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# TWO-DIMENSIONAL NONLINEAR SEISMIC ANALYSIS OF SOIL-WELL-PIER SYSTEM CONSIDERING SOIL NONLINEARITY

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## ABSTRACT

Nonlinear seismic analysis of soil-well-pier (SWP) system of a typical bridge supported on well foundation is carried out considering soil nonlinearity while the well and the pier are assumed to behave linearly. Nonlinear behaviour of soil is captured in two different ways: (a) a rigorous analysis considering nonlinear constitutive model of soil, and (b) an approximate analysis by equivalent-linear method. In the first method, multiple yield surface plasticity model available in OpenSees is used to rigorously model nonlinear hysteretic behaviour of soil. In equivalent-linear method, analysis has been carried out in two steps. In the first step, free-field analysis of soil column is performed in SHAKE2000 for a given acceleration time-history using shear-strain-dependent shear modulus and damping properties of soil to obtain effective shear modulus and damping values at each layer of soil. In the next step, these effective properties of soil are used in finite element model of soil-well pier system and linear seismic analysis is performed in OpenSees. Results of equivalent-linear analysis are compared with those from nonlinear analysis, with the objective to assess suitability of approximate analysis in the design offices.

## Introduction

Well foundations are frequently adopted in Indian subcontinent and other countries like Japan (known as caisson foundation) for the foundation of railway and road bridges on rivers. These are massive structures embedded in river bed and the embedment depth may vary from 30m to 60 m. Many structures supported on caisson foundations suffered severe damage during Kobe (1995) earthquake. Soil-well interaction considering nonlinear behaviour of soil plays an important role for response of such deep foundations subjected to earthquake motions. However, in practice, effect of soil nonlinearity is generally ignored or sometimes is considered approximately by equivalent-linear analysis for simplicity. The present study aims to validate such simple analysis by comparing it with rigorous nonlinear dynamic analysis.

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## FE Modeling Of Soil-Well-Pier System

Well foundation considered in the present study is a typical double-D hollow section (Fig. 1). It is a 50 m deep foundation fully embedded in dry cohesionlless soil. Fig. 2 shows the FE discretization of the entire SWP system. In the dynamic behaviour of the substructure, the superstructure stiffness does not contribute significantly (Chang et al. 2000) and hence only the mass of the superstructure is modeled and is applied at the pier cap. Hydrodynamic mass is not considered in the present study. The mass of water and sand inside the well have been considered in the analysis. In the FE model, soil domain is discretized using four-noded, bilinear, isoparametric finite elements with 2 DOF at each node under plain-strain condition, while well and piers are discretized using two-noded linear beam-column elements with 3DOF at each node. Massless rigid-outrigger elements are added to the embedded part of the well to account for the breadth of the well when interacting with soil. Nonlinearity at interface (i.e., gapping, sliding at the soil-well interface) and in structure is ignored. Viscous boundary proposed by Lysmer and Kuhlemeyer (1969) is used as radiation boundary at two vertical boundaries and at base of the FE model. Ground motion is applied at the base of the FE model in the form of equivalent shear force proportional to velocity of incident wave motion (Joyner and Chen 1975). Analysis is carried out for nine ground motions with free-field peak ground accelerations ranging from 0.12g to 0.68g.

