Within the structural model, loaded SPMT axle loads were traversed across the deck to simulate a moving load along the path of travel. These loads were applied in combination with the lateral loads from the combi-wall. The resulting bending moments and shear demands on the deck and piles were compared with the capacities of the structural elements calculated in accordance with the current Eurocodes.

Field Program

Based on the results of the preliminary evaluation, it was determined that there was potential to undertake loadouts using SPMTs. However, as there is inadequate historic construction or design information, confirmation of pile lengths, soil properties and stratigraphy, together with material properties of reinforcing steel and concrete were required.

Furthermore, a condition survey of the MOF was required to confirm extent of degradation of the structural elements to confirm present structural capacity of the piles and deck elements.

Over a number of site visits, a field program was completed to obtain this data from site. Access to the soil beneath the deck was gained through concrete coring at locations set out close to the tubular piles and combi-wall piles. The retrieved concrete cores were recorded for presence of reinforcement and laboratory testing to confirm cement content and chloride content.

The soil stratigraphy information was obtained through CPT, boreholes and material sampling for laboratory testing. All Boreholes were left open with sleeves to enable magnetometer and parallel seismic testing to determine embedment length of selected piles.

A visual survey of the structure was undertaken to observe condition of the tubular piles and reinforced concrete deck. Access to the soffit was restricted due to tides and working within a confined space. The adjacent "open pile structure" constructed in a similar period to the proposed MOF was visited to obtain information on the structural degradation of tubular steel and reinforced concrete elements.

Due to access constraints, no inspection was carried out on the anchored combi-pile wall elements. The condition of these structures could not be gauged. Furthermore, material testing of the anchored combi-pile wall was not possible, and as such the yield stress of the materials is not known.

Validation of preliminary evaluation

The data gathered from the field program was used to update the structural and geotechnical models developed during the preliminary evaluation. Non-linear P-y curves were calculated to represent the lateral stiffness of the soil, which were represented in the structural model as multilinear springs.

The geotechnical soil stratigraphy model updated using soil properties obtained from the field tests, is schematically illustrated in Figure 6.

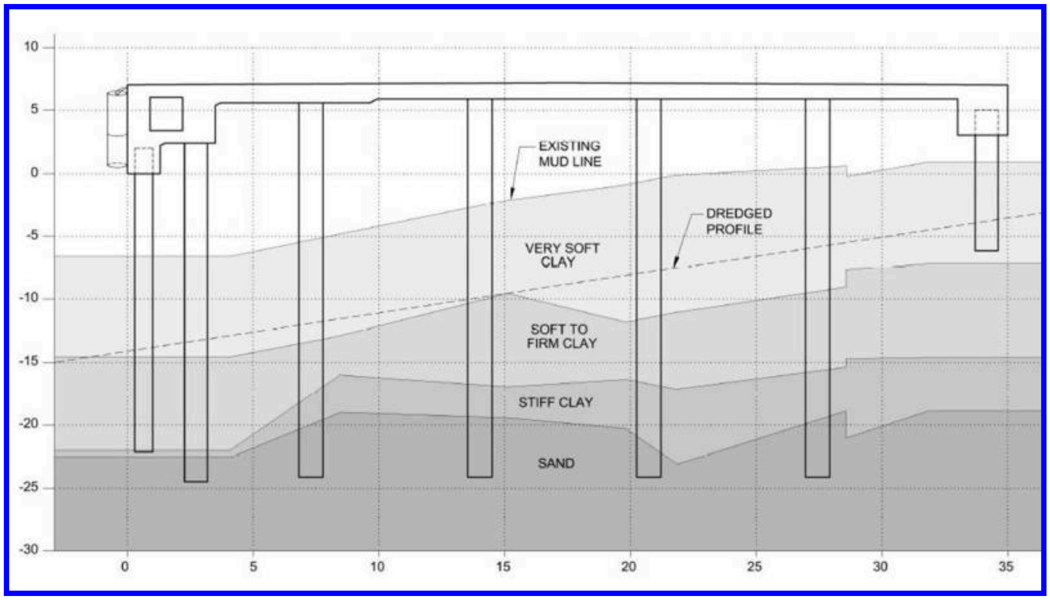

Figure 6 – Soil stratigraphy at the project site based on field tests

The structural models were updated to take into consideration the degradation of the piles. Due to the limited field data, a sensitivity analysis for the effects of corrosion affecting capacity of the pile sections was completed. The recommendations for corrosion rates as per BS EN 1993-5:2007 were adopted.

