Performance-based seismic assessment for pile-supported wharves – A case Study

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ABSTRACT

This study intends to cover aspects concerning the performance-based seismic assessment (PBSA) of pile-supported wharves following the state of the art and current design criteria given by The Port of Long Beach (POLB-WDC-2015) and the American Society of Civil Engineers (ASCE/COPRI 61-14) guidelines. The performance criteria is based on three earthquake levels: Operational Earthquake (OLE), Contingency Earthquake (CLE) and Design Earthquake (DE).

In this analytical study, the displacement demand was estimated by simplified bi-dimensional methods such as the elastic stiffness and substitute structure, as well as tri-dimensional analyses such as the response spectra and non-linear time-history. For the last two methods, the “SuperPile” concept was implemented. Since, the wharf study was assumed to be located in the Coast of Peru, zone of high seismic activity, the Peruvian Seismic Code, Norma E.030-2016, was adopted to determine the seismic demand under the three seismic levels.

The soil-pile interaction was considered by nonlinear springs, *p-y* curves, as per recommendations given by the API RP 2A WSD-2000 for sands.

The current seismic evaluation includes only the inertial loading effects on the structural system, neglecting the kinematic action of soil deformation on piles, as well as soil liquefaction potential or lateral spreading.

A single segment of a marginal wharf 39.2 m wide and 240 m long was evaluated. The wharf is supported by steel pipe piles, with embedded reinforced concrete section at the pile deck connection. Piles were modelled by inelastic beam-column elements with distributed plasticity model. Earthquake accelerograms were matched to the target response spectrum. The SeismoSoft software was used for this study.

The substitute structure method was determined to be a simplified and reliable method for this assessment. It was found that the OLE earthquake level becomes critical for the performance evaluation and the critical plastic hinges are located near the pile-deck connection.

Keywords: Soil-structure interaction, performance based design, seismic wharf design, superpile concept, substitute structure method.

INTRODUCTION

Earthquakes continue to cause severe physical, economic and social damage on ports infrastructure. Performance-based design (PBD) approach is used intensively today for the seismic design, improvement and repair of ports structures, due to the development of seismic design guides/codes such as the International Navigation Association (PIANC), Port of Los Angeles (POLA), Port of Long Beach (POLB), and recently the ASCE / COPRI 61-14.

The current Peruvian seismic code E-030, uses the forces-based design (FBD) approach mainly focusing on building structures; therefore it would be insufficient in a port structures application. The purpose of this research was aimed to cover the lack of seismic design guidelines for ports structures in Peru using a performance-based approach.

The PBD method in the ports industry uses the multi-performance level approach, mostly following research and recommendations from the bridge industry with a strong influence from Priestley et al. (1996, 2007). The performance criteria is based on pile material strain limits for each of the three seismic hazard level: OLE, CLE and DE, as shown in Table 1. It must be verified in each application that the displacement capacity is greater than the displacement demand for each seismic level. The displacement capacity is generally calculated by non-linear static analyses like pushover, while the demand is obtained by equivalent linearization methods such as the substructure method, the response spectrum (RSA), and the non-linear time-history (NLTH) analyses.

PERFORMANCE-BASED DESIGN PRACTICE

The State of California has developed guidelines for port structures such as the Marine Oil Terminals Guideline (MOTEMS, 2013), the POLA (2010), and POLB (2015). The ASCE/COPRI 61-14 standard was developed for use in all US states, but it does not address port structures covered by other guides such as LNG terminals, nor structures for public access.

The current POLA, POLB and the ASCE/COPRI guidelines have established three performance levels: Operating level earthquake (OLE) or "Minimal damage" for a seismic hazard of 72-year return period; Contingency level earthquake (CLE) or "Controlled or reparable damage" for a seismic hazard of 475-year return period; and Design earthquake (DE) level or 'Life safety protection" for a seismic hazard to be consistent with the corresponding local building code.

