Canadian Conference - Pacific Conference on Earthquake Engineering 2023

Vancouver, British Columbia

June 25th – June 30th, 2023

Case Study of Seismic Performance of a High-Rise Concrete Shear Wall Building Located Directly Adjacent to An Active Fault

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ABSTRACT

This study focuses on the seismic design and response of a new (37) story residential tower in San Diego that sits directly adjacent to an active fault. The tower design adopted Performance Based Seismic Design (PBSD) methodologies and was scrutinized by a seismic peer review panel (SPRP) comprised of structural and seismological engineering practitioners and researchers.

Given the adjacent active fault, near-fault effects were extensively evaluated in the development of the ground motions and PBSD approach. Ground motions (GMs) were developed with near-fault effects and the non-linear time history analyses (NLTHA) included simultaneous application of vertical and horizontal time series, in addition to, the permanent vertical ground displacements (VGD) associated with the fault rupture (i.e. fling-step). With the maximum VGD occurring at the fault-side of the building and reducing to zero 200' away from the fault line, the ground displacement contour was idealized as a "tilting" or permanent rotation of the tower towards the fault. Fling-step studies were performed to assess whether the VGDs associated with the fling should be applied in a static vs dynamic fashion. Results of these studies, the PBSD evaluation criteria and GMs for this near-fault site are discussed herein.

Incorporating vertical ground motions in NLTHA is rarely done and it presents many challenges, including but not limited to: numerical convergence, additional degrees of freedom, significant processing times and validation of analytical results is difficult. Ground displacement effects on building structures are sometimes explored in analytical models, but rarely in the context of a NLTHA. Approaches and techniques we developed to support these highly complex analyses are discussed herein, along with our analytical results.

A comparison of structural responses with two-component (horizontal GMs) and three-component (horizontal + vertical GMs) is presented. Compared to the two-component GMs, the three-component GMs are shown to have negligible effects on building drift demands and associated structural responses (i.e. rotational demands on coupling beams and outriggers, core wall flexure and shear demands, et cetera). Vertical GMs were shown to have little effect on the core wall axial-strain demands but significant effects were observed on the column axial demands. Compared to the vertical seismic effects estimated from the empirical equations per LATBSDC, NEHRP, TBI, ASCE 7, et cetera, we found that the vertical GMs were generating vertical seismic axial demands in the columns that were 500% to 1,000% higher than those values. Techniques developed to evaluate the columns for these unprecedented demands are discussed herein, along with the results of that evaluation.

Keywords: High-rise, concrete core walls, near-fault ground motions, vertical ground motions

INTRODUCTION

Located in downtown San Diego, California, this new 37-story development is comprised of two below grade levels, nine podium levels (L1-L9), with the typical tower floor plates starting at L10. Typical floor-to-floor heights are around 10 feet and the total height of the building is 375 feet above grade and 402 feet to the foundation level. Floor areas for the low-rise and high-rise floor plates measure roughly 50,000sf and 20,000sf, respectively. Figure 1 provides the project site plan and shows the location of residential tower within the larger low-rise footprint.

Vertical structural systems consist of mild-reinforced (below L10) and post-tensioned (above L10) two-way concrete slabs spanning between reinforced concrete (RC) columns located on a ~30'x30' grid. Lateral structural systems consist of RC diaphragms spanning between SRCSWs situated throughout the low-rise floor plates (below 10). Within the tower footprint (Figure 1), SRCSWs occur as a central elevator core as well as a blade wall that was introduced to manage torsion. Below grade levels are enclosed by reinforced concrete basement walls that wrap the perimeter of the low-rise footprint.

As identified in Figure 1, there is an active and recently discovered segment of the Rose Canyon Fault (i.e. San Diego Fault) that cuts through downtown and along the west side of the property. The location of the fault was confirmed via fault trenching followed by physical surveys of the exposed fault. A 20' minimum setback was recommended and the fault survey files were referenced into the building plans to ensure the structural envelope remained inside the fault setback. Above the ground surface, a small portion of levels L1-L9 cantilever out over the setback zone and moat condition introduced in this area (Figure 2). Movement gaps around the cantilevering L1 structure were sized to the maximum estimated fault displacements.

