PROPOSAL OF PERUVIAN GUIDELINES FOR THE SEISMIC DESIGN OF PILE SUPPORTED WHARVES

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Abstract

Ports are very important structures for the development of a region, so, it is advisable to implement guidelines or recommendations that can be used for the design of this type of structures. Currently, there is no a Peruvian seismic design code that can be used directly for the seismic design of ports.

The purpose of this paper is to present a proposal for Peruvian seismic design guidelines of pile supported wharves that follows the criteria for performance-based seismic design. The proposal considers the recommendations presented by the Port of Long Beach (POLB-WDC-2015), the Port of Los Angeles (POLA, 2010), and the American Society of Civil Engineers (ASCE/COPRI 61-14) design codes. The performance criteria are defined on three earthquake levels: Operational Earthquake (OLE), Contingency Earthquake (CLE) and Design Earthquake (DE). The current Peruvian Seismic Code, Norma E.030-2016, is used as a base to determine the seismic demand for the three motion levels. The proposal includes recommendations to take into account the soil structure interaction between the piles and the supporting soil by means of non-linear springs known as p-y curves.

As an example, the performance based assessment of a single segment of a marginal wharf with 36.2m width and 240m long is presented. Displacement demands for all seismic levels are estimated using various models, such as the Elastic Stiffness, Substitute Structure, and Tri-Dimensional Model, in which the “super-piles” concept was taken into account. Analyses include response spectral and non-linear time-history.

Keywords: Performance based design; wharves seismic design

1. Introduction

Ports are vital components of infrastructure for the support and economic development of the countries, most of which are located in areas of seismic risk. So, earthquakes may produce severe physical, economic and social damage such as those caused by the earthquakes of Loma Prieta-California 1989, Kobe-Japan 1995, Haiti 2010, Maule-Chile 2010 among others.

After the Loma Prieta earthquake in 1989, research projects started with the objective of improving seismic design criteria, underlining the use of displacements instead of forces as a measure of seismic demand related to the structure damage. By using these displacement based design methodologies it is more likely that a performance based seismic design can be achieved.

The philosophy of performance-based seismic design is being used intensively today for the design, improvement and repair of port structures, thanks to the development of seismic design guidelines and codes such as California Maritime Oil Terminals (MOTEMS, 2013) [1], the Port of Long Beach (POLB, 2015) [2] and the Port of Los Angeles (POLA, 2010) [3]. Also, a mention should be made of the existence of other port structure design guides such as that of ASCE/COPRI 61-14 [4] and the International Navigation Association (PIANC, 2001) [5].

The performance-based seismic design process allows the evaluation of the performance of a structure for events given to certain levels of seismic risk, in relation of engineering parameters of demand such as displacements, curvatures, forces, deformations. Damage can occur in the form of cracking and/or crushing of concrete, fracture of the reinforcement or permanent deformation among others. Also, damage can be related to direct risks such as loss of functionality, accidents, loss of life, repair costs or downtime, in addition to indirect costs due to economic losses. The desired performance objectives of the structure are generally established by the users or owners of the facilities, based on the acceptable level of structural damage and associated risks.

The current Peruvian Seismic Resistant Design Code, Norma E.030 [6], uses the force-based design approach. Therefore, it could not be applied directly for the design and or evaluation of port structures, if a performance-based design procedure would be required.

Many existing port structures in Peru have been designed to standards different than current international codes. Therefore, some of them could be suitable for the seismic evaluation from the point of view of their performance against potential seismic risks. It is then necessary to also address specific aspects to the seismic evaluation of existing structures.

The current paper is aimed for a proposal of seismic guidelines that can be used for the design, evaluation and retrofitting of ports structures in Peru. The presented proposal for Peruvian seismic design guidelines of pile supported wharves follows the criteria for performance-based seismic design and considers the recommendations presented by the Port of Long Beach (POLB) [2], the Port of Los Angeles (POLA) [3] and the American Society of Civil Engineers (ASCE/COPRI) [4] design codes. The current Peruvian Seismic Code, Norma E.030 [6], is used as a base to determine the seismic demand for the motion levels.

2. Seismic Design Guidelines Proposal

2.1 Performance-based Levels

Three performance levels are recommended: Operating Level Earthquake (OLE) or minimal damage, Contingency Level Earthquake (CLE) or controllable repairable damage and Design Earthquake (DE) level or life safety protection.