The performance criteria for each earthquake level is based in terms of operability, reparability, and safety concerns. For the OLE level, it is required that operations continue without interruption, and allows for only minor (cosmetic) or no structural damage, all damage shall be reparable and visually observable and accessible, and repair shall not interfere with wharf operations. For CLE level, the structure shall undergo a temporary loss of operations (2 to 3 months could be acceptable), controlled and ductile behavior, limited permanent deformations, and all damage shall be reparable and visually observable and accessible. For the performance DE or Life Safety Protection level, structure shall not collapse, the wharf shall be able to support the dead loads of the structure including cranes, and the life safety shall be kept.

Strain Limits

The strain limits for solid reinforced concrete piles and steel pipe piles studied here, are briefly summarized in Table 1.

Table 1. Strain Limits as per POLB, POLA y ASCE.

Agency	Pile / location	Component, Deformation	Performance level		
			OLE	CLE	DE
POLA, 2010 POLB, 2015 ASCE/ COPRI 61-14	Solid reinforced concrete, top plastic hinge	Concrete, compression ε_c	0.005	$0.005 + 1.1\rho_s \leq 0.025$	No Limit
		Reinforcement, tension ε_s	0.015	$\leq 0.6\ \varepsilon_{smd} \leq 0.06$	$\leq 0.8\ \varepsilon_{smd} \leq 0.08$
	Steel pipe, buried plastic hinge ≤ 10Dp	Structural steel, compression and tension ε_s	0.010	0.025	0.035

ρ_s= Effective volumetric ratio of confining Steel. ε_{smd} = Strain in the dowel at máximum stress. D_p= Pile diameter.

The concrete compressive strain limit ε_c=0.005 established for OLE, is an increase from a conservative 0.004 used to mark the onset of spalling in early literature (POLA, 2004), since the adjacent deck/beam at the top pile section would provide an additional lateral restraint (Priestley, 2007). Also, it would take into account the extensive redundancy provided by this multiple- piles structure.

The tension strain limit of the longitudinal reinforcement ε_s=0.015, is referred to the strain at which the residual cracking width would exceed 1 mm. (Priestley, 1996). Frequently, this width is considered as the maximum cracking width that could be tolerated under normal environmental conditions without requiring repair.

According to previous studies (Priestley et al., 1996 and 2007), the ultimate concrete compression strain ε_{cu} for CLE level, is generally established when the confining reinforcement fracture begins. This condition is obtained by equating the increase of deformation energy absorbed by the concrete, as a result of confinement, with the deformation energy capacity of the confining steel. As a result, the following expression is obtained: $\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh} \varepsilon_{su}}{f'_{cc}}$; where, $f_{yh}$is the yield stress of the confining reinforcement, $\varepsilon_{su}$is the tension strain at the maximum stress of the confining reinforcement, and $f'_{cc}$is the compressive strength of the confined concrete. The above expression for $\varepsilon_{cu},$ was considered conservative at sections subject to flexion or combined flexo-compression; as well as the additional confinement that the adjacent deck/beam would provide to the pile upper section. Currently, concrete compression strain limit adopted for CLE level is given by: $\varepsilon_{cu} = 0.005 + 1.1\rho_s \leq 0.025$.

For the CLE level, the maximum tensile strain of the longitudinal reinforcement ε_s is limited to a value of 0.06, less than the deformation at the maximum stress ε_{su} (0.09 to 0.012). The reason for this is that, once large tensile deformations is developed, the longitudinal reinforcement is susceptible to buckling when subjected to a reverse load that puts the reinforcement in compression. The buckling typically occurs before the cracking by bending is closed, and while the reinforcing bars are still subject to tensile strain (but with compressive stresses) (Priestley, 2007). In addition, tests on pile-deck connections with dowels show that these bars could fracture when the unit strain exceeds ε_s= 0.06. (ASCE / COPRI 61-14).

The longitudinal reinforcement strain limit of: 0.8 $\varepsilon_{smd} \leq 0.08$ for DE level, prevents the possibility that some of the dowels could be fractured. The fracture of some bars in a pile-deck connection, reduce the capacity of moments and the resistance to the global turning of the structure; therefore, the code requires a stability check to be carried out assuming an articulated connection.

