ACCOUNTING FOR NON-LINEAR DYNAMIC SOIL-STRUCTURE INTERACTION IN THE DISPLACEMENT-BASED SEISMIC DESIGN

R. Paolucci¹, R. Figini², C. di Prisco³, L. Petrini⁴, M. Vecchiotti⁵

ABSTRACT

An engineering procedure is presented in this paper, aiming at taking into account non-linear dynamic soil-structure interaction effects in the displacement based seismic design framework. The procedure applies for structures lying on shallow foundations. Non-linearities occurring at the soil-foundation level during an earthquake are modeled by an equivalent linear visco-elastic approach, through the concepts of secant stiffness and equivalent damping. A series of curves, which describe the stiffness degradation and increase in damping as a function of the foundation rotation, are obtained both experimentally and numerically by employing a constitutive model based on the macro-element concept. An iterative pseudo-static procedure is then developed to introduce these curves into the standard formulation of displacement based design, originally conceived for fixed base structures, and applied to the design of a bridge-pier.

Keywords: non-linear dynamic soil-structure interaction, shallow foundations, displacement based seismic design

INTRODUCTION

The increasing interest of earthquake engineering towards performance-based design approaches has recently led to a greater consideration of the role played by soil-foundation-superstructure interaction on the overall behaviour of the system (ATC-40, 1996; Martin and Lam, 2000; Pecker, 2006). Traditional seismic design tends to neglect non-linear effects taking place in the soil under the foundation, so that the dissipation of energy is assumed to be due exclusively to the ductile behaviour of the super-structure. However, non-linear soil response is almost unavoidable, since overturning moments can get temporarily greater than the foundation static bearing capacity even for moderate seismic loads (Pecker, 2006; Gazetas et al., 2007; Pender, 2007). In this case the non-linear soil-foundation interaction can be relevant, and can lead to a reduction in the seismic demand on the super-structure.

Numerical methods used to solve dynamic soil-structure interaction problems traditionally assume that structure and foundation can be uncoupled (“sub-structuring approach”), so that dynamic and kinematic effects related to the foundation and the soil are generally either neglected or computed separately (Mylonakis et al., 2006). Once the structural problem is solved, the foundation response is usually computed by application of the loads transmitted by the super-structure. In this process, the role of dynamic soil-structure interaction (SSI) is usually accounted for only by a suitable change (elongation) of the fundamental vibration period of the compliant soil-structure system, based on the linear-elastic

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assumption, and by an increase of system damping due to the energy dissipation at the soil-foundation level (Stewart et al., 2003; Gajan et al., 2010).

In this paper a new pseudo-static design procedure, based on an equivalent linear visco-elastic approach, and aiming at introducing non-linear soil-structure interaction in the footing-structure design is described. It is based on curves quantifying the foundation stiffness degradation and the corresponding increase in damping as a function of foundation rotation. The curves have been obtained by fitting the results of both large scale laboratory experimental tests on rigid shallow foundations, and numerical simulations based on a macro-element model. Subsequently, these curves have been introduced in a performance-based structural design method, the Direct Displacement Based Design (DDBD, Priestley et al., 2007), which is also based on an equivalent linear visco-elastic approach. Some design examples are then discussed, referring to the application of the new iterative procedure to bridge piers. The proposed procedure is compared with fixed base design procedures. Results are checked by means of non-linear time history analyses performed with a macro-element based numerical tool, which includes a non-linear characterization for both the structure and the soil-foundation system.

**STIFFNESS AND DAMPING OF A SHALLOW FOUNDATION UNDER CYCLIC LOADING**

The non-linear cyclic behaviour of a shallow foundation resting on a frictional soil can be modeled either by means of an elasto-plastic formulation or by an equivalent linear visco-elastic characterization, in which the two key concepts of secant stiffness and equivalent viscous damping are used. Since the response of a shallow foundation supporting a structure subject to seismic loading is dominated by the rotational component (Paolucci et al., 2008; Gazetas et al., 2007), only the rotational secant stiffness \( K_F \) and the rotational damping ratio \( \xi_F \) are considered.

