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SPECTRUM MATCHING SCALING PROCEDURES APPLIED TO NONLINEAR ANALYSIS OF WHARVES

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Abstract

Wharf structures are essential structures in the economy of a country. Even though their importance, they have received less attention compared with ordinary buildings. These structures are characterized by significant torsional response under bi-directional earthquakes excitations. This behavior arises for the changes in lengths of piles from the land side or waterside. On the other side, they support a complex state of static and dynamic load due to water Streams, impact with containers and craine effects, etc. The ASCE/COPRI 61 – 14 [1] demand that analysis and design of wharves structures have to been done with nonlinear – time history analysis and this motivates this research to present a procedure for selecting and scaling of ground motions for nonlinear dynamic analysis.

A recently developed scaling procedure called the modal pushover-based scaling (MPS) [2] has been successfully implemented in the seismic analysis of single and multi-story buildings with symmetric or unsymmetrical structural plan distribution. An extension of the well-known Spectrum Matching methodology is presented here to analyze wharf structures. Sophisticated non-linear dynamic analysis is conducted on four different types of wharves subjected to one suite of 7 ground motions each. The suite corresponds to near-field records that represent the seismicity of Los Angeles area. Pairs of wharves with length varying from 300 to 600m are analyzed; for each length, two embankment systems are defined as 2:1 and 3.5:1. The performance of these structures is compared with benchmark values defined as the mean values of Engineering Demand Parameters (EDPs) due to a more extensive suite of unscaled records. Additionally, this study is compared with the ASCE/SEI 7-10 [3] procedure for selecting and scaling records. Results indicate that SM scaling procedure can result in an improvement of results obtained with ASCE/SEI7-10, with lower underestimation of the platform drifts demand by as much as 10% and underestimation of the material strain demands by as much as 30%. Also, ASCE/SEI 7-10 methodology underestimates the displacements of wharves structures in values ranging from 30-40% and material strain demands of up to 60%, respectively. Moreover, it is found that the MPS procedure provides better estimates of the expected EDPs. Because the performance acceptance in the new standard ASCE/COPRI 61-14 [1] (seismic design of piers and wharves) is solely based on strain limits, the Spectrum Matching (SM) scaling procedure could lead to unconservative results.

Keywords: Modal pushover-based scaling; wharves; performance-based design; Spectrum Matching; The ASCE/SEI 7-10 scaling procedure.

1. Introduction

The ASCE/COPRI 61 – 14 [1] is the current design standard for new and existing marginal wharves. For design wharves of high levels of importance, the process of design requires a performance-based procedure to evaluate the strain of confined concrete and steel for several levels of seismic hazards specified in the document. Hence it requires a response history analysis (RHA) where each signal should be scale following the ASCE/SEI7-10 [3] process to meet the target spectra to met the seismic hazard from the site. From the analysis, the ASCE/SEI7-10 process determine the engineering demand parameters (EDPs) to calculate the strain of the material and compare them with the strain limits specified in the standard asce61-14. Since the scaling process describes for ASCE/SEI7-10 was develop to buildings, the concert in this work is to evaluate the efficiency and efficacy of the scaling process to apply in wharves structures. Since the wharves have rectangular plain shape and different height in elevation, the modal behavior of the structure is highly torsional; the main concern is to validate the ASCE/SEI7-10 procedure to use in wharves structures.

In the past, several new scaling procedures have been developed. In this study, we focus on two particular processes, the first developed for Kalkan and Chopra [2] named modal pushover -based scaling (MPS) procedure, scale seismic signals applying forces with the shape of the first structure vibration mode to obtain the pushover curve. With this curve, the procedure model an inelastic SDF system and scale the signal to reach a target displacement obtained. This procedure was successfully extended to analyses of symmetric structures with different heights and has proved to be accurate and efficient for one component of ground motion. Next, Reyes and Chopra [4, 5, 6] research the incidence on the EDPs for two components of the seismic signal and extend the MPS procedure remaining accuracy and efficiency of the procedure. Reyes and Quintero and later, Reyes et al. [7, 8] research the behavior of single-, multi-story asymmetric plan structures and found that the MPS scaling process is accurate to estimate the structural response.

The second arises like a solution to the distortion in velocity and displacement that result to modifying the ground motion in the frequency domain [9], the spectrum matching procedure adds wavelets to the ground motion in the time domain to get compatible target spectrum ground motion. The initial research was published by Kaul [10] Lilhanand and Tseng [11, 12], but the procedure changed the acceleration, velocity, and displacement forms of ground motion. Next, Abrahamson developed the RSPmatch computer code, and later Hancock and Bommer [13] extended the code adjusting to match the ground motion at target spectre in different damping levels, moreover preventing drift in acceleration, velocity, and displacement waveform. This code was named RSPMatch2005. Al-Atik and Abrahamson [14] developed an improved tapered cosine wavelets, and the new method does require baseline correction and ensure zero final velocity and displacement.

