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NONLINEAR SEISMIC ANALYSIS AND ASSESSMENT OF BC PLACE STADIUM

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ABSTRACT

This paper summarizes the results of dynamic response spectrum and nonlinear static analyses of BC Place Stadium. The scope of the analysis was to review the viability of rocking foundations to dissipate seismic energy in lieu of conventional, anchored foundation upgrades to an existing structure as conventional approach would impact existing, buried utilities. The results indicate the significance of soil-structure interaction and the effect of foundation rocking and soil yielding on the overall response of the building. This behaviour would also result in increased displacement response of the superstructure and permanent foundation displacements.

Introduction

BC Place stadium is located at 777 Pacific Boulevard in downtown Vancouver on the North side of False Creek. The construction of this 60,000 seat stadium commenced in April of 1981 and completed in June of 1983. The stadium has recently undergone interior renovations including several structural upgrades to accommodate a new roof structure. Presently elements of the existing structure that resist seismic forces from the new roof have been upgraded to meet the current seismic design provisions or upgrades have been designed and shall be constructed in conjunction with the new roof. Occupancy of the stadium has not changed from the time of the stadium's original construction, nor has the stadium undergone any significant additions.

When designed, the governing design statue was the National Building Code of Canada, 1980. Seismic analysis of the structure consisted of a static force applied to the frame as prescribed by the building code, and these forces were applied to both two-dimensional computer models as well as hand calculations. Detailing of the concrete reinforcement was completed to a high standard. Subsequent editions of the building codes have increased the magnitude of earthquake for which a designer must consider in the design. Risk involving earthquakes are commonly expressed as a probability of exceedance over a given time period. As shown in Table 1 below, the period of exceedance has increased, which has resulted in an increased in level of seismic forces that must be accommodated by a new structure's lateral load resisting system. Beyond the increase in return period, an importance factor has been incorporated for buildings likely to be used as post-disaster shelters in current BC building code; this additional factor also increases the design forces that must be accommodated in new buildings.

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Table 1.1: Seismic Base Shears of Various Building Codes

Design Statue	Seismic (Elastic) Base Shear	Return Period
NBCC 1980*	$V_E = 0.14W$**	1/100
NBCC 1995*	$V_E = 0.33W$	1/475
NBCC 2005***	$V_E = 0.66W$***	1/2475

*Importance Factor, I_E, of 1.0
**Original seismic design criteria
***Importance Factor (I_E) taken as 1.3 and assuming fixed base shear walls

The seismic assessment of BC Place involved numerous analytical models/methods with emphasis on the soil-structure interaction, as discussed below.

Structural System

BC Place Stadium is a concrete structure that is comprised principally of 54 concrete moment frames orientated radially that support four suspended levels of precast concrete joists and bleacher sections which form the floors of the elevated structure. The weight of the structure is approximately 690,000 kN (155,126 kips) without the addition of the new roof and 800,000 kN (179,856 kips) inclusive of the new roof structure. The eight periphery ramps total 220,000 kN (49,460 kips) of additional weight. The overall plan dimensions of the building are 224m by 183m, with the bowl frames arranged as a super ellipse. The top of concrete event level slab is 3m above sea level. The top of the existing structure is approximately 35m above the event level; with the new roof in place the building will project approximately 82m above the event level (See Fig. 1).