2.2 Performance-based Criteria

The design approach is based on deformation limit criteria and performance objectives associated with levels of structural damage, reparability and life safety for each earthquake level:

Performance for the Operational Level Earthquake (OLE)

No interruption in operations. Forces and deformations should not result in structural damage. All damage must be cosmetic in nature and located where it is visually observable and accessible. Repair should not interfere with dock operations.

Performance for the Contingency Level Earthquake (CLE)

Temporary loss of operations of few months may occur. Forces and deformations could result in limited inelastic behavior and limited permanent deformation. All damage must be repairable, visually observable and accessible for repair.

Performance for the Design Earthquake (DE)

Forces and deformations should not result in structural collapse of the structure, and the wharf should be able to withstand dead loads of the structure including cranes. Life safety must be maintained.

2.3 Strain limits

The strain limits [7, 8] for solid reinforced concrete piles and steel pipe piles associated with earthquake levels and damage control recommended are summarized in Table 1.

Table 1 – Strain Limits

Performance Level	**Pile/location**		
	Solid reinforced concrete, top plastic hinge		Steel pipe, buried plastic hinge <= 10 D_p
	Concrete compression, ε_c	Reinforcement tension, ε_s	Structural tension and compression, ε_s
OLE	0.005	0.015	0.010
CLE	0.005 + 1.1 ρ_s <= 0.025	0.6 ε_{smd} <= 0.06	0.025
DE	No limit	0.8 ε_{smd} <= 0.08	0.035

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

2.4 Ground Motion Levels

The ground motions associated with the three performance levels are: The OLE earthquake is defined as the seismic event that produces ground motions associated with a 72-year return period. The 72-year return period have a 50 % probability of being exceeded in 50 years. The CLE earthquake is defined as the seismic event that produces ground motions associated with a 475-year return period. The 475-year return period have a 10 % probability of being exceeded during 50 years. The DE earthquake is defined as the seismic event that produces ground motions that is consistent with the level given in the Peruvian building code, Norma E030 [6], for the design of ports. This motion could be associated for a 2500-year return period. The 2500-year return period have a 2 % probability of being exceeded during 50 years.

The elastic design accelerations spectrum (Sa) for all performance levels, for a damping factor of 5%, is estimated by the expression:

$$Sa = Z\ U\ S\ C \tag{1}$$

Where: Z is the zone factor, U is the importance factor, S is the soil factor and C is the seismic amplification factor.

The zone factor, Z, for the CLE motion is obtained from the zone factor given in the Peruvian Norm E030 since it correspond to the same return period of 475 years. For the OLE motion the factor should be obtained from a seismic risk study; the application developed by Sencico [9] can be very useful for this purpose. For the DE motion the zone factor is defined equal to the CLE motion.

The important factor, U, for the OLE and CLE motions is set to U=1.0, For the DE motion the importance factor is defined as U=1.5 as given by the Norm E.030 [6] for port structures.

The soil factor is obtained from the Peruvian Norm E.030 [6] based on the soil profile and zone factor. The seismic amplification factor, C, is estimated as defined in the Peruvian Norm E.030 [7] based on the soil profile and period of the structure.

The elastic accelerations spectrum response based on the recommendations presented above for the OLE, CLE and DE levels, given an intermediate soil is presented in Fig. 1.

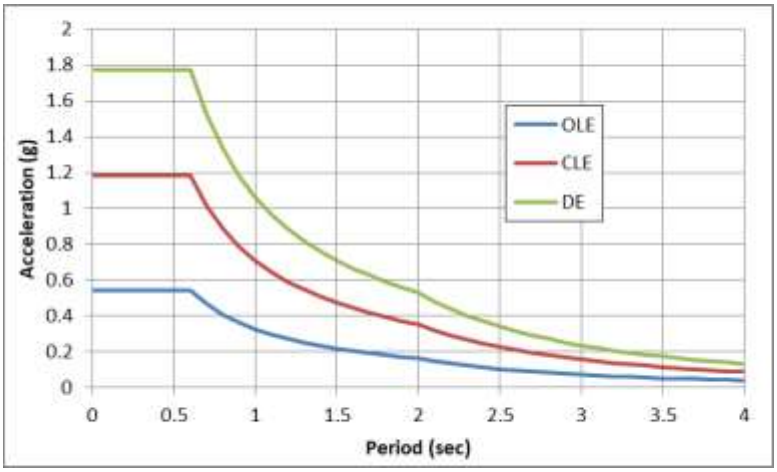

Fig. 1 — Design acceleration spectra for an intermediate soil (ξ=5%)

