Practical issues in time-history analysis of low-rise concrete wall buildings with subterranean levels

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ABSTRACT

In New Zealand, older power stations are commonly low-rise concrete wall structures with rigid basements housing generators, turbines and other electro-mechanical infrastructure at or below ground level. These structures are typically characterised by low concrete strength and non-ductile detailing with low reinforcement ratios. In addition, these structures are acceleration sensitive in the short period range with few distinctive fundamental modes, but rather exhibit a large number of modes with low mass participation. Thus, the traditional equivalent static method based on the premise of single mode response is not suitable for analysing this type of structure. A linear time-history analysis appears to be a viable alternative. For such structures, the foundation flexibility could increase the spectral acceleration but also provide significant radiation damping. This paper presents the influence of foundation modelling on the accuracy of time-history analysis of low-rise concrete wall buildings with subterranean levels. A detailed finite element model with the foundation explicitly modelled and distributed soil springs acting around the foundation has been developed for a specific case study building. The effects of foundation mass and flexibility on superstructure response are discussed. Based on the findings from the detailed finite element model and to achieve computational efficiency, a simplified macro-element model representing the soil-foundation system is proposed through the representative stiffness matrix considering cross coupling components. Recommendations have been given for practicing engineers in analysing low-rise concrete wall buildings with subterranean levels.

1 INTRODUCTION

In New Zealand, it is common for industrial concrete wall buildings, e.g. a power station, to have one or more subterranean levels to provide additional spaces for services and equipment. To accurately predict the seismic response of the superstructure we need to consider the influence of substructure system and the surrounding soils (Tileylioglu et al. 2010). When a building is founded on a shallow foundation, it is reasonable to assume a fixed-base condition for the superstructure at ground level with free-field ground motion excitation, as the free-field motion is approximately identical to the foundation input motion. However, the presence of subterranean levels could alter the superstructure response due to the following Soil-Structure Interaction (SSI) effects (Wolf and Obernhuber 1985).

- Foundation flexibility. Flexibility of foundations will elongate the system's fundamental period and affect the base shear and structure deformation.
- Kinematic interaction. Kinematic interaction results from the presence of relatively stiff foundation elements on or in the soil. It alters the propagating shear wave velocity and reflects the seismic wave, which causes the ground motion at the foundation level to deviate from free-field motion.
- Foundation damping. It results from the relative movement of the foundation and soil and is associated with energy dissipation through radiation away from the foundation and hysteric damping within the soil.

Generally, the consideration of SSI effects caused by kinematic interaction and foundation damping serves to reduce the shaking input to the structure relative to the free-field input motion. Rivera et al. (2008) found that high-frequency seismic waves were filtered by the kinematic interaction between rigid and massive foundations and the supporting soil. This interaction resulted in smaller motions than those in the corresponding free field. Segaline et al. (2022) investigated the effect of dynamic SSI in a building with subterranean levels using physical reduced-scale models of a building under different configurations of above-ground and underground stories. The experimental results suggest that the interaction of the foundation in terms of motion compatibility and the vibrations transferred from the surrounding soil to the superstructure invariably produces a reduction in effective base motion.

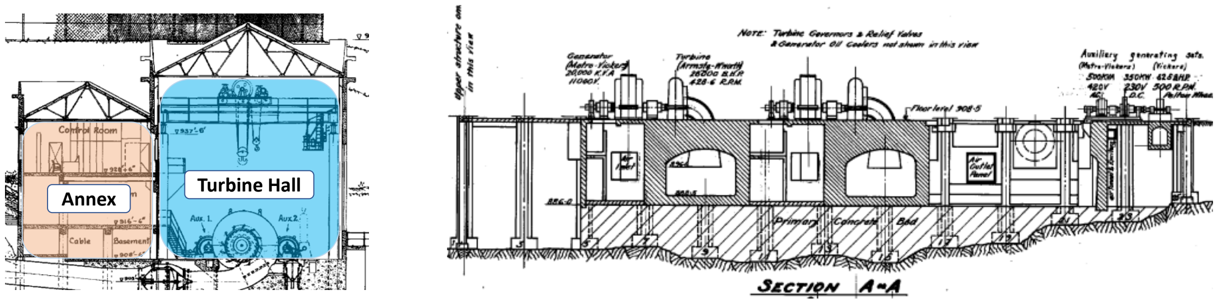
However, it is notable that foundation flexibility can reduce or increase spectral accelerations and seismic forces, but it can also increase lateral displacements and secondary forces caused by P-Δ effects (Moghaddasi et al. 2012). Low-rise concrete wall buildings are relatively rigid structures which are acceleration sensitive in the short period range with few or no distinctive fundamental modes. The short fundamental periods could be on the ascending branch of the site-specific response spectrum, in which case the increase in period caused by foundation flexibility can increase spectral acceleration and therefore seismic forces. This has been confirmed by Bandyopadhyay et al. (1995) and NZS/API 650 (2020) for tank structures where the impulsive base shear is directly calculated from the constant acceleration zone while the tank period lies in the rigid zone of the design spectrum. Thus, the effects of SSI, specifically the inclusion of foundation flexibility, on superstructure response should be evaluated in structural analysis.

