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Seismic assessment including soil-structure interaction of a three-storey building with piled foundations on soft soil

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ABSTRACT

The use of soil-structure interaction (SSI) results in period lengthening of the structural system, with reduction in seismic demand. This paper presents a case study of a detailed seismic assessment of a three-storey, nominally ductile concrete building, with piled foundations on soft soils that are prone to liquefaction. The aim is to assess the seismic capacity incorporating SSI with sensitivity analysis using p-y derivation methods from the NZ Transport Agency (NZTA), Federal Highway Administration (FHWA) and Naval Facilities and Engineering Command (NAVFAC). Site observation of settlement was used to correlate with the adopted structural period from period lengthening. For forward prediction analysis where site behaviour is not known, the structural period may be adopted based on the largest individual modal mass from eigenvalue analysis.

1 INTRODUCTION

A detailed seismic assessment of a three-storey, nominally ductile concrete building with piled foundations on soft soils that are prone to liquefaction is carried out to estimate the seismic capacity in % New Building Standard (%NBS), and assess localised failure modes of the building. Analyses were undertaken using a simplified approach in which piles were represented by pinned supports, and a more complex approach considering the influence SSI. Effects of SSI were incorporated into the model using p-y derivation methods from NZTA, FHWA and NAVFAC; the utilisation of these methods allowed for investigation of the model's sensitivity to varying lateral soil stiffness.

Analysis results were compared to determine whether the simplified pinned support approach is sufficient for assessments or if more computationally extensive SSI models are warranted. SSI of foundation beams was also modelled using a standard beam-on-nonlinear-Winkler-foundation approach. Linear dynamic analysis

in SLS and ULS seismic design cases were carried out to investigate the effect of SSI throughout all foundation elements of the building. The effect of modelling pinned supports versus consideration of SSI is discussed with reference to the case study. The structural period due to period lengthening from SSI is adopted based on correlation with site observation of settlement and based on the dominant individual modal mass from eigenvalue analysis.

2 BUILDING DESCRIPTION

The building was designed in 1964, is three storeys high and rectangular on plan. It has overall dimensions of 35.5 m x 6 m approximately. Storey heights are roughly 2.7 m giving an overall building height of 8.1 m. Viewed externally the elevations consist of block veneer with window openings and exposed bands of concrete beams. The roof is of lightweight construction comprising timber framing and metal cladding. The structural model as per Figure 1 provides an overview of the building form.

