EVALUATION OF MARINE STRUCTURES FOR KINEMATIC EFFECTS

by:

Julie A. Galbraith[1], M. Ali Naeem[2], William M. Bruin[3]

ABSTRACT

In performance-based seismic design, a marine structure is designed for inertial loading effects associated with the dynamic response of the structure. Some waterfront structures also experience seismically induced kinematic effects, associated with soil liquefaction and lateral spreading. This paper explores some of the approaches for incorporating these kinematic effects into the seismic design and analysis of marine structures.

Although most performance-based design provisions for piers and wharves require that kinematic effects be considered in seismic design, there is generally not detailed guidance on how these effects should be considered analytically. In addition, there are various opinions and practices across the industry in accounting for these soil-structural demands. Considerable judgement is required by the design professional in deciding how to include the kinematic loads into the structural analysis.

Combination of inertial and kinematic earthquake effects is one of the major decisions that needs to be made in the analysis and design process. Determination of the coupled load combination can be done with analytical methods or by engineering judgement and experience. Further, how to practically combine the kinematic and inertial loading is also an important decision. This paper discusses three analytical methodologies; including, superposition of results, post-inertial kinematic response, and post-kinematic inertial response. This paper will provide commentary on how these approaches can be implemented, and the advantages and shortcomings of each method.

Important analysis parameters are also discussed; including application of the kinematic loading, soil structure interaction, and modeling of nonlinear structural behavior using pile hinges.

[1] Simpson Gumpertz & Heger, Inc., Oakland, CA, USA, jagalbraith@sgh.com
[2] Simpson Gumpertz & Heger, Inc., Oakland, CA, USA, manaeem@sgh.com
[3] Simpson Gumpertz & Heger, Inc., Oakland, CA, USA, wmbruin@sgh.com

1. INTRODUCTION

Although most performance-based design provisions for waterfront structures require that kinematic effects be considered in seismic design, there generally is not detailed guidance on how these effects should be considered analytically. In addition, there are various opinions and practices across the industry in accounting for how to incorporate these soil-structural demands. Considerable judgement is required by the design professional in deciding how to include the kinematic loads into the structural analysis.

1.1 Description of Kinematic Effects

Kinematic effects in the context of marine and waterfront structures occur when lateral soil movements, typically associated with liquefaction-induced lateral spreading of marine slopes during seismic shaking, load the foundation elements of the structure. A less common occurrence is associated with cyclic plastification of clay-type soils that results in loss of stiffness. Kinematic loading of marine structures, and its effect on seismic performance, has been observed and its significant impact on waterfront infrastructure has been documented following recent earthquakes in Haiti (TCLEE 2012), Japan (COPRI 2014), Chile (COPRI 2013), and New Zealand. Marine structures like marginal wharves or pier landings, which are often founded on marine slopes, are particularly vulnerable to this hazard. For many structures, kinematic displacements can often be the governing seismic load condition, especially when combined with inertial demands. This seismic load condition is dynamic, occurring when a liquefiable layer is mobilized by ground shaking. The magnitude and characterization of the kinematic movement is dependent on project specific factors such as foundation pile density, slope angle, soil layering, cyclic behavior of sloped soils, as well as surcharge loading conditions. Structural response is dependent on the magnitude and depth of soil movements, relative soil layer stiffnesses at the slip plane, stiffness of the non-liquefied or crust layer, as well as inertial response. Because of the influence of these factors, kinematic soils are typically addressed on a project-by-project basis.

1.2 Code Design Requirements

At present, performance-based seismic design standards for marine and waterfront structures (ASCE 61-14, CBC 2016, POLA 2010, POLB 2014) require the consideration of seismically induced kinematic loading in the design of foundation systems. These standards to some degree imply that coupling of the inertial and kinematic loads should be performed, but no specific guidance is provided. Exact phasing of the kinematic slope mobilization with respect to the inertial shaking is not well known nor is there a professional consensus about how to combine these conditions. As is reported in Dickenson et al. (Dickenson 2016), scaling factors for combining the uncoupled inertial response with the uncoupled kinematic response range from 20% to 100%, provided by various guidelines, for surface transportation applications. Dickenson also concludes that the selection of weighting factors for combining kinematic and inertia loads should be done judiciously.

