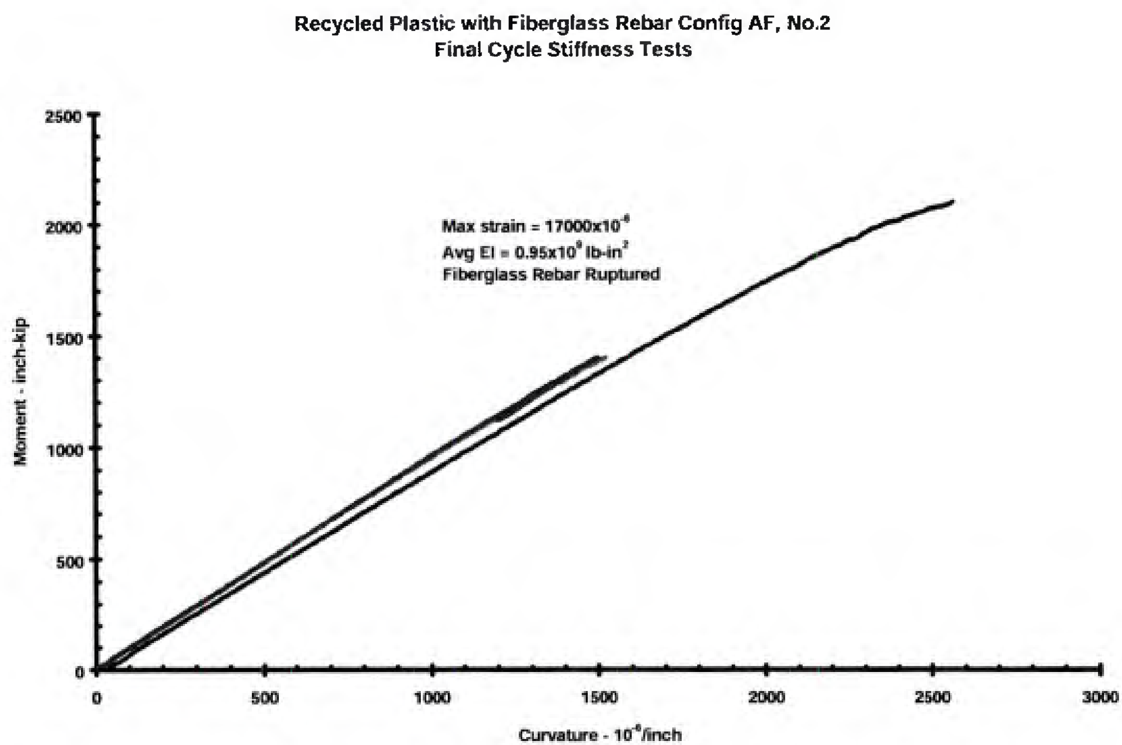


Figure A6. Final cycle moment-curvature response of Configuration AF, specimen number 2. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

Recycled Plastic with Steel Rebar Cage Config BF no.1

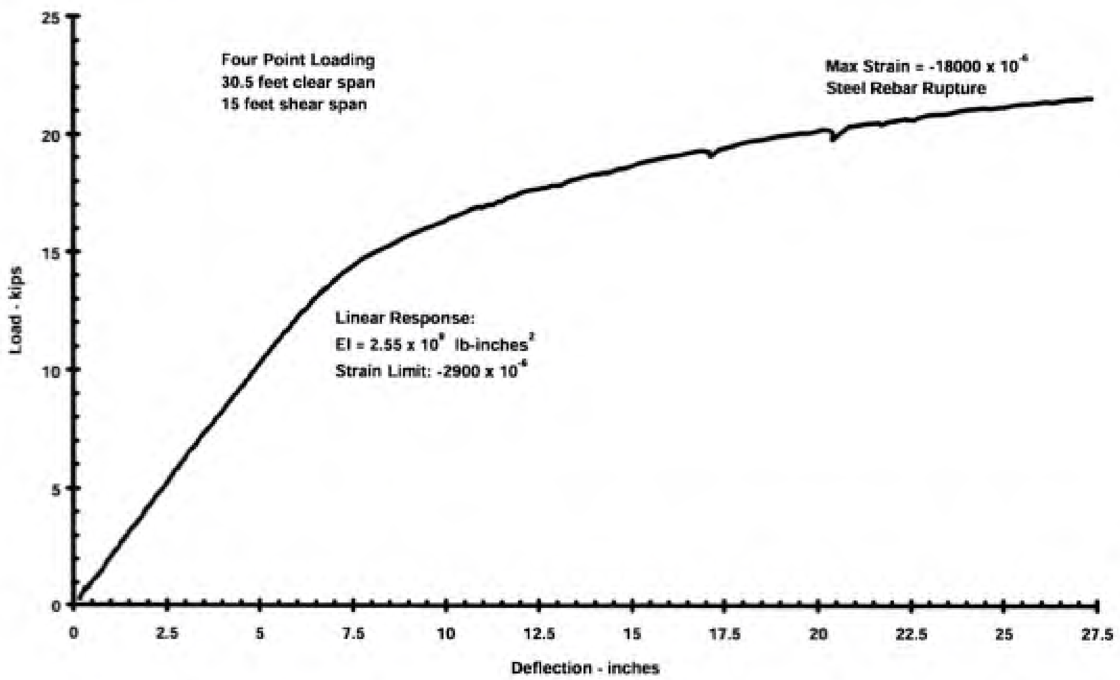


Figure A7. Load-deflection response of Configuration BF, specimen number 1, test setup A. (1 in-kip = 445 N, 1 inch = 2.54 cm)

Recycled Plastic with Steel Rebar Cage Config BF no. 1

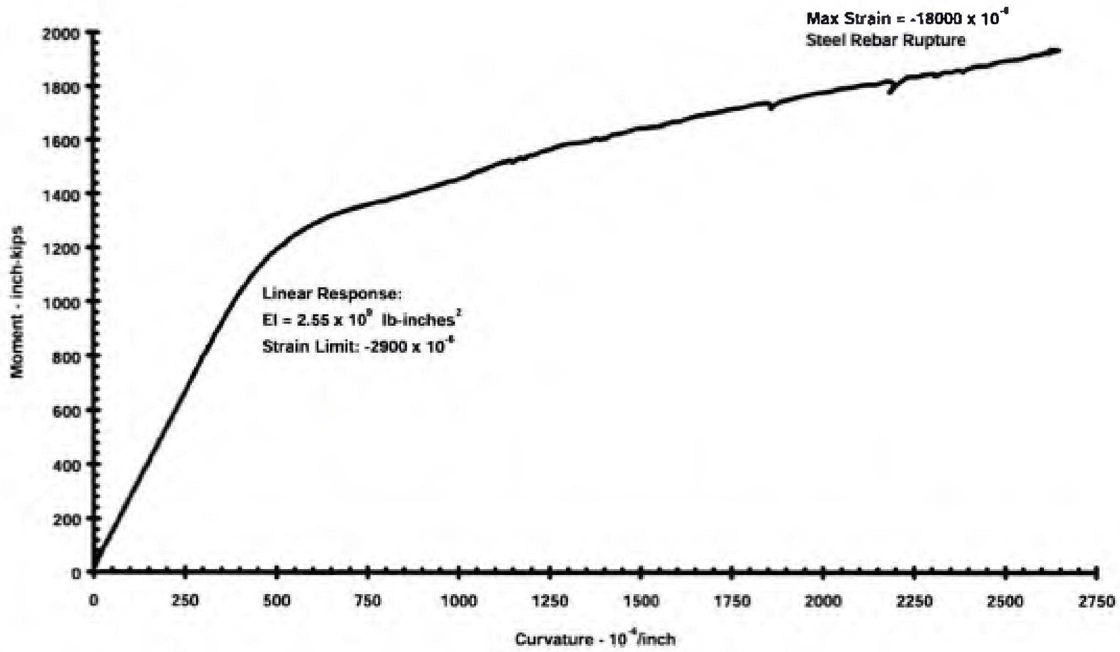


Figure A8. Moment-curvature response of Configuration BF, specimen number 1. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

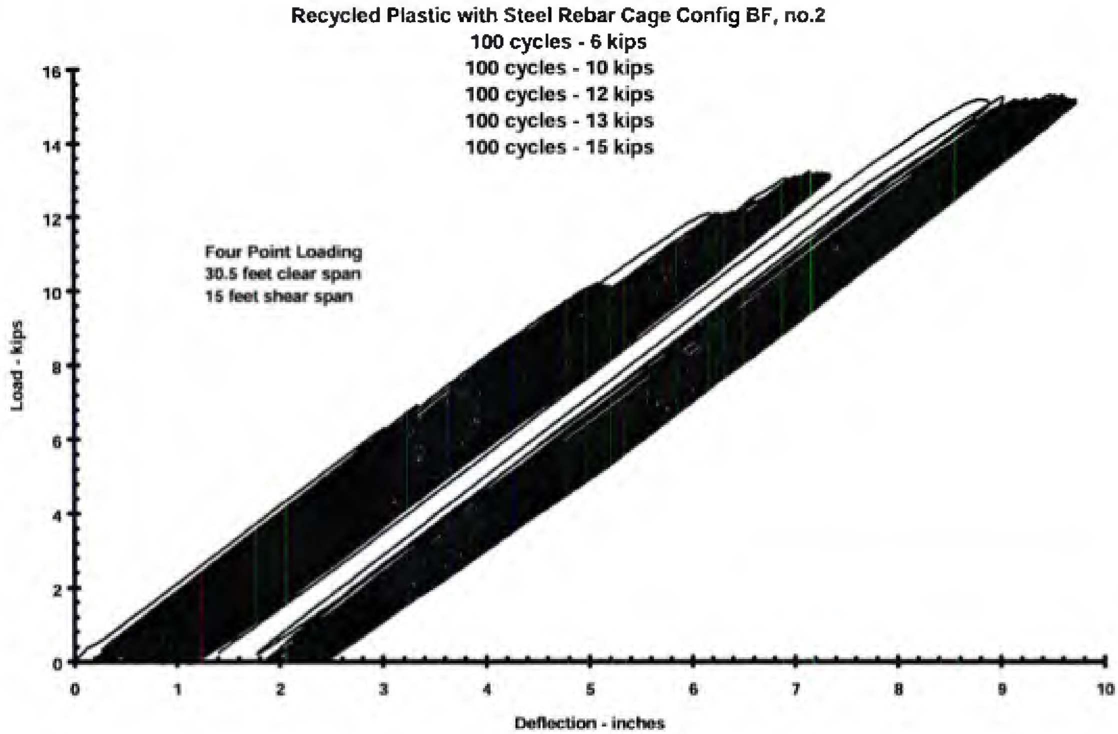


Figure A9. Cyclic load-deflection response of Configuration BF, specimen number 2. (1 in-kip = 445 N, 1 inch = 2.54 cm)

