Case Study: Application of the Observational Method Using High Strain Dynamic Tests

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CASE STUDY: APPLICATION OF THE OBSERVATIONAL METHOD USING HIGH STRAIN DYNAMIC TESTS

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ABSTRACT

In-situ High Strain Dynamic Testing (HSDT) was developed more than 40 years ago. When a hammer or drop weight strikes the top of a pile, a compressive stress wave travels down its shaft at a speed that is a function of the elastic modulus and mass density. The impact induces a force and particle velocity at the top of the pile that can be measured using accelerometers and strain gauges. This paper discusses the theory behind using HSDT instruments and the procedures to calculate the capacity of the pile. The role of selecting the proper instrument to record them is also discussed. Finally, a case history involving the use of HSDT as an instrument for the observational method is presented.

BEGINNINGS OF DYNAMIC TESTING

As the need for more efficient piles became apparent, concrete piles were introduced in the early 20th century. Isaacs (1931) recognized the possible tension stresses that may develop in a concrete pile tip when driven to a soft material and encouraged a more exhaustive study of the stresses in piles due to driving. He proposed that one dimensional wave action be applied to the practical application of stress-wave theory to pile driving analysis. In the late 1940's E.A.L. Smith (1960) produced the first general solution for the practical application of stress wave theory (Hussein et al. 2004). With his landmark paper, Smith (1960) formed the basis for modern wave equation analysis. Smith's numerical model divides the pile, hammer and driving accessories into discrete elements, where each element has a mass equal to that of the corresponding portion of the real system (Figure 1). These elements are connected with springs that have the same stiffness as the corresponding element. The method also models the interface between the pile and the soil using a series of springs and dashpots along the sides of the elements and on the bottom of the lowest element to model side friction and toe bearing.

The first attempt to make dynamic stress measurements in pile driving was made by Glanvielle et al (1938). Strain measurements were made using piezoelectric force transducers on concrete piles and recorded on an oscilloscope. Around 1960 a large research project was performed by the Michigan Department of Highways (Goble et al. 1980). They used specially designed force transducers to measure the force at the pile head and also added a strain accelerometer to the transducer. The primary purpose of the test was to evaluate hammer performance.

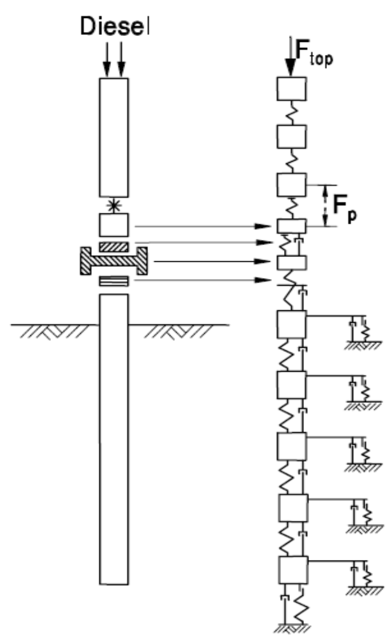

Fig 1. Discrete Element System (Long, 2011)

HIGH STRAIN DYNAMIC TEST (HSDT)

When a hammer or drop weight strikes the top of a pile, a compressive stress wave travels down its shaft at a speed that is a function of the pile's elastic modulus and mass density. The impact induces a force and particle velocity at the top of the foundation. In 1964, an extensive program of stress wave measurement started at Case Institute of Technology (now Case Western Reserve University). Measurement techniques and equipment were developed to measure the particle velocity and force in the pile, producing a large volume of literature (Goble et al. 1980). During the following decades, these procedures and equipment have been tested and also enhanced.

HSDT EQUIPMENT

Pile Driving Analyzer (PDA)

The PDA is a digital microcomputer as shown in Figure 2. Using the transducers that are fixed on to the pile (accelerometers and strain gauges), the PDA provides signal conditioning and an external device for recording the signals (Likins, 1984). The PDA contains software that allows the user to automatically process the data and estimate capacity through the CASE method. The software also calculates the stresses produced by the hammer impact and presents the structural integrity of the pile at the moment of the test.

