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Approximate seismic displacement capacity of piles in marine oil terminals

Rakesh K. Goel*

Department of Civil and Environmental Engineering, California Polytechnic State University, San Luis Obispo, CA 93407, USA

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Aabtract. This paper proposes an approximate procedure to estimate seismic displacement capacity − defined as yield displacement times the displacement ductility − of piles in marine oil terminals. It is shown that the displacement ductility of piles is relatively insensitive to most of the pile parameters within ranges typically applicable to most piles in marine oil terminals. Based on parametric studies, lower bound values of the displacement ductility of two types of piles commonly used in marine oil terminals − reinforced-concrete and hollow-steel − with either pin connection or full-moment-connection to the deck for two seismic design levels – Level 1 or Level 2 − and for two locations of the hinging in the pile − near the deck or below the ground − are proposed. The lower bound values of the displacement ductility are determined such that the material strain limits specified in the Marine Oil Terminal Engineering and Maintenance Standard (MOTEMS) are satisfied at each design level. The simplified procedure presented in this paper is intended to be used for preliminary design of piles or as a check on the results from the detailed nonlinear static pushover analysis procedure, with material strain control, specified in the MOTEMS.

Keywords: marine structures; seismic displacement capacity; seismic ductility; seismic analysis; seismic design, piles.

1. Introduction

Seismic design of marine oil terminals in California is governed by the Marine Oil Terminal Engineering and Maintenance Standard (MOTEMS) (Eskijian 2007, MOTEMS 2007). The MOTEMS requires design of such facilities for two earthquake levels: Level 1 and Level 2. The return period of the design earthquake for each level depends on the risk level. For example, Level 1 and Level 2 design earthquakes for high risk terminals correspond to return periods of 72 and 475 years, respectively. The acceptance criteria for piles in the MOTEMS are specified in terms of maximum permissible material strains. The maximum permissible material strains depend on the earthquake level – Level 1 or Level 2. For reinforced-concrete piles, the material strain limits also depend on location of the plastic hinge – pile-deck or in-ground. The material strain limits in two of the commonly used piles – reinforced-concrete and hollow-steel – are summarized in Table 1. These strain values are based on the latest revisions (after public comment period) to the MOTEMS

* Professor, E-mail: rgoel@calpoly.edu

Table 1 Material strain limits in the MOTEMS

Pile Type	Material	Hinge Location	Level 1	Level 2
Reinforced-Concrete	Concrete	Pile-Deck	$\varepsilon_c \leq 0.004$	$\varepsilon_c \leq 0.025$
		In-Ground	$\varepsilon_c \leq 0.004$	$\varepsilon_c \leq 0.008$
	Steel	Pile-Deck	$\varepsilon_s \leq 0.01$	$\varepsilon_s \leq 0.05$
		In-Ground	$\varepsilon_s \leq 0.01$	$\varepsilon_s \leq 0.025$
Hollow-Steel	Steel		$\varepsilon_s \leq 0.008$	$\varepsilon_s \leq 0.025$

and have been modified from the previous version as follows: (1) concrete compression strain limit for Level 1 have been reduced from 0.005 to 0.004 for both pile-deck and in-ground hinge formation; and (2) steel tensile strain limit for Level 2 has been increased from 0.01 to 0.025 for in-ground hinge formation.

Since the acceptance criteria in the MOTEMS is specified in terms of maximum permissible strains, estimation of seismic displacement capacity of piles in marine oil terminals requires monitoring material strains during the nonlinear pushover analysis. However, most commercially available structural analysis programs do not have the capability to directly monitor strains during such analysis. Therefore, there is a need to develop simplified procedure to estimate seismic displacement capacity of piles in marine oil terminals that ensure that material strains do not exceed the values specified in the MOTEMS yet do not require direct monitoring of strains during seismic analysis.

In order to fill this important need, this investigation is aimed at developing a simplified procedure for estimation of displacement capacity using displacement ductility, instead of strain limits, for piles in marine oil terminals. The simplified procedure is developed for piles which are connected to the deck either by pin-connection or full-moment-connection and assumed to be fixed at the bottom at a distance equal to depth-to-fixity below the mud line; initial results of this study were reported in a recent publication (Goel 2008). Simplified procedures for piles with other types of pile-deck connection (such as dowel connection for reinforced concrete piles and concrete-plug connection for steel piles) are available in Goel (2010). The simplifying assumption of fixing the pile at a distance equal to depth-of-fixity below the mud line, and thus avoiding explicit modeling of soil, is appropriate for estimating the displacement capacity because such assumption is commonly used in estimating is seismic displacement demand for marine oil terminals; next section discuses details of this simplification.

2. Simplifying assumption

Fig. 1(b) shows typically used mathematical model of a free-head pile of Fig. 1(a) supported on bedrock (or other competent soil) and surrounded by soil between the bedrock and mud line. In this model, the pile is represented by beam-column elements and soil by Winkler reaction springs connected to the pile between the bedrock and the mud line (Finn 2005). The properties of the beam-column element are established based on the pile cross section whereas properties of the reaction springs are specified based on geotechnical data (e.g., see Priestley *et al.* 1996, Dowrick 1987, Finn 2005). Fig. 1(c) shows the height-wise distribution of bending moment under lateral load applied to the pile tip. Note that the maximum bending moment occurs slightly below the mud line at a depth equal to D_m, typically denoted as the depth-to-maximum-moment below the mud line Fig. 1(c). Lateral displacement at the pile top can be calculated based on this bending moment distribution or from a discrete element model implemented in most commonly available computer programs for structural analysis.

An alternative approach to modeling the detailed pile-soil system (Fig. 1(b)) is the effective fixity approach (Priestley *et al.* 1996, Sec. 4.4.2; Dowrick 1987, Sec. 6.4.5.3). In this approach, the pile is assumed to be fixed at a depth below the mud line equal to the depth-to-fixity, D_f, and no soil reaction springs are included (Fig. 1(d)). The depth-to-fixity is defined as *the depth that produces in a fixed-base column with soil removed above the fixed base the same top-of-the-pile lateral displacement under the lateral load*, F, as that in the actual pile-soil system (Priestley *et al.* 1996). Both the axial load, P, and top-of-the-pile moment, M (not shown in Fig. 1(d)) need to be considered. The depth-to-fixity, which depends on the pile diameter and soil properties, is typically provided by the geotechnical engineer, estimated from charts available in standard textbooks on the subject (e.g., Priestley *et al.* 1996, Dowrick 1987), or from recommendations in several recent references (e.g., Chai 2002, Chai and Hutchinson 2002).