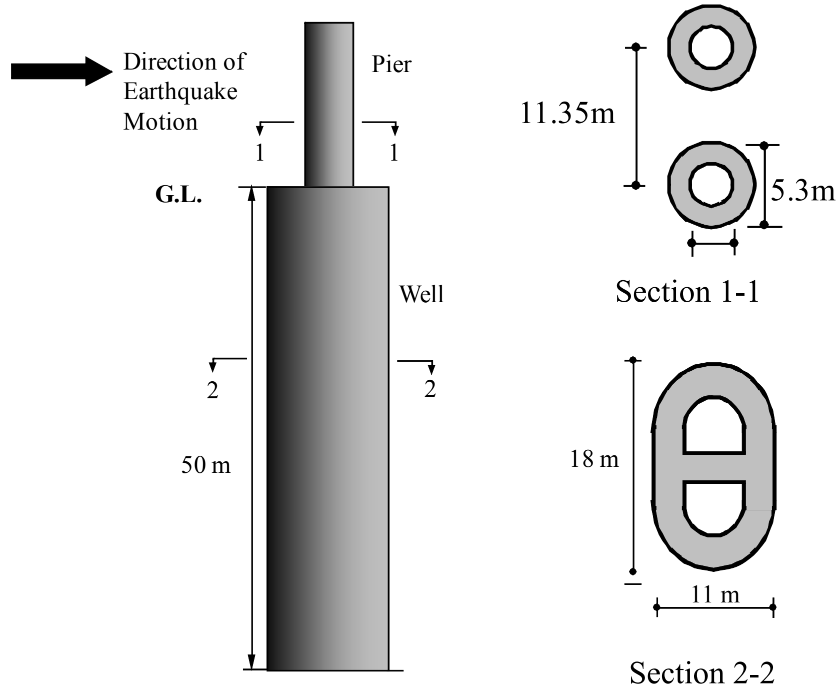


Figure 1. Schematic of the well foundation used in the present study

## Material Properties

The soil domain is modeled up to the bed rock which is assumed at 100 m depth from the ground surface. The soil profile considered in the present study consists of three layers of cohesionless soil (Table 1). Table 1 shows unit weight and Poisson's ratio and reference shear modulus at a reference confining pressure of 80 kPa of these layers.

