
Influence of soil–structure interaction and nonlinearity on seismic performance of a concrete substation

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ABSTRACT

Detailed seismic assessments in New Zealand often adopt a fixed-base assumption and linear material behaviour when modelling existing structures. However, for infrastructure sited on soft ground, these assumptions may not reflect actual performance. In this study, we present a nonlinear time-history analysis of a 1940s reinforced concrete substation building owned by Vector Ltd, with a basement embedded in soft soils and subject to realistic seismic loads. Undertaken collaboratively with Vector, the study highlights how infrastructure owners can adopt advanced data-driven seismic-assessment processes that align resilience, asset-safety and sustainability goals. A detailed finite-element model was developed, incorporating soil springs along the basement walls to explicitly simulate soil-structure interaction. Comparative analyses between fixed-base and flexible-base conditions quantify the influence of soil-structure interaction (SSI) on building response. To optimise utilisation of existing structural capacity and avoid unnecessary strengthening works, critical members were modelled with nonlinear behaviour. The effect of element nonlinearity on global seismic performance is examined. The results underscore the importance of integrating SSI and structural nonlinearity into detailed seismic assessments of existing buildings. The work also demonstrates how program-efficient client–consultant collaboration enables an assessment-led solution, which protects the operational continuity of critical energy-supply infrastructure while minimising construction impacts, embodied carbon, and disruption within the urban environment.

1 INTRODUCTION

Seismic assessment practice in New Zealand follows a tiered hierarchy of analytical complexity, as defined by NZS 1170.5 (SNZ, 2004) and the Engineering Assessment Guidelines (the guidelines) (2017). It is recommended by the guidelines that a Simple Lateral Mechanism Analysis (SLaMA) is used as a baseline tool to establish the expected failure hierarchy and available global ductility of a structure. Force-based procedures, such as Equivalent Static Analysis (ESA) and Modal Response Spectrum Analysis (MRSA), are then undertaken using this assumed ductility to determine seismic ratings. However, these approaches fail to address commonly encountered issues such as buildings with mixed ductility systems, or concentrations of ductility demand, often resulting in conservative estimates of structural performance. To capture the inelastic behaviour, Nonlinear Time-History Analysis (NLTHA) has been increasingly recognised (Hashemi et al.,

2024). By explicitly modelling material nonlinearity, NLTHA provides more realistic insight into both local and global response, accounting for changing member stiffness, additional damping and load redistribution in the structure following the initiation of non-linear mechanisms.

The subject of this study is a historical reinforced concrete substation. The basement level of the building is embedded in soft soil and supported by small-diameter gravity piles, making consideration of soil–structure interaction (SSI) essential. Previous research, e.g. by Wolf and Oberhuber (1985), identified three primary SSI effects: elongation of the system period, kinematic interaction that modifies incoming seismic waves, and additional energy dissipation through radiation damping and soil hysteresis. In practical engineering, however, it is not feasible to explicitly capture all these mechanisms. Since several studies (e.g. by Rivera et al. (2008) and Segaline et al. (2022)) showed that kinematic interaction and foundation damping could reduce the seismic input compared to free-field excitation, these effects were intentionally neglected to ensure a conservative and practicable analysis.

In this study, the influence of structural nonlinearity was investigated by comparing the responses from nonlinear (NL) analyses with those from an elastic linear (EL) model. Further comparisons between fixed-base (FB) and soil-supported models were undertaken to quantify the effect of soil-structure interaction (SSI) on both global and local structural response. The influence of linear and nonlinear soil springs is also examined. Finally, seismic ratings obtained from elastic and nonlinear assessments were compared, and the elimination of unnecessary strengthening works (together with the associated societal impacts) resulting from the adoption of an advanced, assessment-led approach is discussed.

2 BUILDING AND SITE DESCRIPTION

The building considered is a three-storey, L-shaped reinforced concrete substation located in the Auckland CBD. The gravity-load-resisting system of the structure comprises a mix of steel beams and reinforced concrete frames and walls, while its lateral resistance is provided by a combination of reinforced concrete wall piers, spandrels, shear walls, and moment frames, as shown in Figure 1 and Figure 2(a). The basement of the building is embedded in soil and is supported on reinforced concrete piles, see Figure 2(b).

