DEEP FOUNDATIONS FOR A 5000 METER LONG BRIDGE

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FUNDACION PROFUNDAS PARA UN PUENTE DE 5000 METROS

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SYNOPSIS: Foundation design and construction for the new Howard Frankland Bridge over Tampa Bay in Florida entailed many challenges. Highly variable and irregular subsurface conditions along the approximately 5000 m long bridge coupled with high design loads (particularly ship impact forces) dictated the use of deep foundations. Over 30500 lineal meters of driven prestressed concrete piles (610 and 762 mm square), bearing in the unpredictable limerock or the overlaying clay formation, were required in the 112 substructure units carrying the bridge. Prior to production pile driving, 112 test piles were installed, and dynamically tested, along the bridge alignment to assess pile drivability and establish production pile lengths at each pier location. Dynamic pile tests were also performed on many production piles to verify design parameters and as means for construction inspection and control. This paper presents discussions on the design and construction of the foundations with findings from the extensive dynamic pile data obtained.

1. INTRODUCTION

Built in the 1950s, the Howard Frankland Bridge is a concrete structure which carries two lanes of vehicular traffic in each direction along highway I-275 between the cities of Tampa and St. Petersburg in Florida, USA. It crosses the navigable waters of the Old Tampa Bay in an East-West direction. The structure consists of 315 spans supported by driven concrete piles. By the late 1980s, increased population and commercial activities in the area necessitated the construction of a companion bridge to ease traffic congestion on the existing Howard Frankland Bridge. The New Howard Frankland Bridge is approximately 5000 m long carrying 4 lanes of traffic parallel to the existing structure. Although the overall length is similar to the existing bridge, the new bridge has one third the number of spans. This was made possible by employing state-of-the-art prestressed concrete T-beams and high capacity piles. Similar to the existing structure, the roadway of the existing bridge is elevated about 4 meters above water level throughout most of the length except over the channel approximately midway along the bridge where the vertical clearance is 15 meters.

Due to the extremely variable subsurface conditions along the bridge alignment, the substantial number of piers, and requirements of high structural and ship impact loads, a comprehensive pre-construction loading test program was undertaken as part of the foundation design phase. The program included installation, dynamic monitoring and static testing of driven and cast-in-place piles. The primary objectives of the program were: assessment of constructability of driven prestressed concrete piles and drilled shafts, evaluation of load carrying characteristics of subsurface formations for most cost effective foundation solution, and evaluation of dynamic pile testing, wave equation analysis and other computer based methods as foundation design and construction control tools. Dynamic testing of driven prestressed concrete piles (610 and 762 mm square) and static compression, tension and lateral load tests were performed on driven piles and 914 mm diameter drilled shafts at nineteen test site locations along the proposed bridge alignment. Testing results were previously presented elsewhere (Sheahan and Poepsel 1987, Poepsel and Sheahan 1988). Findings from the testing program included foundation design parameters, recommendations to use driven prestressed concrete piles, and recognition of a need to extensively utilize dynamic pile testing, wave equation analysis and other computer programs during foundation construction.

Foundation construction commenced with the driving of 112 test piles at production pile locations in piers along the bridge alignment. Dynamic testing results of these piles were used to estimate production pile lengths and diving criteria for each pier. After approximately 20% of the production piles were driven, the bridge was redesigned to include one additional traffic lane which necessitated the redesign of the foundations to accommodate the structural change. Eventually, more than 30500 lineal meters of driven concrete piles were used in the foundation. This paper presents discussions on the foundation design and construction with particular emphasis on results of pile dynamic measurements.

2. SUBSURFACE CONDITIONS

Subsurface investigations included 70 borings drilled to depths between 8 and 35 m for this project and 120 borings of similar depth, drilled for the existing bridge in 1939. The 1939 borings provided visual descriptions only while penetration testing and rock coring was used in recent borings. Borings from the existing and new bridge sites show conditions are very irregular over the length of the structure, sometimes differing substantially even within the same foundation unit. The depth of water was relatively consistent, ranging between 5 and 7 m.