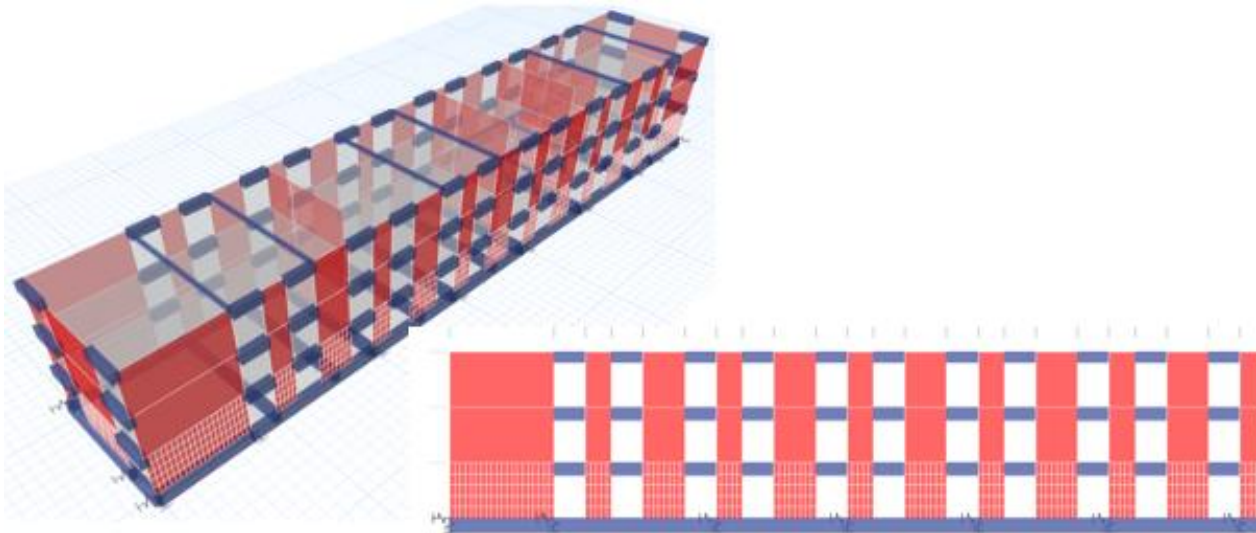


Figure 1: Structural model – 3D view and long elevation

2.1 Structural System

The structural system consists of reinforced concrete walls and reinforced blockwalls with non-ductile detailing. The concrete walls are 150 mm thick and singly reinforced with 9 mm rebar at 190 mm spacing each way, while blockwalls are lightly reinforced with 12.7 mm and 19 mm rebar vertically. Lateral loads are resisted in both directions by the reinforced concrete walls and reinforced blockwalls, via diaphragm action of the suspended concrete slabs.

Reinforced concrete beams typically located at the long elevations, support concrete slabs at each floor. Concrete beams are 300 x 650 mm with 2-19 mm bars top and bottom typically. 150 mm thick concrete slabs have been detailed as two-way spanning with 16 mm bars at 100 mm spacing. Slab reinforcement is anchored into the beams and structural walls.

The substructure consists of ground beams and concrete piles. Ground beams at 300 x 700 mm size are reinforced with 2-19mm bars top and bottom and 6mm stirrups at 375 mm centres. The ground beams support the load-bearing blockwalls above, for this reason, they are key elements to be assessed. Piles are 400 mm square, 7.5 m long with varying reinforcement and well confined with 6 mm stirrups at 75 mm centres. Figure 2 provides a snapshot of the record information. The ground slab is 100 mm thick, with a partial connection to the ground beams via a notch/key detail. The slab is assumed to be ground-bearing. No information was available on the ground slab reinforcement.

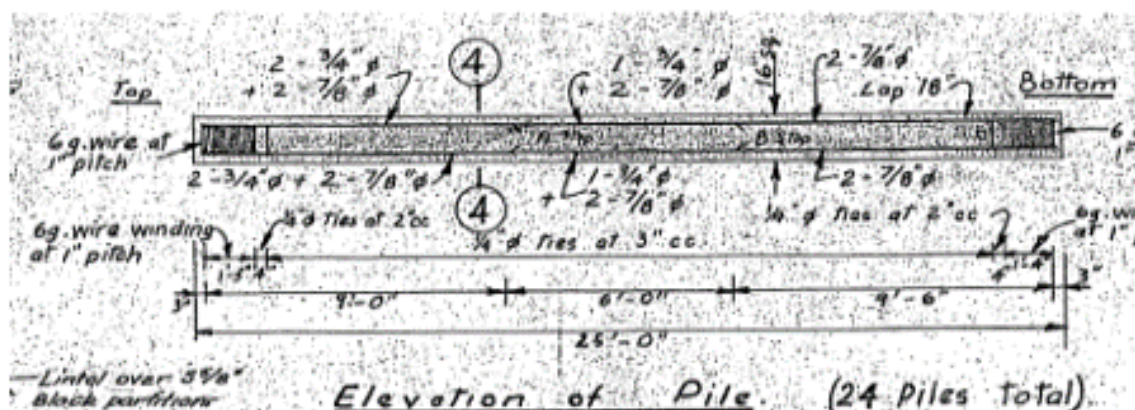


Figure 2: Record information

2.2 Geotechnical Conditions

Site specific geotechnical information consist of historic drawings, which indicate that two boreholes were undertaken at the eastern and western ends of the building, circa. 1964. The boreholes were terminated at depths of 8.5 m and 6.1 m. Due to the previous boreholes terminating at shallow depths relative to the pile lengths, a borehole was undertaken to a depth of 15 m to ensure sufficient information throughout the length of the pile and the zone of influence below the toe. This borehole was used in conjunction with Cone Penetration Test (CPT) records situated within 50 m of the site, sourced from the New Zealand Geotechnical Database (NZGS). The generalised geotechnical profile underlying the site and inferred engineering soil parameters are summarised in Table 1.