The equivalent fixity model is typically used for estimating displacement of piles that remain within the linear-elastic range. For piles that are expected to be deformed beyond the linear-elastic range, however, nonlinear analysis of the discrete soil spring model approach of Fig. 1(b) is preferable (Priestley *et al.* 1996, Sec. 4.4.2) because the plastic hinge forms at the location of the maximum bending moment, i.e., at the depth-to-maximum-moment, D_m, and not at the depth-to-fixity, D_f. A recent investigation has developed equations for estimating lateral displacement of equivalent fixity model of the nonlinear soil-pile system by recognizing that the plastic hinge forms at the depth-to-maximum-moment (Chai 2002); expressions for estimating displacement ductility of pile-soil system are also available (Priestley *et al.* 1996, Sec. 5.3.1). However, calculation of lateral displacement capacity of nonlinear soil-pile systems using these approaches requires significant information about the soil properties. Another paper developed relationship between curvature ductility and displacement ductility of fixed-head concrete piles (Song *et al.* 2005) but did not

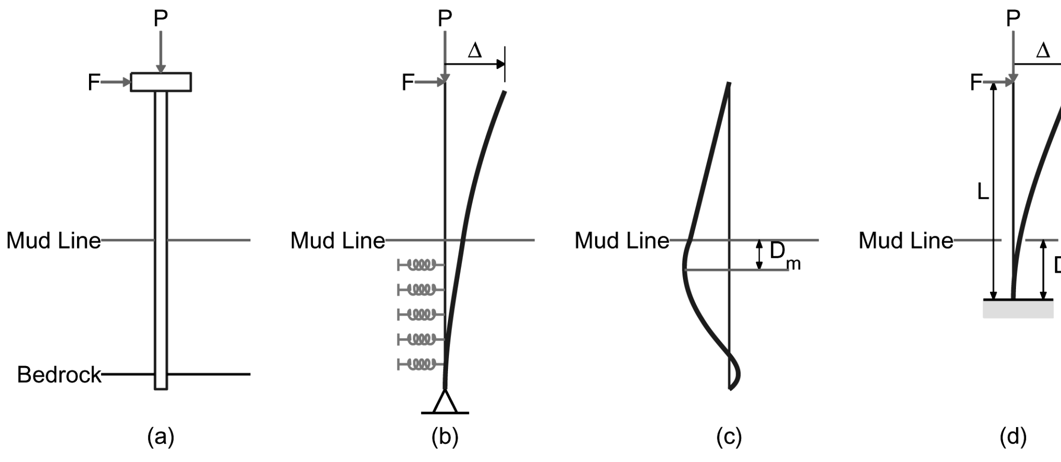


Fig. 1 Simplified model of the pile-soil system for displacement capacity evaluation (a) Pile supported on bedrock, (b) Mathematical model of the pile, (c) Height-wise variation of bending moment, (d) Equivalent fixity model for displacement calculation

include flexibility of piles above the mud line as is the case for piles in marine terminals.

This investigation uses a simplifying assumption that the equivalent fixity model may directly be used to estimate lateral displacement capacity of nonlinear piles. Clearly, such an approach implies that the plastic hinge would form at the depth-to-fixity, D_f, which differs from the actual location at the depth-to-maximum-moment, D_m. It is useful to note that D_f is typically in the range of 3 to 5 pile diameter whereas D_m is in the range of 1 to 2 pile diameter (see Priestley *et al.* 1996). Obviously, the displacement would be lower if the cantilever base was located at D_m below the mud line compared to if it was located at D_f in the equivalent fixity model: plastic displacement is given by $\Delta_p = \theta_p(L_a + D_f \text{ or } D_m)$ where θ_p is the plastic hinge rotation and L_a is the cantilever length. However, it must be noted that additional deformation would occur in the actual pile due to curvature below the depth-to-maximum-moment (see Fig. 1(c)). Fixing the cantilever at D_f below the mud line compensates for these additional deformations.

Seismic analysis of large marine oil terminals using analytical models that include detailed modeling of soil effects on each pile, such as those in Fig. 1(b), becomes cumbersome. Therefore, analyses to estimate seismic displacement demands of piles often utilize the equivalent fixity approximation. The depth-to-fixity below the mud line is generally recommended by geotechnical engineer on the project for typical piles and the local soil conditions and linear/nonlinear soil behavior.

Since the seismic displacement demand is often estimated from analysis of a system that utilizes equivalent fixity approximation for piles, the same approximation must be used for estimating the seismic displacement capacity for consistency. Therefore, it is appropriate to utilize the assumption of equivalent fixity for estimating seismic displacement capacity of piles as proposed in this investigation.

It is useful to emphasize that the approximate approach proposed in this investigation is intended to be used for preliminary design of piles or as a check on the results from the detailed nonlinear analysis. The final evaluation of piles may be conducted by analysis of detailed pile-soil system using more advanced methods (e.g., Castelli and Maugeri 2009, Imancli *et al.* 2009, El Naggar *et al.* 2005, Allotey and el Naggar 2008, Chiou and Chen 2007).

3. Current procedure in the motems

The displacement capacity of piles in the MOTEMS is estimated from nonlinear static pushover analysis. In this analysis, a force of increasing magnitude is applied statically in the transverse direction (perpendicular to the pile) permitting the materials in the pile – steel and/or concrete – to deform beyond their linear-elastic range. The displacement capacity is defined as the maximum displacement that can occur at the tip of the pile without material strains exceeding the permissible values corresponding to the desired design level.

The displacement capacity of a pile at a selected design level in the MOTEMS is obtained from the procedure proposed by Priestley *et al.* (1996) as illustrated in Fig. 2. This procedure requires development of the pile section moment-curvature relationship. The moment-curvature relationship may be developed from any standard moment-curvature analysis programs using material constitutive relationship specified in the MOTEMS; the MOTEMS specifies guidelines for selecting material properties such as concrete and steel strengths as well as stress-strain curves for unconfined concrete, confined concrete, reinforcing steel, and prestressing steel. It is useful to note that the

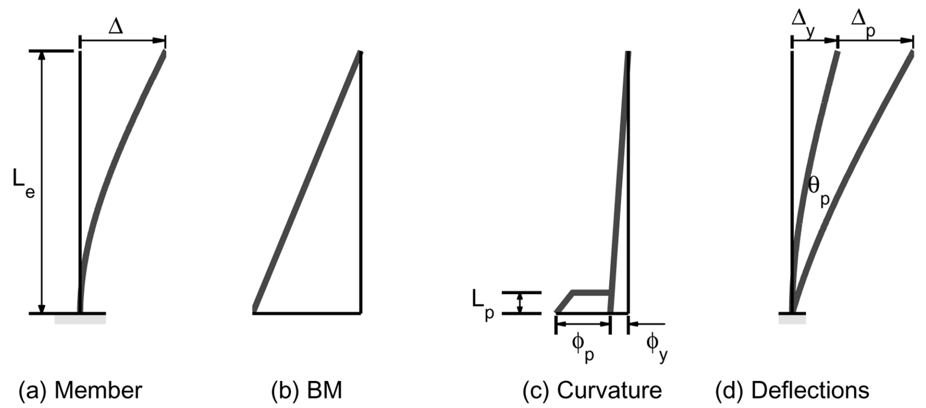


Fig. 2 Deformation capacity of a pile (a) Deflected shape, (b) Bending moment diagram, (c) Curvature distribution, (d) Yield and plastic displacements

formulation presented next is for a cantilever, i.e., a pile with a pin-connection to the deck. Similar formulation is available for piles with full-moment-connection (e.g., see CALTRANS 2006).

