Lateral Load Analysis of Waterfront Structures Supported on Plumb Piles

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ABSTRACT:

Analysis of wharves/piers under lateral loads is often conducted by assuming the piles to be fully restrained at a presumed distance below the mudline. Once a reasonable point of fixity for each pile is estimated, the wharf/pier is modeled as a one-story structure comprising a slab supported by columns of different heights that are fully restrained at their bases. Strictly, this simplified approach is valid only if the response of both the soil and the piles are linear because the location of the point of fixity varies as the pile and/or the supporting soil softens with increase in lateral displacements. However, representing the flexibility of the supporting soil by assuming the piles to be restrained at invariable points of fixity may produce plausible results in relation to elaborate analysis methods.
In this paper, case study of a marginal wharf structure supported on plumb piles is presented. The lateral resistance of the wharf computed from static lateral load (pushover) analysis was compared with the demand estimated using low to moderate seismic design spectra. It was demonstrated that, for the earthquake demands considered, modeling the soil-pile support by using points of fixity instead of a coupled nonlinear system leads to overestimation of the lateral displacement demand by only 20 percent.

KEYWORDS: displacement-based design, wharves, piers, numerical modeling, simplified approach.

1. INTRODUCTION

The behavior of pile-supported marginal wharves under lateral loads is highly dependent on the supporting soil conditions. For decades, the effect of soil stiffness/strength on the lateral response of wharves has been taken into account by estimating the locations at which the piles are assumed to be fully restrained against rotation. Once a reasonable point of fixity is estimated for each pile, wharf structures were analyzed as one-story structures comprising a slab supported on columns of different heights that are fixed at their bases. The main reason for the wide use of this simplified approach had been the lack of nonlinear elements in commercial structural software.
The development of lateral load-displacement relations for soils, typically referred to as the p-y curves, allowed engineers to obtain more rational estimates of the point of fixity for piles. Commercial software developed for geotechnical analysis of deep foundations under lateral loads enabled modeling the soil as a series of nonlinear spring elements that provide lateral restraint for the piles. Although, the notion of point of fixity is a useful device for treating piles as columns, it should be recognized that the so-called point of fixity of a pile does, in fact, vary.

The lateral secant stiffness of the plumb pile embedded into a soil mass, K_{lat}, can be obtained by imposing displacement imposed at the top, Δ_{top}, and calculating the corresponding shear force required at the top V_{top}. Using these results, the lateral stiffness can be estimated as follows:

$$K_{lat} = V_{top}/\Delta_{top} \tag{1}$$

If it is assumed that the pile remains elastic -with a flexural stiffness of EI- and fixed against rotation at the top, an equivalent column length, L_{eq}, that would yield a lateral stiffness equal to K_{lat} can be calculated using the following equation:

$$L_{eq} = \{(K_{lat}) / [12*(E*I)]\}^{1/3} \tag{2}$$

It is apparent from Eq. (2) that even if the pile was to remain elastic, the depth to the point of fixity would increase as imposed lateral displacements increase. This is because as the supporting soil softens under larger imposed displacements, greater soil mass is required to restrain the pile. Similarly, for the same soil conditions and the same imposed displacement, inboard (shorter) piles would have a deeper point of fixity than outboard (longer) piles. Given that the location of the point of fixity varies with the displacement demand, determination of a simplified approach for obtaining reasonable estimates for the lateral force-displacement response of wharf structures is essential for preliminary design.
In this manuscript, a marginal wharf was analyzed under low to moderate design seismic demands by treating the structure as a one-story building with columns of different heights that are fully restrained at the selected points of fixity. Results from this simplified analysis were compared with those from a more detailed numerical model in which the non-linearity of the supporting soil and the structure were taken into account simultaneously.

2. DESCRIPTION OF THE CASE STUDY

2.1. Geometry

The case study structure is a pile-supported marginal wharf with a total berthing length of 560 m. The framing system consists of longitudinal pile caps that support precast concrete panels spanning in the transverse direction of the structure as shown in Figures 1.a) and 1.b). The panels and the pile caps are provided with sufficient transverse reinforcement projecting into the cast-in-place (CIP) concrete topping to ensure composite behavior and diaphragm action for the superstructure.

