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Designing a settlement tolerant building using simplified soil structure interaction

Z. Teh & A. Palermo
WSP Opus, Christchurch.

M. Lowe
WSP, United Kingdom.

ABSTRACT

The Ministry of Education's (MOE) guidance document 'Designing Schools in New Zealand, Structural and Geotechnical Guidelines' sets out key considerations including designing to accommodate settlement and for usability following earthquakes, including ease of repair and relevelling, particularly for liquefaction-prone sites.

This paper presents a case study design of a single storey steel moment resisting frame building with shallow foundations on a liquefaction-prone site with low bearing capacity. The aim is to estimate differential settlements in the SLS and ULS using a well understood and industry-accepted analysis method, to demonstrate compliance with SLS requirements of the MOE guidance and better understand structural implications in terms of residual capacity. Modelling of the superstructure will involve spring stiffness for base fixity and moment connections. The Beam on Nonlinear Winkler (BNWF) methodology with reference to the PEER 2005/04 report, will be applied in relation to soil-structure interaction. The robustness of the structural concept and detailing is validated through a series of loading conditions.

1 INTRODUCTION

Structural designers of school buildings in New Zealand are required to make reference to the Ministry of Education's (MOE) document 'Designing Schools in New Zealand, Structural and Geotechnical Guidelines' (MOE, 2016). Key structural design considerations include accommodation of settlement and maintaining operation following earthquakes, which is critical for liquefaction-prone sites.

The three main mechanisms for liquefaction-induced building settlement are ejecta-induced, shear-induced and volumetric-induced ground deformation. Shear-induced settlement due to bearing pressures from earthquake loading will be estimated using the Beam on Nonlinear Winkler Foundation (BNWF) methodology with reference to the Pacific Earthquake Engineering Research Centre (PEER) report 2005/04 (Harden et al, 2005), while volumetric-induced ground deformations will be directly taken from geotechnical liquefaction analysis.

Briefly, the PEER 2005/04 report presents three springs to describe nonlinear inelastic behaviour of the soil-foundation system. Node springs are applied to represent vertical load-displacement, horizontal passive load-displacement against the side of the footing and horizontal shear sliding at the base of a footing. Moment-rotation behaviour is implicitly captured by defining vertical springs along the length of the footing.

A single storey moment resisting frame with shallow foundations on a liquefaction-prone site is designed with SSI applied based on spring stiffness derived from PEER2005/04 methodology. SLS performance criteria considers floor slopes, while ULS performance criteria considers strength design due to ULS loading and differential settlement. The implication on post-earthquake residual capacity on a distorted structure due to differential settlement, is assessed by applying gravity loads and a second ULS loading with further settlement. The aim is to demonstrate design of a settlement tolerant building with a structural system that is robust and easily repairable and relevellable.

2 GROUND CONDITIONS

Cone penetration tests to 20m depth indicate uniform sand profile across the site. Ground water level of 0.7m depth was used for liquefaction analysis. Key findings of the geotechnical assessment used in the structural analysis:

- Total settlement of up to 150 mm and 60 mm should be adopted in the design of the foundation system when considering ULS and SLS design events respectively.
- Differential settlement of approximately 60 mm and 30mm for a ULS event and SLS event respectively should be accounted for foundation design. These values are taken as the upper bound for conservative design.
- Liquefaction occurs closely after SLS shaking. There is a low risk of lateral spreading.
- Coefficient of subgrade modulus in the seismic liquefied case, K_s 100-1600 kN/m^3
- An ultimate static bearing pressure (non-liquefied) was calculated to be 500 kPa, while the ultimate liquefied seismic bearing pressure was found to be 19 to 24 kPa for a shallow foundation option.

