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

# SITE SPECIFIC REPORT

*SSR-2410-SHR*

## IMPACT LOAD TESTS, STRUCTURAL ANALYSES AND LOAD LIMITATIONS OF THE MARGINAL WHARF

### TRIDENT REFIT FACILITY
### NAVAL SUBMARINE BASE
### BANGOR, WASHINGTON

by

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## *EXECUTIVE SUMMARY*

In response to Work Order N6843897PO9001 from the Trident Refit Facility (TRIREFFAC) the Naval Facilities Engineering Service Center (NFESC) conducted impact load tests on the Marginal Wharf at SUBASE Bangor. NFESC has also conducted finite element analyses of the pier to determine if five TRIREFFAC cranes can safely conduct lift operations. These cranes are the Lorain LRT 300D, Grove RT 655, P&H CN 150, Grove RT 880, and Grove TM 890.

Soft areas were identified near deck mods of the loading platform, near the approach addition, and at the base of the ramps. The piles, pilecaps, rail girders and stringers do not show degradation from current operations. The pier is in sound condition overall. However the deck slab was not designed to support wheel and outrigger loads of mobile cranes. In addition, some of the structural members are not properly designed in accordance with ACI 318 and the deck slabs are not reinforced for biaxial bending.

NFESC analysts computed the limiting resistance of individual structural members and compiled finite element models of pier systems and subsystems that are were validated to reflect ILM test response. The finite element models were used to determine working load limits for uniform loads and patch loads.

Rail girders, stringers and pilecap girders can support wheel loads of the five TRIREFFAC cranes as well as AASHTO HS20 truck wheels. However, the original deck slabs cannot. Maximum patch loads on the original deck slab should not exceed 20 kips. Dual axle wheel loads less than 16 kips (axle loads less than 32 kips and axle spacing at least 54 inches) and single wheel loads less than 20 kips are the maximum that can traverse the deck. Since the P&H CN150 and the Grove TM890 wheel loads are near the load limits, they should be restricted to 5-MPH speed limit to keep impact loads to a minimum. The Grove RT880 exceeds allowable material capacities throughout the deck and should not be operated on the Marginal Wharf.

Maximum outrigger loads of 100 kips can be placed over piles and pilecaps. The report contains a table of allowable loads that can be placed over the walls supporting the platform, the crane rails, and other structural members. The centroid of the outrigger floats should be placed within 1 foot of centroid of major structural members. Maximum outrigger loads of 70 kips can be placed within 6 inches of the longitudinal stringers of the main deck and loading platforms.

Since the wharf is in excellent condition we expect the north platform can still support its original design loads without restriction. We estimate the wharf can continue to operate under the above restrictions for at least 20 years unless corrosion accelerates in the vicinity of "working" cracks that are identified in the report. The large transverse cracks and the boundaries of deck modifications should be continuously monitored visually for crack growth and steel corrosion. These cracks will continue to grow and the reinforcement will corrode. Eventually the flexural strength in the area will be measurably reduced. We do not recommend any immediate action but the cracked areas should be repaired within the next five years.

NFESC recommends rebuilding piles 42A and 64A on the south pier by composite hard shell encasement rather than replacement. We also recommend that a crane path be devised across the approach, main deck, and ramp to the platforms. The crane path can be made by adding external reinforcement to the existing deck. The upgrade would include embedding carbon reinforcement rods in the main deck over the rail girders and adding carbon laminate to the bottom of the deck slabs. NFESC can assist the NAVFAC Northeast Engineering Field Activity in preparing designs with specifications. This assistance is beyond the scope of the current work order. The upgrade can be designed to remove restrictions on crane wheel loads over the crane path. The pier should be reevaluated in five years.

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## OBJECTIVE

The objective of this project is to assess the structural integrity of the Marginal Wharf at Trident Refit Facility Submarine Base Bangor WA and to determine the crane and wheel load capacity of the deck and transverse pilecap girders and longitudinal stringers. The study specifically addressed restrictions on uniform live load, AASHTO HS20 truck wheel loads, and the wheel and outrigger loads of five mobile truck cranes: Grove RT65S, Grove RT880, Grove TM890, Lorain LRT300D, and P&H CN150.

The Naval Facilities Engineering Service Center (NFESC) conducted load tests and finite element analyses on the Marginal Wharf. The load tests employed the impact load method (ILM) using a falling weight deflectometer (FWD). The purpose of the tests was to provide measures of the structural response in order to validate finite element models that were used to quantify the structural reliability and load limits of the pier.

## BACKGROUND

The Marginal Wharf was constructed in 1945. It is located near the center of the Naval Submarine Base Trident Refit Facility (TRIREFFAC) and is used to refit submarines, effect minor repairs, and support dive operations. The Wharf consists of two piers that extend parallel to the shoreline in the north-south direction, an approach, and a nonfunctional railroad trestle access that is not being analyzed. The Wharf is a monolithic reinforced concrete structure and consists of a deck supported by stringers and pilecaps on 16-inch and 18-inch square piles. Piles over 55 feet long are 18 inch square while the remainder are 16-inch square. The north and south piers are approximately 600 feet and 860 feet long, respectively, and 87 feet wide (Figures 1 and 2). An approach deck (Figure 3) provides vehicular access at the intersection of the north and south piers. The structure's deck consists of a main deck, approach deck and loading platform decks. The approach and main decks are nominally 7-1/2 inches thick and the platform deck is 8 inches thick. The Wharf was built without expansion joints. Figure 4 is a nominal cross section of the original pier. Pile bents are spaced 10 feet on center. Half the pile bents contain batter piles. Figure 5 is a wharf plan with the pile layout.

The main pier deck was originally constructed for train cars and wagon cranes for ship services. The loading platforms were designed for wagon crane operation. The 1944 Design loads included:

- Cooper's E-60 (Figure 6) for the railroad system,
- 600 psf or 12-ton truck or 15,000 pound wheel load for the wharf deck,
- 800 psf or 15-ton wagon crane (19,000-pound maximum wheel load at 5.33 feet center to center) for the platform,
- 40-ton pile load.

1954 modifications to the south pier platform added a 10-inch thick deck section. The approach was also widened with a 9-inch deck in 1954. 1963 modifications to the north pier platform included addition of a crane rail system on a rail beam supported by additional 50-ton capacity, prestressed concrete piles. An 8-inch ramp was also added to the south platform in 1963. The 1963 modification design loads included:

- Crane Loads: A 20-ft gauge, 32-wheel crane with a maximum wheel load of 56,000 pounds plus 15 percent impact on each of 8 wheels.
- Truck Loads: 25,000 pounds axle load plus 15 percent impact.

The rail cars have been suspended from service and the rails were removed from the main deck in 1984. Pierside maintenance and other lifting operations are performed with the track mounted crane on the north pier platform or mobile truck cranes. The forces from the outriggers supporting the truck cranes are not restricted to the girders and do not have the same load distribution mechanisms as the original design loads. The mobile cranes' outriggers place greater loads on the deck slab and stringers than the original design loads.

**Figure 1. North pier of the Marginal Wharf looking south.**

**Figure 2. South pier of the Marginal Wharf looking North.**

**Figure 3. Marginal Wharf approach.**

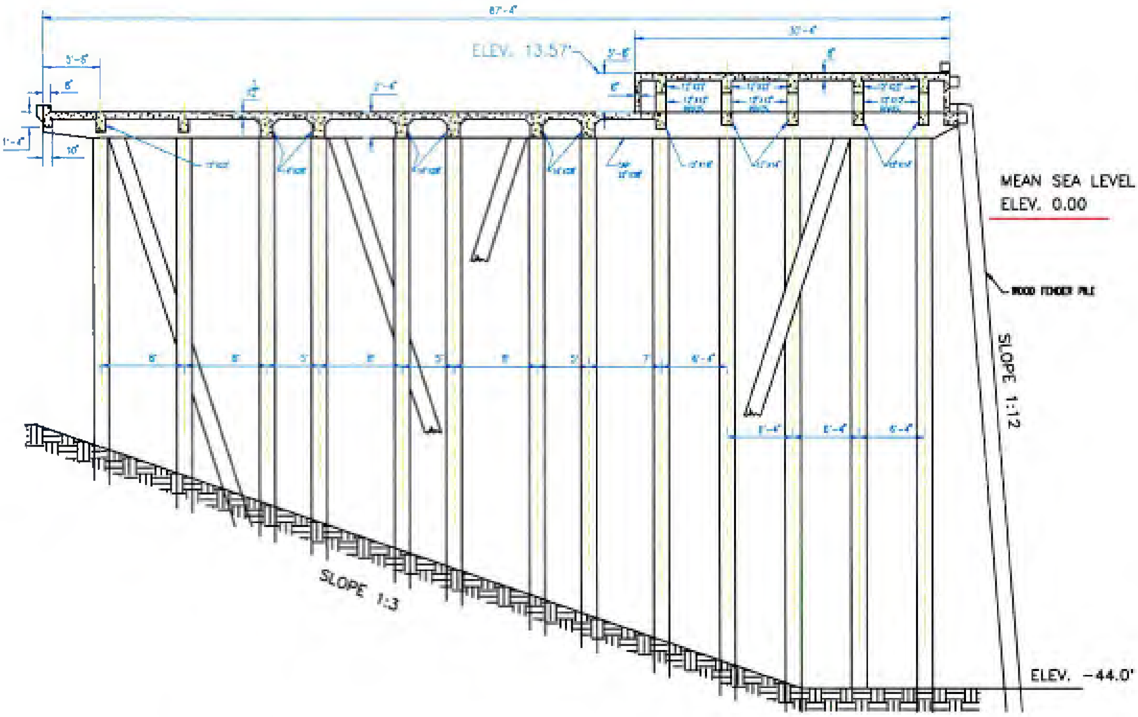



**Figure 4. Nominal cross section (east-west) of original Marginal Wharf.**

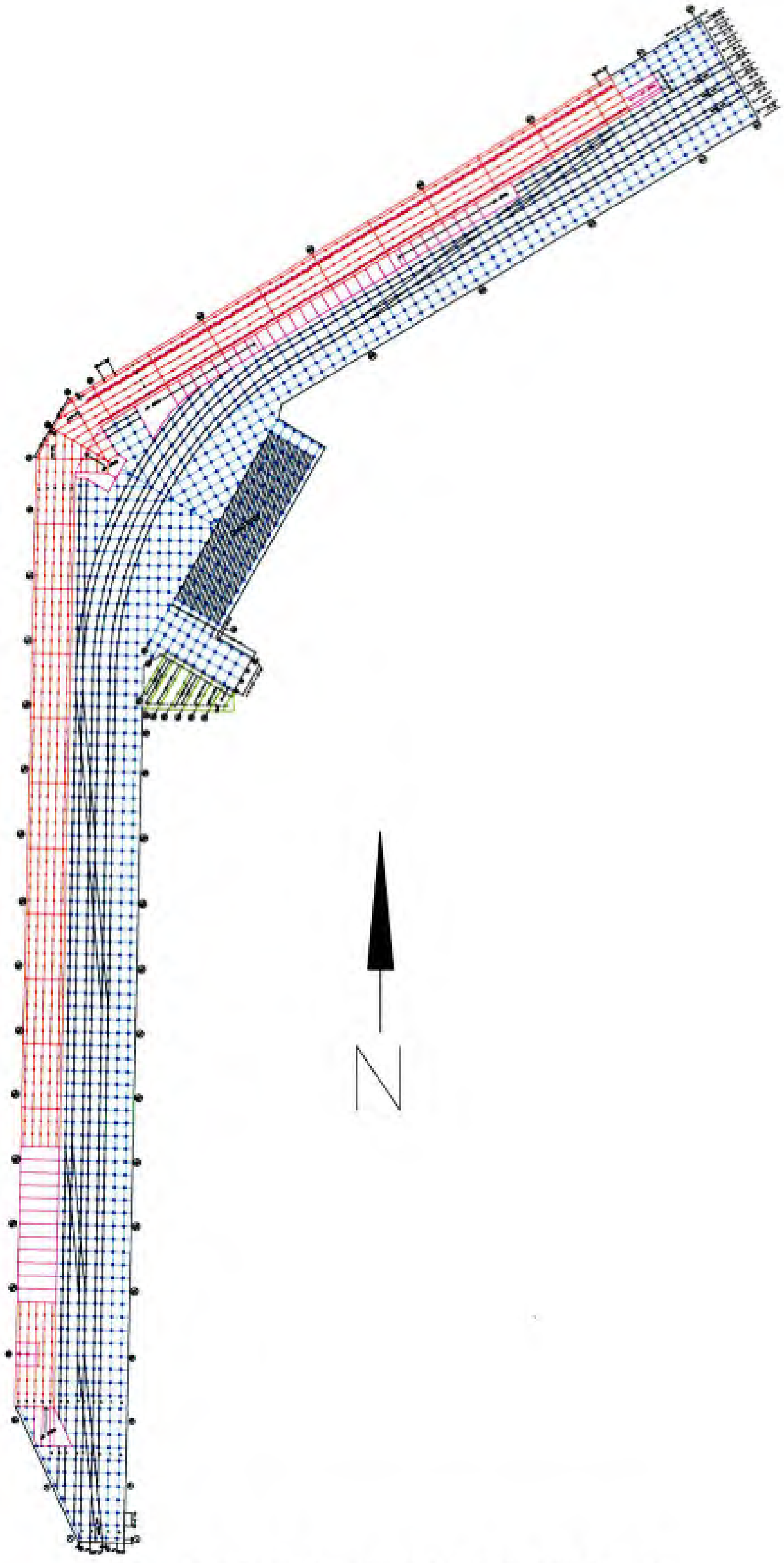



**Figure 5. Plan of Marginal Wharf with pile layout.**

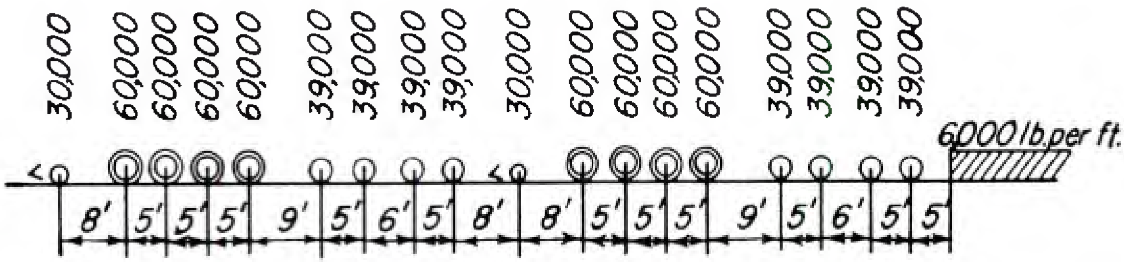



**Figure 6. Cooper E-60 locomotive loading.**

An in depth analysis of the Marginal Wharf was conducted by Johnson Controls in 1994. This analysis set wheel load limits on the wharf primarily because of the thickness of the deck slabs. The Johnson Controls analysis is accurate and thorough. The strengths of individual structural elements (stringers, pilecaps, and deck) are accurately assessed. We take some exception to the unidirectional methodology of load distribution and structural response used. While the methodology is traditional and acceptable, it is very conservative and will under estimate the actual load capacity of the wharf. The NFESC approach of load testing coupled with finite element modeling will produce a more realistic representation of the structural response and the load distribution mechanisms.

Operators of mobile cranes on the south platform place outriggers on, or near, the outside pile Lines, A and E, of the platform (Figure 7). These piles, as well as the 12-inch by 12-inch post supporting the deck above the pile caps must be maintained in excellent condition. Outriggers are placed on wood panel cribbing. While these wood panels will prevent the deck surface from being scarred by the outrigger floats, they are not sufficiently stiff to enhance distribution of the outrigger loads. To spread the outrigger load further than the floats, heavy timber or, preferably, metal cribbing should be employed. The disadvantage of proper cribbing is its heavy weight, which usually requires a forklift for positioning.

**Figure 7. Truck crane on the south platform of the Marginal Wharf.**

## *EXISTING WHARF CONDITIONS*

We reviewed the following documentation in preparation for the tests and analyses:

(Original) Drawings of Marginal Wharf dated May through November 1944.
Alterations to Marginal Wharf (Approach, ramps, elevator) Drawings of April 1953.
Marginal Wharf Modifications (Ramp and Crane Rails/Beams/piles) Drawings December 1962.
Marginal Wharf Structural Repairs Drawings of 31 January 1969.
Marginal Wharf Inspection and Repair (piles and beams) Drawings of July 1984.
Marginal Wharf Surface Repair (train rail removal) drawings by Alpha Engineers of July 1984.
150-TON MOBILE CRANE STUDY REPORT by Whitacre Engineers of August 1986.
MARGINAL WHARF CRANE ANALYSIS (Phase I) by Johnson Controls of 10 February 1992.
Johnson Controls Memo w/calcs to Ken Swartz re Crane Operation of Marginal Wharf 90-ton Grove Crane of 18 Aug 1992.
MARGINAL WHARF CRANE ANALYSIS (Phase 11) by Johnson Controls of 28 September 1992.
Underwater inspection report by Han-Padron Associates in May 1994.
Marginal Wharf Repairs 1994 drawings by Johnson Controls of August 1994.
Underwater inspection report by Russell-Veteto Engineering, Inc. of May 1997.

After the last underwater inspection the A&E team concluded the pier was in good condition. The concrete piles and substructure were typically sound. However, two impact-broken piles (42A and 64A) that had been noted in previous inspections had not been replaced or repaired. Worst case spall damage occurs in a beam supporting the approach and part of the 1954 approach addition. Except for the damage in the 1954 addition to the approach, the marginal wharf spalls and delaminations are minor. We found no reports of failures or damage due to wheel or crane outrigger loads. The marginal wharf does not display any evidence of immediate danger of failure from current service loads.

In addition to the two broken piles there are two structural conditions that could restrict the load carrying capacity of the pier deck at some time in the future. One is the presence of large cracks that have occurred because of the absence of expansion joints in the original construction (Figure 8) or structural modifications to the original deck. Large transverse crack systems occur at Bents 31, between Bents 68 and 69, and between Bents 110 and 111. Attempts to patch and seal these cracks have failed because they are working cracks and continue to widen and grow. We suspect the reinforcing bars at these cracks are badly corroded which may result in a loss of section strength. Similarly, structural modifications to the approach and the platforms have resulted in construction joints that did not integrate with the original deck slab. The "new" construction boundaries have resulted in open cracks. Exposed reinforcing bars are visible on the approach.

**Figure 8. Crack emanating from approach addition (in the foreground) between bents 68 and 69.**

The second condition is the lack of continuity across the rail girders caused by the insertion of rails in the original construction. The train rails prevented placement of reinforcement on the top face of the slab over the rail girder in the strong bending direction (Figure 9). Longitudinal cracks over the extension of these girders on the south end of the South Pier where rails were never placed also indicate missing negative reinforcement. The flexural resistance (negative moment) transverse to the girders is negligible and the shear resistance is reduced because of the slot cut in the deck. The rails were removed in 1984 and the rail slot was filled with "shrinkage compensated concrete" (a cementitious grout). Reinforcing steel was not added to the section. The principal tension stress field in this area is perpendicular to the slot wall surface and will cause the grout to separate from the original concrete (producing a vertical crack). NFESC proposes reinforcing upgrades for negative moment over the rail girders.

**Pile Capacity**

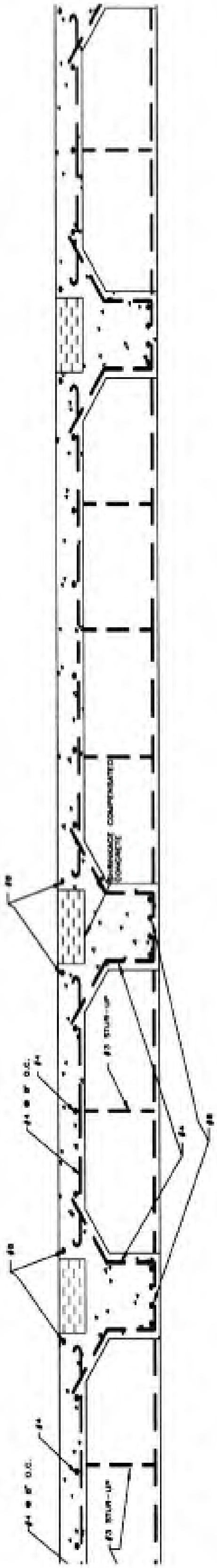
The piles were driven with a single acting steam hammer. The weight of the hammer was 5,000 pounds and the operating stroke was 3 feet. The bearing capacity, P, of the piles was calculated in the original design and construction by the Engineering News formula (a dynamic pile-driving formula):

$$P = 2\,W\,H / (S + C)$$

where W is the weight of the hammer, H is the operating stroke, and S is the average penetration of the last 10 blows. The constant, C, is usually set equal to 0.1 for a steam hammer; but, the original design used 0.3. The bearing capacity, P, would be calculated slightly higher using the former value. In granular soils the formula has a safety factor of about 6. In plastic soils it can be unconservative.