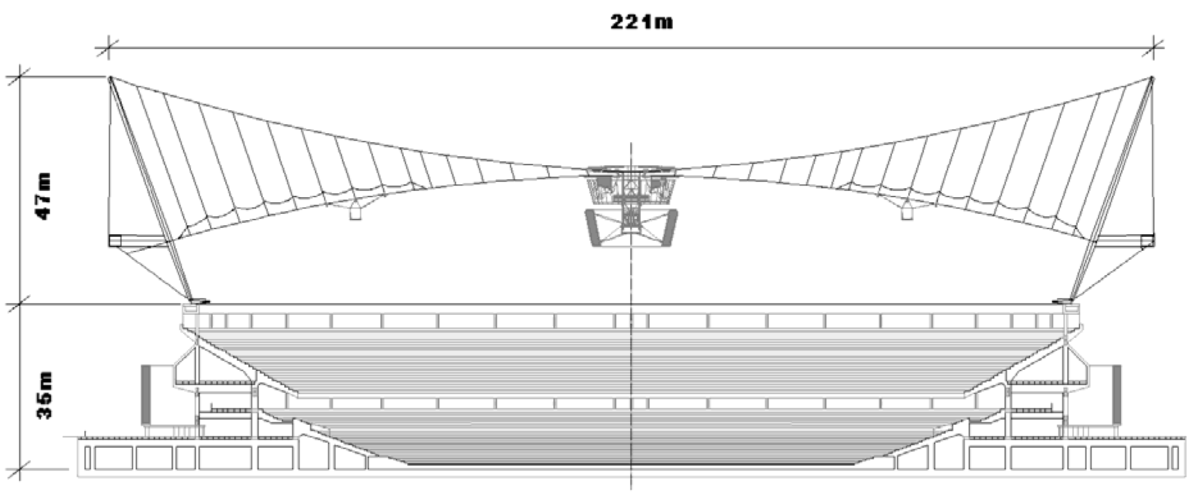

Figure 1. An overall transverse section of BC Place existing bowl and exterior plaza with the new roof (original drawing courtesy of Geiger Engineers).

The radial moment frames act to support the gravity loads imparted by the self-weight of the structure, occupancy, and snow. In addition, these frames provide lateral stiffness to resist wind and seismic effects. Detailing of the steel reinforcement in the concrete frames was found to be generally acceptable although column upgrades at Gridline F were still required to support the new roof structure.

The original design also includes 42 concrete shear walls orientated circumferentially throughout the stadium (See Fig. 2). These walls are located on all five circumferential grid lines and vary in height from the underside of roof level at the perimeter of the building to the underside of precast joists below the Level 3 Suites in the concourse area, and to the underside of the bleachers in the lower bowl. The orientation of these shear walls is intended to resist lateral forces that would otherwise burden the concrete moment frames in a direction perpendicular to their orientation, or the “weak axis” of the moment frames. For reasons of thermal expansion and concrete creep effects, the building was partitioned into eight segments, with expansion joints at Gridlines 2, 9, 16, 23, 29, 36, 43, and 50 (See Fig. 2).

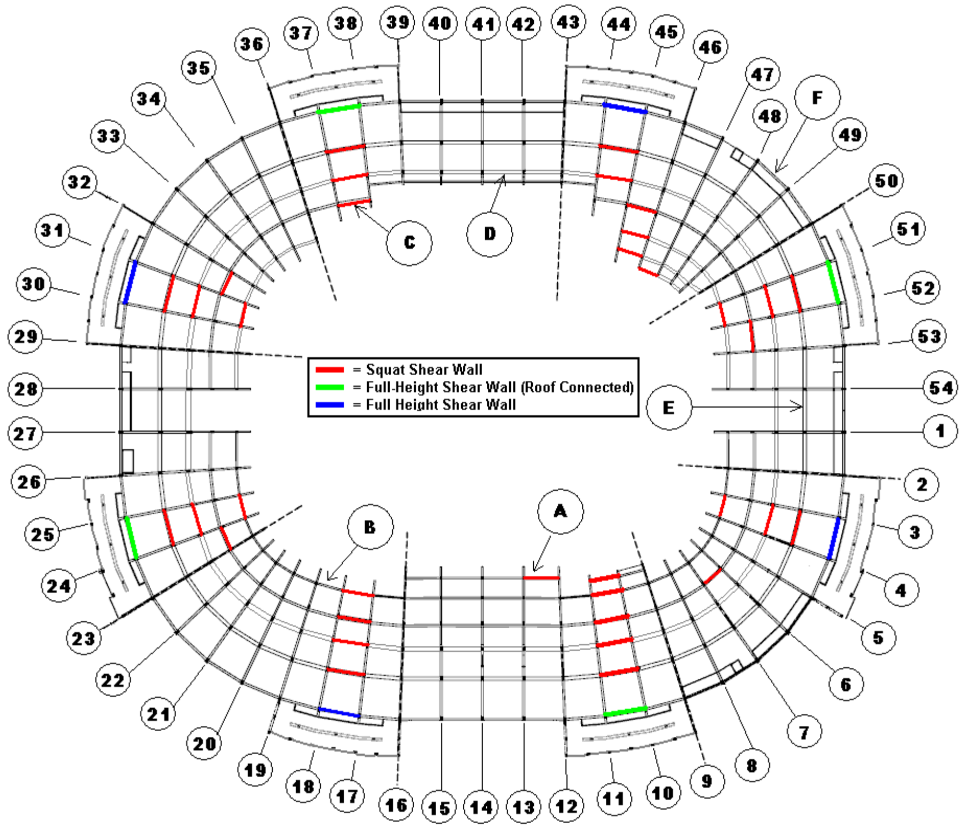

Figure 2. Plan view of BC Place Stadium, the green lines indicate full-height walls that are connected to the roof, the blue lines denote full-height walls not connected to the roof, and the shorter, squat walls are denoted in red. The eight ramp structures at the periphery of the building are self-supported and separate from the main bowl structure but do share common footings. Note the bowl structure is partitioned into eight segments and the expansion joints are noted by the dashed lines.