For time-history analyses, at least three sets of ground motions with two orthogonal components should be considered. The accelerograms should be adjusted to match the spectral response for the OLE, CLE and DE earthquake levels.

2.5 Soil Structure Interaction

The lateral interaction between soil and piles is taking into consideration by using a series of non-linear springs, known as p-y curves as depicted in Fig 2. The p-y curves should be estimated based on the soil properties, as recommended by the American Petroleum Institute [10] or other appropriate references [11, 12, 13].

To consider the uncertainties of the soil properties in the construction project an upper bound (UB) and lower bound (LB) nonlinear soil springs should be taken into account. A factor of 2.0 should be used to estimate the upper bound springs and 0.3 for the lower bound springs.

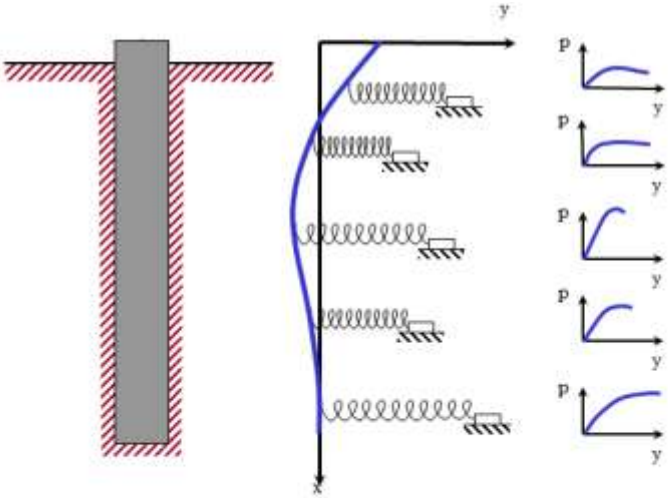

Fig. 2 – Lateral model for soil structure interaction consideration

For the soil interaction between soil and piles in the axial direction, the nonlinear t-z curves should be considered for the side contribution and the Q-w curves should be considered for the axial contribution. The curves should be estimated based on the soil properties, as recommended by the American Petroleum Institute [10] or other references [11, 13]. Similarly to the lateral springs, upper bound (UB) and lower bound (LB) nonlinear soil springs should be taken into account. A factor of 2.0 should be used to estimate the upper bound axial springs and 0.3 for the lower bound axial springs.

2.6 Nonlinear Static Analysis (Pushover)

An incremental lateral load or displacement is applied monotonically until desired displacements or other deformation limit states are obtained. The displacement control point corresponds to the level of the wharf platform.

