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A DESIGN FRAMEWORK FOR SOIL-FOUNDATION-STRUCTURE INTERACTION

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ABSTRACT

A rigorous performance-based design methodology should not just consider the performance of the superstructure but the supporting foundation system as well. Case studies throughout history (eg. Kobe 1995, Kocaeli 1999 and Christchurch earthquakes 2010-2011) have demonstrated that a poor performance at the foundation level can result in a full demolition of the structure and, in general terms, that the extent of damage to, and repairability of, the building system as a whole, is given by the combination of the damage to the soil, foundation and superstructure. The lack of consideration of the modifying factors of soil-foundation-structure interaction (SFSI) and an absence of intuitive performance levels for controlling foundation and soil behaviour under seismic loads has resulted in inadequate designs for buildings sited on soft soil. For engineers to be satisfied that their designs meet the given performance levels they must first, understand how the soil-foundation interaction affects the overall system performance and secondly have tools available to adequately account for it in their design/assessment.

This paper presents performance-based design considerations based on a de-aggregation of super-structure and foundation performance. Several effects and mechanisms of non-linear SFSI are discussed and related to design parameters (super-structure drift and foundation rotation). 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.

INTRODUCTION

The effects of SFSI have been a topic of discussion amongst the structural and geotechnical community for many decades. The complexity of the mechanisms in SFSI as well as the need for inter-disciplinary knowledge of geotechnical and structural dynamics has plagued the advancement and the consequent inclusion of SFSI effects in design.

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

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relative to the main frequency of the structural-foundation system. There have been several studies into linear SFSI with non-linear structures, such as Comartin et al. (2000), which demonstrated that ignoring SFSI could 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 would generally suffer more damage compared to a fixed based equivalent when the super-structure period was 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 a more refined representation of the non-linear mechanisms at the foundation level such as up-lift, soil yielding and sliding, that can provide reliable energy dissipation mechanisms. Several extensive experimental programs have also given great insight into the mechanisms involved with SFSI (e.g. Gajan et al. 2005).

Amongst the development of experimental tests, numerical and analytical models and discussion over the detrimental or conservative effects of SFSI, the concept of purposefully designed yielding foundations has shown considerable promise. Deliberately designing to harness the effects of non-linear SFSI has been successfully demonstrated in laboratory experiments (Liu et al., 2013) as well as in massive structures such as The Rion-Antirion Bridge (Pecker, 2011). Unfortunately, often in practice the buildings and their foundations are decoupled in design and therefore it becomes difficult to determine the SFSI effects and to implement such a design philosophy that is dependent on both the building and the foundation dynamic behaviour.

Several authors have recently suggested integrated design procedures for the super-structure and foundation employing the Direct Displacement-based Design (DDBD) (eg. Sullivan 2010, Paolucci 2013). The integrated approach allows the effects of increased flexibility, shared ductility and modified mode shapes to be considered directly in the design of both the super-structure and foundation. Currently available 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 foundation deformations, however, there is no guidance on how to compute such values.

The earthquake engineering profession is moving towards low damage building designs, however an investigation into the performance of buildings during the Christchurch earthquakes by Giorgini (2014) has shown that buildings were deemed irreparable due to super-structure damage, foundation damage or a combination of both. To maintain consistency in the design of buildings, the foundation performance levels need to be considered with the same rigour as that of the structure.

PERFORMANCE CONSIDERATIONS

A performance-based design is a risk-oriented decision making process where the engineer 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 (desired) 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 exceedance 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 performance levels are satisfied, the design must incorporate not only all of the different mechanisms of the super-structure and in the non-structural elements but

must also adequately consider all of the foundation mechanisms. Figure 1 shows a list of "limitation parameters" which contain the major effects that must be considered in a performance-based design. Controlling the corresponding performance parameters to suitable levels can satisfy these limitation parameters. This can be achieved by controlling the main engineering demand parameters (design parameters) associated with superstructure and foundation behaviour, where key relationships between design parameters and performance parameters are required. Several of these key relationships are presented in the following section.