A schematic representation of the revised structural model is presented in Figure 7.

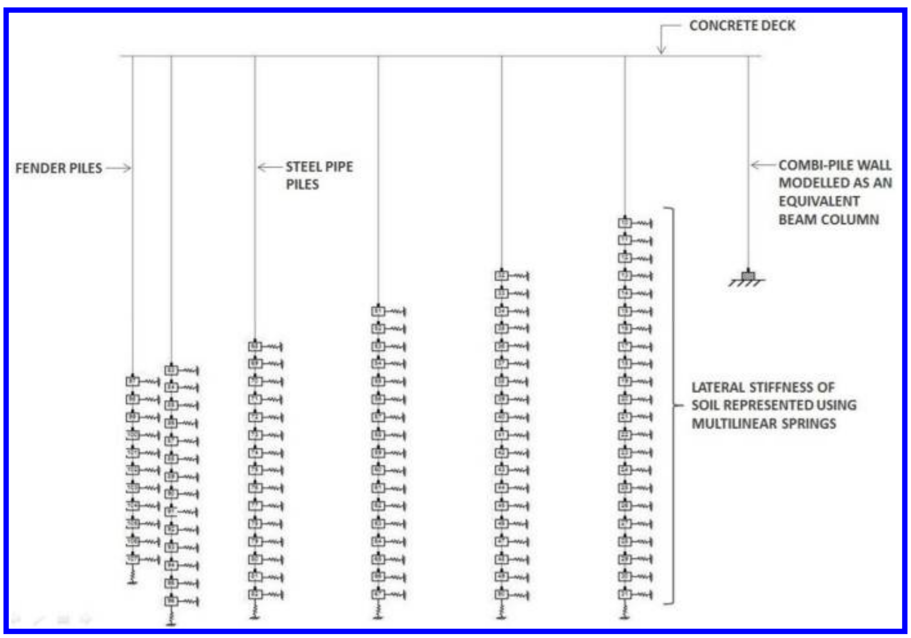

Figure 7 – Schematic representation of finite element model used for validation of preliminary evaluation

Analysis was carried out for the revised model and the results of the analysis are presented in Table 1 and 2.

Table 1 – Comparison of demands on deck from module load out

Description*	Longitudinal Span*		Transverse Span*	
	Column Strip	Middle Strip	Column Strip	Middle Strip
1200 t module				
Bending Moment – Sagging	80	70	76	74
Bending Moment – Hogging	87	38	76	29
Shear Stress	58		65	
830 t module				
Bending Moment – Sagging	81	82	70	69
Bending Moment – Hogging	66	31	56	34
Shear Stress	56		61	

*-All demands are expressed as a percentage of corresponding structural capacities.

Table 2 – Pile Utilization Ratios

Description	Pile Utilization Ratio
1200 t module	0.43
830 t module	0.40

Conclusions

Structural analysis of a potential MOF located in Iraq has been undertaken to determine the potential for load out of modules using SPMTs. Various configurations of SPMT trains have been studied to load out modules of up to 1,200 tonnes in weight.

Limited site specific data or construction information was available to confirm structural capacity of the MOF structural elements: tubular piles, reinforced concrete deck and anchored combi-pile wall.

Provisional studies identified the potential capacity of the MOF to support a uniformly distributed live load of 40 kPa. SPMT configurations were developed to limit the effective ground bearing pressure to less than the permissible 40 kPa. However, several structural elements are loaded close to their capacity, but confirmed potential for the MOF to be used.

Site specific testing was carried out to confirm embedment length of piles and estimate the geotechnical vertical and lateral capacities of the piles and combi-pile wall. Intrusive testing of the concrete deck provided information to confirm that the reinforcement arrangement shown on the reference drawings generally matches the as-built state.