2. COMBINATION WITH INERTIAL RESPONSE

As noted, deciding whether and how to combine kinematic effects with inertial demands is a complicated issue, with minimal research, and currently no specific standard for waterfront structures. The first consideration for the design professional is how the occurrence of liquefaction correlates with ground shaking during the earthquake event. Generally, some level of strong ground motion must occur to initiate kinematic soil displacements, but the level of shaking required is variable and dependent on site-specific conditions. Additionally, the ground deformation from liquefaction is typically not an instantaneous motion, but rather a ramping up of movement to the maximum displacement as the earthquake shaking continues. The responses can be coupled, but generally not in a manner where the maximum magnitudes of both effects occur simultaneously.

Figure 1 illustrates the development of kinematic ground displacement over the duration of a typical earthquake time history. The dark smooth line in the displacement curve reflects the kinematic portion of soil movement. This example shows how the peak ground acceleration (at approximately 23 seconds) is not concurrent with the peak kinematic displacement.

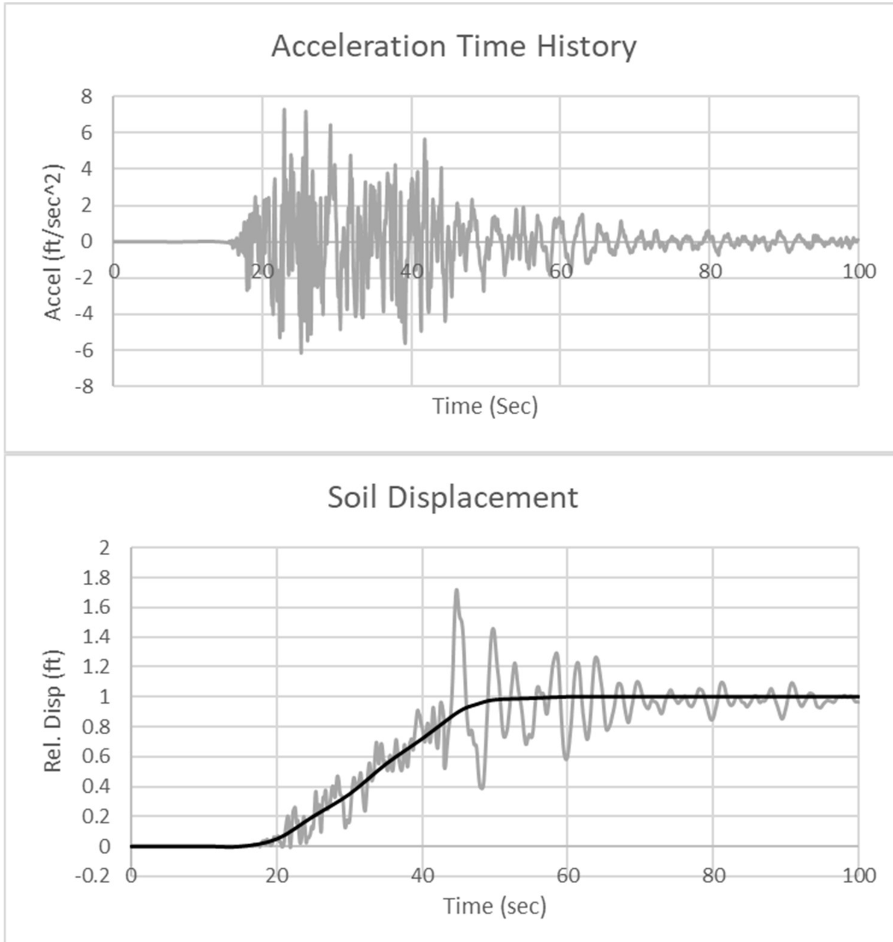

Figure 1: Example Soil Displacement During Acceleration Time History

Most design standards acknowledge some coupling and recommend that a combination of kinematic and inertial earthquake effects be considered. However, what percent of the full inertial demand and what percent of the full kinematic demand to be used in combination is open for interpretation and typically addressed on a project-by-project basis.