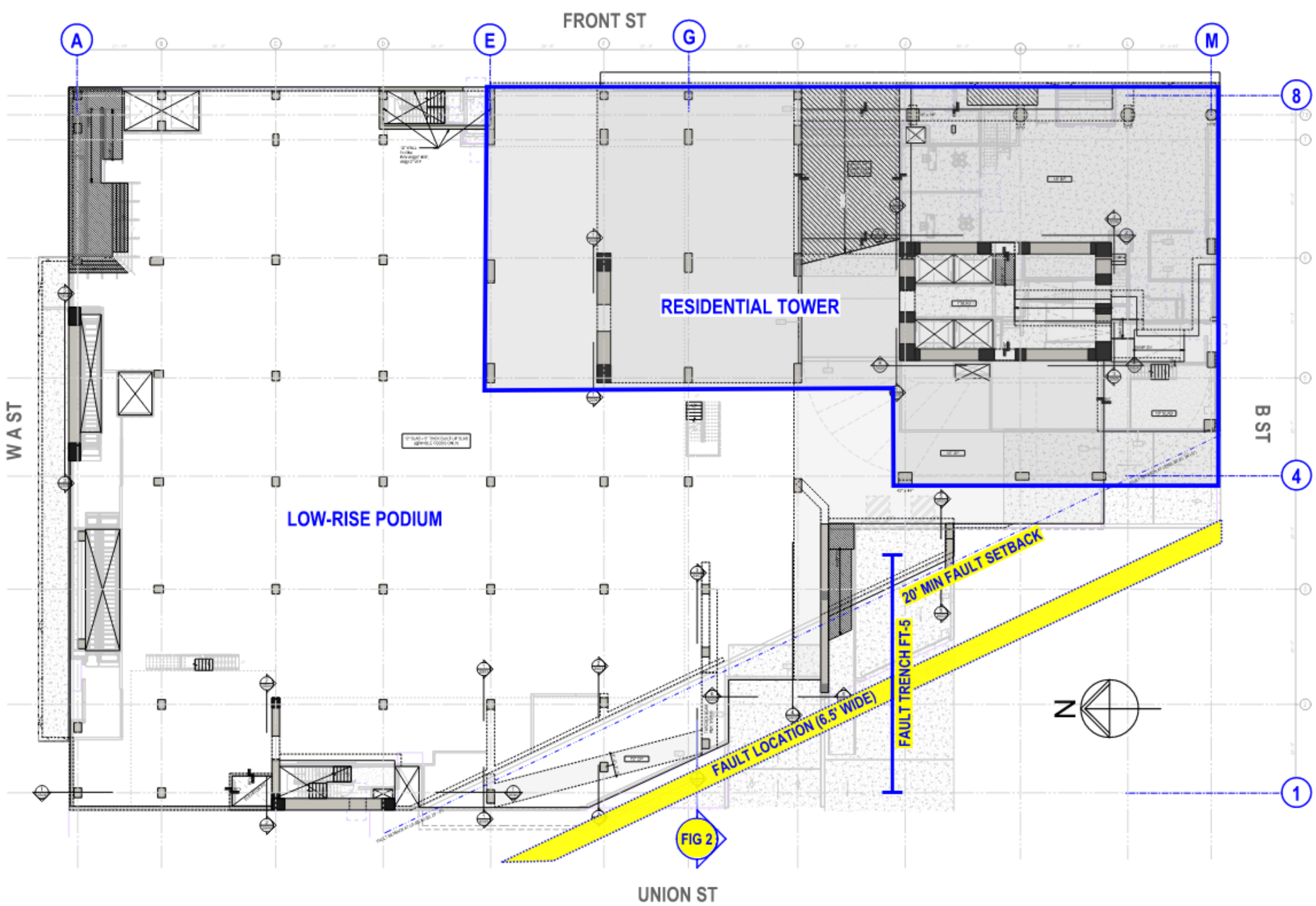

Figure 1. Project site plan.

The structural design was carried out in conformance with the 2019 California Building Code (CBC), ASCE 7-16, LATBSDC 2020, ACI 318-19 and associated reference standards. The building height exceeds the permissible height for Special Reinforced Concrete Shear Wall (SRCSW) systems outlined in ASCE 7-16 and a dual system was not desirable. As such, PBSD methodologies were adopted as an alternative to the prescriptive system and height limitations. In accordance with the City of San Diego requirements, the initial design of the seismic force resisting system (SFRS) was developed as a prescriptive code-based design under the Design Earthquake (DE). The performance of that design is further evaluated for Service Level Earthquake (SLE) and Maximum Considered Earthquake (MCE) scenarios as per the requirements of the LATBSDC 2020. Collectively, these approaches are intended to achieve the following building performance levels: 1) < 10% probability of collapse during 2,475yr MCER event, 2) low probability of life endangerment during 475yr DE event, 3) high probability for building to remain functional when subject to frequent earthquakes (i.e. 43yr SLE event).

Per the PBSD standards and requirements of ASCE7-16 and CBC 2019, our design and the criteria employed in that design was to be scrutinized by a Seismic Peer Review Panel (SPRP) comprised of the following individuals: geotechnical and seismology engineer, practicing structural engineer and a professor of structural engineering.

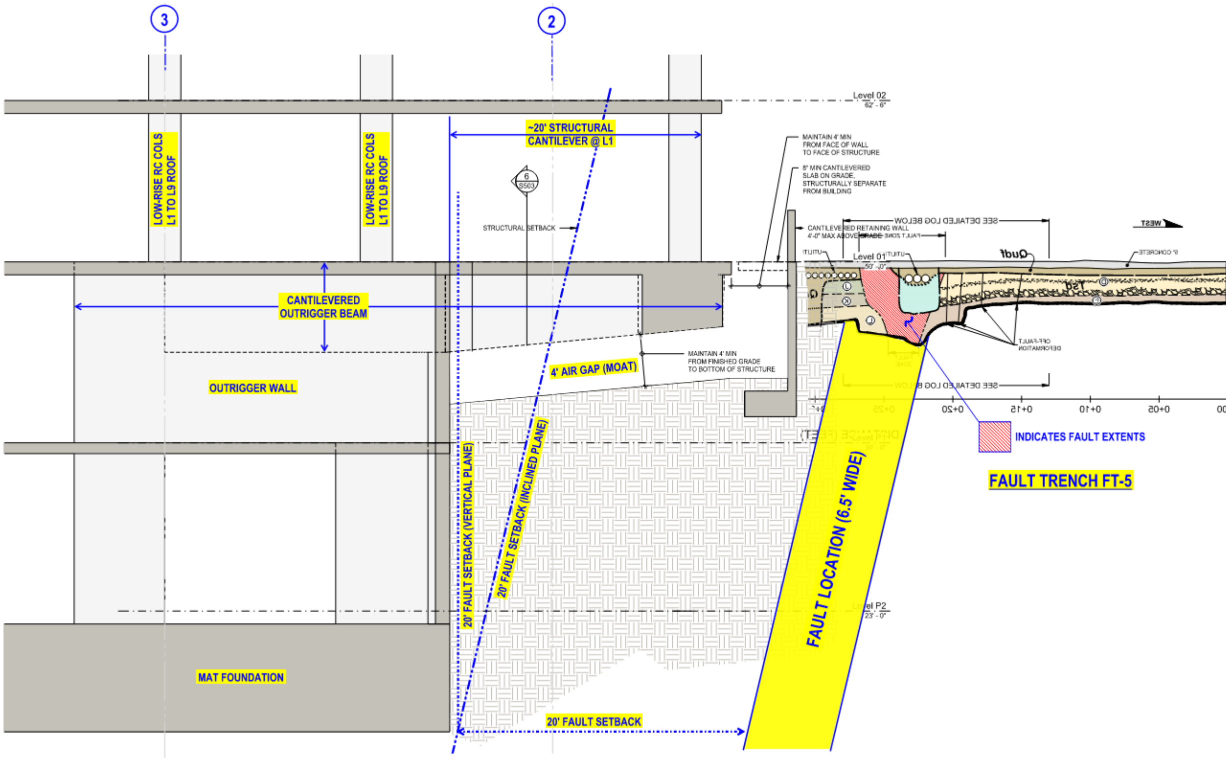

Figure 2. Section @ Structural Cantilever and Fault Setback

SEISMICITY & FAULT HAZARDS

The fault parameters and ground motions used in the design were developed by the project geotechnical and seismology consultants. Probabilistic Seismic Hazards Analyses (PSHAs) informed the site-specific response spectrums and seismic design values. A combination of probabilistic and deterministic methods were used to develop estimated fault displacements presented below.