A limited condition inspection of the structure was completed. Based on limited visual inspection of the marine tubular piles and concrete deck soffit of the adjacent marine structure, structural degradation of these elements appears to be low, and serves as a gauge as to the potential condition at the MOF. No inspection has been carried out on the anchored combi-pile wall elements. The condition of these elements has not been gauged. Furthermore, material testing of the anchored combi-pile wall was not possible, and as such the yield stress of the materials is not known.

Evaluation of the capacity of the anchored combi-wall was undertaken using proprietary geotechnical software, while the deck on pile structure was modelled using a proprietary structural analysis programs. The transfer of lateral loads between the two models was achieved using an iterative methodology. Structural degradation of the piles was considered by undertaking a sensitivity analysis.

The MOF structure is limited by the reinforced concrete deck which has a utilization ratio of 87%. The utilization ratio of Combi-pile was found to be 78% and 83% for uncorroded and corroded sections respectively. Therefore, the MOF structure and combi-pile wall have sufficient capacity to support module load out.

However, the ability of the tie rod to withstand the module load out is uncertain as the extent of degradation and yield strength of tie rod is not known.

It has been recommended to carry out further investigation to confirm the extent of degradation and yield strength of tie rods and a load test simulating SPMT loads to verify the existing conditions and address any hidden or latent defects that are currently unknown during structural evaluation.

Reference

BS (British Standard). (2004). *Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges*, Standard BS-EN 1992-2:2003. United Kingdom.

BS (British Standard). (2004). *Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings*, Standard BS-EN 1992-1-1:2004. United Kingdom.

BS (British Standard). (2009). *Eurocode 3: Design of Steel Structures –Part 5: Piling*, Standard BS-EN 1993-5:2007. United Kingdom.

Kinematic Loading from a Structural Perspective

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Abstract

While both ASCE 7 and ASCE 61 require that kinematic (soil movement) loads be considered, they do not provide detailed guidance on the method(s) by which these loads on the structural system can be evaluated. This study discusses the modes of failure observed in real world kinematic movements, the analytical techniques used to evaluate pile-supported structures based on input soil movements, and appropriate performance limits for operational, life safety, spill prevention, and collapse prevention. The combination of kinematic and inertial loads, including simultaneous loading as well as post-kinematic inertial response is addressed. Acceptability of in-ground soil hinging and shearing is discussed in regards to acceptable performance levels. Significant parameters in the evaluation of kinematic loads are also addressed and recommendations are made on appropriate techniques for evaluation of kinematic loading.

INTRODUCTION

When examining a marine structure for seismic hazards, the engineer must consider seismic shaking and ground movement effects which can occur on soft sloping soil. In practice, this ground movement is commonly referred to as “kinematic” movement and is caused by liquefaction, slope instability, or the combination thereof. This movement typically consists of layer(s) of soil which slide past each other downslope, increasing with each seismic cycle cumulatively. There is a limited pool of research studying the cause, prediction, and effects of kinematic movement, and most of this research is focused on the geotechnical aspects of kinematic loading, not evaluation criteria for pile-supported structural systems. This paper focuses on a practical understanding of the evaluation methods, expected performance, and acceptable criteria which should be used by practicing marine structures engineers when evaluating pile-supported marine structures.

CODE GUIDANCE

Typically, practicing engineers would look to available code documents to select a proper approach to evaluating structural performance. In the case of kinematic loading, there is limited available code guidance. For publically accessible structures, ASCE 7-10 is the typically adopted model document. Section 15.5.6.2 states that

"The design shall account for the effects of liquefaction and soil failure collapse mechanisms"; however, detailed guidance is not provided. For private facilities, ASCE 61-14 provides guidance which is more detailed in the geotechnical commentary (C4.7.2), but is limited structurally to Section 4.7.2, which states "Kinematic loading from permanent lateral ground deformation on the foundation shall be evaluated". The commentary (C4.7.2) states that "As a minimum, deformation profiles along the length of the various pile rows should be provided to the structural engineer to estimate strains and stresses in the piles for the purpose of checking performance criteria"; however, it is not clearly stated what strains or shear performance result in acceptable performance criteria. While it may be inferred that the strains and shears evaluated for inertial response should also be used for kinematic loading, there are philosophical differences in the performance for each loading, which are addressed further in this document.

