Integrating soil-structure interaction within performance-based design

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ABSTRACT: The uptake of a performance-based design methodology requires consideration of not just the performance of the superstructure, but the supporting soil and foundation as well. Case studies throughout history (eg. Kobe, Kocaeli & Christchurch earthquakes) demonstrate that a poor performance at the foundation level can result in a full demolition of the structure. For designers to have confidence that their design satisfies the given performance levels, they must first understand how soil-foundation-structure interaction affects the performance and secondly have tools available to adequately account for it in their design.

This paper provides an overview of the effects and mechanisms of soil-foundation-structure interaction especially in relation to the non-linear effects. Following this a performance-based design framework is presented which addresses the discussed effects and is supported with a design example of a six storey building.

1 INTRODUCTION

The earthquake engineering profession is moving towards low damage building designs, however research shows that even when a superstructure is designed to be low damage, a failure at the foundation could still render the building irreparable. To maintain consistency in the design of buildings, there is a need to have an integrated superstructure - foundation design methodology which ensures that overall building performance levels are met as well as superstructure and foundation performance levels.

There are now over fifty years of research into the effects of soil-foundation-structure interaction (SFSI). While the common moot of whether it is beneficial or detrimental is still debated, there have been considerable advancements in the understanding of SFSI effects. A series of earthquakes (eg. Kobe, Kocaeli & Christchurch earthquakes) have demonstrated the importance of SFSI with the full demolition of buildings due to poor foundation performance. Several extensive experimental programs have given great insight into the mechanisms involved with SFSI (e.g. Gajan et al. 2005). Early numerical and analytical studies using linear elastic analysis showed that the increased damping and increased flexibility from rocking and sliding of the foundation caused a modification to behaviour with the overall effect being dictated by the frequency content of the earthquake record. There have been several studies into linear SFSI with non-linear structures, such as Comartin et al. (2000), which demonstrated that ignorance of SFSI can result in the wrong part of the structure being retro-fitted. Studies by Nakhaei and Ali Ghannad (2008) showed that by modelling SFSI the structure will generally suffer more damage compared to a fixed based equivalent when the super-structure period is less than the predominant period of the record and vice-versa. This research also concluded that SFSI effects are more prominent for slender structures due to the larger elongation of the natural period.

The development of lumped-plasticity soil-foundation interface models (non-linear Winkler beam and macro-element models) has allowed the consideration of non-linear mechanisms at the foundation level such as up-lift, soil yielding and sliding that can provide reliable energy dissipation mechanisms. The successful design of structures with energy dissipation at the foundation level has been demonstrated in centrifuge tests (eg. Deng et al. 2012) and in practice with the Rion-Antrion Bridge (Pecker 2011) by changing the hierarchy of strength. Formal design procedures accounting for SFSI have been suggested by several authors, all employing the Direct Displacement-based Design (DDBD) procedure (eg. Sullivan 2010, Paolucci 2013). These procedures focus on achieving a particular design drift for mainly single-degree-of-freedom (SDOF) structures and have little guidance on controlling settlements and the change in foundation behaviour due to frame-action. The Model Code for DDBD (Sullivan, 2012) suggests designers should limit the degradation in soil stiffness to minimise residual displacements, however, there is no guidance on how to compute such a number.

For engineers to be satisfied that their designs meet the given performance levels, they must first of all be able to quantify the effects of the superstructure on the foundation performance and vice-versa into the non-linear range. Secondly they need to have tools available to adequately account for it in their designs. This paper summarises the major mechanisms and effects involved with SFSI and secondly it builds on existing literature to present a design procedure for considering these effects in design.

2 SFSI EFFECTS AND MECHANISMS

There are six major non-linear SFSI effects (Figure 1), the first three being effects relating to rigid foundations and the last three relating to flexible foundations.