The total displacement capacity of the pile is computed as

$$\Delta = \Delta_y + \Delta_P \tag{1}$$

in which Δ_y is the yield displacement and Δ_p is the plastic displacement of the pile. The yield displacement can be estimated as

$$\Delta_y = \frac{\phi_y L_e^2}{3} \tag{2}$$

where ϕ_y is the yield curvature computed from

$$\phi_y = \frac{M_y}{EI_e} \tag{3}$$

with M_y being the yield moment and EI_e being the slope of the initial elastic portion of the bilinear idealization of the moment-curvature relationship and L_e is the pile "effective" length defined as the length between a critical section and the point of contra-flexure (or point of zero bending moment). The "effective" length for a cantilever becomes equal to its total length.

The plastic displacement is computed from

$$\Delta_p = \left(\frac{M_u}{M_y} - 1\right)\Delta_y + L_p(\phi_u - \phi_y)(L_e - 0.5L_p) \tag{4}$$

which includes components due to the elastic displacement resulting from the increase in moment from M_y to M_u, i.e., post-yield stiffness of the moment-curvature relationship, and due to plastic rotation θ_p of the pile. In order to compute the plastic rotation, it is assumed that a constant plastic curvature, $\phi_p = \phi_u - \phi_y$, occurs over a plastic hinge length L_p of the pile (see Fig. 2(c)). Therefore, the plastic rotation is given by

$$\theta_p = L_p\phi_p = L_p(\phi_u - \phi_y) \tag{5}$$

The values of M_u and ϕ_u in Eq. (4) are largest values of pile section moment and curvature, respectively, without exceeded the material strains at the selected design level.

The MOTEMS specify the formula for estimating the plastic hinge length required in Eq. (4). If the hinge were to form against a supporting member, i.e., at the pile-deck intersection, the plastic hinge length is computed from

$$L_p = \begin{cases} \rho L_e + 0.022 f_{ye} d_{bl} \geq 0.044 f_{ye} d_{bl} & (f_{ye} \text{ in MPA}) \\ \rho L_e + 0.15 f_{ye} d_{bl} \geq 0.3 f_{ye} d_{bl} & (f_{ye} \text{ in ksi}) \end{cases} \tag{6}$$

in which L_e is the "effective" length defined as the distance from the critical section of the plastic hinge to the point of contra-flexure, $\rho = 0.08$ is the constant that specifies plastic hinge length as a fraction of the length L_e, f_{ye} is the expected yield strength of the reinforcing steel, and d_{bl} is the diameter of the longitudinal reinforcement. The second term in Eq. (6) is intended to account for strain-penetration effects in reinforcing steel in reinforced concrete members. If the plastic hinge forms in-ground, the MOTEMS provides a chart to estimate the plastic hinge length that depends on the pile diameter, subgrade modulus, effective stiffness of the pile, and the distance from ground to the pile point of contra-flexure above the ground. It is useful to note that Eq. (6), as specified in Priestley *et al.* (1996) or in the MOTEMS (2006), does not explicitly impose an upper limit even though there may be some experimental evidence that the plastic hinge length should not be greater than the pile diameter.

The plastic hinge length formula of Eq. (6) specified in the MOTEMS is based on the recommendation by Priestley *et al.* (1996) for reinforced concrete sections. The MOTEMS do not provide recommendations for plastic hinge length for hollow steel piles.

4. Approximate procedure

An approximate procedure is proposed in this investigation to compute the displacement capacity of piles commonly used in marine oil terminals. This approach computes the displacement capacity as

$$\Delta_c = \mu_\Delta \Delta_y \tag{7}$$

where Δ_y is the yield displacement of the pile and μ_Δ is the displacement ductility of the pile. The yield displacement may be estimated from

$$\Delta_y = \begin{cases} \dfrac{M_y L_t^2}{6EI_e} & \text{for piles with full-moment-connection to the deck} \\ \dfrac{M_y L_t^2}{3EI_e} & \text{for piles with pin-connection to the deck} \end{cases} \tag{8}$$

in which M_y is the effective section yield moment, EI_e is the initial slope of the idealized bilinear moment-curvature relationship, and L_t is the total length of the equivalent fixity pile model. The displacement ductility is selected such that the material strains remain within the limits specified in the MOTEMS. The guidelines to select the displacement ductility are developed next for reinforced-concrete piles and hollow-steel piles with either pin-connection or full-moment-connection to the deck by parametric studies on nonlinear finite element models of piles fixed at the depth equal to depth-to-equivalent-fixity below the mud line.

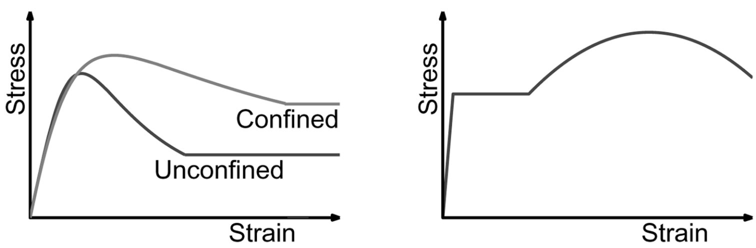
5. Analytical model

As noted previously, this investigation utilizes the equivalent fixity model (Fig. 1(d)) to estimate displacement capacity the pile-soil system (Fig. 1(a)) instead of the more elaborate model containing entire pile length with Winkler reaction springs below the mud line (Fig. 1(b)). Therefore, an analytical model of the cantilever (with free or fixed head) was developed in the structural analysis software Open System for Earthquakes Engineering Simulation (*OpenSees*) (McKenna and Fenves 2001). The length of the pile, L_t, was selected to be equal to the free-standing height plus the depth below the mud-line equal to the depth-to-fixity. The pile was modeled with the *nonlinearBeamColumn* element in *OpenSees*. The *nonlinearBeamColumn* element uses a force-based, distributed-plasticity approach with integration of section behavior over the member length. The pile was modeled with five elements, each with seven integration points. The section is defined with fibers of confined concrete, unconfined concrete, and steel reinforcing bars for reinforced-concrete piles and steel for hollow-steel piles. The nonlinear axial-flexural behavior of the element is determined by integration of the nonlinear stress-strain relationships of various fibers across the section, whereas linear behavior is assumed for shear and torsional.