In Fig. 1 examples of curves of the \( K_F \) decay and \( \xi_F \) increase as a function of the rotation angle \( \theta \), obtained from the data processing of experimental test measures on both dense and medium dense sands (TRISEE, Faccioli et al., 1998 and PWRI, 2005), are shown. In Fig. 1a the values of the ratio \( K_F / K_{F,0} \) are plotted for high dense and medium dense sand, respectively, where \( K_{F,0} \) is the initial rotational stiffness of the system. In the same graphs the values obtained by numerical analyses performed by the macro-element model developed by di Prisco et al. (2003) are also shown. It is worth noting that, even for low values of the rotation angle (for example 0.001 rad), the rotational secant stiffness of the foundation reduces by a percentage varying from 40% to 60%, depending on the relative density of the soil. This result (for dense sand) depends on the large initial stiffness value. In Fig. 1b the damping factor \( \xi_F \) is shown, with values varying between 5% and 10% for rotations up to 1 mrad, and increasing significantly for higher rotations up to about 20% for dense sands and 30% for medium dense sands.

To investigate the effect of loading path and generalize the experimental results, a series of numerical simulations was performed using the di Prisco et al. (2003) macro-element model. It is an elasto-plastic model, based on a bounding surface plasticity formulation, which is suitable for the description of footings subject to cyclic loading. It does not include a modeling of the uplift phenomenon. A set of curves obtained by interpolation of numerical results obtained by the mentioned macro-element is shown in Fig. 2. The vertical load \( N \) was kept constant, while the overturning moment \( M \) was varied and the horizontal load \( V \) was maintained equal to zero. The variation of \( K_F / K_{F,0} \) and \( \xi_F \) is plotted as a function of \( \theta \) for different values of \( N_{max} / N \), where \( N_{max} \) is the foundation vertical static bearing capacity. These results justify the significant dispersion of experimental and numerical data plotted in Fig. 1, in which no distinction about the loading path has been done.
Figure 1. Decay of shallow foundation rotational secant stiffness (a, top) and variation of damping (b, bottom) for dense (left) and medium dense (right) sands, as a function of foundation rotation. Results from TRISEE and PWRI large scale cyclic tests are considered, as well as numerical results from the di Prisco et al. (2003) macro-element model.

The numerical results were then interpolated by the curves displayed in Fig. 2, having the following expressions:

\[
\frac{K_F}{K_{F,0}} = \frac{1}{1 + a \theta^m}
\]

(1)

\[
\xi_F = \xi_{F,\text{min}} + (\xi_{F,\text{max}} - \xi_{F,\text{min}}) \left[1 - \exp(-b \theta)\right]
\]

(2)

In the previous equations, \(\xi_{F,\text{min}}, \xi_{F,\text{max}}, a, m \) and \(b\) are non-dimensional parameters and vary as a function of the sand relative density and of the static safety factor \(N_{\text{max}} / N\). For large values of \(\theta\), Eq. (2) is modified with respect to numerical results by introducing a saturation value for \(\xi_F\) in order to take into account the effect of uplift. In fact, the di Prisco macro-element model does not consider uplift, so that it gives rise to hysteretic cycles increasing in size as the deformation increases. The result is a damping factor which in turn increases indefinitely as the deformation gets larger. On the contrary, experimental tests show that hysteretic cycles in the moment-rotation plane tend to assume an ‘S’ shape as footing uplift starts. Beyond a certain deformation threshold, the area enclosed in the hysteretic cycles keeps constant, so that also the associated damping tends to saturate. The saturation value, \(\xi_{F,\text{max}}\), is calibrated...
based on the available laboratory test results: it is equal to 0.25 for dense sands and to 0.37 for medium dense sands. The minimum value of $\xi_F$, called $\xi_{F\text{,min}}$, has been fixed equal to 0.036 regardless of sand relative density. The values of $a$, $m$ and $b$ both for dense and medium dense sands and for different values of the static safety factor are summarized in Table 1. It is worth noting that, since curves expressed by Eqs. (1) and (2) have been obtained by interpolation of numerical results from cyclic tests, some care must be taken in their use, especially when extended to seismic loading. In particular, it is suggested to limit foundation rotation to 0.02 rad and to impose a saturation value also for the ratio $K_F / K_{F\text{,0}}$. From several calculations based on design examples, it has been found that a reasonable lower limit for $K_F / K_{F\text{,0}}$ is 0.15 (this threshold has been indicated in Fig. 2).

![Figure 2](image-url)

**Figure 2.** Influence of the loading path on the rotational secant stiffness degradation (top) and damping increase (bottom), for dense (left) and medium dense (right) sand. Analytical interpolations of numerical results obtained by the di Prisco et al. (2003) macro-element model.