Limestone bedrock locally referred to as limerock, is from the

Tampa Formation and was typically found at depths between 5 and about 40 meters below the mudline. Rock was not encountered in the depth of 35 m investigated at a few locations. Overburden from the east end bent to about 25% of the project length was consistently between 5 and 9 m thick with minor local variations. West of that area the overburden is deeper and more variable. Voids and solution cavities filled with soft clays or loose sand were common. Core recoveries in the limestone varied between 0 and 50%. Unconfined compression strengths ranged between 3.4 and 49.6 MPa. Standard Penetration Tests (SPT), when taken, were greater than 100 blows per 0.3 m.

Overlaying limestone are medium stiff to hard sandy, phosphatic clays and calcareous cemented sands from the Hawthorne Formation. The thickness of this material varied between 1 and 15 m with no clear pattern of deposition over the project. SPT results ranged between 2 and more than 100 blows per 0.3 m. Unconfined compression strengths ranged between 48 and 95 kPa.

Surficial soils are geologically more recent deposits of loose to medium dense sands and slightly clayey sands which contained abundant shell fragments in the near surface intervals. While the thickness of this deposit is generally between 3 and 20 m, more than 35 m were sometimes found in some areas on the west side of the project. SPT values averaged 10 blows per 0.3 m in the near surface intervals but increased to 30 or more below about 10 meters. In some areas these sandy deposits lie directly on the limestone. Shear strength tests on samples of the clayey sand indicated friction angles of 30 to 37 degrees.

3. FOUNDATION DESIGN

For design, four ship impact zones are designated across the length of the structure. Zone D extended 230 m on either side of the ship channel centerline; Zone C extended 230 m beyond the limits of Zone D and Zone B extended 230 m beyond that. The remainder of the structure is in Zone A. Design ship impact loads applied at angles of zero and 30 degrees to the pier axis, transverse to bridge centerline, are 9000, 5340, 2225 and 900 kN for Zones D, C, B and A, respectively.

Impact loads had a significant influence on the number of piles required in each footing. One 35 pile group was used in the 12 Zone D piers. Ten 20-pile piers were in Zone C. Zone B contained 10 piers with 8 piles each. The 78 piers in Zone A each had two 5-pile groups. The two end bents had no impact loads and each had a single row of 7 piles. Designs in Zones D, C and B were governed by lateral loads from ship impact while the Zone A piers were governed by vertical loads. All piles were precast prestressed concrete. Piles in Zone A were 610 mm square solid sections while all others were 762 mm with a 457 mm circular void throughout the length except for the top and bottom 1.5 m. Maximum vertical compression loads were 1780 kN for the 610 mm piles and 2670 kN for the 762 mm piles with a minimum factor of safety of 2. Lateral load designs were governed by the moment capacity of the piles when soil models were analyzed using the COM624 computer program (Reese and Sullivan 1980). Pile heads were embedded more than a meter into the footings to assure a fixed condition. Since pile splicing was anticipated but the location of the splice was unpredictable, a special detail was developed to provide full moment capacity across the splice's length. The structural designer used a STADD program to model the superstructure and substructure units including the pile group. Loads on each pile were estimated using this frame analysis.

Footing sizes were minimized and inclined piles were used to minimize the foundation costs while spreading load to reduce or eliminate group effects. Footings were analyzed with a pile removed from the group to represent loss of bearing in limerock. This test for redundancy showed that none of the remaining piles would be stressed beyond ultimate.

Analyses after redesign for the widened structure showed that footings in Zones D, C and B were satisfactory although vertical loads were increased. An additional pile was required in the center of the each original 4-pile group in Zone A.

Wave equation analyses (Hussein et al. 1988) were performed during design for driveability analysis and to specify minimum hammer characteristics for construction.

Project foundations specifications allowed flexibility while controlling foundation construction. One pile in each pier was driven as a test pile with dynamic monitoring to confirm pile capacity and establish casting lengths.