Performing time-history analyses on a soil profile model is one method for understanding the degree of coupling and the determination of combinations. The geotechnical and structural engineer can identify combinations by comparing a time history acceleration to the resulting kinematic motion, in terms of a percentage of the simultaneous maximum of each. Common points to identify are at the peak acceleration as well as at the peak kinematic displacement. Intermediate combinations may be worth consideration as well. For example, referring again to Figure 1, a combination of 100% inertial plus 20% kinematic could be interpreted at approximately 23 seconds; and 100% kinematic plus 30% inertial at approximately 50 seconds. A third combination of 75% inertial plus 80% kinematic at approximately 42 seconds may also be relevant for this ground motion. This process could be completed for multiple earthquake ground motions or based on an average of a ground motion set.

These time-history comparisons would be used only for determining the coupled load combinations; a nonlinear static pushover analysis and response spectrum curve would still be used for calculating inertial demand displacements and assessing structural performance. This procedure, although a very credible approach for estimating load combinations, is not commonly used, mainly because of the significant level of effort in performing these time histories. More commonly, geotechnical engineers may provide load combinations based simply on engineering judgement, with knowledge of soil type and probable earthquake scenarios. Some designers may simply choose to combine 100% kinematic and 100% inertial demands, recognizing that this is the most conservative assumption of load coupling.

The second consideration for the design professional in developing combined kinematic and inertial load combinations is recognizing that the damage from each loading type often does not occur in the same location of the foundation structure. For example, under inertial earthquake demands, the pile hinge is typically at the deck connection and/or the mudline elevation, as shown in Figure 2. Under kinematic loading, the hinge usually forms in the soil just below the liquified layer, where the stiffness of the underlying soil is significantly higher than the liquefied layer. For this configuration, it is rational to not combine the two effects; the maximum loading and inelastic hinging for each condition is occurring at completely different locations on the pile, suggesting not much coupling of response with respect to global seismic performance.