The compressive stress-strain behavior of concrete, both confined and confined, was modeled with *Concrete01* material in *OpenSees* (Fig. 3(a)). The crushing strain of the unconfined concrete was selected to be equal to 0.004 and that for confined concrete was selected to be that corresponding to the rupture of confining steel using the well established Mander model (see Priestley *et al*. 1996). The stress-strain behavior of steel was modeled with *ReinforcingSteel* material in *OpenSees* (Fig. 3(b)). Further details of the material models and *nonlinearBeamColumn* element are available in McKenna and Fenves (2001).

6. Results of parametric study

This section presents results of parametric studies to investigate the sensitivity of the displacement



(a) Concrete model (b) Steel model

Fig. 3 Material models

ductility on various pile parameters such as pile length and diameter, longitudinal reinforcement ratio, transverse reinforcement ratio, and axial force level for reinforced-concrete piles; and pile length and diameter, wall thickness, and axial force level for hollow-steel piles. Following is a summary of this study.

6.1 Reinforced-Concrete piles

Fig. 4 presents variation of displacement ductility with pile length, L_t, for three values of pile diameters: 61 cm, 91 cm, and 107 cm. The results are presented for piles with axial force equal to 5% of its capacity, 1% longitudinal reinforcement, and 0.6% transverse reinforcement. These parameters correspond to typically used piles in marine oil terminals. The presented results indicate that the displacement ductility of the pile is essentially independent of the pile length as apparent from very little variation in the ductility with length as well as pile diameter as apparent from almost identical curves for the three pile diameters considered.

In order to understand the aforementioned trend, i.e., independence of the displacement ductility of the pile length and diameter, it is useful to re-examine the equation that defines the displacement ductility. For this purpose, let us utilize Eqs. (1), (4) and (8) to express the displacement ductility as

$$\mu_\Delta = \left(\frac{M_u}{M_y}\right) + 3(\mu_\phi - 1)\left(\frac{L_p}{L_e}\right)\left(1 - 0.5\frac{L_p}{L_e}\right) \tag{9}$$

in which μ_ϕ is the section curvature ductility at a selected design level. For piles with little or no post-yield straining hardening in section moment-curvature relationship, $M_u \simeq M_y$, and Eq. (9) further simplifies to

$$\mu_\Delta = 1 + 3(\mu_\phi - 1)\left(\frac{L_p}{L_e}\right)\left(1 - 0.5\frac{L_p}{L_e}\right) \tag{10}$$

Eq. (10) indicates that the displacement ductility depends on the plastic hinge ratio, L_p/L_e, and section curvature ductility, μ_ϕ. For long piles typically used in marine oil terminals, the ratio L_p/L_e

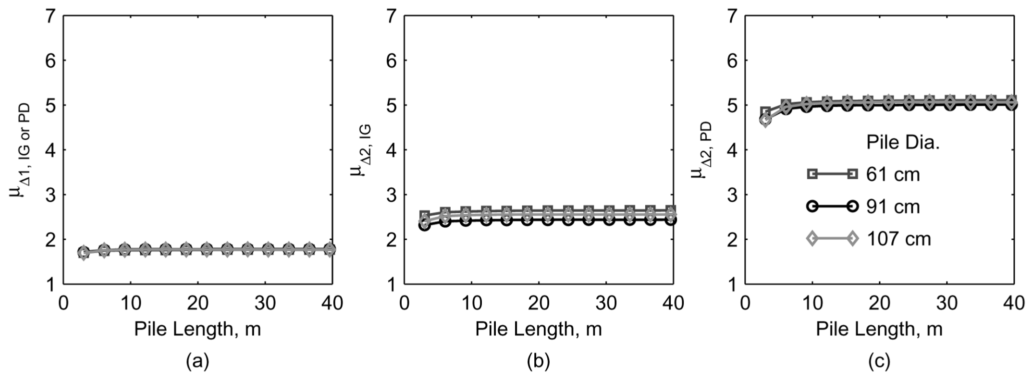


Fig. 4 Variation of displacement ductility with pile length and pile diameter of reinforced-concrete piles for seismic design (a) Level 1 for in-ground (IG) and pile-deck (PD) hinge formation, (b) Level 2 for IG hinge formation, (c) Level 2 for PD hinge formation

may become essentially independent of the length as the second term in Eq. (6) becomes much smaller than the first term. This indicates that the displacement ductility is essentially independent of the pile length which is consistent with the observation from results presented in Fig. 4.

The displacement ductility does not directly depend on the pile diameter but on the section curvature ductility, μ_ϕ. The results presented in Fig. 5 indicate that the section curvature ductility is essentially independent of the pile diameter. Therefore, Eq. (10) leads to the conclusion that the displacement ductility is also essentially independent of the pile diameter, which is consistent with the finding in Fig. 4 that the pile displacement ductility is independent of the pile diameter.

Fig. 6 present the variation of the displacement ductility with the reinforcement ratio. The results presented are for a pile with 91 cm diameter and 15 m length. The values of longitudinal reinforcement varying between 0.5% and 2% and transverse reinforcement between 0.5% and 1.5% were considered; these ranges of longitudinal and transverse reinforcement correspond to those for typical piles in marine oil terminals.

The results presented in Fig. 6(a) show that the displacement ductility decreases with increasing

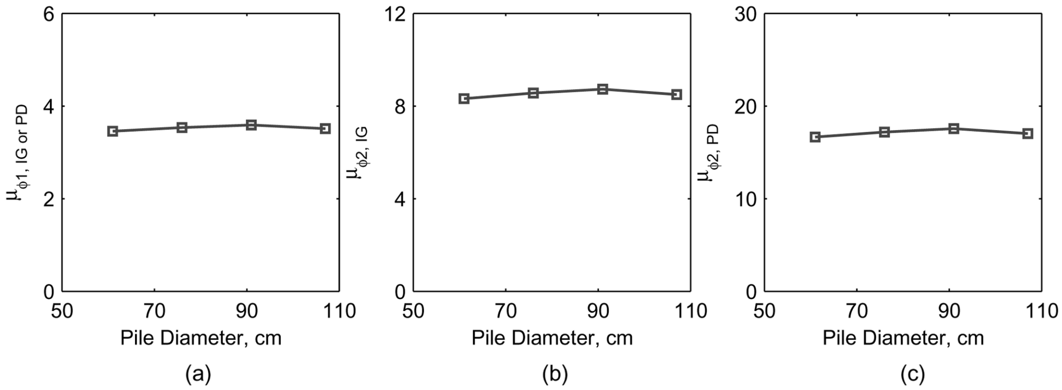


Fig. 5 Variation of section curvature ductility of reinforced-concrete piles with pile diameter for seismic design (a) Level 1 for in-ground (IG) and pile-deck (PD) hinge formation, (b) Level 2 for IG hinge formation, (c) Level 2 for PD hinge formation