Accelerometers

The accelerometers, shown in Figure 3, measure the acceleration in the pile. The acceleration is integrated with respect to time to compute velocity. There are two types of accelerometers commercially available for the HSDT, which are piezoelectric and piezoresistive.

Piezoelectric transducers, as their name implies, contain piezoelectric materials that produce a charge output when they are compressed, flexed or subjected to shear forces (deformed). The piezoelectric material cannot be centro-symmetric (containing a center of inversion), since the deformation of these type of materials will not be accompanied by any change in its internal electric field. Typical piezoelectric materials tend to be crystals formed of ionic bonds with moderate to highly complicated geometries (Thomas et al. 2010).

Piezoresistive transducers are based on the idea that a mechanical input (pressure, force, acceleration) applied to a mechanical structure of some kind (beam, plate, diaphragm) will cause the structure to experience mechanical strain. The resistor is wired through an electrical circuit configuration called "Wheatstone Bridge". When the resistor undergoes strain, then the electrical resistance of the piezoresistor is changed, this material property is called "piezoresistance" (Thomas et al. 2010).

Strain Gauges

Strain gauges (Figure 3) use piezoresistive transducers with a full bridge network. Their purpose is to obtain the force produce in the pile by the hammer blow. The shielding protects it from electrical interference and noise problems. Using a strain gauge that is mounted into the pile, the force is calculated using the known material modulus and cross sectional area. The strain gauge must be installed aligned with direction of the force in the pile to ensure that the strain is measured properly.

Based on experimental results, the calibration accuracy of both the PDA strain and acceleration sensors are approximately ±2% or less. Both transducers are available in cabled and wireless models. If not using wireless sensors, the strain sensitivity when using PR accelerometers is changed slightly by the cable length. Installation time is usually 20-30 minutes per pile. At least two strain transducers and accelerometers installed opposite to each other are required for the test, in calculations; average values between the two are used. Piezoresistive accelerometers are intended for high acceleration applications such as steel piles driven by hammers without cushions; piezoelectric accelerometers generally work well for concrete and timber piles, and for steel piles driven by hammers with a hammer cushion (PDA Manual of Operation, 2004). The transducers should be bolted into the pile at a distance of at least two diameters from the pile top. This will assure that if the pile is slightly curved or the hammer impact is not concentric there is enough distance for the forces to equate.

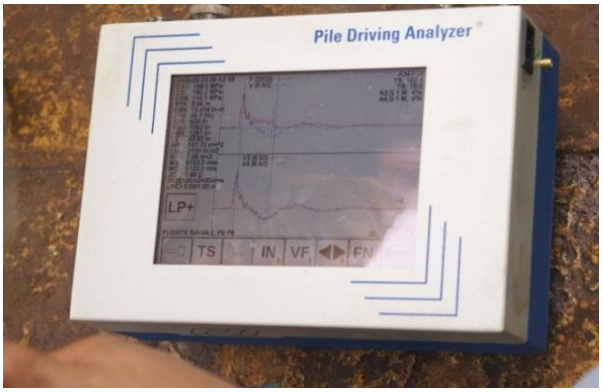

Fig 2. Pile Driving Analyzer (courtesy of CPR Asociados)

Fig 3. Installed Strain Gauges (left) and accelerometer (courtesy of CPR Asociados)

CASE METHOD FOR ESTIMATING PILE CAPACITY

The soil resistance was initially computed from the rigid body equation:

$$R = F(t) - m \cdot a(t) \tag{1}$$

Where R is the total pile resistance, F(t) and a(t) are the force and acceleration measured as function of time and m is the pile mass. Assuming the pile is linearly elastic and has a constant cross section; this equation can be modified and be expressed as:

$$R = \frac{1}{2}\left[F(t_1) + F(t_2)\right] + \frac{1}{2}\left[V(t_1) - V(t_2)\right] \cdot \frac{EA}{c} \tag{2}$$

where, F is the force measured at gauge location, V is the particle velocity measured at gauge location, t_1 is time of initial impact, t_2 is time of reflection of initial impact, E is the pile's elasticity modulus, c is the wave speed of the material, A is the pile area at gage location and L is the length of the pile below the gages.