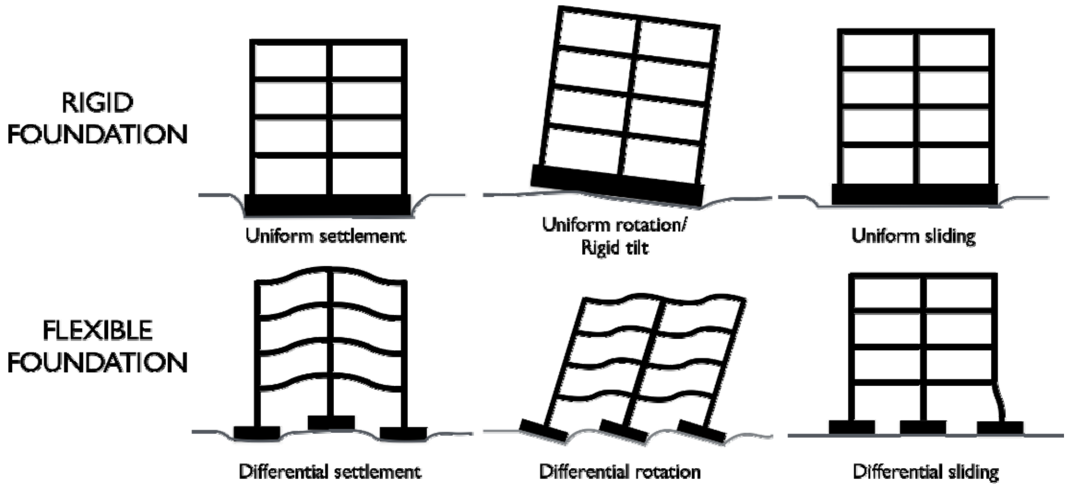

Figure 1. The effects of SFSI on buildings

2.1 Rigid foundation

The main contributor to uniform settlement is the shake down of the foundation through cyclic rocking (Figure 2). The yielding of soil on the compression edges eventually drives the foundation into the ground. It has been seen in several experimental research programs (eg. Taylor and Williams 1979) that the amount of settlement is dependent on the axial load ratio (*N*) (Eq. 1) and the rotation of the foundation.

$$N = \left(\frac{foundation\ axial\ capacity}{foundation\ axial\ demand}\right) \tag{1}$$

The dependence of settlement on axial load can be seen clearly from simulations using the macro-element proposed by Chatzigogos et al. (2009) in Figure 3, where the lightly loaded foundation (N=10) rocks backwards and forwards with some uplift and negligible dynamic settlement while the heavily loaded foundation (N=1.2) shakes itself into the ground. Empirical relationships have been derived to determine the dynamic settlement by Gajan et al. (2005) by relating it to the half cycle amplitude of foundation rotation and by Deng et al. (2012) to the cumulative footing rotation. If they are to be used in design, these relationships need to be in a form similar to Figure 4 where the dynamic aspects of the ground motions are accounted for and the settlement (δ_f) can easily be predicted and controlled through the foundation peak rotation design parameter ($\theta_{f,p}$) .

