Lateral response of single piles in liquefying soil

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ABSTRACT: The lateral response of single piles embedded in a cohesionless soil profile with a horizontal ground surface and no crust layer, subject to dynamic excitation of underlying bedrock has been investigated. During earthquakes, structures supported on pile foundations will interact with the surrounding soil. In saturated cohesionless soils, soil-pile interaction is a challenging and multi-variable problem, and therefore not well understood. This is because saturated cohesionless soil is prone to liquefaction when subject to dynamic loads. These variables include changing soil stiffness and damping, changing soil-pile-structure resonant frequency and changing loads. Nonlinear three-dimensional finite element models were developed in the OpenSeesPL software to represent these phenomena. Based on these models an attempt to setup an elastic equivalent pseudo-static method for approximating the response of piles embedded in liquefiable soils subject to kinematic and inertial loads is outlined.

1 INTRODUCTION

New Zealand currently does not have any specific code or guidelines that deal directly with the design of foundations against liquefaction. The New Zealand design and loadings code NZS 1170.5 explicitly states that it does not address the effects of liquefaction resulting from earthquake motions (Standards New Zealand, 2004). Any guidelines for designing pile foundations in liquefiable soils will have to consider: the strength and stiffness of liquefied soil; the effects of damping in the liquefied soil; the change in natural frequency of the soil-pile-structure system; the definition and application of kinematic and inertial loads onto the pile; and the stability of the pile during liquefaction.

During earthquakes, structures supported on pile foundations will interact with the surrounding soil. This interaction can be split up into kinematic and inertial interactions (Wolf, 1985, Pender, 2012, Madabhushi et al., 2010, Bowen and Cubrinovski, 2008). Kinematic interaction comes from the deformations transferred to the pile shaft from movement of the surrounding soil. Inertial interaction is the result of the response, generated by the accelerations from the ground motions, of the mass attached to the pile (Pecker and Pender, 2000).

The combination of inertial and kinematic loads that act on a pile at any given moment is difficult to assess. The dynamic response of pile foundations is often decoupled to kinematic and inertia loading during design. It is sometimes assumed that peak inertia loads and lateral crust loads act at different times, which could be significantly unconservative (Brandenberg et al., 2005). Generally the pile response near the ground surface is dominated by inertial loads, while the pile response deeper into the ground is dominated by kinematic loads (Madabhushi et al., 2010).

The purpose of this paper is to understand and better quantify how piles respond during earthquakes when the soil is prone to liquefy. This will be carried out by analysing the response of piles embedded in saturated cohesionless soils subject to sinusoidal excitation of the underlying bedrock (i.e. at the base of the model). Nonlinear numerical models developed in OpenSeesPL, will be used for this investigation. OpenSeesPL is a three-dimensional finite element package developed within the OpenSees framework for modelling linear or nonlinear soil-pile interaction (Elgamal et al., 2010, Lu et al., 2010).

The first part of this paper considers the free-field site response in order to obtain the liquefaction potential, degraded soil stiffness and associated damping, maximum ground surface accelerations and maximum ground displacements. The second part looks at the pile response subject to inertial and kinematic interaction. The last part goes through a first cut at setting up an elastic equivalent pseudo-static method for kinematic and inertial interaction based on the nonlinear OpenSeesPL analyses.

2 MODEL SETUP

The soil profile used in the OpenSeesPL analyses was based on the Seal Beach site in California where Scott et al. (1982) carried out full-scale pile tests. It consisted of approximately 6m of saturated medium dense silty sand in the upper layer. Underlying this were layers of silt, clayey silt, sand and siltstone (Scott et al., 1982). The soil properties are given in Table 1.

The pile used in the OpenSeesPL analyses was a 610mm diameter steel-pipe pile embedded 9.8m into the ground. The pile had a wall thickness of 13mm and stiffness (E_pI_p) of 212MN-m^2. The total vertical weight on the pile was approximately 250kN or mass of 25 tonnes.

The free-field site response and pile response were analysed by applying sinusoidal excitations with acceleration of 0.3m/s² to the base of the model (i.e. underlying bedrock).