Recently, Pantoja et al. [15] conclude that ASCE/SEI 7-10 [3] scaling process can result in underestimation of drift platform demand by as much 40% and materials strain demands as much 60%; in contrast, they found that MPS provide conservative estimates of those values although more dispersion. In this paper, we extend the previous results by adding the SM results; the objective is to compare the EDPs obtained to analyze two geometric configurations of marginal wharves supports by two types of soil; these characteristics were chosen to be representative of practical application. The structures were analyzed with RHA after to scale, and to select seven ground motion with MPS and SM procedure, the results were compared against the ASCE/SEI 7-10 [3] (ASCE7 hence-forth) ground motion scaling procedure. The efficiency and accuracy of each process were evaluated by comparing the EDPs against a broad set of 30 ground motion.

2. Modal-pushover scaling (MPS) procedure

The MPS procedure requires three phases: (1) computation of target roof displacement and pushover analyses, (2) scaling phase, and (3) selection phase. A step-by-step list of the general procedure is described in Pantoja et al. [15] and is reprint here for convenience.

2.1 Target roof displacement and pushover analyses

(1) For a given site, define the target spectra $\hat{A}_x$ and $\hat{A}_y$, in this study taken as the median of the 5-percent damped pseudo-acceleration response spectra of two components of the ground motions.

(2) Compute the natural frequencies ω_n (periods T_n) and modes ϕ_n of the first few modes of linear-elastic vibration of the structure. For each ground motion component direction (x or y), identify the first, second, and third modes as the three modes with the larger effective modal mass. In the case of a one-story structure –as a marginal wharf- only one or two modes may have the largest effective modal mass; in such cases, only these few modes should be used.

(3) Develop the base shear-roof displacement, $V_{bn} - u_{rn}$, relationship or pushover curve by nonlinear static analysis of the structure subjected to the n^{th}-“mode” invariant force distribution given by Eq. (1):

$$s_n^* = \begin{bmatrix} \mathbf{m}\phi_{xn} \\ \mathbf{m}\phi_{yn} \\ \mathbf{I}_o\phi_{\theta n} \end{bmatrix} \qquad (1)$$

Where $\mathbf{m}$ is a diagonal matrix of order N with $m_{jj} = m_j$, the mass lumped at the j^{th} floor level; $\mathbf{I}_O$ is a diagonal matrix of order N with $I_{ojj} = I_{oj}$, the moment of inertia of the jth floor diaphragm about a vertical axis through the center of mass (C.M.); and subvectors ϕ_{xn}, ϕ_{yn}, and $\phi_{\theta n}$ of the n^{th} mode ϕ_n represent x, y, and components of ground motion, respectively. This step should be implemented only for the first three “modes” in the direction under consideration; this step could be omitted for the higher-“modes” if they are treated as linear-elastic [16].

(4) Idealize the $V_{bn} - u_{rn}$ pushover curve as a bilinear or trilinear curve, as appropriate, and convert it into the force-deformation, $(F_{sn}/L_n) - D_n$, relationship for the n^{th}-“mode” inelastic SDF system using the well-known formulations [12, chapter 20] shown in Eq. (2):

$$\frac{F_{sn}}{L_n} = \frac{V_{bn}}{M_n^*} \; ; D_n = \frac{u_{rn}}{\Gamma_n \phi_{rn}}; \Gamma = \frac{L_n}{M_n} = \frac{\phi_n^T \mathbf{M}\iota}{\phi_n^T \mathbf{M}\phi_n}; \quad \mathbf{M} = \begin{bmatrix} \mathbf{m} & 0 & 0 \\ 0 & \mathbf{m} & 0 \\ 0 & 0 & \mathbf{I}_o \end{bmatrix}; \; \iota_x = \begin{bmatrix} 1 \\ 0 \\ 0 \end{bmatrix}; \text{and } \iota_y = \begin{bmatrix} 0 \\ 1 \\ 0 \end{bmatrix}; \qquad (2)$$

Where F_{sn} is a nonlinear hysteretic function of the nth modal coordinate [16]; M_n^* is the effective modal mass for the n^{th}-“mode”; **1** and **0** are vectors of dimension N with all elements respectively equal to one and zero, and ϕ_{rn} is the value of ϕ_n at the roof.

(5) Establish the target roof displacement $\hat{u}_r$. For a system with known T_n, damping ratio ξ_n, and force-deformation curve (Step 3), determine the peak deformation D_n for the n^{th}-“mode” inelastic SDF system due to each of the unscaled ground motions $\ddot{u}_g(t)$ by solving: $\ddot{D}_n(t) + 2\xi_n\omega_n\dot{D}_n(t) + \frac{F_{sn}}{L_n} = -\ddot{u}_g(t) \rightarrow D_n$

Determine $\widehat{D}_n$ as the median of the D_n values. Calculate roof displacement in the direction under consideration of the n^{th}-“mode” as $\hat{u}_{rn} = \Gamma_n\phi_{rn}\widehat{D}_n$, and compute the roof displacement in the direction under consideration $\hat{u}_r$ from values of $\hat{u}_{rn}$ using a suitable modal combination method (e.g., complete quadratic combination). In practical applications, the target deformation $\widehat{D}_n$ can be computed as $\widehat{D}_n = C_{Rn}\widehat{D}_{no}$, where C_{Rn} is the inelastic deformation ratio, estimated from empirical equations (13), and $\widehat{D}_{no} = (T_n/2\pi)^2\hat{A}_n$ with $\hat{A}_n$ is the target pseudo-spectral acceleration at period T_n.