4. DYNAMIC PILE TESTING

All 112 test piles and more than 100 additional production piles were dynamically tested with a Pile Driving Analyzer™ (PDA) according to the Case Method. Subsequent dynamic data analysis was performed according to the CAPWAP® Method. Dynamic pile testing is performed under impact pile driving during the initial installation process, or restrike. Testing is often performed during pile installation for the purposes of evaluating hammer and driving system performance, monitoring of pile driving stresses and assessment of pile structural integrity, and evaluation of pile static bearing capacity. Dynamic testing is also performed during pile restrikes to evaluate time dependent pile capacity changes (Preim et al. 1989) and for appraisal of structural integrity (Rausche et al. 1988).

Modern dynamic pile testing is based on the measurement of pile strain and acceleration under hammer impacts. The equipment consists of two reusable strain transducers and two accelerometers, bolted at opposite sides of the pile at a minimum distance of one meter below its head, and a Pile Driving Analyzer. The PDA is state-of-the-art field digital computer which applies Case Method equations to the pile measured data and computes some 30 variables in real time between hammer blows after providing signal conditioning and calibration to the measured signals. Field computed results include: maximum energy transferred to the pile (Likins and Rausche 1988), maximum dynamic pile compression and tension stresses and a pile structural integrity assessment factor (Hussein and Rausche 1991), and pile driving resistance and static bearing capacity (Rausche et al. 1985). This method of pile testing has been incorporated into many standards and specifications (ASTM 1989) and is now routine procedure in modern deep foundation practice worldwide (Goble 1994).

Dynamic data of pile strain and acceleration recorded with the PDA in the field is further analyzed with the CAse Pile Wave Analysis Program (CAPWAP) for a comprehensive evaluation of soil behavior during pile driving and under static loading conditions (Rausche et al. 1994). The analysis is done in an interactive environment using measured data and wave equation type analysis in a system identification process using signal matching techniques. CAPWAP analysis results include static pile capacity, soil resistance distribution along the pile shaft and under its toe, soil damping and quake values associated with shaft resistance and end bearing, axial forces along pile length at ultimate resistance, and a simulated static loading test result of load-movement relationships at the pile head and toe.

5. INSTALLATION AND MONITORING OF TEST PILES

A test pile driving and monitoring program was undertaken prior to production pile construction. The program included installation and dynamic testing of eighty 610 mm and thirty two 762 mm square prestressed concrete piles. All piles were driven in production pile locations in all piers along the bridge length. The purpose of the program was to determine casting pile length and installation procedure and driving criteria at each pier location. Wave equation analyses were performed to evaluate pile driveability and establish minimum hammer energy required. Piles of both sizes were evenly divided between the east and west halves of the structure. Pile lengths generally increased going from east to west along the project site, although there were some locations where piles of longer than anticipated length were needed. In some cases piles which drove easily to full penetration were extracted and piles of longer length driven in their place, or they were spliced for added length. For piles that did not quite meet the driving criteria at the end of driving, restrike testing was performed to evaluate the effect of time on pile capacity increases. On the east side, pile lengths generally ranged between 13 and 25 m with a few over 30 m. Piles on the west side were generally longer with lengths ranging between 20 and 44 m. Two single acting air hammers were used to install the piles. The 610 mm piles were driven with a Conmaco 200E-5 model having a ram weight of 89 kN with a mechanism to work with half or full 0.9 m strokes with corresponding rated energies of 41 and 82 kJ, respectively. The 762 mm piles were installed with a Conmaco 300E-5 hammer having a ram weight of 133.5 kN and the possibility of 0.67 and 1.02 m strokes, corresponding to rated energies of 91.1 and 136.0 kJ. A variety of plywood cushion thicknesses (between 900 and 1800 mm) were tried in the field in order to maximize energy transfer to the piles and at the same time keep dynamic driving stresses below allowable limits. In some cases jetting was used to assist in pile installations.