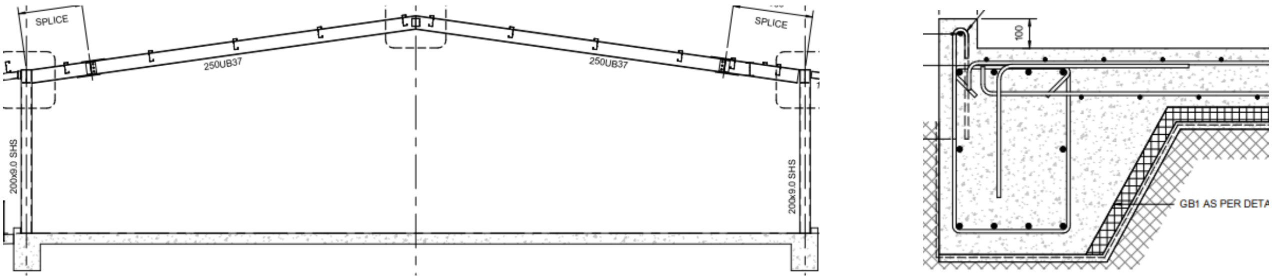
3 DESIGN CONCEPT

A single storey lightweight building with typical configuration of 15m x 6m grid is considered, with the short direction taken as 15m and the long direction taken as 36m. To afford future flexibility in internal layouts all columns are located on the perimeter and all internal partitions are non-load bearing elements.

The building importance level is 3 (IL3) as it is a school with a capacity for more than 250 occupants. It is to be designed for a 1 in 25 year return period for SLS1 and 1 in 1000 year return period for ULS limit states.

The lateral resisting system comprises two-way moment frames designed for a ductility of 1.25, with roof plane bracing to transfer loads in the long direction. Columns are located on the perimeter foundation beams. The two-way moment frame system ensures loads are distributed to all columns, mitigating concentration of loads and settlement. The two-way moment frame system also means that the structure has a similar response in each orthogonal direction instead of a braced longitudinal system that is usually designed. Figure 1 provides an overview of the structural system.

The substructure consists of edge foundation beams 500 mm x 700 mm supporting the columns, with a solid (not ribbed) internal ground-bearing slab. A solid slab was chosen as it is easier to repair (by crack injection) and relevel (grout injection), compared to a system with internal ground beams. The intent is for future relevelling to mainly involve work to the edge beams that are easily accessible.



(a) Typical section. (b) edge beam 500x700

Figure 1: Structural system

The edge beams form the main substructure elements in supporting loads from the roof and external walls. As the edge beams will settle more than the slab, the slab adjacent to the beams need to be designed for this effect. The slab provides rotational restraint to the edge beam, especially when passive pressure is applied to the beams simultaneously with base shear from the column. The slab also acts as a tie in the short direction and is expected to deform in sympathy with ground deformation in the liquefied case.

A 1m thick geo-syntethic reinforced gravel raft was recommended by the geotechnical engineer. This is now common practice as per MBIE Residential Technical Guidance Part C (MBIE 2015). This reduces the potential for surface expression of liquefaction within the building footprint, reduces the potential for liquefaction induced differential settlement, improves the bearing capacity and subgrade stiffness, and generates less vibration compared to other ground improvement methods. Ground beams are keyed 450mm into the gravel raft for sliding resistance. The action point for passive resistance of the edge beams is lower than the slab which produces an eccentric moment, and has been considered in the design.

SLS and ULS performance limit objectives were considered as follows:

- SLS: Table C7.1 NZS1170.5 Supp 1:2004 Acceptable Serviceability Limit State Criteria for Earthquakes, for plasterboard walls with exposed surface finish indicates Height/300 limit for cracks sufficiently visible to need repair. For perimeter glazing a Height/250 performance limit is required to avoid broken glass. Assuming similar rotation of the wall due to settlement, 0.33% performance criteria for differential settlement was adopted. Vertical differential settlement of 0.5% as a trigger for relevelling.
- ULS: Life safety criteria with no collapse. Strength design based on differential settlement from building response plus geotechnical liquefaction assessment. Vertical displacement applied under column supports to model differential settlement.

4 SOIL-STRUCTURE INTERACTION

Lateral and vertical springs were modelled using the PEER 2005/04 report, involving BNWF with vertical load-displacement (Qzsimple1) and horizontal passive load-displacement (Pysimple1) springs. Horizontal sliding resistance considered friction and adhesion derived from gravity and active pressures.

4.1 Vertical load-displacement spring, Qzsimple1

Qzsimple 1 spring was derived from Equations 3.5, 3.6 and 3.7 of PEER 2005/04 as per Figure 2.

For ULS using q_{ult} = 24kPa, z_{crit} = 130mm, z_{50} = 0.125z_{crit}, c = 12.3, n = 5.5 and C_d = 0.05, a multilinear spring was obtained. Keeping the same parameters and using z_{crit} = 60mm, the SLS multilinear spring was obtained.