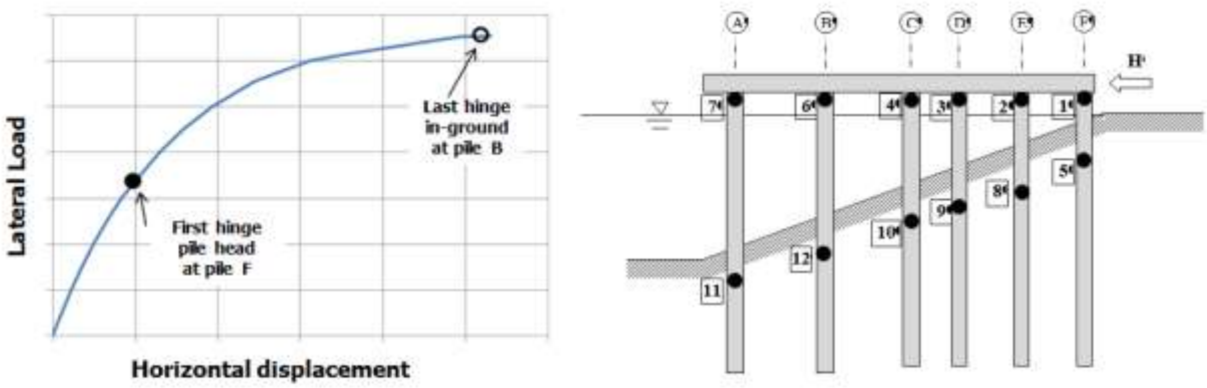

Fig. 3 – Typical pushover curve and development of plastic hinges

This method not only allows the estimation of displacement capacity of the structure for the different earthquake levels but also provides the lateral stiffness parameters of the piles needed to calculate the demand for displacements. The method, also known as Pushover Analysis, is sufficiently suitable for seismic analysis of wharves supported on piles, in which the dynamic response of its cross-section is generally governed by the response of the first mode.

The nonlinear analysis is carried out in the transversal direction of the wharf. The analysis will also provide information about the sequence of development of the plastic hinges in the piles. Since the wharf structural system is based on strong beam (deck) and weak column (pile) concept, all plastic hinges are designed to occur in the piles. A typical pushover curve is presented in Fig. 3.

To evaluate the seismic capacity of the piles, as ductile members, the expected properties of the materials should be used. The stress-strain model for confined and unconfined concrete widely used today is the model developed by Mander et al. [14], as shown in Fig 4. This is a unified stress-strain model for concrete subjected to static or dynamic loads, either monophonically or cyclically. The stress-strain model of the reinforcing steel has a linear elastic initial portion, a horizontal creep portion and a hardening range in which the stress increases with deformation as shown in Fig 4.

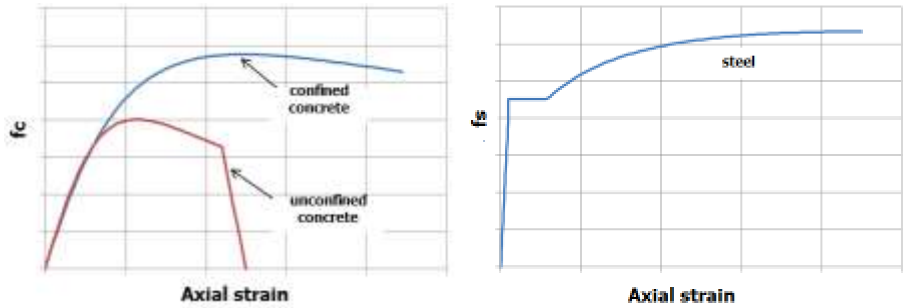

Fig. 4 – Typical stress strain curves for material behavior

For the static nonlinear analysis in addition of the dead load a 10% of the live load should be taken into account. In addition the dead weight of one third of the free pile length should be considered.

2.7 Demand Analysis

The displacement demand is calculated by simplified methods: 2D models such as the initial stiffness method, and the substitute structure method, and by 3D models such as the superpile method.

2.8 Initial Stiffness Method

The initial stiffness method is an equivalent method of analysis that is applied to a cross section in the transverse direction. The transverse elastic stiffness, k_i, is obtained from the bilinear approximation of the pushover curve at the point of first yielding, as depicted in Fig. 5.

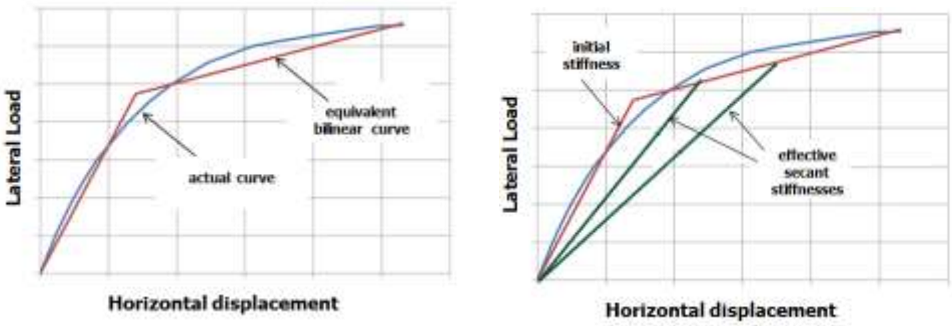

Fig. 5 – Effective stiffnesses for the initial stiffness and substitute structure methods

The period of the structure, T, associated to the initial stiffness can be obtained by the expression:

$$T = \sqrt{\frac{m}{k_i}} \tag{2}$$

Where: m, is the mass of the cross section being analyzed.