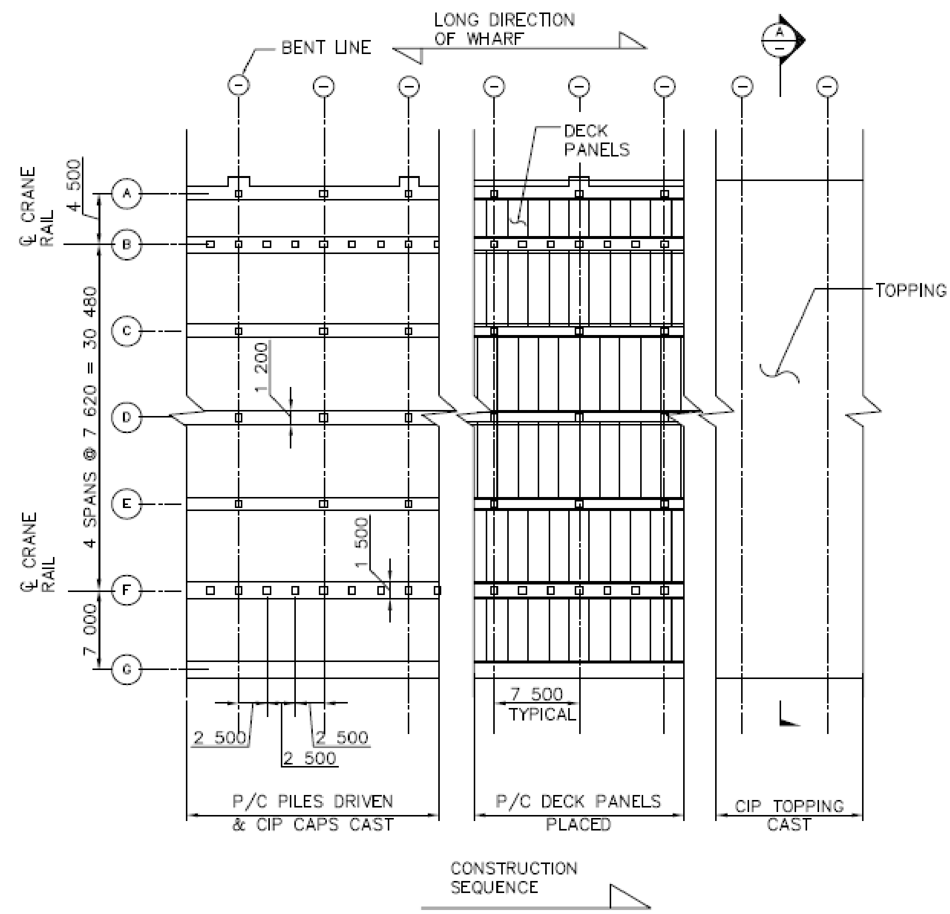


Figure 1.a) Plan of the case study structure

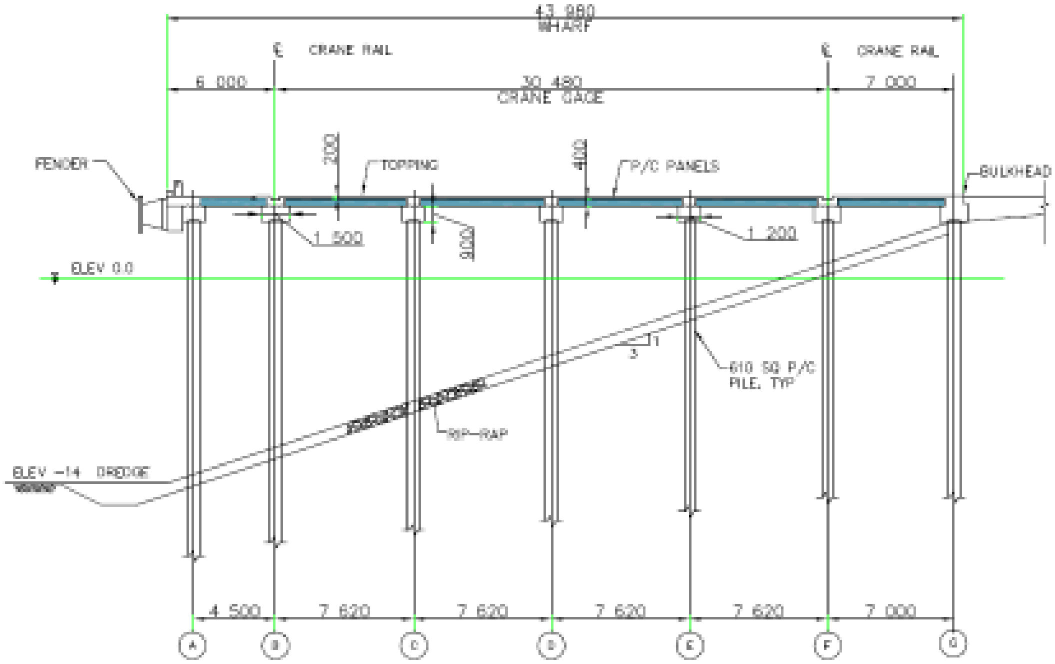


Figure 1.b) Transverse section (A-A) of the case study structure

2.2. Soil Properties and p-y Curves

For simplicity, soil layers were assumed to be parallel to the mudline in the transverse direction of the wharf. The soil properties and layer thicknesses, and the corresponding parameters used in order to estimate the p-y curves for the soil at various depths are listed in Table 1. The numerical model developed by Matlock (1970) was used in the computation of the p-y curves for soft clays, whereas the numerical model developed by Reese et al. (1974) was used for modeling of sand.

Table 1 Base soil properties/parameters

Soil layer	**Soil ype**	**Thickness**	**Effective unit weight**	**Lateral subgrade modulus (static)**	**Undrained cohesion**	**Strain factor**	**Friction angle**	**p-y model used**
		(m)	(kN/m^3)	(MPa/m)	(kPa)	(e_{50})	(degrees)	
I	Soft Clay (ML)	5.0	5.8	7	10	0.02	0	Matlock (1970)
II	Soft Clay (ML)	4.0	6.2	8.1	20	0.02	0	Matlock (1970)
III	Stiff Clay	4.0	6.2	27	40	0.005	0	Matlock (1970)
IV	Stiff Clay	6.0	6.2	136	60	0.005	0	Matlock (1970)
V	Sand	-	8.4	33	N/A	N/A	36	Reese et al (1974)

The calculated p-y curves for the upper two layers of soil are shown in Figure 2. These curves were assumed to represent the expected soil conditions at the site, which will be referred as to the “base soil” conditions. Recognizing the uncertainties involved in the estimation of soil lateral resistance, two additional sets of p-y curves were obtained in order to establish upper and lower bounds for the estimated displacement response of the structure. The first additional set of p-y curves, representing “strong soil” conditions, was obtained by multiplying the “p” values for the base soil by a factor of two. Thus, the lateral resistance and stiffness of the strong soil are twice as much as those of the base soil. The second additional set of p-y curves, representing “weak soil” conditions, was obtained by dividing the “p” values for the base soil by a factor of two. Hence, the lateral resistance and stiffness of the weak soil is one- half of those of the base soil.