(where q_{ult} = ultimate resistance of the q-z material in compression; z_{50} = the displacement at which 50% of q_{ult} is mobilised during monotonic loading; c,n = constants that control the shape of the q-z^p curve; C_d = ratio of the maximum drag (suction) force to the ultimate resistance of the q-z material)

Alternatively, for a single storey lightly loaded building where plastic yielding of soil is not expected, the liquefied K_s was used. In this case study the Qzsimple1 spring was applied under the slab.

4.2 Horizontal passive load-displacement spring, Pysimple1

Pysimple1 spring was derived from Equation 3.8 of PEER 2005/04 as shown in Figure 2.

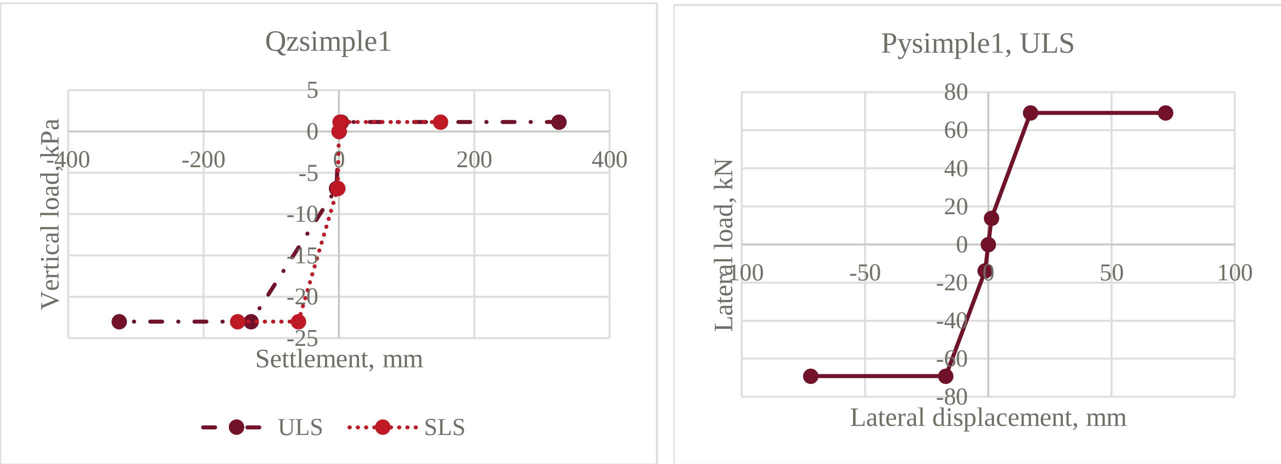


Figure 2: Qzsimple1 and Pysimple1 backbone derived from PEER2005/04

4.3 Friction

Friction between the slab underside and sub-base considered the self-weight of the slab with a coefficient of friction of 0.1 to account for the damp-proof membrane (dpm) layer. Based on a 200mm thick concrete slab a friction spring of 0.48 kPa was applied in the horizontal directions. This ignores slab uplift which would be minimal for the lightweight single storey structure.

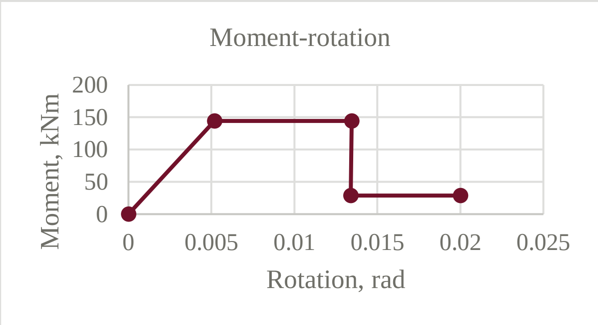
Adhesion in the direction of loading to the external beam face (one side only without dpm), was included using active soil pressure with a coefficient of friction for concrete-gravel of 0.55. Based on 400 mm beam contact face, active coefficient of 0.3 and soil density of 18 kN/m^3, a concrete-gravel adhesion of 0.59 kPa was applied as a line spring uniformly distributed (kN/m) along the length of the edge beam.

However for a single storey lightly loaded building where horizontal translation is likely to be minimal, horizontal restraints were applied to the model instead of friction springs.