Then, the displacement demand, D, is obtained from the accelerations spectra, A, using the equation:

$$D = A * \left(\frac{T}{2\pi}\right)^2 \tag{3}$$

To consider the amplification of the transverse response due to the longitudinal excitation, the transverse displacement should be multiplied by the dynamic amplification factor (DMF). For a single segment the following expressions can be used:

$$\begin{gathered} DMF = 1.8 - 0.05\ L_l/B >= 1.1, \text{ for OLE} \\ DMF = 1.65 - 0.05\ L_l/B >= 1.1, \text{ for CLE, DLE and upper bound springs} \\ DMF = 1.50 - 0.05\ L_l/B >= 1.1, \text{ for CLE, DLE and lower bound springs} \end{gathered} \tag{4}$$

Where: L_L is the length of the wharf in the longitudinal direction. B is the width of a wharf unit.

2.9 Substitute Structure Method

This methodology uses an iterative process in order to estimate the displacement demand [15], see Fig 5. The displacement demand estimated in the transverse direction is modified by the dynamic magnification factor in order to obtain the total displacement demand.

The effective secant stiffness, ke, is obtained from the pushover curve for a wharf segment. Also, the method includes the estimation of an equivalent elastic damping that represents the combined effects of elastic and hysteretic damping.

The following steps are considered in each iteration:

1. Assume transverse displacement of the wharf, $\Delta_{t,n}$
2. Estimate secant stiffness from the pushover curve, $ke_{,n}$
3. Estimate effective period of the structure (Eq. 2)
4. Compute displacement ductility, $\mu_n = \Delta_{t,n}/\Delta_{ys}$
5. Estimate equivalent damping

$$\xi_{eff,n} = 0.10 + 0.565\left(\frac{\mu_n - 1}{\mu_n \pi}\right) \tag{5}$$

6. Obtain the displacement from the acceleration spectra (Eq. 3)
7. Adjust the displacement for the estimated damping using the correction factor:

$$R_\xi = \sqrt{\frac{10}{5+\xi}} \geq 0.55 \tag{6}$$

8. Compare the new displacement with the assumed one (Step 1). Iterate until converge (error less than 3%) is achieved.

2.10 Superpile Concept and 3D Analysis

Usually a typical dock segment contains several hundred piles; so modeling each of these piles as individual members would result in an impracticable matrix system. A sufficiently precise alternative is to use an equivalent simplified structure with four “superpiles” (Fig. 6). Two superpiles in the landside represent the seismic piles and two piles in the seaside represent the gravity piles. This equivalent system provides translational and rotational stiffness similar to the actual structure. This simplified representation is based on the assumption that the stiffness of the slab in the direction perpendicular to its plane is significantly large compared to the flexural stiffness of the piles that support them. Likewise, the platform is considered sufficiently rigid in its plane.

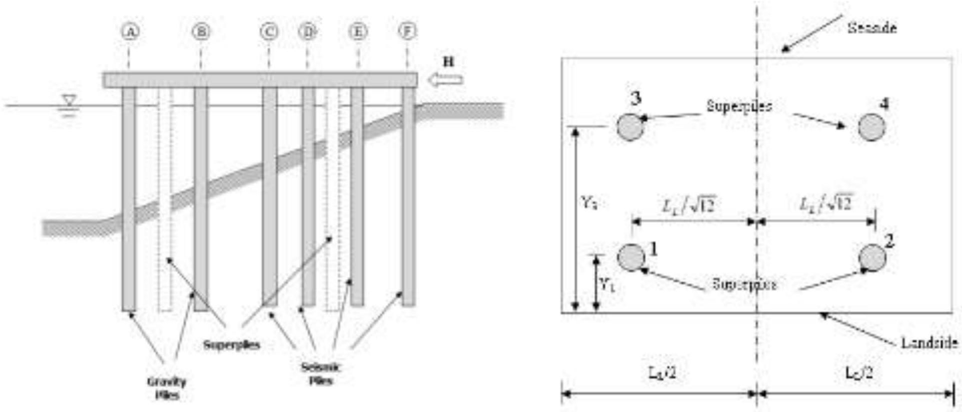

Fig. 6 – Superpile concept

In this way, the real three-dimensional structural can be represented by a two-dimensional model with inelastic individual piles for the cross-section and another two-dimensional plan model with four super-piles, each with two horizontal springs (Fig. 6). The stiffnesses of the superpiles is calculated from the pushover curve of the piles represented. The coordinates of the superpiles in the transversal direction are computed based on the following expression;