**ITERATIVE PROCEDURE TO ACCOUNT FOR NON-LINEAR SSI IN THE DISPLACEMENT-BASED DESIGN APPROACH**

The procedure, aiming at introducing non-linear soil-structure interaction effects in the direct displacement based design (DDBD), is based on the idea that it is possible to apply the standard DDBD, for the description of which we make reference to Priestley et al. (2007), to the equivalent 1 dof oscillator. To take into account soil-foundation non-linearities, iterations are needed, and at each iterative step soil stiffness and damping are updated as a function of the current foundation rotation, which is caused by the actions (base moment) calculated throughout DDBD.
The iterative procedure, denoted by DDBD + SSI (Direct Displacement Based Design with Soil Structure Interaction), is summarized by the following points and by the flow chart of Fig. 3:

1 – Definition of desired performance. This can be sub-divided in: (a) system global performance (total lateral target displacement $\Delta d$), (b) foundation performance (foundation rotation under a certain limit: $\theta < \theta_{LIM}$), (c) structure performance (limited structural ductility: $\mu < \mu_{LIM}$).

2 – Structure design. The geometry of the pier is assigned (diameter $D$), along with a first tentative reinforcement content. Based on these quantities, the pier yield displacement $\Delta y$ and limit base shear $V_{b,LIM}$ are evaluated.

3 – Foundation design. The geometry of the foundation is assigned (width $B$). Then the vertical static bearing capacity $N_{max}$ and the safety factor $FS$ are evaluated. The foundation limit moment $M_{F,LIM}$ is also estimated (e.g. following the Nova and Montrasio (1991) formulation). The initial elastic rotational stiffness of the foundation $K_{F,0}$ is computed (e.g. according to Gazetas, 1991).

4 – Construction of the curves of foundation rotational stiffness and damping. Once the values of $FS$, $K_{F,0}$ and soil relative density are known, the curves of stiffness degradation and increase in damping (Eqs. (1) and (2)) can be calibrated.

5 – Evaluation of the equivalent 1 dof oscillator properties. At the first iteration the oscillator is considered fixed base; subsequently the vibration period, the equivalent damping and the equivalent input are modified taking into account soil-structure interaction. If $\Delta d$ is defined on strain limits, then it coincides with the structural distortion $\Delta S$. Otherwise the structural distortion is calculated as $\Delta S = \Delta d - \Delta F$ (where $\Delta F$ is the lateral displacement due to foundation rotation, at the first iteration set equal to zero).

The structural ductility is computed as $\mu = \Delta S / \Delta y$. Then the structural damping is calculated as $\xi_s = 0.05 + f(\mu)$, as in standard DDBD. Then equivalent damping and displacement input at the corner period $T_{CO}$ are calculated as in Priestley et al. (2007):
\[ \xi_{eq} = \frac{\xi_S \Delta_F + \xi_S \Delta_S}{\Delta_F + \Delta_S} \]  \hspace{1cm} (3)

\[ \Delta_{c, \xi_{eq}} = \Delta_{c, 5} \left( \frac{0.10}{0.05 + \xi_{eq}} \right)^{0.5} \]  \hspace{1cm} (4)

At the first iteration Eq. (3) becomes \( \xi_{eq} = \xi_S \). In Eq. (4) \( \Delta_{c, 5} \) is the corner spectral displacement for 5% damping (see Eurocode 8-Part 1, CEN (2003)).

6 – Application of the DDBD to the equivalent oscillator. The standard DDBD procedure is performed to calculate the design base shear \( V_{b,d} \):

\[ T_{eq} = T_{CO} \frac{\Delta_d}{\Delta_{c, \xi_{eq}}} \]  \hspace{1cm} (5)

\[ K_{eq} = m_2 \left( \frac{2\pi}{T_{eq}} \right)^2 \]  \hspace{1cm} (6)

\[ V_{b,d} = K_S \Delta_S = K_{eq} \Delta_d \]  \hspace{1cm} (7)

Eq. (5) is valid for the linear branch of the design displacement spectrum. Eq. (7) is clearly based on the assumption of in series system.