In this paper, the influence of foundation flexibility and foundation mass on the superstructure response is investigated using a detailed finite element model with the foundation explicitly modelled and distributed soil springs around the foundation for a specific case study building. For practical engineering, it is desirable to have a simpler modelling approach which is less time-consuming but without compromising the accuracy of structural analysis considering SSI effects. Thus, a simplified single macro-element describing the whole soil-foundation system and linked to the base of the superstructure is developed for dynamic analysis of structures taking into account the influence of the subterranean levels. Recommendations have been given for modelling and dynamic analysis of low-rise concrete wall buildings with subterranean levels.

2 FINITE ELEMENT MODEL DEVELOPMENT OF STRUCTURE-FOUNDATION SYSTEM

2.1 Building description

A powerhouse structure with construction details typical of older hydroelectric power stations in New Zealand has been selected as the case study building for this paper. The powerhouse is a reinforced concrete structure with a single storey Turbine Hall and an integrated three-storey Annexe Block (see Figure 1 below) constructed in the 1920s. There is a rigid basement structure housing generators and other electro-mechanical infrastructure below ground level under the Turbine Hall.



(a) Main section above ground (b) Foundation section under ground level of Turbine Hall

Figure 1: Illustration of the building configuration

The seismic load resisting system of the building is a combination of cantilever columns with reinforced concrete infill walls of the Turbine Hall and perimeter reinforced concrete walls of the Annexe (see Figure 2 below). In the transverse direction (E-W), seismic loads at roof level are resisted by bending in the primary columns predominantly cantilever from ground level. In the longitudinal direction (N-S), seismic loads are resisted by the longitudinal concrete walls of the Annexe and primary columns with concrete wall infills of the Turbine Hall.

The foundation system consists of a rigid basement structure under the central 8 of 12 bays of the Turbine Hall and square concrete piles under the Annexe. The rigid basement structure includes thick boxed concrete walls, with cast in equipment. The lower levels of the basement excavation for the powerhouse have been backfilled with a weak mix of mass concrete which covers a significant portion of the building footprint. All the piles and basement structure are resting on the rock level. Details of the foundation are shown in Figure 1(b).