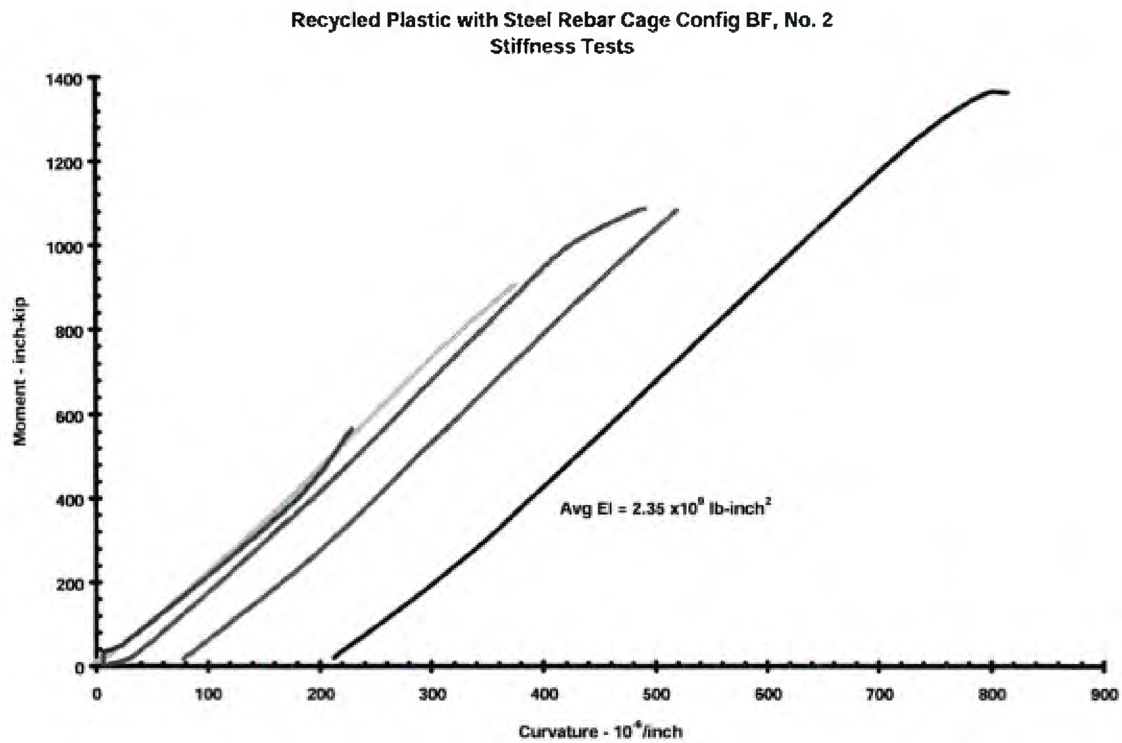


Figure A10. Moment-curvature response of Configuration BF, specimen number 2. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

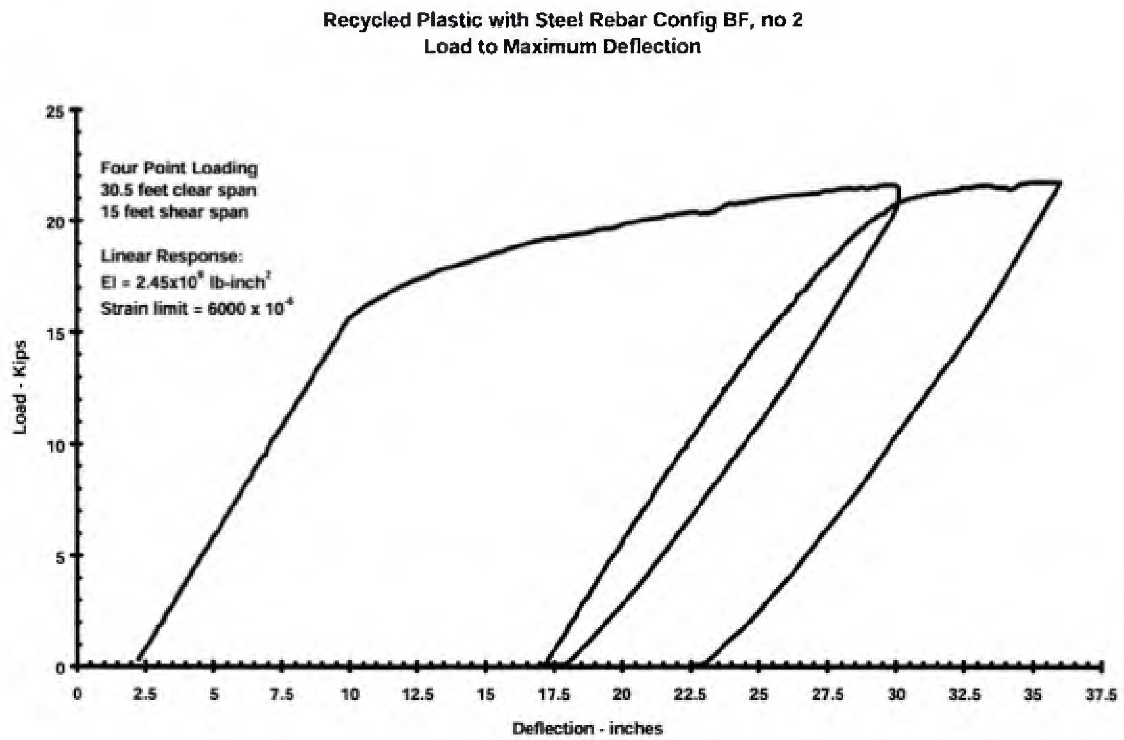


Figure A11. Final cycle load-deflection response of Configuration BF, specimen number 2. (1 in-kip = 445 N, 1 inch = 2.54 cm)

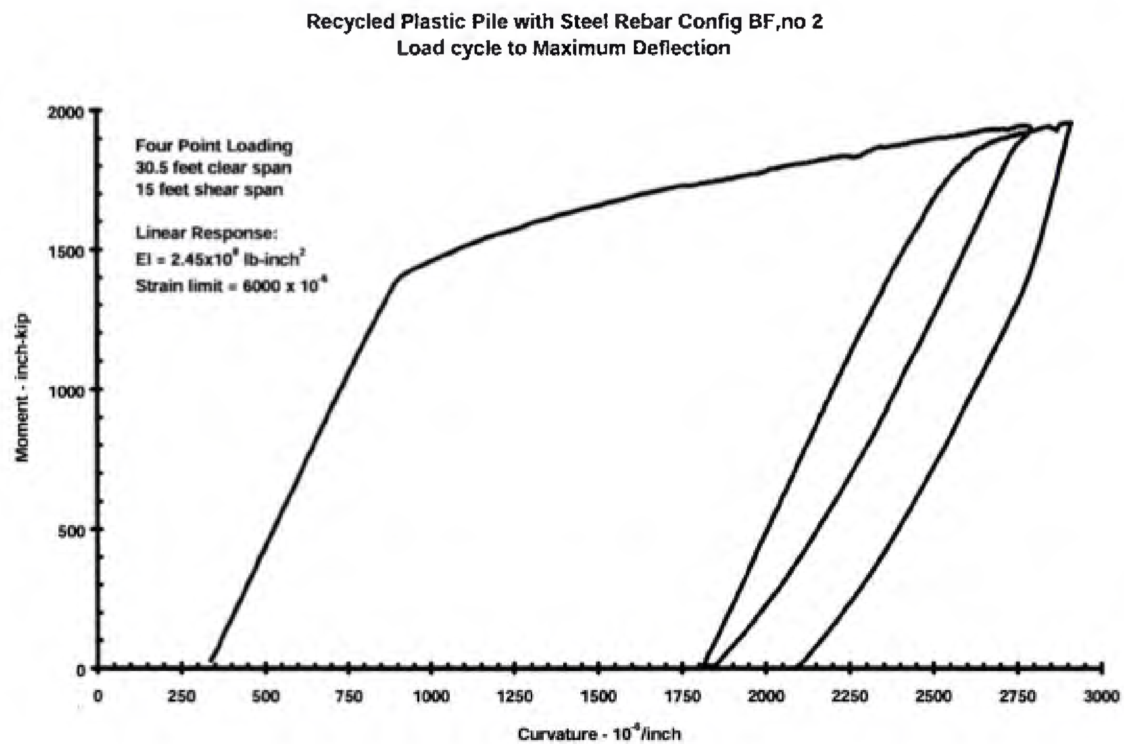


Figure A12. Final cycle moment-curvature response of Configuration BF, specimen number 2. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