Table 1 – Soil properties for elastic site response

Soil Description	Depth (m)	Shear modulus, G (MPa) at bottom of layer	Density (ton/m^3)	Shear wave velocity, V_s (m/s) at bottom of layer	Damping ratio, β (% of critical damping) **
Medium dense silty sand	0 – 6	9	2.0	65	2
Medium dense silty sand with some clay	6 – 8	20	2.0	100	2
Firm clayey silt	8 – 9	26	1.7	120	2
Fractured siltstone	9 – 19	70	2.1	180	2
Bedrock (base of model)	19+	250,000 *	2.5	10,000 *	2

* The bedrock stiffness is very high in order to represent the rigid base of the OpenSeesPL model

** Damping is based on Rayleigh damping of 3% at 1Hz and 6Hz

3 FREE-FIELD SITE RESPONSE

Nonlinear free-field site response analyses have been performed using OpenSeesPL in order to calibrate and verify elastic equivalent soil properties to be used in a one dimensional elastic wave propagation method (referred to as "1-D elastic wave propagation" in the text and "Lysmer" in figures). This method is based on notes presented by John Lysmer to classes taught at Berkeley in 1989 (provided by M.J. Pender) and forms the basis of the Shake software programme.

The maximum ground surface acceleration obtained from elastic soil response and recommendations from Youd et al. (2001) were used to determine the liquefaction potential of the soil. The top two soil layers were found to be prone to liquefaction. The stiffness of fully liquefied soil is expected to be in the order of 2% to 10% of the initial stiffness (Bowen and Cubrinovski, 2008, Cubrinovski et al., 2009). This was verified in nonlinear OpenSeesPL analyses by applying a sinusoidal excitation with a frequency of 1.5Hz and acceleration amplitude of 0.3m/s² to the base of the model (refer to shear modulus ratio time history after 6 seconds in Figure 1).

The ground surface acceleration time history, effective confining pressure time histories over the top 2m and shear modulus ratio time history at 0.8m depth are shown in Figure 1. It can be estimated that the soil stiffness at full liquefaction is approximately 10% of the initial stiffness value. Also, it can be seen that the reduction in soil stiffness is related to the excess pore water pressure in the soil.

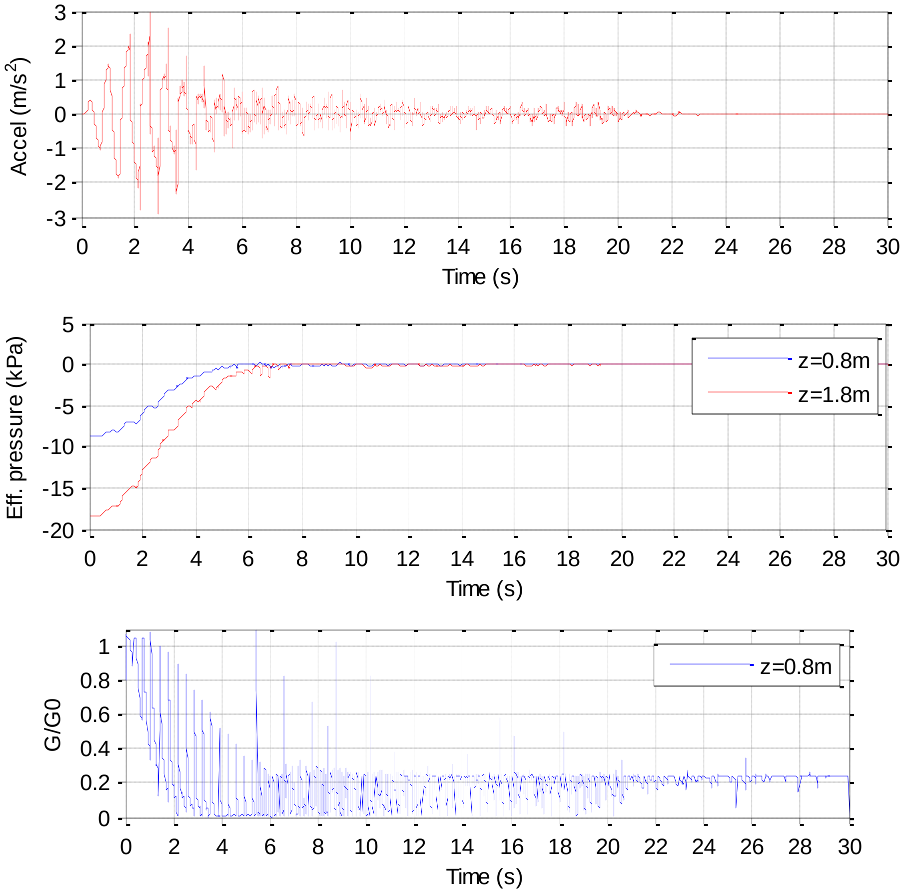

Figure 1 – Nonlinear OpenSeesPL free-field ground surface acceleration time history; effective confining pressure time histories in the top two metres of soil (compressive stress taken as negative); and shear modulus ratio (G/G0) time history at a soil depth of 0.8m (excitation frequency = 1.5Hz)