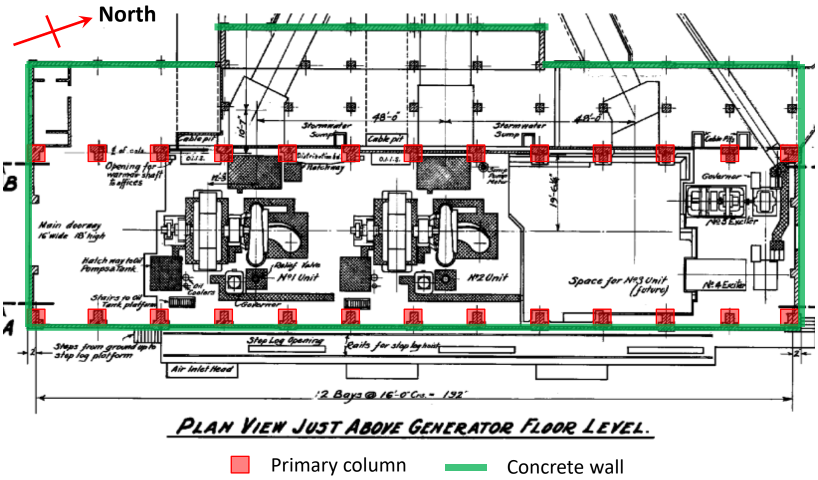


Figure 2: Primary lateral-load resisting elements of the building

2.2 Modelling of the building and foundation system

To understand the seismic response of the powerhouse structure, a finite element model was developed using DIANA FEA 10.6 (DIANA 2022). This building is characterised by low concrete strength and low reinforcement ratios which is typical for older industrial concrete structures in New Zealand. Ductility of the structure will be limited by the low reinforcement content and non-ductile detailing. Thus, the whole structure is considered force-controlled and structural nonlinearities are not considered for any structural components in the modelling. Effective stiffness modifiers have been incorporated into the concrete elements following ASCE41-17.

The foundation system including basement structure and piles have been explicitly modelled. The surrounding soil and its interaction with the structure is not directly modelled but instead represented by distributed soil springs and damping. To investigate the impact of modelling foundations (i.e. the subterranean levels and SSI effects) on the superstructure response, a superstructure-only model with fixed base at ground level has also been developed.

3 TIME-HISTORY ANALYSIS AND RESULT DISCUSSIONS

Figure 3 shows the mass participation ratio of the structural modes and cumulated mass participation ratio up to 90% from eigen analysis of the case study building. The building does not have any dominant fundamental modes, but rather a large number of local modes with low mass participation. Thus, the seismic response is complex when considering the large number of low mass participation modes which will not be excited simultaneously. The traditional equivalent static method based on the premise of single mode response is not suitable for analysing this type of structure and a time-history analysis is more appropriate.

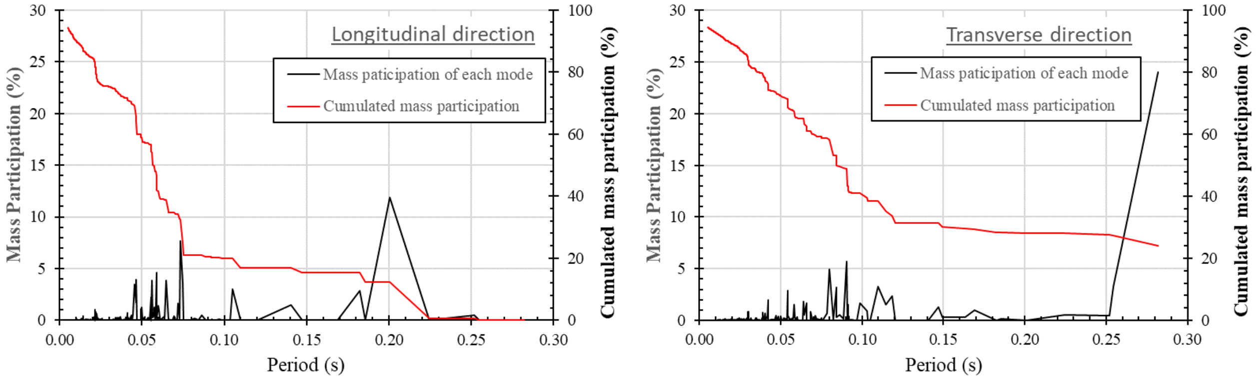


Figure 3: Mass participation ratio of the building modes

3.1 Selection and Scaling of ground motions

Information about the selected ground motion records is shown in Table 1 below. Three ground motions were selected based on a site-specific seismic hazard study of the site considering the seismic sources with major contributions. The scaling of the ground motions was based on the method outlined in NZS 1170.5 to match the site-specific response spectra for the site. A structural period range of interest from 0.03s to 0.6s is considered in the scaling covering more than 70% of the cumulated mass participation and considering period elongation of the structure system. Figure 4 shows the target spectrum and scaled response spectra of the selected ground motions.

Table 1: Earthquake records and scaling factors

Record Name*	Magnitude	Hypocentral Depth (km)	Scaling Factor $k1 \times k2$	Event Type
IRIGM 487	7.1	14	1.88	Crustal
LLOLLEO	8.0	40	0.89	Subduction interface
MYG008	7.0	65	2.78	Subduction slab

* The ground motion records were downloaded from NGA-West 2, COSMOS and K-NET for crustal, subduction interface and subduction slab, respectively.