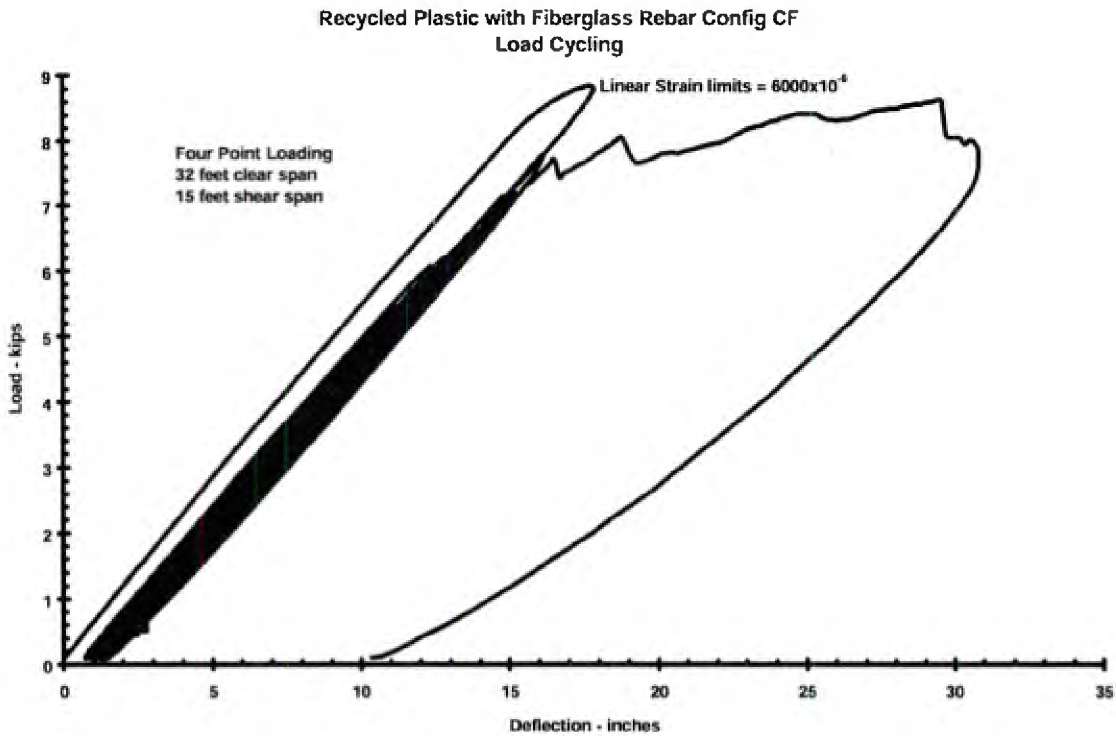


Figure A13. Cyclic load-deflection response of Configuration CF, test setup B. (1 in-kip = 445 N, 1 inch = 2.54 cm)

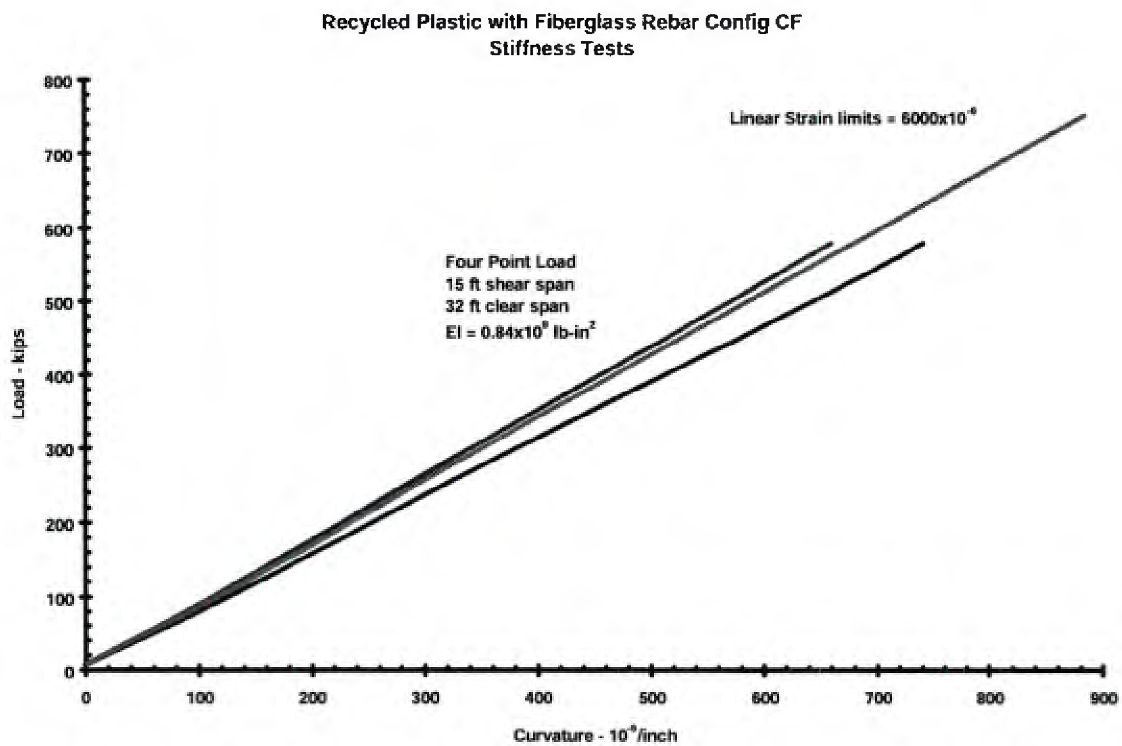


Figure A14. Moment-curvature response of Configuration CF. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

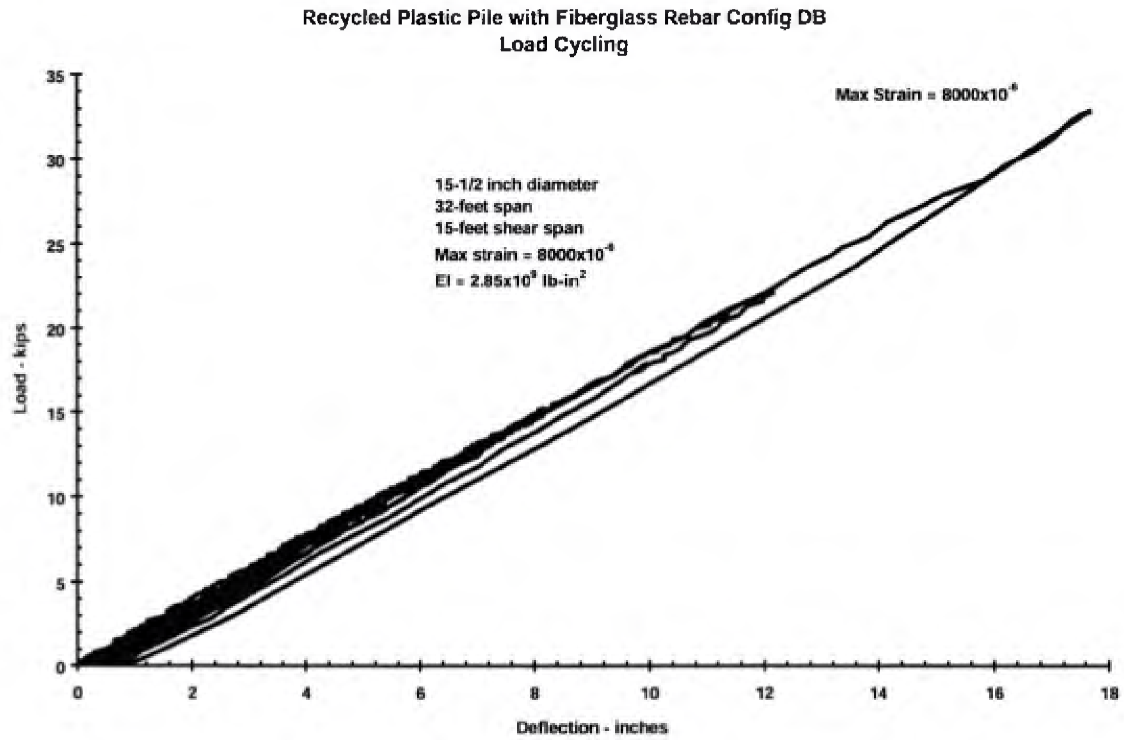


Figure A15. Cyclic load-deflection response of Configuration DB, test setup B. (1 in-kip = 445 N, 1 inch = 2.54 cm)