Fault Displacements

Based on the fault rupture scenarios that were evaluated, average fault induced ground displacements for the MCE scenario were estimated as follows:

1. Horizontal Displacement: 1.22m or 48in
2. Vertical Displacement: 0.41m or 16in (33% of Horizontal)

The geotechnical engineer recommended that these fault-induced ground displacements be rationalized as occurring under the following scenarios:

1. Along a discrete knife edge of the fault,
2. Across the observed 6 ½' wide fault zone,
3. Over a larger deformation area extending outside the observed fault and recommended 20' wide setback zone, with approximately half the anticipated deformations occurring within the setback zone and half of the deformations occurring within a zone extending 200' from the edge of the setback.

Scenarios 1 and 2 informed the fault width and setback requirements as well as the movement gap dimensions surrounding the ground level structural cantilever and moat condition. Beyond those considerations, it was not necessary to consider Scenarios 1 & 2 in our analyses as they are considered to occur directly in the vicinity of the 6 ½' wide fault zone and outside the building footprint (i.e. no influence on building).

Scenario 3 served to inform the ground displacement contours that would be developed and directly evaluated as part of the MCE_R analyses (Figure 3). As the fault-induced ground displacements were considered to be a very extreme scenario, associated with a very rare event (return interval exceeding MCE_R event), direct evaluation of these in the context of the SLE and DE analyses was not appropriate. However, the foundation system selection did considered the requirements outlined in Table 12.13-3 of ASCE 7-16 and a mat foundation was confirmed acceptable as the differential settlement or tilt (i.e. 10.6''/200', .0044L) was within the maximum acceptable limits for such foundation applications.

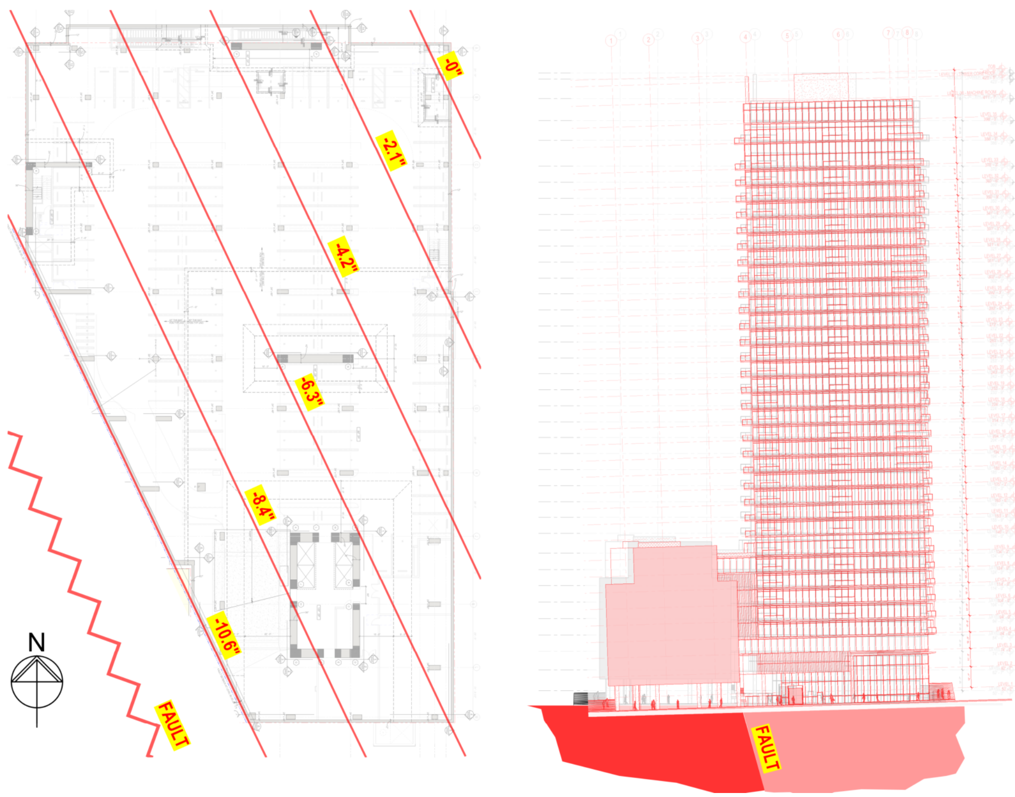

Figure 3. Fault-Induced Vertical Ground Displacements & Base Tilting Action

Much discussion was had on whether the vertical ground displacements should be applied in a dynamic or static fashion; because the displacements were predicted to occur over time interval that is considerably longer than the fundamental structural period, static application was deemed appropriate (see Fling-Step analysis).

Seismicity & Ground Motion Development

Our design was carried out in conformance with the 2019 California Building Code (CBC) and seismic design parameters were developed in accordance with ASCE 7-16 and the LATBSDC. Based on the site-specific studies, the site is characterized as Site Class C and seismic design parameters were developed for the three hazard levels evaluated: Service Level Earthquake (SLE, 43yr event), Design Earthquake (DE, 475yr event) and the Maximum Considered Earthquake (MCE_R, 2475yr event).

SLE and DBE evaluations were carried out via a linear response spectrum analysis based on site-specific response spectrums. For the MCE_R performance verifications, 11 pairs of grounds motions (GM) were developed and scaled to the target Site-Specific MCE_R Spectra presented in Figure 4.

GM scaling consisted of a hybrid approach that utilized period-dependent linear scaling to develop pair- and component-specific target response spectrum that were used in spectral modification. As the site is considered Near-Fault, records with polarized demands were rotated to fault-normal (FN) and fault-parallel (FP) orientations as per ASCE7-16 section 16.2.4. The FN/FP orientations were determined for the first-mode and lengthened periods by the inspection of the orbital response plots. Pairs with naturally occurring polarization were rotated such that the maximum direction was aligned with project FN direction.