CASE STUDY

Structure Geometry

A single segment marginal wharf of 240 m long and 39.2 m wide described by Figures 1 and 2 was studied. The structure is supposed to be located in a zone of high seismicity in Peru. For the purpose of assessment, the wharf is considered to be previously designed.

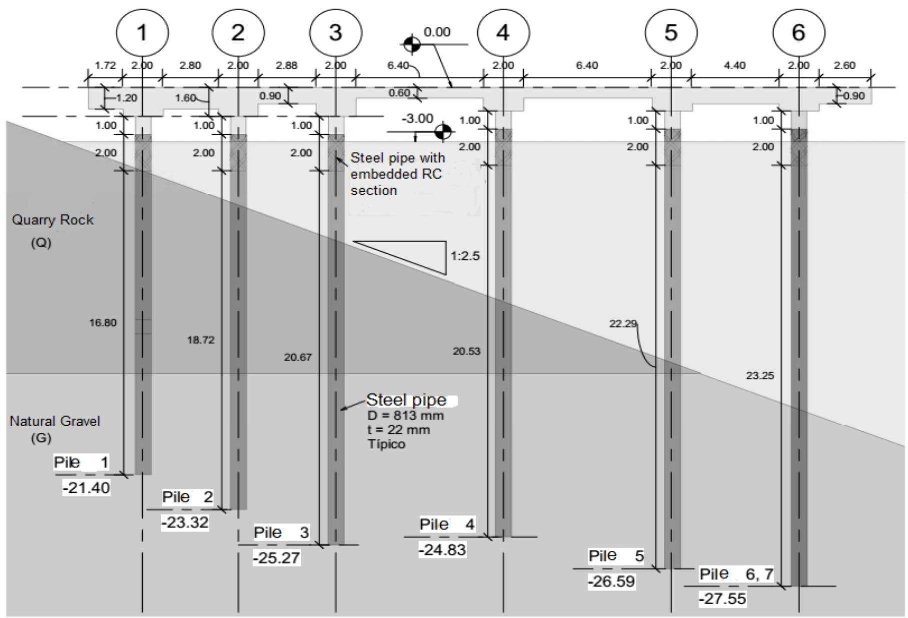


Figure 1. Cross-section of case study.

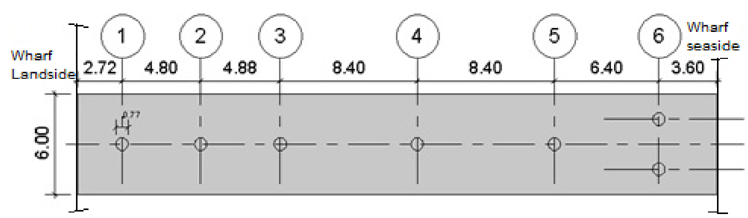


Figure 2. Plant view of a typical wharf strip 6 m long. Axis 6 contains two piles spaced at 3 m.

The wharf segment is supported by 280 piles, each pile with three different sections. The top reinforced concrete section at the pile-deck connection of one meter long is detailed in Figure 3. Below, a compound transition-section of 2m long of steel pipe 813 mm diameter and 22 mm thick is embedded with reinforced concrete similar to the top section. The lower-section is an extension of the steel pipe.

Material properties

The pile seismic capacity, except shear, is obtained by using the most probable (expected) material properties. Unconfined concrete compressive strength of: $f'_{ce} = 1.3f'_c = 52\ MPa$; the yield strength of longitudinal reinforcing steel: $f_{ye} = 1.1f_y = 462\ MPa$; yield strength of the confining steel: $f_{yhe} = 1.0f_{yh} = 420MPa$, and yield strength of structural steel (pipe): $f_{ye} = 1.1f_y = 264\ MPa$ are used in this case.