2.2 Scaling phase

(6) Compute the scale factor *SF* for each record in the direction under consideration by solving the following nonlinear equation: $u_r - \hat{u}_r = 0$, where u_r is the peak roof displacement in the direction under consideration from the scaled records. Because this equation is nonlinear, *SF* cannot be determined *a priori*, but requires the following iterative procedure:
   a) Select an initial value of the scale factor *SF*, and compute deformation $D_n(t)$ for the n^{th}-"mode" inelastic SDF due to the scaled record by solving: $\ddot{D}_n(t) + 2\xi_n\omega_n\dot{D}_n(t) + F_{sn}/L_n = -SF \times \ddot{u}_g(t)$ $\rightarrow D_n(t)$
   b) Compute roof displacement of the n^{th}-"mode" in the direction under consideration: $u_{rn}(t) = \Gamma_n \phi_{rn} D_n(t)$
   c) Compute roof displacement in the direction under consideration: $u_r = max(|\sum_n u_{rn}(t)|)$
   d) Estimate error: $\varepsilon = u_r - \hat{u}_r$
   e) Adjust the value of the scale factor *SF*, and repeat steps a) to d) until ε is less than a tolerance value.

   In this study, step 6 was implemented by a numerical algorithm. By executing steps a) to e), separately for the x and y components of the record, scale factors SF_x and SF_y are determined. Note that pushover curves (step 4), and target roof displacement (step 5) will be different for the two horizontal components of the ground motion.

2.3 The selection ground motion phase applies to wharf structures.

(7) Select the first *k* records with the lower values of
   $Error = \frac{\hat{A} - SF_x A_x + \hat{A} - SF_y A_y}{max(\hat{A}_x - SF_x A_x + \hat{A}_x - SF_y A_y)} + \frac{\hat{A}_{T_n} - SF_x A_{x,T_n} + \hat{A}_{T_n} - SF_y A_{y,T_n}}{max(\hat{A}_{T_n} - SF_x A_{x,T_n} + \hat{A}_{T_n} - SF_y A_{y,T_n})}$, where $\widehat{A}$, A_x, and A_y are vectors of spectral values $\hat{A}_i$ at different periods T_i between $0.2T_1$ and $1.5T_1$; $\hat{A}_{T_n}$, A_{x,T_n} and A_{y,T_n} are vectors of spectral values for the first three periods of vibration $T_{n,i}$.

3. Spectrum Matching Scaling process

Spectrum matching process was extended by J.C. Reyes et al. [7] to account for three-dimensional structures and two horizontal components simultaneously. The step by step procedure is described next for convenience:

(1) For a given site, select ground motions compatible with site-specific seismic hazard conditions governing the seismic design.
(2) Compute the response spectrum A(T) for each ground motion for various damping values (e.g., 2%, 5%, and 10%) at evenly spaced periods T_i in a logarithmic scale over the period range from $0.2T_1$ to $1.5T_1$ (in this study, i =1, 2, 3,..., 100).
(3) Determine the target pseudo-acceleration response spectrum $\hat{A}(T)$ as the median spectrum determined in step 2 for various damping ratios. Define $\hat{A}$ as a vector of spectral ordinates $\hat{A}_i$ at 5% damping level at the same periods T_i.
(4) Estimate the scaling factor SF to minimize the difference between the response spectrum (step 2) and the target spectrum (step 3) for 5% damping by solving the following minimization problem for each ground motion:

$$min\|\ln(\hat{A}) - \ln(SF \times \hat{A})\| \rightarrow SF \qquad \|.\| = Euclidean\ norm$$

   Required for this purpose is a numerical method to minimize the scalar functions of one variable. Such methods are available in textbooks on numerical optimization [17]. This minimization ensures that the scaled response spectrum is as close as possible to the target spectrum. At the end of steps 1–4, implemented separately for the two horizontal components of each ground motion record, scaling factors SF_x and SF_y are determined for the x and y components of the ground motion, respectively.

(5) Compute the difference between the scaled spectrum $SF \times \mathrm{A(T)}$ and the target spectrum for 5% damping (step 2) for each ground motion. Define the error E_{SM}, and rank the scaled records based on their E_{SM} value; the record with the lowest E_{SM} is ranked the highest.

$$E_{SM} = \left\|\ln(\hat{A}_x) - \ln(SF_x \times A)\right\| + \left\|\ln(\hat{A}_y) - \ln(SF_y \times A)\right\|$$

(6) From the ranked list, select the first k records with their scale factors determined in step 4. In this study, we used k = 7 because previous research shows that a minimum of seven records is sufficient for unbiased estimates of EDPs from nonlinear RHAs [18, 19].

(7) Modify each scaled ground motion, independently, by adding wavelets in the time domain to match the target spectrum for various damping values: 2%, 5%, and 10%. In the present research, this step is implemented using the non-commercial computer program RspMatch2005 [13]. These modified ground motions are used to conduct nonlinear RHA of the structure. Note that this step should be implemented for each horizontal component of ground motion, separately. The median spectra computed in step 3 for the two horizontal components of records shall be used as target spectra for two orthogonal directions in 3D analyses.