Since the soils in the region are granular we suspect the piles are capable of sustaining higher service loads, if necessary, than the design limit of 40 tons. Since pile l/r is less than 130, the buckling loads exceed 60 tons. This means that the pile capacity is governed by the pile driving formula. Based on the load tests, we suggest the pile limits be increased to 50 tons for short term loads like wheel and outrigger loads.

**Figure 9. Cross section detail of main deck above rail girders after rail removal.**



## *ILM METHODOLOGY*

ILM was developed as a means of structural excitation for performing condition assessment. A FWD provides a rapid means of applying an impulse load. The FWD is a trailer-mounted, computer-controlled, load-testing device (Figure 10). NFESC uses a small Kawasaki "Mule" to tow the trailer and house the computer controls and data processor. NFESC developed the ILM test and finite element analyses (FEA) methodology for analyses of Naval waterfront structures. These combined technologies identify areas of most sensitive structural response to vehicular loading, provide quantification of structural condition, and provided data for setting facility load limits. Finite element models that are validated with ILM results are capable of predicting structural response to all loads.

**Figure 10. Falling weight deflectometer and tow vehicle on Marginal Wharf.**

The FWD used to obtain the impact load response was a Dynatest model HWD 8081. A portable computer monitored, digitized and stored the electrical analogs of load pulse and pier deflection response. Deflection data was recorded at 7 locations along a transducer beam that projected from the load point along the longitudinal axis of the FWD trailer. For example, Figure 11 is the load and deflection histories for slab panel of the platform bounded by pile Lines C and D and bents 25 and 26 with the impact load at the center. Peak deflections from each sensor time history determine the deflected basins that characterize the stiffness of the structural elements in the vicinity of the load application (Figure 12). Analysts compare these deflections with those generated by the FEA using the same loading.

## *ILM PROCEDURE ON THE MARGINAL WHARF*

NFESC analysts developed a grid that covered the accessible surface of the wharf south platform, main deck and approach. Load tests are conducted at the grid nodes. Grid lines were located with reference to transverse pile bents and longitudinal pile lines. The test grid was laid out so that tests were performed over piles, midway between piles, midway between bents on longitudinal stringers and girders, and the middle of deck slab panels. Load points were marked on the deck surface along chosen grid lines. Pile bents and lines have alphanumeric designations consistent with existing drawings. Tests are given the same alphanumeric code as the load point.

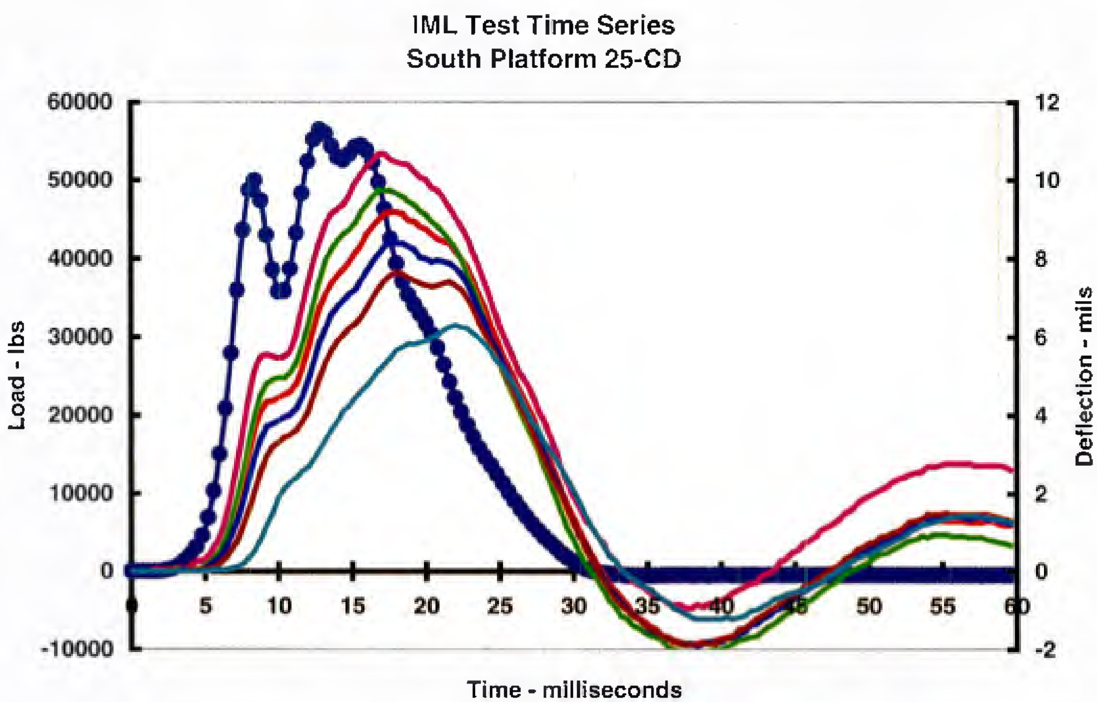



Figure 11. ILM load and displacement time histories.

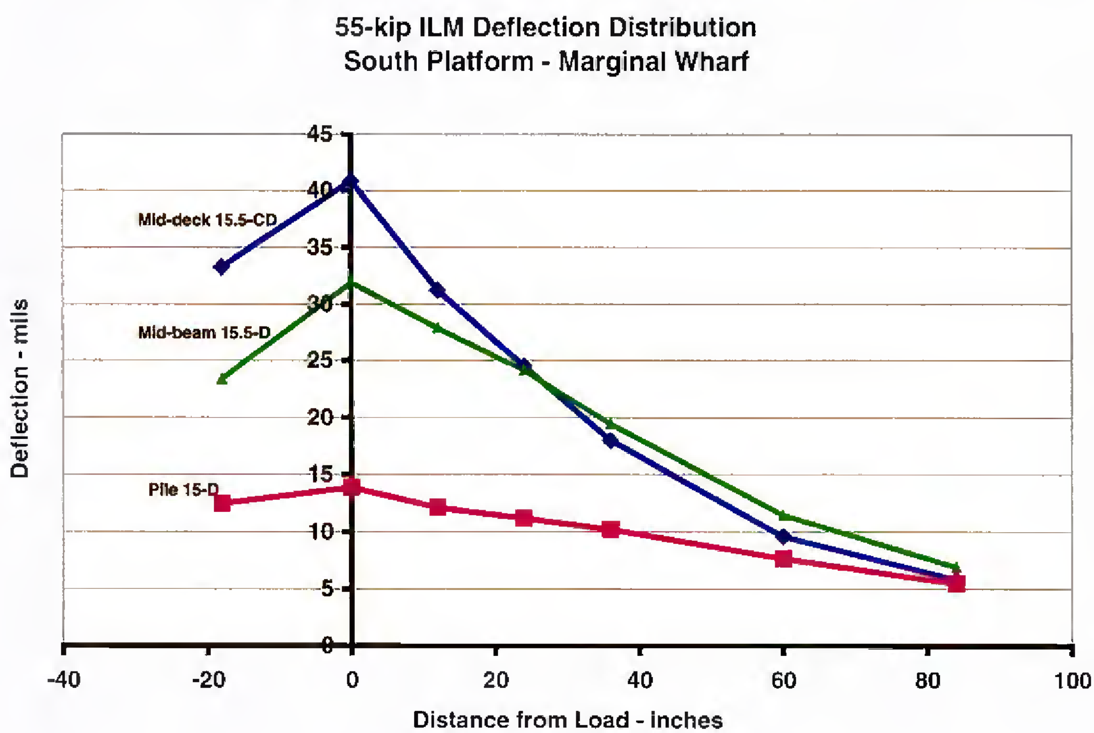



Figure 12. ILM deflection distribution.

Each load test consists of at least 3 impact load applications preceded by a small load to set the load platen. Peak loads ranged from 55 to 60 kips. Varying load levels are applied at random locations to check the linear load-deflection response of the structure (Figure 13). Displacement sensors are positioned along the transducer beam at 18 inches aft of the load point, at the load point, and at 12, 24, 36, 60 and 84 inches forward of the load point. The transducer beam is oriented along the longitudinal lines of the test grid and perpendicular to the pile bents.

After the data processor has digitized and converted the analog signals, peak values are printed and stored on data disks for further analysis. A portable printer prints the load deflection data with correct conversion to engineering units of kips and mils. The FWD operator usually makes a cursory examination of the peak load and deflection values after each load series looking for unusually large deflections and indications of nonlinearity or random results. All data is stored in computer files compatible with the database program, EXCEL©. Each test will generate up to 1,200 data values.

For reasons of safety and instrumentation stability, tests were not conducted near heavy equipment, electrical cables, or on the ramps to the loading platform. Tests could not be conducted in locations that were not accessible to the tow vehicle, FWD, and instrumentation. The SOD platforms that were recently added were not tested because the impact load may separate the new concrete from the old deck slab. Tests could not be conducted on the asphalt overlay along the edge of the platform (Line D) between Bents 76 and 82 because the asphalt dampens the impact of the falling weights and the layer tilts the FWD outside its tolerances. The north platform was not tested because the impact load of the FWD is not large enough to excite the massive pile system supporting the crane rails.

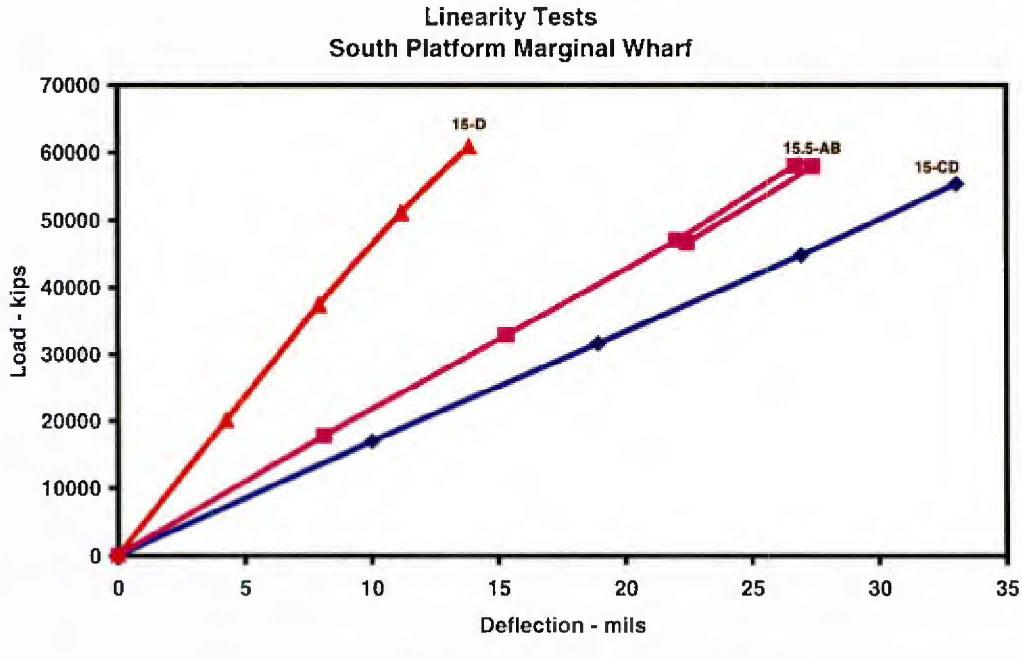



**Figure 13. Check for linear response of load-deflection.**

## *FINITE ELEMENT ANALYSIS (FEA) MODEL DEVELOPMENT*

NFESC analysts performed elastic analyses of the Marginal Wharf using the computer programs ABACUS and STARDYNE. FEA provides a means of efficiently and accurately determining vertical load response. We employed elements with orthotropic properties of steel-reinforced concrete and modified the properties and model geometry to reflect measured response to impulse loading. Results consisting of deck, pilecap, stringers, rail girders, and pile reactions are compared to limiting ACI values.

Initial finite element analysis includes geometry and dimensions taken from available drawings. We assigned concrete (3500 psi strength) properties to the finite element members. Calculated strengths of structural elements were obtained using ACI 318 methodologies. The cross sections of piles, pilecaps and stringers were fully represented without deterioration.

A large range in concrete stiffness was reflected in the ILM response that corresponds to a spread in concrete strength. The initial modulus of elasticity, $E_c$, for concrete was taken as 3,400,000 psi (3500 psi concrete strength). The $E_c$ values were adjusted to 4,000,000 psi (5000 psi concrete strength) to best reflect the ILM response in the refined models.

The finite element models were contrived to reflect the behavior of systems and subsystems of the Marginal Wharf. The pier deck was modeled with plate elements that accounted for bending, membrane, and shear deformation. The pilecaps, stingers, rail girders and piles were modeled with beam elements. Detailed models of small areas used all 3-D elements. Each model had appropriate boundary conditions that reflected neighboring structural areas. We tested 19 FEA models. Figures 14, 15, 16, and 17 are graphic representations of four of these models. The FEA model element nodal grid and the ILM test grid and deflection measurement points were designed to coincide.

The mudline under the pier is approximately 10 ft. below the deck near the shore and dredged on a 1:3 slope to approximately 55 feet below the deck at the outboard edge (Figure 4). The piles were fixed in the finite element model at 10 feet below the mudline. For loads applied near the middle of deck panels we determined the bending and torsional stiffness of the rail girders and pile caps restricted much of the effects of applied vertical loads to the loaded deck panels. Therefore, using symmetrical and antisymmetrical boundary conditions at terminating pile bents, the Marginal Wharf was analyzed in sections so that subsystems could be modeled in detail.

ILM loads, wheel loads, and outrigger loads are patch loads. That is, the loads are applied over an area as opposed to a point load. ILM loads were applied over 12-inch diameter patch and wheel loads were applied over an 8-inch by 12-inch rectangular patch. Patch loads are represented in FEA as pressure loads over an area or as multiple point loads applied to adjacent element nodes in close proximity to each other.

The modified finite element model that best reflected pier behavior was used to determine response to wheel and outrigger loads placed in critical locations on the pier deck. We analyzed five crane wheel and outrigger configurations (Figure 18), AASHTO HS20 truck loading (Figure 19), as well as uniform live loading. Dead load was applied in all load cases.

**Wheel Loads**
The weakest structural members of the Marginal Wharf system for patch loads are the deck slabs. The wheel loads of the larger 4-wheel (2 axles) cranes are the more difficult to support than AASHTO HS wheel configuration and cranes with 3 or more axles. We looked at the wheel loads of the cranes and AASHTO HS20 truck at critical locations at the centers of the deck slab panels of the south platform, ramps, main deck and approach. The maximum wheel loads of all the cranes exceed the original design requirements of the Marginal Wharf system. All vehicles do not have simultaneous wheel loads on adjacent deck panels of the main deck except the Grove TM 890.

**Outrigger Loads**
Outrigger spread is too large for the floats to fit in neighboring slab panels. We applied individual, 100-kip patch loads over a 24-inch square area to represent outrigger loads. We also applied the same outrigger loads spread over a 48-inch by 48-inch area to represent cribbing that is compliant with the concrete surface. Since the FEA is linear, we extrapolated to different outrigger load levels and used superposition to determine load effects of multiple outriggers. We analyzed individual patch loads applied over piles, on pilecaps between piles, on beams between pile bents, and in the middle of deck slab panels.

Table 1. Strength of Marginal Wharf Structural Elements

| Element | Longitudinal Flexural Strength +/- (1) | Transverse Flexural Strength +/- (1) | Shear (2) | Wheel Punching Shear Strength (3) | Outrigger Punching Shear Strength (3) | NOTES |
|---|---|---|---|---|---|---|
| Main Deck | | | | | | |
| 7-1/2-inch Deck | +43.8 / -37.0 | +51.5 / -43.1 | 7.1 | 70.3 | 153.6 | Deck between pile line (or rail girder) "K" and inboard curb |
| 7-1/2-inch Deck | +39.1 / -37.0 | +61.9 / -N.A. | 7.1 | 70.3 | 153.6 | Deck between rail girders. No negative moment reinforcing over rail girders. |
| 12 in. x 22 in. Stringer | +1178 / -1178 | | 58.5 | | | |
| Long. Rail Girder | +3109 / -1586 | | 73.0 | | | Does not meet ACI minimum negative flexural reinforcement requirements |
| 22 in x 28 in. Pilecaps | | +1564 / -3001 | 49.5 | | | Does not have sufficient reinforcement to meet ACI minimum flexural and shear reinforcing requirements |
| Approach | | | | | | |
| 7-1/2-inch Deck | +43.8 / -37.0 | +51.5 / -43.1 | 7.1 | 70.3 | 153.6 | |
| 9 – inch Deck Mod | +65.4 / -N.A. | +173.2 / -173.2 | 9.2 | 101.5 | 187.6 | |
| 22 x 28 inch Pilecap | +1564 / -3001 | | 65.1 | | | Does not have sufficient reinforcement to meet ACI minimum flexural and shear reinforcing requirements |
| 20 x 28 inch Mod Pilecap | | +1017.9 / -2274.5 | 59.1 | | | Does not have sufficient reinforcement to meet ACI minimum flexural and shear reinforcing requirement. |
| Platform | | | | | | |
| 8-inch Deck | +45.1 / -45.1 | +65.9 / -65.9 | 8.9 | 79.5 | 170.4 | |
| 10-inch Deck Mod | +255.2 / -255.2 | +71.2 / -71.2 | 11.3 | 136.3 | 242.3 | |
| 10-inch deck elevator mod | +138.4 / -138.4 | +55.1 / -N.A. | 11.3 | 136.3 | 243.3 | No reinforcing ties to original deck |
| Orig. Ramps | +45.1 / -45.1 | +65.9 / -65.9 | 8.9 | 79.5 | | |
| Ramp additions | +255.2 / -255.2 | +71.2 / -71.2 | 11.3 | 136.3 | | |
| 12-in x 22-in Stringer | +1178 / -1178 | | 48.5 | | | |
| 12-in x 14-in Tie Beam | +663 / 663 | | 8.2 | | | Does not meet ACI minimum shear reinforcing requirements |
| Pilecap | | +1564 / -3001 | 65.5 | | | Does not have sufficient reinforcement to meet ACI minimum flexural and shear reinforcing requirements |

(1) Values in in-kips for beams or in-kip/ft width of slab. (2) Values in kips for beams or kips/ft width of slab. (3) Values in Kips.
(+) Values are for midspan and (-) values are over supports and intermediate supports. Values do not include ACI material reduction factors.