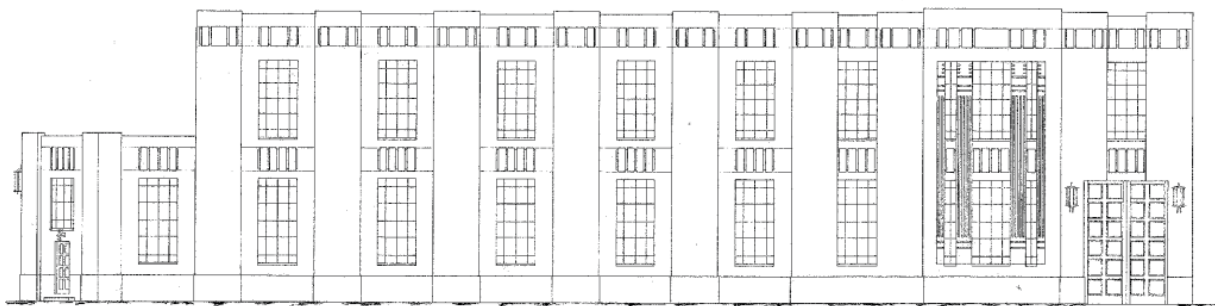


Figure 1: Structural elevation

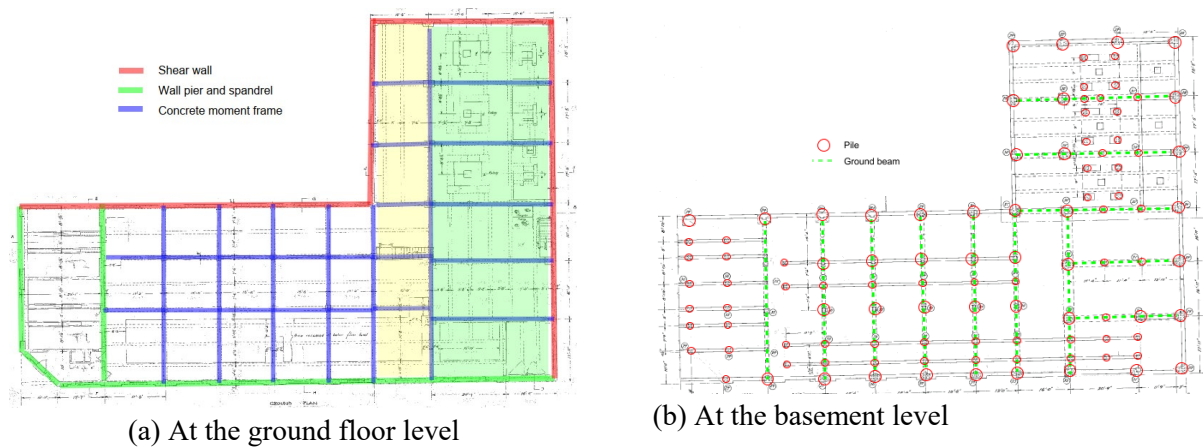


Figure 2: Structural plans

Beyond its structural and geotechnical complexity, the substation has notable societal and operational importance. Unlike typical buildings with high occupancy, where seismic assessment focuses primarily on life safety to occupants, the primary driver for this facility is operational and functional continuity, providing an uninterrupted electrical supply to the Auckland CBD. As a critical infrastructure asset, any significant seismic disruption would trigger widespread economic and life safety consequences for commercial, transport, and emergency services.

3 METHODOLOGY

3.1 Modelling of the Structure

To simulate the seismic response of the building, a detailed model was developed using finite element analysis (FEA) software. The assessment followed a progressive modelling approach consistent with the guidelines. An initial linear-elastic analysis was undertaken on the structure with an assumed FB condition (see Figure 3) and subjected to 100% Ultimate Limit State (ULS) seismic loading for Importance Level 3 (IL3). Three sets of bi-directional excitations were applied to the longitudinal and transverse directions alternatively. This analysis, together with a SLAMA, was used to understand the dynamic characteristics, base shear demands, and primary failure mechanisms of the building. To account for flexural cracking, effective section stiffnesses were implemented for reinforced concrete members in accordance with NZS 3101 (SNZ, 2006).

