Displacement Modification Factors for Seismic Design considering Foundation Deformations

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2015 NZSEE Conference

ABSTRACT: It is a requirement of performance-based design to have an estimate of the expected level of deformation of the building. For many years this estimation has ignored the influence of soil-foundation-structure interaction (SFSI), however it is widely accepted that SFSI modifies the displacements of the building. Advanced tools exist to simultaneously model the soil, foundation and structure using a non-linear time-history analysis that can provide reasonable estimates of the deformations throughout the system. However these numerical tools require considerable knowledge of soil and structural dynamics and results are strongly impacted by the characteristics of the ground motion time history. The alternative approach of determining the in-elastic displacements based on the response of an equivalent linear structure is still widely adopted and forms the basis of many seismic design codes.

Design procedures that adopt equivalent linear properties require the engineer to quantify both the effective response period and a displacement modification factor often presented in the form of an equivalent viscous damping. The two properties are interdependent and by no means intrinsic to the building. Equivalent viscous damping requires calibration based on the assumptions of the effective response period to match some behavioural aspects of the linear system. The Direct Displacement-Based Design procedure defines the response period based on the secant stiffness to design displacement and calibrates the equivalent viscous damping of the equivalent linear system to match the non-linear displacement. Existing curves quantify the foundation hysteretic energy dissipation in terms of an equivalent viscous damping, based on pseudo static cyclic loading of foundations. These curves, however, use an equal viscous-to-hysteretic energy dissipation assumption based on cyclic loading to peak displacement and do not attempt to account for the erratic dynamic loading from earthquakes. This paper provides an overview of the displacement-based approach to design buildings considering soil-foundation-structure interaction. The secant (to design displacement) stiffness assumption with equivalent viscous damping is discussed with reference to alternative considerations of equivalent linear analysis methods. New foundation displacement modification factors are proposed based on non-linear time history analysis results using an elastic single-degree-of-freedom structure with a non-linear macro soil-foundation interface element.

1 INTRODUCTION

It is widely accepted that foundation deformations or “soil-foundation-structure interactions” (SFSI) modify the displacements of the structure. The quantification of this modification has been attempted by many researchers (e.g. Veletsos and Verbic 1974; Nakhaei and Ali Ghannad 2008; Moghaddasi et al. 2011). This existing literature has highlighted a series of important parameters that influence the level of modification, namely structure-to-soil stiffness ratio, structure height-to-foundation length ratio and building period-to-ground motion predominant period. However design orientated expressions to estimate displacements are not provided and are critically needed.

A performance-based design methodology, as adopted in many seismic codes around the world, imposes limits on deformations and displacements, therefore there is a need to quantify the modification of displacements due to SFSI in the context of design. The prediction of the inelastic displacements of a structure can employ a full non-linear model of the structure and soil system or the more pragmatic approach where the system is converted to an equivalent linear single-degree-of-freedom (SDOF) system and the linear response spectrum can be utilised. While the full modeling of the structure and soil provides a robust and rational approach to the prediction of displacements, it is hindered by the requirement that the design must be complete to formulate the numerical model, thus resulting in a time-consuming iterative process. The full modeling is further hampered by several non-trivial decisions, such as ground motion selection, level of model complexity and material physical properties. Simple equivalent linearisation techniques therefore remain a cornerstone in in-elastic design, even in the face of ever growing computational power.

The linearisation of a non-linear system requires an estimate of an effective vibrational period and an estimate of a displacement modification factor often provided as an equivalent viscous damping. Values of 3-5% are often considered suitable levels of viscous damping to account for small cycle hysteresis and foundation energy dissipation through kinematic interaction of the soil and foundation. While equivalent viscous damping (EVD) has also been used to account for non-linear behaviour and hysteretic energy dissipation. Many linearisation procedures rely on a conversion from hysteretic energy dissipation to viscous energy dissipation. Equivalent linear site response analysis tools use EVD and a degraded stiffness to model the hysteretic behaviour of the soil. The EVD is based on an equal energy dissipation assumption, assuming resonant cycles often to 65% of the peak response with a period based on the secant stiffness to 65% of peak response.

The equivalent linearisation of structures uses a similar approach. Jacobsen (1960) pioneered the development of an equivalent structure concept, whereby a non-linear structure could be approximated by a linear structure with additional damping. Jacobsen’s equivalent structure used the initial stiffness and matched the viscous energy dissipation to that of the hysteretic energy dissipation, with the requirement that the structure would be at steady-state resonance. Although this procedure gives a good approximation to the displacements for structural periods equal or greater than the excitation period (Dwairi et al. 2007), the additional requirements of steady-state resonance do not make it fully applicable to earthquake loadings. The Direct Displacement-based Design (DDBD, Priestley et al., 2007) procedure puts considerable emphasis on quantifying hysteretic energy dissipation as an equivalent viscous damping. DDBD uses a period shift based on the secant-to-peak stiffness and an adjusted area based damping that is calibrated on non-linear time history results.