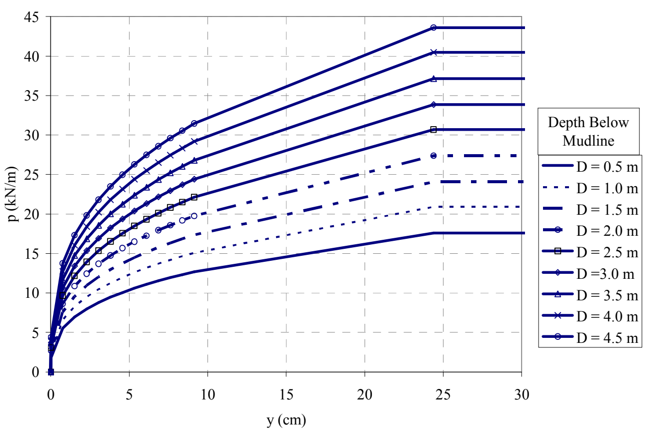


Figure 2.a) Calculated p-y curves for soil layer I

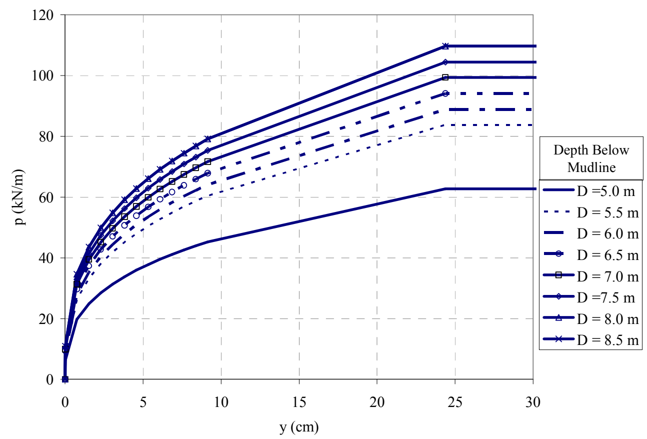


Figure 2.b) Calculated p-y curves for soil layer II

2.3. Pile Properties

The plumb piles used in the case study structure consist of 610-mm-square precast members prestressed through 24 low relaxation strands of 12-mm-diameter and an effective pre-stress of 1,100 MPa (after losses). The 28-day concrete compressive strength was assumed as 42 MPa and the modulus of flexural rupture was taken as 4.0 MPa. Each pile was assumed to be embedded into the pile cap by 75 mm and connected to the superstructure through 8-D25 dowels with nominal yield strength of 420 MPa.

Taking into account the weight of the superstructure and an additional live load of 4.5 kN/m^2 assumed to be present during an earthquake, the cracking moment for a typical pile was calculated as 510 kN-m, while the cracking moment calculated at the pile-to-pile cap connection was 240 kN-m. Furthermore, the yielding moment computed for the pile was 750 kN-m, whereas that for the pile-to-pile cap connection was 510 kN-m.

2.4. Design Acceleration

The elastic design response spectrum used in the analyses is consistent with that recommended by ATC-40 (1996). The spectrum is defined in terms of the parameters C_a and C_v, which represent the effective peak ground acceleration, and response acceleration for a system with a period of one second. C_a and C_v are both expressed as a fraction of the acceleration of gravity, g. The analyses were conducted for two extreme levels of ground motion demands shown in Figure 3: (i) Level 1, intended to represent low seismic demand, for which $C_v = C_a = 0.1$, (ii) Level 2, intended to represent moderate seismic demand, for $C_a = C_v = 0.5$.

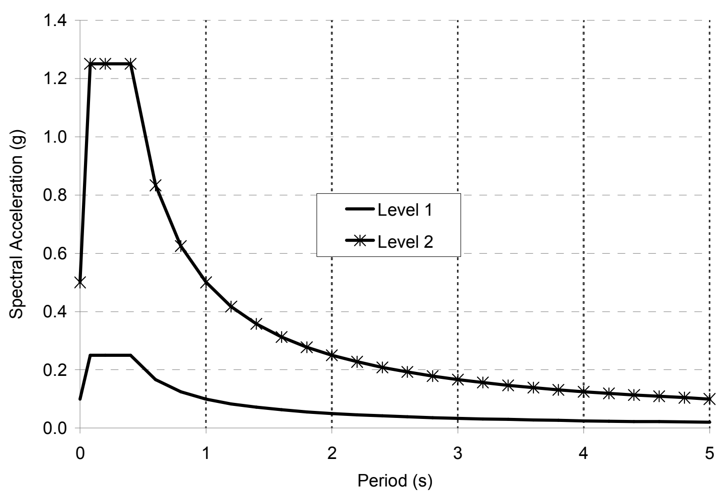


Figure 3 Design acceleration spectra

3. ALTERNATIVE ANALYSIS METHODS

In this study, two alternative methods were used in order to conduct static lateral load-displacement (pushover) analyses in the transverse direction of the wharf. Because the structure is long and modular, only one bent was modeled using the structural analysis program SAP2000®. The piles were modeled as elasto-plastic flexural elements with a yielding moment of 750 kN-m along their, and 510 kN-m at the pile-to-pile cap connection. The superstructure was modeled as a beam element with the combined cross-sectional properties of the pile cap, precast panels, and topping. The weight of the superstructure was calculated as 7,000 kN per bent (i.e., per 7.5-meter-long wharf segment), which includes a 4.5 kN/m^2 uniform live load as mentioned previously.