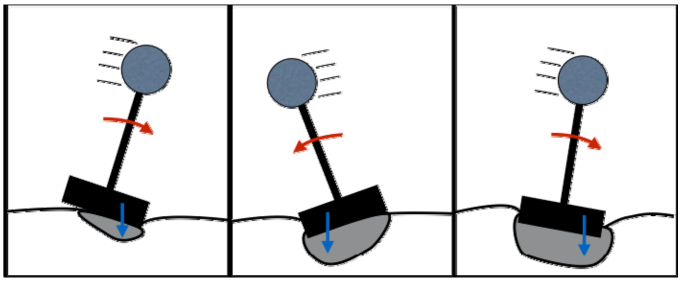

Figure 2. Shakedown of foundation through cyclic loading

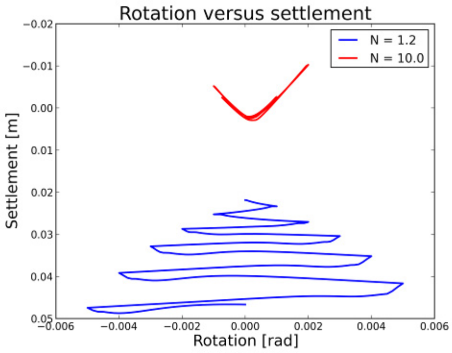

Figure 3. Settlement under cyclic loading – macro-element results

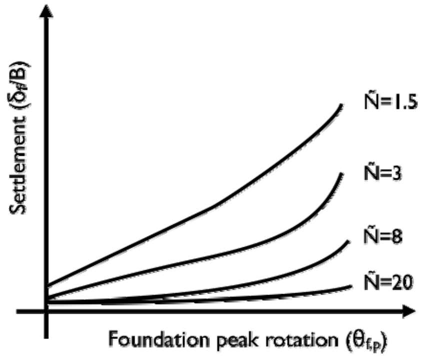

Figure 4. Relating settlement to peak foundation rotation

Uniform rotation effects include residual rotations, increased flexibility and toppling. Foundation rotation is the result of three separate mechanisms; the first being elastic-compliant rotation where the soil deforms in a recoverable manner. The second mechanism is foundation uplift, a geometric non-linear elastic mechanism, where the tension edge actually lifts off the soil. The final mechanism is a non-linear inelastic mechanism, where the soil yields under the compression edge in an irrecoverable manner. Since only the soil-yielding mechanism results in any irrecoverable displacement, the residual rotation can be determined based on the contribution of this mechanism, which is dependent on the axial load ratio and the foundation peak rotation. This behaviour is demonstrated in Figure 5 by a macro-element pushover analysis, where the lightly loaded foundation undergoes uplift, which results in an unloading stiffness less than the elastic loading stiffness. The heavily loaded foundation suffered from large amounts of plastic displacement culminating in large residual rotation. To allow designers to limit foundation residual rotations they need to be able to predict the expected amount of residual rotation for a given foundation. The relationships in Figure 6 are proposed to control the foundation residual rotation through the foundation peak rotation and axial load ratio.

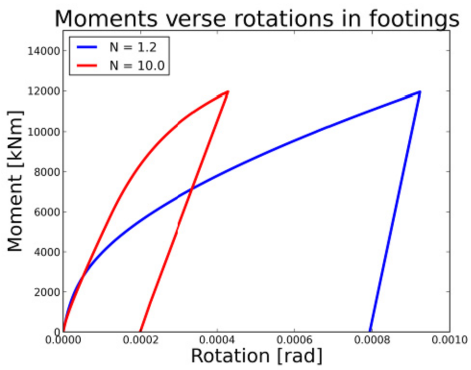

Figure 5. Push-over tests on foundation macro-element under constant axial load

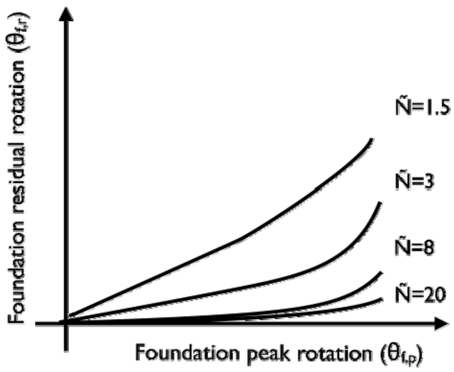

Figure 6. Relating foundation residual rotation to foundation peak rotation

All three mechanisms discussed above contribute to the increased flexibility of the building. The elastic stiffness (K_{r0}) can be approximated from solutions by Gazetas (1991) (Eq. 2) where G is the soil shear modulus, v is the Poisson's ratio, I_y is the area moment of inertia of the foundation about the axis of rotation and L and B are the half dimensions of the foundation.

$$K_{r_0} = \frac{G}{1-\nu} I_y^{0.75} \left(3 \left(\frac{L}{B} \right)^{0.15} \right) \quad (2)$$

The contribution from the two non-linear factors can then be dealt with by reducing the rotational elastic stiffness by a ratio which is dependent on the foundation rotation and axial load and including an equivalent hysteretic damping to account for the increased energy dissipation. Relationships for medium and dense sand have been developed by Paolucci et al. (2009) (Figure 7).