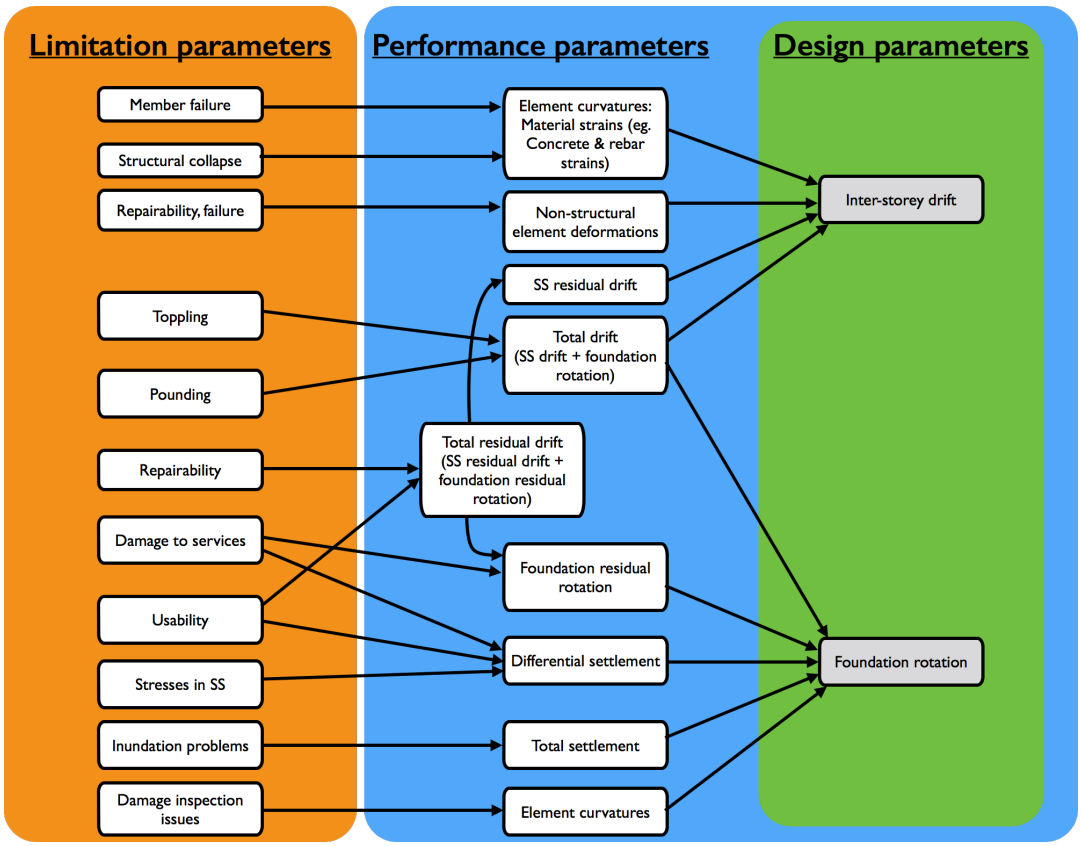


Figure 1. Deaggregation of performance-based design for foundation and superstructure

The performance parameters tend to have different performance limits depending on the performance level and Table 2 suggests some values taken from literature. Table 2 lists the inter-storey drift as the sole performance parameter for the transient performance of the structure, as the element curvatures and non-structural element deformations implicitly limit this value, so have not been listed.

Table 2. Performance limits

Performance parameters	No damage	Repairable	No Collapse
Inter-storey drift ($\theta_{SS,P}$)	0. 7%*	2.5%*	20% strength loss*
Super-structure 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 (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*

SFSI EFFECTS AND MECHANISMS

Four major non-linear SFSI effects have been considered in this design procedure (Figure 2), the first two being effects associated to rigid foundations and the last two relating to flexible foundations.