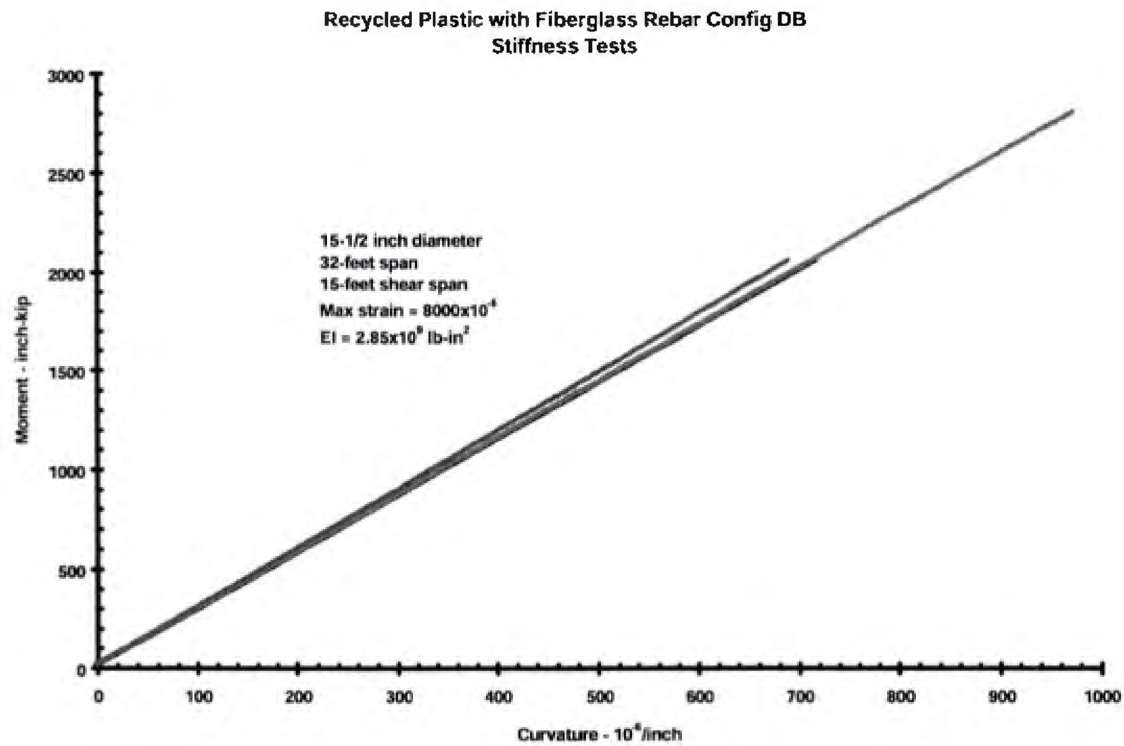


Figure A16. Moment-curvature response of Configuration DB. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

Recycled Plastic with 6-inch Schd 40 Pipe Config EF

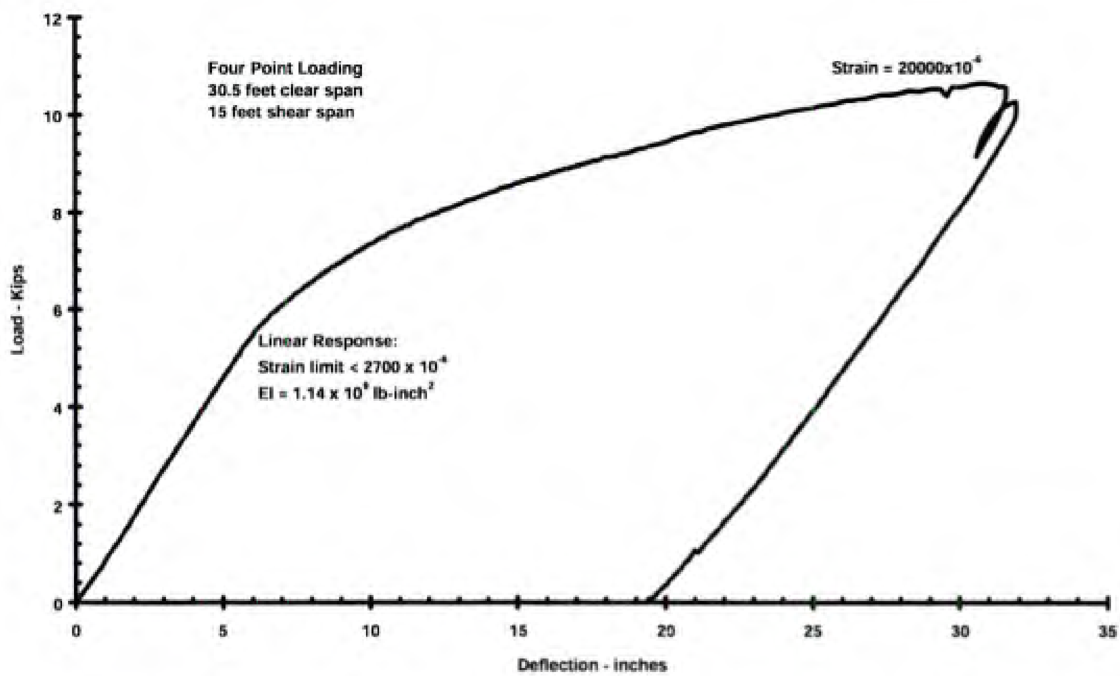


Figure A17. Load-deflection response of Configuration EF, test setup A.
(1 in-kip = 445 N, 1 inch = 2.54 cm)

Recycled Plastic with Sch 40 Pipe Config EF

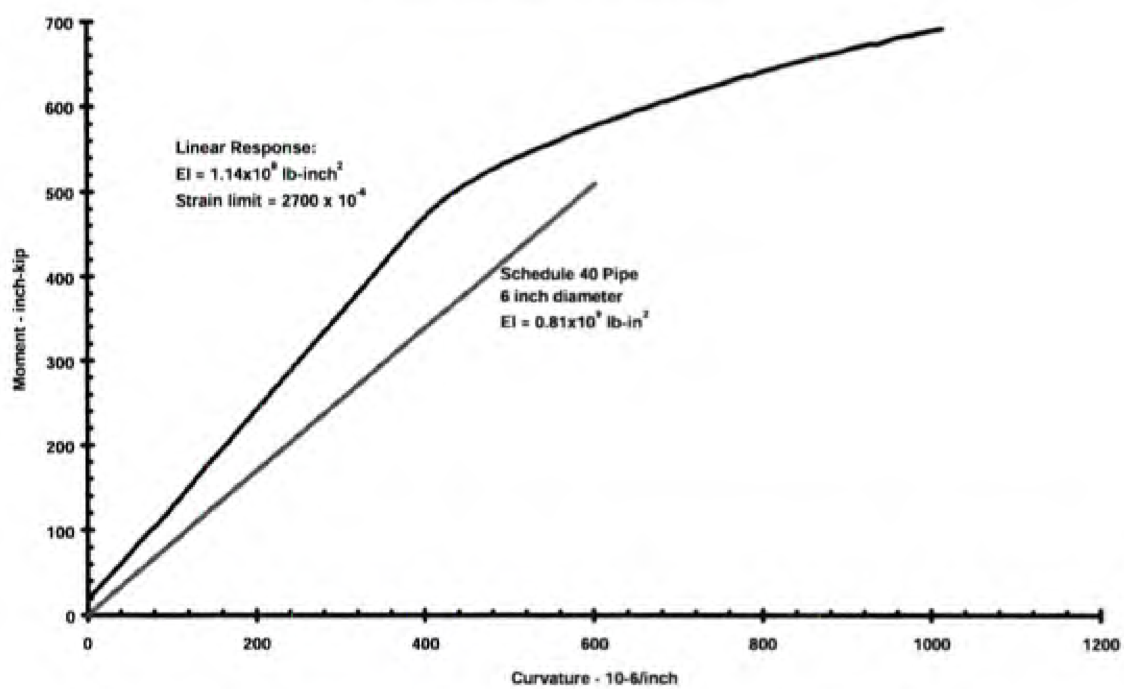


Figure A18. Moment-curvature response of Configuration EF. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

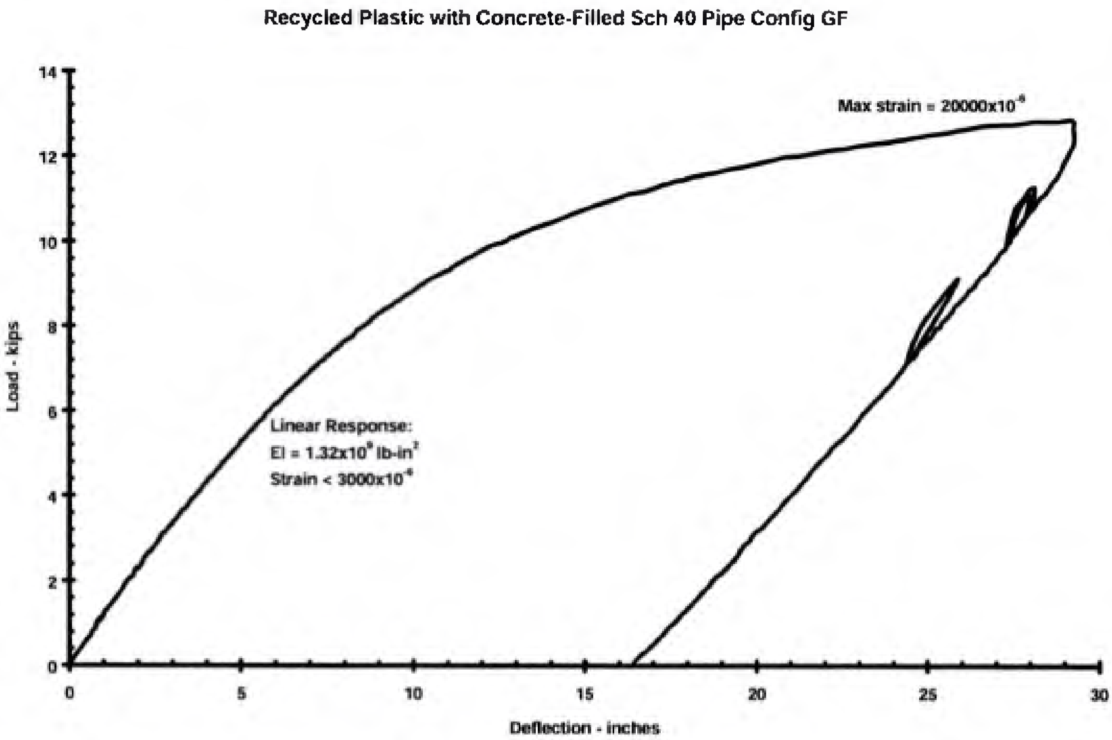


Figure A19. Load-deflection response of Configuration GF, test setup A. (1 in-kip = 445 N, 1 inch = 2.54 cm)