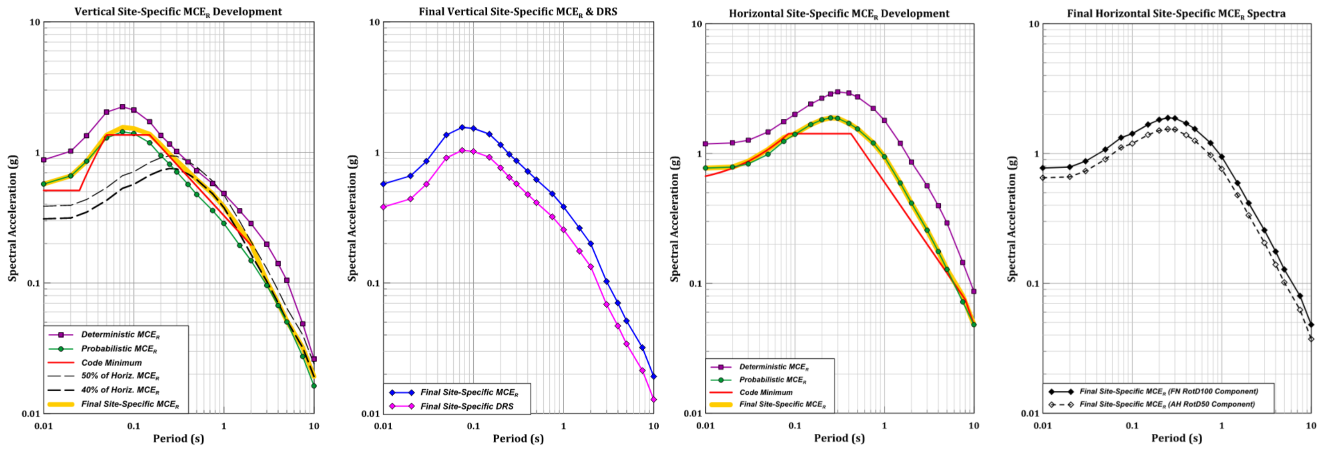

Figure 4. Final Site-Specific MCE_R Vertical (left) & Horizontal (right) Spectra

Despite the regular form of the structure (i.e. no significant vertical transfers or gravity-induced lateral demands, et cetera), due to the near-fault location, the SPRP mandated that vertical seismic effects be evaluated via application of vertical ground motion time histories and not by way of the empirical methods typically used to estimate these effects (e.g. equations 5c-5f of LATBSDC 2020). The MCE_R vertical response spectra developed in accordance with ASCE7-16 was found to produce large V/H ratios with average return periods exceeding 4,000yrs (i.e. not consistent with 2,475yr MCE_R event). For these reasons, the vertical response spectra acceptance criteria for this project was based on 40% of the site-specific MCE_R RotD50 response spectrum in the vertical period of interested (0.09-1.0s).

Fling-Step Analysis

Fling-step analyses were performed using the site-specific fault displacement estimates and the GMs were modified to incorporate fling-step effects. A comparison was made using the ratio of spectral responses for GMs with and without fling-step modifications as presented in Figure 5. For periods less than eleven seconds, the ratio of responses with and without fling-step modifications are less than 10%. The fundamental structural periods are around four seconds and well inside the period range for which the dynamically applied fling had little effect on the structural demand parameters. Based on analysis software limitations, had we proceeded with the fling-modified GMs, it would not have been possible to simulate the tilting of the building with a single input motion – this could have only been realized with multiple input motions with varying degrees of permanent ground displacements across the building footprint. It was therefore determined that the fling-step effects should be evaluated in a static fashion by tilting the building to the estimated fault displacement contours (Figure 3) in advance of running the GMs.

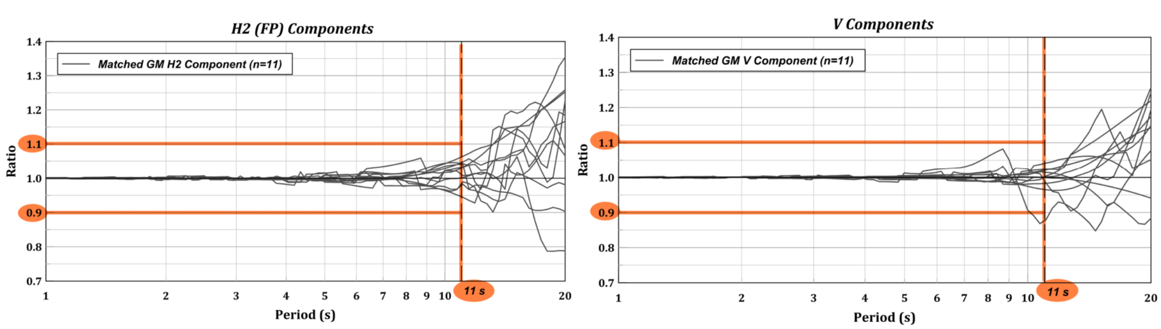

Figure 5. Ratio of Spectral Responses for GMs with & w/o Fling-Step Modifications

ANALYTICAL MODEL DEVELOPMENT

Based on our extensive experience with PBSD and NLTHAs, we were concerned with the SPRPs analysis requirements for this project and the high-degree of complexity these would introduce to the analytical models.

Incorporating vertical ground motions in NLTHA is rarely done and it presents many challenges, including but not limited to: numerical convergence, additional degrees of freedom, many modes required to achieve desired modal mass participation, extremely long processing times, and validation of analytical results can be difficult rationalize (e.g. vertical seismic force demands). Ground displacement effects on building structures are sometimes explored in analytical models, but rarely in the context of a NLTHA. As we have encountered significant challenges with vertical NLTHA analyses in the past, we thought it was prudent to run tandem analyses of only horizontal GMs and horizontal + vertical GMs. Aspects of the various models developed for these analyses are discussed in the following.

General Modelling Aspects (SLE, DE, MCE)

For the SLE and DE analyses, a 3-dimensional model was developed in ETABS, which included core walls, podium walls, basement walls, slabs, and columns. The stiffness modifiers for DE analysis were obtained from ACI 318-19, except that for coupling beams, effective stiffness for flexure was calculated from LATBSDC 2020. The effective stiffness values for the SLE analyses were obtained from Table 3 of LATBSDC 2020.

For the MCE_R analyses, a 3-dimensional non-linear model was developed in PEFORM 3-D and effective stiffness values were obtained from Table 3 of LATBSDC 2020. For backstay effects, upper bound and lower bound stiffness assumptions were evaluated at the main diaphragm transfer levels (L9, L1, B1) in accordance with LATBSDC 2020.

Fiber sections were used to model the axial-flexure response of core and podium walls and the basement walls were modeled with elastic fiber sections. Coupling beams were modeled with a shear hinge and shear response of core walls, podium walls, and basement walls were assumed to be elastic.

Viscous damping was employed as a combination of modal damping (2.3% assigned to all modes) and 0.2% Rayleigh damping over the period range of 0.1T to 2.0T (where fundamental period(s), T = ~4s).

Modelling for Tilt Effects (MCE)

To address the fault-induced vertical ground displacement or "tilt" (Figure 3), the mat foundation was explicitly modeled in PERFORM-3D with elastic shell elements (Figure 6) with each node supported on linear point springs. Since soil-structure interaction was not intended to be captured in the MCE_R analyses, the stiffness of the support springs was not calculated based on the modulus of subgrade – instead, "infinitely" stiff support springs were employed and the stiffness was calibrated such that the dynamic properties of the tilted model aligned with those of the fixed base (or "un-tilted") model. With the spring stiffnesses constant, variable artificial loads were calculated and applied to each spring/node such that the vertical ground displacement at any node would match the displacement contour across the larger foundation (Figure 3).