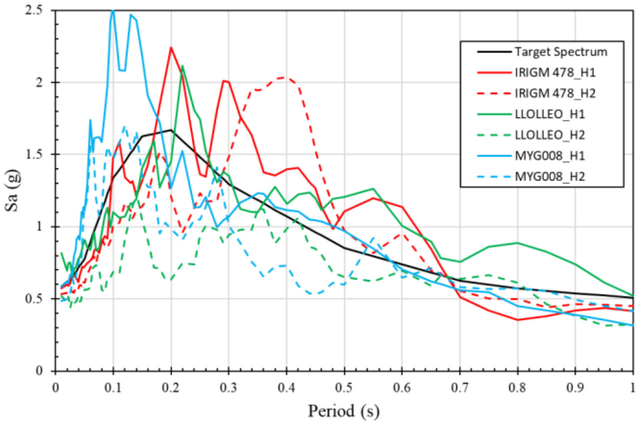


Figure 4: Scaled acceleration response spectra of the selected ground motions

3.2 The effect of foundation flexibility on superstructure response

Table 2 shows the analysis results of the developed models with and without foundation effects included. The difference in total base shear is in the order of 10-20%. It is interesting to note that the maximum base shear is increased in the longitudinal direction while it is decreased in the transverse direction after modelling the foundation system without radiation damping consideration. The difference in the critical fundamental period range of different directions can explain this. The critical periods in the transverse direction are longer than that of the longitudinal direction which could shift the dominant period range from the ascending branch of the site-specific response spectrum to the descending branch. In terms of deflection of the superstructure, the inclusion of the foundation system in the analysis increases the maximum displacement in both directions. These findings confirm the significant influence of foundation flexibility on the structure response and hence it should be considered in the seismic analysis of short-period structures.

Table 2: Time-history analysis results of model with and without foundation system

Model*	Max. Base Shear (kN)		Average Base Shear (kN)		Max. Displacement (mm)**	
	Longitudinal direction	Transverse direction	Longitudinal direction	Transverse direction	Longitudinal direction	Transverse direction
Full model	31,900	25,900	24,100	20,800	13	53 (150)
Superstructure-only model	25,700	28,600	21,700	23,100	8	26 (124)

* "Full model" is the model with foundation system and "Superstructure-only model" is the model with fixed base at ground level.

** The displacement is reported at the top of the primary lateral load resisting elements for the Turbine Hall. The numbers outside the bracket correspond to the wall elements, while those within the brackets pertain to the primary columns.

3.3 The effect of foundation mass on superstructure response

An assumption of massless foundation has been proposed by Clough and Penzien (1975) and is widely used in foundation modelling of seismic analysis. The foundation mass is treated as a separate type of mass on which the earthquake forces do not act on subterranean levels. However, the foundation mass may provide an inertial effect on the superstructure response, but the relative formulation method cannot be used (Gazetas et al. 1995).

Table 3 summarises the time-history analysis results for the developed model with and without foundation mass. The consideration of massless foundation changes the first mode period slightly. The change of total base shear at ground level is limited to within 10% difference. This example shows that the consideration of subterranean mass in relative formulation has minimal effect.

Table 3: Time-history analysis results of massed and massless foundation model

Foundation model	Period (s)	Max. Base Shear (kN)		Average Base Shear (kN)	
		Longitudinal direction	Transverse direction	Longitudinal direction	Transverse direction
With mass	0.21-0.4s	31,900	25,900	24,100	20,800
Massless	0.2-0.36s	29,200	27,500	23,500	22,300

4 DEVELOPMENT OF A SIMPLIFIED MODELLING APPROACH INCLUDING SSI EFFECTS

A simplified model of the case study building is developed in the widely used software CSI SAP2000 as shown in Figure 5. The superstructure is modelled elastically, and the foundation system is represented via a single macro-element attached to the base of the model.