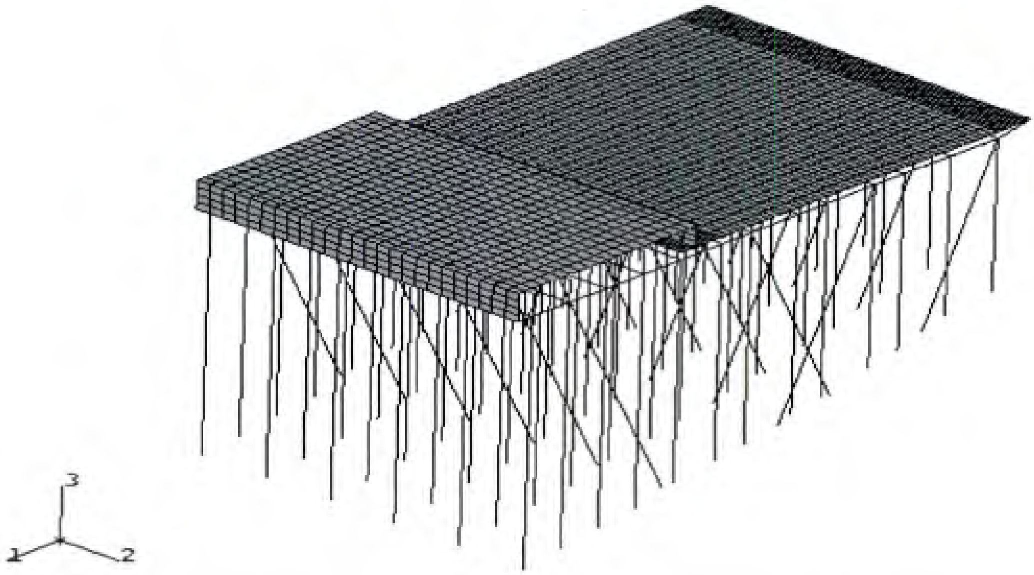



**Figure 14. ABAQUS FEA model of typical Marginal wharf structural systems.**

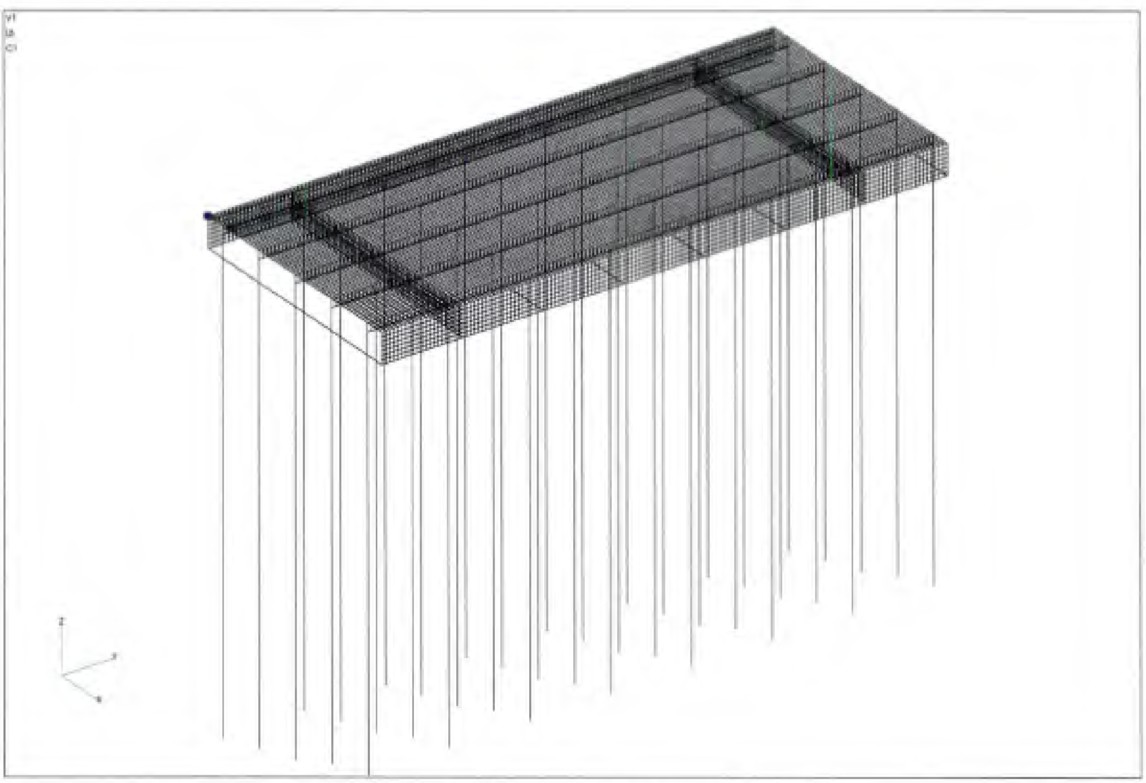


**Figure 15. Detailed FEA model of Marginal Wharf platform between shear walls.**

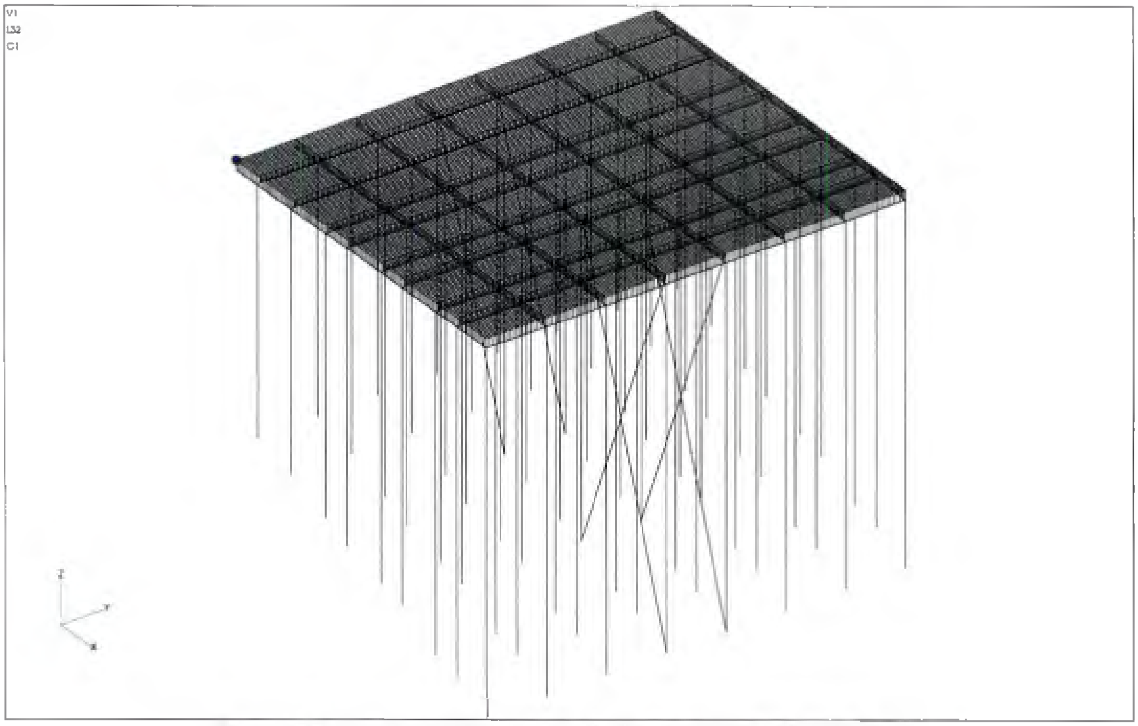



**Figure 16. Detailed FEA model of Marginal Wharf main deck.**

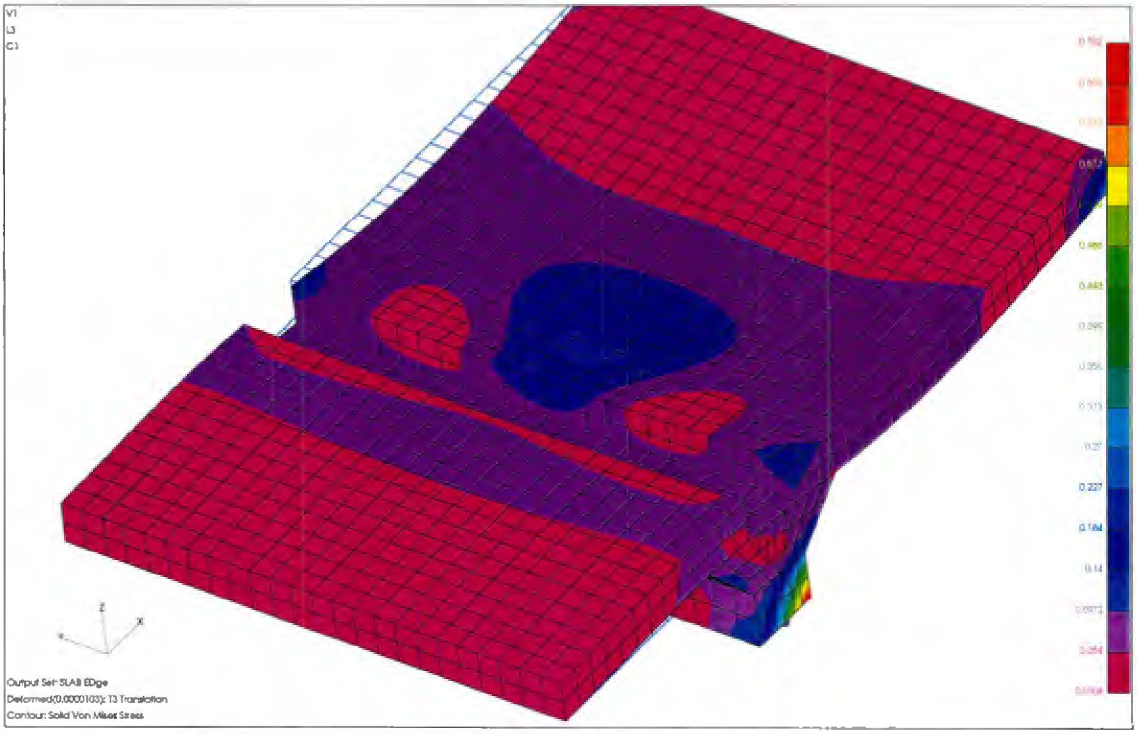



**Figure 17. FEA model detail of slabs and rail girder slot in main deck.**

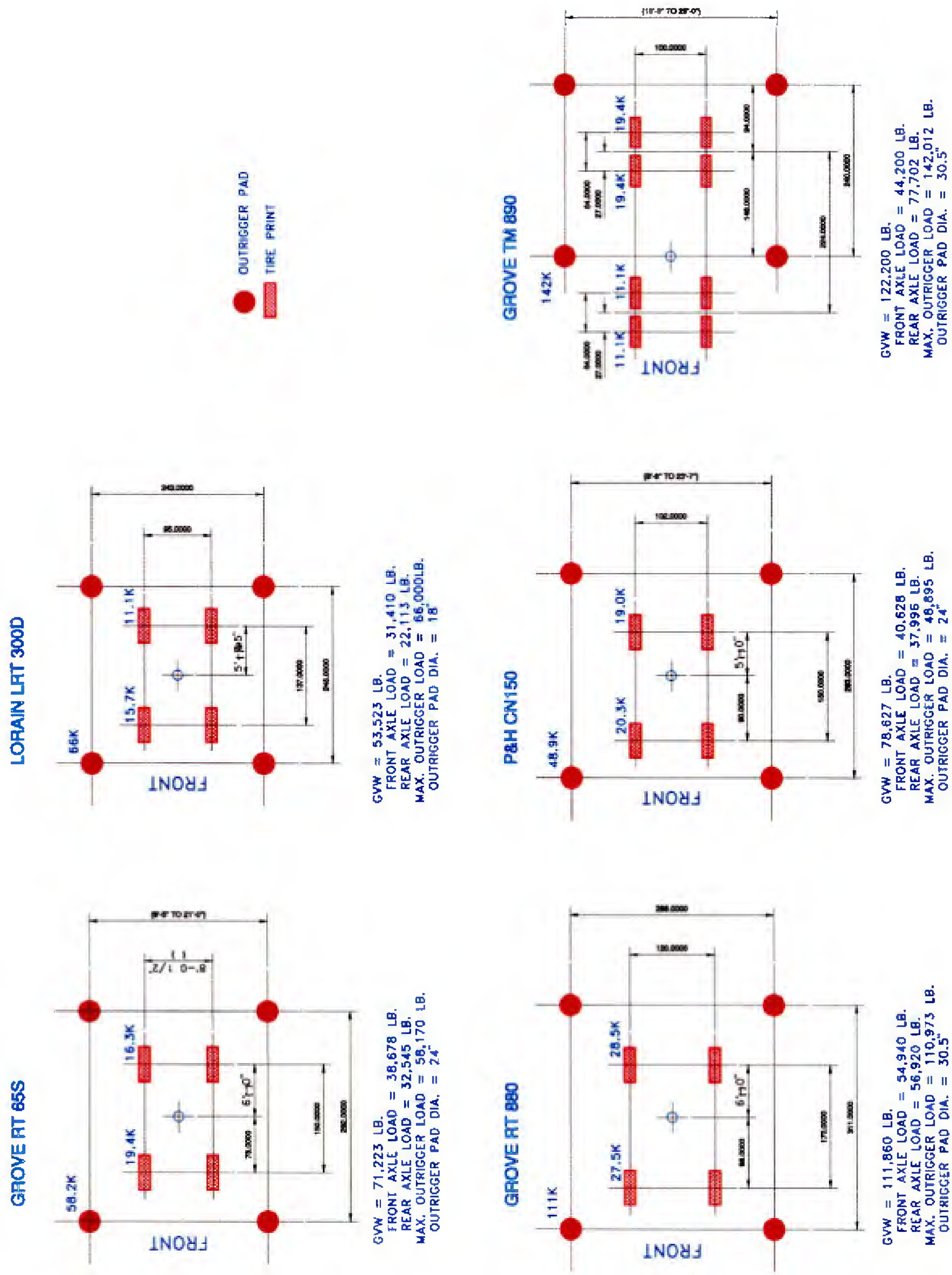



**Figure 18. Crane outrigger and wheel configurations.**

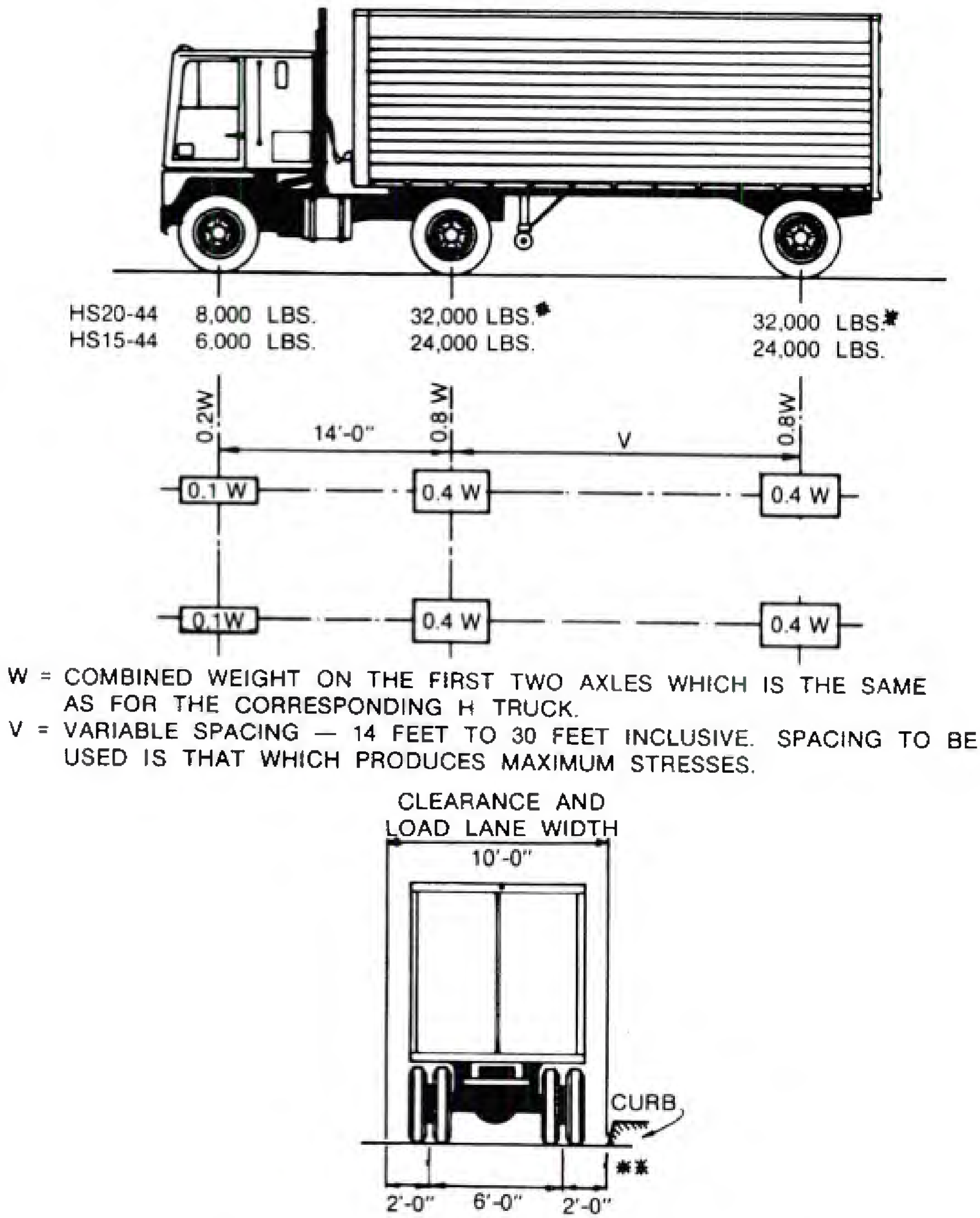



**Figure 19. Standard AASHTO HS truck loading.**

## *RESULTS AND ANALYSES*

### Overview of ILM Tests

While over 8,100 impact load tests were conducted on the Marginal Wharf including over 65,000 channels of load and deflection data, the methodology of ILM inherently evaluates a structure at discrete points. Therefore, it is not possible to cover 100 percent of a deck surface even if the entire deck is accessible, which is not the case for the Marginal Wharf. The large number of tests provides a statistical basis for determining the service load limits based on a worse case scenario of all the areas tested. We do not expect to find significantly different conditions in those areas not tested.

A broad range of responses was elicited by ILM testing. The structure acquires its rigidity primarily from the concrete. We found the less stiff ILM response to best match the finite element model as homogeneous concrete sections. The response also indicates that the concrete strength (and stiffness) was greater than the design value (3,500 psi). We suspect that the concrete strength averages 4,500 psi in most areas. The marginal wharf response for the most part was very solid. Some softness was found in the base of the platform ramps, at boundaries of deck modifications, and across transverse cracking. Loss of steel area was not noticeable because of the lack of stiffness attributed to reinforcing. Concrete cracking and deterioration correlated to the softest response from ILM.

Figure 12 shows characteristic ILM deflection shapes for midspan slab, midspan beam, and pile response for the south platform that are typical for sound, undamaged concrete. Peak deflection adjacent to the load was plotted in a surface plot for a graphic summary of pier response to vertical load. These summary surface plots are intended to isolate structural elements. Along the stringers, the midspan load response reflects the structural status of the stringer between pile bents. The midspan response on the slab primarily reveals the status of the slab. The impact load responses on the pile bents provide indications of the status of pile and pile caps. Figures 20 and 21 are surface summary plots for the deck and pile/wall support respectively of the south platform. Figures 22, 23, and 24 are similar response plots for the deck slab, stringers/pilecaps, and piles, respectively, of the wharf approach. Figures 25 through 28 are plots for the apex area of the main deck. Figures 29 through 36 are plots for the South Pier main deck and the area adjacent to the south platform ramp while Figures 37 through 44 are plots of the North Pier main deck. These provide a comparison of all areas to a peak test load of 55 kips. They are compared to like FEA model summary plots to quickly locate soft responses that would indicate loss of materials or loss of strength. The vertical axis of the summary graphic is measured peak deflection. The horizontal axes identify longitudinal test lines (not to scale) and the pile bent designation along each test line. If there were no obstacles (e.g., bollards, cleats, material, and equipment), a load test was conducted at each bent and midspan between bents along each longitudinal line. Nonvalued areas on the plots are locations where load testing was not conducted.

We did not observe excessive deflections in the pile bent areas that would indicate a pile support problem. Areas of very sound and stiff support include the 18" piles, the batter pile locations, and the rail girders along the main deck. There was larger than normal displacement in the vicinity of 42-AB and 64-AB (Figure 21) on the south platform as a result of the damaged piles at 42A and 64A respectively. The soft response of the 1954 addition of the approach was unexpected because it has a thicker deck slab (Figures 19 through 21). Some of the approach soft response can be explained by the severe deterioration of support beams at the boundary of the addition with the original approach (Line I near Bent 169) but it is mostly caused by the smaller pile cross sections. The approach is still sound for purpose for which it was designed.

Boundaries between original and the modification slabs on the approach and on the loading platform do not seem to be monolithic. There is a lack of continuity across the construction joints. For example the response is softer than normal in the vicinity of the 10-inch slab addition on the south end of the loading platform and the elevator mod (Figure 20).

Since the pier was constructed without expansion joints, load and thermal induced movement have caused severe cracking which will be followed by accelerated corrosion and localized weakening of the deck. Transverse cracks have caused some slight softening that indicates lack of continuity across the cracks. The most severe crack is between Bents 68 and 69 (Figure 8) emanating from the approach. It is a working crack that continues to grow and widen. It should be monitored regularly because corrosion of the reinforcing will accelerate and compromise the deck span.

The deck areas at the bases of the ramps also demonstrate softening and cracking due to repeated overloading (Figures 29 and 37).

Load response indicates the cementitious material used to fill the rail slots has separated from the original concrete. Analysis also shows there are cracks running parallel to the slots through the deck adjacent to the slot.