4. Structural behavior of pile-supported Wharves

In this study, we focus on wharf structures; this denomination describes structures where their longer plan dimension is parallel to the shoreline. Their structural configuration is prestressed slabs supported in reinforced concrete girders to conform a stiff superstructure that must accomplish stringer durability requirements adds to high levels of dead by their storage use daily and live load due container vessels approach and craine work. Prestressed long piles often support this superstructure; this is to reach competent load-bearing soil layers. Their requirements to manage large vessels lead to their plain dimension has long-width ratios ranging between 6 and 18, on the other hand, their low soil load-bearing, and draft requirements lead to longer-slender waterside piles and shorter-stiffer landside piles. The landside piles are much stiffer than waterside piles due to shorter unsupported lengths; demands at the top of the piles are greater because of the stiff deck superstructure, and connections are usually weaker than the piles themselves. This configuration results in a structure with a strong beam-weak column with a torsional modal response where the plasticity after seismic effects is concentrated in the landside pile to cap interface. Pantoja et al. [15].

5. Geometric description of structures

Two cross-sectional structures were studied to be representative of common container terminals, and two long plain dimensions of 315 m and 630 m respectively, were considered to harboring one or two Panamax vessels. Fig. 1 shows soft clays soil characteristics dominated for mudline slope 1:3.5 (vertical to horizontal), and on the other hand, Fig. 2 shows medium clays soil characteristics dominated for mudlines slope 1:2. These cross-sections are uniformly spaced at 7.5 m on axis centers by 43 transverse bents for 315 m and 85 transverse bents for 630 m, respectively, moreover each cross-section has two longitudinal bents at 30.48 m on-center supported by piles space at 2.5 m to support large crane wheel loads. The framing system consisted of piles and transverse pile caps that support 350mm- thick precast/prestressed concrete panels spanning in the longitudinal direction of the wharf. The panels and the pile caps have sufficient transverse reinforcement projecting into a 200mm-thick cast-in-place (CIP) concrete topping to ensure composite behavior. Piles were pre-tensioned through 24-12.7mm (1/2 in.)-diameter grade 270 strands, while pile-to-cap connection consisted of 12-25mm (#8) longitudinal bars. Table 1 lists the geometric configuration of 4 wharf structures analyzed. The soil layer was considered uniform through depth; Table 2 describes the mechanic properties of soil considered Pantoja JC et al. [15].

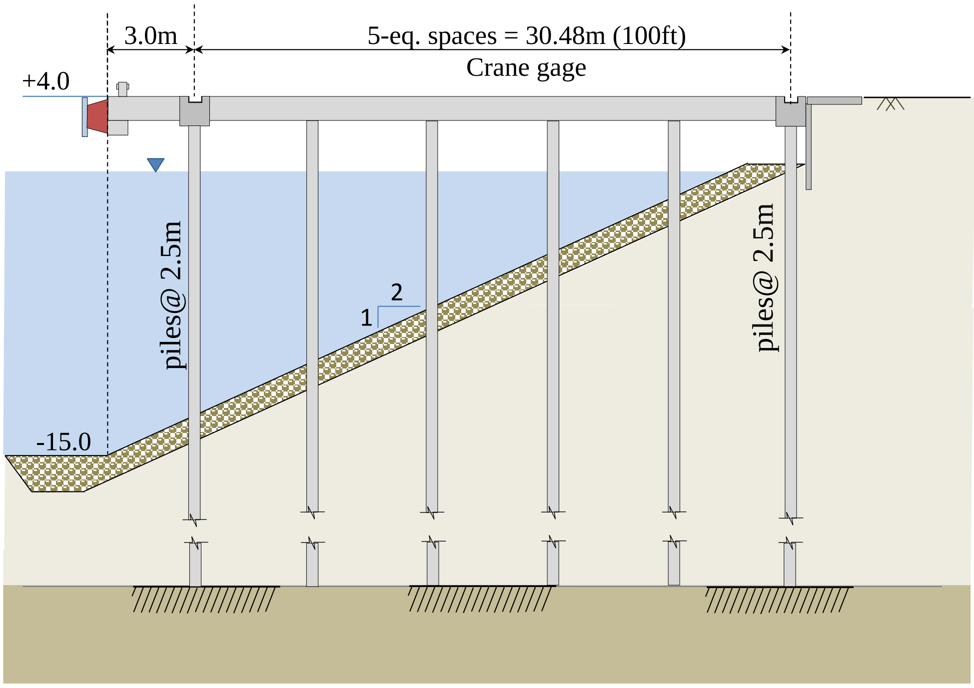

Fig. 1 – Cross-sectional view of case study structures on medium dense soil: Wharf 1 (315m long) and Wharf 3 (630m long), Pantoja JC et al. [15].