It is important to understand the differences and implications of different linearisation methods and the variations in the levels of damping. It is clear that EVD is inherently linked to the assumption of the period shift and the parameter which the equivalent model attempts to match (Jennings, P.C. 1968). In design it is imperative to use the correct EVD for the assumed period shift and the intended matching parameter. Figure 1 demonstrates three different period shift assumptions and the values of damping that would be obtained if the equivalent linear response matched the non-linear displacement response. However the application of these damping levels and equivalent periods would result in poor estimates of accelerations.

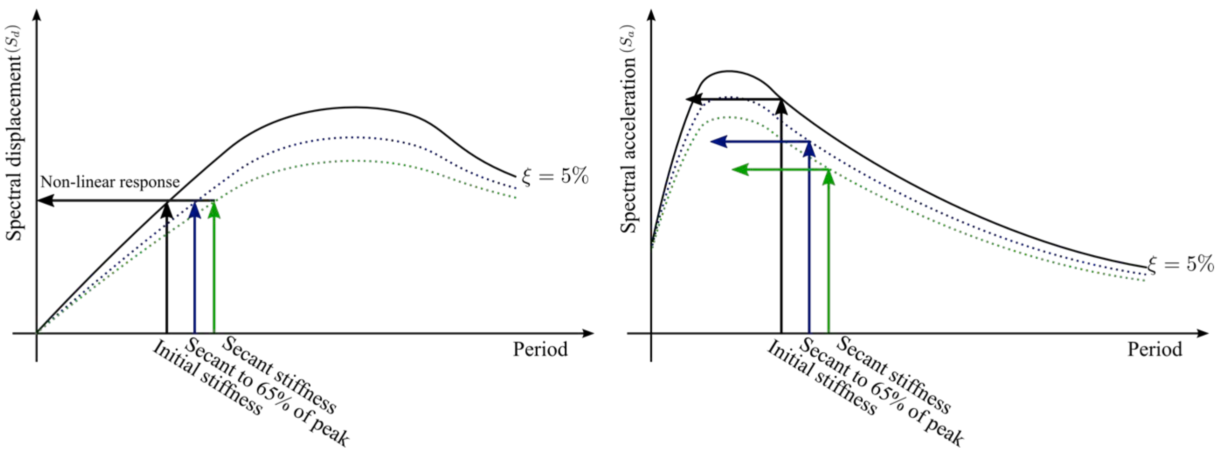

Figure 1. Dependence of period shift on equivalent viscous damping.

Recent developments in DDBD have bypassed the step of estimating the EVD, due to the imperfect correlation with area-based hysteretic energy and the variation of sensitivity of spectral displacement to viscous damping between ground motions (Pennucci et al. 2011). Pennucci et al. (2011) presents Displacement Reduction Factors (referred to here as Displacement Modification Factors, DMF) for different hysteresis models and demonstrates the agreement with earlier EVD estimates. Importantly, by removing the need for estimating the reduction in spectral displacement due to viscous damping, Pennucci et al. (2011) demonstrates that conflicting EVD expressions from Priestley and Grant (2005) and Dwairi et al. (2007) that used different spectral reduction formulations and ground motions, result in indistinguishable DMFs. Pennucci et al. (2011) concludes that while the EVD is dependant on the damping sensitivity of the ground motions, the inelastic DMF is not.

The use of DMFs further alleviates the issue of whether to average the response displacements (Dwairi et al. 2007) of a series of ground motion records or the EVD values (Priestley and Grant 2005), which are co-related through a non-linear expression.

Equal energy/equivalent area based viscous damping equations exist for foundation rotation based on experimental data from cyclic loading tests (Paolucci et al. 2009) and from finite element cyclic loading tests (Adamidis et al. 2013), which provide a provide a useful point of reference to gauge the level of “damping” from hysteretic behaviour. Both studies also recognised the axial load ratio ($\widetilde{N}$), defined as the ratio of axial load to cause bearing capacity failure (N_{max}) and the applied axial load (N), as an important parameter in the variation of energy dissipation. To make a more rational estimate of the displacements of a non-linear system there is a need for DMFs to be calibrated against a series of non-linear time-history results for a given period shift.