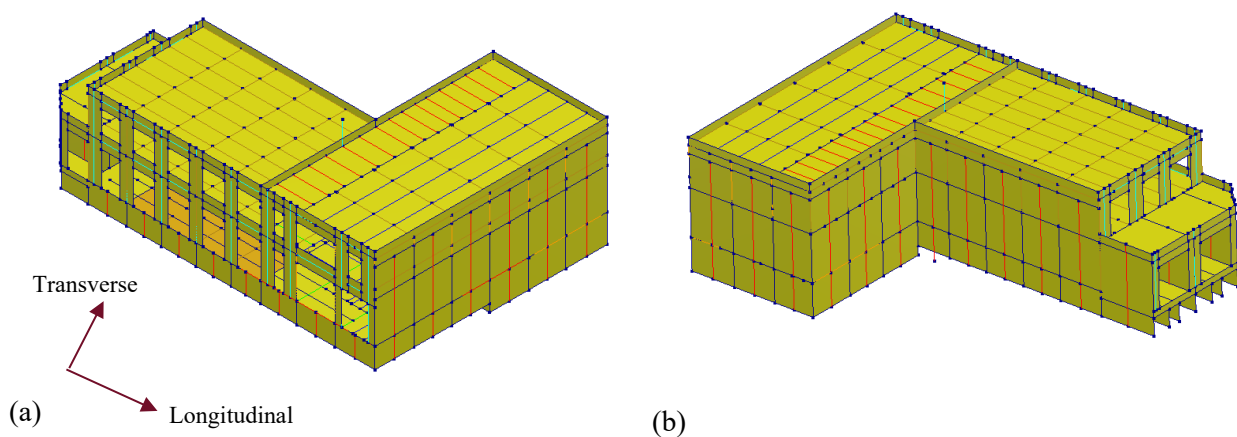


Figure 3: Numerical model (a) Eastern and (b) Western sides

The initial assessment identified that the seismic rating of the building was governed by the flexural failure of wall piers on the eastern face, which is a ductile failure mechanism. Tension demand at the interior corner of the L-shaped diaphragm was also identified as a structural weakness, with stress concentrations that could be more appropriately simulated through nonlinear diaphragm modelling. On this basis, a refined NL model was subsequently developed. Table 1 summarises the concrete material properties used in modelling. The compressive strength and elastic modulus were obtained from the core sample testing, while the tensile strength was calculated in accordance with Section C5 of the guidelines. The concrete nonlinearity was modelled using the rotating crack model developed based on the Maekawa cracked concrete curve (Maekawa et al., 2003), paired with the JSCE tension softening (JSCE, 2010). The strain-stress curve of the Maekawa model is shown in Figure 4(a).

Table 1 Concrete material properties used in modelling

Concrete	Compressive strength f'_{c_prob} (MPa)	Tensile strength f_{t_prob} (MPa)	Fracture energy* G_F (N/m)	Elastic modulus E_c (GPa)	Average density ρ_c (kg/m ³)
Col. and wall	17.6	2.31	130	15.6	2400
Beam	20.5	2.46	133	20	2400
Slab	28.7	2.95	254	25	2400

* Fracture energy from CEP-FIP 2010 (FIB, 2013)

Given the smooth and straight reinforcement detailing in the wall piers and columns, their post-yield behaviour is governed by flexural rocking at the base. Consequently, a simplified reinforcement post-yield stress-strain relationship developed by Opabola et al. (2019) and Opabola and Elwood (2020) was adopted, as illustrated in Figure 4(b). The yield rotation (θ_y) and overturning rotation (θ_o) were calculated in accordance with the equations provided in Section C5 of the guidelines. The beams and walls are reinforced with hooked-end bars and were therefore assumed to follow conventional reinforcement stress-strain behaviour. Due to the use of smooth bars, strain hardening was conservatively neglected.