7 – Calculation of foundation rotation. Once base shear is determined, the moment acting on the foundation \( M_F \) can be easily computed, as well as the foundation rotation \( \theta \) and the lateral displacement due to foundation rotation \( \Delta_F \):

\[ M_F = V_{b,d} H \]  \hspace{1cm} (8)

\[ \theta = \frac{M_F}{K_F} \]  \hspace{1cm} (9)

\[ \Delta_F = \theta H \]  \hspace{1cm} (10)

At the first iteration of equation (9) \( K_F \) is set equal to \( K_{F,0} \), thus \( \theta \) is equal to the value of rotation on an elastic soil \( \theta_0 \). If \( \theta_0 > 0.1 \) mrad, then it is imposed that \( \theta = \theta_0 = 0.1 \) mrad. This is made to prevent that a high value of base shear obtained from the fixed base solution forces the initial guess of footing rotation to an excessive value, thus compromising the whole iterative procedure.

8 – Update of soil stiffness and damping. Introducing the current value of rotation \( \theta \) in Eqs. (1) and (2), the new values of \( K_F \) and \( \xi_F \) are determined.
Figure 3. Flow chart summarizing the DDBD+SSI procedure for the seismic design of a bridge pier with circular cross-section of diameter D and shallow foundation width B.

9 – Check of θ convergence. The procedure comes back to point 5 and a new iterative step is performed, until the convergence on foundation rotation θ is achieved.

10 – Check of foundation performance and limit moment condition. The conditions θ < θ_{LIM} and M_F < M_{F,LIM} are checked. If they are not satisfied the procedure comes back to point 3, and the foundation width B is increased.

11 – Check of structure performance and limit base shear condition. The conditions μ < μ_{LIM} and V_{b,d} < V_{b,LIM} are checked. If they are not satisfied the procedure comes back to point 2, and the pier diameter D is increased (alternatively, if possible, reinforcement content is increased).

It is worth noting that the whole procedure makes reference to values at system peak response, so that no information is obtained concerning residual displacements and rotations, both for the structure and for the foundation. However, this is an intrinsic limit of a pseudo-static procedure, which can be overcome by imposing as maximum allowable displacements the permanent ones. Concerning foundation behaviour, there are well consolidated reference values of maximum permanent displacements and rotations for static loading (e.g. Lambe and Whitman, 1969), while there are few experiences that allow to establish limits for the dynamic case. As a reference, the Japanese norm for highway bridges (PWRI, 2003) allows a foundation permanent rotation angle of 0.02 rad. This threshold was fixed after the Kobe 1995 earthquake, when, based on repairing times and costs, only piers with a residual inclination less than 0.02 rad (∼1°) were repaired (M. Shirato, PWRI, personal communication). The same threshold was also used in Adapazari, Turkey, after the August 1999 earthquake, to select the repairable buildings among those...
which suffered permanent foundation overturning (M. T. Yilmaz, METU Ankara, personal communication).

**EXAMPLE OF APPLICATION**

As a practical example, the proposed procedure has been applied to the seismic design of a circular bridge pier, lying on square shallow foundations. The design has been carried out considering three alternative approaches: force-based design considering fixed base conditions (FBD), Direct Displacement Based Design considering fixed base conditions (DDBD), and the previously proposed DDBD+SSI procedure, which takes into account non-linear soil-structure interaction effects. The structure and the foundation are assumed to be uncoupled in the first two design methods: this means that the structure is first designed, then, once the actions acting at the structural base are known, the foundation is dimensioned. The DDBD+SSI procedure considers coupling between the foundation and the super-structure, so that it provides a combined design solution for both the structure and the footing.

The super-structure parameters have been adapted from a case studied by Restrepo (2007). The distributed weight of the bridge deck, including asphalt, is $W_{\text{deck}} = 175$ kN/m. The deck span length is 50 m, so that the mass supported by each pier is 8750 kN. The bridge is excited in the transverse direction, so that the pier is assumed to behave as a 1 dof oscillator, with mass $m_s$ equal to the deck mass plus one third of the pier mass, according to Priestley, et al. (1996). The bridge deck is considered to respond elastically, while inelastic action is intended to concentrate at the bottom of the pier. The pier height is 20 m. Typical conditions for shallow foundations soil are chosen: a dense sand, characterized by friction angle $\phi = 35^\circ$, shear modulus $G = 80$ MPa, and shear wave velocity at ground surface $V_s = 200$ m/s. The seismic input is defined by the Eurocode 8 design spectrum (Eurocode8-Part 1, CEN (2003)), with a peak ground acceleration on firm soil $a_g = 0.5g$.