The maximum surface accelerations were found to occur in the initial loading stage during the buildup in excess pore water pressure. The ground surface accelerations at full liquefaction were significantly lower. During the initial loading stage, nonlinear OpenSeesPL analyses found the stiffness of the soil at a depth of 0.61m (i.e. one pile diameter) to be 20% of the initial stiffness at that depth. The distribution of stiffness with depth had changed from the initial square root distribution to a linear distribution. Therefore based on the OpenSeesPL analyses it is assumed that the maximum ground accelerations occur when the soil has a shear modulus equal to approximately 20% of the initial shear modulus. Figure 2 compares the initial shear modulus distribution with that obtained from the nonlinear OpenSeesPL analysis during the first four seconds of the input when the ground surface acceleration was largest.

Using a shear modulus equal to 20% of the initial modulus with a linear distribution and an associated damping value of 15% the resonant soil frequency and maximum ground surface acceleration were calculated using the 1-D elastic wave propagation method. Figure 3 shows the ground acceleration response calculated using the 1-D elastic wave propagation method compared with the nonlinear OpenSeesPL response. The resonant frequency (1.6Hz) and amplification from the 1-D elastic wave propagation method match the OpenSeesPL analyses reasonably well.

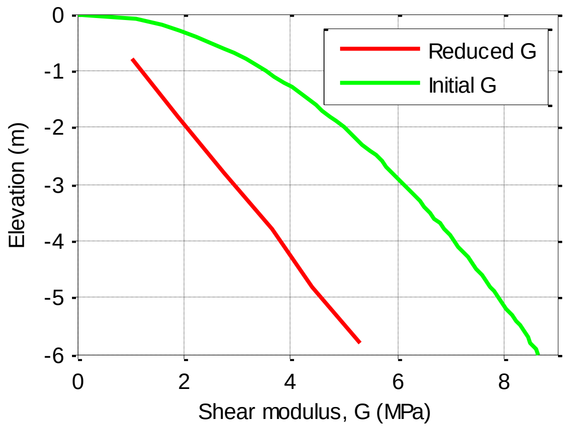

Figure 2 – Initial shear modulus distribution throughout top soil layer vs. shear modulus distribution during max loading in nonlinear OpenSeesPL analysis

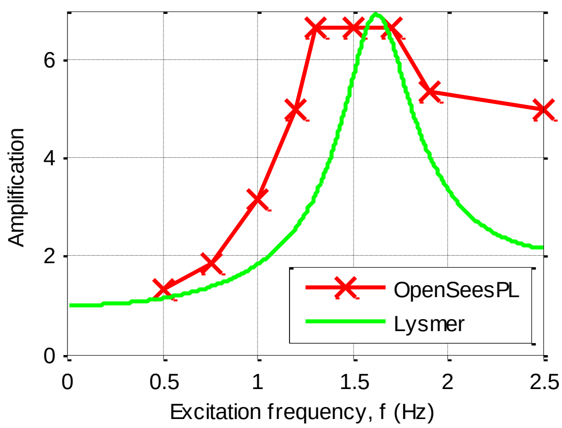

Figure 3 - Free-field ground surface acceleration amplification relative to bedrock acceleration from nonlinear OpenSeesPL analyses vs. elastic equivalent 1-D elastic wave propagation (Lysmer)

The OpenSeesPL amplification curve indicates that the natural frequency of the soil ranges between 1.3Hz to 1.7Hz. The authors do not fully understand why there is a plateau between 1.3Hz and 1.7Hz. However, one possible reason could be because the soil stiffness, and therefore natural frequency, is constantly decreasing during the initial transient response stage. The maximum amplification shown in Figure 3 occurs when the soil stiffness has degraded to a point where the natural frequency of the soil-pile system equals the loading frequency.

4 SOIL-PILE INTERACTION

We now consider the pile response when subject to sinusoidal base excitations.