1.1 Displacement modification considerations in direct displacement-based design

The fundamental steps from Paolucci et al. (2013) for a DDBD considering dynamic non-linear soil-foundation-structure interaction are shown in Figure 2, where the design loads for a non-linear, multiple degree-of-freedom structure are determined based on an equivalent linear SDOF. The first step uses the displaced shape of the structure-foundation-soil system (Δ_i) based on a first mode response at peak design response and mass distribution (m_i), to determine an equivalent SDOF mass (m_e), effective height (H_e) and equivalent design displacement (Δ_d) using Equations (1-3). The non-linear behaviour is converted to an equivalent linear behaviour in steps two and three, where the secant-to-peak stiffness is used as the effective stiffness and the EVD is determined based on a weighted average of the displacement contributions from the foundation and superstructure and the hysteretic behaviour (Equation 4). The effective period (T_{eff}) is determined based of the interception of the design displacement with the reduced spectral displacement, where the spectral displacement is reduced based on the EVD (ξ_{sys}) or the DMF (η) could be used directly as suggested by Pennucci et al. (2011). The final step is to compute the base shear from the effective stiffness (K_{eff}) and the design displacement (Equations 6-7).

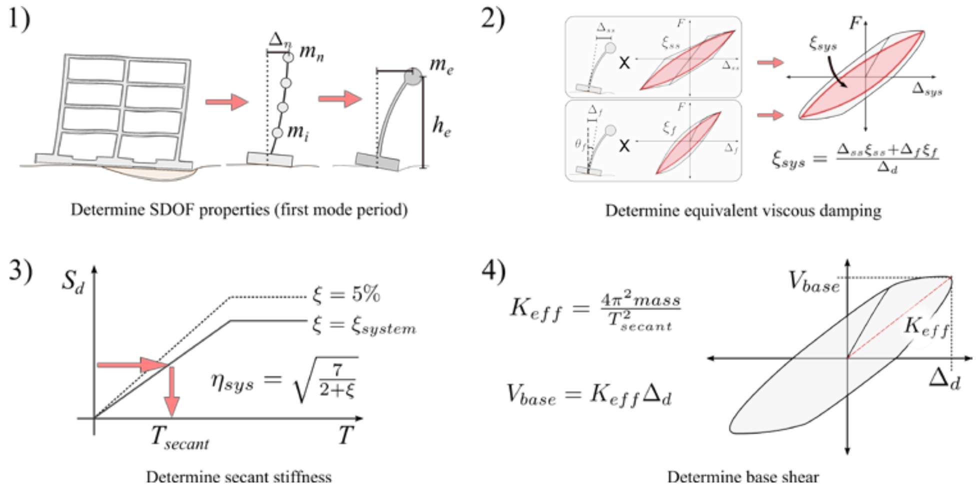

Figure 2. Direct displacement based design procedure considering non-linear SFSI
after Paolucci et al. (2013)

$$\Delta_d = \frac{\sum_{i=1}^{n} m_i \Delta_i^2}{\sum_{i=1}^{n} m_i \Delta_i} \tag{1}$$

$$m_e = \frac{\sum_{i=1}^{n} m_i \Delta_i}{\sum_{i=1}^{n} \Delta_d} \tag{2}$$

$$h_e = \frac{\sum_{i=1}^{n} m_i \Delta_i h_i}{\sum_{i=1}^{n} m_i \Delta_i} \tag{3}$$

$$\xi_{sys} = \frac{\Delta_{ss}\xi_{ss} + \Delta_f \xi_f}{\Delta_d} \tag{4}$$

$$\eta = \sqrt{\frac{7}{2 + \xi_{sys}}} = \frac{\Delta_d}{Sd_{\xi=5\%}(T_{eff})} \tag{5}$$

$$K_{eff} = \frac{4\pi^2 m_e}{T_e^2} \tag{6}$$

$$V_{base} = K_{eff}\Delta_d \tag{7}$$

The majority of EVD and DMF equations proposed for use in DDBD have been calibrated against time history analysis. To maintain consistency with other works, a similar calibration process to that used by Pennucci et al. (2011) was adopted.

2 METHODOLOGY

This paper presents only the relationships for the displacement modification factors, DMF, for the secant-to-peak effective period, while the EVD could be determined based on the inversion of Equation 5. The relationships were based on results from an extensive parametric study using an experimentally validated modeling technique to mimic the structure-foundation-soil system and was subjected to 40 ground motions.