The following design performance are defined: (i) system limit drift $\Delta_d = 0.03H$, (ii) maximum foundation rotation $\theta_{\text{LIM}} = 0.01$ rad, and (iii) maximum structure ductility demand $\mu_{\text{LIM}} = 3.2$. Design outputs are represented by pier diameter $D$, pier longitudinal and transverse reinforcement content and foundation width $B$. The pier reinforcement content is determined by performing a moment-curvature analysis, based on the design actions calculated by each considered design approach. In the force-based design (FBD) case, reinforcing steel yield strength is $f_y = 455$ MPa, while concrete compression strength is $f'_{c} = 30$ MPa. Material strengths in the DDBD and DDBD+SSI cases are higher than those used in the FBD, according to Priestley et al. (2007, section 4.2.6): concrete compression strength is $f'_{ce} = 1.3\cdot f'_{c} = 39$ MPa, while steel yield strength is $f'_{y} = 1.1\cdot f_y = 500$ MPa. The confined concrete stress-strain curve is determined by standard procedures for all the considered design approaches (see Priestley et al. (2007), section 4.2.2). The foundation height is assumed to be constant and equal to $h_f = 1.5$ m, while the foundation embedment depth is 1.7 m. In the fixed base FBD and DDBD cases, foundation design is carried out following Eurocode 7 (CEN, 2004) and Eurocode 8 Part 5 (CEN, 2003) prescriptions. The maximum possible actions transmitted by the super-structure ($V_{b,\text{LIM}}$) are taken from the pier force-displacement curve calculated after the moment-curvature analysis, according to capacity design principles, then the vertical static bearing capacity $N_{\text{max}}$ and the design resistance under eccentric loading $R_{d}$ are calculated according to the Brinch-Hansen formulation (Brinch-Hansen, 1970). Furthermore, foundation rotation $\theta$ is computed by dividing the foundation design moment by the rotational elastic stiffness of the soil, calculated according to Gazetas (1991), in which a reduced soil shear modulus has been considered ($G_{\text{red}} = 0.5G$). The results of the design process conducted by the three considered design approaches are summarized in Table 2.
The design results have subsequently been checked by means of a series of non-linear time history analyses. They have been performed by the macro-element based numerical tool described in Figini (2010) and Figini et al. (2010). The super-structure is modelled as a 1 dof oscillator characterized by a “Thin” Takeda hysteresis rule, in which the initial stiffness $K_i$ and the post-yield stiffness $rK_i$ are taken from the bi-linear force-displacement relationships obtained from moment-curvature analyses. The unloading stiffness is defined as $K_i \mu^{0.5}$, according to Priestley et al. (2007). Soil-foundation behaviour is modelled by the 3 dof macro-element described by Figini et al. (2010), with the proposed reference set of parameters to account for soil plasticity, foundation uplift and stiffness degradation. Soil elastic impedances have been determined based on the formulas in Gazetas (1991).