Nonlinear OpenSeesPL analyses have been used to develop an elastic equivalent pseudo-static method for determining both the kinematic and inertial pile interaction. The pseudo-static method is based on the 1-D elastic wave propagation method outlined above; an approximate technique for evaluating the response of piles to inertial loading; and the elastic continuum pile analysis method given in Budhu and Davies (1987) for elastic soils. Budhu and Davies (1987) developed a simple set of equations for analysing laterally loads piles in homogeneous elastic sandy soil (Pender, 2012, Budhu and Davies, 1987).

An approximate technique for evaluating the inertial response of piles is described in Pender (2012) (referred to as the "inertial pseudo-static method" in this paper). In dynamic soil-pile interaction both the system stiffness and damping resist the dynamic loads. Therefore the technique includes calculating the soil-pile stiffness and damping matrices and combining these into an impedance matrix. By solving the impedance matrix the total equivalent system damping can be calculated (Wolf, 1985). The dynamic response amplification is calculated by solving the standard single degree of freedom (SDOF) equation (Pender, 2012).

Pile head acceleration amplification response curves due to inertial interaction calculated in OpenSeesPL (equal to the pile head acceleration divided by the free-field ground surface acceleration at each loading frequency) and using the inertial pseudo-static method are shown in Figure 4. For the inertial pseudo-static method, the equivalent elastic stiffness and strain dependent material damping of the soil was assumed to be the same as the free-field soil during the maximum ground acceleration stage (G/G0 = 0.2 and damping = 15%). The natural frequency of the soil layers above the bedrock was assumed to be equal to the resonant frequency from the 1-D elastic wave propagation method in Figure 3 (1.6Hz).

The pile natural frequency determined using the inertial pseudo-static method is approximately 1.9Hz. As this is greater than the soil natural frequency, it has been assumed that frequency dependent radiation damping is present at pile resonance. This may be the reason that there is not a sharp peak in Figure 4.

The inertial response curve calculated using these assumptions in the inertial pseudo-static method is generally in good agreement with the nonlinear OpenSeesPL curve.

Pile head accelerations are shown in Figure 5. The pseudo-static response curve has been calculated by multiplying the inertial pseudo-static response curve in Figure 4 with the free-field ground surface accelerations calculated using the 1-D elastic wave propagation method in Figure 3. The pseudo-static accelerations are in reasonable agreement with the OpenSeesPL accelerations. By referring to Figure 3, it can be seen that the soil and pile resonant frequencies are approximately equivalent.

The pseudo-static method using the assumptions outlined, gives reasonable agreement with the nonlinear OpenSeesPL analyses, below the resonant frequencies. For greater excitation frequencies it must be recognised that the soil will not be 'excited' as much. Therefore the stiffness degradation will be less and the soil-pile natural frequency greater. That is possibly why OpenSeesPL gives greater accelerations above the resonant frequencies.

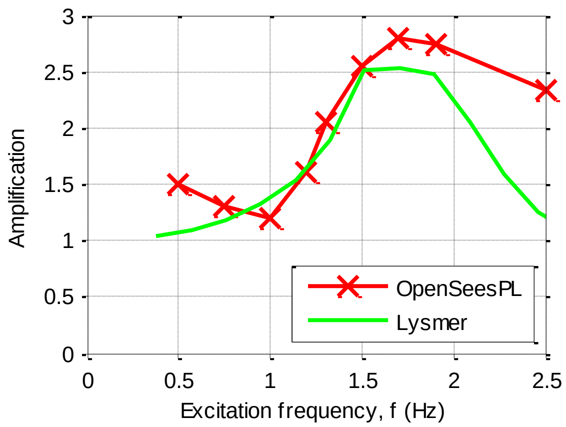

Figure 4 - Pile head acceleration amplification, in nonlinearly behaving soil, relative to free-field ground surface acceleration using OpenSeesPL and inertial pseudo-static method

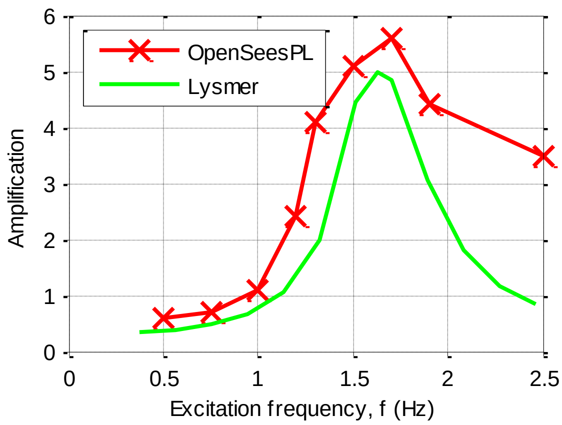

Figure 5 - Pile head acceleration, in nonlinearly behaving soil, calculated using OpenSeesPL and inertial pseudo-static method multiplied by the 1-D elastic wave propagation method