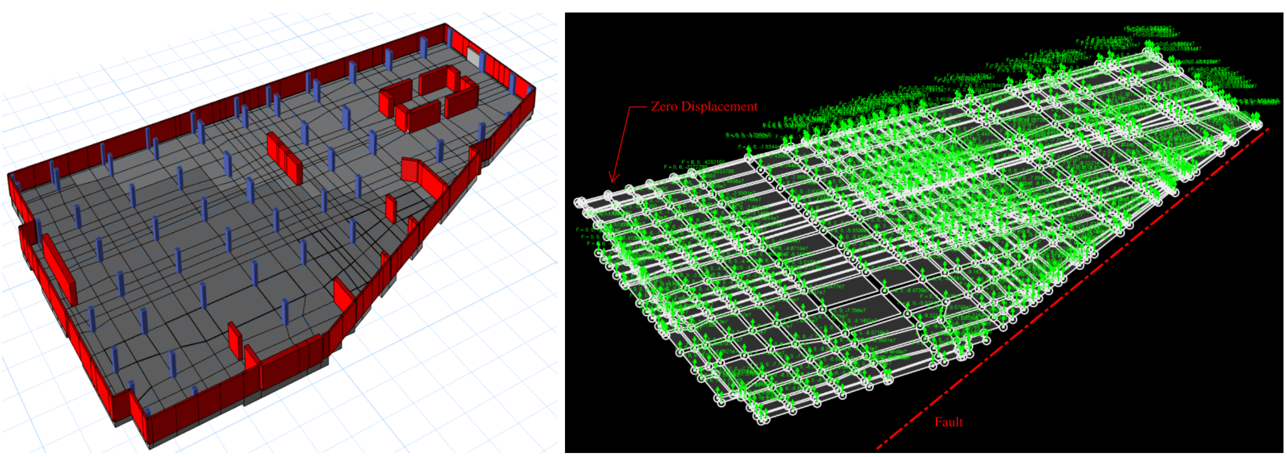

Figure 6. PERFORM-3D Foundation Meshing, Point Springs with Assigned Loads

Modelling the tilt proved to be relatively straightforward and the tilt effects on the structural response were quite intuitive. From our initial analyses looking at tilt effects alone, we observed that the structural elements remained in the linear range and the response to tilt was essentially rigid-body action with the ground/foundation tilt occurring as a uniform drift ratio up the height of the tower (i.e. structural drift = ground tilt = 0.44%).

Modelling for Vertical GMs (MCE)

With the tilt effects addressed, we then looked to equip the PERFORM-3D with the additional characteristics needed to run the vertical GM time histories. Using our original model, another model was created and the following updates were incorporated:

1. To adequately define the vertical stiffness and mass distribution characteristics:
    a. All diaphragms were revised to semi-rigid using elastic shell elements and effective stiffness properties;
    b. All lumped story masses were revised to more distributed masses assigned to each shell element node;
2. Damping model revised based on numerical and convergence issues we encountered with the initial runs.

While these updates are seemingly straightforward, they came with many challenges and required many initial iterations just to obtain a model that would converge and achieve sufficient vertical Model Mass Participation (MMP) with only 99 modes (PERFORM-3D limits). After many sensitivity studies looking at incremental increases in MMP with increasingly more refined floor-shell element mesh sizing (and vertical mass distributions), and associated effects on structural response parameters; we determined that the final meshing of the low-rise and typical tower floors presented in Figure 7 was adequate but only achieved vertical MMP of 27% with 99 modes. Following these updates, the first mode period associated with the vertical vibration of the building was 0.17 second.

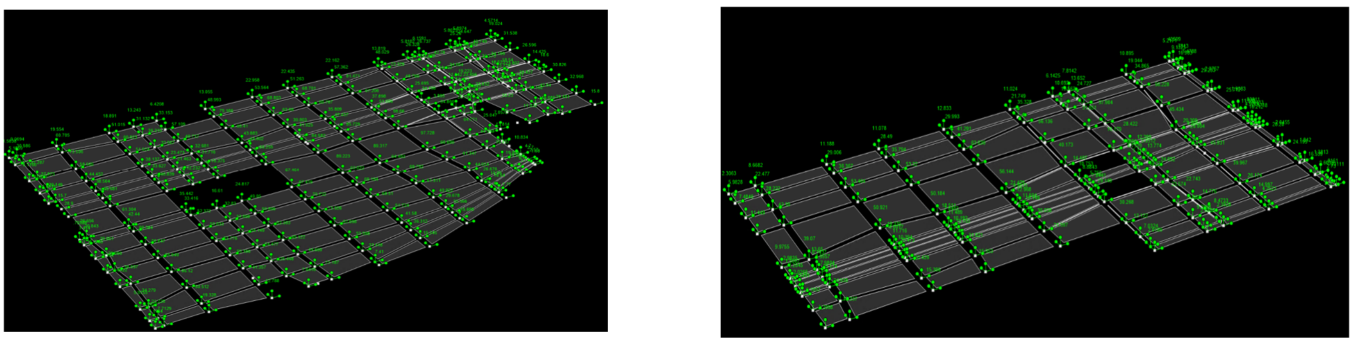

Figure 7. PERFORM-3D – Meshing of Low-Rise (Left) and Typical Tower Floor Plates (Right)

With the many degrees of freedom that were introduced to the model (i.e. meshing of all floors, distributed vertical masses, et cetera), we experienced very long runs times and struggled with convergence. After much discussion with the SPRP, we determined that the best course of action was to supress the very high-frequency modes by damping them out. To do this, we revised the damping model to employ only Rayleigh damping with coefficients selected to provide roughly 20-30% Rayleigh damping ratio at 20 Hz. The final damping model is plotted in Figure 8.