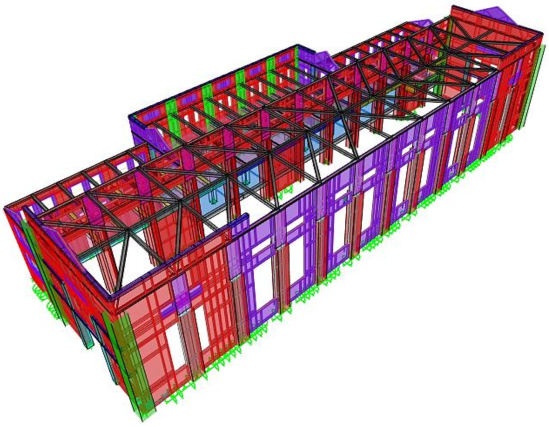


Figure 5: 3D view of SAP2000 superstructure model

4.1 Ground model consideration

The building was founded on rock at 6m to 10m depth with the top layer consisting of backfilled material which are mostly silty gravels, with some cobbles and boulders. The shear wave velocity was measured in two ways: MASW in three locations and downhole measurement in one of the two machine drilled boreholes. From MASW as well as the downhole measurement, the top layer has a shear wave velocity of approximately 200 m/s and that of the rock layer from the downhole measurement is 772 m/s. The physical observations of the machine drilled boreholes and the direct downhole measurements clearly demonstrate the presence of a prominent velocity interface. Both the downhole and MASW data do not extend to 30 m depth. The extrapolated Vs30 values are 909m/s and 991m/s respectively from Boore (2004) and Boore et al, (2011). A representative shear wave velocity of 500m/s is adopted for the rock considering it as a homogenous half-space. It should be noted that the accuracy of the shear-wave velocity is not critical for calculating foundation damping as the damping is restricted to 20% for the purpose of assessment (ASCE7-22).

The seismic site classification to NZS 1170.5:2004 can be considered as “Soil Class C” based on the characteristics of the surface deposit. However, as the foundation input motion will occur at the rock layer and the basement is significantly stiffer than the backfilled soil, the Soil Class is considered as “B” based on the shear wave velocity of the rock layer. Accordingly, the site-specific spectrum and ground motions were based on the Vs30 of the rock layer. This assumption is then further verified through a soil-structure-interaction analysis.

4.2 Soil-Structure-Interaction Modelling

To simplify the modelling approach, only the superstructure is explicitly modelled in SAP2000. The static stiffnesses (K) and radiation damping coefficients (C) for the subterranean levels and surrounding soils are introduced by a single macro-element. As discussed in section 3.3, the foundation mass can be neglected in the seismic analysis of low-rise wall building with subterranean levels. Thus, the whole soil-foundation system including foundation flexibility is described using a massless lumped parameter model (a combination of springs and dashpots) at the centre of rigidity at the ground level.

The foundation stiffness and the radiation damping are determined from Gazetas (1991) for surface foundations on homogenous half space based on an average shear wave velocity of 500 m/s, average unit weight of 2300 kg/m^3 and a Poisson's ratio of 0.2.

Due to the large stiffness contrast between the backfill and the rock, the embedment effect is neglected (the shear modulus of the backfill is at least 1/5th to 1/10th of the rock layer as it varies to the square of the shear velocity). The building has a basement that covers approximately 50% of the footprint. The footings of the individual columns are relatively small (typically 1200 mm square) and are neglected for the sake of simplification according to ASCE7-22.

The parameter H/V_sT (H is the effective building height, V_s is the shear wave velocity and T is the period range of interest) being negligible for constant-acceleration zone structures, and the rotational degree-of-freedom at the foundation can be neglected (NEHRP 2012). Therefore, only stiffness and damping for horizontal translational degrees-of-freedom are calculated for the transverse and longitudinal directions, respectively, according to Gazetas (1991) using below Equations 1 to 2.

$$K_y = [2GL/(2-\nu)](2+2.5\chi^{0.85}), \qquad K_x = K_y - \left[\frac{0.2}{0.75-\nu}\right] GL \left[1-\left(\frac{B}{L}\right)\right] \tag{1}$$

$$C_x = (\rho V_s A_b), \qquad C_y = (\rho V_s A_b).\widetilde{c_y} \tag{2}$$

where, χ is $A_b/4L^2$. A_b, B, and L are the area, half-width and half-length of the circumscribed rectangle for the basement foundation such that $L > B$. $\widetilde{c_y}$ considers the frequency dependence of the damping coefficients as depicted by Gazetas (1991) and equals $f\left({}^{L}/_{B}, {}^{\omega \mathrm{B}}/_{V_s}\right)$.