$$y_r = \frac{\sum n_i V_i y_i}{\sum n_i V_i} \tag{5}$$

Where: *ni*, is the total number of piles along the length of the wharf, *yi* is the distance of the pile to landside, and *Vi* is the lateral force in pile *i*, when the seismic piles reach yielding.

The model with superpiles is used to develop the three-dimensional modal spectral response analysis (RSA). In general, three significant modes are considered, two translational and one rotational. Full Quadratic Combination (CQC) is recommended to combine the independent modes.

The effects of simultaneous seismic actions in the horizontal orthogonal directions should be considered, by combining 100% in one direction and 30% in the other direction.

$$\begin{gathered} E = \pm 1.0E_1 \pm 0.3E_2 \\ E = \pm 0.3E_1 \pm 1.0E_2 \end{gathered} \tag{6}$$

Where E_1 and E_2, are the seismic actions in the main horizontal directions.

Also, the model with superpiles can be used to perform a nonlinear time-history analysis (NTHA). The NTHA method should be used in conjunction with another simplified method in order to verify the results. The results of the NTHA must be within the range of 20% with respect to the results obtained by other methods. The demand for displacements obtained by this method must be obtained by the simultaneous application of orthogonal horizontal excitations (Eq. 6). At a minimum, it is recommended to use three sets of records compatible with the spectrum, for this case the envelope will be considered. If seven or more records are used, the average value of the results should be used.

The hysteresis rule of superpiles should be appropriate for the pile material. To model piles of reinforced concrete, prestressed concrete or steel piles with a concrete plug, models with degradation are recommended, such as the modified Takeda hysteresis rule with parameters alpha of 0.3 and beta of 0.5. Elastic damping should be represented by tangent stiffening damping equivalent to 10% of the critical damping.

3. Case Study

3.1 Structure geometry

As an example, a single segment marginal wharf of 240 m long and 39.2 m wide is presented. The cross section of the wharf is depicted in Fig 7.

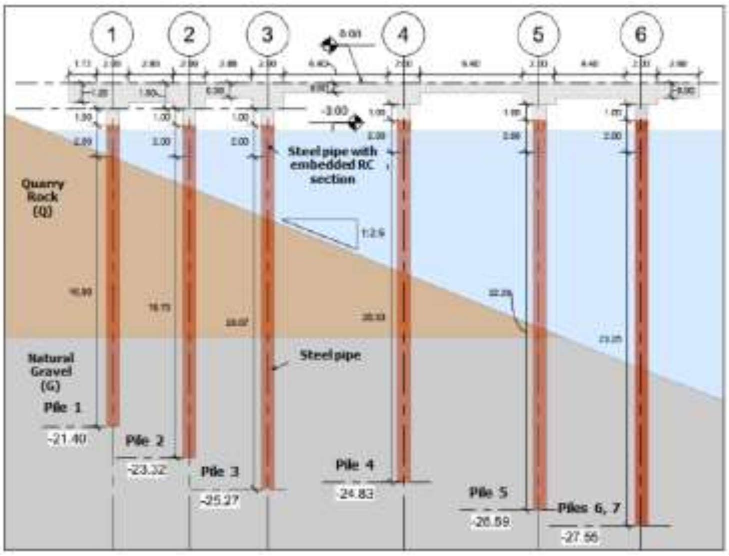

Fig. 7 – Cross section of case study

The wharf segment is supported by 280 piles, each pile with three different sections. The top reinforced concrete section at the pile-deck connection is 1m long. 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. Further details of the geometry, material properties, soil profile among others are presented by Palma, et al [16]

3.2 Seismic Hazard

The structure is located in the area of high seismicity of Peru. The supporting soil corresponds to an intermediate profile, so the factors to be applied in Eq. 1 are:

- OLE level, Z=0.207, U=1.0, S=1.05
- CLE level, Z=0.45, U=1.0, S=1.05
- DE level, Z=0.45, U=1.5, S=1.05

The acceleration spectrum is presented in Fig 1. For time-history analyses, three Peruvian sets of real ground motions each with two orthogonal components (E.030) were considered. The accelerograms were adjusted to fit the spectral response OLE, CLE and DE, by using the SeismoMatch software [17].