The system non-linear DMF (η_{sys}) for each building was determined by the ratio of the average non-linear displacement of the 40 records and the average of the spectral displacement at the secant-to-peak period (Equation 8).

$$\eta_{sys} = \frac{\overline{\Delta_{NL}}}{Sd_{\xi=5\%,T_e}} \tag{8}$$

It was essential to model the super-structure as a flexible system since the flexibility of the super-structure contributes significantly to the dynamic response of the system and would therefore apply more realistic loading to the foundation. Having both a superstructure displacement component and a foundation displacement component creates a further complication to determining EVD or DMF expressions for DDBD, compared to previous studies. While the total displacement of the system should match the equivalent linear SDOF, the foundation DMF only applies to the displacements from the foundation. If the foundation displacement contribution is small, then the influence of the non-linear deformations and hysteretic energy dissipation is small. The foundation DMF (η_f) was back-calculated using Equation 9 which was derived from Equations 4 and 5 to maintain consistency with the displacement weighted EVD formulation. To avoid the use of EVD in the DDBD procedure suggested by Paolucci et al. (2013), Equation 10 could be used based the design displacement being equal to the system displacement (Δ_{sys}) and directly using the foundation DMF (η_f) and displacement (Δ_f) and superstructure DMF (η_{ss}) and displacement (Δ_{ss}). For the calibration of the foundation DMF the super-structure was modelled elastically with 5% damping so the DMF was equal to 1.

$$\eta_f = \sqrt{\frac{\Delta_f}{\frac{\Delta_{sys}}{\eta_{sys}^2} - \frac{\Delta_{ss}}{\eta_{ss}^2}}} \tag{9}$$

$$\eta_{sys} = \sqrt{\frac{\Delta_{sys}}{\frac{\Delta_{ss}}{\eta_{ss}^2} + \frac{\Delta_f}{\eta_f^2}}} \tag{10}$$

Due to the non-linear relationship between the foundation displacement contribution and the influence on the displacement modification factor, only results where the foundation contributed 40% or greater to the displacement were included in the regression analysis. The removal of some of the dataset meant that small variations in displacements were not attributed to a large displacement modification factor. Due to the removal, foundation displacement modification factors may be less accurate when used in situations where the foundation contributes less than 40% to the displacement, however in these situations the foundation DMF makes very little difference to the predicted system displacement.

2.1 Numerical model

The numerical model used in this study consisted of a lumped mass super-structure (M_{ss}) attached to a soil-foundation interface element (Figure 3). The super-structure was modelled elastically with a horizontal linear dashpot (C_{ss}) set at 5% of critical damping between the foundation and the super-structure. The vertical displacement from the super-structure was slaved to the foundation node providing a perfectly rigid super-structure axial stiffness. The foundation mass was not modelled since the DMFs are for use with a SDOF response spectra. The foundation radiation damping was modelled with horizontal (C_{VV}), vertical (C_{NN}) and rotational (C_{MM}), linear dashpots between the foundation and surrounding soil based on the radiation damping equations from Gazetas (1991). The initial stiffnesses (K_{NN}, K_{VV}, K_{MM}) for the soil-foundation element model were based on the stiffnesses suggested in Gazetas (1991), where for embedded foundations the contact area of the sidewalls was assumed to be zero as the numerical model was developed for shallow foundations on the surface. The modification to the stiffnesses due to uplift was captured using the uplift formulation from Chatzigogos et al. (2011). The uplift model has the advantage of capturing the vertical displacement of the centre of the footing, allowing for the vertical inertia and damping to contribute to the behaviour. The soil yielding was modelled using the plasticity model and model parameters from Figini et al. (2012). The plasticity model uses a bounding surface and vertical mapping rule and it has been experimentally validated to reasonably accurately capture the rotational and settlement behaviour (Figini et al. 2012, Millen et al. 2015). The input parameters for the uplift and plasticity models are summarised in Table 1 and Figure 4 shows the foundation rotational stiffness degradation curves that the macro-element produces, where the curves for different axial load ratios are indistinguishable when normalised by the pseudo uplift angle. The pseudo uplift angle given in Equation 11 is not the actually uplift angle of the foundation but the ratio between the moment required to cause foundation uplift and the elastic rotational stiffness, thus providing a link between the foundation stiffness and strength and the applied axial load (N). P-delta effects were not considered in the analyses, since the P-delta effects are considered separately in DDBD and the influence of P-delta forces depends on structural geometry which cannot be captured by the simple displacement modification factors suggested here. All analyses were carried out using the non-linear time history analysis software Ruaumoko3D (Carr 2014).

$$\theta_{uplift} = \frac{M_{uplift}}{K_{mm,elastic}} \tag{11}$$

Table 1. Plasticity model and uplift model parameter values.