The concrete reinforcing of the existing lateral load resisting system is well detailed, suggesting that the frames will behave in a ductile manner. The frame-wall arrangement adds complexity in analyzing the structure, and the segmented bowl sections create the potential for pounding between bowl segments. In order to control this drift and mitigate pounding soil anchors at the base of the shear walls are required, however, anchoring the shear wall foundations affect the dynamic characteristics of the structure as the anchored walls exhibit rigid response increasing the dynamic shear on these elements.

Response Spectrum Analysis

Response Spectrum Analyses (RSA) were conducted on a full three-dimensional model of BC Place Stadium based on spectral acceleration values established in accordance with British Columbia Building Code (BCBC 2006) for a “Site Class C” soil classification with an importance factor, I_E, of 1.3.

The RSA linear elastic method provided the design team with anticipated seismic demand in the structure by accounting for the stiffness of the building and its mass distribution. This analysis also confirmed that the dynamic characteristics of the roof are such that the roof is not sensitive to stiffness/mass changes in the bowl structure below. A series of soil springs produced by the geotechnical consultant was introduced into the structural model to account for the structure boundary condition at the base.

Bounding the seismic demand on the structure was achieved using several modal combination methods. Modal combination methods that accounted for rigid mode response of the structure produced elastic base shears that were commensurate with the code prescribed base shears assuming the walls are fixed to the ground. The shear walls in most locations are squat or nearly squat in their proportions. The analysis generated shear demand on these walls that exceeded their sliding resistance and overturning moments resulting in significant uplift. In effect, the loads attracted in the (fixed-base) squat walls were quite large, and more advanced analysis was required as the squat walls were subject to foundation rocking/sliding.

Soil-Structure Interaction

Observing the results obtained from non-linear static push-over analyses (See below), we completed iterations of the structural model in an attempt to produce results similar to those of the non-linear push-over models. These modifications included implementing equivalent linear soil springs to mitigate the forces attracted by the squat walls and revising the shear stiffness of the walls to better approximate the peak shear and moment demand attracted by the existing shear walls including foundation-soil interaction (soil springs were estimated based on site specific measured shear moduli; an example of upper and lower bound soil modulus of subgrade is shown in Fig. 3).

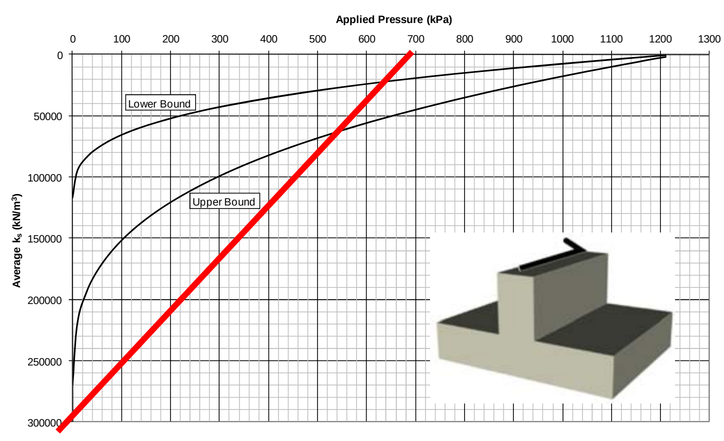

Figure 3. Average modulus of subgrade reaction for shear walls (red line indicates equivalent linear spring).

In essence, the engineering team worked to idealize the complexities of non-linear soil-structure interaction within the linear RSA model. The linear idealization of soil properties provided limited information and further advanced analysis is required; however, useful information of the fundamental dynamic properties of the existing structural system were attained from these analyses. The estimated range of fundamental period of the existing structure (with a new roof) was computed as 0.9-1.1 seconds assuming rocking foundations (for the fixed base structure the fundamental period was about 0.5 seconds). The higher period associated with foundation rocking substantially reduced the anticipated peak seismic demand on the building (See Table 2 and Fig. 4).