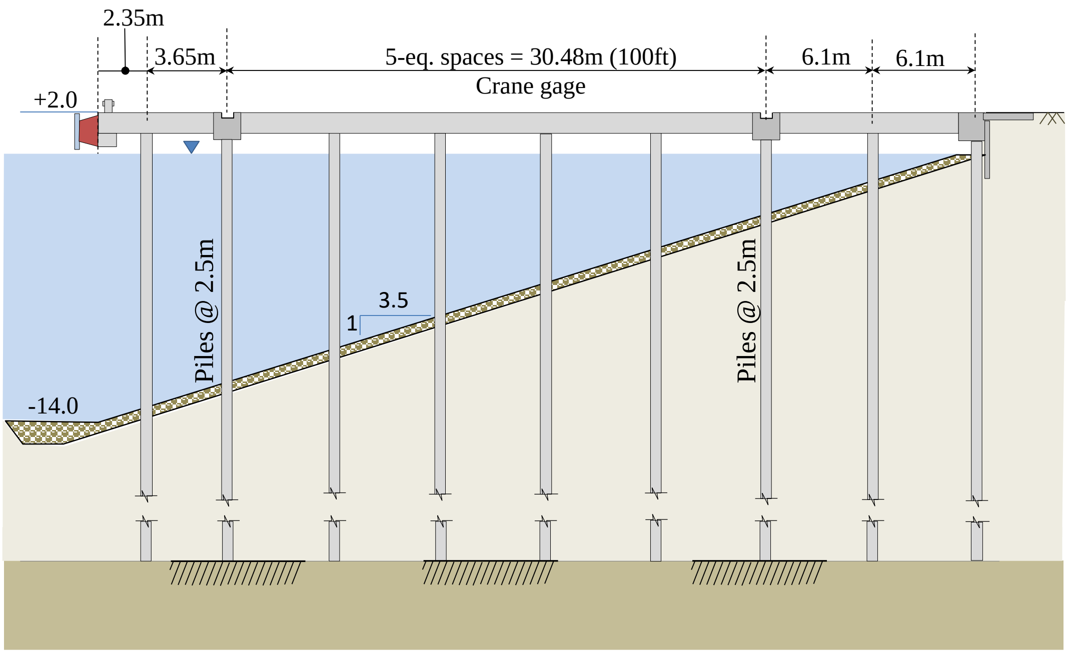

Fig. 2 – Cross-sectional view of case study structures on soft soil: Wharf 2 (315m long) and Wharf 4 (630m long), Pantoja JC et al. [15].

Table 1 – Geometric characteristics of wharfs structures.

Structure name	Length plain dimension [m]	Mudline slope
Wharf 1	315	1:2
Wharf 2	315	1:3.5
Wharf 3	630	1:2
Wharf 4	630	1:3.5

Table 2 – Soil properties

Property	Medium dense soil (1:2 slope)	Soft dense soil (1:3.5 slope)
Unit weight, kN/m^3	10.2	10.2
Cohesion, kN/m^2	30	45
Friction angle, degree	18	22
Soil type	Clay	Soft clay

6. Structural design and computational models

Each wharf structure was design by displacement-based provision describes in ASCE/COPRI 61-14 [1] considering seismic actions due target acceleration spectrum. The reinforced and prestressed concrete elements were design based on ACI318-19 [20]. Wharf structures were model using the program PERFORM3D [21] using an equivalent fixed base for piles to conduct RHA. Pile and pile caps were represented by linear elastic elements with plastic hinges at their ends; each pile was fixed at a certain depth below mudline. The depths to pile fixity were selected to produce similar force-displacement responses as compared to refined analysis that considers fibers to model the strain – stress curves of materials or the variation of nonlinear soil behavior trough depth. Pantoja JC et al. [15] conclude that the use of an equivalent fixed base for piles reduces the computational demand significantly, and it is also conservative for the analyzed structures.

Figs. 3 and 4 show the calculated effective modal mass along with a schematic representation of the first three mode shapes and corresponding periods of the four wharf structures modeled using an equivalent fixed base for piles. It is observed that there is a strong coupling between the longitudinal displacement and the plan rotation in the first and third mode, while transverse displacements dominate the second mode of vibration. These are consistent with the fact that the structure's center of stiffness is expected to be near the middle of the wharf in the longitudinal direction due to symmetry, while in the transverse direction, the center of stiffness is closer to the land side piles Pantoja JC et al. [15].

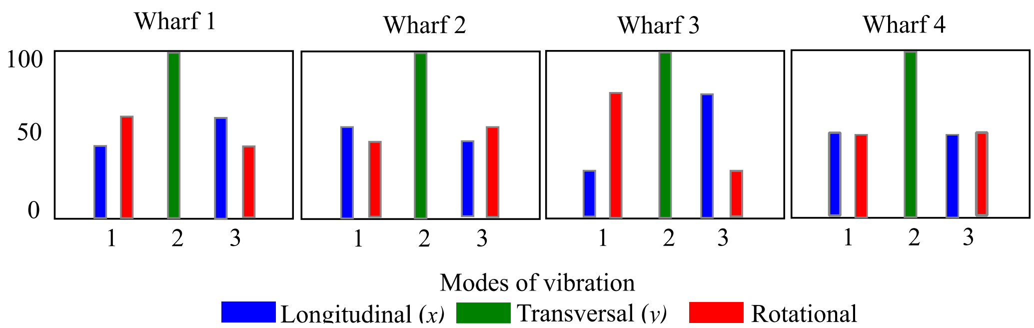

Fig. 3 – Effective modal mass for the case study structures, Pantoja JC et al. [15].