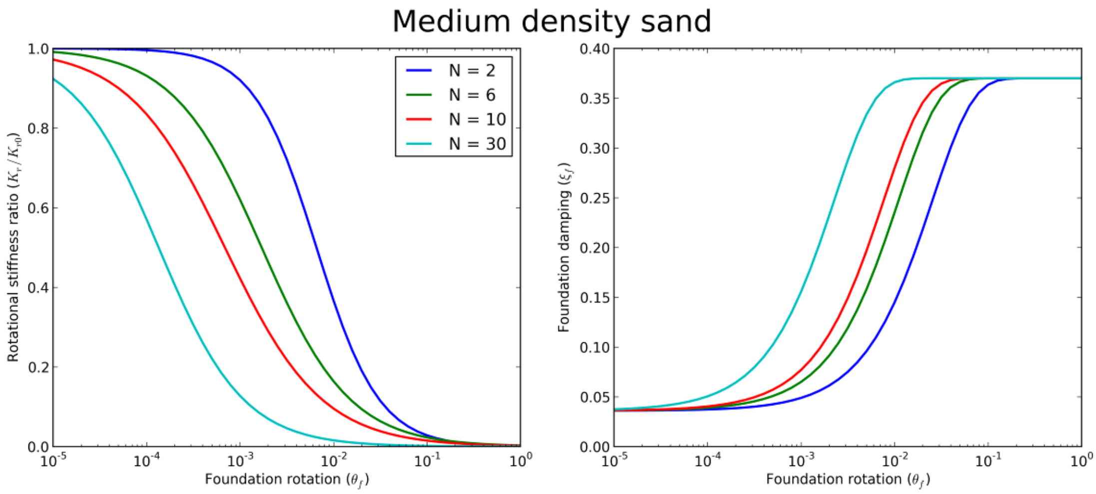

Figure 7. Foundation stiffness degradation and hysteretic damping (adapted from Paolucci et al. 2009)

Addressing the toppling of the foundation, experimental tests by Gajan et al. (2005) showed that the foundation moment is stable, predictable and ductile and the likelihood of collapse is not increased through allowing foundation uplift. In fact experimental research by Deng et al. (2012) demonstrated that rocking foundations can survive higher intensity shaking than their fixed based counterparts. This is due to the re-centring nature of a rocking foundation as opposed to a yielding super-structure which has a limited ductility.

The third global effect is foundation sliding, which occurs either due to sliding of the foundation on the surface soil if the interface friction is low or through soil shearing. For frictional soils sliding is dependent on the axial load ratio, while for purely cohesive soils sliding is limited by the shear strength of the soil. The amount of sliding is also dependent on the amount of embedment as the embedded foundation would be restricted by the soil on the side walls. Given that the majority of foundations are embedded, the contribution from sliding is quite low in non-liquefied soils and therefore this mechanism will be neglected.

2.2 Flexible foundation

For the designer to control the differential effects of SFSI they must consider a series of different mechanisms. The procedures for such considerations are not within the scope of this paper; however the mechanisms will be discussed in brief.

The differential settlement effect is due to several mechanisms; the first being frame-action where exterior columns and coupled walls experience additional cyclic vertical loads due to the seismic overturning moment (Figure 8 - left). The cyclic vertical load can result in additional settlements in the exterior columns and is a function of the static vertical load and the additional seismic vertical load. Differential settlement can also occur due to different footings experiencing different static loads (Figure 8 – centre), having different foundation sizes, varying soil conditions or varying dynamic loads (Figure 8 – right).