Although the ILM tests exhibited a wide scatter of response, displacement magnitudes remain below what would be expected from an impaired structure. The test responses over the pilecaps for the most part were very consistent. This is interpreted as a demonstration of pile soundness.

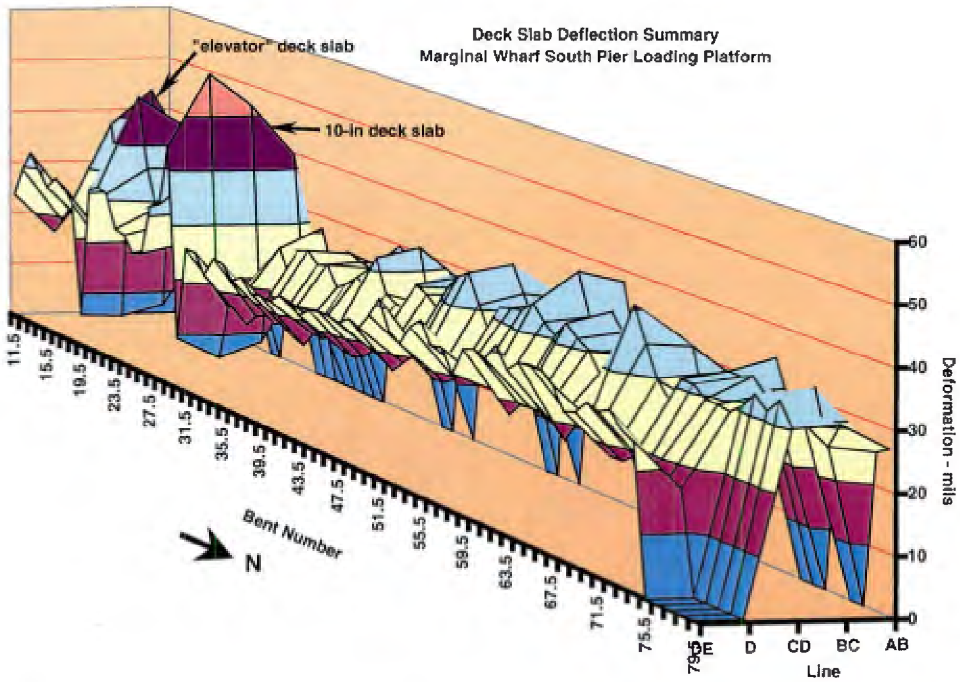



Figure 20. Summary of ILM maximum deflection response. Deck and stringers of south platform.

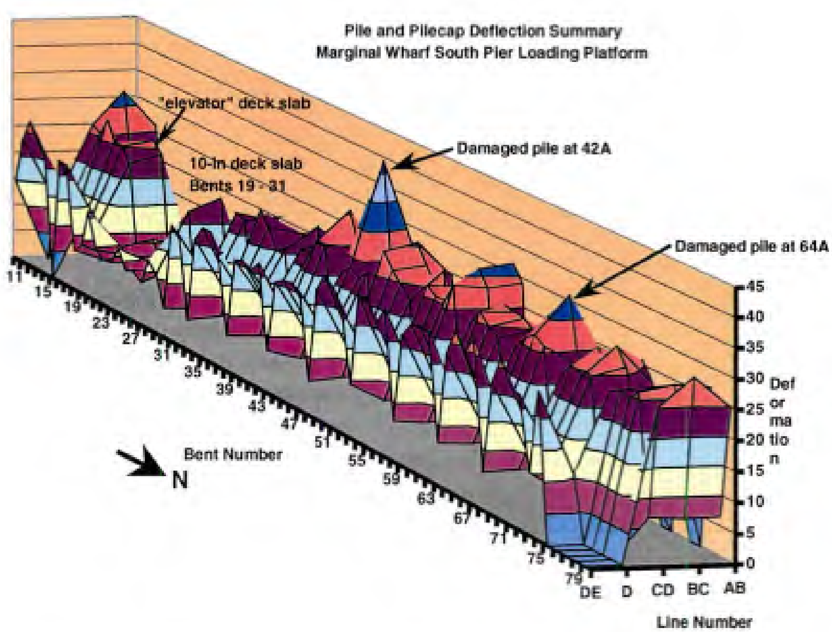



Figure 21. ILM maximum deflection response summary. Piles and 8-inch walls of south platform.

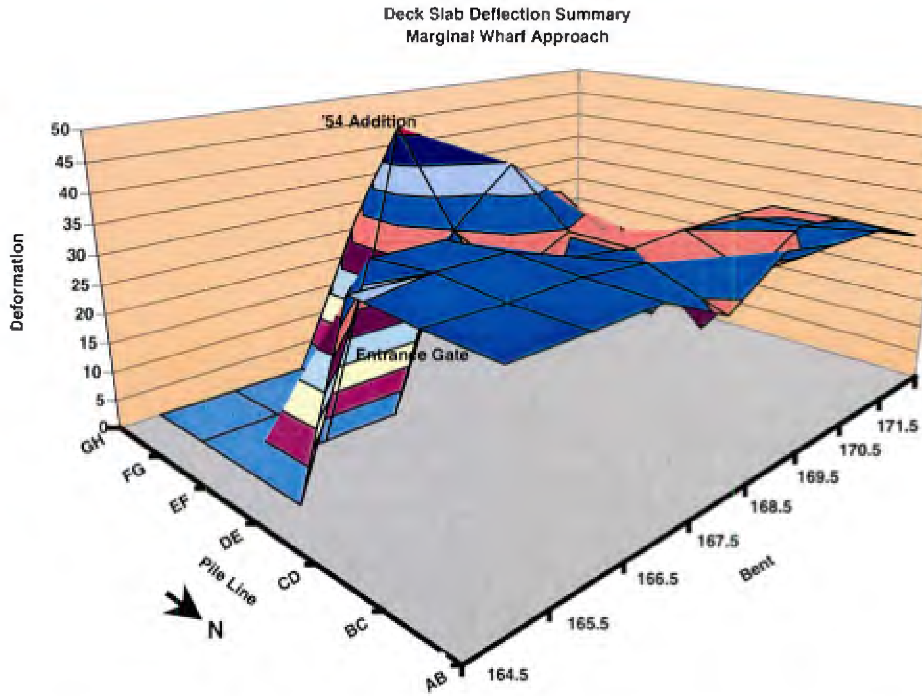



**Figure 22. ILM maximum deflection response summary. Approach deck slab.**

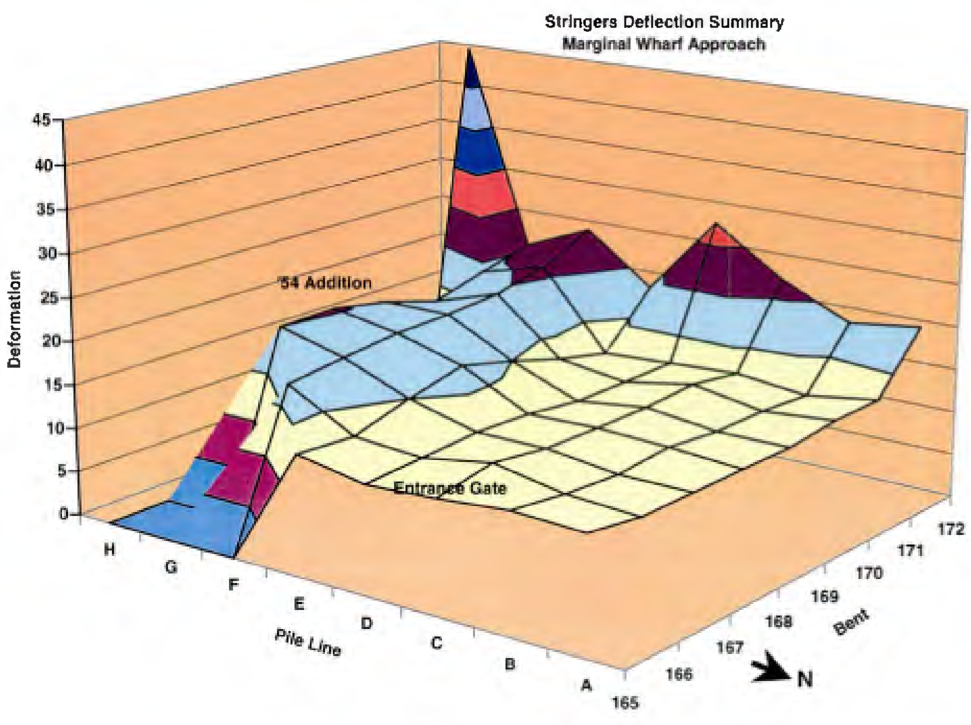



**Figure 23. ILM maximum deflection response summary. Stringers/pilecaps of wharf approach.**

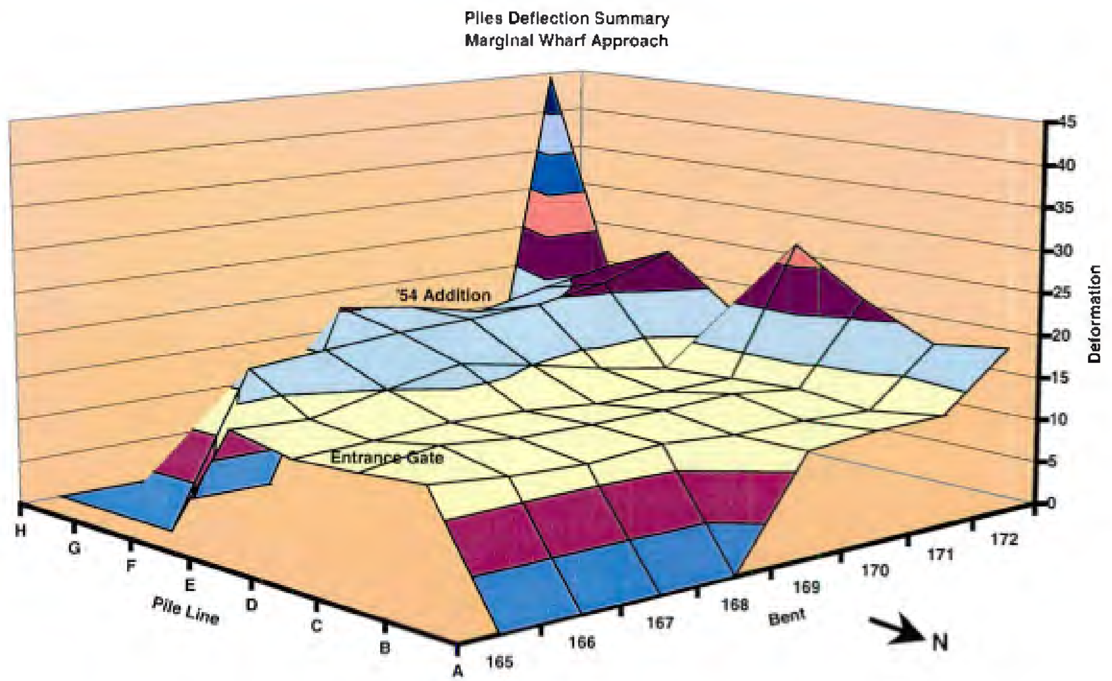



Figure 24. ILM maximum deflection response summary. Piles of wharf approach.

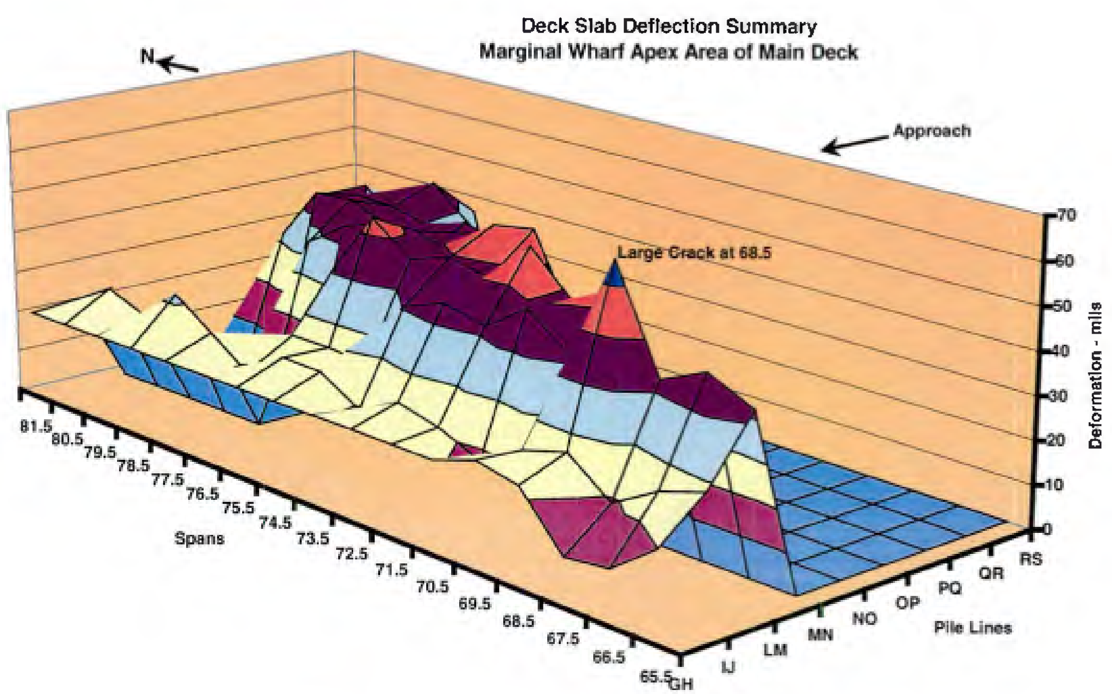



Figure 25. ILM maximum deflection response summary. Deck slab of wharf apex area.

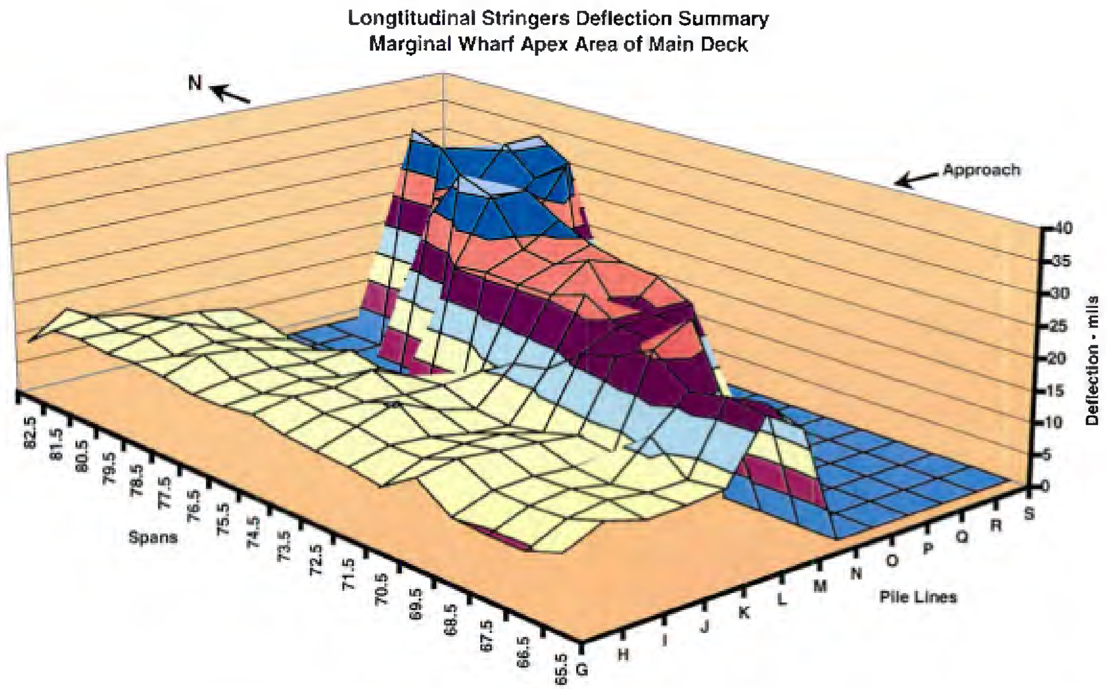



Figure 26. ILM maximum deflection response summary. Longitudinal stringers of wharf apex area.

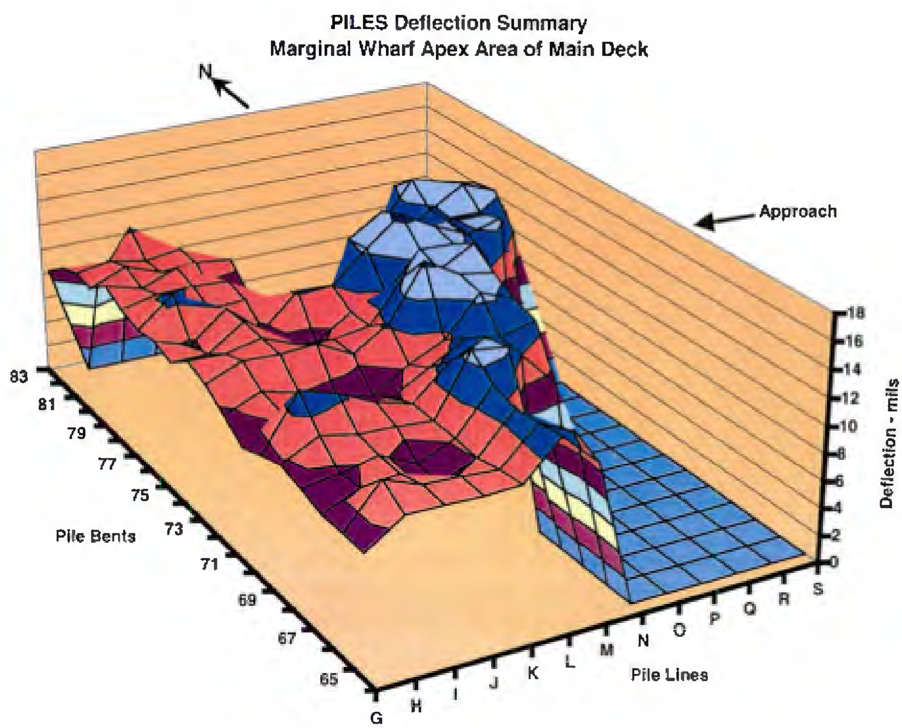



Figure 27. ILM maximum deflection response summary. Piles of wharf apex area.

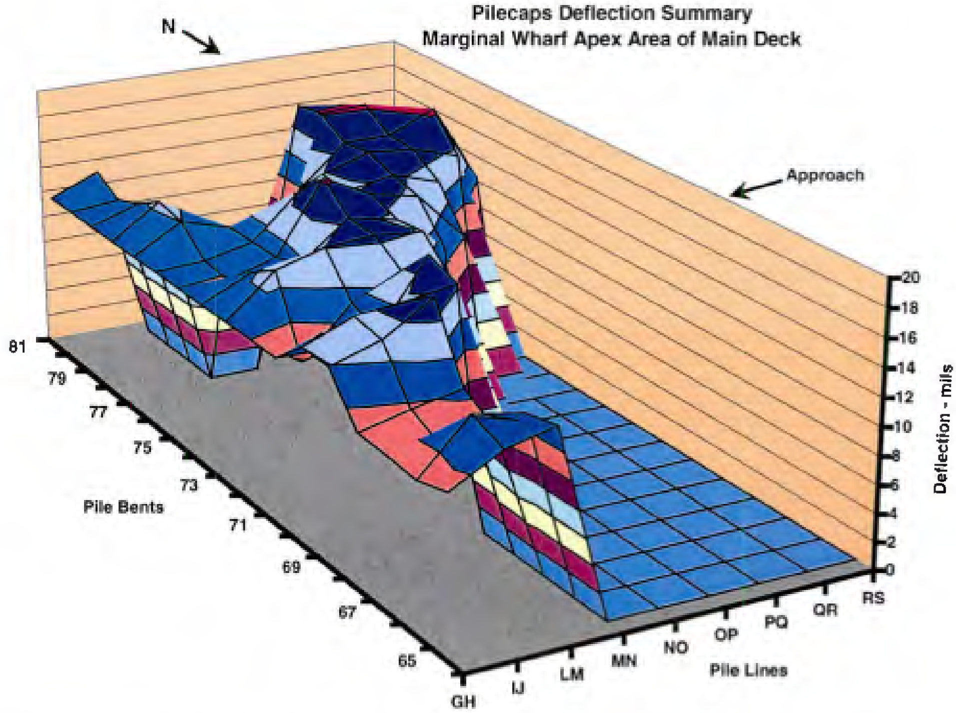



Figure 28. ILM maximum deflection response summary. Pilecaps of wharf apex area.