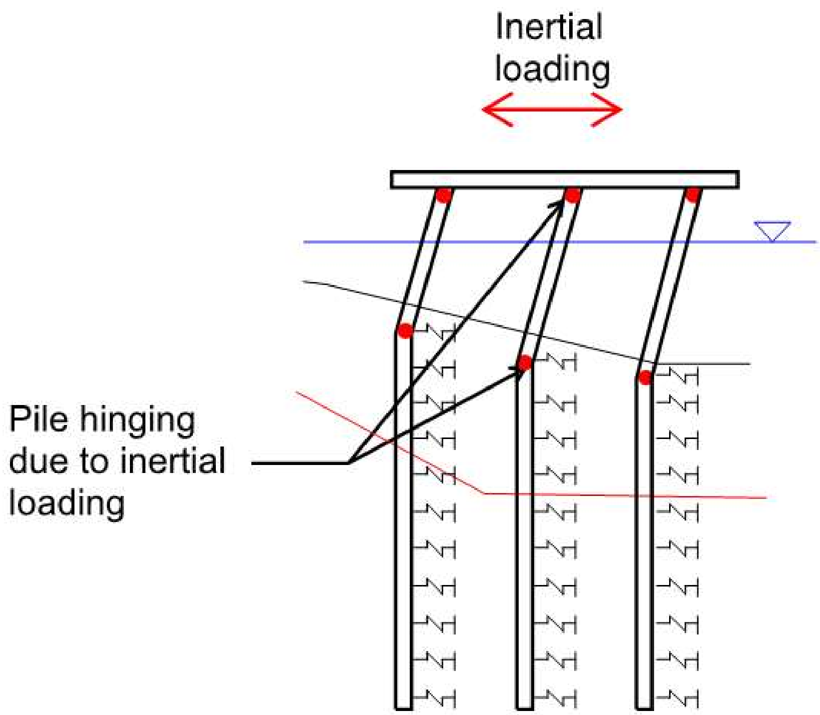

Figure 2a: Typical Pile Hinge Locations for Inertial Loading

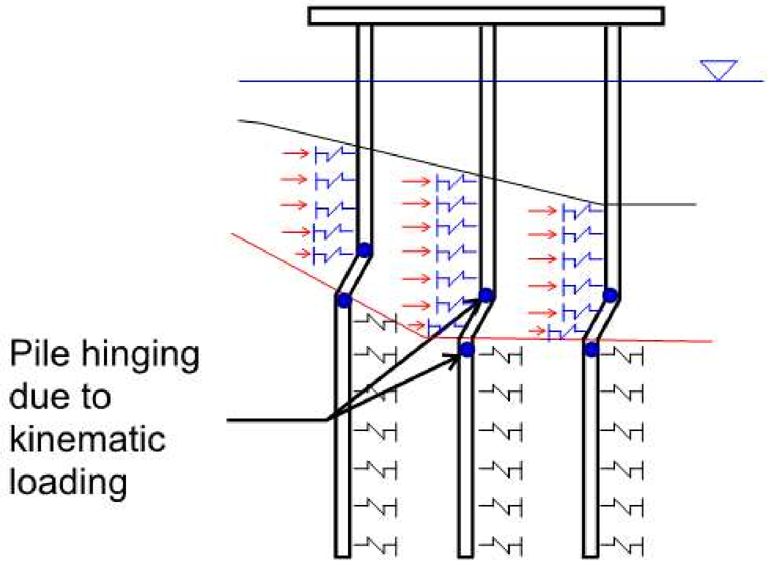

Figure 2b: Typical Pile Hinge Locations for Kinematic Loading

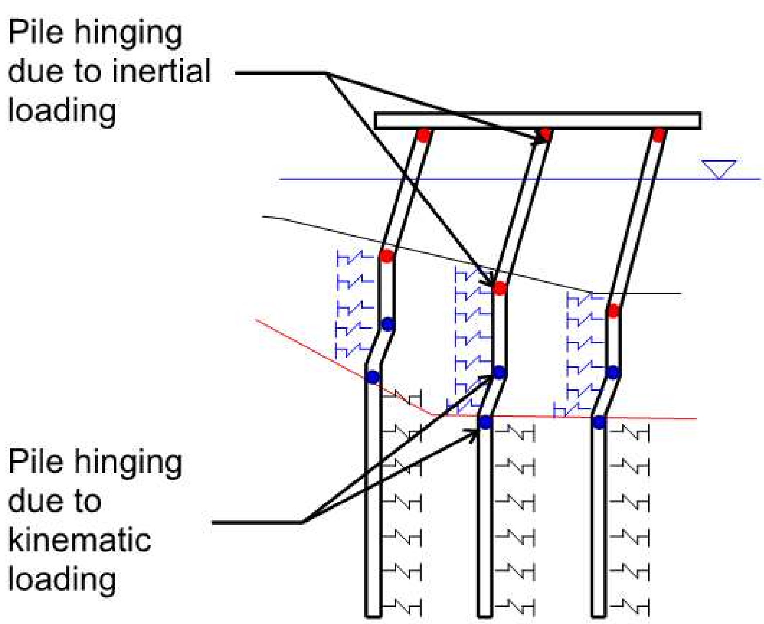

Figure 2c: Combined Pile Hinge Locations for Inertial and Kinematic Loading

When choosing to combine kinematic and inertial responses, there are multiple methods with varying capabilities and limitations. The concepts behind each of these methods, and the advantages and disadvantages of each, are discussed in the following subsection.