Kinematic loading is addressed by some port and bridge codes in California. Both Port of Los Angeles (2010) and Port of Long Beach (2015) discuss kinematic loading; however, it should be noted that these port codes are typically focused on the response of concrete piled marginal wharfs used for container transport. These codes also provide empirical limits at which the kinematic loading can be ignored. These limits should only be considered where similar well confined reinforced concrete piles are used. California Building Code (CBC) Section 31F (aka MOTEMS, CBC 2013) also provides guidance specific to marine structures subject to kinematic load application as well as in combination with inertial response, as discussed below.

The California Department of Transportation (Caltrans) has sponsored research and developed design criteria for kinematic loading. The Caltrans research is specific to bridge bents, but is applicable to marine structures, especially trestles or other viaducts. The research has examined both the best methods for modeling of soil loading as well as combination of kinematic and inertial loads (as discussed below). A simple single pile bent was examined using nonlinear time history analysis and compared with elastic or nonlinear static evaluations to determine recommended methods and combinations.

KINEMATIC LOAD APPLICATION

Modeling of kinematic loading of marine pile supported structures is a complicated evaluation which requires close coordination of the structural engineer with the geotechnical engineer. Much of the input information used for the evaluation of the structure will be developed by the geotechnical engineer, including the following:

- Soil lateral P-Y spring stiffness
  - May include static, seismic, and/or liquefied strengths of soils.
  - Best estimate (not upper/lower bound) properties should be used for kinematic and combinations of kinematic with inertial, unless determined otherwise by the geotechnical engineer. If upper / lower bound properties are used, they should be applied to the slope stability

analysis as well as the inertial response (thus two failure planes will be developed)

- In some cases, such as structures with batter piles, axial stiffness (skin friction T-Z and/or end bearing Q-W springs) may also be required.
- Slope failure plane surface, typically provided as a curved line through the soil profile which identifies the depth to zero soil movement.
- Displacement magnitude and distribution over the vertical height of the soil column. The taller the distribution of displacement (transition region) along the pile, the lower the shears within the pile at the slope failure plane surface.
- Input on soil layers which have negligible strength and can be ignored. Often weak sedimentary soils within the first few feet of the mudline will fluff into mixture or provide very weak loading of the pile.

There are two basic approaches to structural evaluation of kinematic loads on piles: pressure (force) input of load or displacement input of load. For the pressure input method the pile is modeled with soil springs or effective fixity below the slope failure plane, while above the failure plane the maximum pressure from the P-Y springs is applied directly to the pile, as shown in Figure 1.a. As the P-Y springs typically vary in strength with depth, the pressure distribution on the pile is not linear. At the failure plane the full P-Y spring pressure may be used or a lower pressure associated with smaller displacement within the transitional soil displacement can be used; however, it is typically simpler and conservative to model using the full P-Y load. The displaced shape of the pile will typically take the form of a peak moment (and possible hinge location) just below the slope failure plane and an additional peak moment at the pile to deck connection, as shown in Figure 1.c.

As the pressure method does not capture restraining of the pile movement by the soil above the failure plane, it is generally considered the more conservative methodology. Where soil movement only occurs within a shallow upper layer of soils, the pressure method may be preferable as it is considered easier to perform. The geotechnical engineer should provide input on the most appropriate method to be used in the structural analysis.

The other method for applying kinematic load is to introduce P-Y springs above the slope failure plane which are two node springs. One node of the spring is attached to the pile and the other is displaced per the soil displacement distribution. The displacement of the spring results in a lateral load placed on the pile. This method differs from the pressure method in that the transition region (where soil movements change from zero to the full lateral displacement) may be easily incorporated into the system. The method also accounts for resisting capacity of the sliding soil above the slope failure plane against movement of the top of the pile (which is especially influential for stiff upper crust layers). As the curvature of the pile is typically at or just below the failure plane (especially in the case of a hinge formation), the top of the pile may displace laterally more than the forcing soil displacement. In this case the upper soils will restrain the piles, causing the formation of two hinges within the soil, as shown in Figure 1.d. Typically, formation of the second hinge in the soil will protect the pile to deck connection as a mechanism forms which allows the top of the structure to move with the soil block and effectively see no differential movement.