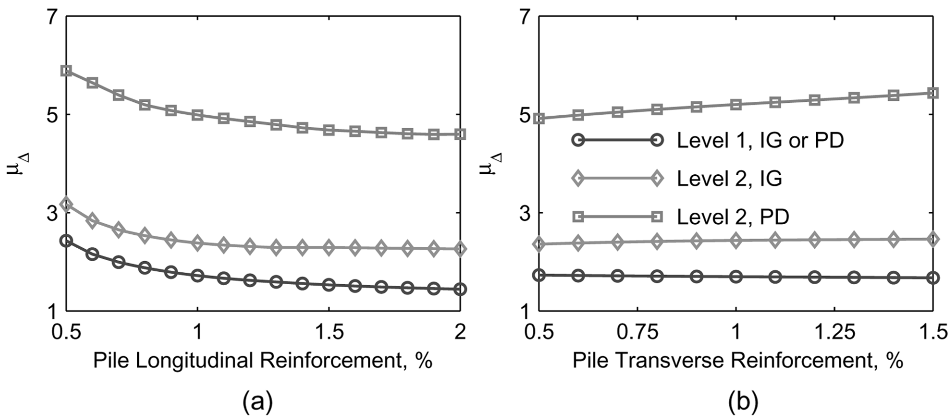


Fig. 6 Variation of displacement ductility of reinforced-concrete piles with pile reinforcement ratio (a) Longitudinal reinforcement, (b) Transverse reinforcement

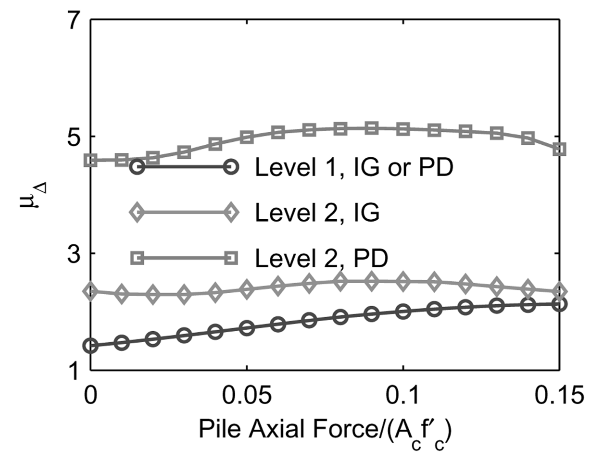


Fig. 7 Variation of displacement ductility of reinforced-concrete piles with pile axial load ratio

longitudinal reinforcement ratio for values up to about 1%. For longitudinal reinforcement ratio in excess of about 1%, as may be the case for seismic piles in marine oil terminals, the displacement ductility of piles is much less sensitive to the value of the longitudinal reinforcement ratio. For such values, the displacement ductility may be considered to be essentially independent of the longitudinal reinforcement ratio. The results presented in Fig. 6(b) show that displacement ductility of piles does not depend significantly on the transverse reinforcement ratio. This becomes apparent from essentially flat variation of the displacement ductility with pile transverse reinforcement ratio.

Fig. 7 presents variation of displacement ductility varies with axial force in the pile. The presented results are for a pile with 91 cm diameter and 15 m length for values of axial force level varying from zero to 15% of the pile axial capacity. These results show that the displacement ductility for Level 1 corresponding to in-ground or pile-deck hinge formation tends to increase with increasing pile axial force. However, the ductility for Level 2 corresponding to in-ground or pile-deck hinge formation appears to be insensitive to the axial force values.

6.2 Hollow-Steel piles

Fig. 8 presents variation of displacement ductility with pile length for three values of pile diameters – 61 cm, 91 cm, and 107 cm – which are representative of typical piles used in marine oil terminals and pile wall thickness of 1.27 cm. These results indicate that the displacement ductility of the hollow-steel pile is also essentially independent of the pile length and diameter as apparent from no variation in the ductility with pile length and almost identical curves for the three pile diameters considered.

As mentioned previously for reinforced concrete piles, the displacement ductility of hollow-steel piles is independent of the pile diameter because the section curvature ductility is essentially independent of the pile diameter. This is confirmed by the results presented in Fig. 9 for variation of section curvature ductility with pile diameter.

In order to understand the effects of the pile wall thickness on the displacement ductility, variations of the displacement ductility with pile length for three values of pile thickness are presented in Fig. 10. The results presented are for a pile with 91 cm diameter and axial force equal

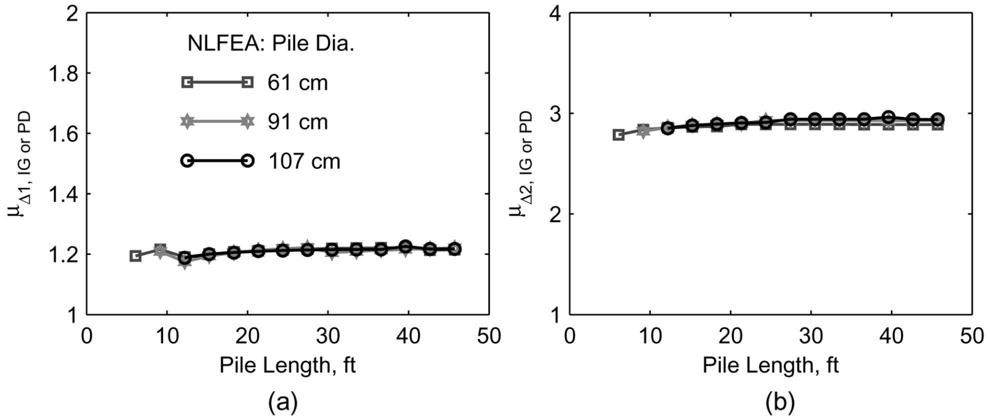


Fig. 8 Variation of displacement ductility of hollow-steel piles with pile length and pile diameter (a) Level 1 for in-ground (IG) or pile-deck (PD) hinge formation, (b) Level 2 for IG or PD hinge formation