All test piles were dynamically monitored during initial installation and some were also tested during restrike. In all cases, the hammer was used with the shorter stroke to drive the pile to a depth where appreciable resistance was encountered and the PDA indicated stresses in the pile at levels that would warrant switching to full stroke. There were cases, however, where the full pile installation was done with the shorter hammer stroke due to the lack of driving resistance. Generally, compression stresses in the piles ranged between 13 and 25 MPa and tension stresses were less than 7 MPa. There were a few cases, however, where the dynamic axial stresses exceeded the concrete strength resulting in pile damage. Figure 1 presents plots of pile force and velocity records of two piles. The top plot represents data from a hammer blow towards the end of driving of a 29 m long, 610 mm pile showing partial pile damage at a distance of 25 m below the pile top. The bottom plot shows a completely broken 25 m long, 762 mm pile. Generally, the hammers transferred an average of 45% of their rated energies to the piles. This transfer ratio is within the range of well performing hammers under similar conditions. Due to the relatively massive pile splices used (a splice consisted of a concrete collar which more than doubled the pile area for a length of approximately 2.25 m). Tests were performed to study the effect of the splice on energy transfer and pile driveability. It was found that the splice generally reduced the energy transfer to the pile section below by approximately 12% as compared to the energy reaching the pile section above the splice. Figure 2 includes plots of pile force, velocity and energy taken at locations just above and below the splice of a 26 m long (full length), 610 mm pile. Whenever possible, piles were driven until they encountered refusal driving resistance (i.e., blow counts of 240 blows per 0.3 m or more). End of driving capacities for the 610 and 762 mm size piles averaged 5350 and 6700 kN, respectively. Shaft resistance accounted for an average of 11% of total pile capacity. CAPWAP analysis indicated that soil damping factors were about 0.33 s/m along the shaft and under the toe of both pile sizes. Skin quakes were 2.8 and 3.0 mm for the 610 and 762 mm piles, respectively. Toe quakes for the 610 mm piles averaged 7.4 mm and for the 762 mm piles averaged 8.2 mm. Along the west side of the project, piles did not behave in a consistent manner. Pile capacities at the end of driving of the 610 mm piles ranged between 1100 and 5400 kN and were generally less than 4450 kN. Skin friction ranged between 5 and 70% of total pile capacities. Soil damping factors ranged between 0.3 and 1.0 s/m. Skin quakes averaged 3 mm and toe quakes averaged 8 mm. The 762 mm piles on the west side had end of driving capacities ranging between 2700 and 7100 kN and were generally greater than 4450 kN. Soil damping values