Table 1: Soil parameters used in design

Unit	Material Description	Typical Depth to Top of Layer (m)	Unit Weight, γ (kN/m ³)	Friction Angle, Φ (°)	Cohesion, c' (kPa)	Undrained Shear Strength, S_u (kPa)
1	SAND with minor silt	0.0	16 – 17	26 – 28	0	-
2	SILT with some sand. Interbedded peat layers	2.2	15 - 16	24 - 26	0 - 5	12 - 25
3	SAND with some gravel and trace silt	5.6	17 – 19	30 - 32	0	-
4	GRAVEL with some cobbles and minor sand	10.0	18 - 19	34 – 36	0	-

The liquefaction hazard at the site has been assessed for design earthquake scenarios derived in accordance with NZS 1170.5 (Standards New Zealand, 2004), and updates and clarifications to the Ministry of Business, Innovation and Employments residential guidance (MBIE, 2014). The earthquake scenarios were derived assuming a design life of 50 years and, based on site records, a groundwater table depth of 2 m. Liquefaction potential of the site soils was based on the results of the recorded SPT blow counts undertaken in the borehole, and CPTs near the site. The analysis was undertaken using the methodology developed by Boulanger & Idriss (2014) and considered a 0.19g/M6.0 SLS design event and 0.35g/M7.5 ULS design event; soil unit 3 was identified as being susceptible to liquefaction for both SLS and ULS design events.

Free field ground subsidence was estimated based on the methodology of Zhang et al.(2002). As the onset of liquefaction occurred at a Peak Ground Acceleration (PGA) less than the SLS design event, free field ground subsidence is anticipated to be 20 mm to 80 mm in a SLS design event and in the order of 100 mm to 250mm in an ULS design event.

From the pile details noted in the original drawings, the 7.5 m piles were founded into soil unit 3, a layer of material considered susceptible to liquefaction. While liquefaction at the base of the pile is expected to be minimal in a SLS design event, a large proportion of this unit could liquefy under an ULS design event; resulting in substantial decreases to the end bearing capacity of the piles. Furthermore, the liquefiable layer at the base of the pile could result in downdrag, inducing an increased axial force and the potential for large magnitudes of settlement.

The existing capacity of the piles was estimated based on soil parameters derived from the site investigations and correlations to SPT 'N' blow counts. In an ULS seismic event in which liquefaction was triggered, due to downdrag the net skin friction resulted in increased axial loads on the pile. Therefore, in an ULS design event, the existing piles are reliant on end bearing, which is also degraded due to the effects of liquefaction. The residual strength of the liquefied material at the base of the pile was estimated from empirical correlations published by Idriss and Boulanger (2008).

Effects of SSI and lateral pile loads were incorporated into the model using a range of force-displacement (p-y) methods; NZ Transport Agency Research Report 553 (Murashev et al., 2014) , Federal Highway Administration (FHWA, 1986) and Naval Facilities and Engineering Command (NAVFAC, 1986).

Each of the p-y derivation methods were used to generate bi-linear soil spring profiles. Report 553 directly relates the raw, uncorrected, SPT 'N' blow count to the coefficient of subgrade reaction. NAVFAC and FHWA both assume that the modulus of subgrade reaction increases linearly with depth and are based off the constant of horizontal subgrade reaction. As empirical relationships based on the SPT 'N' blow counts were used to determine the relative density, this causes a correlation between all methods. By multiplying the modulus of subgrade reaction by the spring spacing the stiffness of the spring was obtained. Differences in each of the methods underlying assumptions, and derived empirical formula, result in variations to the calculated spring stiffnesses.

The yield force of the soil spring, corresponding to the ultimate pressure from the soil on the pile, was determined by Report 553 as a function of the pile diameter, spring spacing, the Rankine passive pressure and the scaling factor accounting for increased pressure on a single pile from wedge effects. As the NAVFAC method does not account for limiting the yield force the same methodology as Report 553 was adopted. The yield force from the FHWA was determined by the minimum value of either a shallow wedge failure or a deep horizontal plastic flow failure mechanism.

Only the method from Report 553 accounts for the degradation of soil strength and stiffness due to liquefaction. For liquefied soils the piles springs were reduced by a degradation factor (β_L) of 0.05 and the yield strength of the soil was reduced to the residual strength of the liquefied soil, 30 – 50 kPa. This methodology was adopted for the FHWA and NAVFAC p-y derivation methods as well.

A sensitivity analysis was undertaken considering variations to the geotechnical properties. As each of the methods produced different values, by considering the two methods which give the upper and lower bound results, minor variations pertaining to the model's sensitivity were encompassed. Figure 3 summarises the static spring stiffness and limiting yield force of the soil along the pile for each of the considered methods.