3.3 Nonlinear static pushover analysis

The pushover analysis is performed in the transversal direction, for one strip segment of 6 m, where loading is monotonically increased. The pushover curves for both lower bound and upper bound soil springs is shown in Fig. 8. This process is performed by the SeismoStruct software [18], which allows to identify when the strain limits are reached. The seismic mass/weight of the strip considered in the analysis is 737.39 ton. The mass estimated is due to inertial effects, no lateral spreading was included.

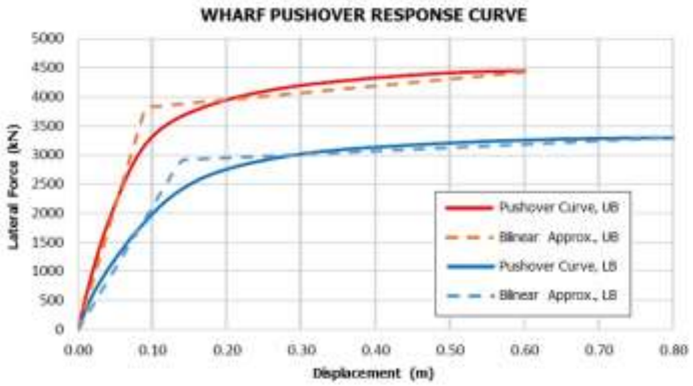

Fig. 8 –Pushover response curve for UB and LB soil springs

3.4 Demand analyses

The displacement demand is calculated by the initial stiffness method, the substitute structure method and the “superpile” model, in which an RSA and NLTH procedures are performed.

Displacement demands, for all the performance levels, estimated using the Initial Stiffness method are presented in Table 2. In Table 3, the results for the Substitute Structure method are presented. The results correspond to the last iteration as described in item 2.9. For OLE level there is no iteration needed since the demand is elastic.

Table 2 – Displacement Demand, Initial Stiffness Method

Parameter	**Upper Bound Soil Springs**			**Lower Bound Soil Springs**		
	OLE	**CLE**	**DE**	**OLE**	**CLE**	**DE**
Δt (m)	0.066	0.148	0.222	0.094	0.209	0.314
DMF	1.49	1.34	1.34	1.49	1.19	1.19
Δd (m)	0.099	0.199	0.297	0.140	0.250	0.373

Table 3 – Displacement Demand, Substitute Structure Method

Parameter	**Upper Bound Soil Springs**			**Lower Bound Soil Springs**		
	OLE	**CLE**	**DE**	**OLE**	**CLE**	**DE**
Assumed Displacement (m)		0.120	0.204		0.169	0.281
Effective Stiffness (kN/m)	42128	32759	19727	21155	17406	10674
Effective Period, T_n (sec)	0.831	0.943	1.215	1.173	1.293	1.651
Displacement ductility, μ_n		1.295	2.194		1.223	2.039
Effective damping, ξ		0.141	0.198		0.133	0.192
Spectral displacement (m)	0.0675	0.166	0.321	0.095	0.228	0.437
Correction factor, R_ξ		0.724	0.635		0.740	0.643
Computed Displacement (m), Δt	0.0675	0.120	0.204	0.095	0.169	0.281
Total Displacement (m), Δd	0.101	0.161	0.273	0.142	0.201	0.334

The results for the "superpile" method are presented in Table 4. The pushover curves for seismic and gravity piles are presented in Figure 8. A Takeda hysteresis rule with parameters alpha of 0.3 and beta of 0.5 were considered for the Nonlinear Time History Analysis.