**Table 2. Summary of main parameters for bridge pier seismic design.**

<table>
<thead>
<tr>
<th>Structure</th>
<th>FBD</th>
<th>DDBD</th>
<th>DDBD+SSI</th>
</tr>
</thead>
<tbody>
<tr>
<td>$m_s$ (tons)</td>
<td>1043</td>
<td>955</td>
<td>973</td>
</tr>
<tr>
<td>$D$ (m)</td>
<td>3.4</td>
<td>2.2</td>
<td>2.5</td>
</tr>
<tr>
<td>$N_u$ (kN)</td>
<td>13204</td>
<td>10615</td>
<td>11153</td>
</tr>
<tr>
<td>$V_{u,d}$ (kN)</td>
<td>2775</td>
<td>1647.5</td>
<td>1612</td>
</tr>
<tr>
<td>$M_y$ (kN)</td>
<td>55500</td>
<td>32950</td>
<td>32240</td>
</tr>
<tr>
<td>Longitudinal rebar</td>
<td>100ø26 (ρ = 0.6%)</td>
<td>110ø26 (ρ = 1.54%)</td>
<td>100ø26 (ρ = 1.08%)</td>
</tr>
<tr>
<td>Transverse rebar</td>
<td>φ12, s = 50 mm (ρv = 0.3%)</td>
<td>φ12, s = 70 mm (ρv = 0.3%)</td>
<td>φ12, s = 70 mm (ρv = 0.3%)</td>
</tr>
<tr>
<td>$\Delta_y$ (m)</td>
<td>0.1861</td>
<td>0.3335</td>
<td>0.2886</td>
</tr>
<tr>
<td>$V_{by}$ (kN)</td>
<td>2738</td>
<td>1700</td>
<td>1931</td>
</tr>
<tr>
<td>$\Delta_u$ (m)</td>
<td>0.6166</td>
<td>0.7880</td>
<td>0.7704</td>
</tr>
<tr>
<td>$V_{b,LM}$ (kN)</td>
<td>2850</td>
<td>1747</td>
<td>1976</td>
</tr>
<tr>
<td>$\Delta_S$ (m)</td>
<td>0.5583</td>
<td>0.393</td>
<td>0.251</td>
</tr>
<tr>
<td>$\mu$</td>
<td>3</td>
<td>1.15</td>
<td>0.84</td>
</tr>
<tr>
<td>$\xi_S$ (%)</td>
<td>-</td>
<td>6.9</td>
<td>5</td>
</tr>
<tr>
<td>$T_S$ (s)</td>
<td>1.06</td>
<td>3</td>
<td>2.45</td>
</tr>
<tr>
<td>Foundation</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$B$ (m)</td>
<td>11.5</td>
<td>9.5</td>
<td>7.5</td>
</tr>
<tr>
<td>$V_d$ (kN)</td>
<td>2850</td>
<td>1747</td>
<td>1612</td>
</tr>
<tr>
<td>$M_y$ (kN)</td>
<td>57000</td>
<td>34940</td>
<td>32240</td>
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<tr>
<td>$N_{max}$ (kN)</td>
<td>284360</td>
<td>173500</td>
<td>95112</td>
</tr>
<tr>
<td>$N_{eo}$ (kN)</td>
<td>18069</td>
<td>13935</td>
<td>13299</td>
</tr>
<tr>
<td>$FS$</td>
<td>15.7</td>
<td>12.5</td>
<td>7.15</td>
</tr>
<tr>
<td>$R_d$ (kN)</td>
<td>18691</td>
<td>18408</td>
<td>-</td>
</tr>
<tr>
<td>$M_{F,LIM}$ (kNm)</td>
<td>78812</td>
<td>58321</td>
<td>40274</td>
</tr>
<tr>
<td>$\Delta_F$ (m)</td>
<td>-</td>
<td>-</td>
<td>0.126</td>
</tr>
<tr>
<td>$\theta$ (rad)</td>
<td>0.00146</td>
<td>0.00160</td>
<td>0.0061</td>
</tr>
<tr>
<td>$\xi_F$ (%)</td>
<td>-</td>
<td>-</td>
<td>10.7</td>
</tr>
<tr>
<td>$T_F$ (s)</td>
<td>-</td>
<td>-</td>
<td>1.73</td>
</tr>
<tr>
<td>Equivalent 1 dof</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\Delta_d$ (m)</td>
<td>0.5583</td>
<td>0.393</td>
<td>0.377</td>
</tr>
<tr>
<td>$\xi_{eq}$ (%)</td>
<td>6.9</td>
<td>6.9</td>
<td>6.88</td>
</tr>
<tr>
<td>$T_{eq}$ (s)</td>
<td>1.06</td>
<td>3</td>
<td>3</td>
</tr>
</tbody>
</table>
Five displacement spectrum compatible accelerograms have been selected as seismic input. They fit on average the Eurocode 8 displacement spectrum, Type 1, corresponding to $a_g = 0.5g$ and soil class C, for periods up to 4 s (see Fig. 4).

![Figure 4. 5% displacement spectra of records used for non-linear time history analyses](image)

The average results of non-linear time history analyses (NLTHA) are reported in Table 3 and compared with design results. An example of results obtained from the dynamic simulations considering the model excited by the Duzce accelerogram are shown in Fig. 5.