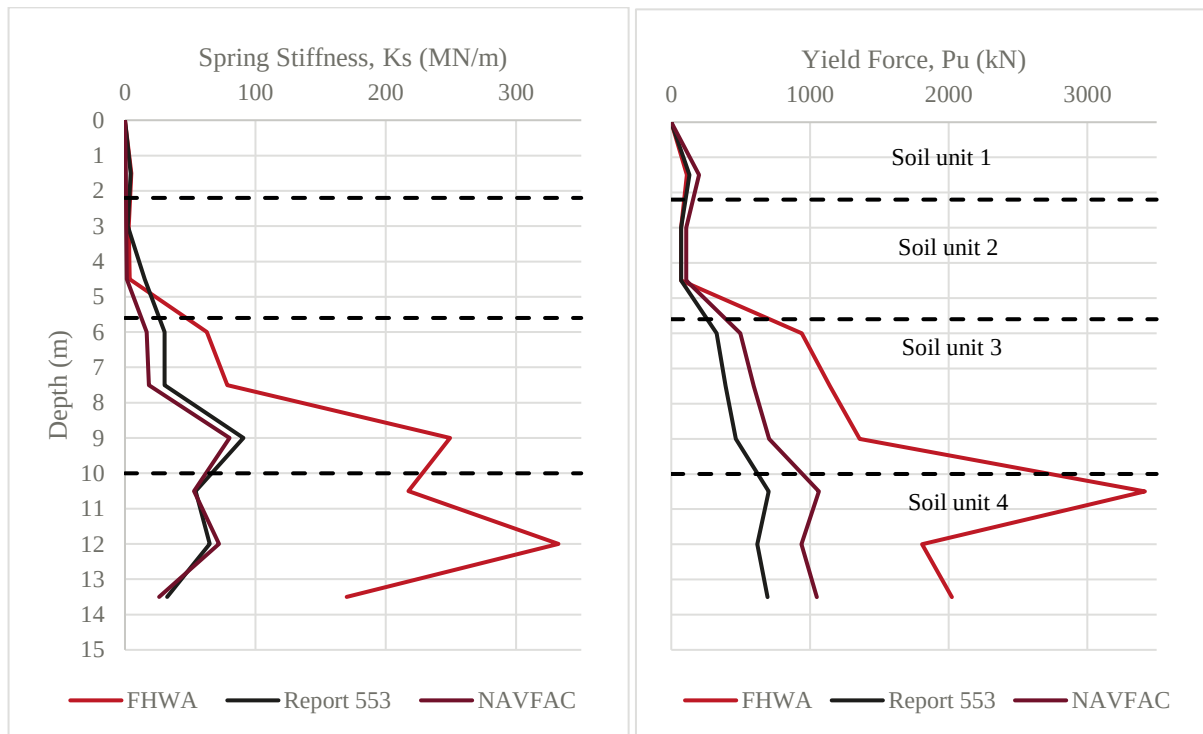


Figure 3: Static soil-pile spring stiffness and limiting yield force

3 ASSESSMENT

Reference was made to The Seismic Assessment of Existing Buildings (NZSEE, 2017). The following design parameters were adopted: Importance level 2 with a 50-year design life; site soil class D, clause 3.1.3 NZS 1170:2002; hazard factor, $Z = 0.3$; expected concrete compressive strength, $= 30 \text{ MPa}$; effective stiffness (ULS), wall $I_e = 0.4I_g$ from Table C6.6 NZS3101; and expected yield strength D rebar $= 270 \text{ MPa}$

A displacement-based assessment using the Simple Lateral Mechanism Analysis (SLaMA) procedure from NZSEE The Seismic Assessment of Existing Buildings (NZSEE Guidance) was used. This was used to gain an understanding of the hierarchy of failure in relation to the superstructure and substructure – whether wall yielding occurs before pile yielding. A comparison with the Force-based procedure was made based on system ductility and hierarchy of failure estimated from SLaMA.

In both methods, soil structure interaction (SSI) was considered, given the soft ground conditions with liquefaction anticipated closely after SLS level of shaking. Simplified models for representing the foundation elements (Winkler-spring) were considered appropriate as opposed to finite element modelling due to the level of uncertainty in ground conditions across the footprint which may not be captured fully from geotechnical site investigations.

3.1 Displacement-based Assessment

The following methodology based on the NZSEE Guidance was applied:

- Section analyses of the reinforced blockwalls and reinforced concrete walls were performed to obtain global Force-Displacement curves, ignoring torsional effects.
- From the pushover analysis (Figure 4) system displacement ductility in the short direction was assessed at 2.5 (41 mm ultimate/15 mm yield displacement) and in the long direction 1.7 (71 mm ultimate/41 mm yield displacement). A system displacement ductility of 1.5 was adopted for both directions. This is considered reasonable given the non-ductile detailing of blockwalls and concrete walls.