The strain rates in the dynamic test are in the order of 100 times higher than in the static test. The large strain produced during the test, mobilizes significant inertial forces, as well as forces related with the rheology of the material involved, particularly the soil pile interaction and the viscous nature of the soil itself (Chaure 2004; Rodriguez et al. 2008). The shear strength of the soil is related to the strain rate at which is sheared (Mitchell et al. 1968; Berre and Bjerrum 1973; Terzaghi et al. 1996). To account for this phenomenon, this dynamic resistance must be subtracted from the total resistance calculated using equation (2). Goble et al. (1975) found that the dynamic resistance component could be approximated as a linear function of a damping factor times the pile toe velocity, and that the pile toe velocity could be estimated from dynamic measurements at the pile head. The static pile capacity equation can be expressed as:

$$R_S = R_T - J\left[V(t_1)\frac{EA}{c} + F(t_1) - R_T\right] \tag{3}$$

R_S is the pile's static resistance and J is the dimensionless damping factor that is dependent on the soil type near the toe. The CASE damping factor ranges from 0.10 – 0.15 in clean sand to 0.70 or higher in clays

CAPWAP MATCHING TECHNIQUE

After the force and acceleration data is recorded by the PDA it is analyzed using the Case Pile Wave Analysis Program (CAPWAP). The pile is modeled as a series of continuous segments and the soil resistance is modeled by elasto-plastic springs and dashpots. Since the acceleration and force in the pile are known, the only unknown remaining is the soil resistance magnitudes and distribution. Reasonable estimates of soil resistance distribution, quake and damping are made and the measured acceleration is input in the program. As a result, a force in the pile will be calculated, which is compared to the force measured in the field. Adjustments are made in the soil assumptions and the calculations are repeated until the force calculated matches the force measured in the field.

Fellenius (1988) conducted a study with 19 individual participants associated with all the commercial organizations that performed CAPWAP analysis at the time; they had all received training from either Garland Likins or Frank Rausche. The credentials of the participants ranged from initial training to very experienced (Garland Likins and Frank Rausche participated in the study independently of each other). The participants were given 4 blows of 4 different projects with sites that contain both sand and clay. Although some details differ between the individual analysis there was a considerable qualitative agreement. The COV for total pile capacity from the 4 sites was: 5%, 6%, 13% and 15%. The increase in the last two is due to the complexity of the profile, which required more expertise in the use of the program. The study comes to show that if applied properly the program is able to provide confident data.

ADVANTAGES OF HSDT

In addition to providing the static axial capacity of the pile, the HSDT also calculates the compressive and tensile stresses in the pile, measures the energy imparted by the hammer and determines the extent and location of structural damage (Rausche and Goble, 1979). Using the computer software CAPWAP the resistance of the soil per layer can be obtained.

If we perform a risk and reliability type of analysis, higher factors of safety are applied to systems with less redundancy. HSDT costs and time is less than that of a static pile load test (Cheney and Chassie, 2000) making it better suited for sites with several piles. The potential for more tests per total number of piles could decrease the required factor of safety and thus, the cost. Because HSDT is a nondestructive test, the pile tested is part of the foundation system and reflects the in-situ soil conditions. The need of only a hammer to test a pile is a very important advantage especially in offshore piles since the cost of mobilizing equipment is much higher.

Long et al. (1999) compiled data from sites where both static pile tests and HSDT were performed and compared them. As we can see from Figure 4, the results obtained by the CASE method (labeled PDA in the authors plot) and the CAPWAP show a good correlation.