5 BASE FIXITY

Steel Construction Institute SCI Publication P398 (SCI P398, 2015) was used to design the 400 x 400 x 30 thick base plates with 6M30 Gr24 holding down bolts. Bolts are capacity protected with failure mechanism involving base-plate yielding.

The rotational stiffness of the base-plate was obtained using Eurocode 3 BS EN 1993-1-8:2005 (EC3, 2005) Cl. 6.3, with stiffness coefficients k_{15} = 25.2 and k_{16} = 1.2. Residual strength of 20% post-ultimate was assumed as indicated in Figure 3.



(a) Moment-rotation of base-plate

$$S_j = \frac{Ez^2}{\mu \sum_i \frac{1}{k_i}}$$

where:

k_i is the stiffness coefficient for basic joint component i;

z is the lever arm, see 6.2.7;

μ is the stiffness ratio $S_{j,ini}/S_j$, see 6.3.1(6).

(b) Rotational stiffness S_j from Eq 6.27 BS EN 1993-1-8

Figure 3: Base rotational stiffness

6 ANALYSIS AND DISCUSSION

Briefly, the superstructure elements are first designed based on fixed base restraint with pushover analysis to evaluate its robustness. SSI is then considered in the model to assess the effect of differential settlement for SLS and ULS with loads applied using the equivalent static method. Finally, with the structure distorted due to differential settlement and response from the initial design event, two load conditions are further considered –a second SLS earthquake, and a second ULS earthquake with further differential settlement.

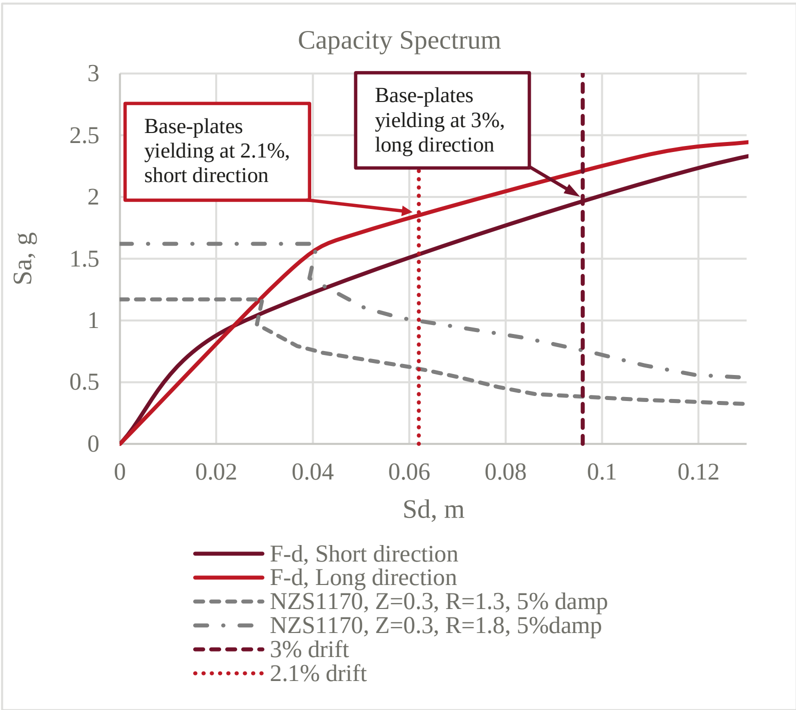
SAP2000 v20 software (CSi, 2018) was used to model the building with the steel frame and ground beams as line elements, and the slab as shell element. The Qzsimple1 vertical spring was applied to the slab shell element. The ground beams and slab are constrained in the model similar to a raft condition. Individual demands in the ground beams and slab are obtained in addition to the raft displacement.

6.1 Fixed Base Restraint and Pushover Analysis

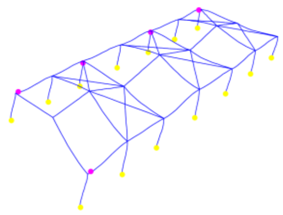
For strength design, fixed base restraint was applied to obtain upper bound design of the superstructure. The steel framing consists of 250UB rafter and 200 SHS column sizes. Using the equivalent static method, maximum reactions at the base from the governing static case were obtained: 140 kNm moment, 64 kN compression and 84 kN shear. SLS and ULS drifts were respectively in the order of 0.3% and 1.1%. Ground beams at slab edges supporting columns, are sized at 500 mm wide and designed for torsional demands arising from column base fixity.