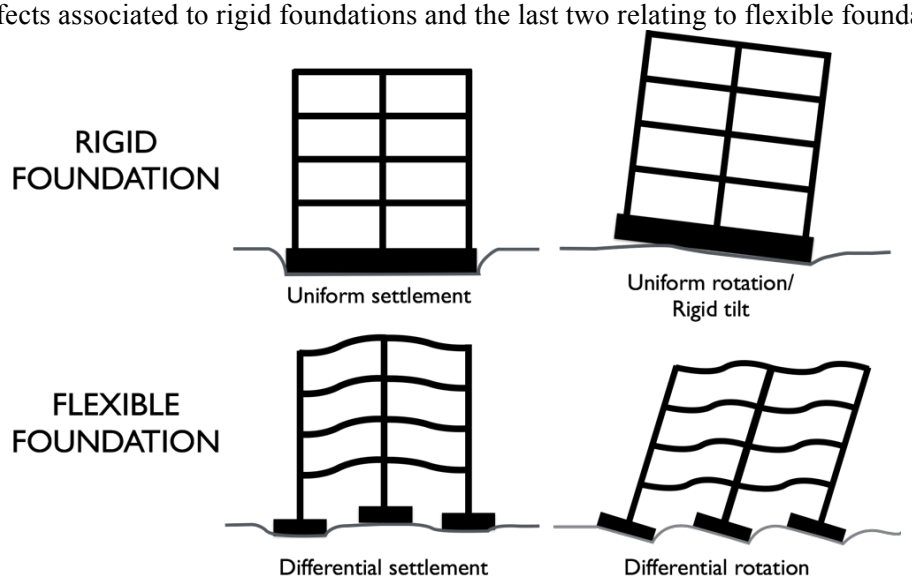


Figure 2. The effects of SFSI on buildings

Rigid Foundation

The main contributor to dynamic uniform settlement is the shake down of the foundation through cyclic rocking (Figure 3). 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 ($\widehat{N}$) (Eq. 1) and the rotation of the foundation.

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

The dependence of settlement on axial load can be seen from simulations using the macro-element proposed by Chatzigogos et al. (2009) in Figure 4, 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 from experimental testing have been derived to determine the dynamic settlement by either relating it to the half cycle amplitude of foundation rotation (Gajan et al. 2005) or to the cumulative footing rotation (Deng et al. 2012). For settlement to be controlled in design using the peak foundation rotation, the empirical relationships need to be in a form similar to the relationships qualitatively shown in Figure 5, 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 ($\theta_{f,p}$).

