A 0.3 to 0.6 m thick layer of compacted aggregate base course (ABC) over the compacted sand subgrade was planned for placement to provide a firm, level surface for support of the steel mats upon which the seven track beams would be placed. Fig. 8 shows the final move route design and the estimated track beam loads. Fig. 9 shows the distribution of jack loading points on the two outer (edge) beams and the center beam.

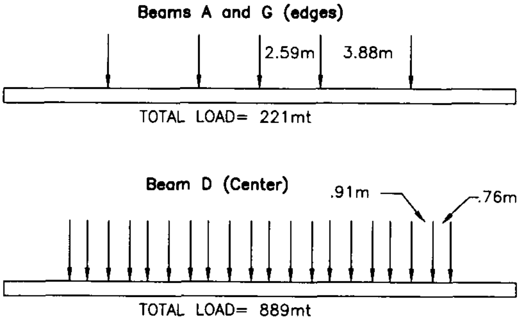

FIG. 9 Hydraulic Jack Distribution Along Track Beams

The point loads from the jacks are distributed through the track beams and further through an underlying mat formed from H-beams welded together into 1.5-m wide sections. The steel mats, the compacted ABC and the compacted soil form a stiff upper layer that reduces stresses transmitted to the deeper sands. For analysis purposes, the relative stiffness of the steel and ABC was ignored. Settlements were conservatively calculated by treating the load applied to the soil as a flexible load distributed evenly over the ground surface.

The geotechnical evaluation focused on settlement of the sand and the potential influence of the thin soft/loose layers encountered in some of the borings. Elastic layer methods using the dilatometer modulus (M-value) were used to calculate settlements. The loading was modeled as strip loads with a uniform pressure. For analysis, a 1.5 m strip 21.3 m long (the width of the steel mat) was assumed with a uniform pressure of 143 kp to 163 kp. The analyses indicated potential for settlements of about 50 to 63 mm in the center of the strip and about 25 to 38 mm at the edge. The movers indicated such settlements could be easily accommodated by the zoned jacking system.

To check the validity of using the DMT and elastic layer methods for settlement calculation, the large-scale field test described earlier was used. The test created a conical-shaped mound with heights of 1.5, 3 and 4.5 m. Elastic theory was used to calculate the expected settlement under the center of the pile using the DMT M-value profile derived from DMT soundings performed earlier at the mound test location. After reaching each incremental height, settlement of the

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original ground at the center of the pile was recorded. Table 1 summarizes the calculated settlements and field-measured values. The agreement between the calculations and field measurements was extremely close. The results gave confidence in the use of the M-values and elastic layer methods to calculate settlements for the lighthouse relocation.

Table 1. Comparison of Calculated and Measured Settlements for Large-scale Sand Pile Load Test

SURFACE STRESS AT CENTER OF LOADED AREA, kp	CALCULATED SURFACE SETTLEMENT, mm	MEASURED SURFACE SETTLEMENT, mm
25	3.5	4.8
50	8.6	7.9
75	15.2	16.3*

*After 12 days; 11.2 mm at initial loading

One benefit of having good confidence in the sand settlement was an ability to evaluate the thickness of compacted ABC needed. The project budget had been based on using 61 cm of ABC. Calculations indicated that reducing the thickness of the compacted ABC to 30 cm would have a negligible effect on the settlement.

The data from the additional DMT soundings performed during the construction were used to calculate the effect of the very thin, very loose zones. These additional calculations indicated the potential for an additional 19 to 25 mm of settlement in two local areas where the zone was thickest. After discussions with the movers, it was determined that the additional estimated settlement posed no significant concern for the move system.

Move Route Preparation

The move route was lightly vegetated with grass over the first 250 m, crossed an asphalt-paved parking area on one side for about 180 m, and then proceeded through scrub woods and underbrush for the last 450 m. Topographic relief was minor; elevations ranged from about 2.1 m to 2.7 m. Man-made dunes with tops at about elevation 6 m were present on the ocean side for the first 70 m. Some excavation into the land side of these dunes was required; the dunes were restored after construction. The planned move route grade began at elevation 2.4 m at the existing location and rose gently to elevation 2.9 m at the new location. At its new location, the lighthouse is about 0.6 m higher.