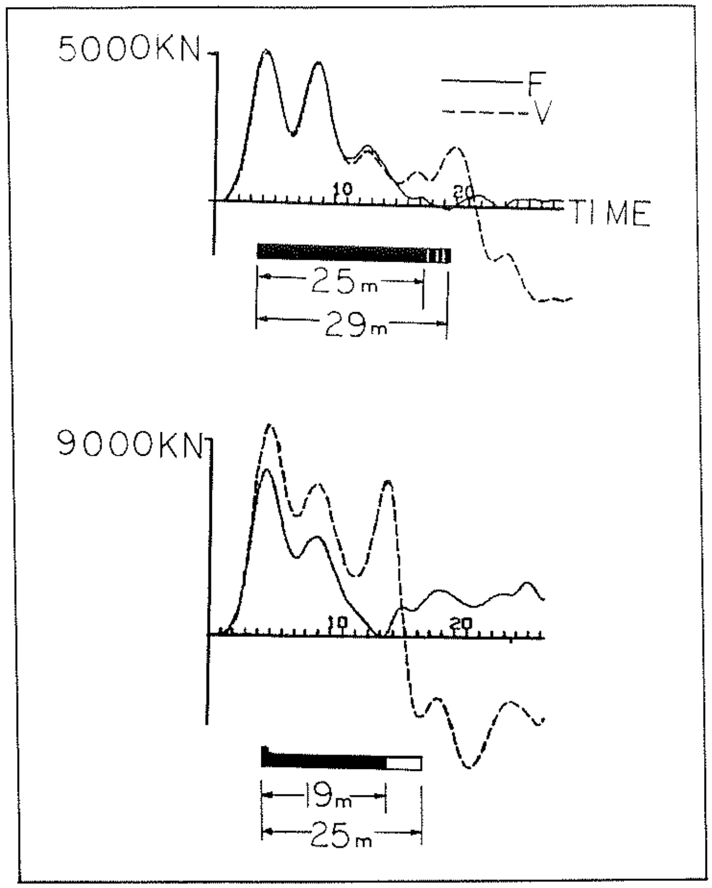

Figure 1: Dynamic Data Indicating Pile Damage

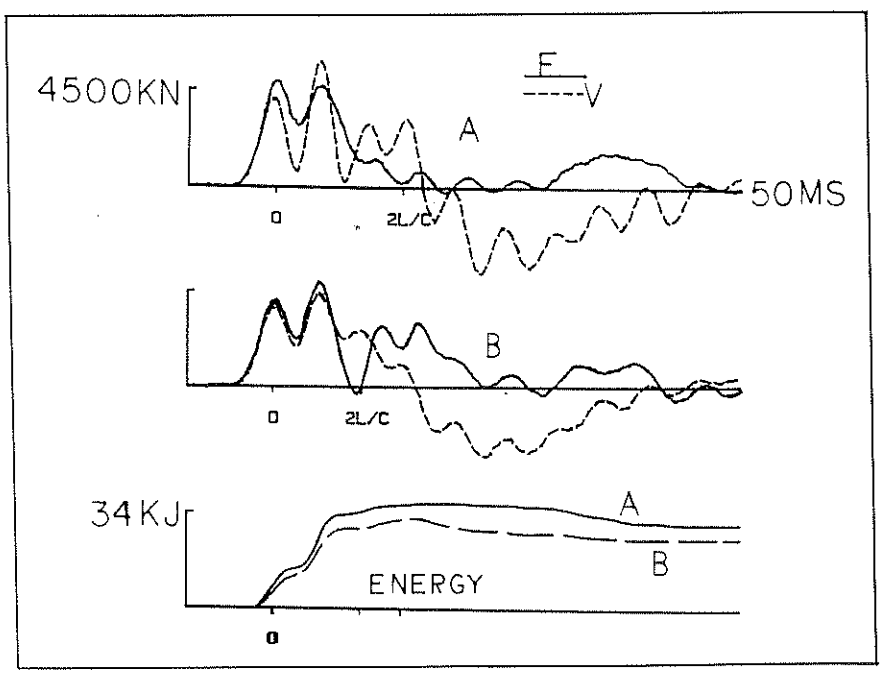

Figure 2: Dynamic Data from a Spliced Pile: Above (A) and Below (B) Splice Location

averaged 0.4 s/m. Soil quakes along the skin averaged 2.5 mm and under pile toes averaged 8.2 mm. Analysis of dynamic data obtained from restrike tests on piles located in the west side indicated that soil damping increased by 65%, skin quakes did not change, and toe quakes decreased by 60% as compared to end of drive values. Pile capacities generally increased with time. Plots of pile dynamic data along with computed soil resistance forces at the end of driving and beginning of restrike of a 762 mm pile are presented in Figure 3. Figure 4 presents the percent increase in pile capacity as a function of time for various pile lengths of both sizes on the west side of the bridge. This figure shows that the amount of pile capacity increase with time is variable across the site due to the varied soil conditions.

6. PRODUCTION PILE DRIVING

Based on dynamic monitoring in the load test program, production hammers for the 762 mm and 610 mm piles were required to have rated energies of at least 136 and 82 kJ, respectively. These required energies were greatly influenced by the soil elastic deformation (i.e., quake). Wave equation analyses on the Conmaco hammers proposed by the constructor, which were rated with these energies, showed they were suitable. Analysis results also showed target energies required at the pile tops for end of drive conditions were 58 and 34 kJ for the 762 and 610 mm piles, respectively. For each hammer these transferred energies were about 42% of the maximum rated energy. These values were confirmed during the driving of the test piles.