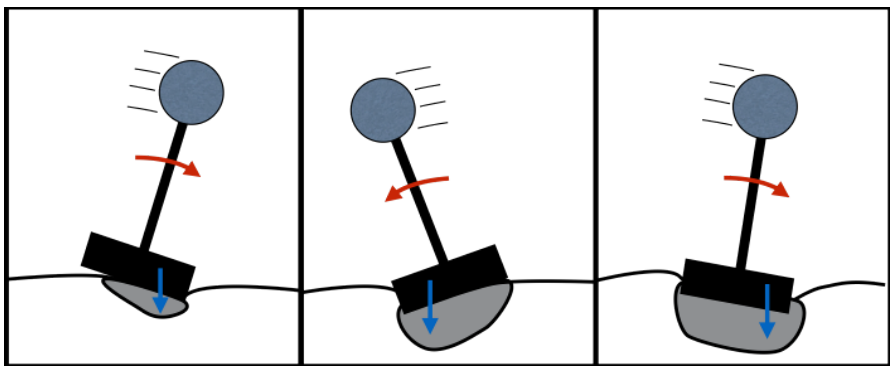


Figure 3. Shakedown of foundation through cyclic loading

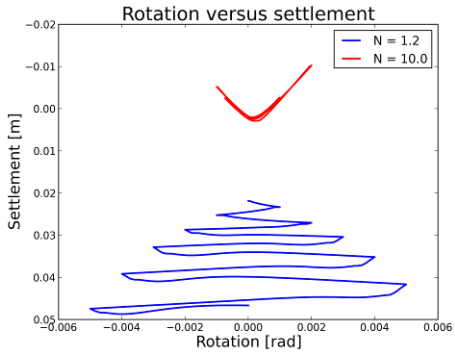


Figure 4. Settlement under cyclic loading – macro-element results

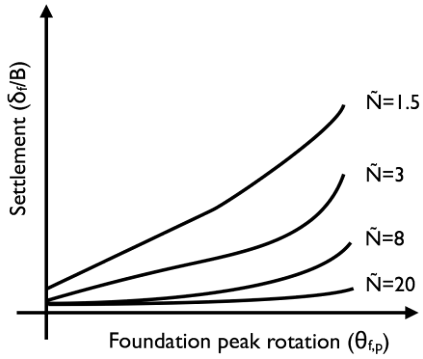


Figure 5. Relating settlement to peak foundation rotation

Uniform rotation causes a modification in the dynamic behaviour due to increased flexibility and often results in detrimental effects such as residual rotations, increased displacements and toppling. The modification in dynamic behaviour can be incorporated into the design procedure by adjusting the natural period of the building and the modes of displacement, while the resulting residual effects need to be addressed further.

Foundation rotation is the result of three separate mechanisms. The first being elastic-compliant rotation where the soil deforms in a recoverable manner and it is the dominant mechanism for very small rotations (Figure 6 - left). The second mechanism is foundation uplift, a geometric non-linear elastic mechanism, where the tension edge actually lifts off the soil, which tends to be prevailing for lightly loaded foundations (Figure 6 - centre). The final mechanism is a non-linear inelastic mechanism, where the soil yields under the compression edge in an irrecoverable manner and is dominant for heavily loaded foundations (Figure 6 - right). 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 7 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 8 are proposed to control the foundation residual rotation through the foundation peak rotation and axial load ratio.