Uplift limit factor (α)	5
Uplift stiffness factor (ε)	0.65
Uplift stiffness factor (δ)	0.75
Uplift stiffness factor (γ)	1.5
Uplift stiffness factor (ζ)	1.5
Bounding surface shear factor (μ_{BS})	tan(φ)
Bounding surface moment factor (ψ_{BS})	0.48
Bounding surface shape factor (ξ_{BS})	0.95
Plasticity modulus factor (p_1)	0.4
Reload stiffness factor (p_2)	1.0
Plastic potential shear factor (λ_{BS})	2.5
Plastic potential moment factor (χ_{BS})	3.0

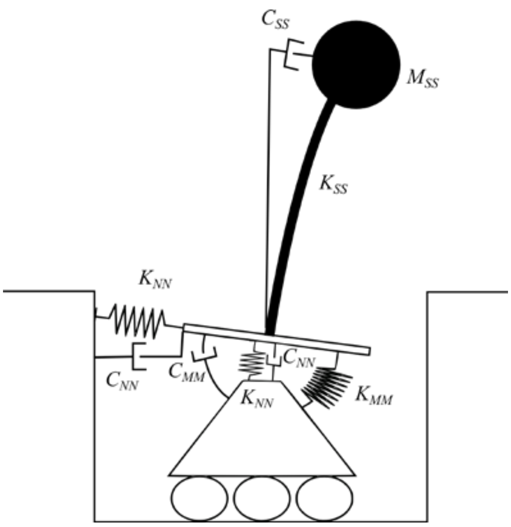

Figure 3 Numerical model setup

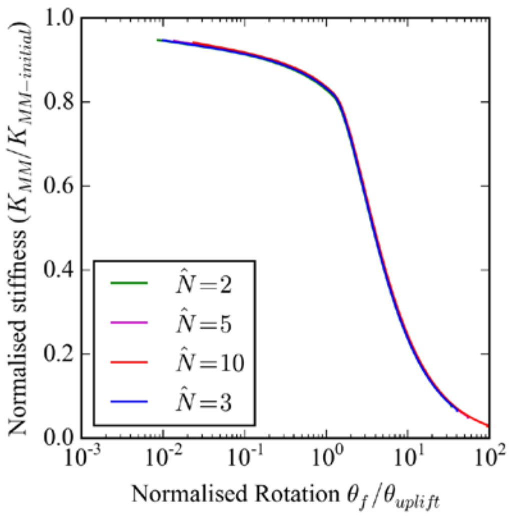

Figure 4 Foundation stiffness degradation curves

2.2 Inputs for parametric study

The parametric study used super-structure and soil parameters that were varied to reflect a range of realistic building and soil types where SFSI may be of interest. Table 2 describes the range and limitations on the parameters, while Equation 12 provides a limit on the length of foundation perpendicular to axis of foundation rotation (L) based on the expected over-turning moment, foundation shear and the foundation capacity from work by Nova and Montrasio (1991). The foundations were sized so that the expected moment demand would be approximately the moment capacity, while the $\pm$ 0.5 term provided a random variation to the foundation length in an attempt to remove any bias on the results from the imposed limitation.

Table 2. Range on input parameters for parametric study.

Parameter	Range
Shear wave velocity (V_S)	$100 \le V_S \le 360$ *m/s*
Soil mass density (ρ_S)	$1.6 \le \rho_S \le 1.9$ t/m^3
Poisson's ratio (v)	v = [0.2, 0.3]
Soil internal friction angle (φ)	$30 \le \varphi \le 40$
Effective height (H_e)	$\max(2, 9.1T_{ss}^{1.33}) \le H_e \le \min(20, 26.8T_{ss}^{1.33})$ m
Design hazard factor (Z)	Z = 0.4 for use in NZS 1170.5:2004
Spectral acceleration (S_a)	$S_a = f(T_{ss}, Z, N = 1, R = 1)$ from NZS 1170.5:2004
Axial load ratio ($\widehat{N}$)	$\widehat{N}$= [2, 3, 5, 10]
Length of foundation perpendicular to the axis of rotation (L)	$\max(2, H/5) \le L = Eq.1 \le \min(20, H/2)(m)$
Length of foundation parallel to the axis of rotation (W)	$0.33H \le W \le 3H(m)$
Embedment of foundation (D)	D = [0, 0.2L] (m)
Axial load capacity (N_{max})	$N_{max} = f(\varphi, L, W, D)$ Salgado (2008)
Vertical weight (N)	$N = N_{max}/(\widehat{N})$
Seismic mass (M_{SS})	$M_{ss} = N/g$
Structural stiffness (K_{SS})	$K_{ss} = 4\pi^2 M_{ss}/T_{ss}^2$

$$L = \frac{3\, S_a(T)\, H_e(1 \pm 0.5)}{\sqrt{\left(1 - \frac{1}{\widehat{N}}\right)^2 - \left(\frac{3\, S_a(T)}{4 \tan \phi}\right)^2}} \qquad (12)$$