3.1. The Detailed Analysis Method

In the detailed analysis method, the piles were modeled as frame elements connected to lateral nonlinear springs spaced at 0.5 m along the upper 9.0 m below the mudline (top two layers of soil), and at 1.0 m beyond the upper 9.0 m (Figure 4.a). The nonlinear soil springs were modeled using the p-y curves computed at relevant depths.

3.2. The Simplified Analysis Method

In the simplified analysis method, the structure was modeled as a one story building with columns of different heights that are fully restrained at the presumed points of fixity (Figure 4.b). The height of each column was estimated using L-Pile®, a commercially available program for the analysis of piles under lateral loads. The piles were modeled using linear-elastic uncracked members assumed to be fixed against rotation at the top and embedded into the soil layers summarized in Table 1. Lateral displacements were imposed at the top of the pile until the maximum in-ground moment reached the cracking moment capacity of the pile (M_{cr} = 510 kN-m). The equivalent pile height to the point of fixity, L_{eq}, was then computed by substituting Δ_{top} at M_{cr} and the corresponding V_{top} into Eqs. (1) and (2).
Conservative estimates for the lateral stiffness of the soil-structure system are expected to be obtained by using points of fixity computed as described above. For a pile subjected to lateral loads, the restraint provided by the superstructure is higher than that provided by the supporting mass of soil. Therefore, the bending moment demand at the top of the pile is greater than the in-ground moment. Moreover, the cracking moment capacity at the pile-to-cap connection is usually less than that for the prestressed pile itself. Consequently, cracking at the top of the pile will occur prior to in-ground pile cracking, which results in a more flexible pile that requires less soil mass to be fully restrained than that estimated using the simplified method.