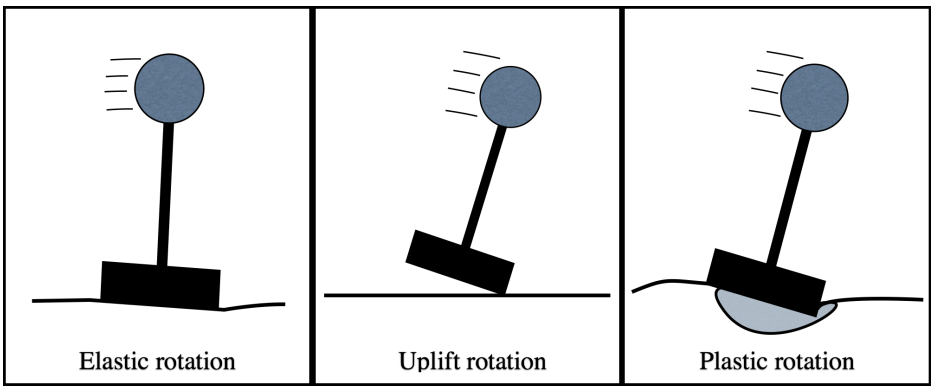


Figure 6. Mechanisms of foundation rotation

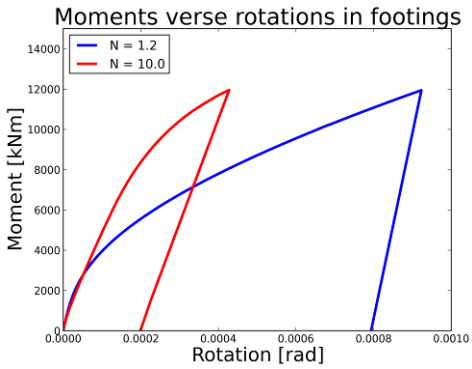


Figure 7. Push-over tests on foundation macro-element under constant axial load

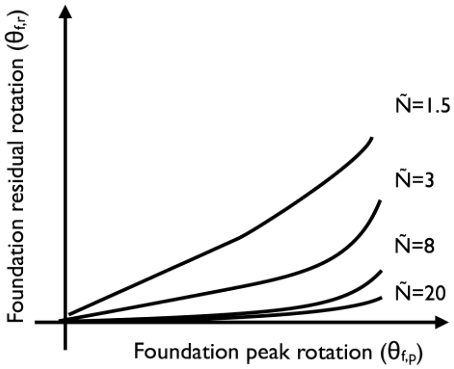


Figure 8. Relating foundation residual rotation to foundation peak rotation

All three mechanisms discussed above contribute to the increased flexibility of the building. The elastic stiffness (K_{r_0}) 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.

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

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) and for clay by Adamidis et al. (2013).

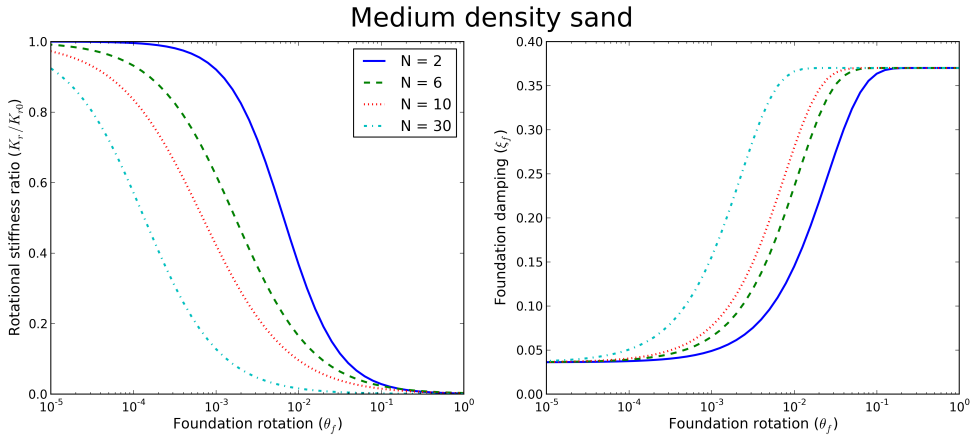


Figure 9. Foundation stiffness degradation and hysteretic damping (adapted from Paolucci et al. 2009)

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 was due to the re-centring nature of a rocking foundation as opposed to a yielding super-structure which has a limited ductility.

Flexible foundation

Frame structures involve several additional mechanisms; the first being frame-action where exterior columns and coupled walls experience additional cyclic vertical loads due to the seismic overturning moment (Figure 10 - 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 (Figure 13 qualitatively demonstrates the interaction in a way that could be used in design). The cyclic

vertical load can also cause assymetric rotational stiffness and strength as seen in Figure 7 where the rotational stiffness is dependent on the axial load ratio and this changes with the direction of loading. The resulting asymetric behaviour is demonstrated in Figure 11 where in this case the positive rotation is accompanied by a reduced axial load causing uplift which was not observed in the negative direction where increased axial load keeps the foundation compliant. Differential movements can also occur due to different footings experiencing different static loads (Figure 10 – centre), having different foundation sizes, varying soil conditions or varying dynamic loads (Figure 10 – right).