2.3 Ground motion selection

The calibration procedure used 40, carefully selected earthquake records all recorded at least 20km from the epicentre (Table 3, Figure 4). In contrast to Dwairi et al. (2007), where a random set of earthquake records were used, here the records were scaled and selected based on their normalised least squared variation from the design spectrum between periods of 0.4s and 5.0s for site class C. The motions were scaled to spectra with a Z factor of 0.3 (i.e. PGA value according to NZS1170.5: 2004 approach) to provide large foundation rotation values and scaled to Z=0.15 to obtain lower values of foundation rotation.

Table 3. Ground motion parameters.

ID	Record Name	Epi dist [km]	Mw	Scale factor	Vs30 [m/s]	Arias I [m/s]	PGA [g]	Earthquake	Year	Station
1	NGA0176_1	36	6.5	2.56	250	0.26	0.12	Imperial Valley-06	1979	El Centro Array #13
2	NGA0143_1	55.2	7.4	0.42	767	11.52	0.84	Tabas, Iran	1978	Tabas
3	NGA0985_1	28.2	6.7	1.76	297	0.67	0.24	Northridge-01	1994	LA - Baldwin Hills
4	NGA0457_1	38.2	6.2	2.72	350	0.34	0.19	Morgan Hill	1984	Gilroy Array #3
5	NGA3276_1	69.5	6.3	1.9	212	0.32	0.15	Chi-Chi, Taiwan-06	1999	CHY037
6	NGA0183_1	28.1	6.5	0.74	206	1.59	0.6	Imperial Valley-06	1979	El Centro Array #8
7	NGA1208_2	55.2	7.6	1.44	442	0.76	0.18	Chi-Chi, Taiwan	1999	CHY046
8	NGA0175_1	32	6.5	1.76	197	0.38	0.14	Imperial Valley-06	1979	El Centro Array #12
9	NGA3266_1	61.5	6.3	2.86	226	0.11	0.12	Chi-Chi, Taiwan-06	1999	CHY026
10	NGA0175_2	32	6.5	2.12	197	0.33	0.12	Imperial Valley-06	1979	El Centro Array #12
11	NGA1484_1	78.4	7.6	1.14	273	1.14	0.25	Chi-Chi, Taiwan	1999	TCU042
12	NGA0172_1	35.2	6.5	2.68	237	0.27	0.14	Imperial Valley-06	1979	El Centro Array #1
13	NGA0172_2	35.2	6.5	3.8	237	0.21	0.13	Imperial Valley-06	1979	El Centro Array #1
14	NGA1495_1	35.9	7.6	0.94	273	1.7	0.24	Chi-Chi, Taiwan	1999	TCU055
15	NGA0178_1	28.7	6.5	0.84	163	1.13	0.27	Imperial Valley-06	1979	El Centro Array #3
16	NGA0186_1	68.9	6.5	3.34	208	0.18	0.11	Imperial Valley-06	1979	Niland Fire Station
17	NGA1153_1	171.4	7.5	3.68	275	0.1	0.1	Kocaeli, Turkey	1999	Botas
18	NGA1493_1	41.2	7.6	1.28	455	0.94	0.22	Chi-Chi, Taiwan	1999	TCU053
19	NGA1000_1	31.7	6.7	2.72	270	0.19	0.1	Northridge-01	1994	LA - Pico & Sentous
20	NGA0176_2	36	6.5	2.92	250	0.26	0.14	Imperial Valley-06	1979	El Centro Array #13
21	NGA0767_2	31.4	6.9	0.84	350	1.35	0.37	Loma Prieta	1989	Gilroy Array #3
22	NGA1762_2	48	7.1	1.58	271	0.68	0.15	Hector Mine	1999	Amboy
23	NGA2646_2	41.7	6.2	3.82	474	0.12	0.11	Chi-Chi, Taiwan-03	1999	TCU109
24	NGA1236_1	68.8	7.6	1.72	273	0.7	0.14	Chi-Chi, Taiwan	1999	CHY088
25	NGA0882_1	32.3	7.3	2.32	345	0.64	0.14	Landers	1992	North Palm Springs
26	NGA0970_1	50.8	6.7	2.5	309	0.23	0.12	Northridge-01	1994	El Monte - Fairview Av
27	NGA3275_2	61.5	6.3	1.32	233	0.51	0.2	Chi-Chi, Taiwan-06	1999	CHY036
28	NGA2715_1	39.9	6.2	2.48	273	0.24	0.14	Chi-Chi, Taiwan-04	1999	CHY047
29	NGA0832_2	75.2	7.3	1.56	271	0.75	0.15	Landers	1992	Amboy
30	NGA3269_1	56.8	6.3	2.52	545	0.11	0.14	Chi-Chi, Taiwan-06	1999	CHY029
31	NGA1636_2	84	7.4	2.36	275	0.42	0.13	Manjil, Iran	1990	Qazvin
32	NGA0292_1	30.4	6.9	0.88	1000	1.19	0.25	Irpinia, Italy-01	1980	Sturno
33	NGA1633_2	40.4	7.4	0.58	724	7.57	0.5	Manjil, Iran	1990	Abbar
34	NGA0726_1	26.5	6.5	3.4	191	0.18	0.12	Superstition Hills-02	1987	Salton Sea Wildlife Refuge
35	NGA0836_1	123.9	7.3	3.58	271	0.24	0.11	Landers	1992	Baker Fire Station
36	NGA0138_1	74.7	7.4	2.56	339	0.28	0.11	Tabas, Iran	1978	Boshrooyeh
37	NGA1026_2	39.3	6.7	2.92	361	0.23	0.15	Northridge-01	1994	Lawndale - Osage Ave
38	NGA1481_2	73.1	7.6	1.32	273	1.03	0.17	Chi-Chi, Taiwan	1999	TCU038
39	NGA2694_2	50.4	6.2	2.64	229	0.15	0.1	Chi-Chi, Taiwan-04	1999	CHY015
40	NGA0015_2	43.5	7.4	2.14	385	0.59	0.18	Kern County	1952	Taft Lincoln School