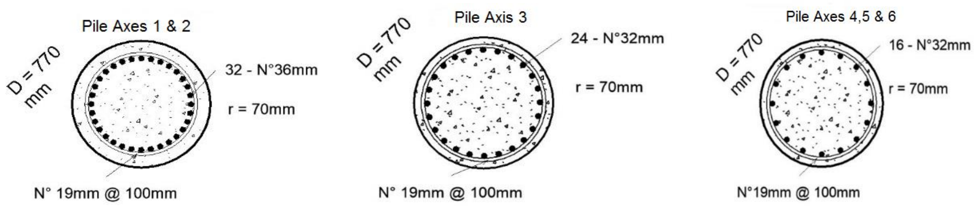


Figure 3. Cross-section details for piles top- section.

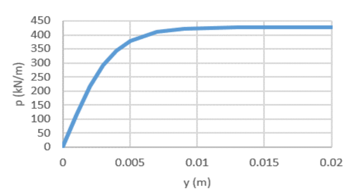
The nonlinear concrete model follows the constitutive relationship proposed by Mander et al. (1988) and the hysteretic rules proposed by Martinez-Rueda & Elnashai (1977). The model for reinforcing steel as per Rodd-Restrepo (1995), and for the steel pipe as per Menegotto-Pinto (1973) are used. The hysteresis model of Modified Takeda (fat) with α= 0.3 and β=0.5 was used to represent the inelastic characteristics of the "SuperPile" for the NLTH, where α is the unloading stiffness degradation parameter and β is reloading stiffness parameter. These are appropriate parameters to model flexural ductile reinforced concrete members. Piles are modelled by inelastic beam-column elements using a distributed plasticity model, and the deck is modelled by linear elastic beam elements.

Soil-pile interaction

The soil-pile interaction is simulated using a series of non-linear *p-y* springs, as recommended by the American Petroleum Institute (API RP 2A WSD, 2000). The upper soil stratum is quarry rock and the lower is natural gravel with assumed effective soil weight of 9 kN/m^3 and 11 kN/m^3 , and angle of internal friction of 40 degrees and 35 degrees, respectively.

A curve *p-y* for pile in axis 1 and at a depth of 3 m is shown in Figure 4, where *p* is the lateral soil resistance in *kN* per linear meter, and *y* is the lateral deflection in meter. The upper bound (UB) and lower bound (LB) soil springs have been used, as per wharf design practice. (POLB, 2015). Later, the *p-y* curves are adjusted in the form of bilinear curves.

Parameter	y (m)	p (kN/m)
H (m)	0	0.000
3.00	0.001	116.103
D (m)	0.002	216.189
0.81	0.003	292.007
ϕ' (deg.)	0.004	344.006
40.00	0.005	377.285
A	0.007	409.769
0.9	0.009	420.978
pu	0.011	424.712
473.93	0.013	425.941
k (kN/m^3)	0.015	426.344
39701.85	0.018	426.503
	0.02	426.528



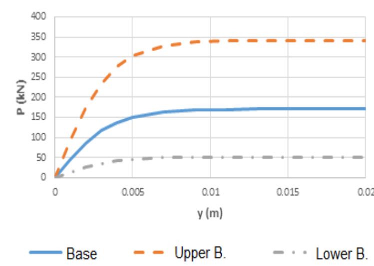
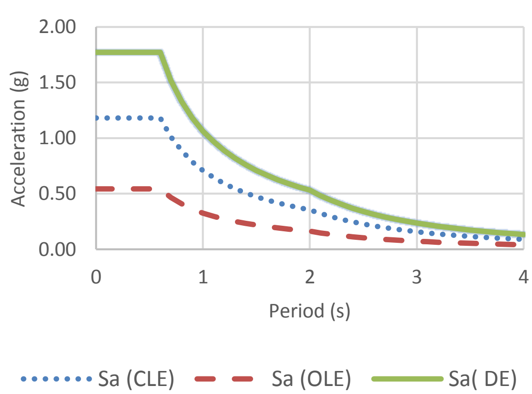


Figure 4. Typical p-y curve (left) and p-y curves for upper and lower bound soil springs.

A typical pile discretization below ground is 10@0.4m, 5@0.8m, 3@1.6 m, and 1@3.8 m. The calculated seismic mass for the 6 m wharf strip is 737.4 m-t, including concrete deck, 10% of design uniform live load (4kN/m^2), and 1/3 of the pile mass.