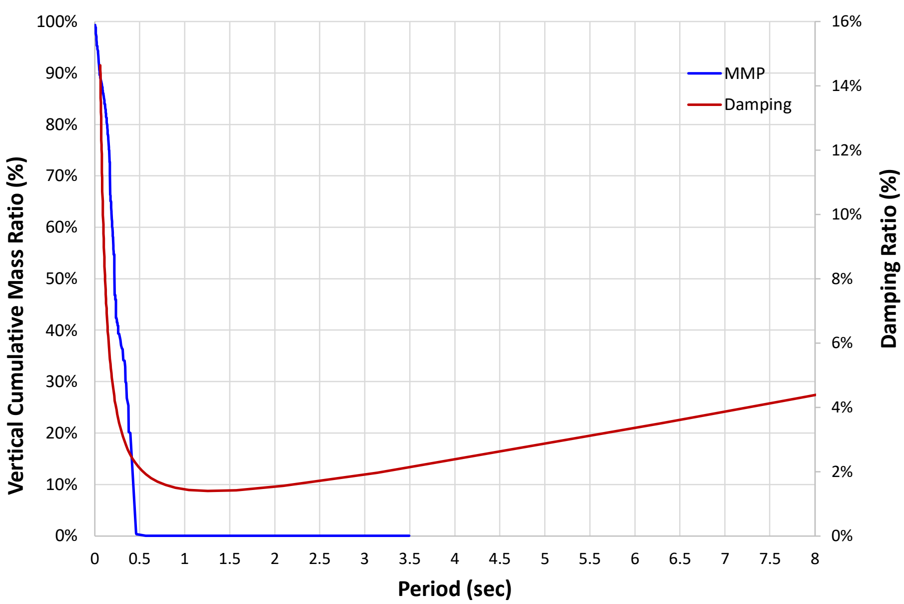

Figure 8. Plot of Vertical MMP and Damping Ratio vs Period   PERFORM-3D.

RESULTS & FINDINGS

Although we were now able to successfully run our PERFORM-3D with vertical GMs to completion, it still required 4-5 weeks to execute the suite of eleven GMs. As we still needed to run more design iterations, there was interest in determining exactly what demand parameters are influenced by the vertical GMs in hopes that we could focus on those separately and allow design iterations to proceed with the more simplified model (i.e. with horizontal GMs only). Initial results of key structural demand parameters from the NLTHAs with and without vertical GMs are plotted in Figure 8.

Through inspection of these plots and others, we determined that the vertical GMs have a negligible effect on building drift demands and associated structural responses (i.e. rotational demands on coupling beams and outriggers, core wall flexure and shear demands, et cetera). The vertical GMs were shown to have little effect on the core wall axial-strain demands but significant effects were observed on the column axial demands (see Table 1). Based on these findings, we proposed (and the SPRP agreed) that all structural components aside from the columns be evaluated with the more simplified model utilizing only the horizontal GMs.

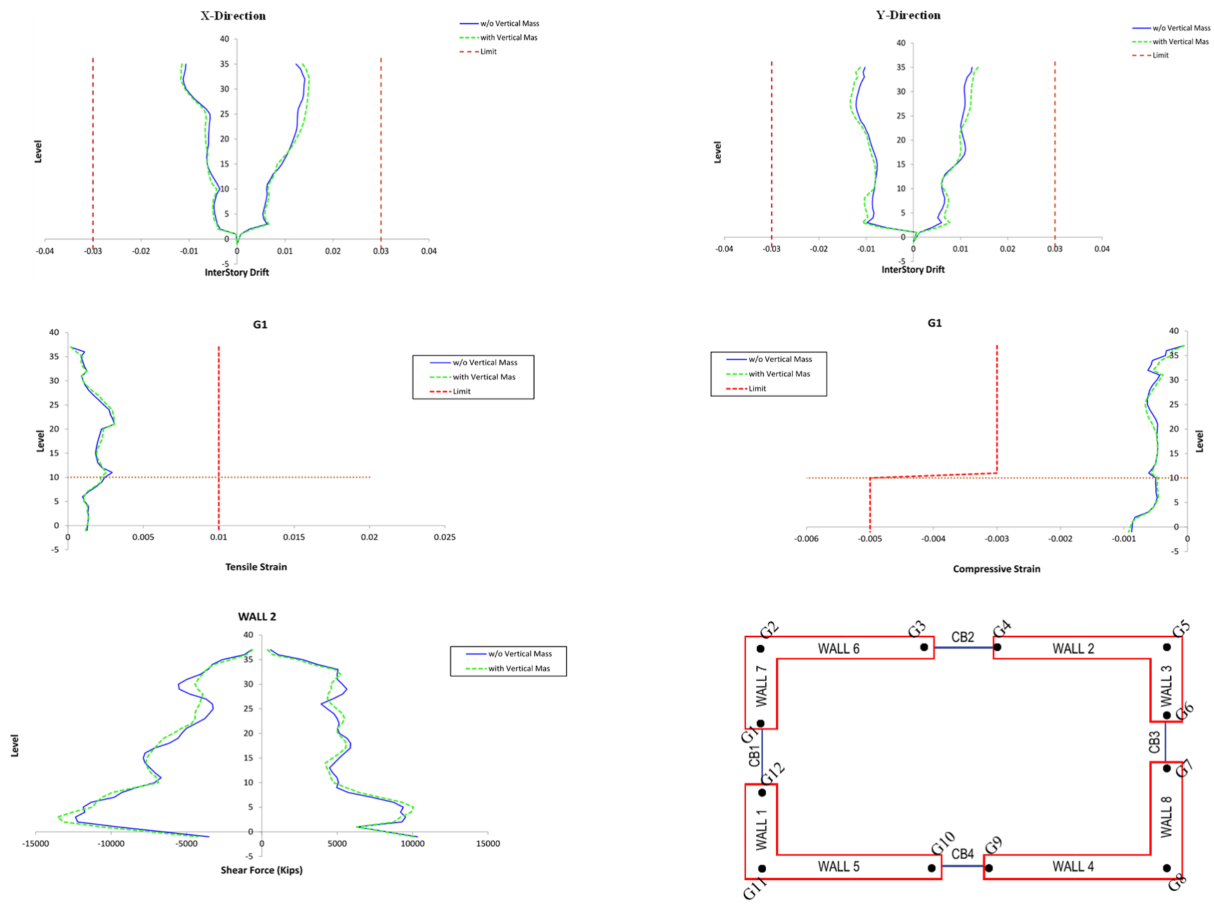

Figure 9. Comparison of average demands from the model with vertical ground motion with those determined from the model with horizontal ground motions. Blue and dashed green line indicate demands generated from PERFORM-3D runs without and with vertical GMs, respectively.