In the case of structures with only part of the structure within the sliding soils (such as long jetties), the piles outside of the sliding soils may restrain the top of deck above the sliding soils, resulting in high moments at the pile connections (see Figure 2.a, which was originally a plumb pile). In this case, additional moment hinges may occur at the pile to pilecap connection. Where piles occur within and exterior to ground movement, it is critical to model the complete system so as to capture any restraint. In these cases, a 3D model of the entire system is recommended.

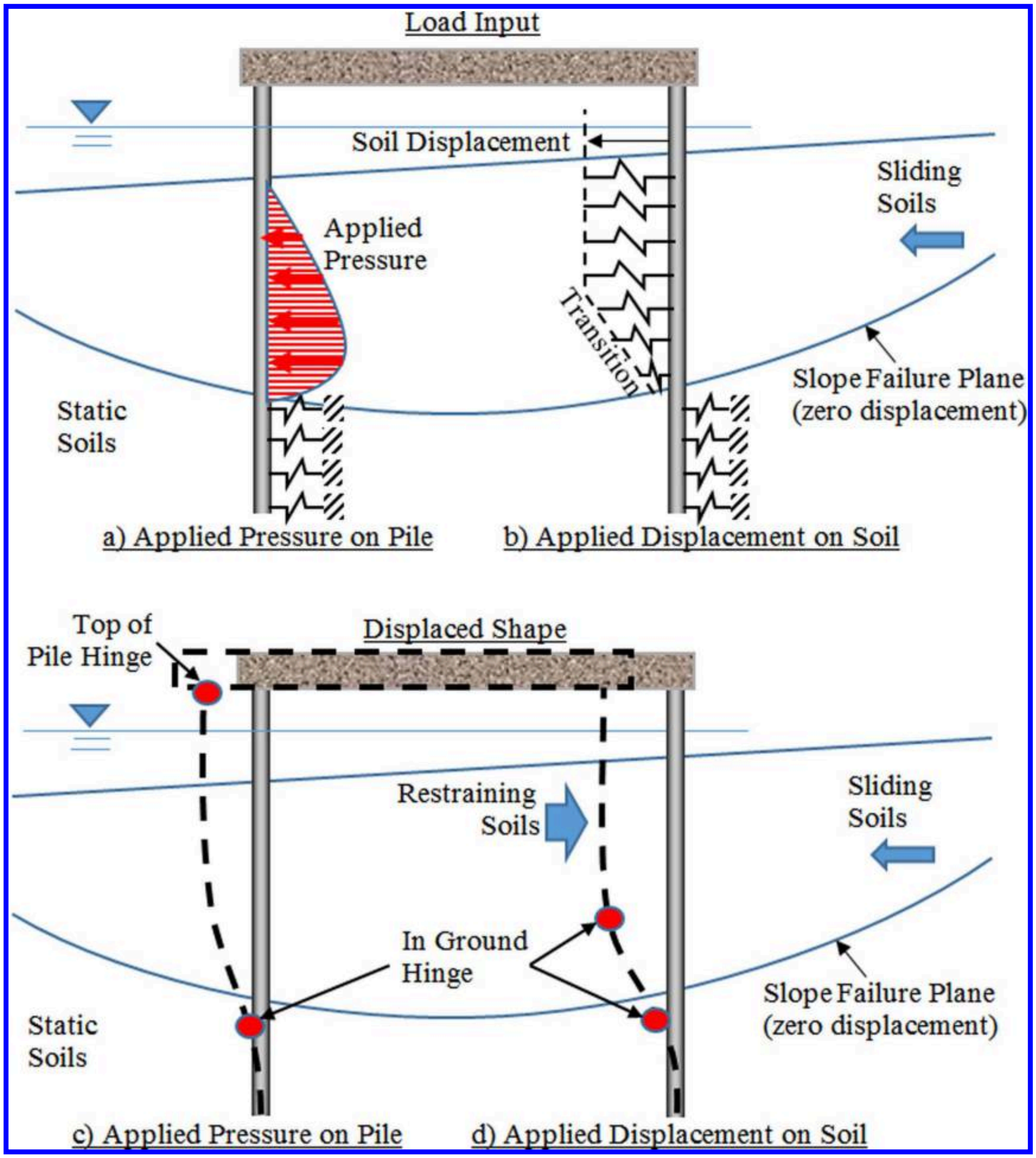

Figure 1: Example of Structural Evaluation Approaches

Modeling of the transition region can be critical for piles with low shear capacity, such as timber or poorly confined concrete piles. A shorter transition region acts to concentrate the shears at the slope failure plane, resulting in higher shears on the pile. A largely distributed transition can be overly advantageous as it may reduce the overall pressure on the pile, though typically the P-Y maximum pressure is reached within less than 1" of lateral displacement of the spring.

Acceptable strain levels are not always stated for each code document; however they may be inferred to correspond to the inertial strain limits stated. However, when even small soil movement (6 inches or less) occurs under a lower level event (such as an operational earthquake event) it may be difficult to satisfy the lower level strain limits since small displacements may result in the maximum P-Y spring pressure. Several current codes (MOTEMS, POLA) provide a lower limit of displacement at which the kinematic movement can be ignored. These limits should be used with caution based on consideration of the redundancy and ductility of the piles. Similarly, shear capacity for kinematic loading is not addressed separately by these codes. Further discussion of the philosophy of damage when strains or shear capacity is overcome within the soil is provided below.

KINEMATIC LOADING RESPONSE

While the analytical evaluation of kinematic loads is relatively independent of the system material, the system response can vary greatly depending on the material type, as shown in Figure 2. While observed damage at the pile to deck connection is typically noted, additional damage within the soil varies based on material.