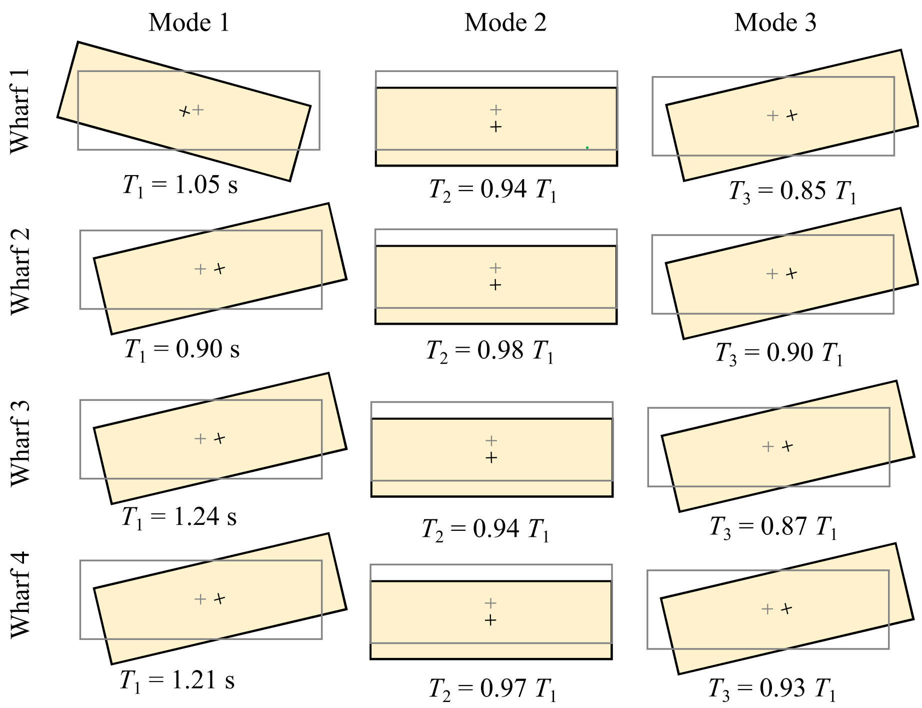

Fig. 4 – Schematics of the calculated first three mode shapes and periods of case study structures, Pantoja JC et al. [15].

7. Assessment of ASCE 7, SM and MPS procedures

The ASCE/COPRI 61 – 14 [1] specify stringer requirements to analyze wharf structures under seismic ground motion. For structures with high levels of importance, response history analysis is required to account for the nonlinear response of individual materials and behavior of structure as global. In past years, investigation shows that the scaling procedure specifies in ASCE/COPRI 61 – 14 [1] standard produce underestimation of benchmark displacement and strain material leading to an unconservative estimation of structural safety. For this analysis, we select the list of ground motion records used in Pantoja et al. [15]. For comparison purposes. They corresponded to near-field earthquakes with moment magnitudes between 6.5 and 7.5, and fault distances ranging from 3.6 to 12.8 km. The ground motions were prescaled for a 1.5 factor to guarantee the nonlinear structural response. This modified signals we call "unscaled records" henceforth. Seven sets of scaled ground motion were obtained by applying SM, MPS, and ASCE/SEI7-10 [3] procedures and used later in RHA. Structural EDPs of four wharf structures were calculated with each set and compared with the results of ASCE/SEI7-10 [3] procedure. The first goal is to establish accuracy defined by comparing the median of the calculated EDP for the four wharves structures subjected to each group of seven scaled ground motion records with the median value of the EDPs but for the structures subjected to all 30 records (benchmark values henceforth), on the other hand, second goal is to establish efficiency defined by the amount of dispersion in the calculated EDP. Table 3 shows the benchmark values obtained at the C.M of each structure of the values of relative displacement, relative velocity, and absolute acceleration; also, Table 4 shows the maximum benchmark strain of concrete and reinforcement on pile-to-cap connections. The locations of the reference points, schematically identified in the same table, corresponding to the piles that are closest to the landside of the wharf and near the ends (P1 and P3) and middle (P2) in the longitudinal direction.

Table 3 – Benchmark values at the C.M of the case structures, Pantoja JC et al. [15].

Wharf	Displacement mm		Velocity m/s		Acceleration m/s^2	
	x	y	x	y	x	y
1	160	140	0.69	0.64	26.0	24.9
2	180	160	0.78	0.69	22.8	21.4
3	140	140	0.64	0.60	24.8	23.9
4	180	160	0.79	0.69	22.8	21.4

Table 4 – Calculated benchmark material strains (ε_c and ε_s for concrete and steel) at critical pile-to-cap connections [units = 1000μ], Pantoja JC et al [15].