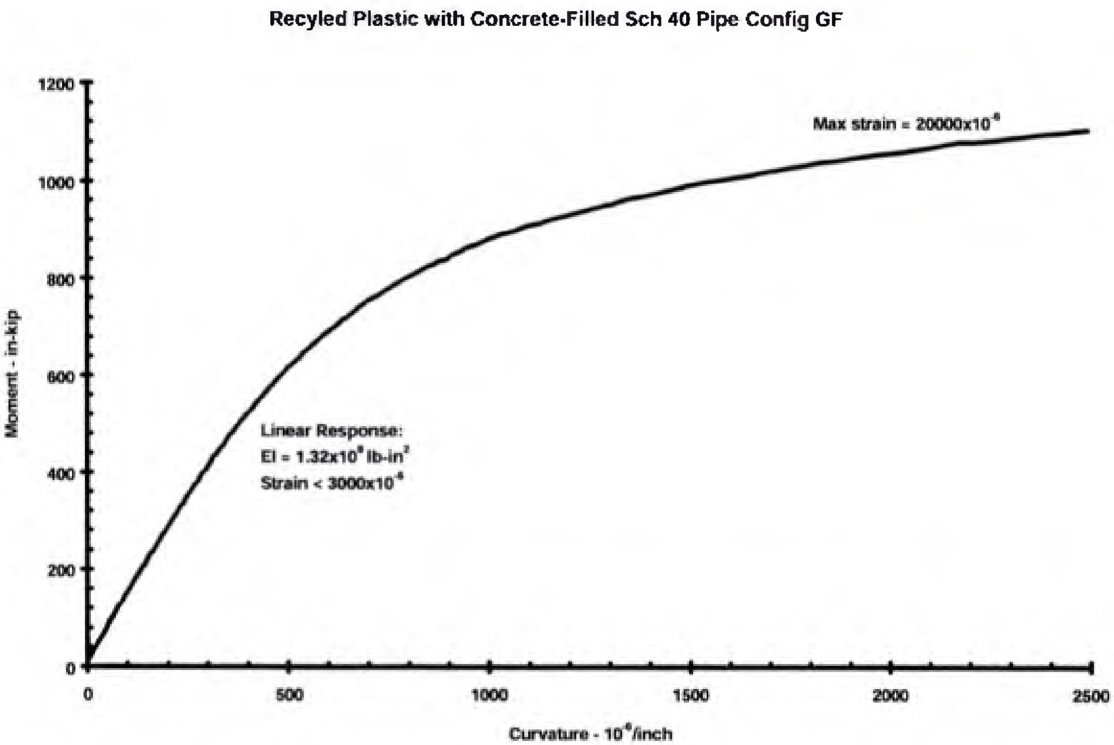


Figure A20. Moment-curvature response of Configuration GF. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

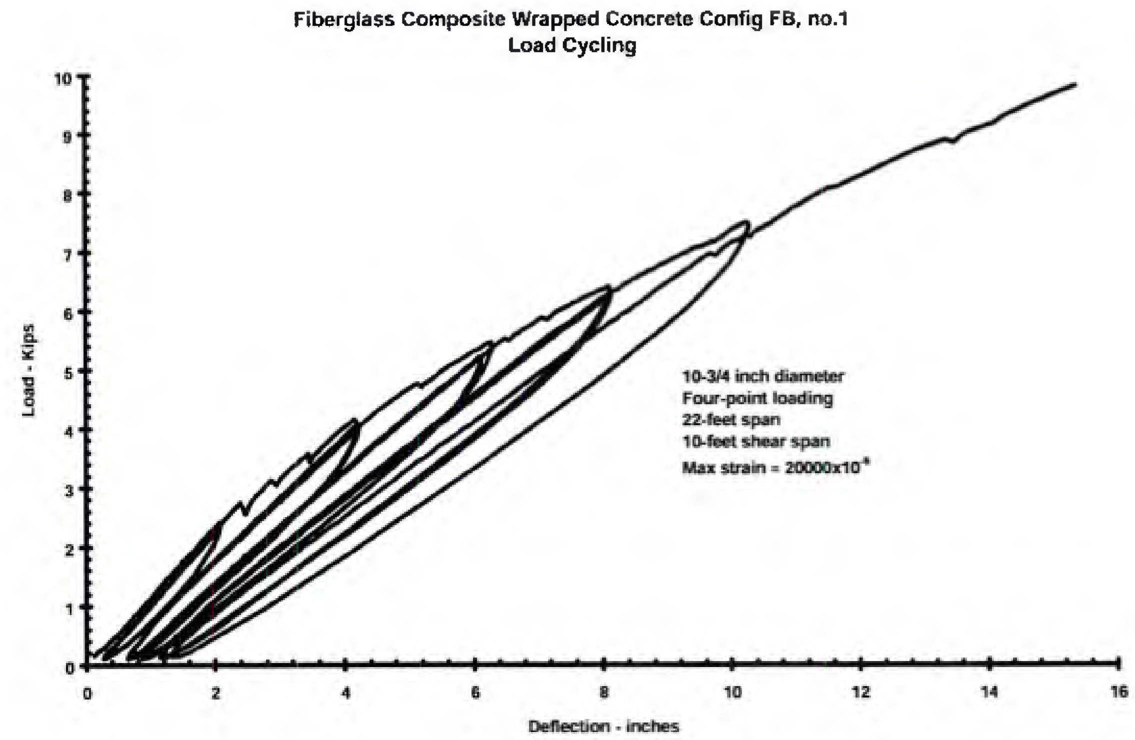


Figure A21. Cyclic load-deflection response of Configuration FB, specimen number 1, test setup C. (1 in-kip = 445 N, 1 inch = 2.54 cm)

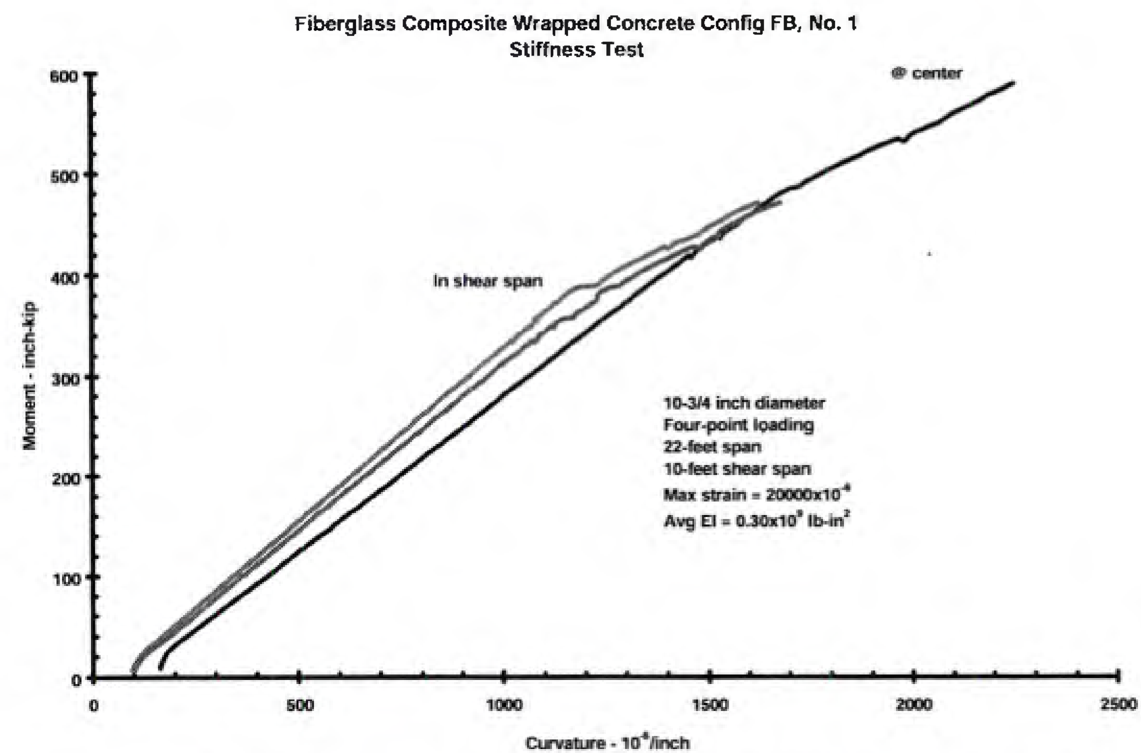


Figure A22. Moment-curvature response of Configuration FB, specimen number 1. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