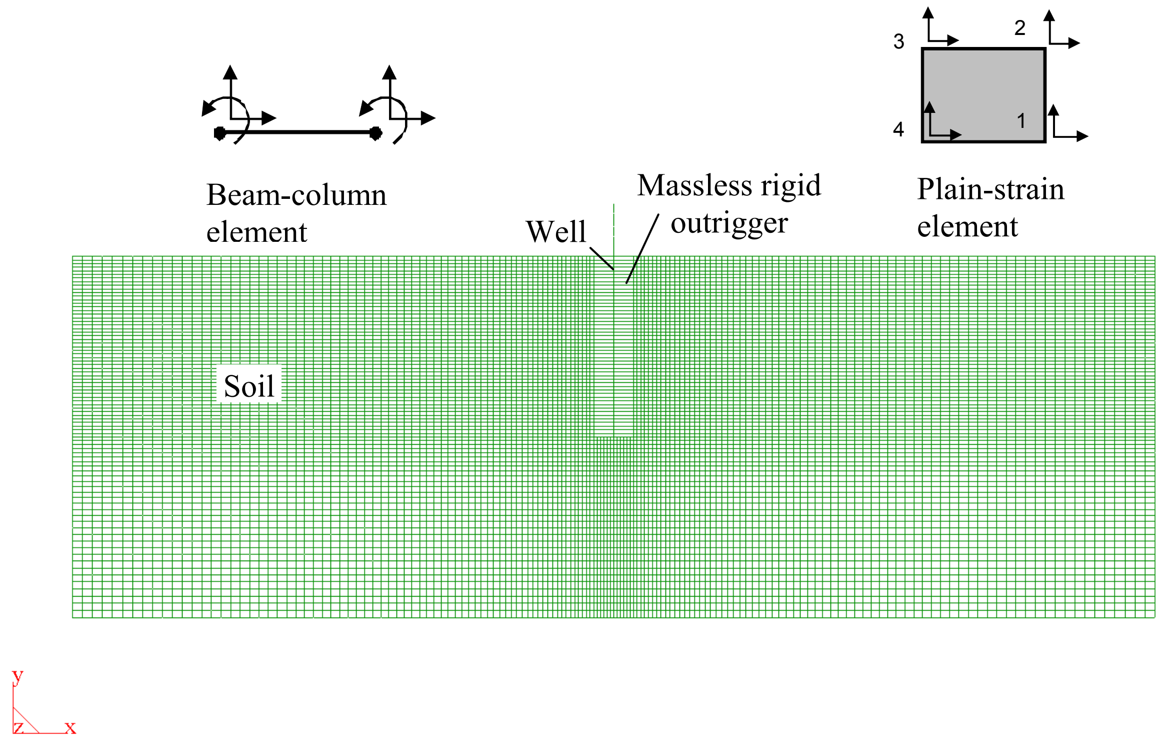


Figure 2. FE discretization of SWP system

## Selection of Earthquake Motions

Nine earthquake motions recorded at different geographical locations are selected for the dynamic analysis (Table 2). These ground motions represent different source mechanism and epicentral distances. These are recorded at ground level at rock- outcrop as free-field motions during strong earthquake with magnitude 6.5 and above. The ground motions with PGA ranges from 0.1g to 0.3g, 0.3g to 0.5g and 0.5 g to 0.7 g are scaled for 0.2g, 0.4g and 0.6g, respectively. These three sets of ground motions are termed as small, medium and large ground motions, respectively. In the present study since only horizontal ground motion is considered, the system will have mainly the vertically propagating shear wave. Moreover, the ground motions are recorded at rock site as free-field motion and the wave propagating medium can be assumed as homogeneous, undamped and elastic half-space. In such a case, the amplitude of the free-surface total motion is twice the amplitude of the incident motion at any location in the half-space (Kramer 1996). As a result, for each earthquake motions, the incident seismic motions (acceleration) are taken as half the scaled free-surface motion (acceleration). This incident seismic acceleration is integrated to obtain velocity time history of the incident seismic wave which is used to determine the equivalent nodal forces at the base of the finite element model.

Table 1: Parameters for constitutive model

| Layers | Depth | Type of Soil | Unit Weight | Poisson's ratio | $G_r$* | $\phi$ * | $\gamma_{\max}$ * | $\phi_{PT}$ * | *contrac** |
|---|---|---|---|---|---|---|---|---|---|
| Layer 1 | 0m-20 m | Medium sand | 1.9 | 0.33 | $7.5\times10^4$ | 33 | 0.1 | 27 | 0.07 |
| Layer 2 | 20m-50m | Medium-Dense Sand | 2.0 | 0.35 | $1.0\times10^5$ | 37 | 0.1 | 27 | 0.05 |
| Layer 3 | 50m-100m | Dense Sand | 2.1 | 0.35 | $1.3\times10^5$ | 40 | 0.1 | 27 | 0.03 |

* $G_r$ = reference shear modulus specified at confining pressure of 80 kPa
$\phi$ = Angle of internal friction
$\gamma_{\max}$ =Octahedral shear strain at which the maximum shear strength is reached
$\phi_{PT}$ = Phase transformation angle
*contrac* = a nonnegative constant defining the rate of shear induced volume contraction or pore pressure build up

## Dynamic Analysis of SWP System

Dynamic analysis of the soil-well pier system is performed in open source code OpenSees, the Open System for Earthquake Engineering Simulation, developed specially to simulate the performance of structural and geotechnical systems subjected to earthquakes (Mazzoni et al. 2006). Nonlinear behaviour of soil is considered by detailed nonlinear analysis and by simple equivalent-liner analysis.

### Nonlinear Dynamic Analysis

In rigorous nonlinear analysis, nonlinear response of soil is simulated using elasto-plastic pressure-dependent multi-yield surface (nested-yield surface) constitutive model (Yang et al. 2008). In this model, a set of Drucker-Prager nested yield surfaces with a common apex and different sizes form the hardening zone are used to simulate nonlinear behaviour of drained (or dry) as well as undrained cohesionless soil (Fig. 3). However, in this study only drained (or) dry conditions are simulated assuming that ground water table is absent up to the bed rock level. The values of the parameters required for the constitutive model are taken from the table given in the user's manual (Yang et al. 2008).

Analysis has been performed in several steps to simulate the initial condition of the SWP model. In the first step, gravity due to the self weight of soil and weight of the embedded portion of well are applied statically considering soil as linear and elastic. Vertical boundaries of soil domain are restrained in horizontal direction only, and the base of the FE model is restrained in both vertical and horizontal directions to develop confining pressure to all the soil elements during the gravity analysis. In the second step, the soil constitutive model has been switched from linear elastic to elastoplastic using the command "updateMaterialStage" available in OpenSees.