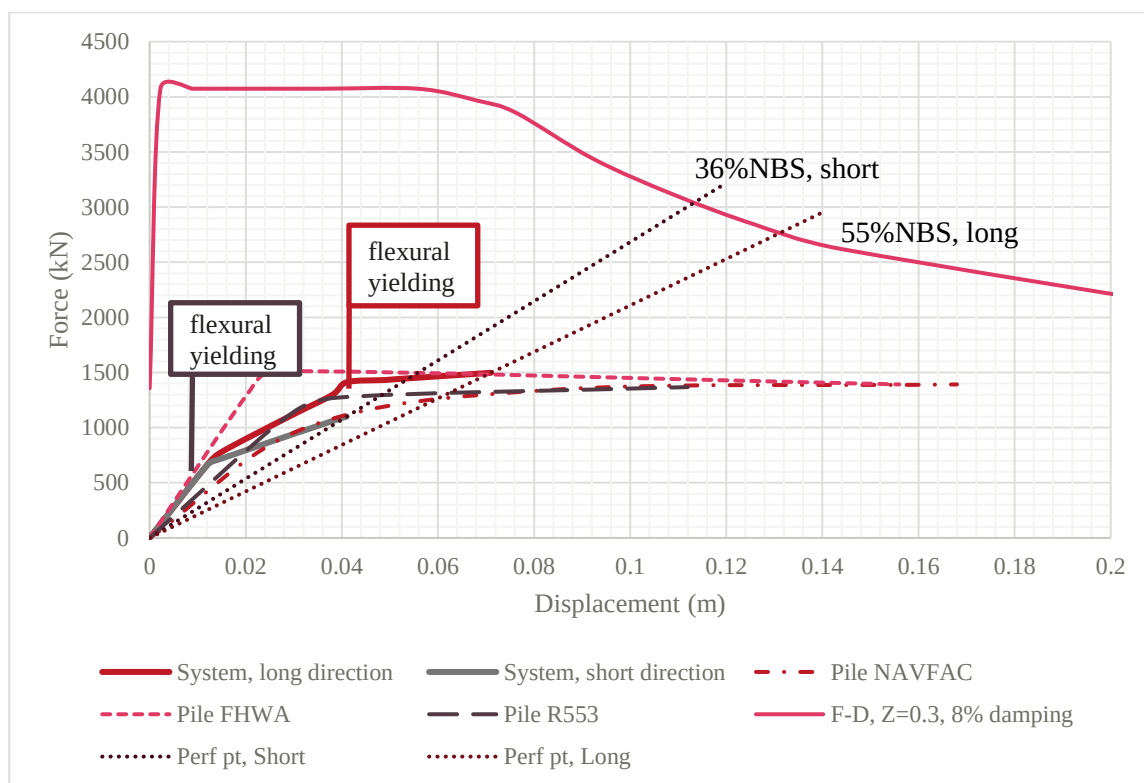


Figure 4: F-D vs capacity curve

- Piles were idealised as pin support for the ULS case. This gave a lower bound to the assessed seismic capacity of the superstructure elements.
- Upper bound values were obtained by considering SSI with soil in the non-liquefied state (SLS/static soil condition) with peak inertial shear loads (ULS demand) assumed to occur prior to triggering of liquefaction. For lateral soil stiffness, soil-pile interaction was analysed using the derived p-y values.
- A soil-pile-pushover analysis was undertaken with bilinear lateral spring supports defined at 1.5 m lengths, with hinges defined at these spacings, and pin support idealised at the pile tip. Spring spacing of 1.5 m was chosen as it generally coincides with the change in pile rebar densities and the soil strata. Table 2 summarises the yield displacement and hinge location for the p-y methods. The pile shear capacity is governed by section capacity and is consistent for all three p-y methods.

Table 2: Pile yield displacement and location of hinge for p-y methods

p-y Method	Yield displacement, δ_y (mm)	Location of hinge
NAVFAC	33	Pile head
FHWA	21	6m depth
Report 553	28	Pile head

The yield displacement of the pile depends on the p-y method used. The Report 553 and NAVFAC methods were generally consistent and progressed in the analyses. The FHWA resulted in a spring stiffness twice that of the other methods, likely more conservative and was not considered further. The Report 553 and NAVFAC methods indicated wall yielding before pile yielding. Based on these methods, the seismic

capacity of the building is considered to be governed by the superstructure. Plotting the 100%ULS force-displacement (F-D) curve based on 8% system damping, the seismic capacity in the short and long directions were estimated at 35%NBS and 55%NBS respectively.