Seismic Hazards

The structure was assessed in accordance with the elastic pseudo-acceleration spectrum response consistent with that recommended by the Peruvian seismic code E.030-2016 for the OLE, CLE and DE levels, described in Figure 5.



S_a=*ZUCS*

Zone Factor Z: Z_{OLE} = 0.207 g, Z_{CLE}= Z_{DE} = 0.45 g

Soil Factor S=1.05 (Soil Profile S_2: *intermediate*)

Seismic Amplification Factor C. T_P= 0.60, T_L= 2.00

$T < T_P \quad C = 2,5$

$T_P < T < T_L \quad C = 2,5 \cdot \left(\frac{T_P}{T}\right)$

$T > T_L \quad C = 2,5 \cdot \left(\frac{T_P \cdot T_L}{T^2}\right)$

Figure 5. Peruvian E.030-2016 design acceleration spectra, (ξ= 5 %)

The Use Factor, U=1, for OLE and CLE level is used, while for DE a factor U=1.5 is used for essential structures, as per E.030. By applying the Use Factor of 1.5, the PGA for DE level is 0.675g.

For the time-history analyses, three Peruvian sets of real ground motions each with two orthogonal components (E.030) were used: Lima 1966 (8.1Mw), Lima 1974 (6.6 Mb), and Pisco 2007 (7.0 ML). The accelerograms were adjusted to fit the spectral response OLE, CLE and DE, by using the SeismoMatch software (2016), which is an application based on the wavelets algorithm proposed by Abrahamson (1992) y Hancock et al., (2006).

Nonlinear static pushover analysis

The wharf displacement capacity is calculated by pushover analysis in the transversal direction, for a one strip segment of 6 m, where loading is monotonically increased. The displacement versus the base shear is represented by the capacity curve shown in Figure 8. This process was performed by the SeismoStruct software (2016), which allows to identify when the strain limits are reached.

Since the wharf structural system is based on a strong beam (the deck) and weak column (pile) concept, all plastic hinges are designed to occur in the piles. The effective top of the pile is located a distance L_{sp} into the deck, to account for strain penetration. The member between the strain penetration and the center of gravity of the deck is a rigid link (i.e. 50x). The length of strain penetration is: $L_{sp}(mm) = 0.0145\, f_{ye}\, d_{bl}$, where d_{bl} is the reinforcing bar diameter (mm), and f_{ye} is the reinforcing expected yield strength (MPa). (POLB, 2015)

Demand Analysis

The displacement demand is calculated by simplified methods such as the initial stiffness method, and the substitute structure method (Shibata & Sozen, 1976), and by dynamic procedures such as the RSA and NLTH analyses.

Substitute structure method

The substitute structure method is widely recommended by most wharf design guides. This iterative procedure replaces the non-linear system by an equivalent linear system or substitute structure defined by the secant stiffness, and incorporates the equivalent damping represented by a combination of elastic and hysteretic damping used to estimate the reduced spectral displacement.

The main differences found among the guidelines is the expressions used to estimate the effective damping ratio $\xi_{eff,n}$ and the Dynamic Magnification Factor (DMF), the latter is used to adjust the displacement demand due to torsional effects. POLB and POLA adopted the expression given by equation (1), based on the work developed by Jaradat & Priestley, (2013); while, ASCE/COPRI and MOTEMS adopted equation (2) developed by Kowalsky (1994), considering the Modified Takeda model (Takeda et al., 1970). Where, μ_n is the displacement ductility at iteration n defined by the ratio of the target displacement Δ_d over the yield displacement Δ_y, and r is the ratio of the second slope over the elastic slope of the pile stiffness curve. A damping correction factor of $R_\xi = \sqrt{(10/(5+\xi))}$ (Eurocode EC8) was used to adjust the displacement demand.