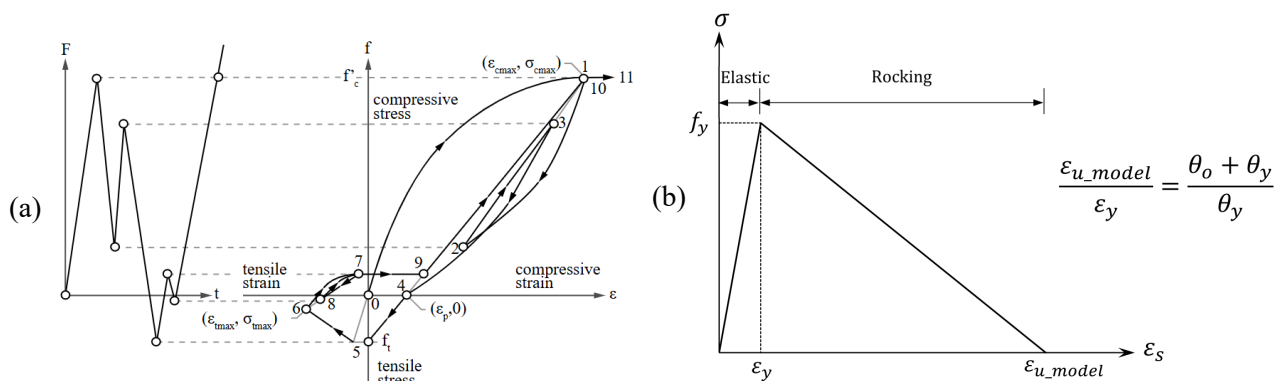


Figure 4: Nonlinear modelling: (a) Maekawa cracked concrete model and (b) reinforcement stress-strain curve for elements with smooth reinforcement

3.2 Soil-Structure Interaction

As mentioned previously, the building's basement level is embedded in soft, reclamation-era soils underlain by historical fill and harbour deposits. It was expected that SSI could have a significant influence on the seismic response. The lateral load was considered to be primarily transferred to the ground through passive soil resistance developed along the basement walls.

The interaction between the structure and the surrounding soil was modelled using area soil springs with nonlinear p - y relationships derived from site-specific geotechnical data. Please refer to Section 4.2 for a detailed summary of these parameters.

3.3 GROUND MOTIONS

Three sets of ground motion records were selected from the PEER (Pacific Earthquake Engineering Research Center) database to represent the seismic hazard. In accordance with NZS 1170.5 (SNZ, 2004), these records were scaled to a target spectrum representing an IL3 event (1/1000-year return period) for Subsoil Class C (Shallow Soil). The upper bound of the period range of interest was set at 1.3 times the fundamental period (T_1) per NZS 1170.5, while the lower bound was selected based on the period required to achieve 90% cumulative mass participation. This method follows the recommendations of Morris et al. (2019) and aligns with ASCE/SEI 41-17 (2017) requirements, ensuring that higher-mode effects are captured, which are particularly significant for irregular structures.

The selected motions were applied as bi-directional components. For each set of records, the principal and secondary horizontal components were rotated to align with the building's primary longitudinal and transverse axes. Figure 5 shows the target spectrum and scaled response spectra of the selected ground motions of the FB system.

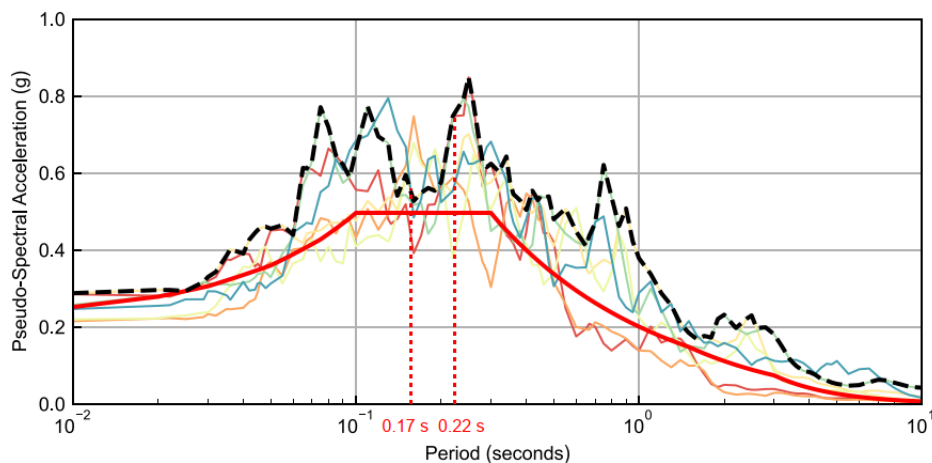


Figure 5: ULS scaled target spectrum and response spectra of selected ground motions

4 RESULTS AND DISCUSSION

4.1 Effect of Structural Nonlinearity

To investigate the influence of structural nonlinearity, a full NL model was developed, and its responses were compared with those obtained from the EL model. To isolate the effects of structural nonlinearity, only the FB support condition is considered in this section. Eigenvalue analysis indicates that the building is relatively rigid, with fundamental periods of 0.17 s and 0.22 s in the longitudinal and transverse directions, respectively.