Table 2. Description of ground motions

| Level | Earth-quake | Magni-tude | Station | PGA (g) | Compo-nent | Epicentral Distance (km) | Name of Ground Motion |
|---|---|---|---|---|---|---|---|
| Small | Northridge, 1994 | 6.69 | LA-Griffith Park Observatory, (USGS 141) | 0.289 | 270 | 25.4 | S1 |
| | Turkey, 1999 | 7.14 | LAMONT 531 Lamont 531 | 0.117 | E | 27.74 | S2 |
| | Uttarkashi, 1991 | 7.0 | Bhatwari, India | 0.246 | 355 | 19.3 | S3 |
| Medium | Chi-Chi, 1999 | 7.62 | CWB 99999 CHY041 | 0.302 | E | 51.15 | M1 |
| | Loma Prieta, 1989 | 6.93 | Corralitos (CDMG 57007) | 0.479 | 090 | 7.2 | M2 |
| | Northridge, 1994 | 6.69 | UCSB 99999 LA 00 | 0.388 | 090 | 14.41 | M3 |
| Large | Chi-Chi, 1999 | 7.62 | CWB 99999 TCU071 | 0.655 | N | 15.42 | L1 |
| | Northridge, 1994 | 6.69 | DWP 77 Rinaldi Receiving Station | 0.633 | 318 | 10.91 | L2 |
| | Petrolia, 1992 | 7.8 | CGS-89156 | 0.685 | 90 | 4.51 | L3 |

The new equilibrium state under soil gravity is obtained iteratively. In the third step, self weight of pier, well cap and other gravity load, if any, are applied statically to the nonlinear soil model. Reaction forces at the boundary nodes are obtained at the end of gravity analysis. In the fourth step, all the restraints along the boundary nodes are removed and the reaction forces obtained from the gravity analysis of SWP system are statically applied at the corresponding nodes. This is assumed to be the initial condition of the SWP system for dynamic analysis. In the fifth step, both horizontal and vertical radiation dampers are added at the nodes of lateral boundaries and base boundary. These dampers have zero-length and one end of these dampers is connected to the boundary nodes and the other end is fixed in space. Finally, seismic analysis of the SWP system is performed by applying horizontal seismic excitation in the form of effective nodal forces applied at the base of the computational soil domain.

In nonlinear dynamic analysis, soil damping is primarily captured through hysteretic energy dissipation. Therefore, no other damping is considered in this case.

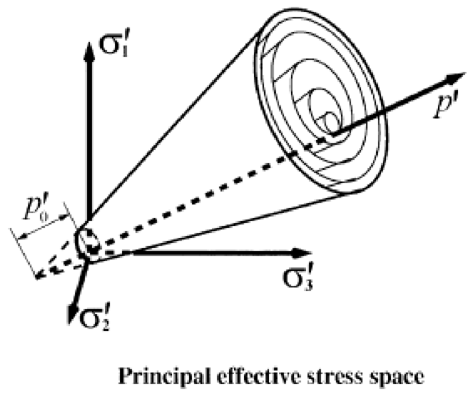


Figure 3. Nested yield surface in principal stress space (Yang et al., 2006)