Wharf	Connection					
	P_1		P_2		P_3	
	ε_c	ε_s	ε_c	ε_s	ε_c	ε_s
1	5.8	11.6	5.7	11.3	5.5	11.0
2	3.0	6.2	3.0	6.2	3.0	6.2
3	6.1	12.3	6.2	12.4	6.0	12.3
4	3.0	6.4	6.0	6.3	3.0	6.3

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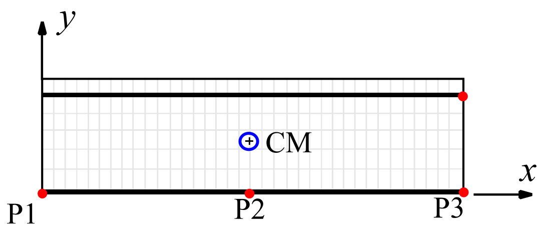

shows the calculated relative displacement, relative velocity, and absolute acceleration at the center of mass for each structure normalized by the corresponding benchmark value. Each geometric median of EDP is identified with a singular marker; for each marker, vertical lines are plotted to denote the magnitude of the median normalize values of the EDPs plus or minus one standard deviation. Pantoja et al. [15] observed that implementation of the ground motion scaling procedure prescribed in ASCE/SEI 7-10 [3] produces underestimation of the benchmark displacements for the case study wharf structures by as much as 40%; thus, indicating that using the standard for this purpose may be unconservative. The MPS procedure, on the other hand, provides more accurate and conservative estimates of the benchmark values. Compared with these results, the SM methodology produces underestimations ranging the 20% to 30 % to displacements. It means an improvement compared with ASCE/SEI 7-10 [3] procedure, but the efficiency is lower than MPS and ASCE/SEI 7-10 [3]. It is observed that MPS procedure produce an estimation by as much as plus or minus 10%, which is acceptable for design purpose. On the other hand, the acceleration values for the three methodologies show a similar efficiency, although MPS values overestimate by as much as 15%; meanwhile, SM reduces the underestimation of ASCE/SEI 7-10 [3] procedure to 10%.

Also, 6 shows the strain of concrete and reinforcement on pile-to-cap connections piles P1, P2, and P3 normalized by the corresponding benchmark value. Pantoja et al. [15] observed that material strain demands estimated using the ASCE/SEI 7-10 [3] ground motion scaling procedures are always smaller, by as much as 60%, than the corresponding benchmark values. By contrast, the MPS procedure provides reasonable and conservative estimates of benchmark values, although with more dispersion. Compared with these results, the

SM methodology shows an improvement compared with ASCE/SEI 7-10 [3], with underestimation by as much as 30%. Also, it is observed that dispersion obtained with SM is similar to ASCE/SEI 7-10 [3] procedure.

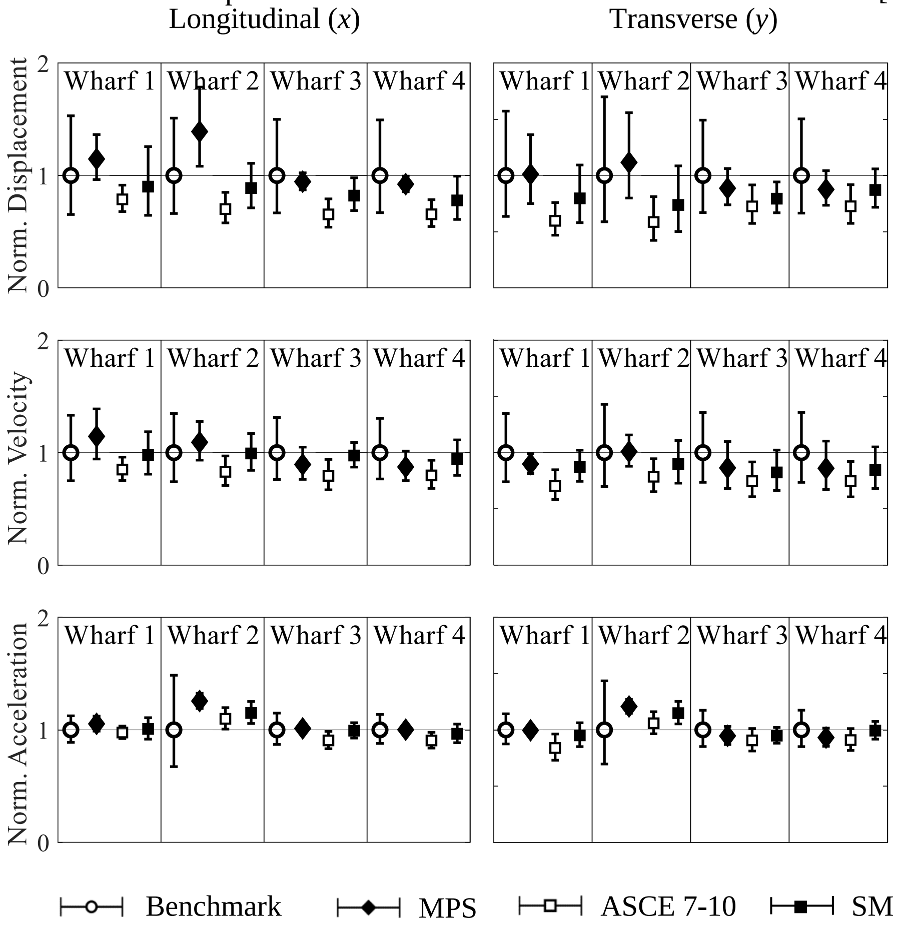

Fig. 5 – Normalized displacement, velocity, and acceleration in the *x* and *y* direction at the C.M. of the case study wharves