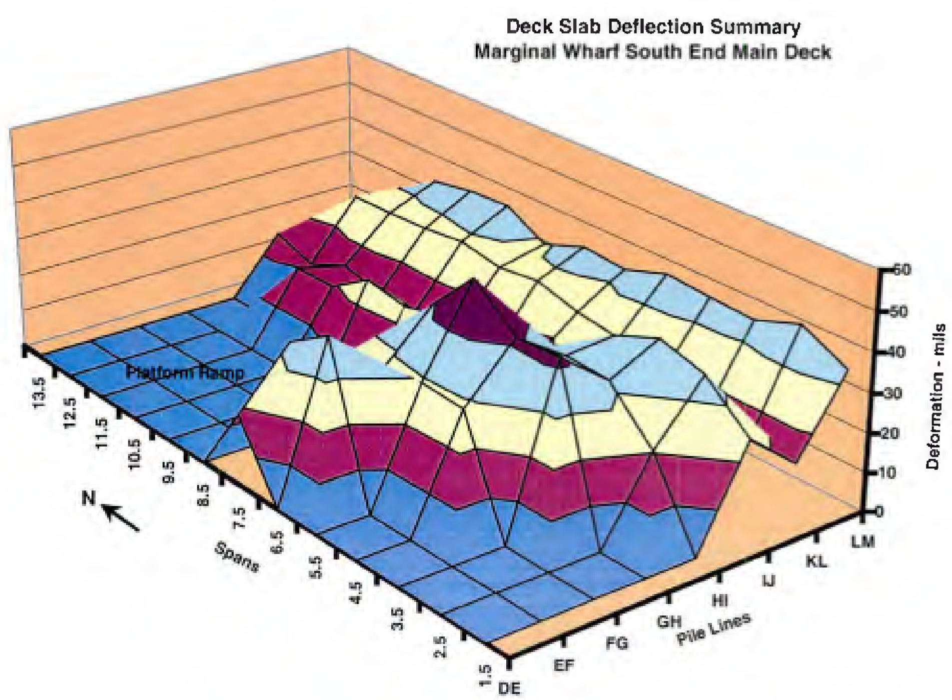



Figure 29. ILM maximum deflection response summary. Deck slab of south end south pier main deck.

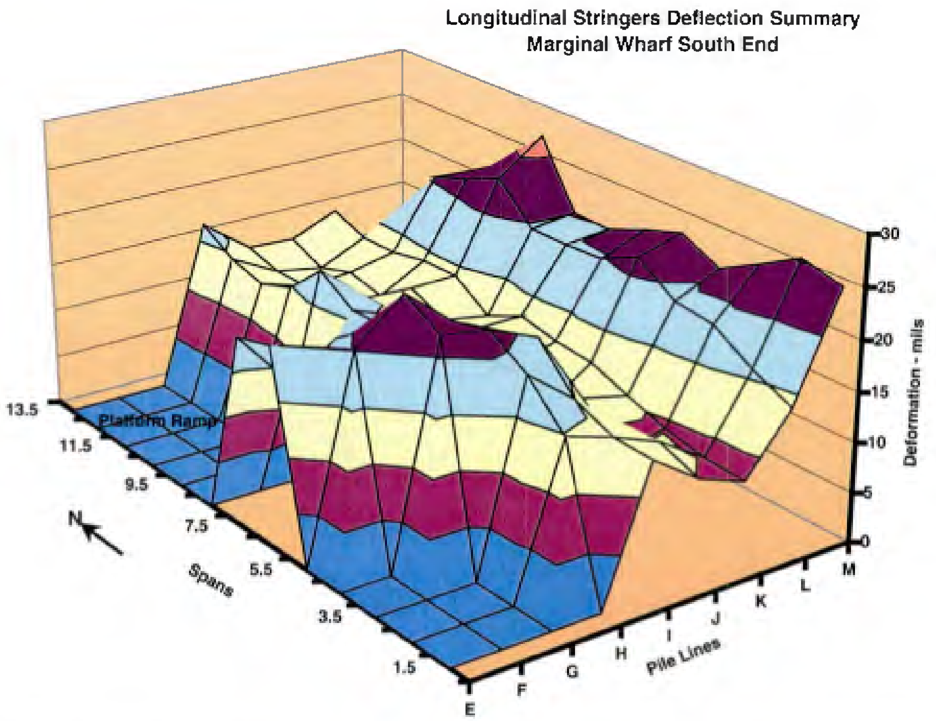



Figure 30. ILM maximum deflection response summary. Longitudinal stringers of south end south pier main deck.

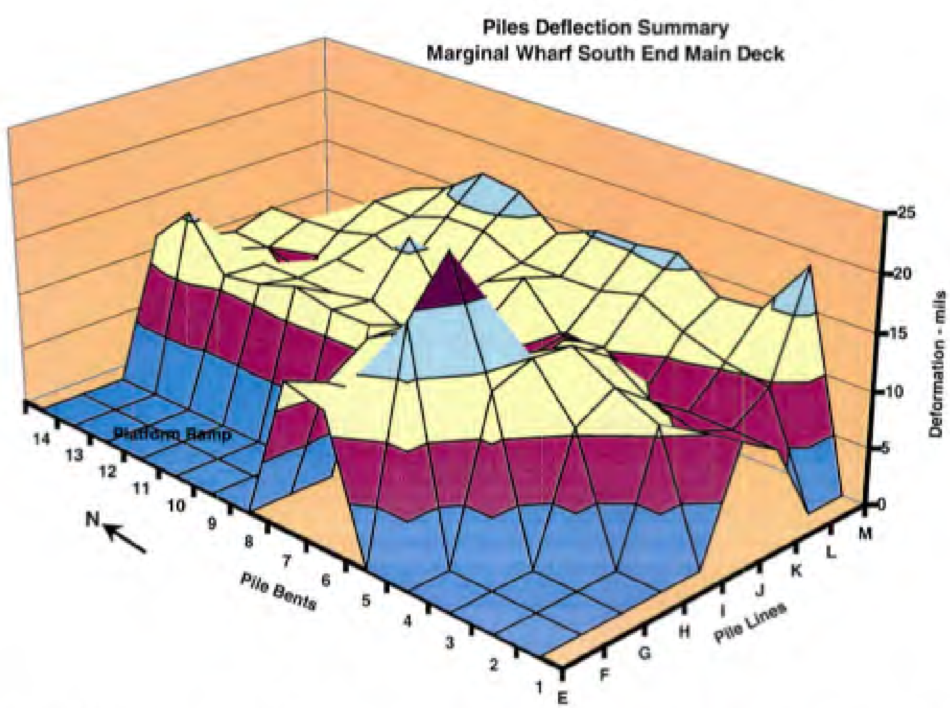



Figure 31. ILM maximum deflection response summary. Piles of south end south pier main deck.

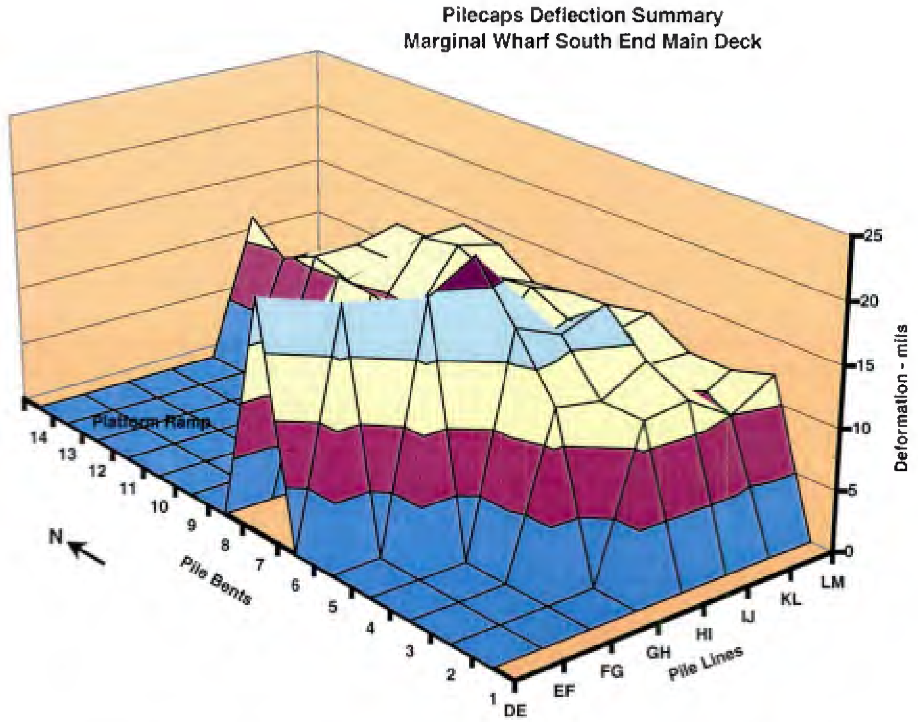



Figure 32. ILM maximum deflection response summary. Pilecaps of south end south pier main deck.

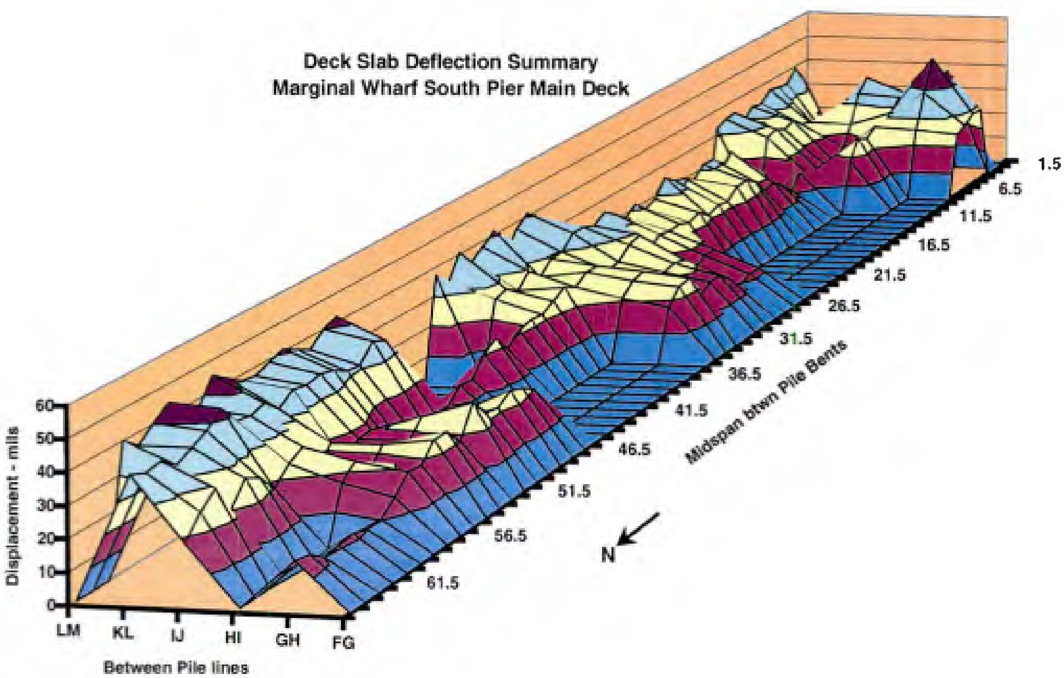



Figure 33. ILM maximum deflection response summary. Deck slab of south pier main deck.

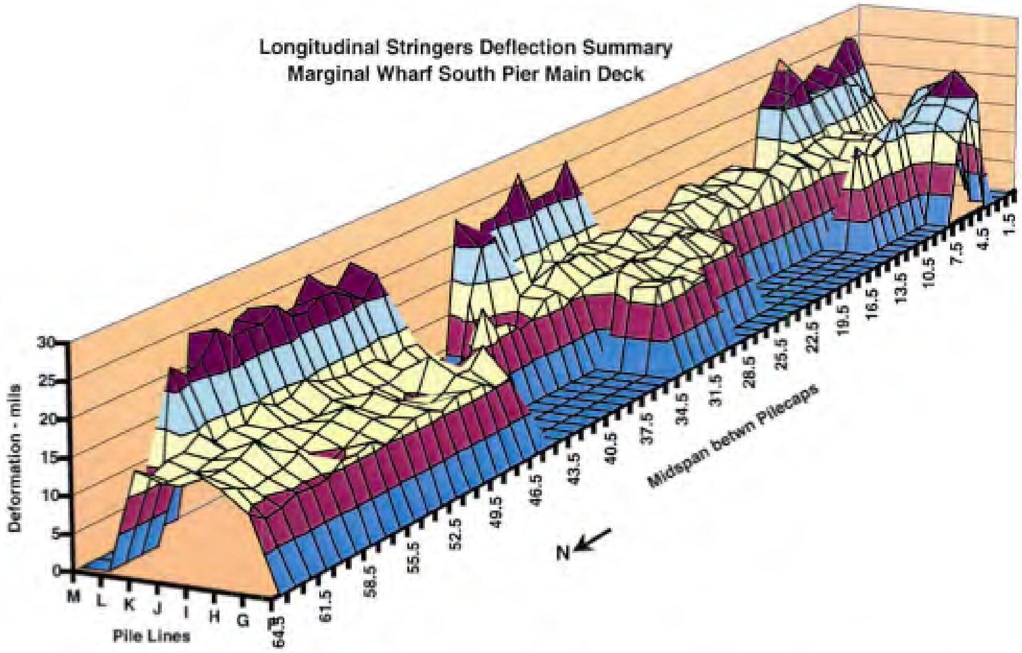



Figure 34. ILM maximum deflection response summary. Stringers of south pier main deck.

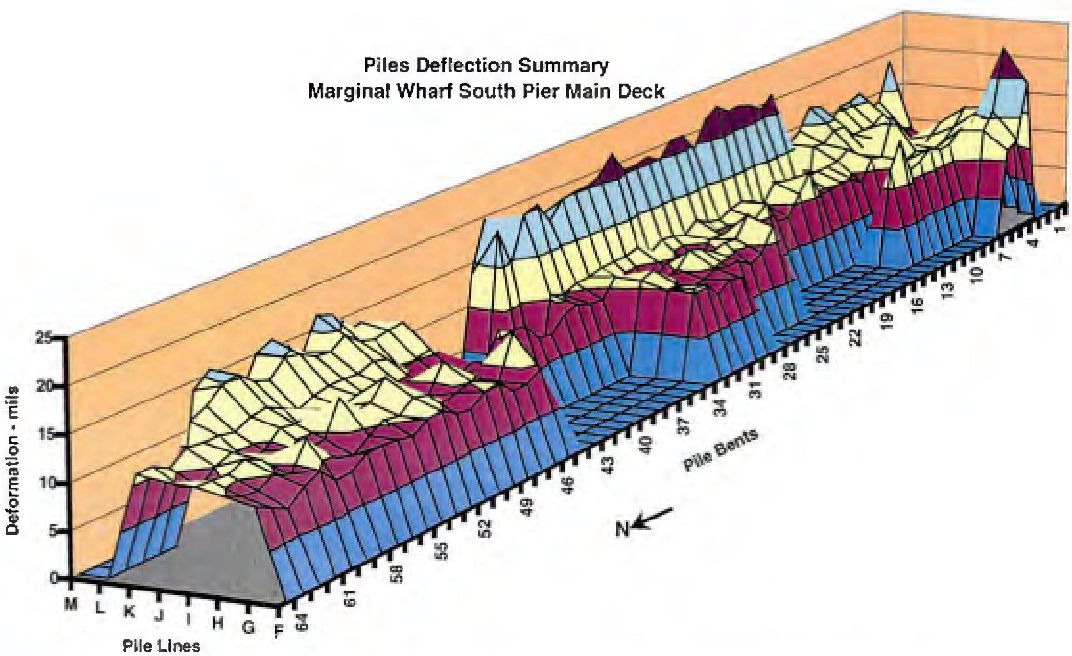



Figure 35. ILM maximum deflection response summary. Piles of south pier main deck.

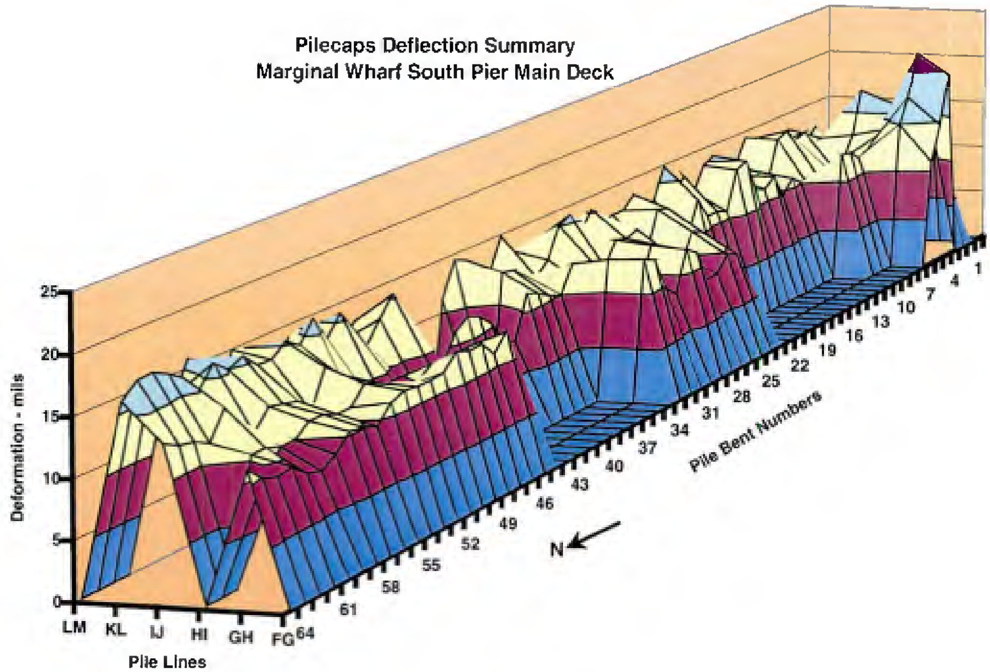



Figure 36. ILM maximum deflection response summary. Pilecaps of south pier main deck.

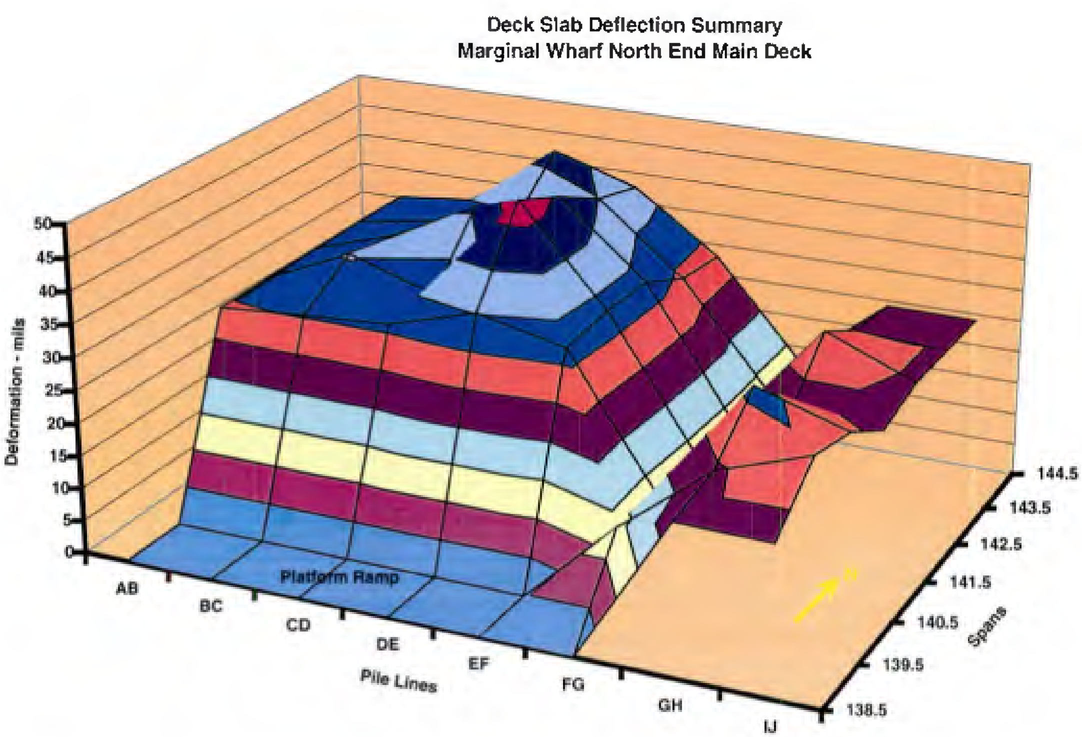



Figure 37. ILM maximum deflection response summary. Deck slabs of wharf north end.