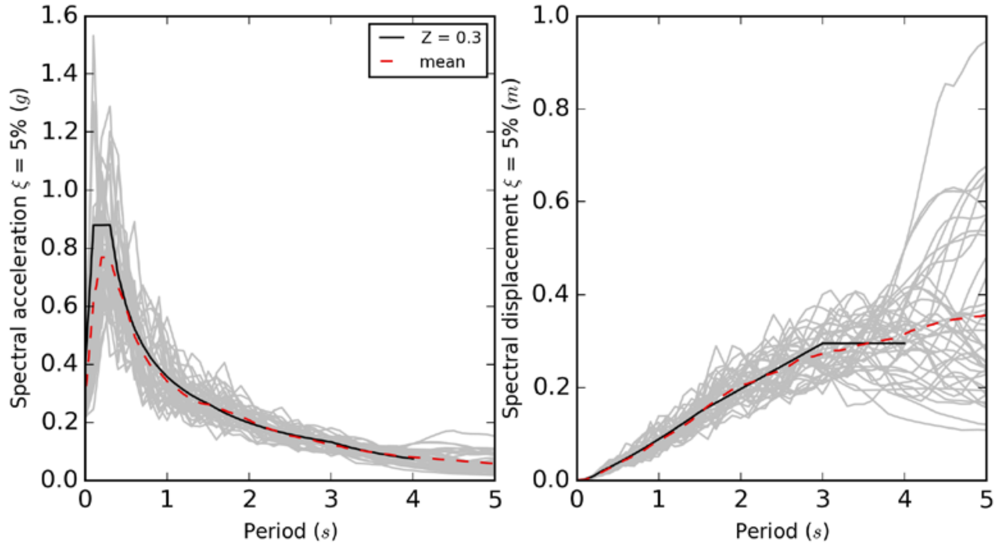

Figure 5. Acceleration and Displacement Response spectra of selected ground motions.