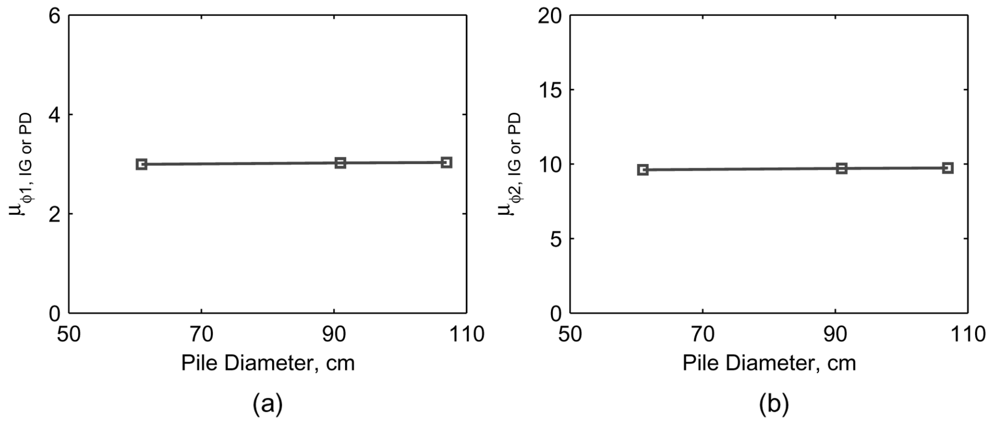


Fig. 9 Variation of section curvature ductility of hollow-steel piles with pile diameter (a) Level 1 for in-ground (IG) or pile-deck (PD) hinge formation, (b) Level 2 for IG or PD hinge formation

to $0.05Af_y$. These results show that the displacement ductility is essentially independent of the pile wall thickness as indicated by essentially identical curves for the three values of pile wall thickness.

Fig. 11 presents the variation of displacement ductility with axial force in the hollow-steel pile. The presented results are for a pile with 91 cm diameter and 15 m length for values of axial force varying from zero to 15% of the pile axial load capacity. These results show that the displacement ductility for Level 1 is essentially independent of the pile axial load. For Level 2, while the displacement ductility may depend on the axial load for very-low axial loads, it becomes essentially independent of the axial load for more realistic values, i.e. However, the ductility for Level 2 appears to be insensitive to the axial force values, i.e., axial loads greater than $0.05Af_y$.

7. Lower bound of displacement ductility of piles

The results presented so far indicate that the displacement ductility of piles (reinforced-concrete or

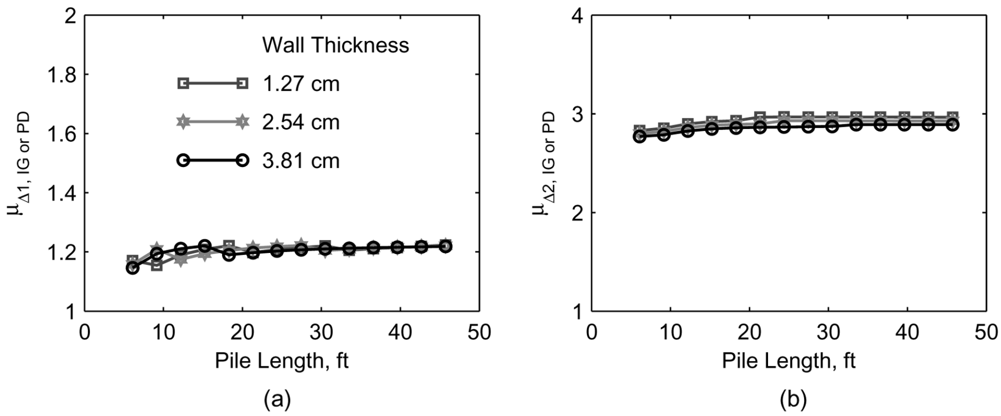


Fig. 10 Variation of displacement ductility of hollow-steel piles with pile length for three values of pile wall thickness (a) Level 1 for in-ground (IG) or pile-deck (PD) hinge formation, (b) Level 2 for IG or PD hinge formation

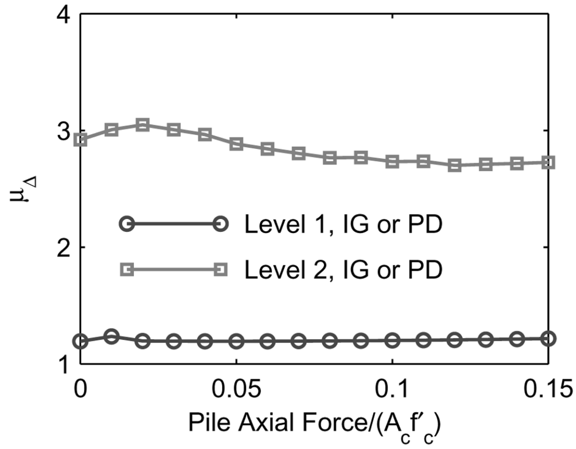


Fig. 11 Variation of displacement ductility of hollow-steel piles with pile axial load ratio

hollow-steel) is relatively insensitive to the various pile parameters within the range these parameters typically used in marine oil terminals. Therefore, the displacement ductility can be used to approximately estimate the pile displacement capacity instead of the material strain limits which are currently specified in the MOTEMS. The results presented in the preceding section indicate that a lower bound of the member displacement ductility may be estimated without any knowledge about various pile parameters.

The results presented in Fig. 12 for pile-deck hinge in the reinforced-concrete pile indicate that lower bound values of the displacement ductility equal to 1.75 for seismic design Level 1 for formation of in-ground or pile-deck hinge, 2.5 for seismic design Level 2 for formation of in-ground hinge, and 5.0 for seismic design Level 2 for formation of pile-deck hinge are appropriate. Note that the displacement ductility for Level 1 is likely to be slightly lower for axial force values lower than the $0.05A_g f_c'$ value considered in developing the results. Similarly, the displacement ductility is likely to be slightly larger for longitudinal reinforcement lower than the 1% value considered in developing the results. The results presented in Fig. 13 for hollow-steel pile indicate that lower bound values of the displacement ductility equal to 1.2 for seismic design Level 1 and equal to 2.75 for seismic design Level 2 are appropriate. Since strain limits for both in-ground and pile-deck hinge formation are identical for the hollow-steel piles, the aforementioned lower bound values apply to formation of hinges at both locations.

Based on the results presented in Figs. 12 and 13, the recommended lower bound values of the displacement ductility of piles are as follows: 1.75 for seismic design Level 1 (in-ground or pile-deck hinge formation), 2.75 for seismic design Level 2 (in-ground hinge formation), and 5.0 for seismic design Level 2 (pile-deck hinge formation) for reinforced-concrete piles; and 1.2 for seismic design Level 1 and equal to 2.75 for seismic design Level 2 for hollow-steel piles regardless of location of the plastic hinge.

The recommendations on displacement ductility of piles in this section are based on analytical studies alone. It is highly desirable that such recommendations be verified by experiments on pile-soil systems (e.g., Budek *et al*. 2004, Roeder *et al*. 2005).