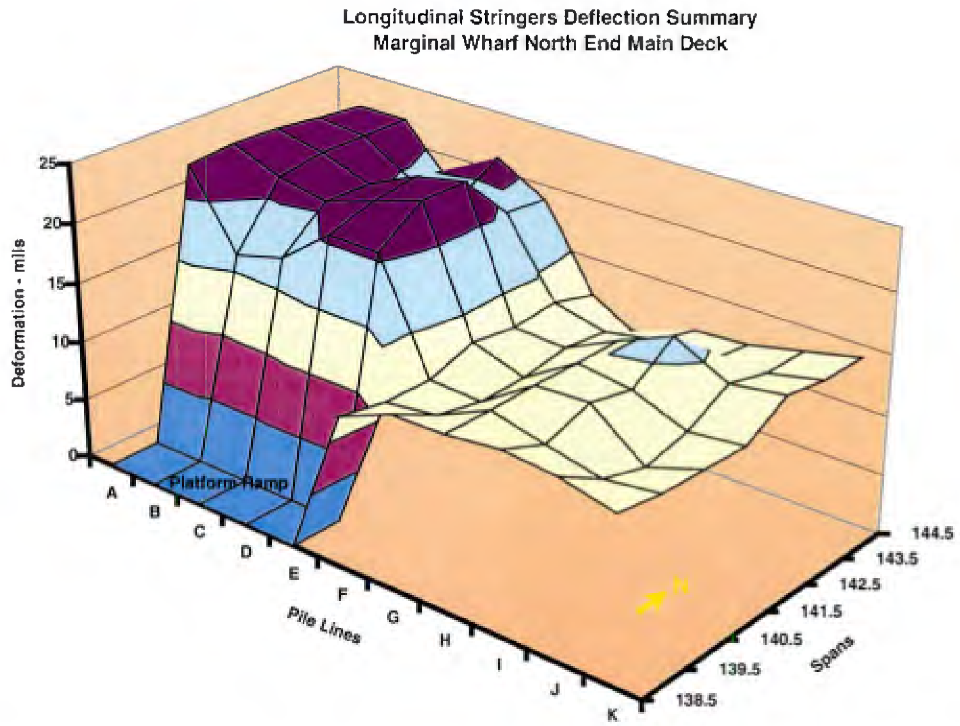



Figure 38. ILM maximum deflection response summary. Stringers of wharf north end.

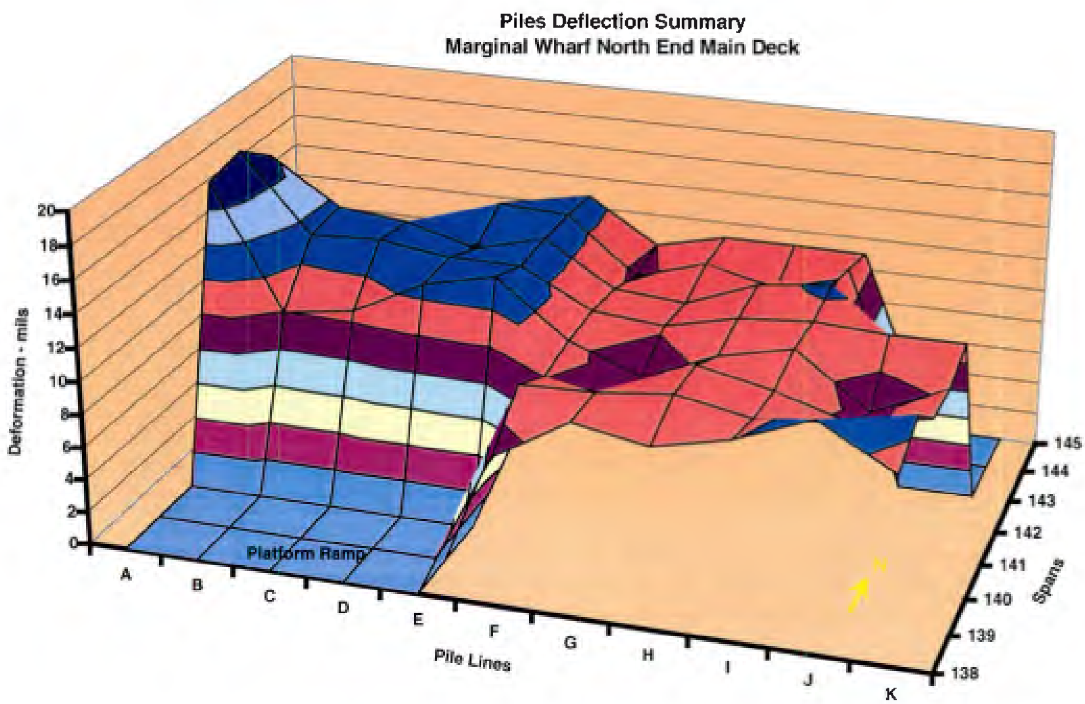



Figure 39. ILM maximum deflection response summary. Piles of wharf north end.

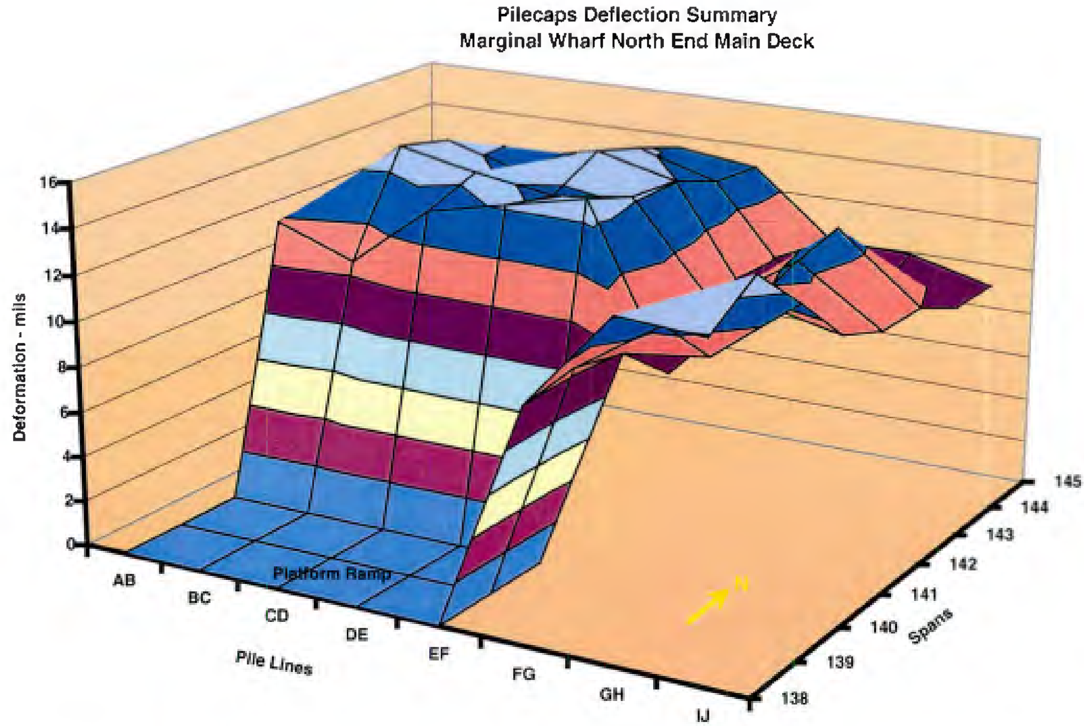



Figure 40. ILM maximum deflection response summary. Pilecaps of wharf north end.

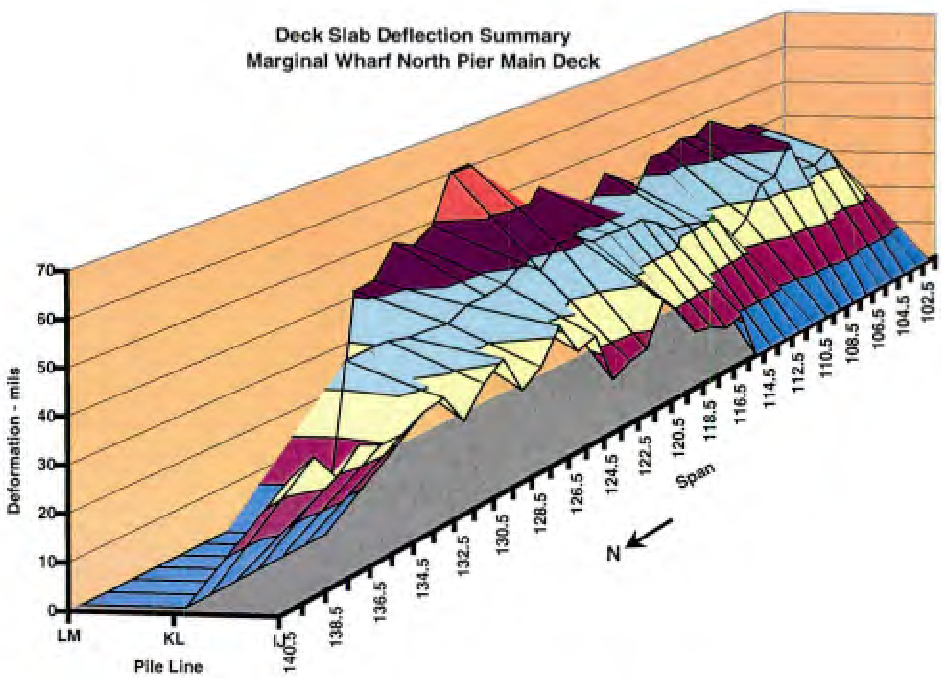



Figure 41. ILM maximum deflection response summary. Deck slabs of north pier.

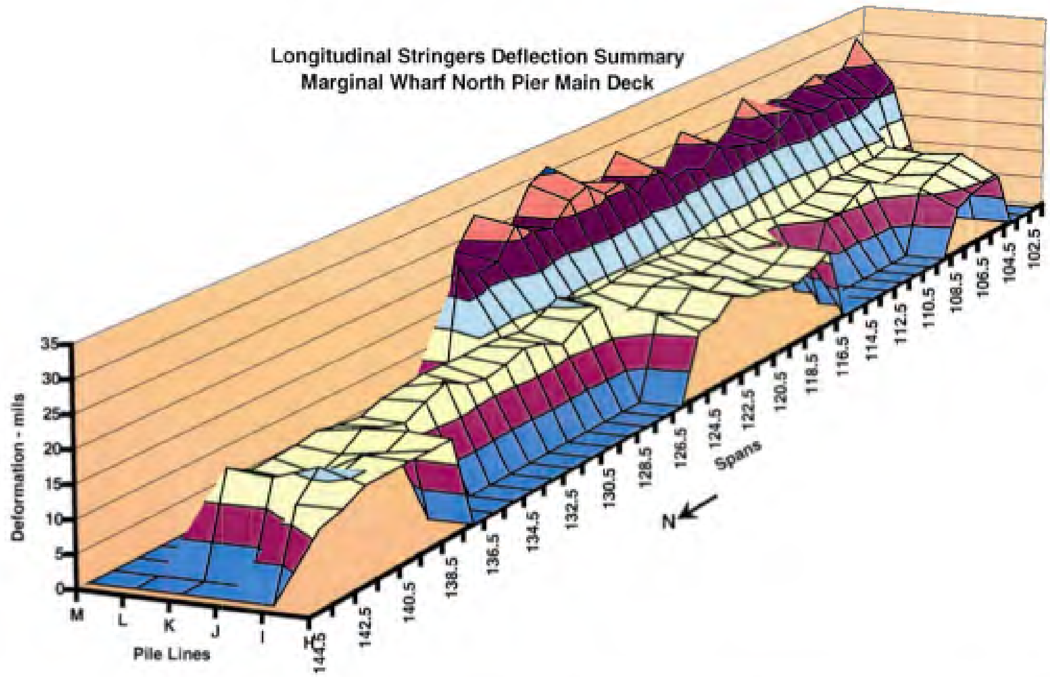



Figure 42. ILM maximum deflection response summary. Longitudinal stringers of north pier.

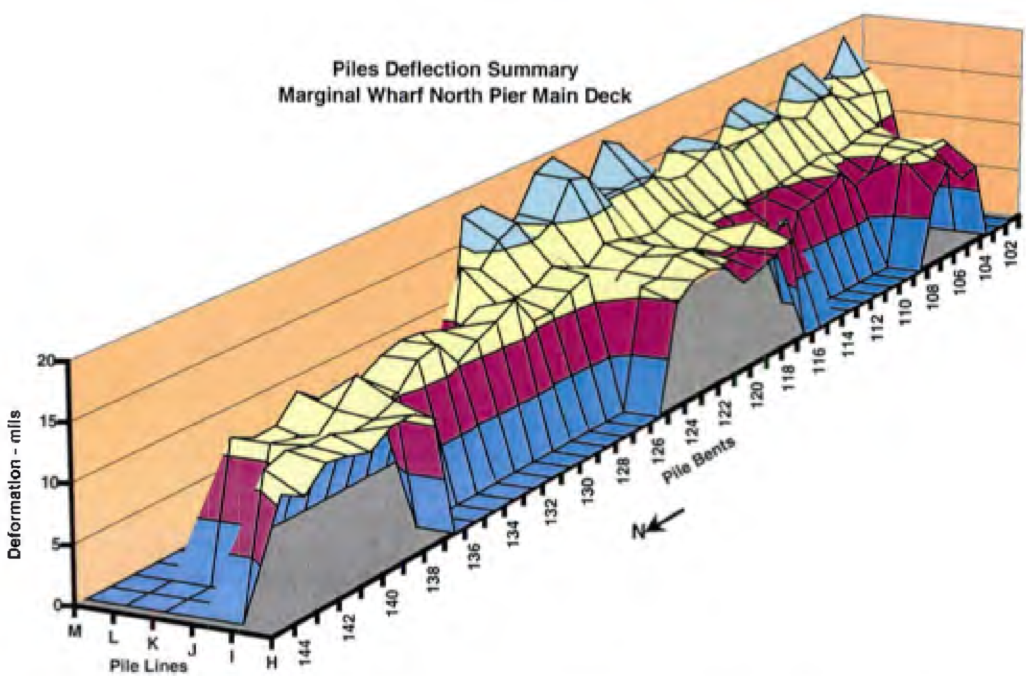



Figure 43. ILM maximum deflection response summary. Piles of wharf north pier.

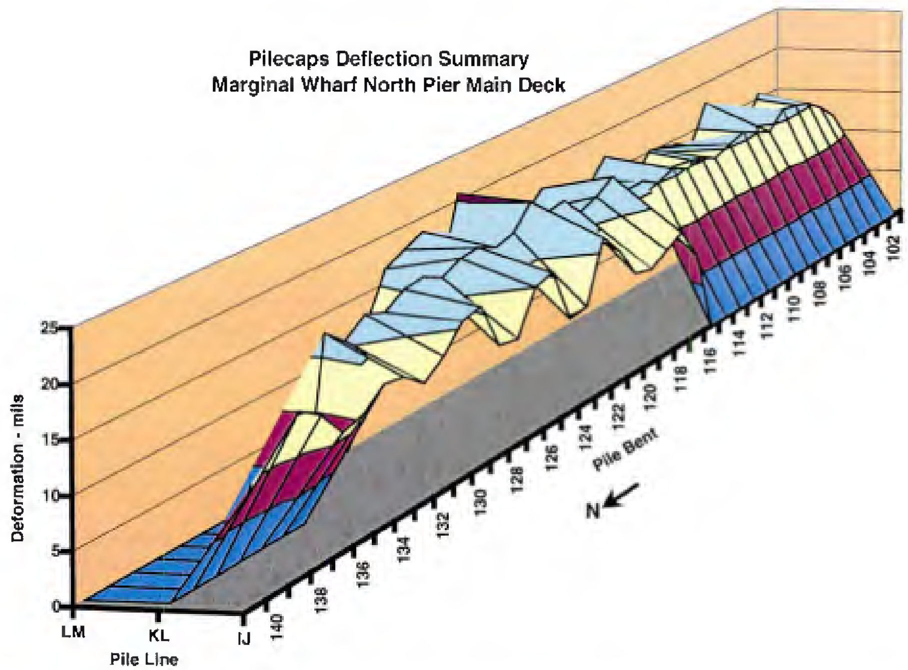



**Figure 44. ILM maximum deflection response summary. Pilecaps of wharf north pier.**

## FEA Results Summary

To verify and validate the finite element models, the FEA response of ILM loading was equated to the ILM test results. Representative sections were checked such as spans 120 to 125 of the north pier, which were analyzed using the models shown in Figures 14 and 16. Summary deflection plots of sections were generated from both FEA and ILM (e.g. Figures 45 and 46) to determine if the FEA was accurately representing the pier response. On the south platform the model stringer, pilecap, and pile response matched reasonably with the test response; however, the deck slabs were stiffer than that predicted by initial models. The material and geometric properties of the basic finite element model were adjusted to reflect measured ILM response. For example, adjustments were required in the deck elements to reflect additional concrete strength.

Applying the validated FEA model we determined critical locations to measure maximum flexural (positive and negative), shear, axial and torsional response of the deck, pilecaps, stringers and piles. All the vehicles (cranes and trucks) have at least four load points at all times (outriggers or wheels) with variable spacing between them. The vehicles could be positioned to represent arbitrary directions of travel. We varied the outrigger spacing from 20 to 25 feet. We made simplifications to the analytical approach based on the geometry of the pier and the structural element properties in order to focus on the critical loading. Whenever geometrically possible, the general approach followed the course of placing maximum loads at midspan and secondary loads on neighboring spans.

FEA Piles Deflection Summary
North Pier Main Deck Section

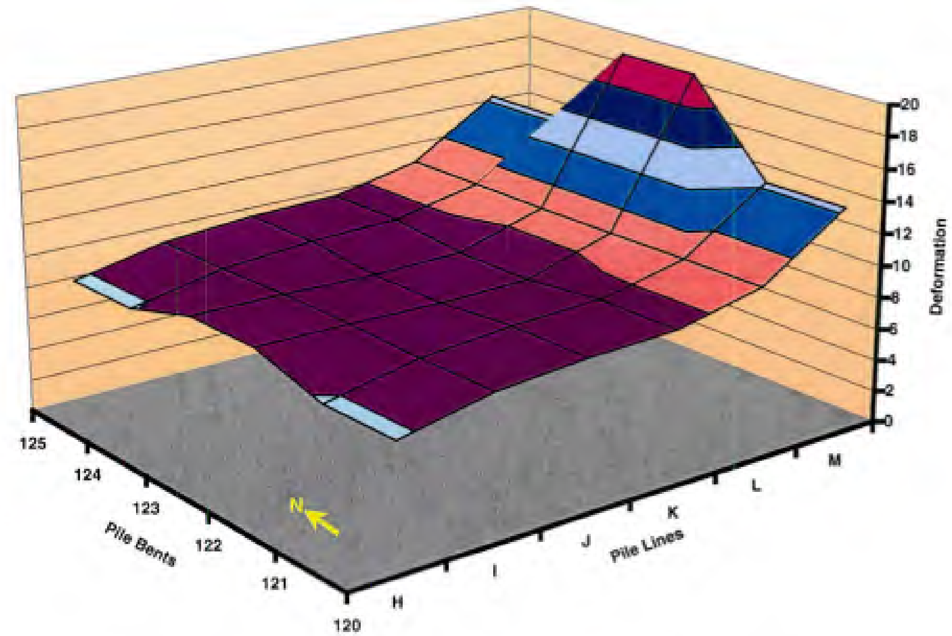



**Figure 45. Peak FEA model piles response of north pier section.**

ILM Piles Deflection Summary
Marginal Wharf North Pier Main Deck Section

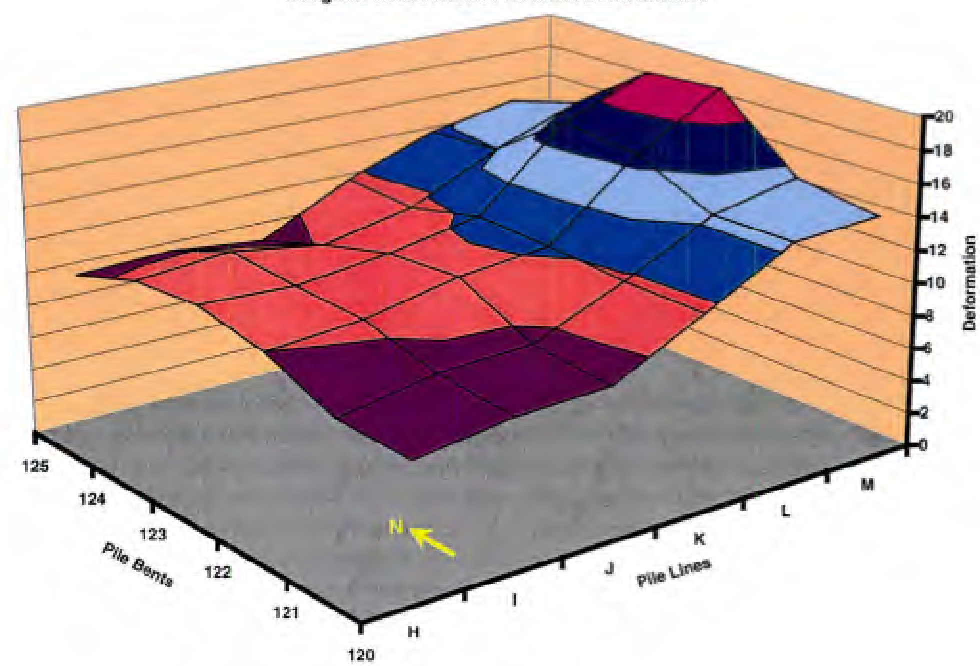



**Figure 46. Peak ILM response of piles of north pier section.**

The post processors of the FEA programs generate graphics to assist the analyst in visualizing the deflection patterns and the distribution of loads into the decks. These include graphics for specific states of plate stress (e.g. X normal, Y normal, ZX shear, etc.) as well as beam forces (e.g. axial forces, moments, torques, shear forces, etc.). They are generated with color contours painted onto the model surface that is deformed in response to the load case under analysis. Figures 47 and 48 are included in this report for demonstration. Figure 47 is an example of the deck's strong axis stress response to a Grove TM890 crane positioned in the middle of the loading platform with the heavier rear axles located on the left. Figure 48 is a similar response to a 100-kip patch load positioned over the centroid of a pile on Line E of the south platform.