<table>
<thead>
<tr>
<th>Design parameter</th>
<th>FBD</th>
<th>DDBD</th>
<th>DDBD+SSI</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_o$ (kN)</td>
<td>Design Value</td>
<td>Mean NLTHA</td>
<td>Design Value</td>
</tr>
<tr>
<td></td>
<td>2775</td>
<td>2759±6.1</td>
<td>1648</td>
</tr>
<tr>
<td>$\mu$</td>
<td>3</td>
<td>1.60±0.15</td>
<td>1.15</td>
</tr>
<tr>
<td>$\Delta_r / \Delta_0$</td>
<td>-</td>
<td>0.182±0.016</td>
<td>-</td>
</tr>
<tr>
<td>$\Delta_{sl}$ (m)</td>
<td>0.56</td>
<td>0.346±0.037</td>
<td>0.393</td>
</tr>
<tr>
<td>$\theta_{\max}$ (mrad)</td>
<td>1.46</td>
<td>2.72±0.40</td>
<td>1.60</td>
</tr>
<tr>
<td>$\theta_{\perm}$ (mrad)</td>
<td>-</td>
<td>0.13±0.10</td>
<td>-</td>
</tr>
</tbody>
</table>
Figure 5. Dynamic response of pier P2 ($H = 20$ m) to the Düzce accelerogram. From left to right: FBD, DDBD, DDBD+SSI. Top: structural response, bottom: foundation rocking response.

FBD yields to different structural dimensions with respect to DDBD: pier diameter $D$ is 3.4 m in the first case and 2.2 m in the second case. This is mainly due to the constant force reduction factor assumed in FBD, independently of pier height, which enforces to enlarge pier diameter in order to respect the prescribed drift limit value. However longitudinal rebar is not so different between the two design approaches. The difference exists in foundation dimensions: the base width is $B = 11.5$ m and 9.5 m, respectively. The global system behaviour is substantially well predicted by the two design methods: structure develops non-linearities, while foundation is considered as a fixed end. Dynamic analyses confirm the latter hypothesis, since soil-structure interaction effects are found to be of limited entity. If design is made using the DDBD+SSI procedure, a further reduction in base width is possible, down to $B = 7.5$ m. Reinforcement content does not change significantly with respect to the fixed base design. The interesting aspect is that ductility demand $\mu$ decreases, so that the structure remains elastic. This behaviour is confirmed also by time-history analyses, and points out a clear advantage in using the DDBD+SSI design procedure: besides reducing foundation dimensions, the structure does not yield during earthquake, because it is protected by the energy dissipation provided at the soil-foundation level. However, the proposed procedure presents some lacks in the prediction of foundation peak rotation $\theta_{\text{max}}$: it is underestimated due to uplift effects, which have not been included in the construction of curves for
rotational secant stiffness degradation and damping increase as a function of foundation rotation. From a design point of view, the interesting aspect is represented by the control of permanent rotations $\theta_{\text{perm}}$, which is guaranteed in case of footing uplift by the reversible character of the phenomenon: the foundation tends to self-centre at the end of the excitation. Finally, it is interesting to observe the difference in system dimensions obtained by the three considered design approaches. In the FBD case, pier diameter $D = 3.4$ m and footing width $B = 11.5$ m. In the DDBD case, pier diameter $D = 2.2$ m and footing width $B = 9.5$ m. In the DDBD+SSI case, pier diameter $D = 2.5$ m and footing width $B = 7.5$ m. The economical advantage given by DDBD+SSI is evident, especially when considering foundation dimensions, but also considering that during the design earthquake the pier designed with DDBD+SSI will remain elastic, while the pier designed with FBD and DDBD will yield and present permanent deformations at the end of the excitation.

CONCLUSIONS

An iterative pseudo-static design procedure has been presented in this paper, which introduces non-linear soil-structure interaction effects in the displacement based seismic design. It is based on the use of curves describing the decay of rotational stiffness and increase in damping as a function of the foundation rotation. These curves have been obtained both experimentally and numerically by employing a constitutive model based on the macro-element concept. The iterative design procedure is based on an equivalent 1 dof visco-elastic oscillator, characterized by a secant stiffness and equivalent damping corresponding to the global foundation-soil-superstructure system, which is assumed to be composed by two systems in series, the foundation-soil and super-structure ones. The approach has been applied to the design of a bridge pier, by comparing standard fixed base design approaches with the new proposed one. Design results have been finally checked by means of non-linear time history analyses performed with a macro-element based numerical tool.

ACKNOWLEDGEMENTS

This work has been partly funded by the Italian Department of Civil Protection under the DPC-RELUIS 2005-09 agreement, in the framework of the Project “Development of Displacement-based approaches for the Seismic Design and Vulnerability Assessment”.

REFERENCES