2.1 Superposition Method

The superposition method is the easiest method for combining inertial and kinematic effects, from an analytical standpoint; but it is also the least accurate, and generally provides very conservative results. This method allows the design professional to perform independent, uncoupled analyses for inertial loading and kinematic loading conditions. The results, in terms of component demands, are then combined additively. This typically results in an overestimation of pile flexure because each analysis starts from zero displacement, ignoring the preloaded wharf state that is accounted for in the other methods, and therefore moving the pile up the elastic portion of the moment-curvature response twice, versus the lower slope of the secondary stiffness portion of the curve.

Also, with this approach, the inertial demand is based on a non-liquified, or static, soil stiffness. For many wharf structures the soil stiffness is significant in computation of the demand displacement (which is a function of the effective period squared and the spectra acceleration at the initial stiffness). In some cases, the difference between static soil stiffness and liquified soil stiffness can result in significant differences in computed demand displacement.

For structures that remain elastic, the superposition of element forces is a reasonable approach. However, in most cases marine structures experience nonlinearity during the design earthquake; and this method will overpredict demands for those piles. Typically, the authors do not use the superposition method in practice, and instead opt for a combined analysis approach.

2.2 Post-Inertial Combination

This method follows the premise that significant ground shaking will occur prior to the liquefaction of the soil. However, this method is somewhat complicated to model, and requires a bit of uncertainty in the assumptions. Soil springs used to represent lateral stiffness of the soil change between initial ground shaking and after entering a liquefied state. This behavior can be accounted for in a more sophisticated structural model that includes staged loading (sometimes called ‘construction staging’) and changes in the support restraint conditions. An inertial demand can be applied with one type of soil spring active, imparting loads and displacements on the structure. This requires the engineer to have already determined the demand displacement. Then, the soil springs can be changed, and kinematic displacements added to the already displaced structure.

The fundamental issue with this method is that calculated inertial demand displacement is not a residual displacement, but an estimate of max displacement during a design seismic event. Applying the kinematic demands to the inertial demand displacement can be quite conservative. A better implementation of this method would be to apply kinematic displacement to the residual inertial displacements, but the residual displacement is typically not calculated in a pushover type of analysis. Due to the complicated nature, and uncertainties involved, the authors generally do not use this approach. The preferred approach of applying inertial demands to a post-kinematic analysis is described in the next section.

2.3 Post-Kinematic Combination

In a broad sense, this method involves application of kinematic demands prior to inertial demands. This method is commonly used in the industry and provides results that best represent actual behavior, short of performing numerous dynamic time-history models. This approach utilizes a single structural model with soil springs representative of a liquified condition. The kinematic displacements (or pressures) are first applied to the structure as a nonlinear static analysis. This will result in a structural displacement that preloads the foundation. The inertial pushover is then started from the end of the kinematic load case. Demand displacements are calculated from the pushover curve using either the Substitute Structure approach (ASCE 61-14 and CBC 2016) or the Coefficient Method (CBC 2016), just as would be done for a site without kinematic effects, except that an initial displacement is present at the start of the pushover. The calculation of displacement demand is simply adjusted to account for that initial displacement associated with the kinematic effects. An illustration of this pushover analysis is shown in Figure 3. Inertial demand displacement is calculated as a function of effective stiffness (K_{eff}) and is added to the kinematic displacement to determine the total demand displacement. For further information on the pushover procedures, refer to the relevant standards.