With a fixed base condition, the failure mechanism will likely occur with yielding of the base-plate before hinging of the columns and rafters. A non-linear static analysis was carried out with 100% push in one direction and 30% push in the other direction, ignoring P-Delta given that gravity loads are minimal. Hinges were defined for the base fixity as per preceding section, with column and beam non-linear hinges modelled to ASCE41-13 (ASCE, 2014) parameters and acceptance criteria.

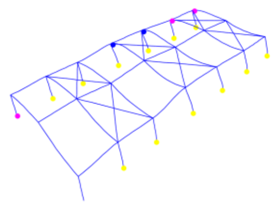
The capacity spectrum plots as per Figure 4 showed a performance point at S_a - S_d of 1.17g,0.03m, with corresponding base shear-displacement of 677kN, 0.046m for the elastic design spectrum at 1/1000 annual probability of exceedance (APE). Global failure mechanism comprised hinging to base-plates followed by column yielding.



(a) Capacity Spectrum



(b) Long direction, 3% drift at collapse with base-plates hinging



(c) Short direction, 2.1% drift at collapse with base-plates hinging

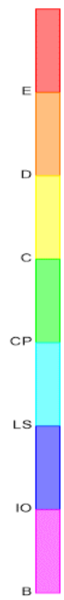


Figure 4: Capacity-Demand plot and collapse mechanism

6.2 SSI springs for Shear-Induced Settlement

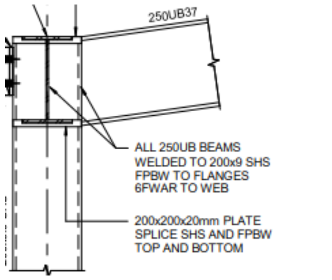
For the liquefied case, shear-induced differential movement due to earthquake loading was estimated using springs derived from the PEER2005/04 method. Peak inertial load is assumed to occur prior to triggering of liquefaction. The application of full earthquake loading on liquefied soils gives an upper bound design. Damping and period lengthening due to liquefaction will result in lower demand, however is not considered here.

Settlement due to earthquake response of the superstructure was respectively in the order of 40mm and 45mm for SLS and ULS cases. These are similar given the lightweight single storey construction. SLS and ULS floor slopes however were more distinct near the edge beams as per Table 1.

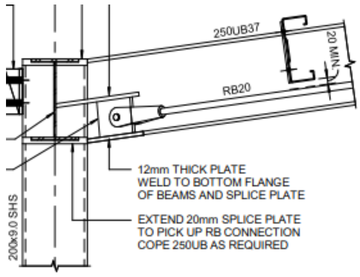
Table 1: Shear-induced settlement with SSI

Shear-induced settlement with SSI	SLS	ULS
Differential settlement	40 mm	45 mm
Floor slope over 6m distance	0.25%	0.4%

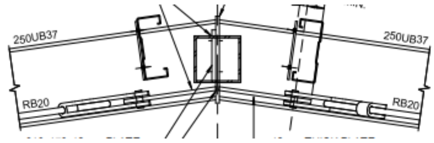
Differential settlement occurs mainly between ridge and eaves locations where the roof tie rod bracing are connected. To allow for this relative movement, connection detailing involved horizontal cleats to one end and vertical cleats to the other end to receive the proprietary rod bracket. Shop-welded column-rafter stubs with bolted splice connections away from the knee joint, avoids site welding and ensures fast installation. Typical connection detailing is shown in Figure 5.