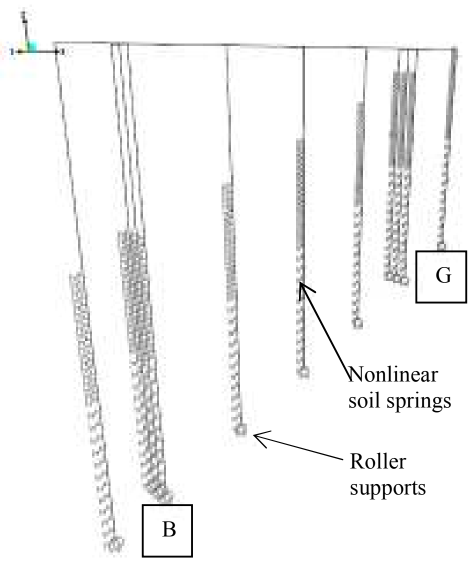


Figure 4.a) Detailed structural model

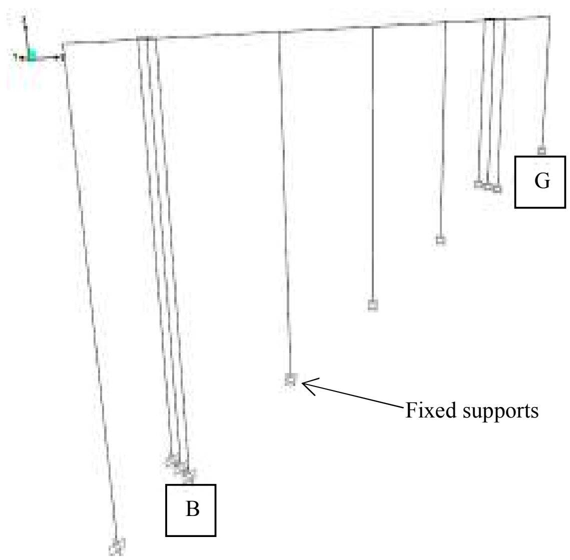
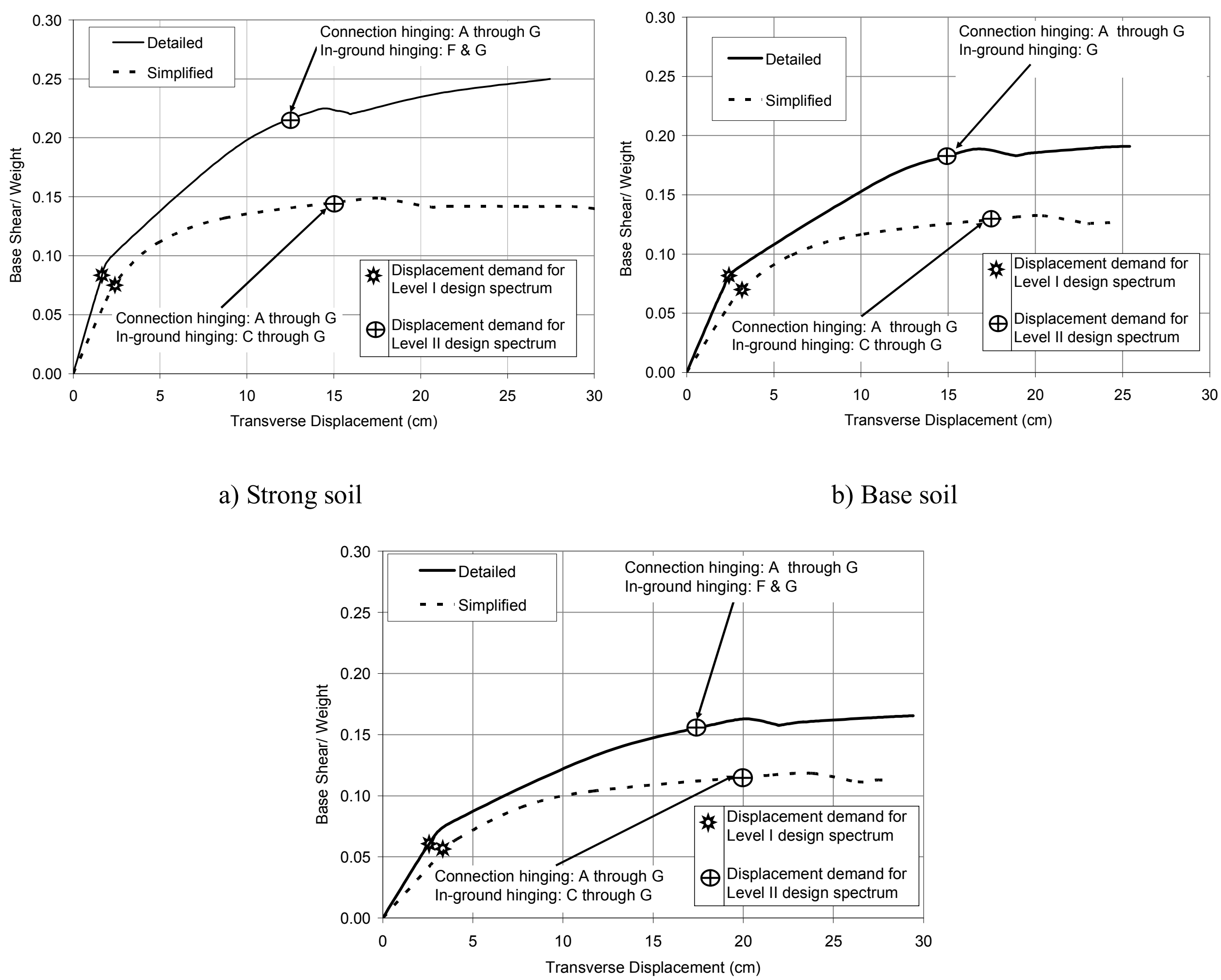


Figure 4.b) Simplified structural model

4. LATERAL LOAD RESPONSE

The distribution of lateral displacement with base shear strength coefficient, which is defined as the ratio of the base shear force to the weight of the structure, was obtained using both alternative analysis methods. The lateral displacement vs. base shear strength coefficient distributions computed for the strong, base, and weak soils are presented in Figures 5a through 5c. It can be observed that using the simplified analysis method leads to underestimation of: (i) the base shear strength by 30 percent for the weak soil and by 40 percent for the strong soil, and (ii) the initial lateral stiffness by 30 percent for the weak soil and by 35 percent for the strong soil. It is of interest to notice that, these observations are consistent with the conservative estimates of points of fixity used in the simplified analysis method.