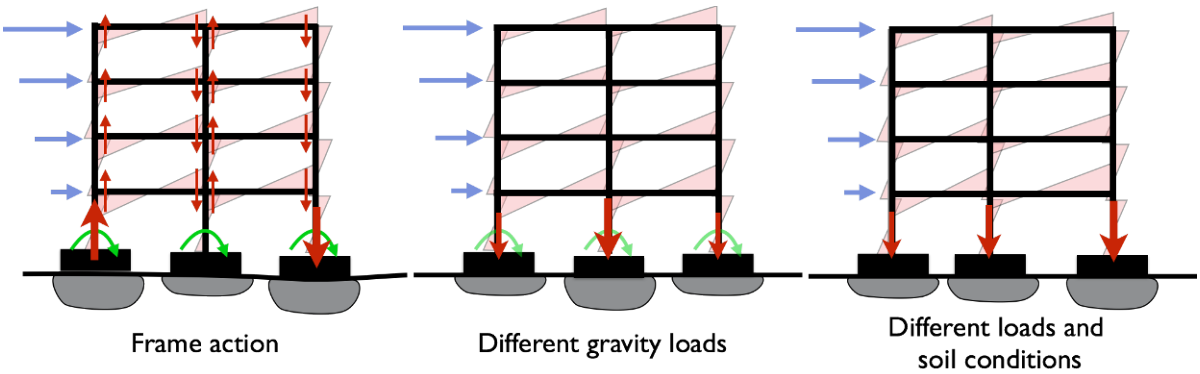


Figure 10. Addtional SFSI mechanisms in frame structures

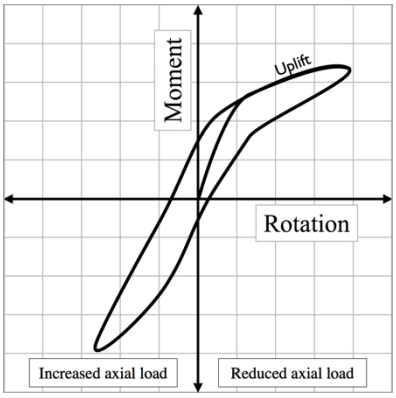


Figure 11. The effects of cyclic axial load on rotational strength and stiffness

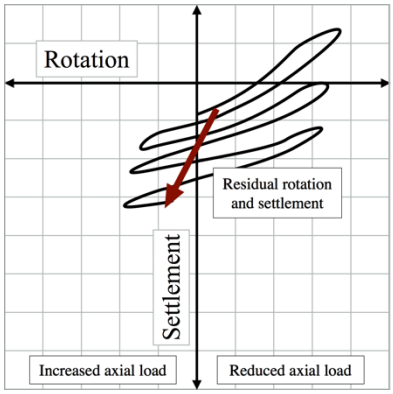


Figure 12. The effects of cyclic axial load on rotation and settlement

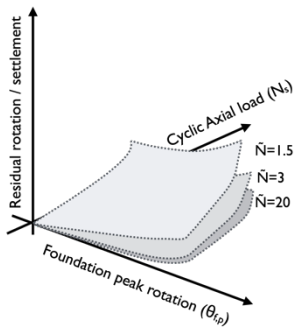


Figure 13. Relating foundation residual deformations to foundation peak rotation and cyclic axial load

Considering differential rotations due to flexible foundations goes beyond conventional SFSI analysis, which assumes rigid foundations. The major difference is with squat structures where an assumed rigid foundation provides 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 potentially larger roof displacements can increase the ductility demand on the beams.

Differential residual rotations or tilt can also occur in frame structures and **Figure 12** demonstrates how a variation in axial load can result in residual rotations due to the asymmetric nature of the loading. For designers to account for such effects the behaviour must be quantified and Figure 13 shows a suggested relationship where the static axial load, cyclic axial load and the peak foundation rotation can be used to predict the settlement and residual rotation.

DESIGN APPROACH

This performance-based design framework has two objectives: 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.

Design procedure

The design procedure presented here has an initial preliminary design considerations phase (Figure 14) 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 design loads are determined and performance limits are checked (Figure 15), which largely follows the design procedure proposed by Paolucci (2013) with the addition of an estimation of foundation rotation and the settlement and residual rotation checks.