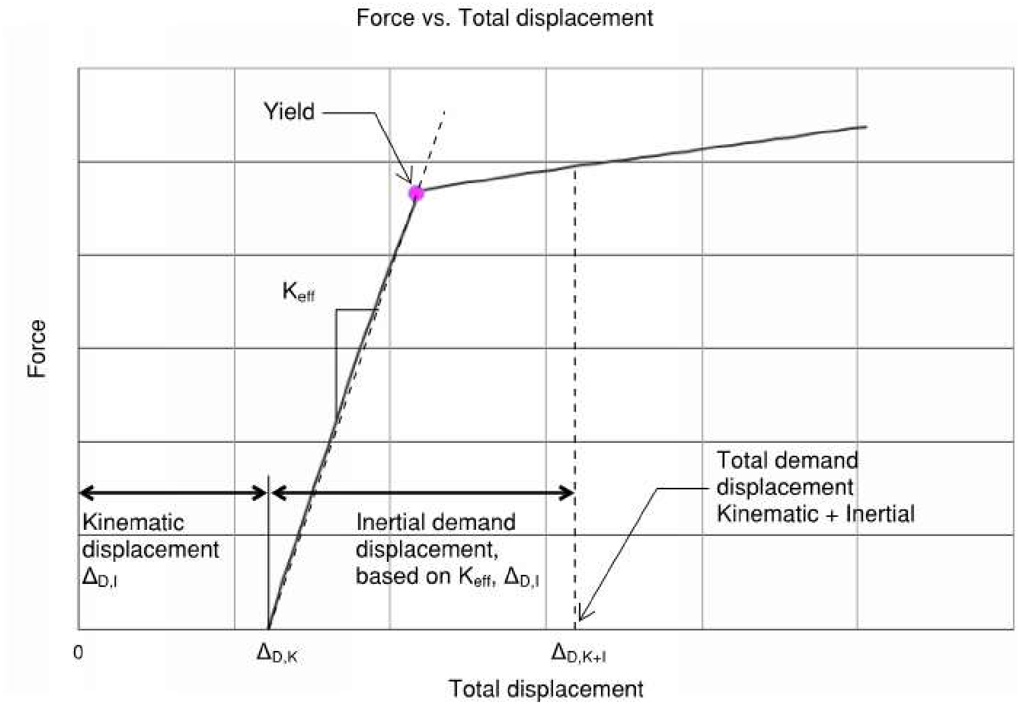

Figure 3: Pushover Curve for Kinematic plus Inertial Analysis

This method also allows for easy adjustments to the magnitude of either kinematic or inertial forces. The percent of inertial demand can be adjusted simply by scaling the response spectrum within the pushover demand procedure. The percent kinematic demand can be adjusted by the magnitude of force or displacement applied in the initial step. For example, if the geotechnical engineer has recommended combining 100% kinematic plus 50% inertial demand; the response spectrum acceleration is multiplied by 0.5 within the demand displacement equation. The actual percentages used in these combinations is determined based on considerations of the site-specific parameters and engineering judgement, as discussed earlier in this paper.

3. MODELING FOR KINEMATIC ANALYSIS

This section has general recommendations for analyzing seismic inertial and kinematic loading.

3.1 Soil Spring Definitions

Soil-structure interaction on a pile supported structure is typically modeled with multilinear springs. The spring properties depend on several factors including soil type, pile cross section, depth, and loading scenario. The geotechnical engineer provides the soil spring stiffness parameters and accounts for the uncertainty of the soil when developing the springs. Seismic inertial analysis is typically performed with both upper bound and lower bound sets of soil spring parameters, whereas kinematic analysis only requires median or best-estimate soil springs.

The best-estimate springs are used for the kinematic condition as they generally are the upper bound stiffness for the liquified soil condition, which is responsible for initiating the lateral spreading and kinematic loads. Stiffer springs (or the inertial upper bound ones) are considered to be an unrealistic load condition as these stiffer soils are not likely to liquefy and produce kinematic load conditions. Lower bound springs are generally not considered in the kinematic analysis despite predicting larger slope movements as these softer springs tend to also have less resistance, resulting in lower demands on foundation elements.