Table 4 – Displacement Demand, Superpile Approach

Parameter	Upper Bound Soil Springs			Lower Bound Soil Springs		
	OLE	CLE	DE	OLE	CLE	DE
RSA method, Δd (m)	0.079	0.172	0.258	0.111	0.241	0.361
NLTH method, Δd (m)	0.071	0.182	0.264	0.070	0.171	0.320

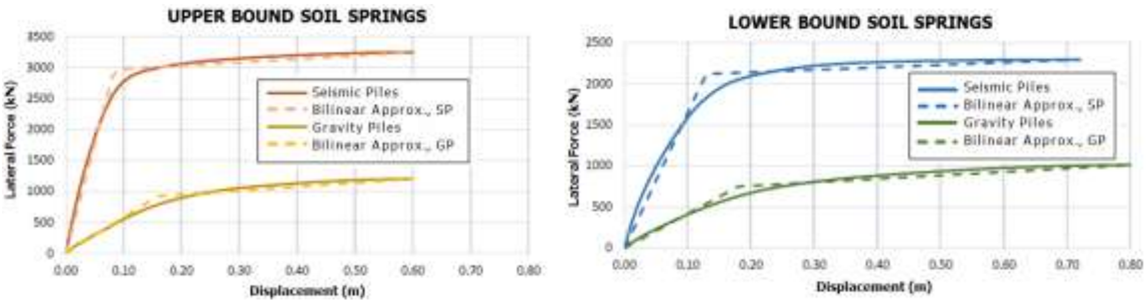

Fig. 9 –Pushover curves for "superpile" approach

3.5 Performance verification

Results for all the models along with the wharf capacity curves, indicating the strain limits for all the performance levels, are presented in Fig 10.

For all the performance levels, the displacement demands are smaller than the capacity, so the example structure would comply with the requirements given in the proposed guidelines.

For the OLE performance levels, the substructure method provides the higher displacement demands. For the CLE and DE performance levels the initial stiffness method provides the higher displacement demands. In general, the OLE level is the one controlling the design since the displacement demands are closer to the capacity.

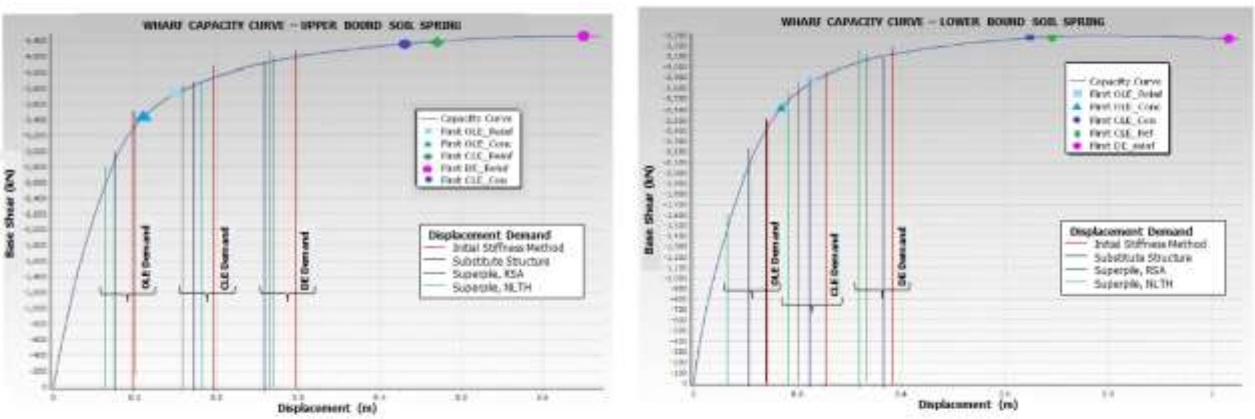

Fig. 10 –Pushover capacity with strain limits and displacement demands

4. Conclusions and Recommendations

A proposal of seismic guidelines to use for the design, evaluation and retrofitting of pile supported wharves structures in Peru is presented in the current paper. The proposed guidelines follow criteria for performance-based seismic design and consider recommendations presented on state of the art seismic codes.

The proposed guidelines should be performed as verification of the structure design. The structure design should be performed considering all the loads that could be acting in the structure like dead, live, mooring berthing, seismic and so on. Load factors and combinations should follow Peruvian and standard international codes.

The simplified models considered in the proposed guidelines should be used for wharf structures with regular geometries. For structures with complicated geometries the analyses, nonlinear pushover as well as nonlinear time-history, considering 3D models of the full structure should be performed.

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