The maximum nonlinear OpenSeesPL pile displacement profiles for a pile with mass (inertial and kinematic interaction) and for a pile without mass (kinematic interaction only) are shown in Figure 6 for an excitation frequency of 1.5Hz (the largest pile displacements and moments occurred at this frequency). The pile head displacement calculated using the Budhu and Davies (1987) method for elastic soil relative to the kinematic pile displacement is also plotted.

OpenSeesPL accounted for nonlinear soil behaviour including build-up of excess pore water pressure during dynamic loading of the soil. Pile response was due to base shaking causing kinematic and inertial interactions between the soil and pile.

The displacement profile of the pile without mass ("OpenSeesPL (no mass)" in Figure 6) is approximately equivalent to the free-field displacement profile of the soil (not plotted). Pender (1993) describes an approximate technique of determining the pile head displacement relative to the free-field ground surface displacement due to kinematic interaction developed by Gazetas (1984), which is summarised in a kinematic interaction factor (Pender, 1993, Gazetas, 1984). By following this technique, the pile is supposed to displace with the soil (i.e. at unity if there was no inertial interaction), which is consistent with the nonlinear OpenSeesPL analyses.

The Budhu and Davies calculated displacements relative to the OpenSeesPL pile displacements without mass are within 10% of the nonlinear OpenSeesPL pile displacements with mass. This is a good match considering the nonlinear soil behaviour was accounted for in the Budhu and Davies method by using the equivalent elastic soil stiffness equal to 20% of the stiffness near the ground surface and assuming a linear stiffness distribution with depth. Also, a lateral load (calculated using the maximum pile head acceleration at resonant frequency - Figure 5) was applied to the pile head to represent the inertial loading due to base shaking.

The Budhu and Davies method is only applicable to piles embedded in homogeneous soils. In this case it was assumed that lateral pile response was confined to the top soil layer, which is equivalent to 10 pile diameters (6m thick). It is noted that Randolph (1981) found that pile deformations and bending moments were generally negligible after typically 10 pile diameters (Randolph, 1981).

Maximum bending moment profiles of the pile with mass (i.e. inertial plus kinematic loads) and without mass (i.e. kinematic loads only) are shown in Figure 7. The Budhu and Davies maximum pile shaft bending moment (also plotted in Figure 7) is 20% smaller than the OpenSeesPL maximum moment. However, if you add the moment due to kinematic loading to the Budhu and Davies moment, at the same depth, the resultant moment is within 10% of the maximum pile shaft bending moment.

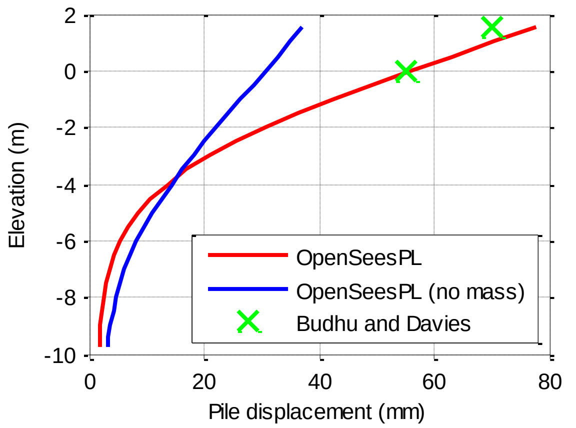

Figure 6 - Nonlinear OpenSeesPL maximum pile displacement profile due to kinematic and inertial interaction ("OpenSeesPL") vs. kinematic interaction only ("OpenSeesPL (no mass)") vs. maximum Budhu and Davies displacements relative to kinematic displacements