Horizontal (P-Y) soil springs provide lateral resistance to plumb piles. Battered piles, which resist lateral loads primarily through tension and compression are modeled with axial (T-Z) springs that represent skin friction on the circumference of the pile. End bearing resistance can either be modeled with a one-directional (Q-Z) spring or a simple vertical support, as recommended by the geotechnical engineer.

Analysis can be sensitive to soil spring spacing and element discretization. Spring spacing should be reduced near locations of anticipated pile hinging such as the mudline elevation and the interfaces of soil layer changes to improve discretization of the foundation system analytical model.

3.2 Application of Kinematic Loads

Kinematic loads can be applied indirectly as fluid pressures or as support displacements on soil springs, as illustrated in Figure 4 and Figure 5, respectively. Both approaches require modification to the soil springs from their static condition, within the liquified soil layer. If soil displacements are expected to exceed the plasticity of the soil, it is a reasonable simplification to apply the soil pressure as the maximum fluid pressure from the liquified layer (and any material above it) and remove soil springs completely within that layer.

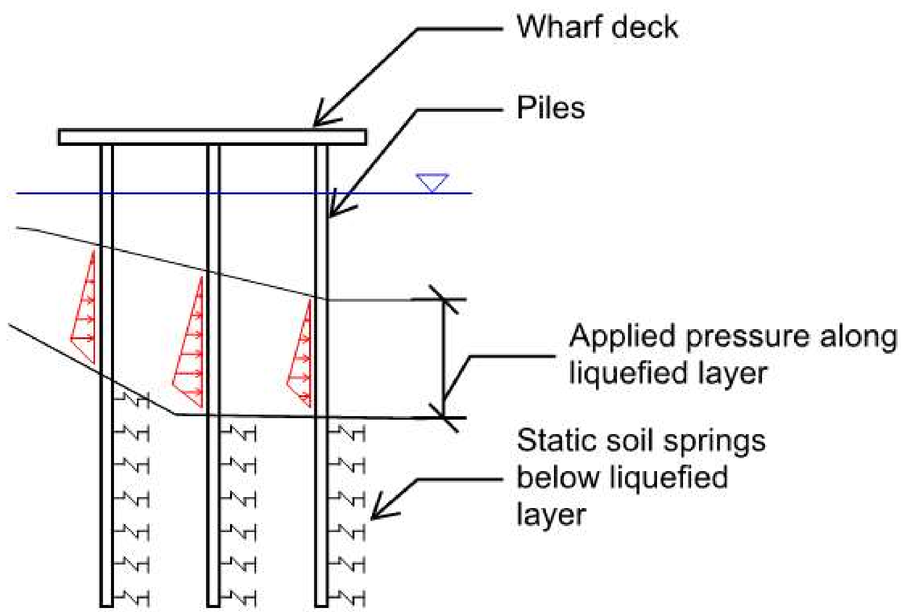

Figure 4: Kinematic Load Applied as Fluid Pressure

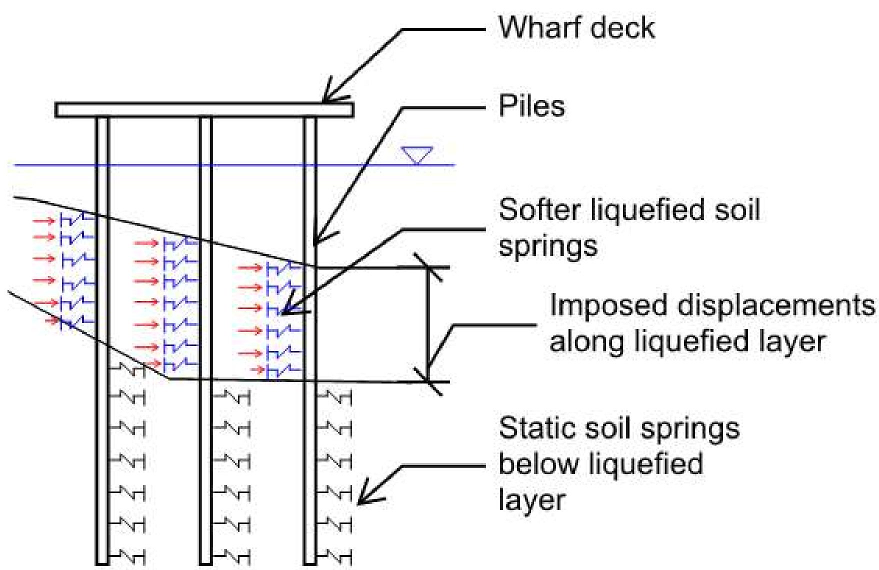

Figure 5: Kinematic Displacement Applied at Softened Soil Spring

The geotechnical engineer provides the kinematic load effects and the recommended soil spring parameters. Their geotechnical analyses may provide complex soil displacement profiles, but these are commonly simplified to a multi-linear profile. Similarly, soil springs are typically modeled as a symmetric tri-linear curve.

3.3 Pile Hinges

Pile hinges are applied consistent with general practice for inertial loading. Hinges are recommended at locations of anticipated damage such as the pile-deck connection, mudline location and soil layer changes. Pile hinges are developed based on cross sectional moment curvature response at relevant axial loads. Performance points can also be defined within the hinge to monitor performance. The limit states are defined by code prescribed material strain limits to represent different damage criteria, such as immediate occupancy, life safety, and collapse prevention. Based on project criteria, these hinge states can also be compared to different demand displacements using corresponding levels of seismic response spectra. An example would be checking "life safety" strain limits for the high intensity but low probability earthquake but limiting strain limits to "immediate occupancy" for moderate intensity but more frequent earthquakes. The recurrence interval that corresponds to each limit state is typically defined in code criterion.