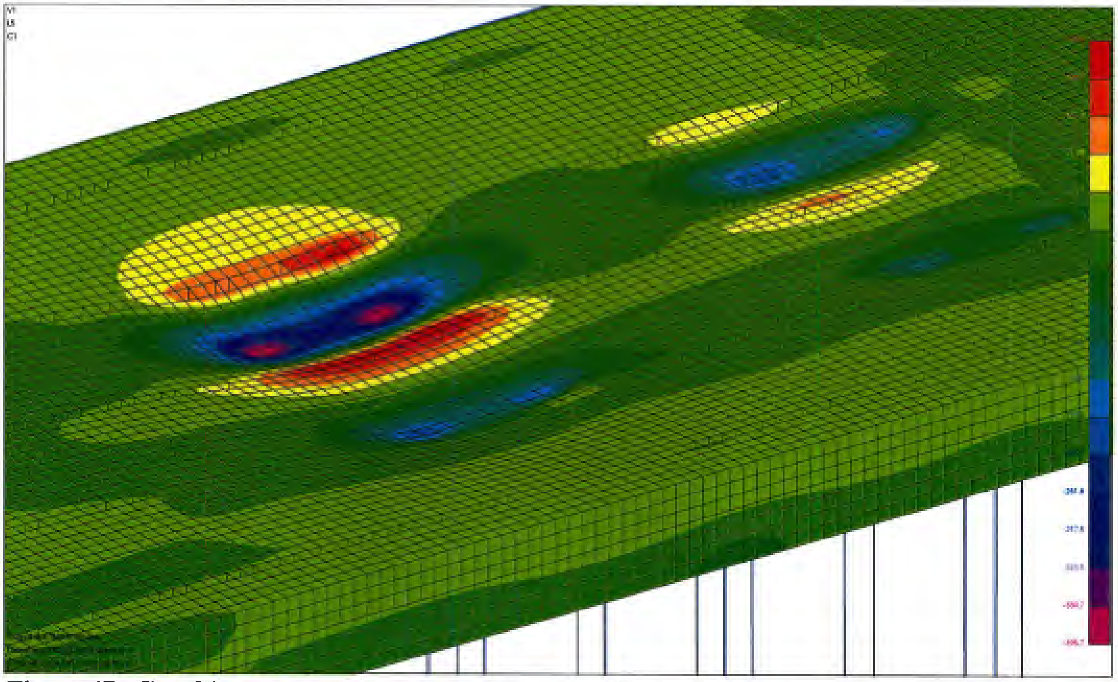


**Figure 47. Graphic representing plate bending stress from wheel loads of Grove TM890 traveling left to right on the south platform.**

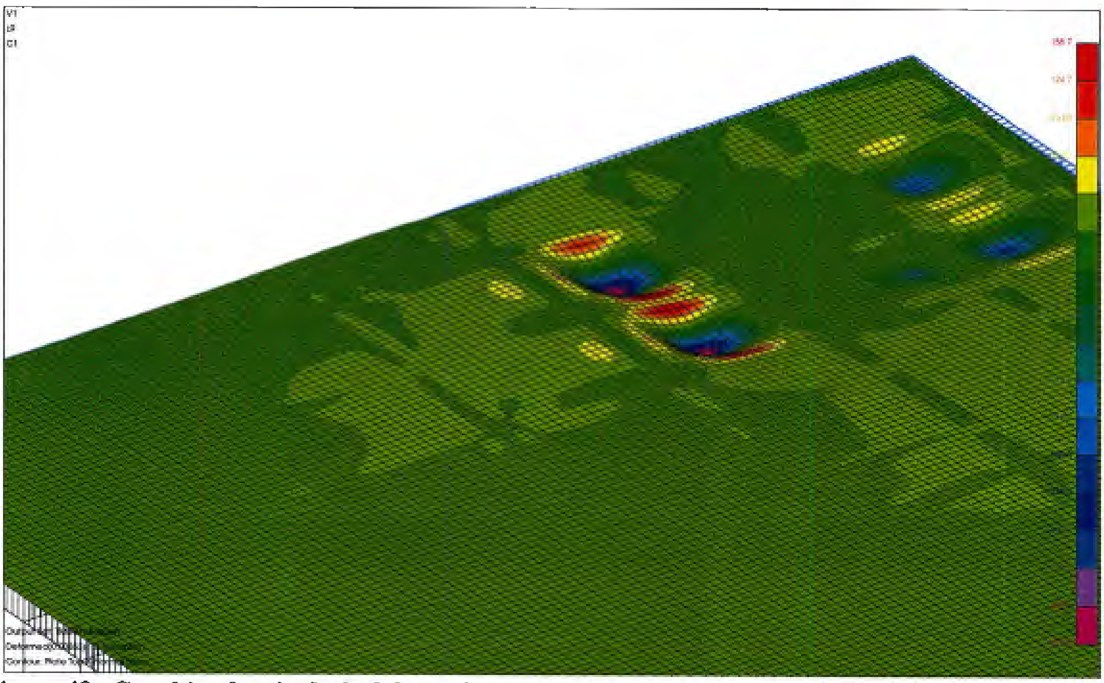


**Figure 48. Graphic of main deck slab maximum bending stress from RT890 crane straddling Line L.**

The deflection shapes depicted in the graphics are accurate, albeit exaggerated representations of the responses as they are revealed on the deck surface. Each color contour represents a stress level in an equivalent homogeneous plate. Contours are used to help to visualize the distribution of load into the model, to locate maximum and minimum values, and to see details of the deflected shape. The stress value (in psi) for each color contour appears with a color code on the right of the graphic, but peak stress values are not listed. Peak values are taken from tabulated element data instead. We limited responses to within the elastic limits of reinforced concrete but beyond the concrete's tensile cracking limits. Thus, specific values of stress do not translate directly to concrete or reinforcing steel stress. Rather, the analyst translates the finite element plate biaxial stress to elastic, cracked section, biaxial moments and shear. These are compared to the ACI-limiting reinforced concrete moment and shear.

The graphics of Figures 47 and 48 demonstrate the confinement of the patch load response to the deck panel between pilecaps and stringers where the load is applied. This is primarily due to the flexural, torsional and axial stiffness of the pilecaps, longitudinal stringers, and piles. The FEA models also clearly show the wharf deck responds to a patch load with two-way biaxial bending. A patch load at the center of a deck panel produces X- and Y- axis (strong and weak axis, respectively) moments of comparable magnitude at the point of load. The weak axis moment magnitude is approximately 2/3 of the strong axis moment. Conversely, the wharf deck slabs were constructed with one-way flexural reinforcing arranged perpendicular to the longitudinal girders and stringers. The lack of reinforcing is the largest load limiting determinant of the wharf.

**Wheel Loads.** The wharf decks possess better load distribution characteristics than allowed by AASHTO wheel load distribution coefficients used in traditional analyses. However, the AASHTO coefficients allow for the probability of two trucks occupying adjacent lanes (side by side) in the same span. This configuration could not be tolerated on the marginal wharf. Cranes and heavy (AASHTO-size) trucks should be well separated while operating on the marginal wharf. The four-wheel mobile cranes apply more intensive wheel loads to the pier deck than AASHTO truck wheel loads. The piles, pilecaps, and stringers are sufficient to carry all the crane and AASHTO truck wheel loading. The P&H CN150, the Grove RT880, and the Grove TM890 produce flexural responses that exceed the ACI limits of the main deck slab. The CN150 and the RT880 only slightly exceed limits. Their application should be allowed but with speed restrictions to avoid impact loads. The deck slab can be upgraded to remove all restrictions on the CN150 and TM890, but the Grove RT880 exceeds ACI limits of all slab configurations. The deck does not have sufficient depth to support the RT880.

**Outrigger loads.** Patch loads applied directly above a pile (or vertical pile/batter pile combo) or to a pilecap will be distributed into immediate neighbor piles through the pilecaps and stringers (Figure 43). The effects of a patch load over the midspan of a girder are confined primarily to the girder, adjacent deck slabs, piles, pile caps and stringers within 15 feet in orthogonal directions. The effects of a patch load applied to the middle of a deck slab panel are confined primarily to the panel and the bounding beams and piles. This means we will find little response at the midspan of the rail girder due to the other three outriggers placed on elements outside adjacent spans. Further, we determined early in the analysis that the deck slabs were not able to support outrigger loads larger than 50 kips without exceeding flexural capacity. Therefore, the analyses concentrated on outrigger loads positioned along stringers and pilecaps.

## *RECOMMENDATIONS FOR SERVICE LOAD RESTRICTIONS*

The load limit recommendations presented in this report are in the best judgement of NFESC engineers and are based on the test load and past operational performance of the structure. The recommendations weigh the critical role the Marginal Wharf performs in support of TRIREFFAC mission. The recommended loads will exceed those allowed by linear, one-dimensional, ACI/ASHTO design-based analyses which engage material and load factors, design coefficients, and simplifying performance assumptions. In short, these recommendations best represent the structural limitations of the Marginal Wharf. Loads in excess of those recommended should not be applied without detailed analyses.

The evaluation of pier readiness and its adequacy to support wheel and outrigger loads is determined by comparing the load effect to the available structural resistance. Basically, the resistance of the pier must equal or exceed the load demands placed on it. The studies by Johnson Controls accurately determined the resistance of all elements of the Marginal Wharf using ACI 318 methodology. We compared calculated resistance (Table 1) to FEA model response to specific loads. The most striking, built-in features that result in restrictions of the wharf are:

1. Small amounts of steel reinforcing (many members do not meet minimum ACI requirements).
2. The shortage of biaxial reinforcing in the deck slab panels
3. The absence of expansion joints which caused transverse cracking through the deck.

The piers were originally designed to the ACI Working Stress Design methodology that is supposed to restrict reinforced concrete to respond in a linear range. Concrete stress was limited to $0.45f_c'$ and ordinary reinforcing steel stress was limited to 20,000 psi. Whereas the stiffness of the structure is controlled primarily by the concrete strength, the reinforcing steel quantity and strength limits the flexural resistance and the flexural resistance governs the load resistance of the Marginal Wharf system because the elements are greatly under reinforced. The ultimate flexural resistance of an under-reinforced concrete element can be expressed by:

$$M_u = A_sF_y\,(d - a/2)$$

Where $A_sF_y$ is the maximum tensile force in the reinforcing (not considering strain hardening), d is the effective depth, a is depth of the compression zone in the concrete, and (d - a/2) is the internal moment arm. Due to small amounts of flexural reinforcing, an increase of concrete strength from 3500 psi to 4500 psi only increases the flexural resistance slightly by changing the internal moment arm (d – a/2). For example, the moment capacity of 12-inch by 22-inch longitudinal stringers in the main deck is 1178 in-kips for concrete strength of 3500 psi and 1188 in-kip for 4500 psi. For those structural members that do not meet the ACI reinforcing minimums, the difference is even less.

Limiting the material stresses to the linear ranges insures a large safety factor against failure and, more conservatively, does not account for the redundancy. FEA is an elastic analysis but it does account for the continuity that is built into the structure. We consider FEA to be an accurate representation of the pier structures for analyzing service load restrictions where the response of the reinforced concrete is restricted to the linear range.

Our analyses provided the load limits based upon ACI-set stress limits on the constituent materials of the reinforced concrete. Our analyses cannot determine damage that will occur with each overload cycle or the remaining life in the structure with continuous overload. Determining the number of cycles to levels of damage is also complicated by the lack of knowledge of past load history. NFESC studies have shown concrete that has been strained more the half its ACI limit strain of 0.003 in compression will "soften" and deteriorate with each load cycle. The relationship of cyclic overstrain to damage is unknown for the wharf since it is a function of strain level. Our laboratory tests on flexural members (piles) have shown fewer than 100 cycles at 70 - 80 percent of the concrete limiting strain results in over a 50 percent loss of load carrying capacity in less than 100 cycles. The loss was characterized by unrecoverable damage to the concrete such as spalling and splitting. Over stressing the reinforcing steel will cause cracking that will continue to widen and grow with each overload cycle. The "working" cracks will lead to steel loss due to accelerated corrosion, which will acerbate the lightly reinforced condition that was constructed into the pier. The life-cycle for this scenario cannot be predicted because it depends on too many unknowns such as corrosion rate. However we expect an additional 20 years of service life if overloads are prohibited.

Recommended patch load limitations are tabulated in Table 2. Recommended uniform loads are graphed in Figure 49. A general restriction should be enforced on all original deck slabs wherein the maximum single wheel load does not exceed 20 kips for wheel spans greater than 10 feet and axle widths greater than 8 feet. Tandem axle (axles space of 54 inches) wheel loads should not exceed 16 kips (32 kip axle load). Crane speed limits should be set at 5 MPH while on the decks to minimize dynamic loads. The Grove RT880 wheel loads exceed ACI structural capacity of the deck slabs and should not be operated on the Marginal Wharf. Cranes and heavy trucks should be separated by a distance such that wheels from two vehicles are not allowed to occupy space in the same deck panel (between adjacent pilecaps and between adjacent longitudinal girders). We recommend that cranes and other heavy vehicles be separated by at least 30 feet on the Marginal Wharf.

## WHARF APPROACH

Uniform load on the approach should be restricted to 550 psf. Single wheel loads should not exceed 20 kips and dual axle wheel load should not exceed 16 kips (32 kips per axle). The Grove RT65S and the Lorain RT300D should be allowed to traverse the decks without restrictions. The P&H CN150 and the Grove TM890 wheel loads slightly exceed ACI concrete and reinforcing limits in weak direction of the deck panels. The latter two cranes are allowable with speed restrictions.

## MAIN DECK

Uniform load on the main deck should be restricted to 550 psf from line K to the inboard curb and 800 psf elsewhere. Patch loads on the deck slab should not exceed 20 kips for wheels and 24-inch and smaller outrigger pads and 35 kips for outriggers placed on 48-inch square cribbing that is compliant with the deck. The Grove RT65S and the Lorain RT300D should be allowed to traverse the decks without restrictions. The P&H CN150 and the Grove TM890 wheel loads exceed ACI concrete and reinforcing limits on the main deck in the vicinity of the rail girders and slightly exceed reinforcing limits in the weak direction of all other concrete deck panels. The latter two cranes can traverse the deck but should be restricted to direct paths to the loading platforms at a restricted speed limit. Crack growth should be monitored along the crane path. The Grove RT880 wheel loads exceed ACI structural capacity of the deck slabs.

The stringers, rail girders and pile caps can support the wheel loads of all the above cranes as well as AASHTO HS20 truck wheels. Outrigger loads placed along longitudinal stringers should not exceed 70 kips (80 kips with cribbing). Outrigger load limitations placed on piles, pilecaps, and rail girders range from 100 to 120 kips (120 to 140 kips on cribbing). Although not listed in Table 2, outriggers placed on pilecaps between adjacent rail girders spaced 5 feet apart (girders on lines F and G, H and I, and J and K) may be as large as 140 kips with or without cribbing. If cribbing is used to bridge across two adjacent rail girders, outriggers load may be as large as 200 kips. An outrigger load place on a pile cap at the location of both vertical and batter piles may be as large as 140 kips with or without cribbing.

## PLATFORMS

The 80-ton, RT880 crane exceeds the ACI flexural capacity in the transverse direction of the deck slabs between the supporting 8-inch walls including the 10-inch deck mod. Outriggers on the original deck slab should not exceed 25 kips for a 24-inch square outrigger and 35 kips for an outrigger placed on 48-inch square cribbing that is compliant with the deck. Outrigger load limits on the modified, 10-inch deck are slightly higher. Outrigger loads placed on top of the 8-inch walls supporting the deck can not exceed 110 kips (120 kips with cribbing). Maximum outrigger loads of 100 kips can be placed over piles on the loading platform except piles 42A and 64A. Piles 42A and 64A should be repaired before placing outriggers over them. The centroid of the float should be placed within 1 foot of centroid of the pile. The outrigger that supports the greatest load (the one the load is rotated over) should always be positioned over an outside pile on Line A or Line E. Maximum outrigger loads of 70 kips can be placed along the longitudinal stringers on Line A and Line E of the south platform.

We recommend aluminum cribbing replace the wood cribbing now in use. An example aluminum cribbing plate is shown in Figure 50. Steel cribbing proposed by Johnson Controls in August 1992 is also satisfactory. Cribbing should be considered for all outrigger loads in excess of 100 kips.

Table 2. Allowable Loading on Marginal Wharf Structural Elements

| Element | Maximum Single Wheel Load (kips) | Maximum Dual Axle Wheel Load (kips) | Maximum Outrigger load (kips) | Maximum Outrigger Load With cribbing (1) (kips) | NOTES |
|---|---|---|---|---|---|
| Main Deck | | | | | |
| Deck | 20 | 16 | 25 | 35 | |
| Stringer | | | 70 | 80 | C.g. of outrigger must be positioned within +/-6 inches of stringer C.L. |
| Rail Girder | | | 120 | 140 | C.g. of outrigger must be positioned within +/-12 inches of girder C.L. |
| Pilecaps | | | 100 | 120 | C.g. of outrigger must be positioned within +/-12 inches of pilecap C.L. |
| Pile | | | 100 | 120 | C.g. of outrigger must be positioned within +/-12 inches of pile c.g |
| Pile w/batter | | | 140 | 140 | Position load within +/-12 inches of batter and vertical pile intersection |
| Bridge rail girder | | | | 200 | Steel or Aluminum cribbing must bridge across two adjacent rail girders |
| Approach | | | | | |
| Deck | 20 | 16 | 20 | 30 | |
| Deck Mod | 25 | 20 | 25 | 35 | |
| Pilecap | | | 100 | 120 | C.g. of outrigger must be positioned within +/-12 inches of pilecap C.L. |
| Mod Pilecap | | | 90 | 105 | C.g. of outrigger must be positioned within +/-12 inches of pilecap C.L. |
| Pile | | | 90 | 120 | C.g. of outrigger must be positioned within +/-12 inches of pile c.g |
| Platform | | | | | |
| Deck | 20 | 16 | 25 | 35 | |
| Deck Mod | 25 | 20 | 30 | 40 | |
| elevator mod | 20 | 16 | 25 | 35 | |
| Orig. Ramps | 20 | 16 | | | |
| Ramp mods | 25 | 20 | | | |
| Stringers | | | 70 | 80 | C.g. of outrigger must be positioned within +/-6 inches of stringer C.L. |
| 8-inch walls Bents 20,21,29,30 | | | 100 | 110 | C.g. of outrigger must be positioned within +/-6 inches of wall centerline |
| 8-inch walls except 20,21,29,30 | | | 110 | 120 | C.g. of outrigger must be positioned within +/-6 inches of wall centerline |
| Pile c.g. | | | 100 | 110 | C.g. of outrigger must be positioned within +/-12 inches of pile c.g. |
| 8-inch walls on piles | | | 100 | 140 | C.g. of outrigger must be positioned within +/-6 inches of wall centerline and pile c.g. For lines A, B, D, and E only. |

(1)Cribbing must be 48" x 48" or larger. Cribbing for bridging adjacent structural elements must be long enough to cover both elements simultaneously.