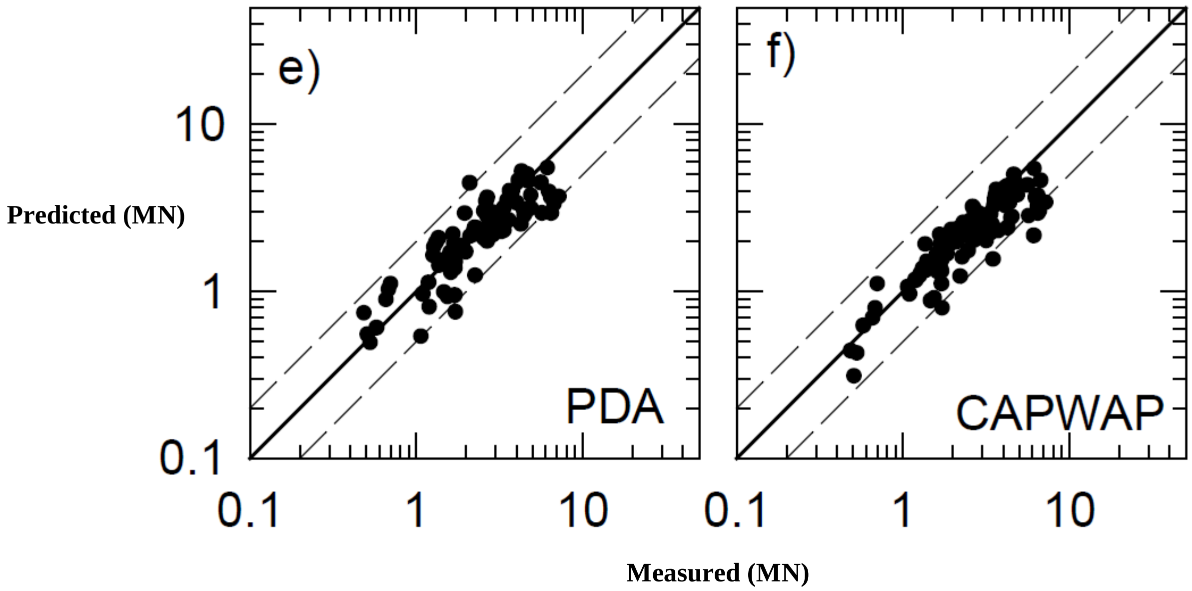

Fig 4. Correlation between Static Load Test and HSDT results (Long et al. 1999)

CASE HISTORY: THE USE OF HSDT IN CONJUNCTION WITH THE OBSERVATIONAL METHOD IN THE BAHIA-SAN VICENTE BRIDGE.

The use of the observational method often permits the maximum economy and assurance of safety (Peck 1969). In the following case history an interesting way of combining HSDT with the observational method is explained.

The Bahía-San Vicente Bridge, with a length of 1,990 meters between abutments, is located in the Chone River estuary in the central part of the Ecuadorian coast. The bridge site is characterized as an estuarine environment formed by alluvial deposits with depths between 25 and 85 meters overlying weathered rocks (Figure 7). The bridge connects the towns of Bahía de Caráquez and San Vicente, located on northwestern Manabí province in Ecuador. The project site is shown in Figure 5. The construction was developed by the Ecuador Army Corps of Engineers and supervised by C.P.R. Asociados C. Ltda., as representatives of the Ecuadorian Ministry of Public Works.