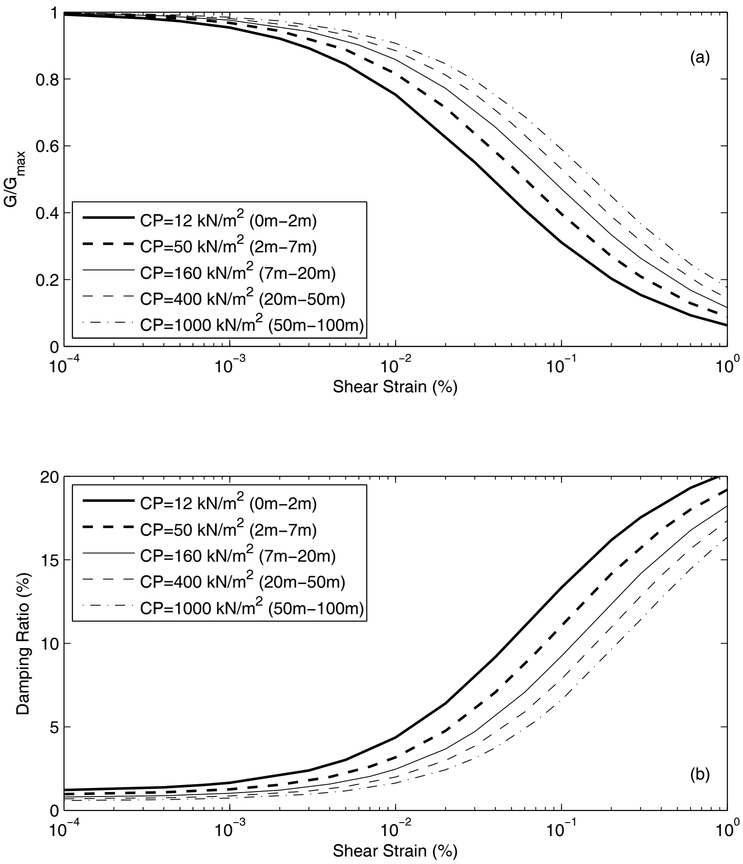


Figure 4. Shear stain dependent (a) shear modulus reduction curves, and (b) damping curves used in equivalent linear analysis in SHAKE for dry soil (Zhang et al. 2005).

## Equivalent-Linear Dynamic Analysis

During ground shaking, shear modulus ($G$) and hysteresis damping ratio ($\zeta$) of soil depend upon the shear strain level in soil and this dependence is highly nonlinear. The equivalent-linear dynamic analysis is performed to account for this nonlinearity in a simple manner. The analysis is performed in two steps. In the first step, free-field analysis of soil column (without the foundation structure) is performed in SHAKE2000 (Ordóñez 2004) for a given input earthquake motion at base. SHAKE uses 1-D wave propagation theory to iteratively calculate the level of maximum strain for each layer and determine the effective dynamic soil properties (i.e., shear modulus and damping) in each layer of soil. In the second step, shear modulus and damping ratio of soil at each layer, obtained from SHAKE2000, are assigned to the soil layers of FE model and linear dynamic analysis is performed in OpenSees for the given ground motions.

Fig. 4 shows the variation of shear modulus reduction ($G/G_{max}$) and damping ratio due to variation of shear strain in dry cohesionless soil at different levels of confining pressure (CP). These curves are obtained from the procedure proposed by Zhang et al. (2005). Ideally, these curves should be calculated for each layers corresponding to the CP of that layer which is time consuming. Stokoe et al. (1995) suggested that the estimated field CP should be within about ±50% of the actual values when selecting curves for design. Therefore, soil profile should be divided into several major units. Average values of CP for each part are compared with CP values of each layer within the unit. If the CP value for each layer is within ±50% of the average value for the corresponding unit, then the average CP is assigned to all layers within the part. Otherwise, the unit is subdivided and new average CP values are calculated. According to this approach, five sets of $G/G_{max}$ and damping curves are needed to characterize the soil profile considered in the present study. $G_{max}$ (in kN/m$^2$) required to estimate G from the shear modulus reduction curve can be obtained from the following equation:

$$G_{max} = G_r \left( \frac{p'}{p'_r} \right)^d \tag{1}$$

where, $G_r$ is the shear modulus at reference confining pressure of $p'_r$ (=80 kPa), and $d$ (=0.5) is a positive constant defining variations of $G$ as a function of instantaneous effective confining pressure $p'$.

In linear time domain analysis stress-strain curve of soil is assumed to be linear and hysteretic energy dissipation does not occur. However, the energy dissipation in the system is approximately captured by considering viscous damping obtained from the equivalent-linear analysis of soil column in SHAKE and is applied in the form of mass and stiffness proportional Rayleigh damping. Rayleigh damping coefficients are determined by considering two target modes, i$^{th}$ and j$^{th}$ having damping ratios ζ. It is common practice to consider the fundamental frequency of the system ($\omega_1$) as the lower target frequency ($\omega_i$) while the higher target frequency ($\omega_j$) can be taken as the odd-integer multiplier, $n$ (i.e., 3, 5, 7, etc.) of the fundamental frequency of the system (Hudson et al. 1994). The parameter n is the closest odd integer greater than, $\omega_{ip}/\omega_1$ where $\omega_{ip}$ is the predominant frequency of the input motion.