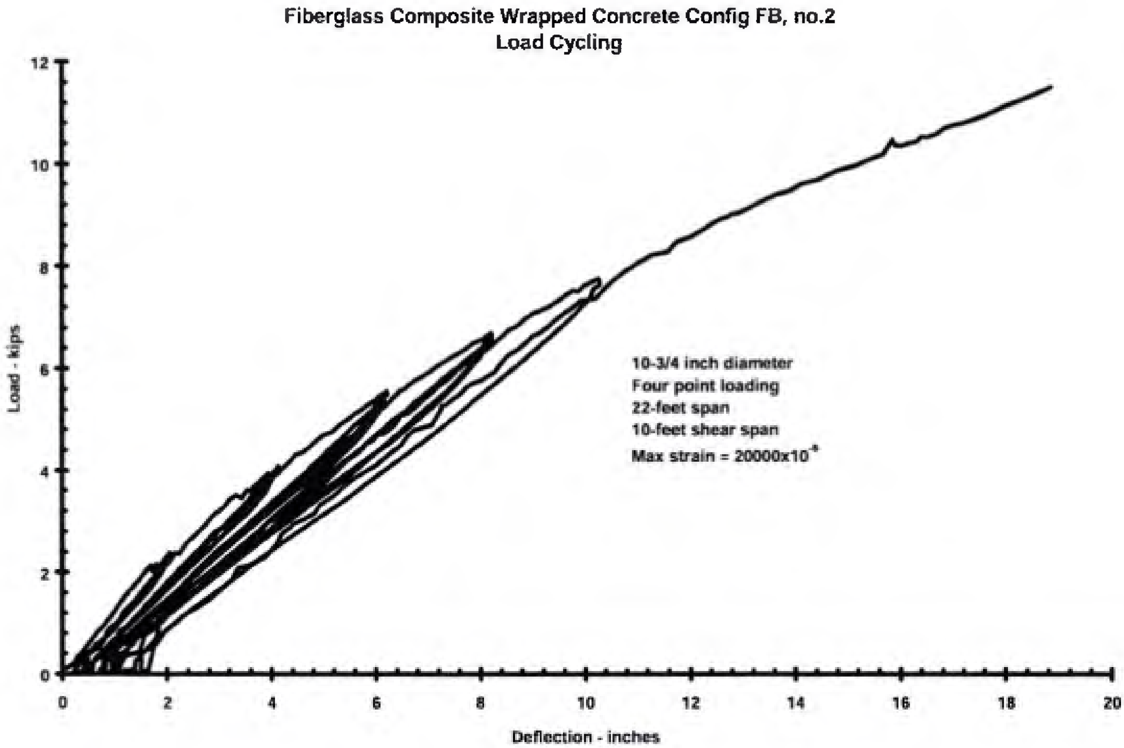


Figure A23. Cyclic load-deflection response of Configuration FB, specimen number 2, test setup C. (1 in-kip = 445 N, 1 inch = 2.54 cm)

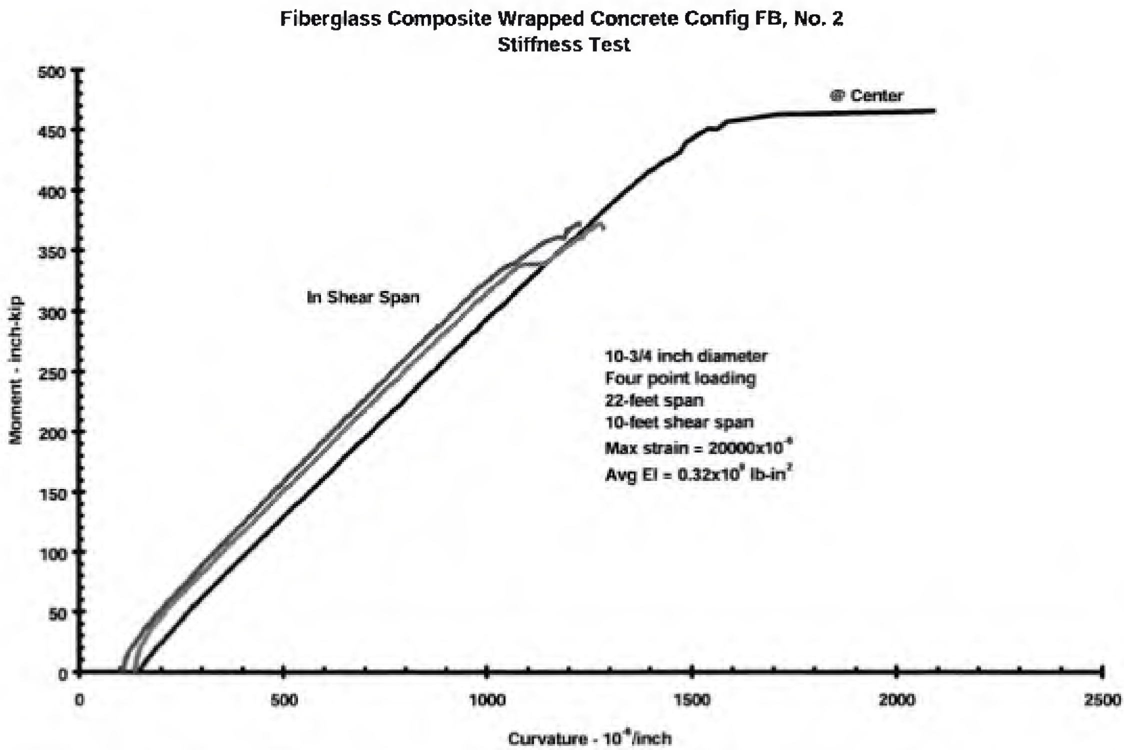


Figure A24. Moment-curvature response of Configuration FB, specimen number 2. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

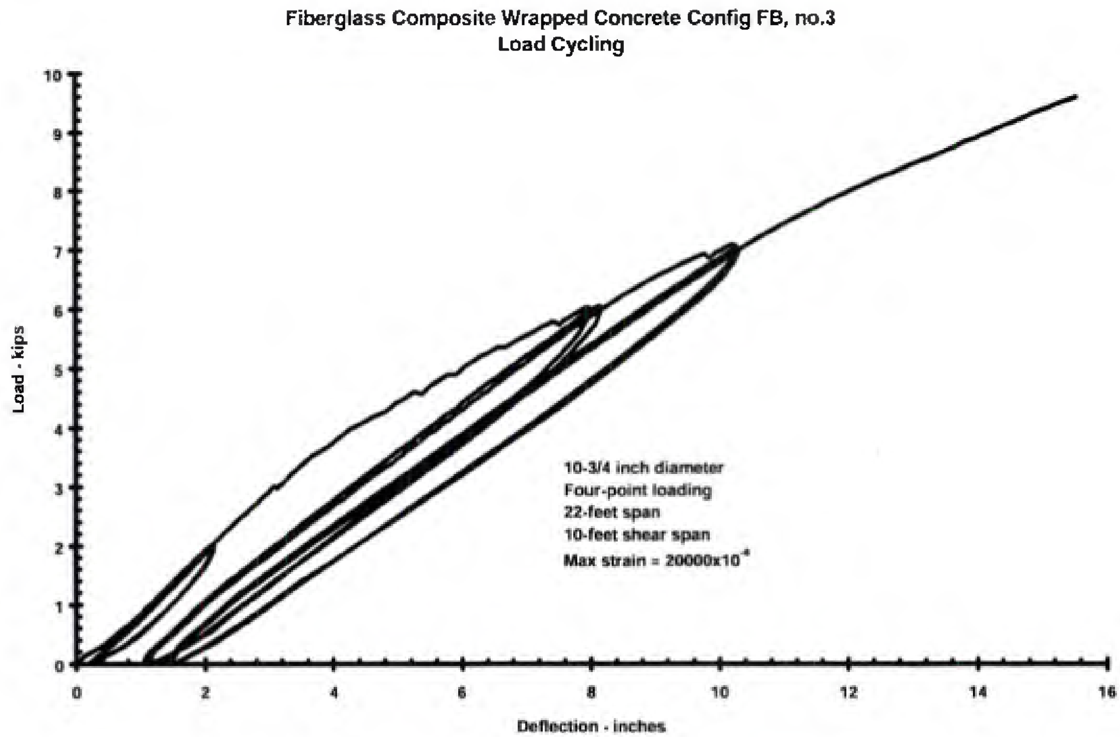


Figure A25. Cyclic load-deflection response of Configuration FB, specimen number 3, test setup C. (1 in-kip = 445 N, 1 inch = 2.54 cm)

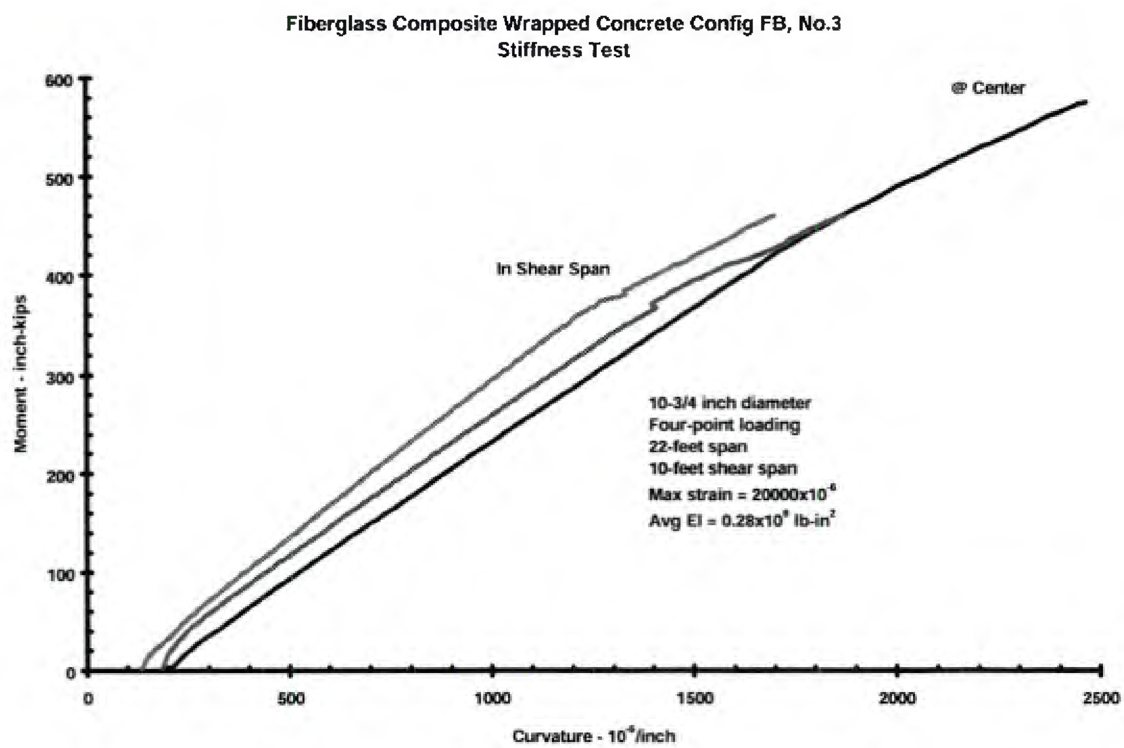


Figure A26. Moment-curvature response of Configuration FB, specimen number 3. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