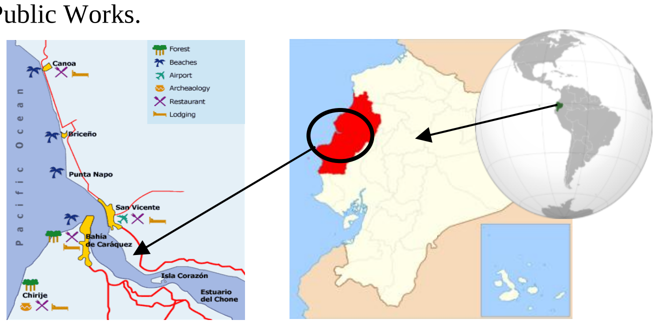

Fig 5. Project Location (courtesy of CPR Asociados)

The roadway section has two traffic lanes, each of them 3.60 meters in width and a 1.5 m shoulder on each side. There is also a 3 m pedestrian and bicycle path on the north side, totaling 13.2 m total deck width. The prestressed concrete beams on which the deck is supported are in turn placed on triple pendulum bearings, which modify the structure's response to lateral loads, leading to lower capacity demand at the foundation level. The foundation was built with open ended steel pipe piles, 1.21 meters in diameter and 20 mm wall thickness. A plan view of the piers is presented in Figure 6, there are a total of 48 piers with a distance of 45 meters between piers, some piers have 8 piles and some have 9. The lengths of the piles vary from 45 to 60 meters.

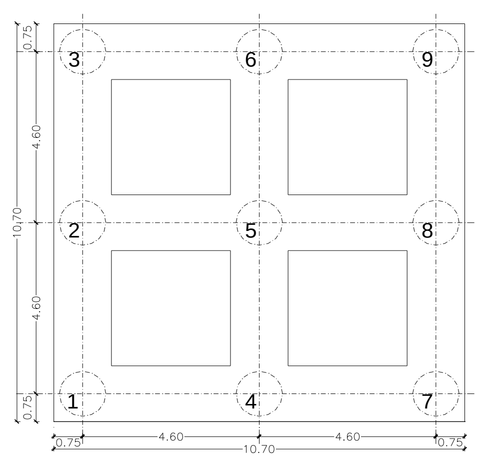

Fig 6. Pier Cross Section (courtesy of CPR Asociados)

The geotechnical conditions can be summarized by the identification of 3 layers under the river bed overlaying the bedrock. The first is a granular layer, mostly loose to very loose sandy silts with a thickness of about 20 to 30 meters and N_{60} values that range from 5 to 30. The second layer consists of clays with thicknesses of 25 to 50 meters interspersed with sands towards the center of the river. The undrained shear strength of the clays varies from 50 to 100 kPa, N_{60} values were reported and they ranged from 12 to 25. The liquid limit varied from 50% to 85%, the plasticity index from 30% to 50% and the moisture content from 40% to 60%. The third layer consists of residual weathered rock with a thickness of 5 to 8 meters before reaching the shale bedrock. A simplified profile between piers 36 and 42 along with N_{60} values from piers 37 and 40 are presented in Figure 7.

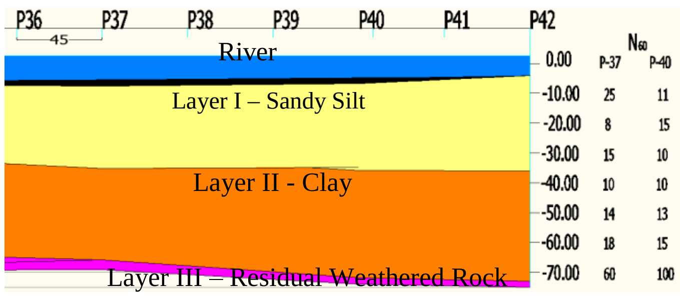

Fig 7. Simplified Profile

The project is situated in the highly seismic region also known as the Pacific Ring of Fire. The 7th largest recorded earthquake as of 2012 (earthquake.usgs.gov), magnitude 8.8 on the Richter scale, occurred off the coast of Ecuador in 1906 (Kanamori, 1977). The most recent important seismic activity took place on August 4, 1998 with a magnitude of 7.2 and a distance 10 km NW from the city of Bahia de Caraquez causing severe damage to the city. Three people perished and forty were injured in the area. Electricity, telephone and water services were disrupted and most buildings with three or more stories were damaged at Bahia de Caraquez. According to the USGS hazard map, the 10% probability in 50 years acceleration in this region is 3.5 ms^{-2}. Due to all these facts, possible liquefaction of the upper layer was a major concern.