The move route preparation began with clearing of vegetation and removing the asphalt pavement. The clearing was restricted to a 33-m wide corridor

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designated by the National Park Service. A very thin surficial layer of sand with roots was stripped and stockpiled.

To provide construction access and an acceptable roadbed on which to transport the keeper's houses and other small structures, a 10-m wide roadway on the edge of the move corridor was first prepared by nominal leveling, then placement and compaction of 15 cm of ABC. The remaining 23 m of the move corridor was later prepared as the move route for the lighthouse transport.

Proofrolling of the 23-m wide move route was conducted using a 4-wheel proofroller cart typically used by the North Carolina Department of Transportation for evaluating highway subgrades. The cart, loaded with stone, weighs approximately 45 mt. The goal of the proofrolling was to aid in identifying locations of the thin, loose sand zones with organics so they could be removed or further evaluated. Although one such near-surface zone was identified and removed, the proofroller generally was not successful at identifying loose zones that were deeper than about 0.75 m. As a result, the additional DMT probes and evaluation program discussed earlier was implemented to address the loose zones and their potential impact on the move performance.

Densification of the upper loose sands above the water table was performed using a vibratory roller with a 1.22 m diameter drum, a static weight of 4053 kg and a dynamic force of 105 kN. Water was added to the exposed sandy soil to aid compaction. The compaction continued until the upper 30 cm of the sand was compacted to at least 98 percent of the modified Proctor maximum dry density (ASTM D 1557). Tests for compaction control were performed using the sand cone method (ASTM D 1556) and the nuclear gauge method (ASTM D 2922).

After the subgrade soils were compacted, the 30 cm of ABC was spread (in 2 layers) and compacted to 95 percent of the modified Proctor maximum dry density.

Because the site location is far from rock quarries, the cost of bringing in ABC was relatively high. To reduce quantities used, only sufficient ABC to cover about ¼ of the move route was brought to the site. After the lighthouse crossed over the first part of the move route, the move route components (steel mats, track beams and ABC) were picked up and moved ahead to the next section of the route. The ABC was only stripped to a depth of about 25 cm to minimize sand contamination. With reuse, the aggregate particles did have some breakage; however, the breakdown did not cause difficulty in achieving compaction or in achieving the intent of a dense, essentially non-yielding subgrade for the steel mats.

To make the transition from the original foundation at elevation 0.3 m to the move route at elevation 2.3 m, a series of steps was built by compacting aggregate base course in layers about 0.3 m thick and 2.3 m wide. The steps were topped

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with the steel mats used for the rest of the move route. The track beams were supported on the steel mats by timber cribbing. The confinement provided by the ABC provided protection against localized bearing capacity deformations of the steps into the sands. A similar step arrangement was used at the transition from the move route on to the new foundation. Fig. 10 shows the step construction concept leading on to the new shallow concrete mat foundation.

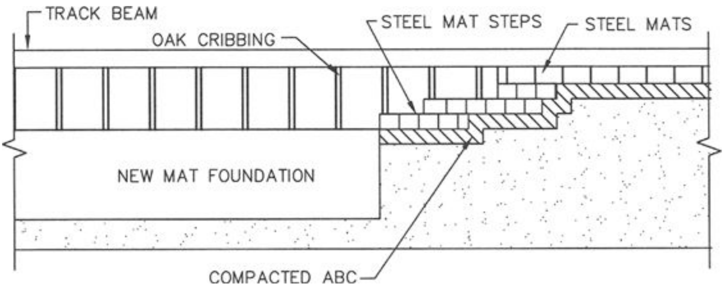

FIG 10. Transition From Move Route to New Foundation

Ground Movements During Move

High precision surveying techniques were used to monitor points on the steel mats as the lighthouse moved. During the first 24 m of the move, which brought the lighthouse off its old foundation, across the steps and on to the move route, the top step mat settled about 30 mm, less than had been predicted for the move route. This area did receive more preparation and compaction than the rest of the route, so settlements further along were expected to be closer to the predicted values.