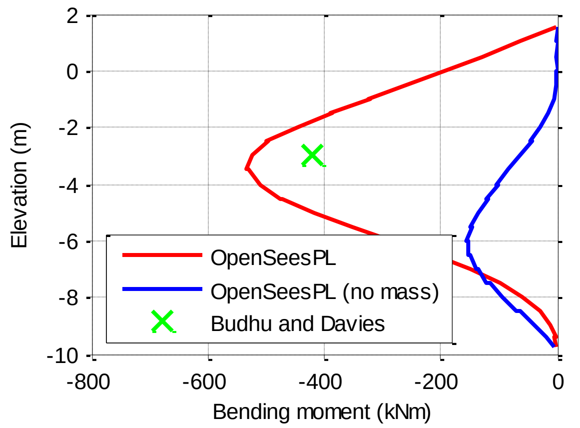

Figure 7 - Maximum pile bending profile for pile subject to kinematic plus inertial loading and subject only to kinematic loading ("OpenSeesPL (no mass)") vs. Budhu and Davies maximum moment

The addition of kinematic and inertial loads is in line with the findings of Tokimatsu et al. (2005) that these two loads are in phase if the natural frequency of the superstructure is greater than that of the ground (Tokimatsu et al., 2005). The natural frequency of pile is 1.9Hz and the natural frequency of ground is 1.6Hz (refer to Figures 3 and 4).

5 PSEUDO-STATIC METHOD FOR KINEMATIC AND INERTIAL INTERACTION

Finally, we outline a pseudo-static method for estimating single pile response.

An attempt to develop an elastic equivalent pseudo-static method to approximately check pile response in liquefiable soils subject to earthquakes has been developed. It is valid for piles embedded in flat ground with a relatively homogeneous soil layer over the top region of the pile (i.e. approximately 10 pile diameters) with no crust layer. The method has the following steps:

1. Evaluate soil stiffness
2. Carry out free-field site response analysis
3. Determine liquefaction potential

4. Estimate "liquefied" soil stiffness (if soil is prone to liquefaction, assume a linear soil stiffness distribution with depth and stiffness at one pile diameter equal to 20% of the "operational" soil stiffness).
5. Estimate strain dependent material damping (equal to approximately 15% to 20% for G/G0 = 0.2)
6. Carry out free-field site response analysis with elastic equivalent "liquefied" stiffness
7. Determine inertial pile response
8. Determine pile head acceleration
9. Calculate pile displacement and moment due to pile head acceleration
10. Estimate maximum pile displacement due to free-field ground displacement by determining the free-field ground displacement (in Step 6) and by determining the kinematic interaction factor (refer to technique outlined in Pender (1993)).
11. Determine pile displacement due to kinematic and inertial interaction
12. Determine maximum pile shaft moment due to kinematic and inertial interaction

This is only the first attempt to set up this method. Further work and research is required to develop the equivalent elastic pseudo-static method for checking pile response due to earthquake loading and refine simplified assumptions.

6 CONCLUSIONS

This paper has looked at the lateral response of piles embedded in flat saturated cohesionless soils, with no crust layer, subject to dynamic excitation of the underlying bedrock. Free-field site response and the effects of kinematic and inertial interaction between soil and pile have been investigated. The main conclusions that can be drawn from this chapter are:

- Maximum ground surface accelerations in liquefiable soils occurs before the onset of full liquefaction. Ground accelerations reduce significantly as the soil approaches full liquefaction.
- When the soil is at full liquefaction, the soil stiffness is in the order of 2% to 10% of the initial stiffness. However, during the transient period where maximum ground surface accelerations occur, the soil stiffness is approximately 20% of the initial stiffness near the surface.
- During the transient period, the distribution of stiffness with depth changes from square root to linear.
- The resonant soil frequency and maximum ground surface accelerations could be approximated using the 1-D elastic wave propagation method of John Lysmer. Equivalent elastic soil properties equal to 20% of initial soil stiffness near the ground surface and a linear stiffness distribution with depth was used.
- Pile head accelerations due to kinematic and inertial interaction can be approximated by multiplying the maximum ground surface accelerations found using the 1-D elastic wave propagation method of Lysmer with the inertial pile head amplification calculated using the inertial pseudo-static method.
- The Budhu and Davies (1987) elastic continuum method for elastic soil can predict pile response due to inertial loading resulting from dynamic loading of the bedrock (that is the displacement relative to the kinematic displacement profile). A lateral pile head load is calculated using the peak pile head acceleration and pile head mass. "Liquefied" nonlinear soil behaviour is represented by an equivalent elastic stiffness equal to 20% of initial soil stiffness near the ground surface and a linear stiffness distribution with depth.

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