## Comparison of Equivalent-Linear and Nonlinear Analysis

Acceleration, displacement and force responses at different locations in pier and well obtained from equivalent-linear analysis are compared with those from nonlinear analysis, and

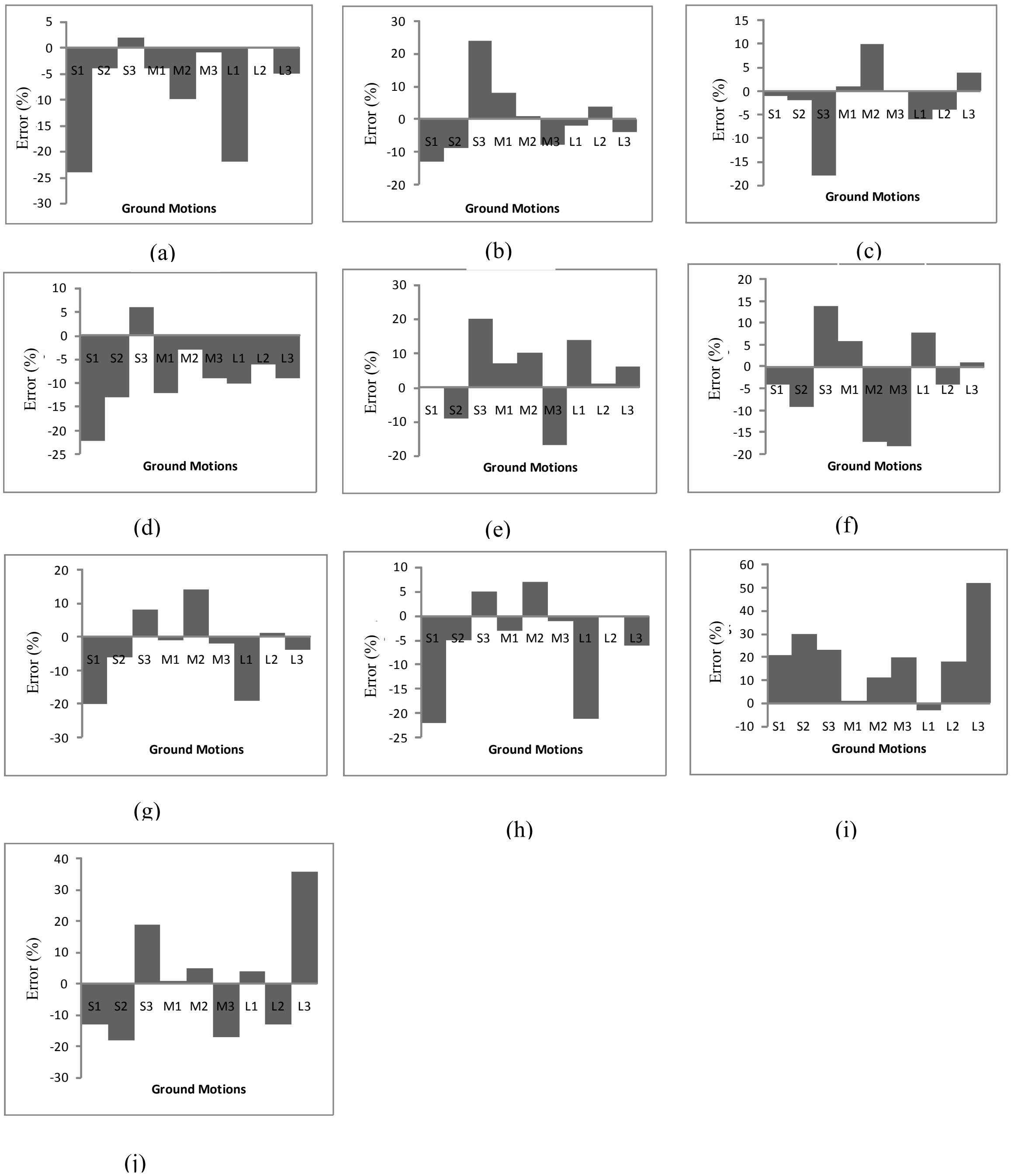


Figure 5. Percentage error in response of well foundation obtained from equivalent-linear analysis: maximum acceleration (a) at pier top, (b) at well top, (c) at well bottom; (d) maximum average acceleration in well; maximum absolute displacement (e) at pier top, and (f) at well top; (g) maximum shear force in pier ;(h) maximum bending moment in pier; (i) maximum shear force in well; and(j) maximum bending moment in well.