Steel pile systems tend to demonstrate the greatest ductility and resiliency against kinematic loading. Steel piles have high shear capacity; therefore piles are unlikely to shear rupture at the failure plane, but may form flexural hinges within the soil or at the deck connection. Welds of the pile to pilecap are susceptible to overloading and should be designed as protected elements (weld strength greater than 1.25 to 1.4 of the pile moment capacity). As an example, the pile shown in Figure 2.a shows a plumb pile which was restrained at the deck; therefore the kinematic load has pushed the in ground pile outwards until significant rotation is apparent. The pile to deck connection was heavily damaged, but allowed for gravity load transfer.

Concrete piles show a range of response dependent on the confinement provided within kinematic loading regions. Concrete piles designed under older codes tend to be poorly confined, with low shear strengths. These piles are susceptible to shear rupture within the soil near the failure plane. This shear overload can occur prior to the full moment capacity of the pile and may result in sliding of the entire structure and vertical settlement if the remaining stub pile above the shear failure does not bear on the stub below. Figure 2.d shows a damaged concrete pile supported structure following the Haiti 2010 earthquake, which was likely due to low shear capacity and lack of maintenance. Modern highly confined piles have a significantly better shear capacity which may result in ductile in-ground response; however, depending on the soil conditions, shear rupture of the pile can still occur.

a) Formerly plumb steel pipe pile [ASCE 2013]

b) Ruptured connection at batter pile [GEER 2010]

c) Leaning timber plumb pile structure partially demolished [Committee on the Alaska Earthquake 1973]

d) Formerly plumb concrete pile in Haiti [TCLEE 2012]

Figure 2: Kinematic Loading Failures by Construction Material

Timber piles have a significantly lower flexural capacity than concrete or wood, but have no NDS code specified direct shear capacity. Timber sections are likely to rupture within the ground. Timber piles are typically pinned to the pilecap; therefore the remaining pile above the soil failure plane is only restrained by the upper soils and is relatively free to sway, similar to the damage shown in Figure 2.c If the superstructure is relatively light, as is typical of most timber structures, collapse due to sway and P-Delta effects may not occur even for large displacements. However,