Referring to the results obtained from the detailed model in Figures 5a through 5c, it can also be noticed that the base shear strength calculated for the strong soil is 35 percent greater than that for the weak soil. Furthermore, the initial lateral stiffness computed by assuming strong soil conditions is more than twice as much as the initial lateral stiffness obtained for the weak soil. Therefore, it can be concluded that the accuracy of the estimates for lateral stiffness and strength of the structure is dominated by the level of uncertainty typically involved in the approximation of the lateral resistance of soil rather than the crudeness of the simplified model.



a) Strong soil

b) Base soil

c) Weak soil

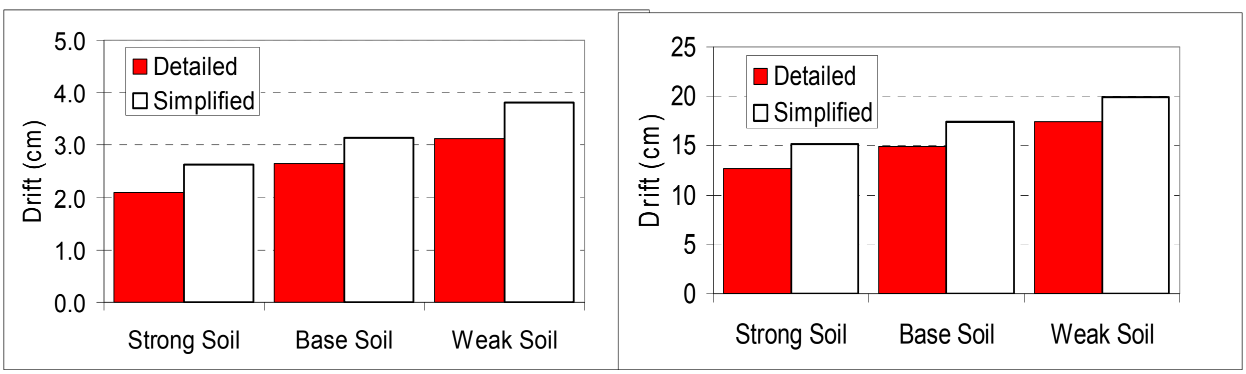
Figure 5 Lateral load (pushover) responses for different soil conditions and modeling alternatives

4.1 Displacement and Plastic Rotation Demand

Figure 6 show the displacement demands for different levels of design acceleration spectra as obtained by following the procedure described in ATC-40 (1996). It is evident that the displacements obtained from the simplified method are only about 20 percent higher than those calculated from the detailed method. This minimal overestimation in the displacement demand, despite the significant underestimation of the initial stiffness can be explained by the greater incursion of the structure into the nonlinear range of response in the case of the simplified method, thus, higher energy dissipation achieved through hysteretic damping.

The results from the detailed model included in Figure 6 indicate that the calculated drift for the weak soil is 40 to 50 percent higher than that for the strong soil under low to moderate seismic demand. Thus, it can be concluded that the accuracy of lateral displacement estimates is dominated by the level of uncertainty involved in modeling the lateral resistance of the soil rather than the assumptions made in the simplified model.