In the case of a major earthquake, the shear strength of the soil will be greatly reduced to a value that is a function of the pre-failure vertical stress (Stark and Mesri 1992; Olson and Stark 2002). To ensure that the piles would not endanger the superstructure in the case that the top layer was liquefied, special attention had to be taken in the design.

In the past, two methods have been used to cope with uncertainties, i.e., either adopt an excessive factor of safety or make assumptions based in general experience. One is wasteful; the second is dangerous (Peck, 1969). Increasing the factor of safety will mean driving the piles 85 meters into rock; this was a very expensive option. In addition, embarking in a very meticulous and large subsurface investigation program to accurately assess the properties of the soil was going to be very time consuming and expensive.

The proposed solution was to monitor the pile installation using High Strain Dynamic Tests and adjust the design according to the results. From this test the total pile capacity as well as the contribution of each layer can be obtained. Then, the contribution from the layers that have liquefaction potential can be subtracted from the total pile capacity. This corrected capacity can then be checked with the demand. Since the piles were steel pipe piles, in the case that the desired capacity was not reached, the pile could be spliced and driven deeper. The speed and cost of the tests allowed them to be performed in as much as 3 piles of each pier. Most of the capacity of the pile was predicted to come from the clay layer so most the tests were performed at least a week after installation to allow the dissipation of pore pressures in the soil in order for it to regain strength. The test was repeated in two of the piles in three different occasions to assess the amount of setup of the soil over time. Figure 8 shows effect of setup in the piles.

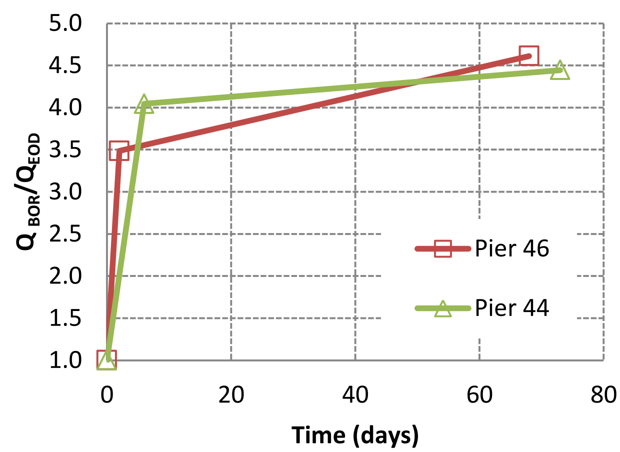

Fig 8. Pile Capacity Gain over Time

It has been shown that all of the elements for the observational method were met, the general nature of the deposit was known but not in detail, the possible unfavorable outcome (not reaching the required capacity) was identified and a possible course of action or modification of the design was available and ready. By carefully monitoring the installation expensive solutions were avoided.

CONCLUSIONS

The background and theory of the High Strain Dynamic Test for determining pile capacity and how it copes with the soil's visco-elastic response is discussed. From the comments about the instrumentation used in the test, it should be noted that different accelerometers may be better suited for different pile hammers.

It has been shown that HSDT can be a very useful tool for monitoring piles and implementing the observational method. The quickness in which the test is performed and the uncomplicated way to setup the test make it a good option for offshore piles or places where the working area is restricted.

Another advantage is that the test is performed in the installed pile without causing any damage. The reduced cost compared to other pile tests allows engineers to constantly monitor the piles and make changes as the construction progresses. Redundancy is becoming more important as we move towards designs based on reliability, HSDT are more suited for multiple tests.

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