Casting lengths for production piles were provided for groups of 10 piers or less at a time based on analysis of test pile driving. Lengths were conservatively estimated since the cost of a single splice was equal to about 9 m of piling. Splices required several days to form, pour and cure before driving. Piles were interchanged between piers since variations within the pier footings often occurred. For example, pile penetrations at Pier 24 East varied between 6.8 and 7.5 m while at Pier 3 East, they varied between 6.5 and 9.5 m. By contrast, penetrations at Pier 24 West varied between 18.5 and 44.3 m while at Pier 4 West they varied between 15.7 and 43 m. The variations at Pier 4 West occurred in the 5-pile group of the south footing alone. Despite these variations, the 33,800 meters of piling driven was within 9% of the estimated quantity in the contract. One hundred forty-two (142) splices were used with as many as 3 on a pile. This was 17% more than estimated. A total of 250 restrikes were authorized on the 1050 piles which in many cases meant elimination of splicing.

During the test pile program, the contractor submitted an alternate structural design for the 762 mm piles, using 4868 kN of prestress per pile as compared to 3329 kN in the designed pile. While the alternate design appeared to be suitable, the allowable driving stresses were affected by the higher prestress. Computed allowable driving compressive stresses

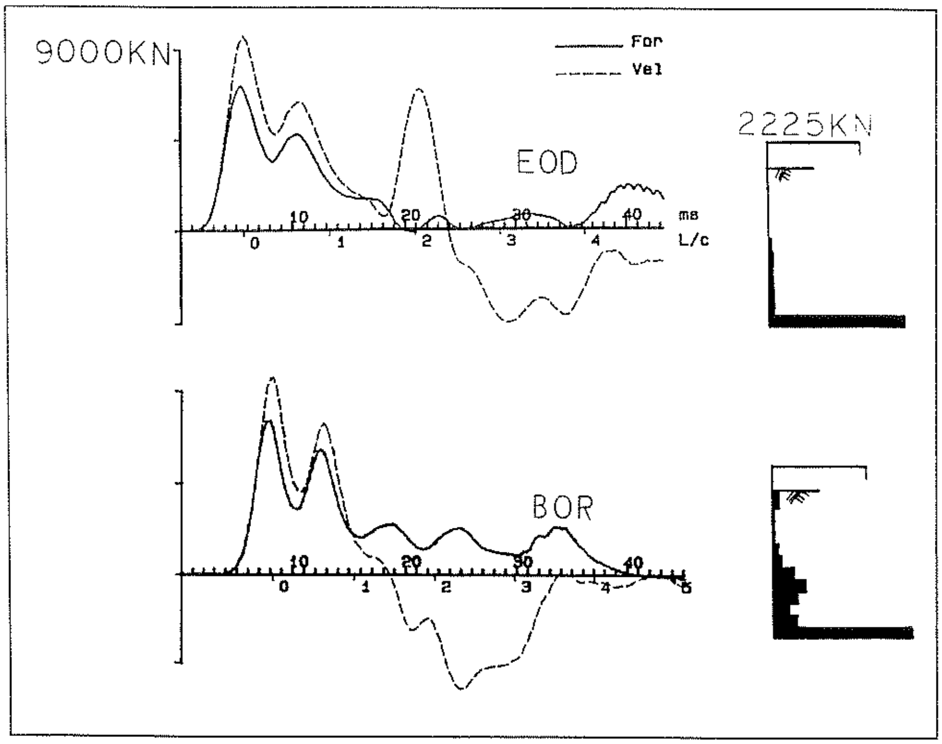

Figure 3: Dynamic Data and Computed Soil Resistances from End of Driving (EOD) and Beginning of Restrike (BOR)