The calculated displacements demands shown in Fig. 6 are also included in Figures 5a through 5c in order to determine the extent of damage of the structure under the selected acceleration spectra. It is observed that for Level I design spectrum, the responses obtained from alternative models are essentially linear, while for Level II spectrum nonlinear response is evident. In both analyses, yielding is achieved at all pile-to-pile cap connections. However, while results from the detailed method indicate in-ground hinging for piles F and G, in-ground hinging for piles C through G was obtained using the simplified method. It is important to mention that the occurrence of in-ground hinging is not desirable under operating level earthquake; this is, however, permitted for contingency level earthquake by standard documents such as MOTEMS (2005). Therefore, using the simplified structural model is clearly a conservative choice for estimating the extent of damage expected in the wharf.



a) Level I acceleration spectrum b) Level II design spectrum

Figure 6 Displacement demand for different soil conditions and modeling alternatives

Notable incursion into the nonlinear range of structural response suggested by the simplified analysis method may raise concerns about the unnecessary effort to detail the pile cap connections for unrealistic high ductility. However, from Figure 7 it is observed that the maximum plastic rotation demands computed at the pile-to-pile cap connections using the simplified method are only 15 percent higher than those obtained from the detailed method. It is important to notice that the overestimation of plastic rotations in the simplified method is expected because hysteretic damping at the pile and/or the pile cap connection is the only source of energy dissipation, while softening of the supporting soil also contributes to the energy dissipation in the detailed method.

As anticipated, maximum plastic rotation demands were observed to occur at the pile-to-pile cap connections located at gridline G (Figure 1). Furthermore, the difference in the rotation demands obtained from the simplified and detailed methods was observed to be less significant for higher seismic demand.

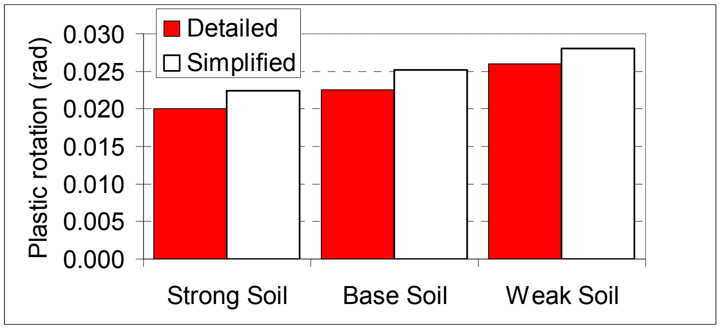


Figure 7 Maximum plastic rotation demand at pile-to-pile cap connection for Level II design spectrum

5. SUMMARY AND CONCLUSIONS

A marginal wharf was analyzed in order to investigate the implications of representing the lateral stiffness of the soil by means of constant points of fixity for the piles. Findings from the study indicate that using such simplification may result in conservative estimates of the lateral load response of the structure. However, the overestimation of the displacement demands under low-to-moderate design spectra is minimal.

Te simplified method was observed to result in slightly conservative plastic rotation demands at the pile-to-pile cap connection. This is expected because the simplified method based on points of fixity for the piles does not take into account the energy dissipation at the supporting soil through lateral displacement reversals.

It was concluded that the extent of imprecision rendered by the simplification of the analysis method is well within the imprecision imposed by uncertainties involved with estimation of p-y curves that represent the lateral response of the soil. Hence, the simplified method presented in this manuscript is recommended as a plausible device to be used in the preliminary design of wharves under low-to-moderate seismic demands.

5. ACKNOWLEDGEMENTS

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REFERENCES

ATC-40 (1996). Seismic Evaluation and Retrofit of Concrete Buildings. Volume 1. Applied Technology Council, Report No. SSC 96-01.

LPILE Plus 5.0 for Windows, A Program for the Analysis of Piles and Drilled Shafts under Lateral Loads, Ensoft, Inc., Austin, Texas.

Matlock, H. (1970). Correlations for design of laterally loaded piles in soft clay. Proceedings, Offshore Technology Conference, Vol. I, Paper No. 1204, Houston, Texas, p. 577-594.

MOTEMS (2005). Marine Oil Terminal Engineering and Maintenance Standards. Marine Facilities Division of the California State Lands Commission.

Reese, L. C., Cox, W. R., and Koop, F. D. (1974). Analysis of laterally loaded piles in sand. Proc. Offshore Technology Conference, Houston, Texas, Vol. II, Paper 2080, p. 473-484.

SAP 2000, Integrated Software for Structural Analysis and Design, Computers and Structures, Inc., Berkeley, CA.