In addition, to account for the foundation flexibility, the lateral stiffness matrix of the basement is obtained from the detailed model in section 3 which consists of basement structure and lateral stiffness of the backfilled soil. The stiffness matrix is obtained by applying unit deformation along the degree-of-freedom investigated at a reference location, while keeping the remaining degrees-of-freedom restrained. The corresponding reactions are the leading and cross coupling terms respectively, this also included any eccentricity effect from the reference location. The reference location is taken as the geometric centre of the basement. Three degrees-of-freedom are considered, two translational and one in-plane torsional. The basement stiffness matrix $[K_b]$ is coupled, but the foundation stiffness matrix $[K_f]$ is uncoupled with only leading terms because the reference location is at the geometric centre of the basement. It is to be noted that $[K_b]$ contains stiffness contribution from the back-filled soil. The matrices are assembled in series to obtain the combined stiffness of the foundation and the basement as shown in Equation 3. The resulting matrix has coupling terms to be input in SAP2000.

$$[K_{eq}] = \left([K_f]^{-1} + [K_b]^{-1}\right)^{-1} \tag{3}$$

4.3 Time-history results of the simplified model

Linear time-history analyses are carried out for all three ground motions. The damping of the structure is represented by Rayleigh damping. Compared to the fixed base structure, the base shear for all the ground motions increased by a maximum of 20% in the longitudinal and negligibly in the transverse directions, when only flexibility is considered. Comparing the base shear in the simplified model to the detailed model results shown in Table 3 “Massless” foundation case, the difference is within 10% in the longitudinal and almost matching in the transverse directions.

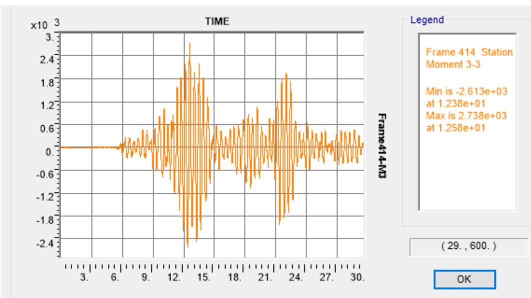
However, when foundation flexibility is accompanied by the radiation damping, the increase in base shear due to the foundation flexibility is offset, resulting in a base shear which is about 10% and 5% less in the longitudinal and transverse directions respectively than that of the fixed base.

The equivalent damping is estimated to be about 9% based on this inversion formula of response reduction $R = (7/(2+\xi))^{0.5}$ where ξ is the damping in percentage.

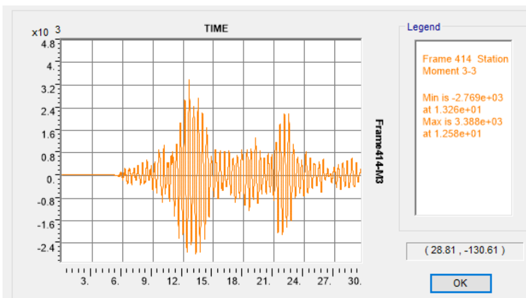
4.4 Attenuating higher mode responses

Amongst the various implicit integration methods for time-history analysis, it was found that Hilber-Hughes-Taylor (HHT) method is more appropriate for short period structures (Hilber et al. 1976). The user would require deciding on the amount of numerical dissipation, the three family factors α, β, γ where α practically varies between -0.3 to 0. A comparative study is carried out to choose the appropriate α value by comparing the results of the HHT with that of the mode superposition method.

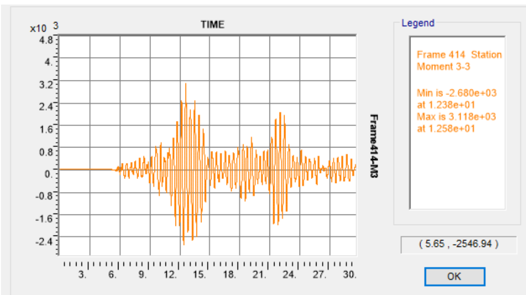
The mode superposition is performed with 5% constant damping across all the Ritz modes considering it as the upper bound benchmark as all the modes are retained. In a two-point Rayleigh damping formulation when contribution from the higher modes is to be retained, the lower modes have less damping. This becomes intractable to cover contribution of all significant modes at the same time. Therefore, a numerical dissipation is required. Figure 6 shows that the maximum bending moment in a critical column is 2738 kN.m from the mode superposition method, and 3388 kN.m (23% higher), 3118 kN.m (13% higher), and 2691 kN.m (less than 2% lower), for α equals to 0, -0.1 and -0.3, respectively. Based on similar reviews for other elements, α of -0.3 will give the best response prediction for the structural components.