Figure 6 compares the base shear demands of the EL and NL models. In the longitudinal direction, the NL model had a maximum base shear of 11,800kN, approximately 30% lower than that of the EL model (16,900kN). In the transverse direction, the reduction is more pronounced, with the NL model exhibiting a peak base shear of 12,800kN, approximately half of that obtained from the EL analysis (25,700kN).

The larger discrepancy in the transverse base shears can be attributed to the alignment of the elastic fundamental period (0.22 s) to the dominant period range of the input ground motions (see spectral ordinates around 0.22 s in Figure 5). As a result, the EL model is likely to experience resonance-like vibrations. In contrast, the development of material nonlinearity in the NL model alters the effective period and increases the energy dissipation through additional hysteretic damping, thereby disrupting resonance-like behaviour and reducing seismic demand.

These results highlight that for a building capable of developing NL behaviour, an elastic analysis could significantly overestimate its seismic demand.

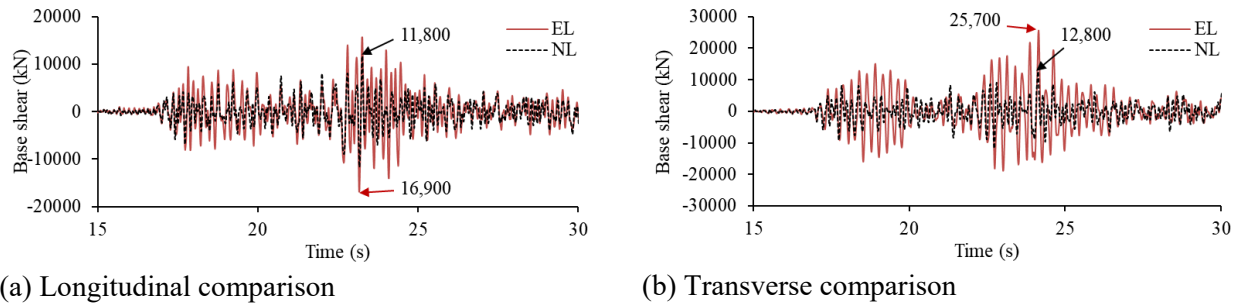


Figure 6: Comparison of the base shears of elastic and nonlinear models in the (a) longitudinal and (b) transverse directions under the critical record

4.2 Effect of SSI

4.2.1 Calibration of linear spring stiffness

Although site-specific p - y curves were available, the appropriate initial soil stiffness was not readily determined because soil stiffness, system period, base shear, and soil-structure displacement are interdependent.

To address this, an iterative procedure was adopted. A secant stiffness was first assumed to estimate the fundamental period, from which the ULS base shear was calculated in accordance with NZS 1170.5 (SNZ, 2004) and applied in a pushover analysis to obtain the resulting displacement relative to the soil. The soil stiffness was then adjusted based on this displacement, and the process was repeated until convergence. The iteration parameters are summarised in Table 2. Based on the converged results, the normal soil stiffness was taken as the secant stiffness corresponding to a displacement of around 50 mm.

Table 2: Summary of the soil spring iterations

Iteration	Direction	Assumed displacement* (mm)	Period (s)	Max. Base shear (kN)	Actual displacement* (mm)
1	Longitudinal	150	1.7	5,000	260
	Transverse	150	1.3	7,000	240
2	Longitudinal	90	1	7,500	110
	Transverse	90	0.8	9,000	160
3	Longitudinal	50	0.8	9,100	40
	Transverse	50	0.6	11,000	50

* The maximum displacement of the building relative to the surrounding soil

4.2.2 Influence of soil spring nonlinearity

To investigate the influence of soil spring nonlinearity, the result of the model with a NL spring is compared with that of the model with a linear spring. The nonlinear spring properties were derived from the site-specific p - y curves, while the linear stiffness was taken as the secant stiffness corresponding to the estimated soil deflection on the same p - y curves. An example of the nonlinear and linear soil springs is shown in Figure 7.