(a) shop-welded column-rafter connection, two directions



(b) roof tie rod connection at column, to vertical cleat

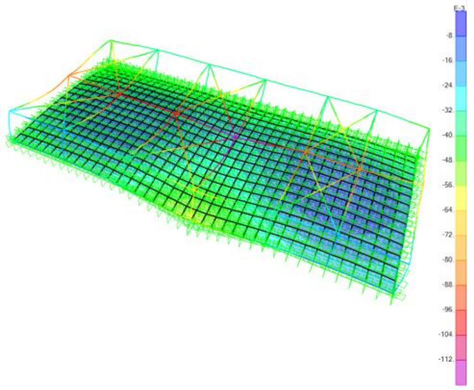


(c) roof tie rod connection at ridge, to horizontal cleat

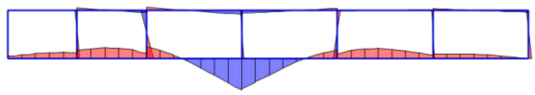
Figure 5: Typical connection details

6.3 SSI with ULS 'full' differential settlement

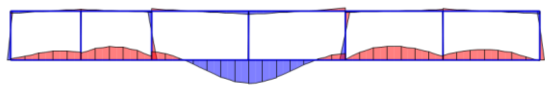
For ULS strength design, 60mm settlement was applied to an internal column and a corner column. This was applied in conjunction with earthquake loading in both directions, which then includes shear-induced settlement to account for 'full settlement'. Figure 6 shows results for this load case (load case b, Table 2).



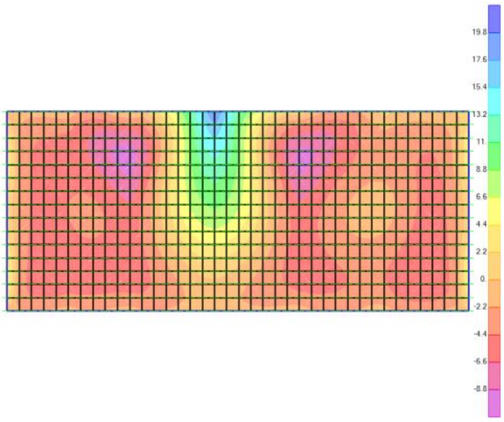
(a) Global settlement, mm due to 60mm applied settlement to column



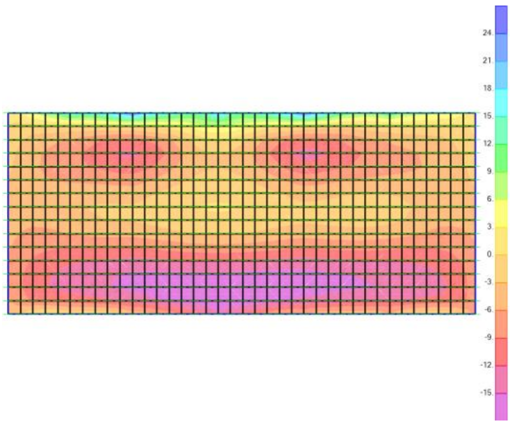
(b) flexural demand due to applied settlement and earthquake loading, 270 kNm max at ground beam



(c) flexural demand due to applied settlement and static load, 145kNm max at ground beam



(d) slab moment, kNm/m in long direction, -9kNm to 20kNm



(e) slab moment, kNm/m in short direction, -15kNm to 24kNm

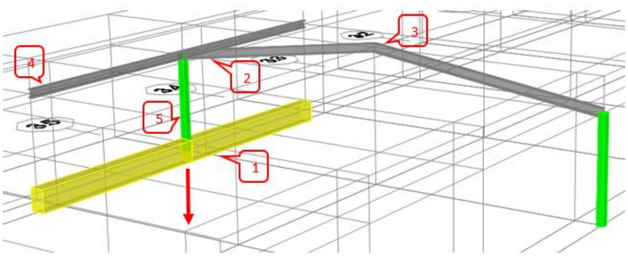
Figure 6: Applied settlement 60mm to internal column

Table 2 indicates the results for SLS, ULS and MCE load cases applied in conjunction with differential settlement. The MCE is based on a return period of 2500 years with differential settlement factored up from ULS based on R factors from NZS1170.5.

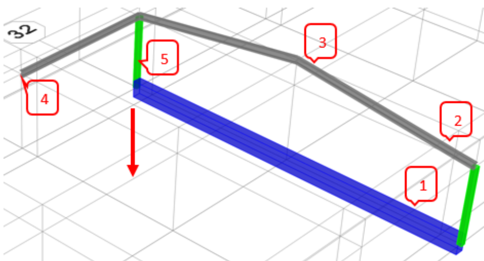
Demand-capacity ratios for the first earthquake loading with differential settlement indicate the capacity of the system is governed by the ground beams and slab. The superstructure elements can withstand up to MCE level of shaking. Element sizing appear to be conservative however the design is governed by static loading given the lightweight construction.