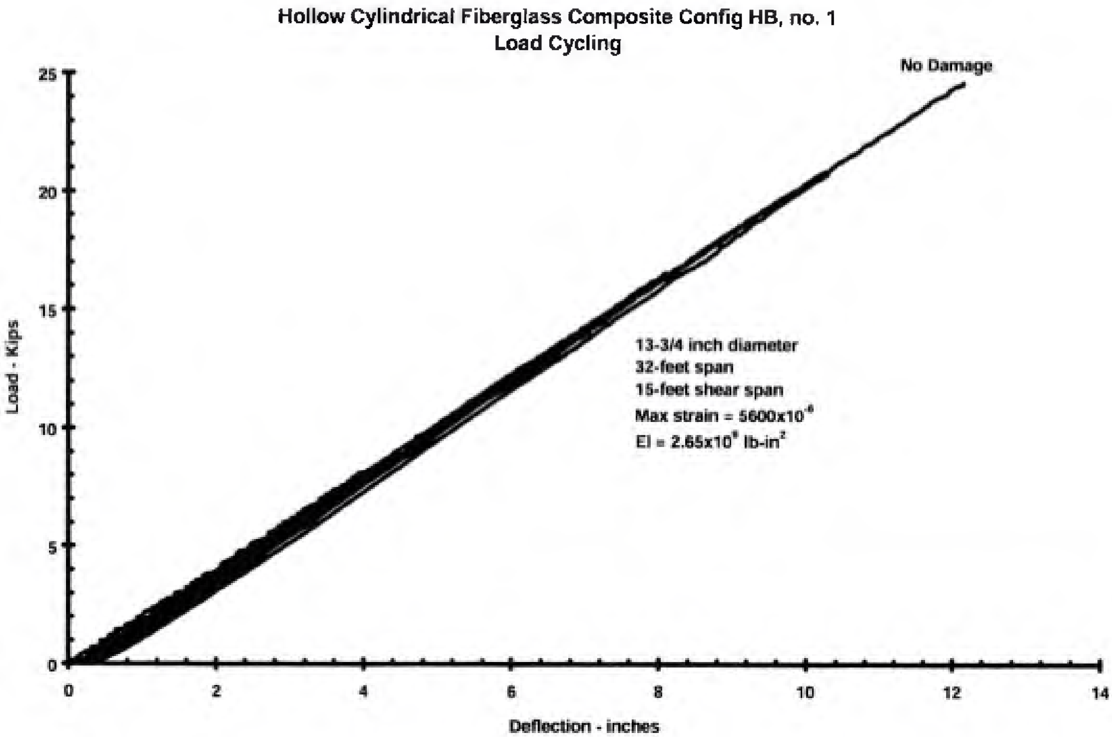


Figure A27. Cyclic load-deflection response of Configuration HB, specimen number 1, test setup B. (1 in-kip = 445 N, 1 inch = 2.54 cm)

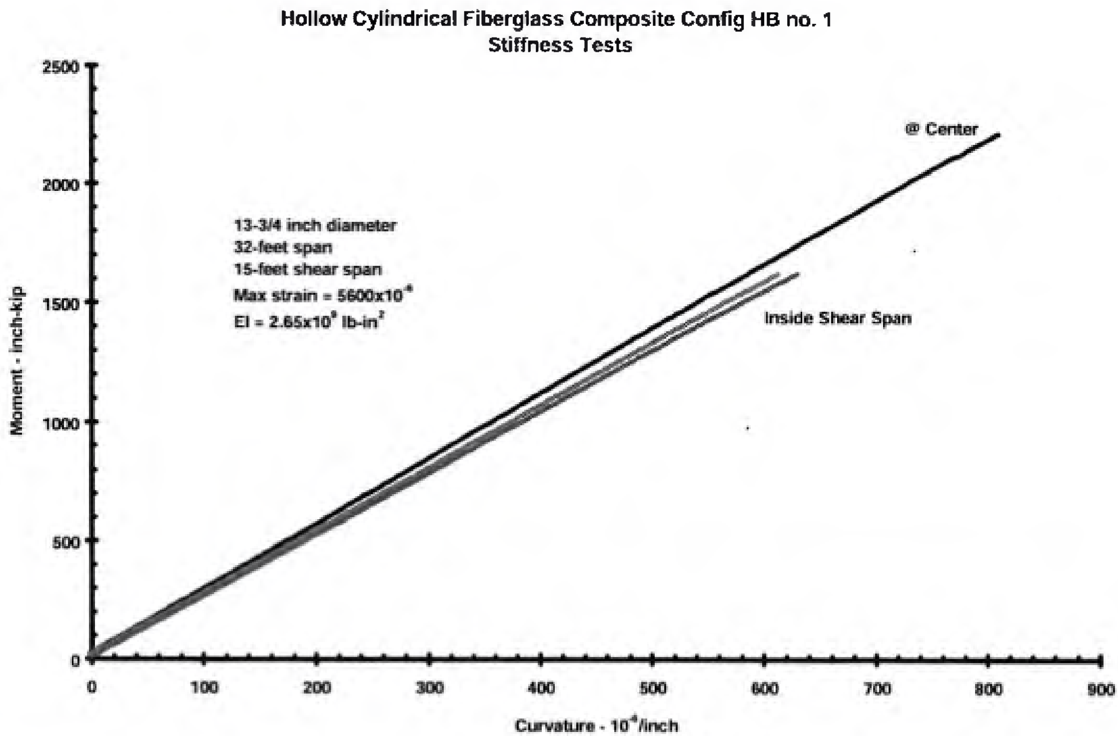


Figure A28. Moment-curvature response of Configuration HB, specimen number 2. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

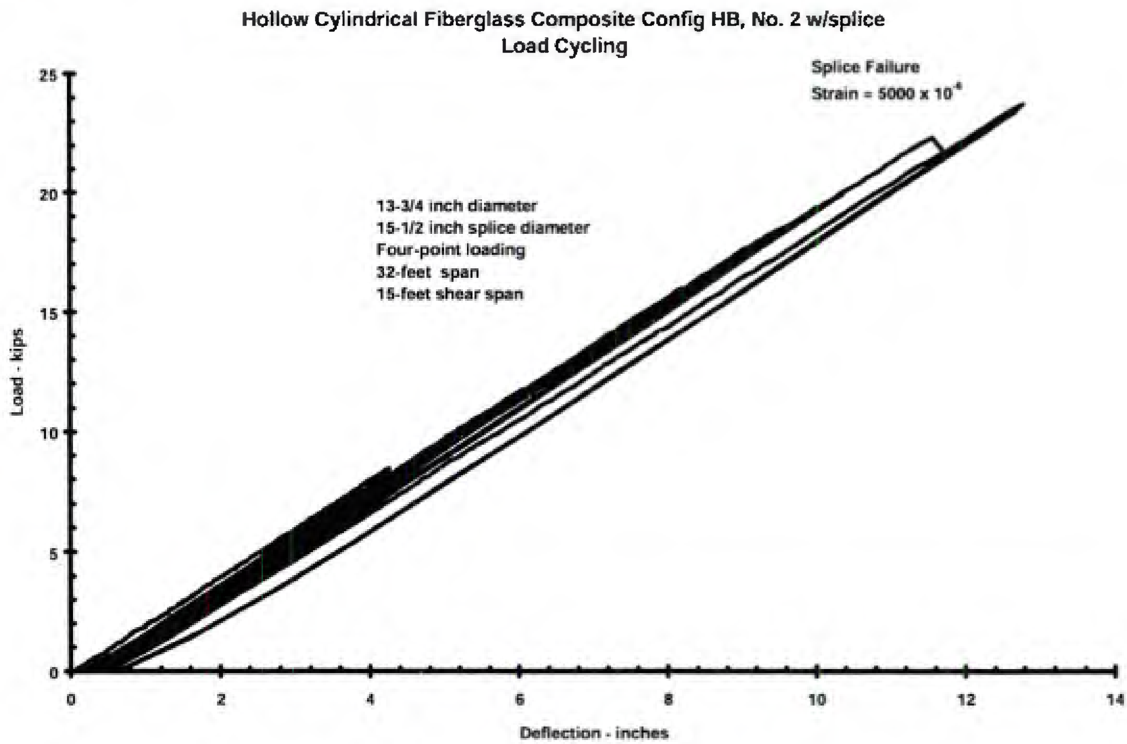


Figure A29. Cyclic load-deflection response of Configuration HB, spliced specimen number 2. (1 in-kip = 445 N, 1 inch = 2.54 cm)