1) Determine building geometry and importance

2) Define design earthquakes (Table 1)

3) Determine performance limit states for design earthquakes (Table 2)

4) Conduct fixed base analysis to get over-turning moment and super-structure residual drift

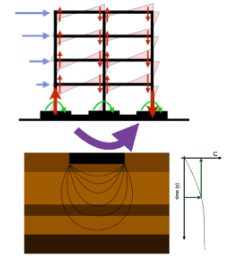
5) Size the foundation for gravity loads

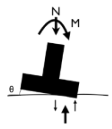
6) Calculate foundation rotation based on foundation sizes and over-turning moment

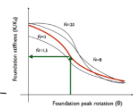
7) Check foundation rotation satisfies performance limits, (increase foundation size and return to step 6 if required)

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

Performance parameters	No damage	Repairable	No Collapse
Inter-storey drift ($\theta_{SS,P}$)	0.007*	0.025*	20% strength loss*
SS residual drift ($\theta_{SS,R}$)	0.002*	0.005*	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 ($\theta_{F,R}$)	0.0033**	0.0067**	P-delta limits
Foundation global settlement (m) ($\Delta_{F,G}$)	0.05***	0.10**	-
Foundation diff. settlement ($\Delta_{F,DIF}$)	0.0033**	0.0067**	P-delta limits
Global peak drift ($\theta_{T,P}$)	Structure specific	Structure specific	Structure specific
Global residual drift ($\theta_{T,R}$)	0.002*	0.005*	P-delta limits*







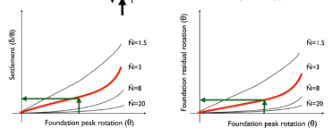


Figure 14. Preliminary design considerations of integrated building-foundation system