Investigation of Column Axial Demands

Upon further inspection of the results generated from the vertical GM analyses, we found that the column axial force demands were extremely large and this needed further investigation. We began by looking at the individual components that make up the cumulative column-axial demands: gravity (dead+live), seismic-outrigger action, and vertical-seismic accelerations. We quickly determined that the gravity demands were not the source of the discrepancy, nor were the seismic-outrigger demand components (outrigger rotations and drifts were not influenced by vertical GMs), and so we began a rigorous evaluation of the vertical seismic effects/demands.

As a sanity check on the demands, we calculated the estimated vertical seismic effects for MCE_R as per the empirical equations of LATBSDC 2020 and ASCE7-16/NEHRP 2009 – these were estimated as 0.203g and 0.215g, respectively.

With some separate analyses and post-processing, we backed out the gravity and outrigger axial demand components to determine exactly what vertical seismic axial demands were generated from the vertical GMs analyses, and then normalized them to the gravity demands (Table 1). Inspection of these results reveals that the vertical seismic forces generated in the columns were 1.17g on average, with the lowest floors experiencing roughly 0.68g, and increasing to 3.45g up the height of the tower. Vertical seismic effects of this magnitude are unprecedented and 500%-1,000% higher than what is predicted by the empirical equations of LATSDC and NEHRP. We briefly considered refining our models to include soil-structure interaction effects (e.g. radiation damping) but determined that this would not significantly offset the magnitude of vertical seismic demands observed in our analyses.

Table 1. MCE_R Vertical Seismic Effects (%g) from PERFORM Vertical GMs

LEVEL	COLUMN ID (See FIGURE 10 for Keyplan)										
	C3	C4A	C4B	C12	C16	C19	C18	C17	C15	C11	MEAN
35	310%	205%	248%	367%	374%	376%	305%	485%	309%	469%	345%
34	212%	189%	253%	335%	319%	292%	296%	314%	244%	173%	263%
33	156%	166%	197%	239%	269%	236%	303%	247%	202%	162%	218%
32	142%	132%	162%	212%	215%	195%	217%	208%	181%	149%	181%
31	144%	106%	145%	188%	191%	173%	190%	184%	155%	144%	162%
30	144%	94%	141%	171%	177%	158%	169%	161%	136%	136%	149%
29	140%	88%	135%	164%	160%	147%	153%	146%	123%	130%	139%
28	136%	85%	126%	157%	140%	137%	142%	136%	116%	125%	130%
27	131%	78%	115%	149%	123%	134%	133%	130%	107%	122%	122%
26	126%	76%	112%	146%	113%	132%	128%	123%	104%	121%	118%
25	120%	76%	106%	142%	109%	128%	123%	117%	101%	126%	115%
24	116%	74%	104%	137%	111%	122%	119%	114%	98%	126%	112%
23	113%	74%	105%	131%	111%	117%	112%	109%	97%	122%	109%
22	109%	76%	102%	126%	112%	114%	106%	108%	95%	121%	107%
21	105%	79%	97%	122%	112%	111%	104%	109%	91%	118%	105%
20	100%	80%	95%	119%	110%	107%	100%	107%	89%	116%	102%
19	96%	78%	94%	118%	109%	105%	98%	106%	87%	112%	100%
18	90%	79%	95%	116%	108%	103%	97%	103%	86%	105%	98%
17	84%	78%	96%	113%	110%	102%	96%	100%	91%	102%	97%
16	81%	78%	98%	113%	111%	102%	93%	98%	88%	98%	96%
15	79%	77%	102%	111%	112%	99%	89%	97%	89%	97%	95%
14	78%	78%	102%	109%	113%	99%	87%	93%	90%	94%	94%
13	76%	82%	103%	109%	112%	96%	85%	90%	89%	93%	94%
12	77%	85%	102%	106%	111%	95%	82%	87%	86%	90%	92%
11	77%	86%	102%	106%	108%	93%	80%	86%	89%	86%	91%
10	76%	88%	101%	103%	104%	87%	77%	83%	96%	83%	90%
9	66%	87%	86%	96%	103%	82%	77%	64%	77%	89%	82%
8	69%	84%	86%	92%	99%	77%	75%	68%	83%	91%	82%
7	71%	83%	85%	89%	97%	72%	74%	67%	91%	93%	82%
6	73%	84%	83%	85%	97%	69%	74%	67%	99%	94%	83%
5	74%	84%	84%	82%	94%	67%	75%	66%	104%	96%	82%
4	75%	84%	82%	79%	92%	65%	75%	64%	96%	95%	81%
3	74%	82%	71%	78%	93%	64%	72%	63%	87%	94%	78%
2	74%	82%	71%	75%	93%	64%	72%	65%	84%	92%	77%
1	74%	83%	66%	56%	89%	36%	42%	66%	84%	91%	69%
P1	73%	81%	65%	55%	86%	36%	41%	63%	84%	92%	68%
MEAN	107%	92%	112%	133%	133%	119%	118%	122%	112%	121%	117%

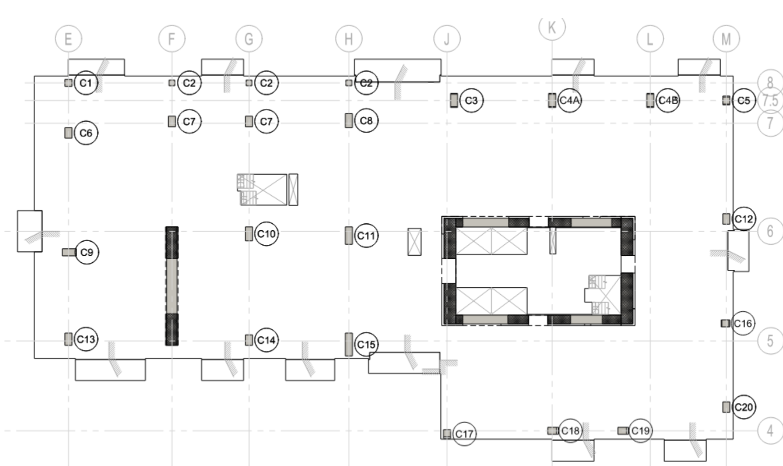

Figure 10. Tower Column Keyplan

Final Column Evaluations

Although we had previously demonstrated that the columns were performing adequately as Force-Controlled elements when subjected to the vertical seismic demands estimated from the empirical equations, our existing design could not be validated with the unprecedented level of vertical seismic demands generated by the vertical GMs. The design team was very skeptical with the results of the vertical GMs and the level of column demands generated by these analyses, but we could not produce anything that would invalidate these results. In the absence of that, we decided to focus on the strain demands of the columns and pursue a deformation-controlled evaluation of these elements.