By applying the above equations in the substitute structure method, the effective damping ratio calculated by POLB (eq. 1) for CLE and DE level is around 14% y 20%, respectively; while by using ASCE/COPRI (eq. 2) lower ratios of 10% and 16% are obtained. The higher damping ratios for DE level is explained by the greater inelastic incursion expected in a more intense earthquake.

$$\xi_{eff,n} = 0.10 + 0.565\left(\frac{\mu_n - 1}{\mu_n \pi}\right) \tag{1}$$

$$\xi_{eff,n} = 0.05 + \frac{1}{\pi}\left(1 - \frac{1-r}{\sqrt{\mu_n}} - r\sqrt{\mu_n}\right) \tag{2}$$

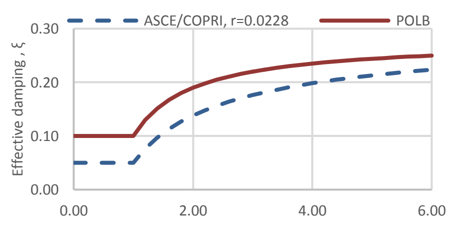


Figure 6. Effective damping ξ and ductility relationship, as per POLB and ASCE.

Considering the accumulative effects for damping and dynamic modification factor (DMF), the final displacements demand under CLE and DE seismic levels differ by no more than 10% between POLB and ASCE. However, for the OLE level, the displacements demand differ substantially between the two approaches (POLB are approximately 24% higher than obtained by the ASCE/COPRI), as it can be seen in Table 2.

Dynamic Analysis

The RSA and the NLTH analyses were carried out for the tri-dimensional analysis by implementing the “SuperPile” concept to simplify the analysis. These provide equivalent translational and torsional stiffness of the actual structure. (Priestley et al., 2007). The 240 m x 39.2 m wharf segment with 280 piles, is represented by an equivalent structure with four “SuperPile”. Each “SuperPile” is represented by 2-D horizontal springs, providing lateral stiffness in transversal and longitudinal direction. “SuperPile” stiffnesses are obtained from pushover curves, as contribution of the classified seismic and gravity piles. For the RSA, linear stiffnesses are used, while for NLTH, stiffnesses represented by bi-linear characteristics and hysteretic model of Modified Takeda are used. “SuperPile” location, as well as stiffness center (SC) and mass center (MC) location under UB soil condition are shown in Figure 7.

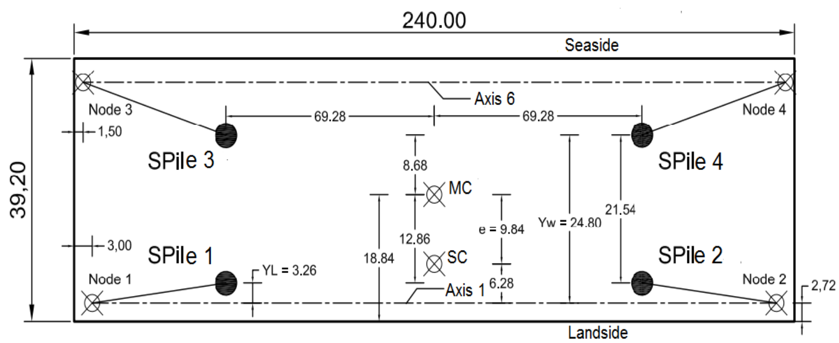


Figure 7. SuperPile location for UB soil springs. .

The displacement demand results calculated by static and dynamic analysis under the three seismic hazards are shown in Table 2. By comparing the displacement obtained by the substitute structure method and by the time-history analysis, the overall differences are around 15% for CLE, and 5% for DE levels, which are within the 20% difference recommended by POLB. For the OLE level, in which the structure behaves in the linear elastic range, the critical damping ratio was reduced to 5% to find consistent values under UB soil condition. For the OLE level, POLB resulted on higher displacements than ASCE, since the calculated DMF are substantially higher using POLB. Also, for OLE level, displacements obtained by RSA and ASCE substitute structure method are consistent between each other, with differences no more than 5%.