3 RESULTS

Figure 6 shows the DMF factors attained for each of the randomly generated buildings and a fitted expression for each level of axial load ratio. The pseudo uplift rotation parameter was used to normalise the peak rotation and thus provide a link between foundation stiffness and strength. The expressions provide reasonable estimates of the expected response however they cannot capture an apparent magnification in the response at normalised rotation values close to 1.0, due to the physical requirement that at zero rotation the DMF factor should be 1.

$$\eta = \sqrt{\frac{1}{1 + 2.5\,(1 - e^{0.05\theta_{norm}})}} \qquad \hat{N} = 2 \qquad (13)$$

$$\eta = \sqrt{\frac{1}{1 + 1.5\,(1 - e^{0.06\theta_{norm}})}} \qquad \hat{N} = 3 \qquad (14)$$

$$\eta = \sqrt{\frac{1}{1 + 2.0\,(1 - e^{0.06\theta_{norm}})}} \qquad \hat{N} = 5 \qquad (15)$$

$$\eta = \sqrt{\frac{1}{1 + 3.0\,(1 - e^{0.03\theta_{norm}})}} \qquad \hat{N} = 10 \qquad (16)$$

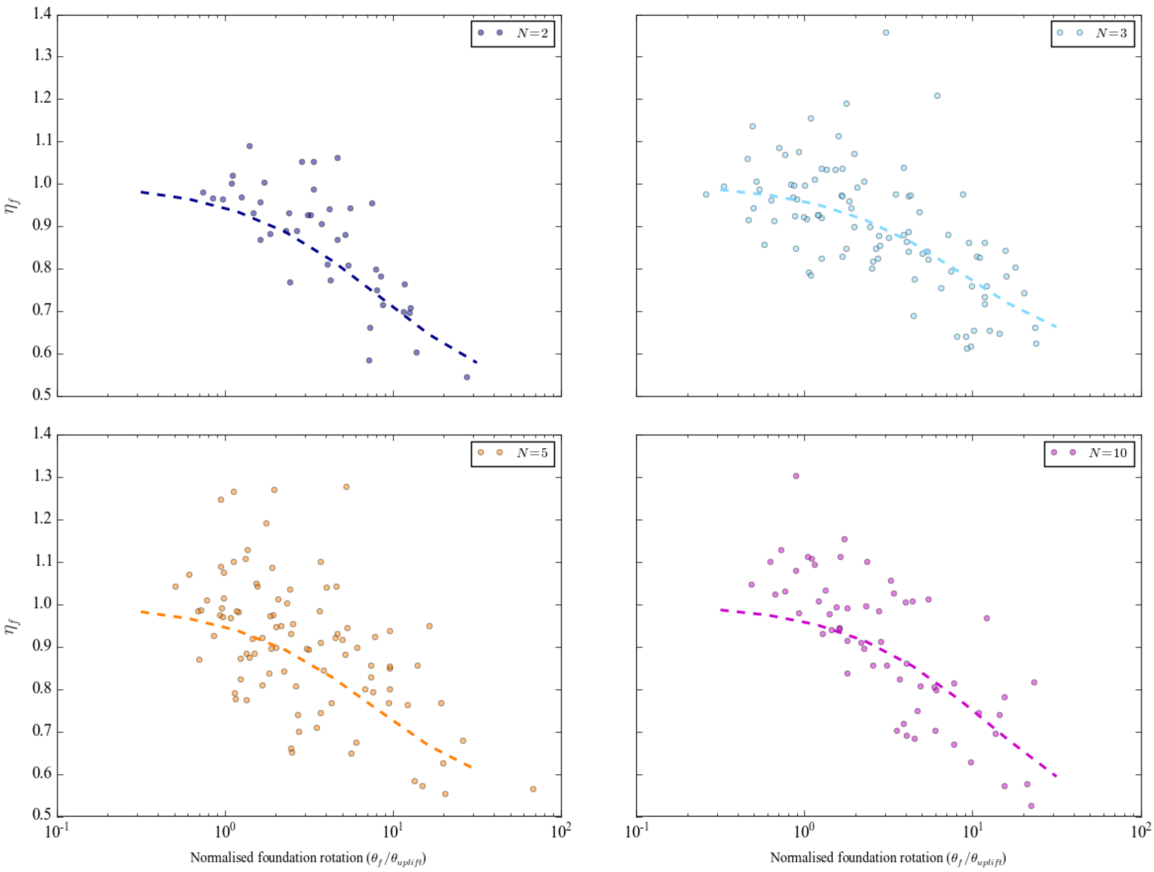

Figure 6. Expressions of Displacement Modification factor, DMF or ηf, vs. normalized foundation rotations derived from parametric study.

4 CONCLUSIONS

The estimation of building displacements is a critical step in performance-based design and the effects of SFSI need to be considered in this estimation. This paper explains the issues with using existing tools for assessing the effects of SFSI on building displacements. A series or relationships between the foundation rotation and the modification to building displacements due to SFSI were developed for specific use in displacement-based design procedures.

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