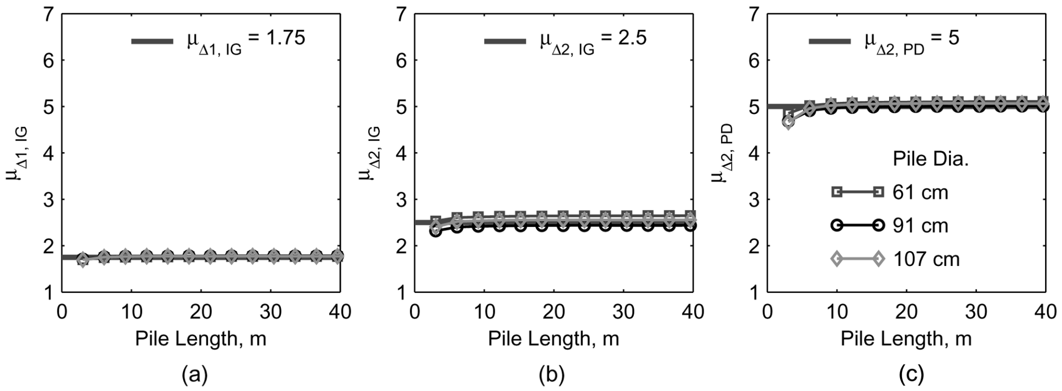


Fig. 12 Lower-bound value of displacement ductility of reinforced-concrete piles for seismic design (a) Level 1 for in-ground (IG) and pile-deck (PD) hinge formation, (b) Level 2 for IG hinge formation, (c) Level 2 for PD hinge formation

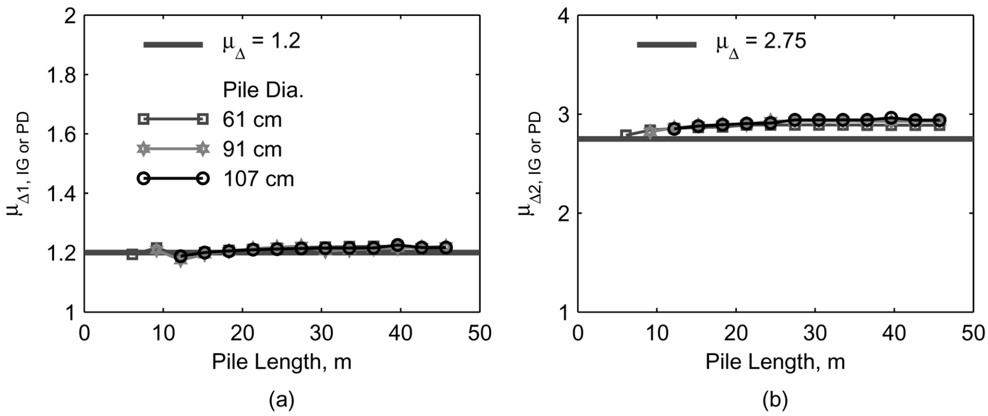


Fig. 13 Lower-bound value of displacement ductility of hollow-steel piles for seismic design (a) Level 1 for in-ground (IG) or pile-deck (PD) hinge formation, (b) Level 2 for IG or PD hinge formation

8. Illustrative example

An illustrative example is presented next to demonstrate that the simplified procedure presented in this paper provides estimate of displacement capacity of the pile shaft that is "accurate" enough for most practical purposes when compared to results from nonlinear pushover analysis. The example structure considered here is adopted from Priestley *et al.* (1996, Sec. 4.6). This structure consists of a circular pile shaft (or columns) of 1.83 m in diameter. The height of the column is 10 m above ground and that of the pile shaft is 15 m below ground with a deck depth of 2 m (Fig. 14(a)). The pile shaft (or column) reinforcement consists of 30 D 36 mm longitudinal bars and D19 mm spirals with a pitch of 115 mm. The material properties are f_y = 414 MPa yield for the reinforcement and f_c' =24.1 MPa nominal concrete strength. The soil is characterized as very dense sand with linearly varying subgrade reaction coefficient of $k_s^* z$ = 10z MPa/m. The column carries an axial load of 8 MN at its tip.

The mathematical model of the column-pile-soil system is shown in Fig. 14(b). The soil below the mud line is modeled with ten discrete springs. The spring values were taken from the example in Priestley *et al.* (1996) and are shown in Fig. 14(b). As mentioned previously, this model was implemented in OpenSees with various column and pile elements modeled with *nonlinear Beam Column* element and Winkler reaction springs modeled with *zeroLength* element. The fiber section properties of the column and pile were based on the specified section dimensions, reinforcement details, and concrete and steel strengths in Priestley *et al.* (1996).

A moment curvature analysis of the pile shaft section with an axial load of 8.85 MN (including axial force and column weight) and idealized bilinear approximation, shown in Fig. 15(a), results in an effective initial stiffness of

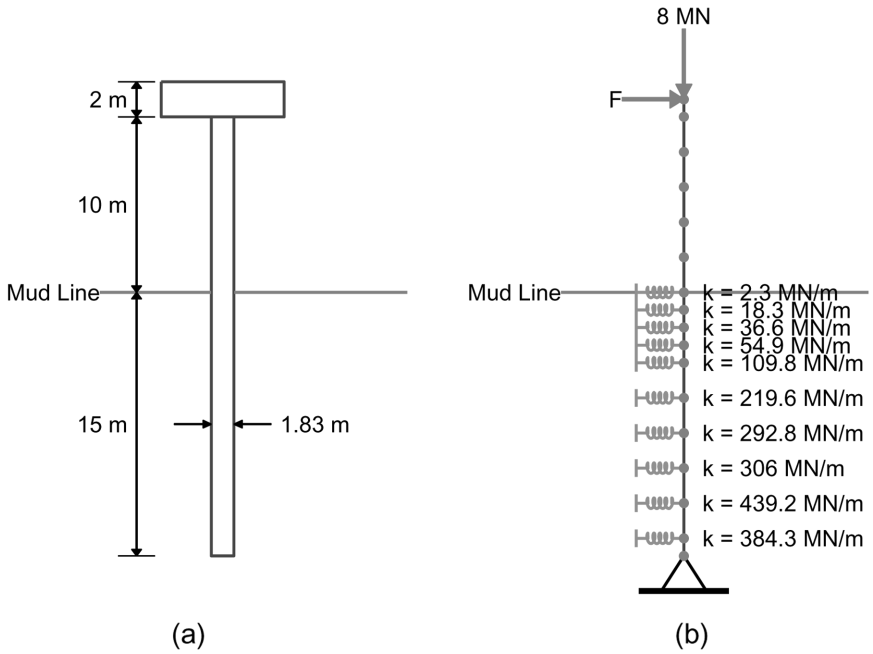


Fig. 14 Illustrative example (a) Pile-soil system, (b) Mathematical model of pile-soil system

$$EI_e = \frac{13473.5}{2.3588 \times 10^{-3}} = 5.71 \times 10^6 \text{ kN-m}^2 \tag{11}$$

Note that the values of M_y, ϕ_y and EI_e in Fig. 15(a) differ slightly from the results presented in Priestley *et al*. (1996) because of slight differences between the pile-section model and material models between this investigation and Priestley *et al*. (1996).