On the third day of the move, a grid of 42 points was marked on the steel mats between the track beams, and initial readings were taken before the lighthouse reached the area. The lighthouse was stopped directly over the area overnight, and the points were surveyed again the next morning, before moving began. Figs. 11, 12 and 13 show the settlements measured. As shown in Fig. 11, a general bowl-shaped settlement pattern was seen with a maximum settlement of 28 mm in the center and less than 6 mm at the edges. Fig. 12, a section along the move route direction, shows that the maximum settlement occurred near the back of the lighthouse. Fig. 13 shows settlements across the move route at two locations. The figure shows that the maximum settlement was near the centerline of the lighthouse. The pattern of settlement matched the analysis expectations and confirmed that the support system behaved as a flexible system. The measured settlement was about half of the calculated values.

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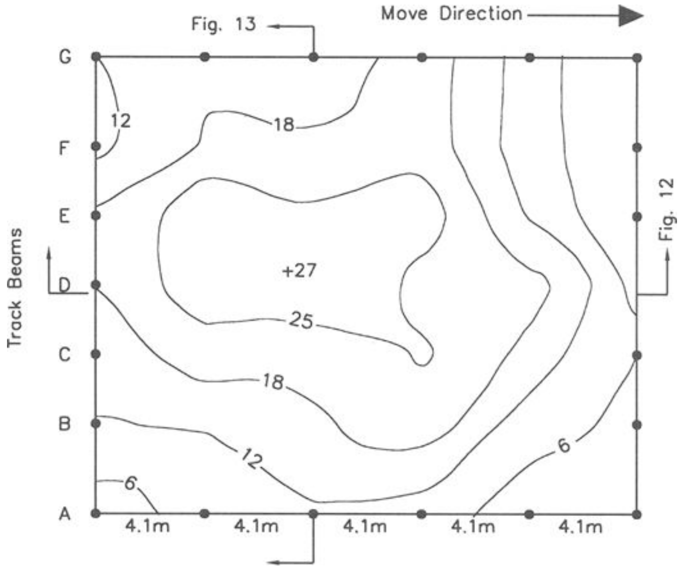

FIG. 11. Contours of Settlement at Location Near Start of Move

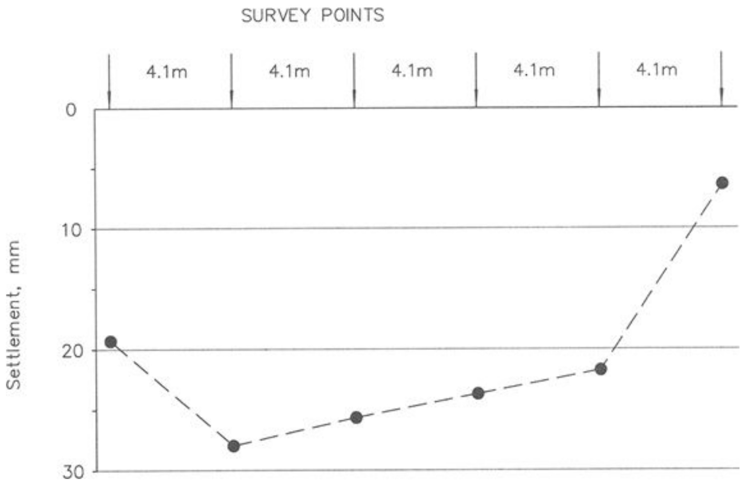

FIG 12. Settlement Under Approximate Centerline of Lighthouse

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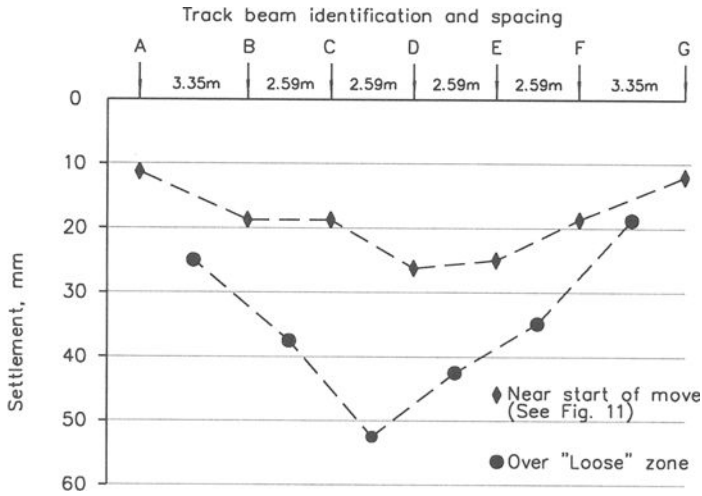