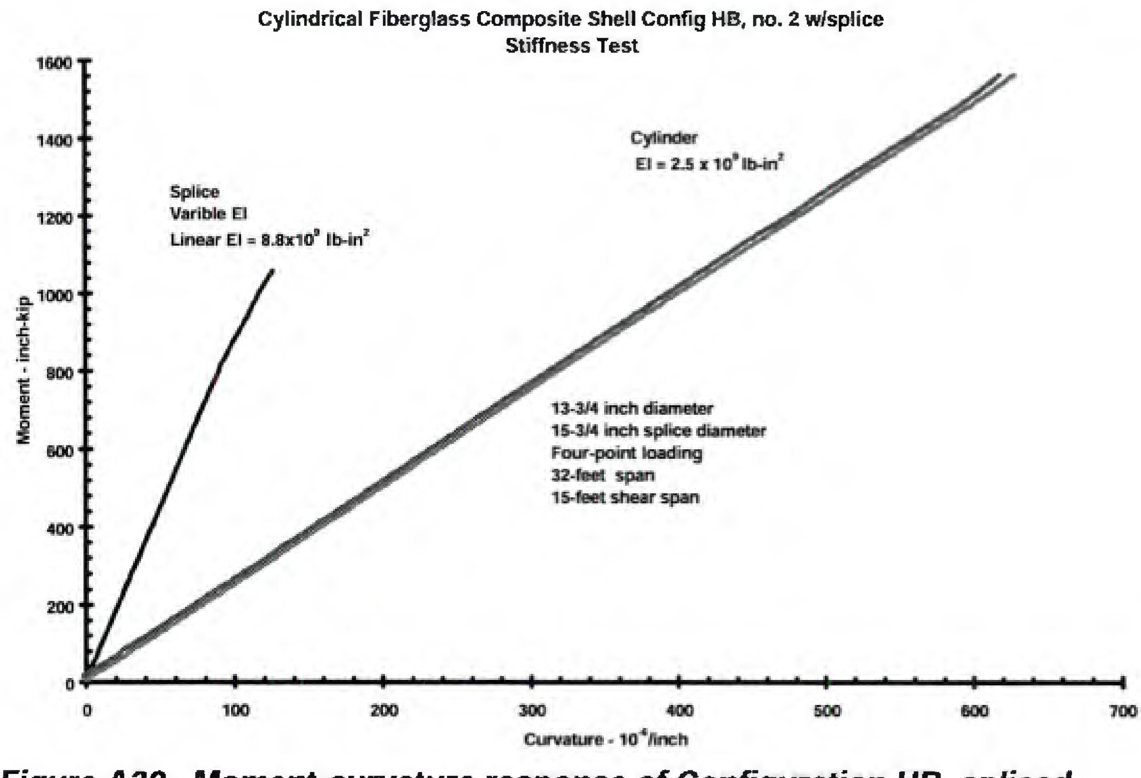


Figure A30. Moment-curvature response of Configuration HB, spliced specimen number 2. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)

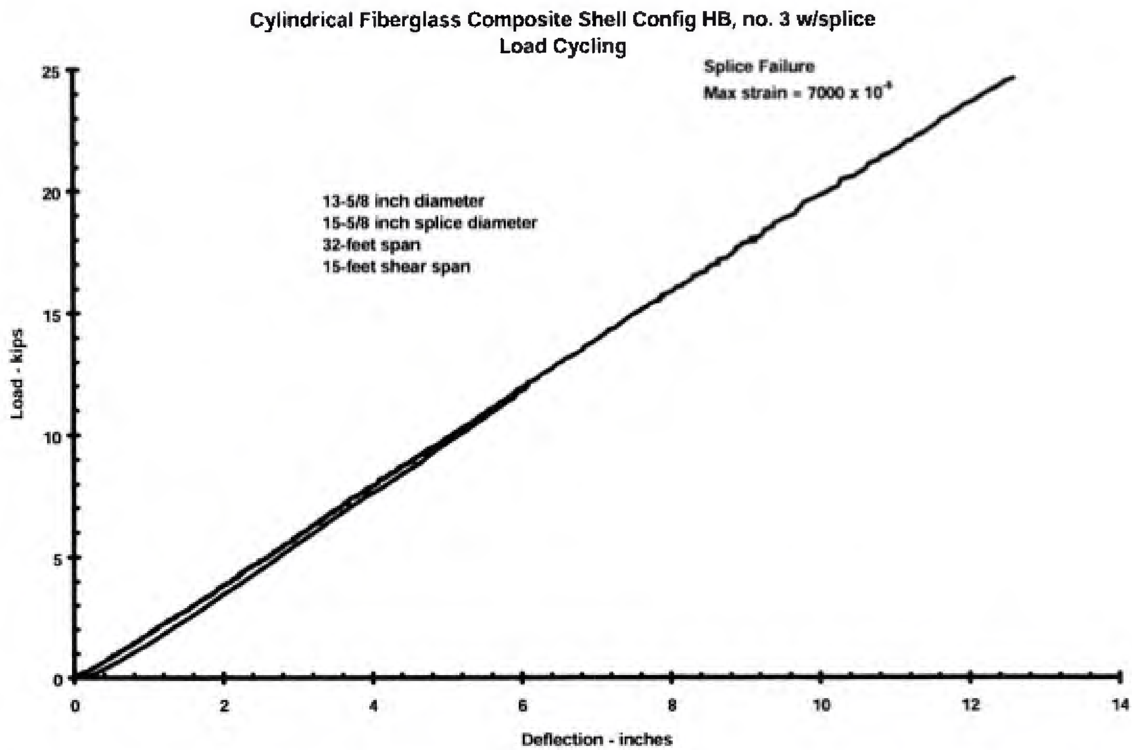


Figure A31. Cyclic load-deflection response of Configuration HB, spliced specimen number 3. (1 in-kip = 445 N, 1 inch = 2.54 cm)

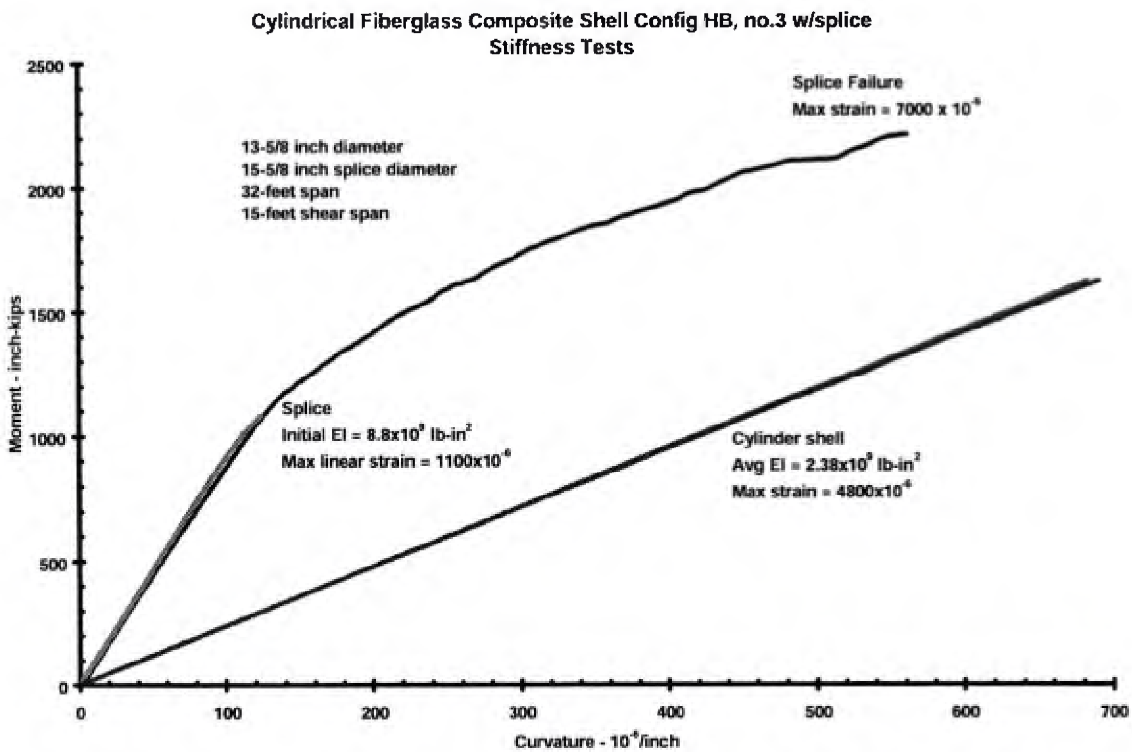
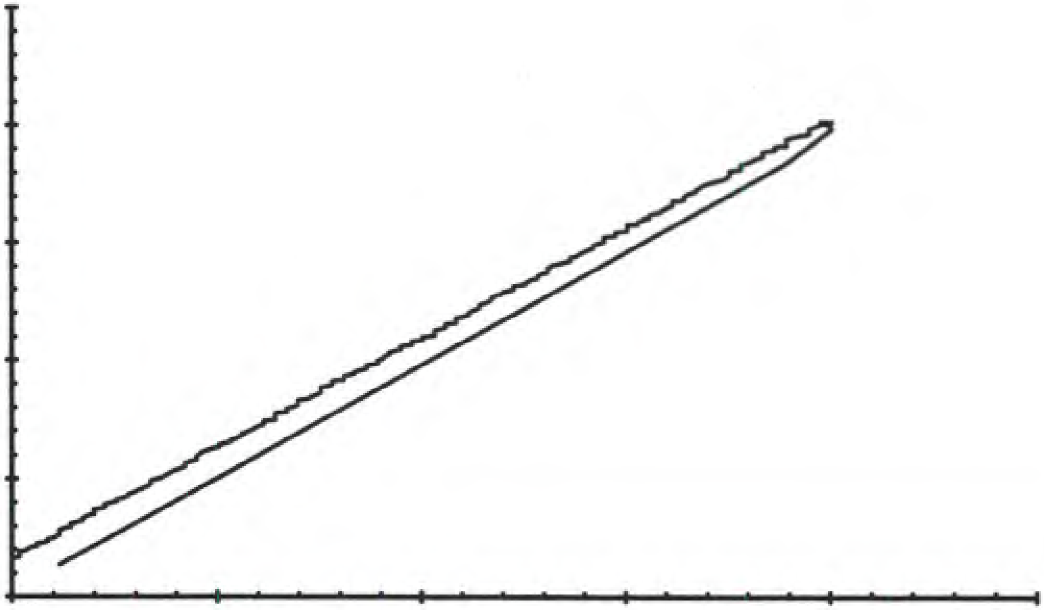


Figure A32. Moment-curvature response of Configuration HB, spliced specimen number 3. (1 in-kip = 113 N-m, 10^{-6}/inch = 0.4 x 10^{-6}/cm)



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121.2
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