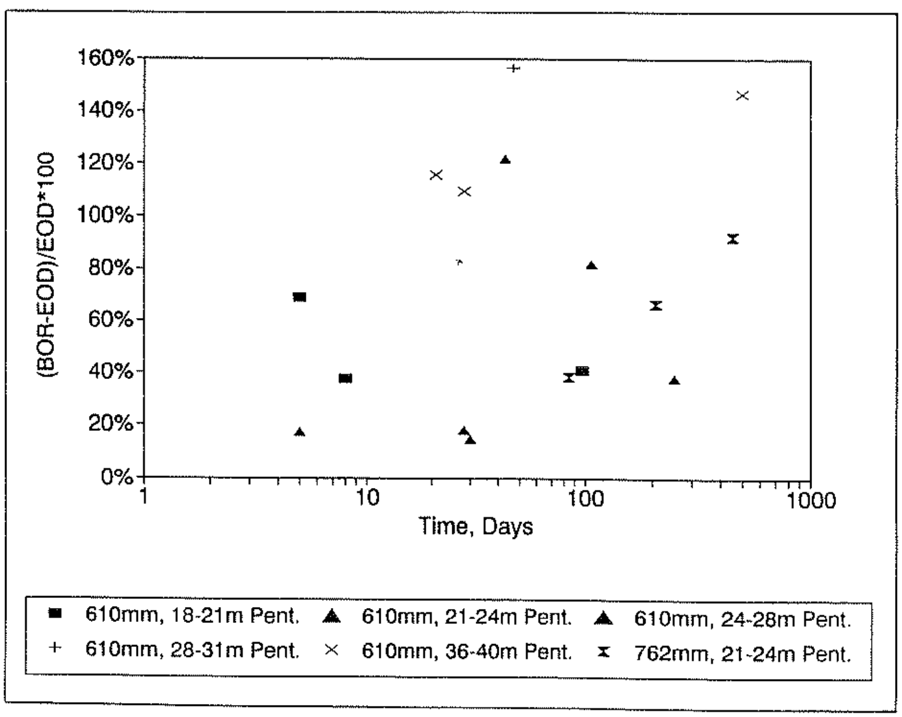

Figure 4: Changes in Pile Capacity with Time, West Side Piles

were reduced by 14% while the allowable tension stresses increased by 36%. Monitoring of the first two piles with the higher prestress levels indicated they were severely damaged while attempting to drive them to bearing. Both piles were removed due to the extent of damage. Prestress levels in all other piles were set according to original design of 3329 kN.

A special splice design was required to maintain moment capacity in the pile. The result was a 2.5 m long, 205 mm thick, heavily reinforced friction collar centered on the spliced section. Dynamic pile testing was utilized in several cases to evaluate whether load was being carried in the splice when it was driven below mudline. Analysis of two 35 m long piles tested indicated that no load was carried by the splice.

Despite the addition of a fifth pile in footings at each pier in Zone A, loads on some piles were about 10% over the 1780 kN allowable for certain loading conditions. In addition, some piles were driven out of position. Often, pile extraction was difficult and new piles in the group would have required increasing the footing dimensions. Some piers were reevaluated using STADD with actual pile locations and inclinations to estimate the new pile loads which resulted. Analysis showed that under some load cases individual piles could receive as much as 25% more than design allowable. In these cases, dynamic pile testing were utilized to verify pile capacity. When the increased capacity could not be completely confirmed, additional STADD analysis was performed with the pile capacity reduced to 10% of allowable. The group was accepted if none of the remaining piles were loaded beyond ultimate.

On one occasion during routine, bi-weekly hammer performance testing using the PDA, transfer energies from one of the hammers were below target values by about 30% although the hammer appeared to be performing suitably otherwise. The cause of the problem was determined to be deterioration in the hammer cushion which could not be easily observed during routine pile driving inspection. Acceptance of piles driven since the previous hammer testing was based on restrike testing and/or wave equation calculations with reduced hammer performance.

7. CONCLUSIONS

Subsurface conditions at the site of the new Howard Frankland Bridge are extremely variable. The existing structure, built in 1955, consists of 20 m spans supported on 610 mm square concrete piles with a 546 kN allowable compression load. The new structure which is designed to resist ship impact loads of up to 9000 kN, consists of 44 m spans supported on 610 and 762 mm square prestressed concrete piles with allowable compression loads of 1780 and 2670 kN, respectively. The ability to have confidence in the use of higher allowable loads with the modern design was justified by a pre-design load test program and the use of dynamic testing to verify capacity, monitor hammer performance and control stresses during driving. In addition, modern computer design methods allowed a rapid reevaluation of the foundations for a widened structure, redesigned after construction of the foundation had begun. These same methods were also used during construction to quickly assess the effects of out of position piles on pile loads.

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