FIG. 13 Settlements Across Move Route

A location along the move route where one of the loose zones was known to be present was also monitored. Several points on the steel mats in line across the move route were surveyed before the lighthouse arrived, then after it had sat over the location overnight. As shown in Fig. 13, these readings indicated a center settlement of about 50 mm and the general bowl-shaped pattern. During the move, the settlements did not cause problems, and few jack adjustments were needed. The actual settlements were about 50 to 70 percent of the predicted values.

Conclusions

The analysis performed for this project demonstrates that settlement behavior of sands under large area loads can be conservatively modeled using confined modulus values from the dilatometer test and elastic layer analyses. By taking into account the stiffness of the steel mats and the ABC layers, it is probable that the predicted settlements would more closely match the actual settlements.

The ability of hydraulic jacking support systems to protect sensitive structures from damage during a move was clearly shown

The success of The Cape Hatteras Lightstation Relocation project also demonstrates that historic coastal structures can be safely relocated to less vulnerable sites. Stabilizing seashores is becoming recognized as a difficult, if not

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inefficient and ineffective, method of structure preservation. The relocation alternate is clearly feasible when seaside structures are endangered by coastal erosion.

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Pile Construction Issues at the P-700 Aircraft Carrier Wharf Project

By Mark R. Tufenkjian,[1] Arthur H. Wu,[2] and Kenneth Woehler[3]

Abstract: The paper summarizes the results of pile installation during construction of the $50 million P-700 Aircraft Carrier Wharf. The project was undertaken by the Navy to homeport a Nimitz-class aircraft carrier at the North Island Naval Air Station in San Diego, California. The existing berthing facilities were modified by creating a new land reclamation area, a new containment rock dike, and a new wharf. The wharf is supported by 450 concrete and steel pipe piles driven through the rock dike and into a dense bearing layer. A major concern during design and construction was the drivability of the piles through the rock dike and into the dense bearing layer. Results of the indicator and production pile programs are presented as they relate to construction issues. A discussion of the construction sequencing and construction difficulties encountered during pile installation are also discussed.

INTRODUCTION

The P-700 Aircraft Carrier Wharf project was undertaken by the Navy to homeport a new Nimitz-class aircraft carrier (USS John Stennis, CVN-74) at the North Island Naval Air Station. In order to accommodate these newer deep draft carriers, the North Island berthing facilities were modified to include a deepened turning basin, a new 13-acre land reclamation area contained on one side by construction of a new underwater rock dike, and construction of a new pile supported wharf. This paper focuses on the construction aspects of the pile installation for the

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wharf. Other aspects of the project have been addressed elsewhere (Wu et. al., 1995; Wu and Hurley, 1997; Wu et. al., 1999; Alcorn, 1998, Schmeltz et. al., 1998). A photograph illustrating the wharf and reclamation area during construction is shown on Figure 1.