Performance Assessment

The displacement capacity is obtained by pushover analysis on the wharf cross-section and correlated to the instant at which the first strain limits are reached for OLE, CLE and DE level. The capacity curve for UB soil condition is shown in Figure 8. For OLE level, the first concrete compressive strain $\varepsilon_c = 0.005$ is reached when the platform displacement is 0.110 m. For CLE level, the first concrete compressive strain $\varepsilon_{cu} = 0.025$ is reached when the platform displacement is 0.430 m, and for DE level, the first longitudinal reinforcement tension strain ε_s=0.8 ε_{smd}=0.072 is reached for a displacement of 0.650 m. Note, these strain limits are reached on the top hinge of pile axis 1 (landside).

Table 2. Displacement demand in meters calculated by different methods.

Method	OLE		CLE		DE	
	UB	LB	UB	LB	UB	LB
Initial stiffness method, POLB 2015	0.099	0.140	0.199	0.250	0.297	0.373
Initial stiffness method, ASCE/COPRI 61-14	0.076	0.105	0.165	0.230	0.246	0.345
Substitute structure method, POLB 2015	0.101	0.142	0.161	0.201	0.273	0.334
Substitute structure method, ASCE/COPRI 61-14	0.077	0.107	0.161	0.222	0.263	0.362
Modal Response Spectra Analysis, CQC	0.079	0.111	0.172	0.241	0.258	0.361
NLTH, 10% damping proportional to tangent stiffness	0.071	0.070	0.182	0.171	0.264	0.320
NLTH, 5% damping, proportional to initial stiffness	0.114	0.104				

From the results, the displacement demand is lower than the displacements capacity for both the upper and lower bound soil condition. The higher ratio demand/capacity corresponds to the OLE level and the UB soil condition, which means this condition becomes critical for the performance evaluation.

In general, the displacement demand for LB soil condition is higher than UB; therefore this soil condition could be critical for the connections design between service lines with the structure.

In Figure 8, it is shown the pushover curve with strain limits, marked on the curve, and the displacement demand, in vertical lines, for the three events obtained by the substitute structure method, as per POLB, 2015. It should be noted that, the strain limits are not exceeded under the displacement demand in the corresponding events; therefore, the wharf meets the performance criteria at all earthquake events.

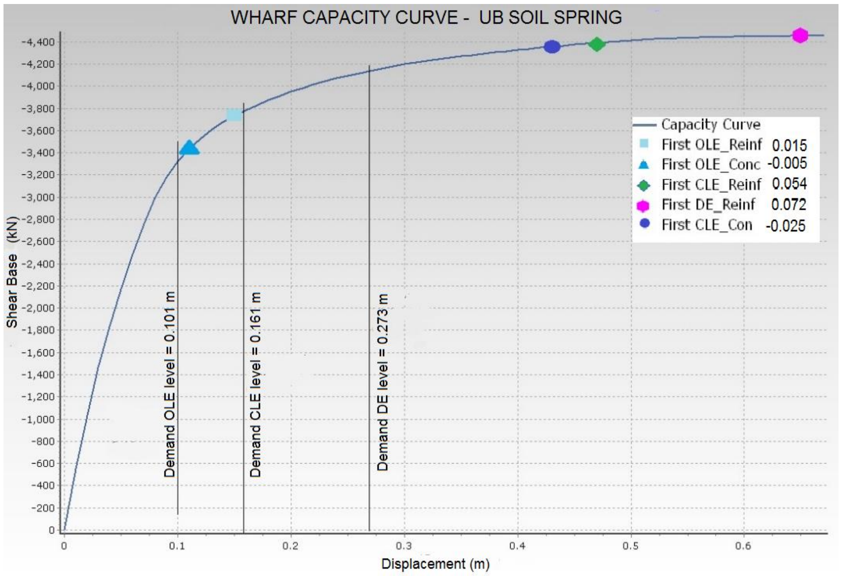


Figure 8. Pushover curve with strain limits and displacement demands under UB soil condition.

Table 3 shows the material strain reached, estimated damage assessment and the performance assessment for each seismic hazard. The first damage start at the pile top at the pile-deck connection from the pile on axis 1 (landside) and then extend to other piles toward the seaside, then the damages extends to the in-ground pile.

Table 3. Performance assessment information.