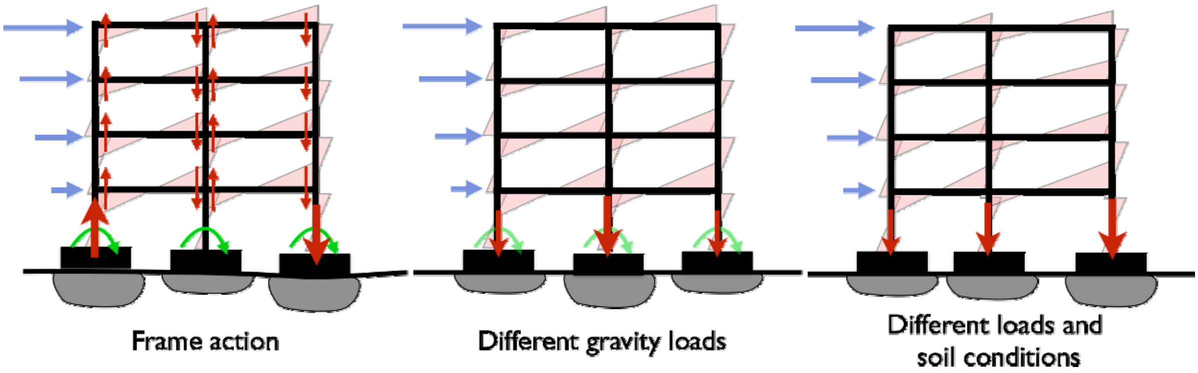

Figure 8. Frame action causing additional settlements under exterior columns

Considering differential rotations due to flexible foundations breaks from conventional SFSI analysis, which assumes rigid foundations. The major difference is with squat structures where an assumed rigid foundation gives a very high rotational stiffness and therefore SFSI effects are minimal, however, squat structures are still influenced by SFSI because the foundation is not rigid in reality. Localised rotation of footings can reduce the ductility demand on ground floor columns but with potentially larger roof displacements there can be an increase in the ductility demand on the beams.

The differential sliding mechanism is of little importance for a tied foundation where the axial stiffness of tie beams is more than adequate to restrict the differential sliding of footings.

3 DESIGN APPROACH

This performance-based design framework attempts to do two things: first integrate the structure and foundation design to have a consistent performance over the whole building and secondly consider the major SFSI effects in the design.

3.1 Performance limits

A performance-based design is a risk orientated decision making process where the engineering can control the performance of a building in terms of deaths, dollars and down-time for given ground motion intensities based on their likelihood, in a consistent manner. The designer must therefore decide on the importance level and life-time of the building to then determine the design level of shaking and design performance levels (see Table 1).

Table 1. Probability of exceedence for performance-based design (adapted from Priestley et al. 2007)

Importance level	No damage	Repairable damage	No Collapse
I	-	50% in 50 years	10% in 50 years
II	50% in 50 years	10% in 50 years	2% in 50 years
III	20% in 50 years	4% in 50 years	1% in 50 years
IV	10% in 50 years	2% in 50 years	1% in 50 years

To be confident that each of these limit states are satisfied, the design must incorporate not only all of the different mechanisms in the super-structure and the non-structural elements but must also adequately consider all of the foundation mechanisms. For each of the limit states the designer should satisfy a set of performance limits such as the recommended values given in Table 2. The values suggested in Table 2 have not been adjusted for the combination of performance limit states where the combined performance of the foundation and super-structure can result in unacceptable behaviour. The development of these limit states is explained in the companion paper by Giorgini et al. (2014).

Table 2. Performance limits

Performance parameters	No damage	Repairable	No Collapse
Inter-storey drift ($\theta_{SS,P}$)	0. 7%*	2.5%*	20% strength loss*
SS residual drift ($\theta_{SS,R}$)	0. 2%*	0. 5%*	P-delta limits*
Foundation peak rotation ($\theta_{F,P}$)	$\theta_{T,P}$ - $\theta_{SS,P}$	$\theta_{T,P}$ - $\theta_{SS,P}$	$\theta_{T,P}$ - $\theta_{SS,P}$
Foundation residual rotation ($\theta_{F,R}$)	0. 6%**	1.6%**	2.0%**
Foundation uniform settlement (m) (δ_F)	Structure specific	Structure specific	Structure specific
Foundation diff. settlement ($\delta_{F,Diff}/B$)	0. 6%**	1.6%**	2.0%**
Total peak drift ($\theta_{T,P}$)	Structure specific	Structure specific	Structure specific
Total residual drift ($\theta_{T,R}$)	0. 2%*	0. 5%*	P-delta limits*

* Sullivan et al. (2012) ** TFR (2007)

3.2 Design procedure

The design procedure presented here has an initial preliminary design considerations phase (Figure 9) where suitable design parameters are determined that should satisfy the performance limits. In steps 5-7 the foundations are sized, compared to a more conventional design process where foundations are sized based on over-strength loads from the super-structure. Following this there is a full design where the true design loads are determined and performance limits are checked (Figure 10), which largely follows the design procedure proposed by Paolucci (2013) with the addition of an estimate of foundation rotation and the settlement and residual rotation checks.