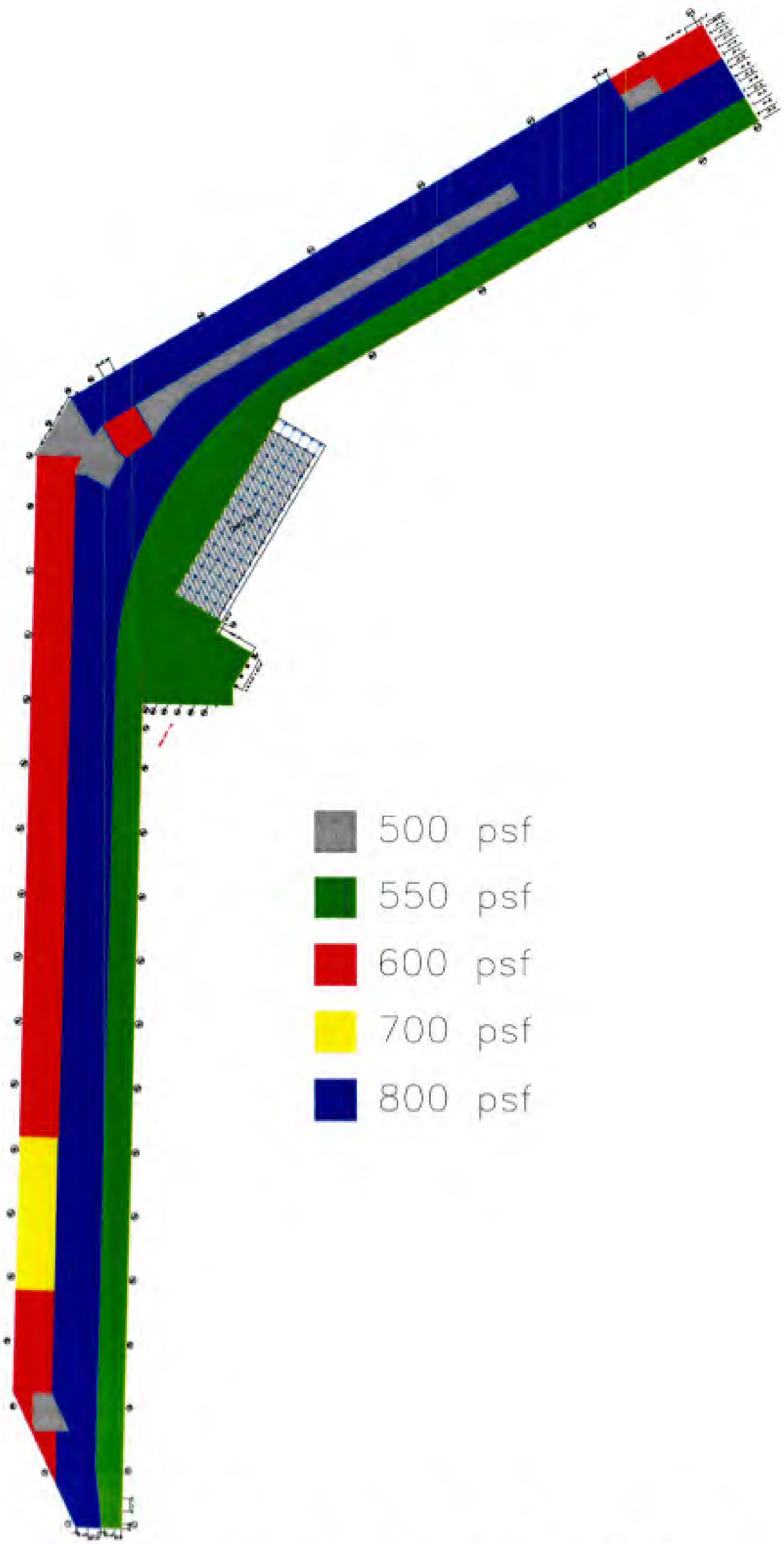



**Figure 49. Schematic of allowable uniform load.**

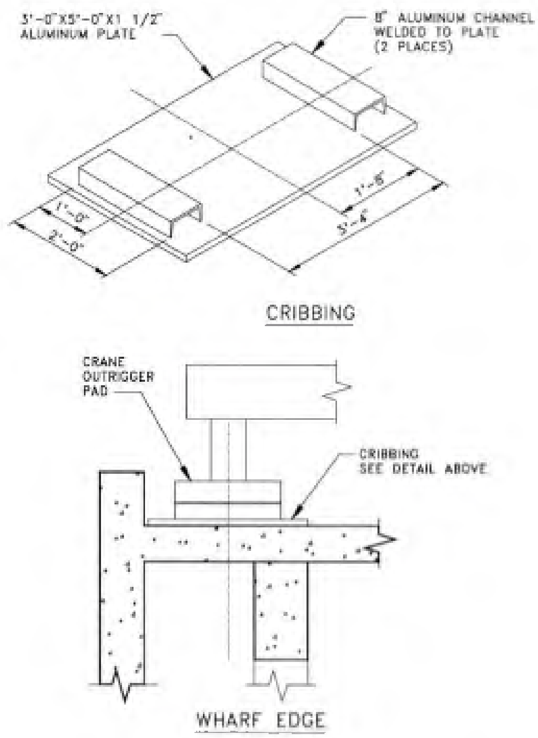



**Figure 50. Aluminum cribbing.**

There was nothing in the test results that would suggest the North Pier platform could not perform as it was originally designed. Portal crane operations can resume on the north platform without restrictions on the original design. The transverse crack (or construction joint) between Bents 110 and 111 is severely deteriorated. To avoid accelerated reinforcing steel corrosion and structural degradation, this cracked area should be repaired within the next 5 years.

## *RECOMMENDATIONS FOR UPGRADING AND REHABILITATION*

Traditional upgrades of Navy pier decks to resist outrigger loads consist of removing and replacing the decks or by adding concrete to the original deck. NAVSTA Norfolk upgraded Pier 7 by replacing the deck. NAVSTA San Diego has added additional concrete to thicken existing decks of two piers. The reconstruction is time consuming and requires the facility to be shut down during the project. NFESC has been developing upgrades using advanced plastic composites as external reinforcing. While the materials are over 10 times more expensive that conventional materials, they are noncorroding, add very little additional dead load to the structure, are easier to install, and pier downtime is usually not required. We have developed similar composite upgrade designs for NAVSTA Norfolk, NAVSTA San Diego, and NAVSTA Pearl Harbor.

NFESC recommends that the two damaged piles along the outboard edge of the south platform be rehabilitated by encasing the damaged section in composite shells filled with nonshrink grout or mortar. It is necessary to return these piles to their original strength because maximum loaded outrigger floats are placed along pile Line A. We also recommend a crane pathway be developed over the main deck to the loading platforms. Carbon composite reinforcement can be added to the top of the deck slab over the rail girders and to the bottom of the approach slab and other slabs of the main deck that are necessary to create a path to the platforms. Main deck load capacity can be increased approximately 20 percent by addition of reinforcing over the rail girders to allow travel of all cranes except Grove RT880.

TRIREFFAC may want to also consider the cost effectiveness of adding additional strength to the soft areas at the base of the ramps, across the boundaries of deck modifications, and in areas transverse cracking. The latter two will have to contend with severely deteriorated concrete, which must be removed before attempting to add reinforcement.

## EMBEDDED REINFORCEMENT

Embedding high strength carbon/epoxy rods into slots filled with epoxy in the top surface of the deck above the rail girders is a relatively easy technique to increase the load capacity of the main deck slab (Figure 51). The traditional alternative approach is to remove more than three inches of concrete from the deck surface, splice in place necessary steel reinforcement, and replace the concrete with shrink resistant grout. High strength carbon rods possess necessary stiffness, strength, and durability to perform better than steel and they are compatible with concrete and epoxy adhesives. The process of cutting slots in the concrete surface and embedding carbon rods in slots is fast and the area can be returned to service within 24 hours after the rods are placed. The required area of high strength carbon rod reinforcing over the rail beams can be satisfied by ¼-inch diameter rods at 6-inches on center (Figure 52). The rods do not require deformations like conventional steel rebar because the rods are embedded in epoxy. NFESC has measured more than 1,200-psi bond strength using the rods embedded in epoxy. A high strength, ¼-inch diameter, carbon composite rod would have a 10-kip ultimate tensile strength and a service limit of 5 kips.

**Figure 51. Embedding carbon reinforcing rods in pier deck.**

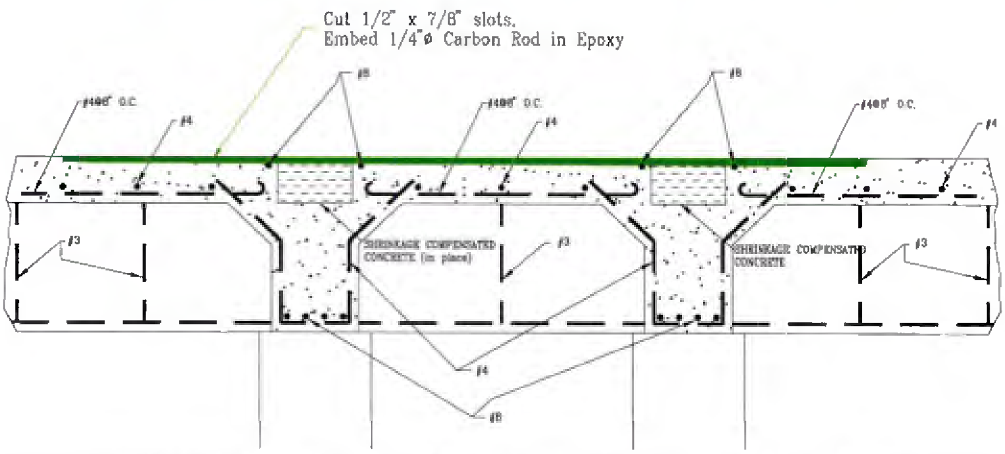



**Figure 52. Embedded carbon composite rod to upgrade main deck strength adjacent to rail beams.**

Reinforcement should only be embedded if the concrete is sound. This includes repair of nonfunctional concrete holes as well as removing and replacing damaged concrete that has chloride, oil and other contamination, spalls, and delaminations.

After concrete repair all loose material is removed from the surface. The surface area to be reinforced is primed with a two-part, penetrating epoxy sealer/primer. The primer will increase the tensile and impact strength of the outer 1/8-inch layer of original concrete. After the primer is cured for 24 hours, the reinforcing grid is laid out by chalk line and slots are cut in the deck with a concrete saw or router. The depth and width of the slots are determined by the diameter of the reinforcing rods and the undulations in the concrete surface. The carbon rods should be completely encased in epoxy within each slot. Allow 1/8 inch between rods and the concrete and at least 3/8-inch clear cover. Slots will be cut approximately 3/4 inch deep and 1/2 inch wide. The carbon rods must be completed immersed in epoxy between high points on the concrete surface. The slots are abrasive blasted to etch and clean the concrete. The slots are primed with the epoxy penetrant/sealant after they are thoroughly cleaned. The high strength carbon rods are embedded in a two-part epoxy that may be filled with as much as 30 percent (by volume) with 60 grit sand. Slots are partially filled with epoxy and the rods are laid in the slots and pressed to the bottom and the slots are filled. The epoxy/carbon rod system must receive ultraviolet radiation (UV) protection preferably in the form of an additive. If a urethane UV protective layer is provided over the epoxy, then the epoxy must be fully cured before adding the UV layer. Sand is sprinkled over the uncured polymer to prevent slipping on the concrete surface. Another UV layer can be derived by mixing three parts of sand to 1 part mixed epoxy (by volume). The sand/epoxy layer can be added immediately before the encapsulatant epoxy is cured.

Costs of embedding composite reinforcing are dependent on extent of concrete repair and preparation but should be in the order of \$200/ft$^2$.

**WET LAY-UP COMPOSITE LAMINATE**

Wet lay-up, carbon/epoxy composite laminate may be also be used to externally reinforce the underside of the Marginal Wharf deck slab. The composite laminate should consist of uniaxial carbon fiber tow sheets in an epoxy resin matrix (saturate). The saturate is required to develop a high interlaminar shear strength and bond with the concrete to develop the concrete shear and tensile strength. The laminate is hand laid and cured in place. The carbon tow sheet fibers should 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$^2$).

A maximum of five carbon fiber plies can be allowed in the orthogonal directions to obtain the required reinforcement area. The first layer is applied after coating the fiber sheet or the concrete surface with epoxy saturate. Successive plies are added between layers of epoxy saturate. Epoxy saturate is hand rolled and brushed into the tow sheet in order to completely wet the carbon fibers (Figure 53). Excess saturate and bubbles are worked out of each layer by squeegee and roller. Holes are not allowed to be cut in the composite when obstructions such as drains, pipe hangers or other hardware are encountered. Instead, tow sheets are to be split along uniaxial fibers to bypass the obstruction when it is not located between strips. Lap splices are allowed if necessary. Laps must be at least 8 inches (20 cm) or more in length. Successive layers of carbon sheets are not to be spliced at the same location. The finished laminate thickness is expected to be 0.2 inches (4 mm) or less.

Costs for laminating external reinforcing to the bottom of the deck is comparable to those incurred to embed a like amount of carbon reinforcing bars.

**Figure 53. Applying uniaxial carbon fiber sheet to underside of pier deck.**

**PILE CONFINEMENT WITH PREFORMED COMPOSITE SHELLS**

Damaged piles 42A and 64A are critically located for supporting outriggers along Line A on the outboard edge of the south platform (see Figure 7). The past two inspection reports have noted the damage and recommended replacement of the piles.

Replacement is the traditional repair method for piles with capacity loss damage. Replacement of piles requires that a deck section is removed and a new pile is driven adjacent to the original. A new pile cap and deck section is cast to tie into the original pile cap and deck. A damaged pile may also be pulled after removing the deck and pilecap above the damaged pile. A new pile is driven in its place and the pile cap and deck is recast. Traditional pile replacement is time consuming and can lead to further damage of the deck in the vicinity of the reconstruction. New concrete can set up galvanic reactions with the original concrete that promotes accelerated corrosion of steel reinforcing in the area of reconstruction. (New concrete galvanic reaction may be the cause of the spalling under the deck at the boundary of the 1954 approach mod.) The area of reconstruction is also lost for mission support during the project and for some time afterwards as the new concrete is cured in place.

Composite shell encasement is an attractive alternative to replacement if the steel reinforcement retains at least 80 percent of its cross sectional area. This technique employs single and two piece preformed fiberglass reinforced vinylester shells to encase the damaged areas of the piles with shrink resistant epoxy grout or concrete injected or pumped between the shell and the existing pile (Figure 54). The grout will restore the concrete cross section and the shell will increase the confinement strength of the concrete. NFESC has employed shells with a circumferential (confinement) strength of 4 kips/inch at a maximum strain of 0.2 percent and a longitudinal strength of 1 kip/inch.

The diameter of the laminate shell must be at least 28.5 inches so that it will encase the 18-inch square pile and leave space for the grout. The shells should be long enough to extend from the pilecaps down beyond the damaged area. Laps and connections must be able to develop the full, circumferential, shell strength and stiffness. Piles must be cleaned of all marine fouling and loose concrete. The concrete cover of existing steel reinforcing does not need to be removed unless the steel is corroded, then the steel must be exposed and cleaned. The shells should be configured to allow placement within 1 inch of the pile cap.


**Figure 54. Placing cylindrical composite shell around rectangular concrete pile.**

The adhesive that is used to join the shell sections must be capable of developing the composite shell tensile strength and interlaminar shear strength. The adhesive must be allowed to cure for 24 hours before the grout is placed in the space between composite shell and the existing pile. During the curing cycle, the shell must be clamped in place to insure a sound adhesive joint. The shrink resistant grout or concrete must be pumped or injected in place. The bottom of the shell must be sealed to keep material from leaking into bay waters prior to set. Grout shrinkage must be no more than 0.05 percent.

Costs of repairing the piles will be dependent on the extent of the damaged area which will govern the length of the composite shell as well as the amount of confinement. The repair costs should be $10 – 15K per pile.

## SUMMARY

The Naval Facilities Engineering Service Center (NFESC) has completed impact load tests on the Marginal Wharf. The ILM data has been analyzed. We have also developed several finite element models that represent the wharf and subsystems. The finite element models were validated with the ILM data and used to determine the working load limits of the pier.

NFESC engineers did not find any evidence of structural failure in due to wheel loads (or outriggers). The structure is structurally sound. We found softening in the approach addition, at the base of the ramps, as well as near deck modifications to the south platform. The piles and stringers do not show degradation from current loading. The wharf is a highly redundant structure and those areas demonstrating softness also demonstrated ability to transfer forces to neighboring structural elements. The wharf could function as it was originally designed without restrictions. However, the deck slab panels were not designed to support the heavy patch loads imposed by wheels and outriggers of large mobile cranes. Further, the deck slabs are not biaxially reinforced.

Rail girders, stringers and pilecap girders can support wheel loads of all five cranes considered as well as AASHTO HS20 truck wheels. However, maximum patch loads on the original deck slabs should not exceed 20 kips. Patch loads on the deck mods can be 25 kips. Tandem axle wheel loads less than 16 kips (axle loads less than 32 kips axle spacing at least 54 inches) and single wheel loads less than 20 kips can traverse the deck without restriction. The Grove RT65S and the Lorain RT300D should be allowed to traverse the decks without restrictions. The P&H CN150 and the Grove TM890 exceed the ACI-set capacity of the decks but can continue to be operated as in the past but should be restricted to a 5-MPH speed limit. Restrictions can be further relaxed by upgrading a path across the approach over the main deck to the loading platform. The Grove RT880 exceeds ACI limits throughout the deck and should not be operated on the Marginal Wharf unless an arrangement can be devised to restrict its transportation to pilecaps, stringers and rail girders. Cranes and other heavy vehicles should be separated such that wheels of two vehicles do not occupy the same deck panel. We recommend a heavy vehicle separation 30 feet.

The deck slab cannot support crane outrigger loads and remain within material limits. Maximum outrigger loads of 100 kips can be placed over piles, pile caps, and rail girders (120 kips). The centroid of the outrigger float should be placed within 1 foot of centroid of the structural element. Maximum outrigger loads of 70 kips can be placed within 6 inches of the centerline of the longitudinal stringers on the south platform and the main deck.

Since the wharf is in excellent condition and we expect that the north platform can still support its original design loads without restriction. We estimate the wharf can continue to operate under the above restrictions for at least 20 years unless corrosion accelerates in the vicinity of "working" cracks. The large transverse cracks, the longitudinal cracks over the rail girders, and the boundaries of the deck modifications should be continuously monitored for crack growth and steel corrosion. These cracks will continue to grow and the reinforcement will corrode. Eventually the flexural strength in the area will be effectively reduced. We do not recommend any immediate action but the cracked areas should be repaired within 5 years to avoid degradation of the reinforcing.

We recommend rebuilding piles 42A and 64A on the south pier by composite hard shell encasement rather than replacement. We also recommend that a crane path be made over the approach and the main deck to the loading platforms. This crane path can be built by externally reinforcing the existing deck. The external reinforcement includes embedding composite carbon rods across the top of the rail girders and applying carbon laminate on the bottom of the deck slabs.

NFESC can assist Northwest Engineering Field Activity of NAVFAC in the preparation of upgrade designs with specifications by separate project. NFESC maintains the ILM data and the FEA models for about five years for future reference and testing other load cases. The pier should be reevaluated in five years. During periods between assessments the deck slabs should be visually monitored for crack growth.