EQ Level	Material	Strain Limits	Maximum strain at demand (UB)	Strain ratio demand / limit	Plastic hinge location	Damage assessment (**)	Performance Assessment
OLE	Concrete	-0.005	-0.0047	0.94	Top of concrete pile	Initiation of cover spalling (axis 1)	Minimal Damage
	Reinforcement	0.015	0.0076	0.51		First yield (axes 1 to 4).	
	Steel pipe	0.01	0.0021	0.21	In-ground, 4Dp (*)	First yield (axis 1)	
CLE	Concrete	-0.025	-0.0078 (cc)	0.31	Top of concrete pile	Cover spalling (axis 1 to 3). Concrete cracking slightly over 1mm. (axis 1)	Controlled and Repairable Damage
	Reinforcement	0.054	0.0165	0.31		Residual cracking slightly above 1mm width. First yield (axes 1 to 5. No buckling, nor fracture of longitudinal reinforcement	
	Steel pipe	0.025	0.0062	0.25	In-ground, 4Dp (*)	First yield (axes 1, 2).	
DE	Concrete	No limit	-0.0148 (cc)		Top of concrete pile	Cover spalling (axis 1 to 4), Concrete cracking over 1mm width (axis 1 to 4). Confined concrete not affected. No fracture of confining reinforcement.	Controlled and Repairable Damage
	Reinforcement	0.072	0.0305	0.42		No fracture of longitudinal reinforcement	
	Steel pipe	0.035	0.015	0.43	In-ground, 4Dp (*)	First yield (axis 1, 2).	

(*) Plastic hinge location is at 4*Dp* for UB and 7*Dp* for LB. *Dp* = pile diameter (0.81 m)

(**) Damage assessment inferred by the authors, following the intent of guidelines/codes, and related investigation (i.e. Priestley, 1996). None of the codes/guidelines explicitly provide correlation between damage and strain limits strain.

For reference, an estimate of the displacement distortion ($\delta = \Delta / Lc$) at the top hinge of the critical pile is calculated, since codes refer to strain limits as the only performance definition parameter. Δ is the displacement capacity of pile head respect to the point of contra-flexure. *Lc* is the distance from the plastic hinge center to the point of the contra-flexure. Distortion values of 1.8%, 6.7% and 10.2% are obtained at top plastic hinge for UB soil condition under OLE, CLE and DE levels.

The displacement capacity distortion of the in-ground plastic hinge, where the relative displacement is obtained between the point of contra-flexure and the in-ground plastic hinge center, for OLE, CLE and DE level are 3.1%, 7.0 % and 10.1%, respectively.

CONCLUSIONS

A performance assessment of a single marginal wharf segment is conducted as per POLB 2015 and ASCE/COPRI 61-14, which in essence have similar requirements. The Peruvian Seismic Code, Norma E.030-2016, was adopted to determine the seismic demand for the three earthquake levels OLE, CLE and DE. Nonlinear *p-y* curves are incorporated to represent the soil-pile interaction. The "SuperPile" concept is implemented for the response spectra and time-history analyses. Displacement demands obtained by the simplified static procedures are consistent with those obtained by dynamic analysis.

From this study, the displacement capacity for DE resulted six times larger respect to OLE, while the capacity for CLE is four times larger than OLE.

For OLE level, it was found that the displacement demand as per ASCE resulted approximately 25% lower respect to those from POLB, and this difference is due to different approach used to estimate the Dynamic Magnification Factor (DMF). At the same OLE level, a better results correlation was found between ASCE and the RSA (less than 5% difference), than between POLB and RSA (27% difference). Results from POLB tend to be more conservative at this demand level. For OLE level, since, the structure behaves close to a linear elastic range, the RSA would provide reliable results.

The effect of soil stiffness impacts on the pile plastic hinges location. For the critical pile, in-ground hinges are located at 4*Dp* for UB soil conditions and 7*Dp* for LB soil condition, where *Dp* is the pile diameter.

From this case study, the OLE level represents the most critical condition for the performance assessment, since at this level were reached the highest strain/displacement ratios (Table 3). From the assessment, it can be concluded that the studied wharf meets the performance requirements for all three earthquake levels, and only reaches controlled and repairable damage states at DE level.

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