In our deformation-controlled evaluation of the columns, the column tensile strain limits were taken as 0.01 as per Table 6-2 of LATBSDC 2020. Compressive strain limits were taken as the strain at peak stress as determined from the confined stress-strain relationship per Razvi and Saatcioglu (1999). The maximum and average column strain demands were evaluated for the suite of GMs and evaluated against the limits. Figure 11 provides a sample of the tensile and compressive strain plots that were prepared for columns C4A and C3 (see Figure 10 for keyplan).

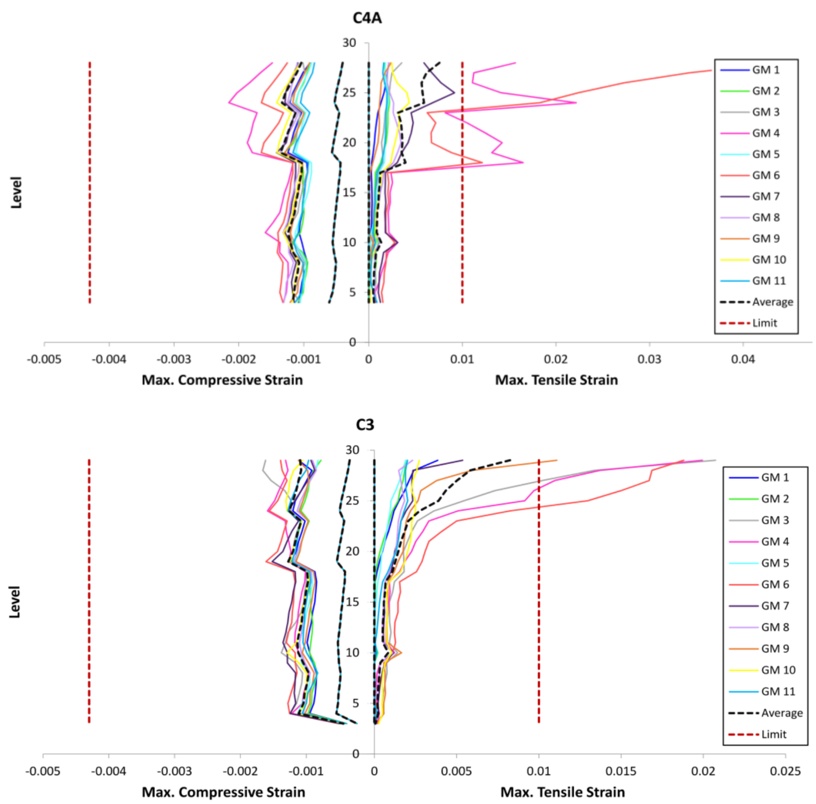

Figure 11. Tensile and compressive strains for two tower columns.

Maximum tensile strains were exceeded only for a few columns at the uppermost levels of the building where the dead and live gravity loads were relatively low, and the vertical-seismic accelerations were most pronounced, at these levels slight increases to the vertical reinforcing were observed.

Maximum compressive strain limits were also only exceeded for a few columns, but at the lowermost levels of the building and within the tower footprint. No increases in cross-sectional area were required for these columns and strain limits could be sufficiently increased with enhancements to confinement reinforcing.

CONCLUSIONS

Through the course of the PBSD and NLTHA analyses we carried out on this project; we encountered many unique challenges when attempting to simulate the near-fault effects in the analytical models we developed to evaluate the structural response. Based on our efforts with such analyses on this project, the following conclusions were gained:

- When developing site-specific vertical response spectra and complimentary vertical GM time series, code-based procedures (such as ASCE7) should be carefully evaluated to ensure that the return periods of the vertical accelerations are consistent with the desired hazard levels for which the horizontal GMs were developed.
- For near fault sites, the effects of fault-induced ground displacements on the structural response should be considered. In development of the PBSD approach, studies should be performed to determine whether these effects should be captured statically vs dynamically. Were fault-induced ground displacements will vary across the larger structural footprint, static application is recommended in the absence of more sophisticated analysis software that would allow multiple input motions across the building plan..
- For vertical-dynamic response analyses, and especially for NLTHAs, sensitivity studies should be performed to balance program convergence/performance against accuracy of analytical model and associated results. Mesh sizing of horizontal elements and associated masses should be calibrated and non-traditional damping models may need to be explored to damp out very high frequency responses that do not materially impact results.
- For more regular structures (i.e. no significant transfers or gravity-induced lateral demands), we have found that compared to the 2-component GMs (horizontal only), the use of 3-component GMs (horizontal + vertical) were shown to have negligible effects on building drift demands and associated structural responses (i.e. rotational demands on coupling beams and outriggers, core wall flexure and shear demands, et cetera). Vertical GMs were also shown to have little effect on the core wall axial-strain demands but significant effects were observed on the column axial demands.

  When tasked with incorporating 3-component GMs on future projects, significantly longer program execution times and convergence issues should be expected. Understanding that the presence of vertical GMs will have limited effects on the majority of the structural elements, we recommend parsing out the evaluation of those elements with a separate analytical model to speed up design and analysis iterations on the other structural elements.
- Compared to the vertical seismic effects estimated from the empirical equations per LATBSDC, NEHRP, TBI, ASCE 7, et cetera, for this project we found that the vertical GMs were generating vertical seismic axial demands in the columns that were 500% to 1,000% higher than those values. Soil-structure interaction effects (e.g. radiation damping) would certainly reduce the demands we observed, but by our estimates, not enough for them to align with demands predicted by the empirical equations. We are not the first to identify such discrepancies and we are aware of ongoing research on vertical seismic effects across the larger structural/seismic engineering community.

  Our studies indicated that the vertical seismic effects are increasingly more pronounced moving up the height of the tower, whereas the empirical equations only provide a single scalar value for all levels of the building. Additional research is needed to determine if a single scalar value is appropriate or if this should vary up the height of the building (or similar).

- If tasked with evaluating column performance under larger forces generated with vertical GMs, consider a strain based vs force based performance evaluation. To satisfy demands, force-based evaluations will likely require significant increases to column sizes and capacities. With the deformation/strain based evaluation pursued on this project, we were successfully able to maintain the initial column sizes and meet the performance objectives with slight enhancements to the column reinforcing details.

ACKNOWLEDGMENTS

The authors acknowledge the valuable feedback from the structural and geotechnical peer review team, which consisted of Leonard Joseph, S.E. (Principal, Thornton Tomasetti), Professors Jose Restrepo and Joel Conte of University of California, San Diego, and Professor Yousef Bozorgnia of University of California, Los Angeles.

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