3.2 Force-based Assessment

- Vertical pile stiffness was based on pile capacity in end bearing and shaft friction, with full capacity assumed to be mobilised at 10% pile diameter. Ignoring a strength reduction factor of 0.5, upper bound values in compression and in the uplift condition at 410 kN and 145 kN (shaft friction only) respectively were used, given that these estimated capacities were close to gravity reactions.
- In the liquefied case, low end bearing values and shaft friction downdrag were predicted from the geotechnical analysis. This condition was idealised as settlement of the substructure as a whole, with support provided by piles in end bearing, and foundation beams mobilised in bearing. The line spring is derived from pile downdrag acting over the bearing area of the ground beam.
- The ground bearing slab with nominal notch connection to foundation beams was also considered to contribute in the settlement and downdrag case. An area spring stiffness corresponding to its self-weight was applied while checking its shear capacity at the notch is not exceeded.
- A force-based linear dynamic analysis was performed to compare with the results from the non-linear process and is considered appropriate for the assessment given its expected nominally ductile behaviour. The modal response spectrum analysis (RSA) was performed with scaling factors to achieve a minimum demand obtained from an equivalent static analysis (ESA), as per the NZS1170.5 requirement.
- A 3D analysis model was created using ETABS 2016 software. The assessed displacement ductility of 1.5 was applied in the RSA model. Pin base condition was considered, and SSI applied using spring stiffnesses for soil-pile and ground beams derived as above.

The pin base condition resulted in significant reactions exceeding the static capacity of piles, pointing to its unsuitability in modelling existing conditions. Modelling of SSI including the ground beams and slabs, gave realistic reactions due to distribution of loads to all available elements. SSI resulted in period lengthening as presented in Table 3, where demands were lower than pin base condition on the order of 10% to 20%.

Table 3: Eigenvalue analysis results

Mode	Structural Period, Pin Base(s)	Individual modal mass	Structural Period, SSI ULS upper bound (s)	Structural Period, SSI SLS static (s)
1, short direction	0.64	0.451	1.38	0.95
2, torsion	0.58	0.42 short, 0.87 torsion **	0.91	0.77*
3, long direction	0.48	0.859	0.75	0.65

*Period adopted based on correlation between settlement observed on site and analysis model

**Largest participating individual modal mass

Site observation of settlement was used to form a correlation with SSI analysis results. Site observations by others attributed approximately 50 mm earthquake-induced differential settlement. A period of 0.77 s based on SSI static springs was adopted where the analysis result was in close agreement with reported settlement.

Further, analyses returned the largest individual modal mass of 0.87 for the structural period of 0.77s, indicating this mode to be the dominant mode for which the period can be reasonably adopted. This suggests

that for forward prediction analysis where the observed behaviour (settlement) is not known, the structural period can be adopted based on the largest individual modal mass participation.

The force-based assessment using SSI and period lengthening resulted in seismic capacities of 25%NBS and 50%NBS in the short and long directions respectively, which was generally in agreement with the SLaMA method.

4 CONCLUSION

A pin base condition resulted in improbable reactions exceeding existing static pile capacities. The use of SSI provides a more realistic assessment of the structural system particularly for soft soil and liquefaction-prone ground conditions. A key outcome of SSI was period lengthening of the structural system resulting in lower demands.

The displacement-based assessment with SLaMA incorporating capacity curves for piles allowed a quick assessment of hierarchy of failure between the substructure and superstructure.

SSI involved a geotechnical sensitivity analysis given the variability of ground conditions and the various p-y derivation methods used in practice. Similarly, structural analysis requires sensitivity analysis using various spring stiffnesses. The analyses progressed with the Report 553 and NAVFAC methods as these generally gave consistent results.

A correlation between site observations and analysis results can be used to inform the parameter to adopt from sensitivity analysis. Settlement or differential settlement that is similar to the analysis results point to the suitability of the spring stiffness used.

For forward prediction analysis where site observation (e.g. settlement) is not available, the structural period adopted from SSI sensitivity analysis may be based on the largest participating individual modal mass from eigenvalue analysis. Further studies are required to validate this assumption.

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