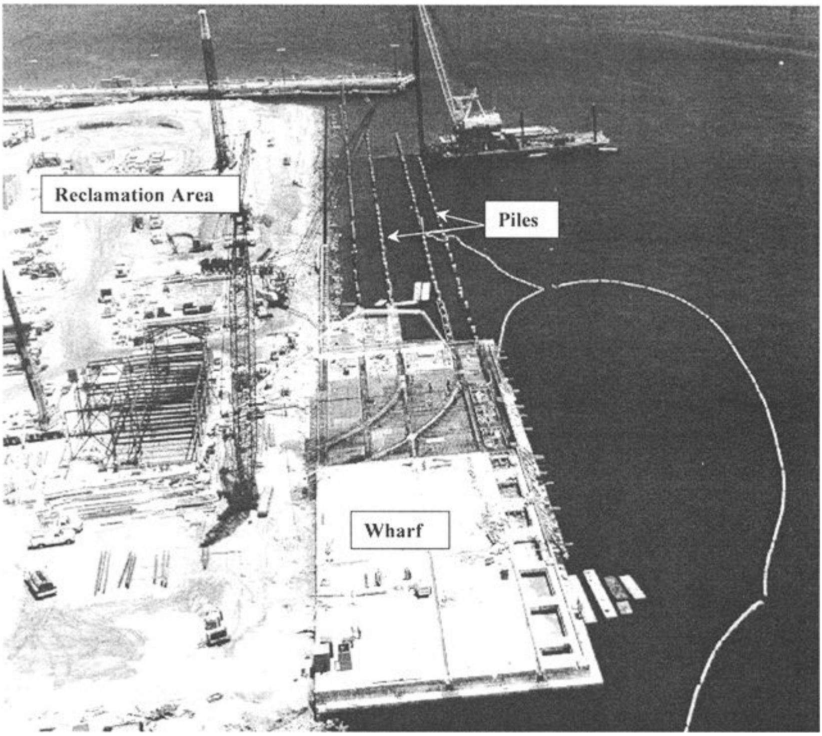

FIG. 1. P-700 Wharf and Reclamation Area during Construction

BACKGROUND

A geotechnical investigation for the project revealed that the seafloor soils in proximity to the proposed dike consisted of bay deposits underlain by beach/channel deposits, which in turn were underlain by the Bay Point Formation. In general, the bay deposits consisted of soft silty clays and loose silty sands about 3 to 7 feet in thickness with SPT blowcounts on the order of 5 blows per foot. The beach/channel deposits typically were loose to medium dense silty sands ranging in thickness from about 5 to 17 feet with SPT blowcounts from 7 to 42 blows per foot. The Bay Point Formation consisted of medium dense to very dense silty sands more than 100 feet in thickness with SPT blowcounts typically greater than 30.

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Dike Design

The designers of the dike determined that the bay and beach/channel deposits were unsuitable for foundation support and had to be dredged. The over-excavated soils beneath the dike were then backfilled with quarry run prior to dike and reclamation construction. The quarry stone sizes specified for the rock dike were between 0.25 inch and 12 inches. The dike and reclamation area backfill were then constructed in multiple lifts such that a continuous slope on the outboard perimeter of the dike was maintained. The outboard slope of the dike was on the order of 1.75:1 (horizontal:vertical). The dike was designed and constructed using a multiple lift configuration to facilitate construction sequencing and to minimize the amount of quarry run needed. Armor stone was placed as shore protection on a portion of the outboard face of the dike. The average size of the armor stone ranged between 12 inches and 24 inches. Figure 2 illustrates a schematic of the multiple-lift sequencing used during construction.

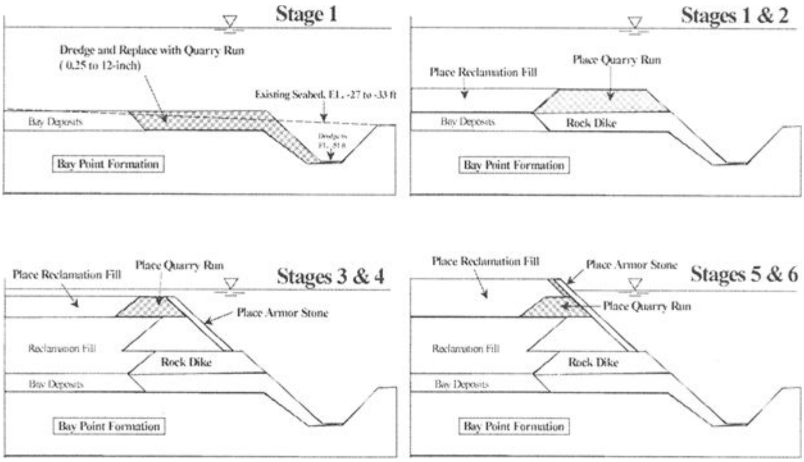

FIG. 2. Multiple-Lift Dike Construction Sequence

Wharf Design

Construction of the pile-supported wharf began immediately following completion of the dike and reclamation area. The wharf is located along the eastern edge of the reclamation area directly above the dike. The wharf is approximately 1,300 feet long and 90 feet wide. The wharf deck is supported by five rows of vertical piles designated as Rows A through E as shown on Figure 3. Battered piles were not used. A longitudinal pile cap that supports the wharf deck connects each of the pile rows.

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