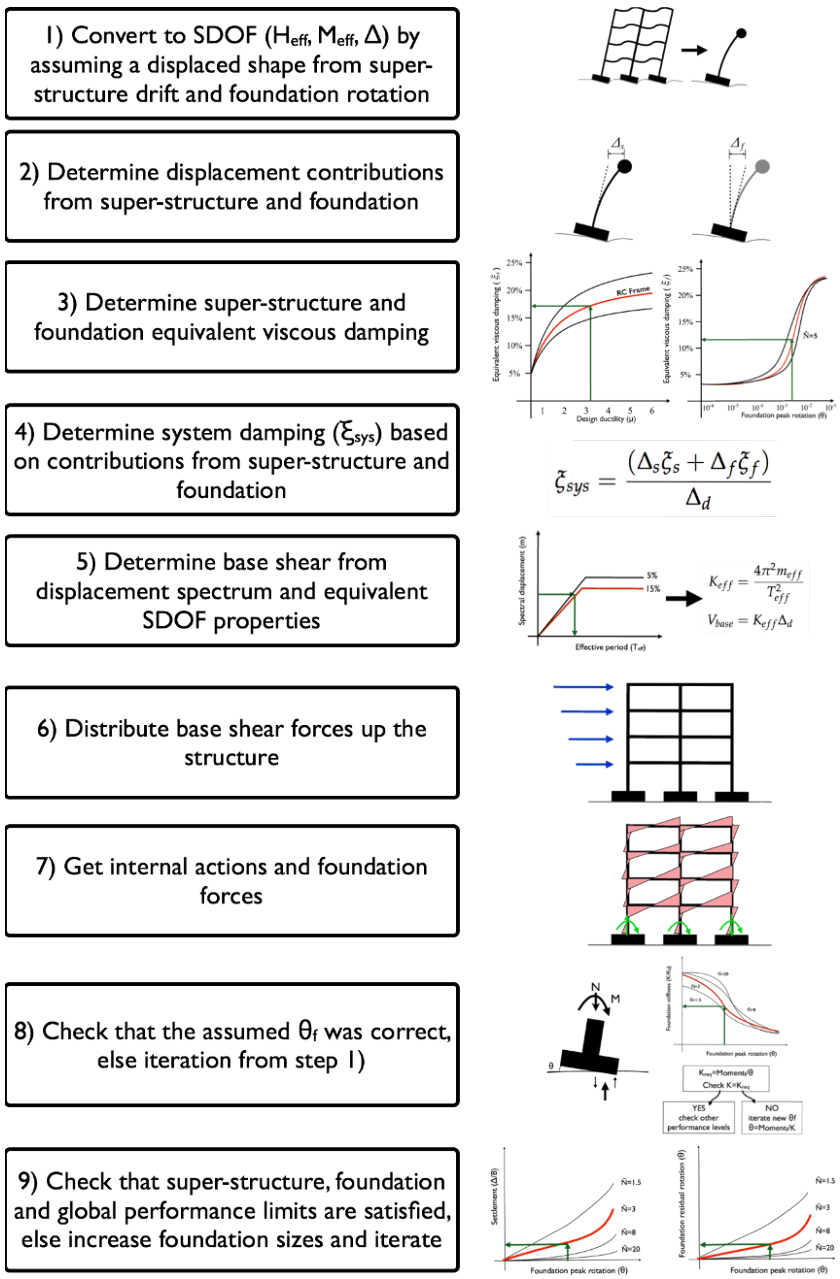


Figure 15. DDBD procedure considering SFSI

The proposed design procedure has been implemented on a case study building consisting of a six-storey frame building in Figure 16 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 inter-storey drift. The ratio of 0.17 was used to give 0.34%, taken as the mean plus one standard deviation from the study presented by Christopoulus and Pampanin (2004). 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 9. Table 4 – left 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 value. No differential settlement checks where made since it was assumed that the foundation would be rigid.

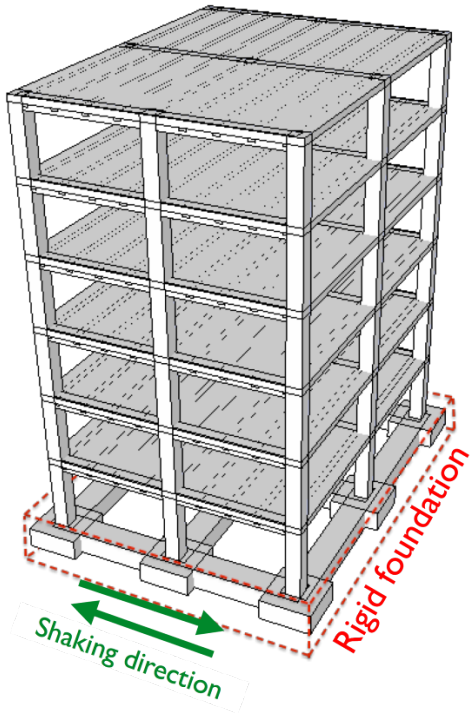


Figure 16. Case study building

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)	410
OTM, fixed base per frame (kNm)	6150
Soil ultimate pressure (kPa)	1720
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)	36000
Approx. foundation peak rotation	0. 14%
Approx. foundation settlement (m)	0.04
Approx. foundation res. rotation	~0%

FULL DESIGN	
Base shear, SFSI per frame (kN)	375
Super-structure drift	2.0%
Super-structure res drift	0.3%
Foundation peak rotation	0.18%
Foundation settlement (m)	0.04
Foundation res. rotation	~0%
Total peak drift	2.2%
Total residual drift	0.3%

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 - right. It can be seen that there was very little change in the foundation rotation compared to the preliminary design, however, there was a small decrease in base shear due to SFSI effects. 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.

CONCLUSIVE REMARKS

This paper presents a series of performance-based considerations that assess the performance of the super-structure and the foundation in a consistent manner. These are incorporated into a displacement-based design procedure that accounts for the major dynamic and residual effects of SFSI. The design procedure relies on a series of key relationships between the design parameters (inter-storey drift and foundation rotation) and the performance parameters. Through these relationships the designer can control the behaviour of the building and thus satisfy the given performance levels. The design framework was demonstrated through a case study design of a six-storey building. Equations for foundation performance in terms of settlement, residual rotations, damping and stiffness degradation still require rigorous development. 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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