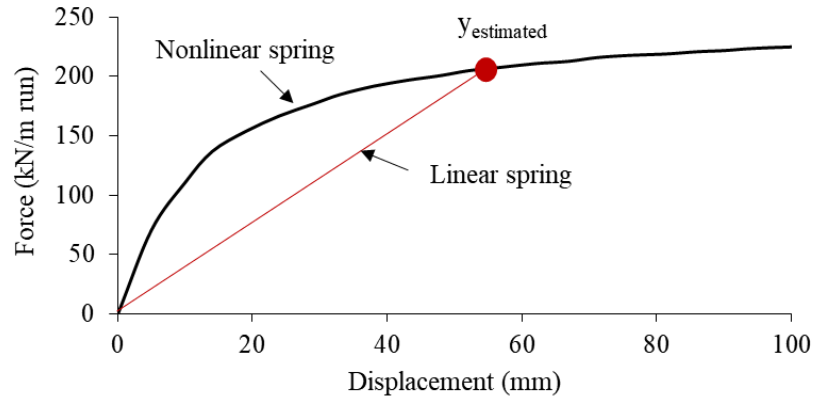


Figure 7: Comparison of linear and nonlinear soil spring curves

Table 3 compares the maximum base shears and displacements of the model with linear and nonlinear soil springs. The results were obtained using an elastic structural model subjected to critical bi-directional records at 100%ULS shaking intensity, with the principal component applied in the longitudinal (L) and transverse (T) directions.

As can be seen, although the nonlinear soil springs resulted in smaller maximum base shear demands, the base shear in each direction could be either larger or smaller depending on the SSI and ground motion characteristics. In addition, the model with nonlinear soil springs exhibited smaller displacements than the linear spring model. This observation indicates that, at the level of earthquake intensity considered, the nonlinear spring model had smaller maximum base shears and less building displacements in both directions compared to the model with the linear spring. It is noted that this trend may change at higher shaking intensities as soil nonlinearity becomes more pronounced.

Table 3: Time history analysis result of the model with linear and nonlinear soil spring.

	Principal direction	Directions	Linear spring	Nonlinear spring	NL/L ratio
Base shear (kN)	L	L	12,400	11,300	91%
		T	7,400	10,000	135%
	T	L	8,600	10,200	128%
		T	13,000	11,600	89%
Displacement* (mm)	L	L	66.7	51.5	77%
		T	89.2	65.8	74%
	T	L	58.3	50.4	86%
		T	72.1	57.6	80%

* The maximum displacement of the building relative to the surrounding soil

The above results show that the model with linear and nonlinear soil springs could have different seismic responses. This implies that linear springs could be insufficient to realistically represent the nonlinear behaviour of structures interacting with soil.

4.2.3 Influence of SSI including structural nonlinearity

To investigate the influence of SSI and structural nonlinearity on the global building behaviour, Table 4 compares the time history results from the elastic fixed-base model with those from the nonlinear model including SSI. The values in brackets denote the ratio of the NL-SSI model results to those of the elastic EL-FB model.

As shown in Table 4, considering SSI significantly elongated the fundamental system periods from 0.17 s to 0.80 s in the longitudinal direction and from 0.22 s to 0.60 s in the transverse direction. Owing to the substantially altered dynamic characteristics, the ground motions were rescaled based on the new period range of interest. On average, the base shears of the SSI model were approximately 76% and 67% of those of the fixed-base model in the longitudinal and transverse directions, respectively. Consideration of SSI was also associated with reduced structural deformations compared with the fixed-base case, indicating that the building behaves more like a rigid structure, due to the relatively soft surrounding soil

These findings reveal that for relatively rigid structures founded on soft soil, the seismic response is fundamentally different from that implied by fixed-base assumptions. Consequently, neglecting SSI will lead to an inaccurate assessment of the governing failure mechanisms for the structure.

Table 4: Time-history analysis results under 100% ULS critical records.

Responses	EL model with FB		NL model with SSI	
	Longitudinal	Transverse	Longitudinal	Transverse
Fundamental period (s)	0.17	0.22	0.8	0.6
Max. base shear (kN)	16,900	25,700	12,400 (73%)	14,500 (56%)
Average base shear (kN)	13,600	16,900	10,300 (76%)	11,400 (67%)
Structural deflection (mm)*	16	25	9 (56%)	12 (50%)

* Peak roof deflection relative to the basement

For the SSI model, total structural displacements related to soil under 100%ULS shakings were approximately 77mm and 78mm in the longitudinal and transverse directions, respectively.