the percentage error are shown in Fig. 5. It can be found that error in the maximum acceleration responses in pier top, well top and well bottom is up to 25%. Similarly, maximum absolute displacement at pier top and well top is satisfactorily predicted by equivalent-linear analysis with a maximum error of about 20%. A maximum of about 22% error in shear force and bending moment in pier are observed. A 50% error was estimated in maximum shear force and bending moment in well obtained by equivalent-linear analysis under ground motion L3. However, a plot of shear force and bending moment envelops along the depth of well shows that the envelops obtained from equivalent-linear analysis satisfactorily match with the force envelops obtained by rigorous nonlinear analysis (Fig. 6) except at the location of maximum force. Therefore, in general, it can be inferred that equivalent-linear analysis can satisfactorily predict response of well foundation under small to severe earthquakes.

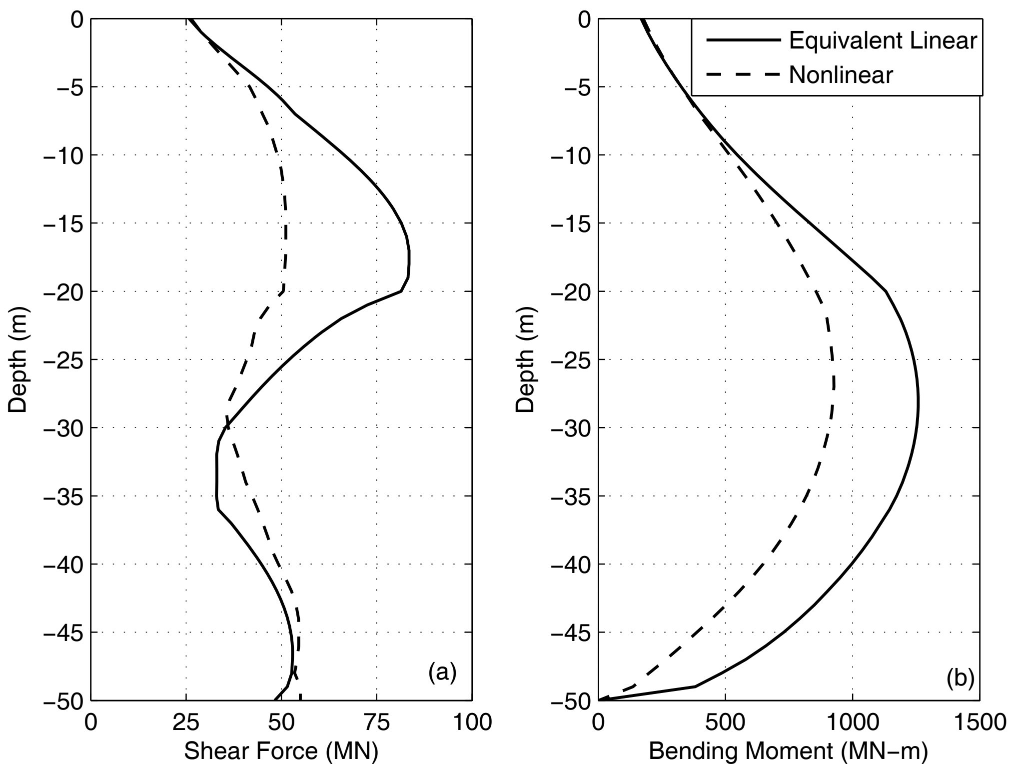


Figure 6. Comparison of (a) shear force envelops and (b) bending moment envelopes in well obtained from equivalent-linear and nonlinear analyses under large ground motion L3.

## Summary and Conclusions

Response of well foundation, commonly used for bridges, has been investigated by complex nonlinear and relatively simple equivalent-linear analysis to assess the suitability of the later in design offices. It is assumed that the foundation soil is dry and cohesionless. Acceleration displacement and force responses obtained from equivalent-linear analysis are compared with those determined by rigorous nonlinear analysis. It is found that equivalent-linear analysis satisfactorily predicts response of both well and pier irrespective of the level of ground shaking, and therefore, it can be can be useful in the design offices for the seismic design of well foundation.

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