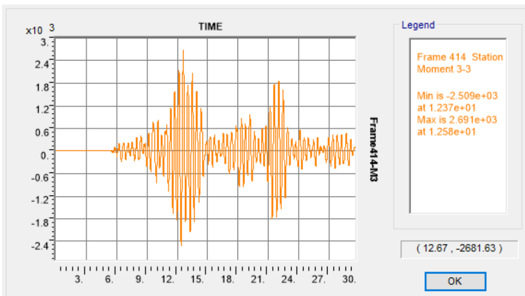
(a) Mode superpostion method



(b) α = 0



(c) α = -0.1



(d) α = -0.3

Figure 6: Comparison of flexural demand of a critical column with different α value

4.5 Rationalising responses

In elastic time-history analysis, the response of short-period structures could be highly erratic in component level because the components do not yield. The erratic response develops as the structure vibrates in a multitude of closely spaced modes, none of which is dominant over the other. Engineering judgement is needed to determine whether the envelope is physical or is a result of the numerical process. Our observation is that often one isolated peak is reported, while the next peak is considerably less and is sustained by several occurrences, as shown in Figure 7 for one of the wall components in the model. The value and the time of occurrence of such single peaks, almost like a spike, are different in the detailed model and simplified model. Therefore, some regularisation of the erratic response is needed. In ASTM E-2126 (2019), three repeated cycles for each drift amplitude are required in cyclic testing of structural component to investigate the structural ductility. The same rule was adopted here and we considered the value of peak which is exceeded at least 3 times. When this is adopted, the results from the detailed model and simplified model in component level are seen to have acceptable match for the purpose of design and assessment.

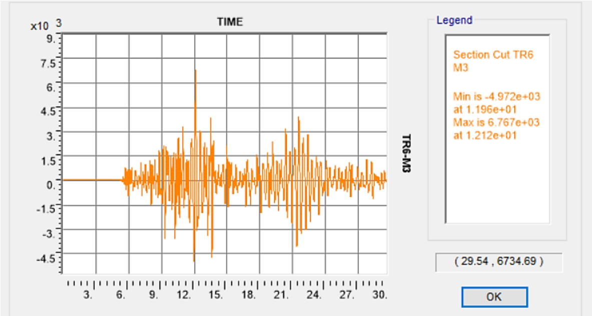


Figure 7: Erratic in-plane flexural demand of a shear wall; note a high isolated positive peak in comparison to the adjacent peaks.

5 CONCLUSIONS AND RECOMMENDATIONS

This paper evaluated the influence of foundation modelling including foundation flexibility and foundation mass on the seismic response of low-rise concrete wall building with subterranean levels using a detailed finite element model. A simplified modelling approach for the foundation system has been proposed using single macro-element representing the stiffness and damping matrix of the foundation system. The following conclusions and recommendations are revealed from the study.

- The equivalent static method based on the premise of single mode response is not suitable for analysing rigid structure with subterranean levels and a time-history analysis is recommended to assess the seismic performance.
- Foundation flexibility is crucial in seismic analysis of rigid structure with subterranean levels. It could increase or decrease the total base shear at ground level but increase the displacement demand.
- The assumption of massless foundation is valid in the seismic analysis using relative formulation approach. If the inertia effect of the foundation mass is to be considered, the formulation of the equilibrium equation should be re-written in absolute formulation approach.
- For rigid embedded foundation such as buildings with basements, the seismic site classification, site-specific spectrum, and ground motions should be based on the characteristics of the founding strata if there is a sharp shear wave velocity contrast.
- The amplification of the acceleration due to the flexibility of the foundation and the basement structure is reduced by the radiation damping. Therefore, for short period structures both flexibility and radiation damping should be considered.

- In elastic time-history analysis, the response of short-period structures is highly erratic and can show one isolated peak like a spike. The envelope of the predicted responses should not be relied upon and engineering judgement is to be exercised. It is recommended to choose the peak repeated three times over the shaking duration as the effective peak response.
- For short-period structures, introducing a numerical dissipation can reduce the impacts of very short period modes which are in the rigid zone as the two-point Rayleigh damping formulation is not sufficient. The HHT method with α = -0.3 will give a similar result as the mode superposition method where all modes can be retained.

6 ACKNOWLEDGEMENTS

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