The analysis of the SSI model was further extended to the Collapse Avoidance Limit State (CALS) by scaling the critical records to 150%ULS. In both directions, the total base shears only increased by about 10%. This result reflects the nonlinear soil response whereby effective system stiffness reduces with increasing loading. The corresponding maximum displacements relative to the ground were 113 mm and 107 mm, i.e. increases by 47% and 37%, respectively. These values remain within acceptable soil performance limits, confirming the robustness of the soil–structure system under severe shaking.

4.3 Assessment Result

A Detailed Seismic Assessment (DSA) was undertaken using both FB and SSI models, with the SSI assessment results independently peer reviewed. Table 5 lists the ratings of critical elements and the overall rating. The results indicate that neglecting SSI and structural nonlinearity leads to overly conservative seismic rating for this building.

Table 5: Summary of Assessment

Element Assessed	EL Model with FB	NL Model with SSI
Diaphragm capacity	<34%NBS*	100%NBS
Wall pier in-plane flexure capacity	60%NBS	>100%NBS
Wall spandrel in-plane flexure capacity	80%NBS	>100%NBS
RC wall in-plane capacities	67%NBS	>100%NBS
Overall rating	60%NBS	100%NBS

*The diaphragm was further assessed using a grillage model, which indicated that while significant damage is expected, the diaphragm is unlikely to govern the seismic performance.

4.4 Sustainability and Operational Disruption Considerations

To illustrate the implications of relying on a conservative force-based assessment using an elastic fixed-base model, a hypothetical strengthening scheme was developed for the failure mechanisms identified in the force-based assessment, using near-surface-mounted (NSM) steel rods. The NSM rods would be interconnected with the existing reinforcement and tied into an existing earthing bar.

Table 6 summarises the estimated quantities of additional steel and embodied carbon implications considering Modules A1-A3 only (NZGBC 2024). These estimates indicate that reliance on a conservative force-based assessment could trigger substantial strengthening demands, with associated cost and environmental impacts. Noting that this hypothetical scheme is not presented as a preferred solution, but rather to illustrate the extent of strengthening that could arise when soil–structure interaction and structural nonlinearity are neglected.

Table 6: Hypothetical strengthening scheme and relevant cost

Items	Strengthening scheme	Approx. steel quantity (kg)*	Embodied carbon estimates (kgCO ₂ e)
Diaphragm	4xHD16, 16m long	100	397
Wall pier	3xHD25 each side, 10m long (54 rods in total)	2,100	8,337
Wall spandrel	2xHD25 each side each level, 22m long (8 rods in total)	650	2,580
RC wall	2xHD10@400CRS, 16m long (13 rods in total)	250	992
Total	N/A	3,100	12,307

* The steel quantities presented are subject to an uncertainty of approximately $\pm 20\%$

In addition to the cost and carbon emission, such a strengthening scheme requires extensive intrusive works, including concrete cutting, surface reinstatement, local demolition, and prolonged access to critical building areas. Significant disruption to building operation would therefore be expected, along with associated construction risks. This would be disruptive to the operation of this critical substation and potentially put the primary electrical assets at risk of physical damage. This highlights the value of advanced assessment methods in avoiding unnecessary intervention and associated social, economic, and environmental impacts.

5 CONCLUSIONS

This paper evaluated the influence of SSI and structural nonlinearity on the seismic response of a relatively rigid building with a basement embedded in soft soil. A detailed finite element model was developed and analysed using NLTHA with recorded earthquake motions. The modelling approach, including the way in which SSI was accounted for and the nonlinear representation of reinforced concrete elements, was described. The benefits of considering SSI and NLTHA in seismic assessment are also discussed. The key conclusions and recommendations drawn from this study are as follows:

- Considering structural nonlinearity enabled existing structural capacities to be more fully utilised, thereby avoiding unnecessary strengthening.
- For relatively rigid structures with a sublevel embedded in soft soil, soil–structure interaction can have a significant influence on seismic response and should be carefully considered to realistically evaluate seismic demand.
- In a time history analysis, structures with nonlinear soil would behave differently compared with those with a linear soil spring. Thus, to more realistically reflect the soil - structure behaviour, nonlinear soil properties should be considered.
- Adopting an advanced assessment-led approach to confirm the structural performance rather than defaulting to strengthening could avoid significant construction and reduce the associated embodied carbon and social disruption.

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