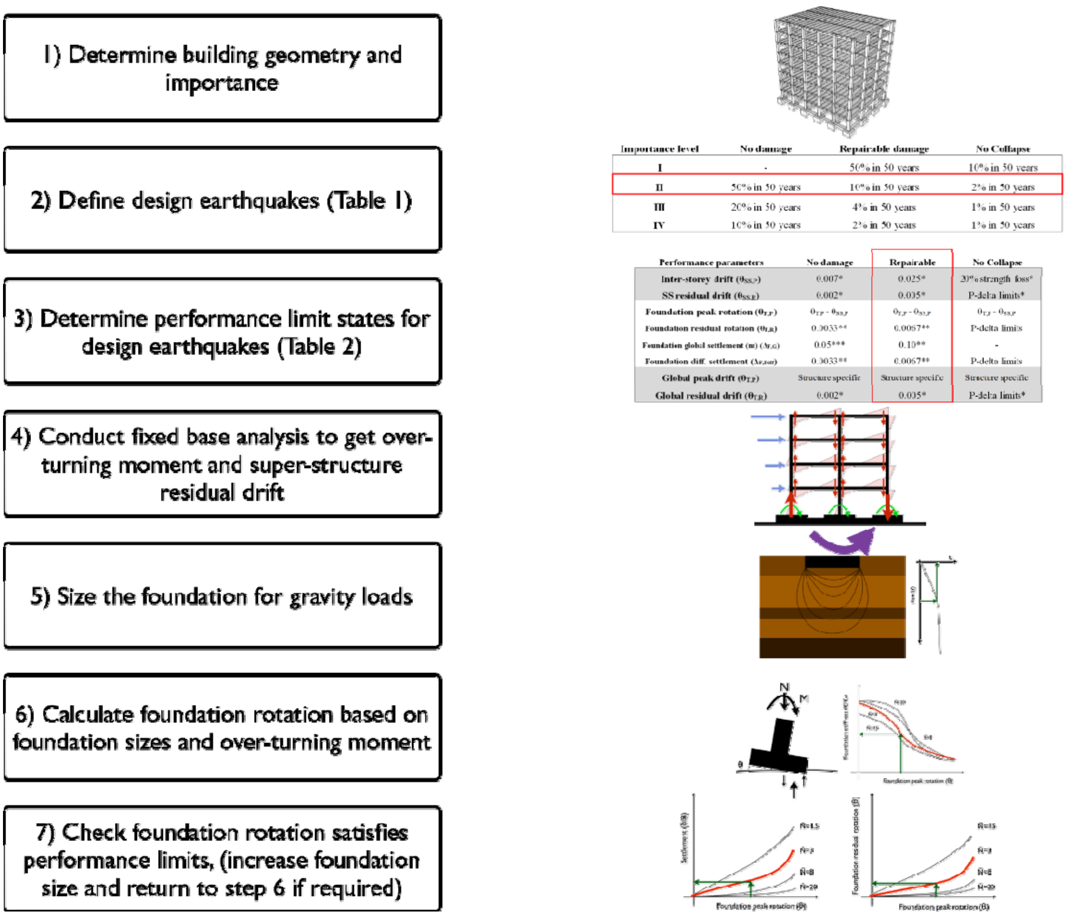

Figure 9. Preliminary design considerations of integrated building-foundation system