Finally, the results obtained shows that SM reduce the underestimation on relative displacement, absolute acceleration and strains of materials compared to ASCE/SEI 7-10 [3] methodology, however, MPS procedure shows a safer estimation of EDP for design purpose, with acceptable overestimation ranging between 10% to 20%. Since the ASCE61-14 [1] standard use performance criteria for seismic design of piers and wharves, the use of ASCE/SEI 7-10 [3] and Spectrum Matching procedures proves to be unconservative for all the case study structures included in this investigation. As conclude by Pantoja et al. [15], more research on the subject is required to ground motion selection and scaling procedure specifically apply to pile-supported wharves.

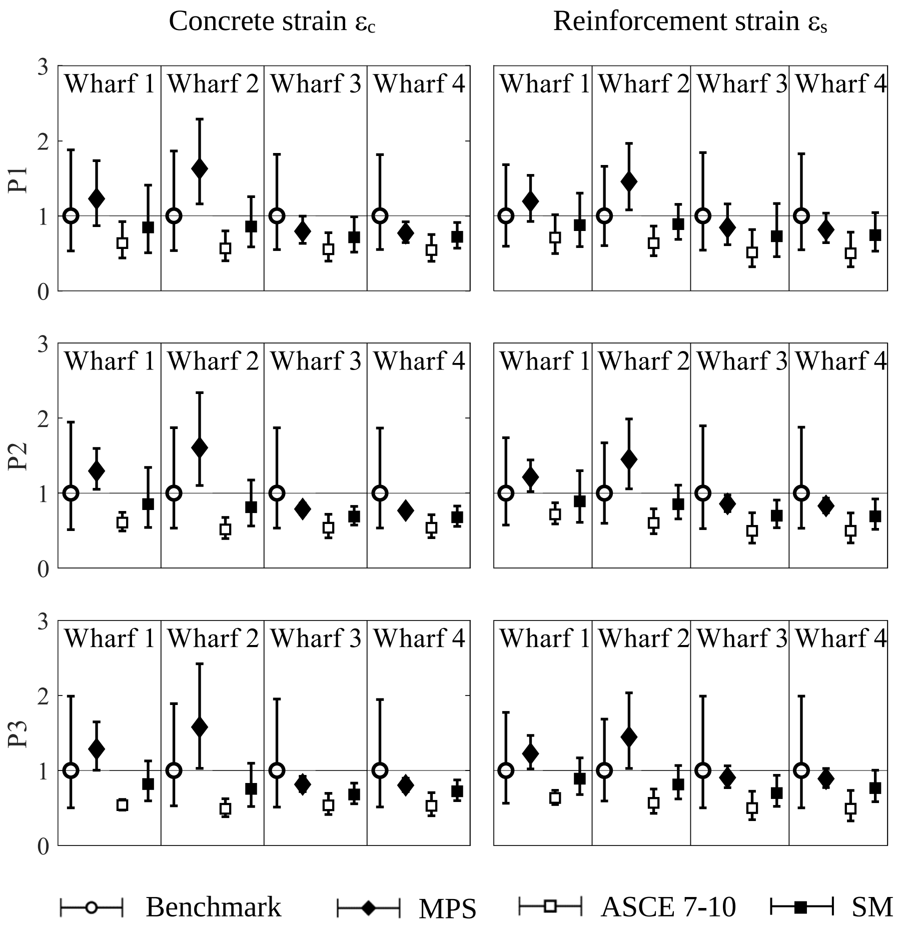

Fig. 6 – Maximum normalized concrete and reinforcement strain demands at various pile-to-cap connections

8. Summary and conclusions

Based on a seismic scenario, 30 unscaled ground motions were selected. Three sets of seven ground motion were scaled by ASCE/SEI 7-10 [3], MPS, and SM selection and scaling procedures and were implemented in nonlinear response history analyses (RHA) of four 3D marginal wharf models. The relative displacement, relative velocity, absolute acceleration at the center of mass, and the strain of concrete and reinforcement on pile-to-cap connections landside piles were calculated; This Calculated Engineering Demand Parameters (EPD) were compared against the results of benchmark (median) values defined calculated applying an RHA of larger set of 30 unscaled ground motions. It was found that implementing the SM scaling procedure can result in an improvement of results obtained with ASCE/SEI 7-10 [3], with lower underestimation of the platform drifts demand by as much as 10% and underestimation of the material strain demands by as much as 30%. However, MPS scaling procedure shows a safer estimation of EDP for design purpose and provide reasonably conservative estimates of benchmark values, although with more dispersion in the case of strain demands. Because the material strain is used as the performance indicator in the ASCE61-14 [1] for the seismic design of pile-supported piers and wharves, it is concluded that modal pushover-based scaling (MPS) scaling procedure can provide unconservative results by as much as ASCE/SEI 7-10 [3].

9. References

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