Table 2: Demand/Capacity Ratios for Load Cases

	(1) Ground beam	**(2) Rafter @knee**	**(3) Rafter @ridge**	**(4) Eaves beam**	**(5) Column**	**Slab**	**(6) Drift**
Capacity ϕM_n	229 kNm	130 kNm	130 kNm	48 kNm	188 kNm	35 kNm	-
Load case			**Demand-Capacity Ratio**				
Fixed base, ULS Seismic	0.30	0.77	0.32	0.63	0.66	-	0.9%
(a) SLS Seismic with 30mm differential settlement							
Internal column	0.65	0.58	0.34	0.31	0.40	0.71	0.3%
Corner column	0.47	0.38	0.31	0.21	0.38	0.43	0.2%
(b) ULS Seismic with 60mm differential settlement							
Internal column	0.33	0.77	0.31	0.54	0.53	0.89	1.2%
Corner column	0.70	0.79	0.35	0.42	0.55	0.80	1.0%
(c) MCE with 83mm differential settlement							
Internal column	0.43	0.86	0.27	0.92	0.60	1.00	1.7%
Corner column	0.82	0.68	0.38	0.42	0.62	0.80	1.5%



Key plan – applied settlement to internal column



Key plan –applied settlement to corner column

As a consequence of Canterbury earthquakes, more refined methods and guidelines have been developed (NZSEE, 2015), however many buildings may be subjected to tolerable displacement settlements that can affect the overall structural performance of the building, i.e. post-earthquake percentage of New Building Standard (%NBS). As a further preliminary investigation, the authors wanted to simulate this scenario. With the structure distorted from the ULS earthquake and differential settlement, two further load cases are applied to assess the post-earthquake residual capacity of the elements:

- Second SLS earthquake, plus differential settlement (total differential settlement 60mm + 30mm) at corner column and internal column respectively (load case d, Table 3)
- Second ULS earthquake, plus differential settlement (total differential settlement 2 x 60mm) at corner column and internal column respectively (load case e, Table 3)

Table 3: Demand/Capacity Ratios for Post-ULS Earthquake on Distorted Structure

	(1) Ground beam	**(2) Rafter @knee**	**(3) Rafter @ridge**	**(4) Eaves beam**	**(5) Column**	**Slab**	**(6) Drift**
Capacity ϕM_n	229 kNm	130 kNm	130 kNm	48 kNm	188 kNm	35 kNm	-
Load case			**Demand-Capacity Ratio**				
Fixed base, ULS Seismic	0.30	0.77	0.32	0.63	0.66	-	0.9%
(d) 2nd SLS Seismic with further 30mm differential settlement (60mm+30mm)							
Internal column	0.56	0.81	0.28	0.90	0.56	0.91	1.6%
Corner column	0.73	0.62	0.36	0.25	0.57	0.80	1.4%
(e) 2nd ULS Seismic with further 60mm differential settlement (60mm+60mm)							
Internal column	1.13	1.00	0.24	1.56	0.69	1.09	2.5%
Corner column	0.98	1.08	0.84	0.46	0.74	1.03	2.2%

Analyses indicate that the structural capacity of the elements are exceeded with the second design level earthquake (load case e), however they would be within their probable capacities when ignoring design strength reduction factors Φ. Drift at 2.2% means that the failure mechanism will be yielding of the base-plates (refer Fig 4). Upgrading of the eaves beam would enable the superstructure to withstand the second ULS event plus further differential settlement.

7 CONCLUSIONS

Differential settlement is a critical load case in assessing the post-earthquake residual capacity of the structural system given that the response of the structural system is distinct between settlement, and static and lateral load cases.

The PEER 2005/04 report provides a practical method for modelling SSI in order to assess differential settlement. SSI modelling also enabled assessment of the post-earthquake residual capacity of elements due to settlement. A limitation of modelling is damping due to the liquefied soil which would result in lower response, which was not considered.

The design intent for the moment resisting frame with perimeter columns and foundation beams with a solid internal slab, is for any repair and relevelling to mainly involve the perimeter beam and slab which are easily accessible. The robustness of the system is demonstrated with application of a second design event in conjunction with differential settlement.

The MOE structural and geotechnical guide provides an example for assessing differential settlement, which mainly focuses on the superstructure. Further consideration should be given to including SSI in assessing the impact of differential settlement on both the superstructure and substructure.

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