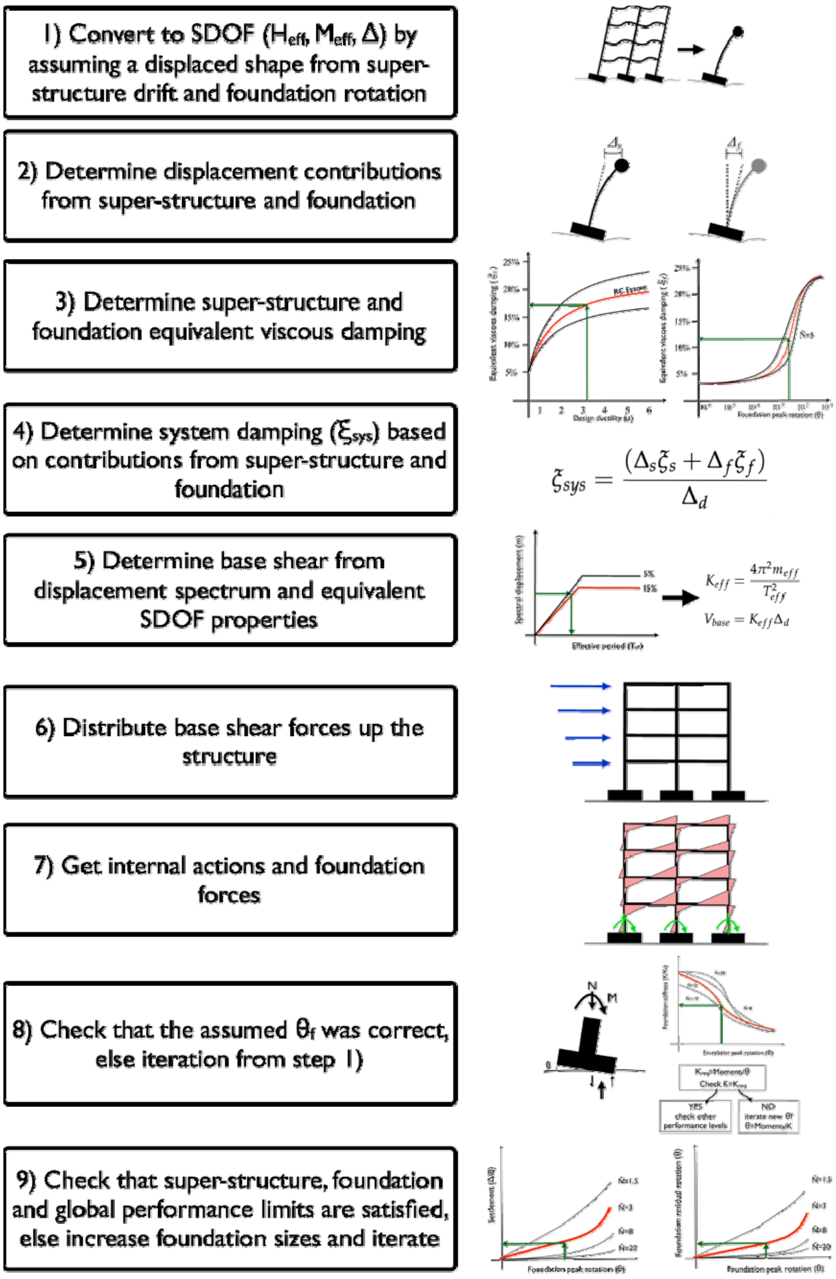

Figure 10. DDBD procedure considering SFSI

4 DESIGN EXAMPLE

The proposed design process has been demonstrated on the six storey frame building in Figure 11 for the shaking in the short direction since this is the more susceptible direction for SFSI effects. The building properties, soil properties and hazard parameters for use with the New Zealand loadings standard (NZS 1170.5) are all given in Table 3. The performance limits suggested for the repairable limit state in Table 2 were designed for in this example, however all limit states should be checked.

The initial base shear and over-turning moment were determined following the DDBD procedure assuming no foundation deformation and designed to the peak super-structure drift of 2.0% (Priestley et al. 2007). The residual drift can be approximated by a ratio of the design drift. The ratio of 0.17, taken as the mean plus one standard deviation from the case study in Christopoulus (2004), was used to give 0.34%. Step 5 was simplified for this example where a factor of safety of three against the factored static loads was deemed satisfactory to control settlements. The foundation rotation was calculated based on the fixed base over turning moment, the elastic rotational stiffness from Eq. 2 and the degradation of rotational stiffness due to non-linear effects from the curves suggested by Paolucci et al. (2009) shown in Figure 7. Table 4 shows the performance limits checks and preliminary sizes where crude estimates of settlement were made using the relationship by Gajan et al. (2005) by assuming three cycles to peak. The residual rotation was determined through a push-over to peak and unload using the macro-element by Chatzigogos (2009) to give an approximate upper bound value. No differential settlement checks where made since it was assumed that the foundation would be rigid.