Based on the soil properties, the depth-to-fixity of the column-pile-soil system below the mud line is equal to three times the pile diameter (Priestley *et al*. 1996). Therefore, the total length of the equivalent fixity model is

$$\begin{aligned} L_e &= \text{Free-Standing Length} + \text{Half the Deck Depth} + 3 \times \text{Pile Diameter} \\ &= 10 + 1 + 3 \times 1.83 = 16.5 \text{ m} \end{aligned} \tag{12}$$

Using Eq. 8(b) for cantilever structure, the yield displacement of the equivalent fixity model is

$$\Delta_y = \frac{M_y L_e^2}{3EI_e} = \frac{(13473.5)(16.5)^2}{(3)(5.71 \times 10^6)} = 0.214 \text{ m} \tag{13}$$

As recommended in this investigation, the displacement ductility of the pile for Level 1 and Level 2 seismic design are 1.5 and 5, respectively, for reinforced-concrete piles. Let us assume that the ductility of 1.5 would control if the hinge were to form in-ground (as would be the case for the selected structure) and 5 would control if the hinge were to from above ground. The estimated displacement capacity of the pile using the simplified procedure presented in this investigation for Level 1 and Level 2 are

$$\Delta_{c1} = (1.5)(0.214) = 0.321 \text{ m} \tag{14}$$

$$\Delta_{c2} = (5)(0.214) = 1.07 \text{ m} \tag{15}$$

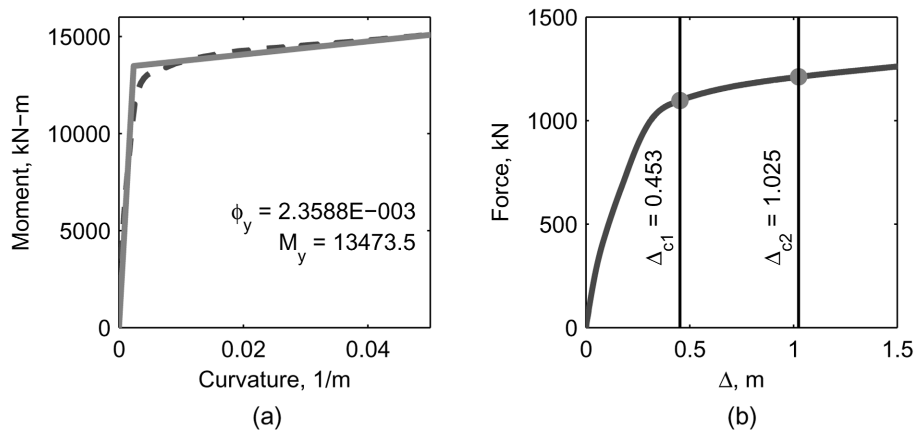


Fig. 15 (a) Moment-curvature relationship of the pile shaft section, (b) Pushover curve of the pile-soil system

A pushover analysis of the nonlinear finite element model of the column-pile-soil system model (Fig. 14(b)) led to the pushover curve of Fig. 15(b). During the pushover analysis, material strains were monitored, and displacement capacity at the selected design level were defined as the maximum displacement at the pile top without exceeding the material strains at the selected design level. The displacement capacities at Level 1 and Level 2 for this analysis are 0.453 m and 1.025 m, respectively (Fig. 15(b)). Let us denote the displacement capacities for the pushover analysis as "exact" values.

The displacement capacities computed form the simplified analysis at Level 2 match the "exact" value quite well: the value from simplified analysis if 1.07 m and "exact" value is 1.025 m. The value at Level 1 of 0.321 m from the simplified analysis is slightly less that the "exact" value of 0.453 m. Obviously, the error in the Level 1 displacement capacity is much larger than that for Level 2 displacement capacity; the slightly larger error for Level 1 occurs due to errors associated with idealizing the moment-curvature relationship with a bilinear curve. However, the simplified procedure provides a conservative estimate of the displacement capacity at Level 1 which may be "accurate" enough for preliminary design or for a quick check on the results from a detailed analysis.

9. Conclusions

This investigation proposes an approximate procedure to estimate seismic displacement capacity – defined as a product of the yield displacement and displacement ductility – of two types of piles – reinforced-concrete and hollow-steel – with either pin-connection or full-moment-connection used in the marine oil terminals. Development of this approximate procedure utilized equivalent fixity approximation for piles. A parametric study is conducted to demonstrate that the displacement ductility is relatively insensitive to various pile parameters such as pile length, pile diameter, longitudinal and transverse reinforcement, and pile axial load level for reinforced-concrete piles; and pile length, pile diameter, wall thickness, and pile axial load level for hollow-steel piles. This observation applies to ranges of these parameters that are applicable for piles typically used in marine oil terminals.

Subsequently, it is demonstrated that lower bound values of the displacement ductility of piles commonly used in marine oil terminals depend on the seismic design level – Level 1 or Level 2 – and location of the hinging in the pile – near the deck or below the ground. Based on the results of the parametric study, lower bound values of the displacement ductility are determined such that the material strain limits specified in the MOTEMS are satisfied at each design level. The recommended lower bound values of the displacement ductility of piles in marine oil terminals are: 1.75 for seismic design Level 1 (in-ground or pile-deck hinge formation), 2.75 for seismic design Level 2 (in-ground hinge formation), and 5.0 for seismic design Level 2 (pile-deck hinge formation) for reinforced-concrete piles; and 1.2 for seismic design Level 1 and equal to 2.75 for seismic design Level 2 for hollow-steel piles regardless of location of the plastic hinge. The applicability of the approximate procedure is finally demonstrated through an illustrative example.

The approximate procedure presented in this paper is intended to be used for preliminary design of piles or as a check on the results from the detailed nonlinear static pushover analysis procedure, with material strain control, specified in the MOTEMS. It is also useful to emphasize that displacement ductility values recommended in this paper correspond to the material strain limits specified in the MOTEMS. Therefore, these recommendations are strictly valid for piles in marine oil terminals designed according to the MOTEMS. However, the framework presented in paper may easily be used to develop such recommendations for other material strain limits.

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