A recommended approach is to first perform separate inertial and kinematic analyses to determine the location of hinging and overall utilization under separate seismic effects As noted above and shown in Figure 2, the first instance of hinging typically occurs in different locations. The compound effects of the kinematic and inertial combination depend on the relative proximity of the expected hinges. In many instances there is only a minor impact of combining inertial and kinematic effects due to the separation of hinge points. However, there are also cases where the in-ground inertial and kinematic hinges occur at close proximity and thus the combination has a more significant impact.

3.4 Non-ductile elements

Non-ductile components such as pile caps, decks and shear in piles should account for over-strength in pile flexural hinge capacity. The over-strength of the pile in flexure is calculated using upper bound expected material strengths instead of design strengths. Piles can have significant over-strength in flexure, which increases the demands on non-ductile components. This approach is consistent with capacity design where the goal is to have a ductile response mechanism instead of a brittle or sudden failure. Piles with a short freestanding height, and piles which penetrate a suddenly dense layer, such as a rock dike are more susceptible to shear demands from either inertial ground shaking or kinematic soil displacement.

4. CONCLUSION

This paper describes some rational approaches for analyzing waterfront structures for kinematic effects, as well as presents some tips for modeling this behavior using a finite element analysis program.

Kinematic load conditions are coupled with inertial loads and need to be considered in combination. To better predict an appropriate degree of coupling, a geotechnical time history analysis may be used to estimate load combinations for various earthquake ground motions for a specific site. However, since this approach involves significant time and resources, a combination is often selected by engineering judgement and experience considering the specifics of the project site.

Three methods of combining the kinematic and inertial effects are discussed herein. The preferred approach is a post-kinematic inertial combination, which involves an inertial pushover analysis using a starting displacement reflective of kinematic effects.

Kinematic effects can be incorporated into a finite element analysis as either displacements or fluid pressures. Some other key modeling parameters, discussed herein, are soil springs, pile hinges, and the consideration of non-ductile components.

5. REFERENCES

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