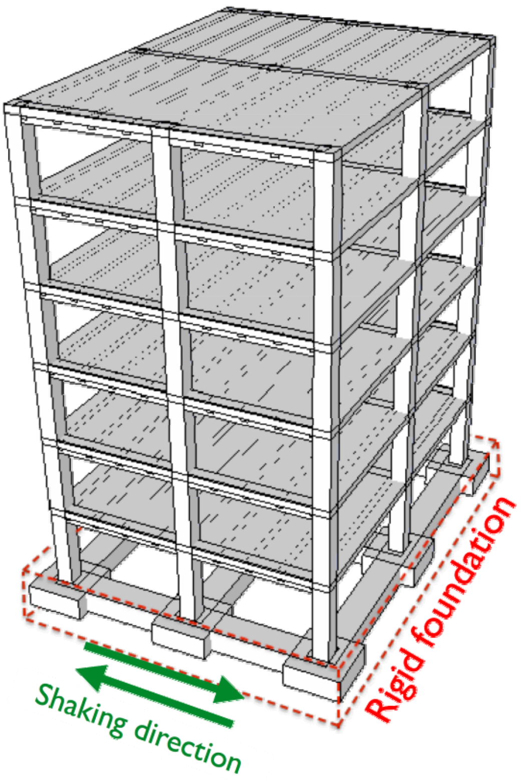

Figure 11. Case study building

Table 3. Design properties

Building:

Building length (m)	12
Building width (m)	16
Bay lengths (m)	6
Storey heights (m)	3.4
Beam depth (m)	0.6
Beam width (m)	0.5
Column depth (m)	0.7
Column width (m)	0.7
Conc. compression strength (MPa)	30
Steel strength (MPa)	300

Soil:

Shear stiffness (MPa)	40
Poisson's ratio	0.3
Critical angle	35
Cohesion (kPa)	0
Relative density %	60
Unit weight (kN/m3)	18

Hazard:

Soil type	D
Hazard factor (Z)	0.3
Return period factor (R)	1
Near fault factor (N)	1
Dead load G (kPa)	4.5
Live load Q (kPa)	2.5

Table 4. Preliminary and final design outputs

PRELIMINARY DESIGN

Base shear, fixed base per frame (kN)	411.5
OTM, fixed base per frame (kNm)	6153.1
Soil ultimate pressure (kPa)	1723
Footing length (m)	2.45
Footing width (m)	2.45
Footing depth (m)	1
Axial load ratio for static Eq loads	11.8
Foundation elastic rot. stiffness (MNm)	36029
Approx. foundation peak rotation	0. 14%
Approx. foundation settlement (m)	0.036
Approx. foundation res. rotation	~0%

FULL DESIGN

Base shear, SFSI per frame (kN)	375
Super-structure drift	2.0%
Super-structure res drift	0.34%
Foundation peak rotation	0.18%
Foundation settlement (m)	0.036
Foundation res. rotation	~0%
Total peak drift	2.18%
Total residual drift	0.34%

Based on the preliminary super-structure drift and foundation rotation the building was reassessed using the full design procedure (Figure 10). The final design values are presented in Table 4. It can be seen that there was very little change in the foundation rotation, however, there was a significant decrease in base shear due to the increased energy dissipation. It should be noted that the foundation damping was estimated through curves by Paolucci et al. (2009), which were based on cyclic loading tests and may significantly over-estimate damping.

5 CONCLUSIVE REMARKS

This paper presented the major effects of SFSI addressing the non-linear, rigid and flexible foundation mechanisms. Relationships were suggested to help the designer control these mechanisms through two design parameters (super-structure drift and foundation rotation). A performance-based design framework was proposed, which accommodates the presented SFSI mechanisms through the proposed relationships to control the effects. This design framework was demonstrated through a case study design of a six storey building. Current equations for foundation performance in terms of settlement, residual rotations, damping and stiffness degradation are still undeveloped. The local deformation effects of SFSI need to be accommodated into this procedure. The design displaced shape and higher mode factors in the current DDBD procedure may need to be revised to account for the influence of SFSI on multi-storey buildings. The performance limits need to be re-assessed to be consistent with the structural type. The intention is that as these relationships develop they will naturally fit into the design framework for immediate use.

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