Table 2: (Elastic) Base Shears from Multiple Modal Combination Methods and Shear Wall Boundary Conditions

		Walls Fixed to Ground		Assumed Foundation Rocking	
	% Base Shear (BCBC 2006)	100%	60%	100%	60%
Dynamic Analysis: Modal Combination Method	GMC (Elastic – Not Scaled)	500,000kN	300,000kN	210,000kN	126,000kN
	CQC (Elastic – Not Scaled)	190,000kN	114,000kN	150,000kN	90,000kN
BCBC 2006 Static (Elastic) Base Shear		550,000kN	330,000kN	290,000kN	200,000kN

The significant decrease in seismic forces achieved through foundation rocking does, however, affect building displacements. When fixed base shear walls are assumed the anticipated displacements at the top of full-height shear walls is on the order of 40mm; when linear soil springs were introduced into the model this displacement increased to over 100mm. These displacements are only approximations given the non-linear properties of soil, and the non-linear analyses discussed below suggest these drifts could be higher notwithstanding pounding between bowl sections.

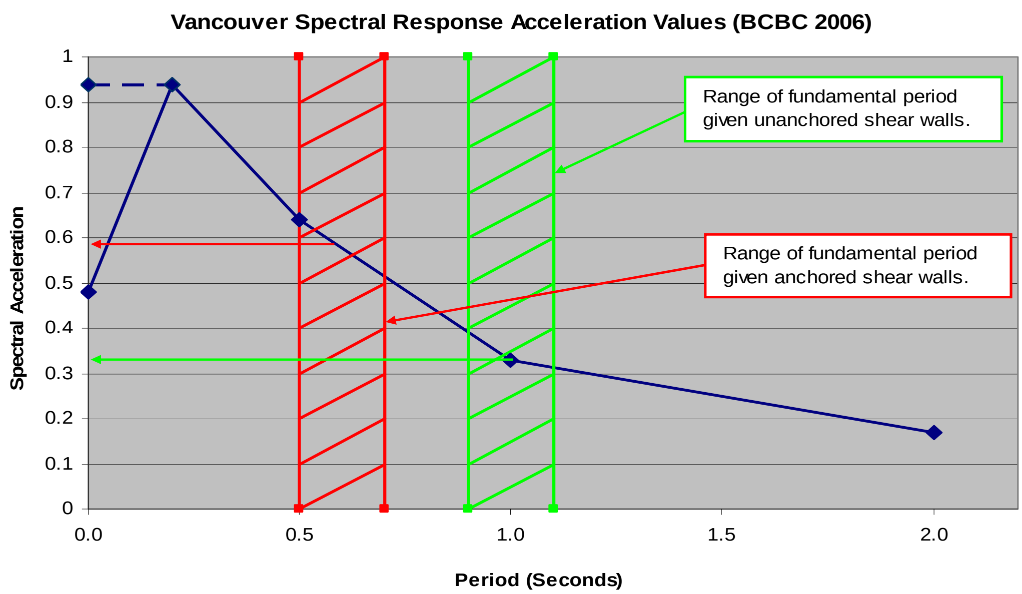

Figure 4. Vancouver spectral response acceleration values for a soil Site Class C.

Given the range of periods established when comparing rocking foundations to anchored footings, the effect of linking the bowl segments together is expected to produce a fundamental period between the foundation-anchored/foundation-rocking period envelope (See Fig. 4).

Further analysis is required to verify these assumptions and confirm that this approach will adequately dissipate seismic forces in lieu of a conventional foundation anchorage.

Nonlinear Static (Pushover) Analysis

Pushover analysis provides a simple method of directly evaluating nonlinear response of a structure at different levels of lateral displacements, ranging from initial elastic response through development of a failure mechanism. It is an attractive pre-requisite to the more complex procedure of nonlinear dynamic response history analysis. Pushover analysis was conducted according to the criteria outlined by FEMA 356 Section 3.3.3. This document defines the force-deformation criteria for hinges used in a pushover analysis.

Pushover Analysis Modelling

The commercially available computer program SAP2000 was used to carry out the pushover analyses. The pushover procedures prescribed in the ATC-40 and FEMA-356 documents are fully integrated into the SAP2000 program. The pushover analyses were carried out for a typical radial and circumferential frame of the BC Place building, as shown in Fig. 5 below. The radial frame represents the frame on bayline 14 including the extension to columns on bayline F for the added gravity load from the new roof structure. The circumferential frame represents the shear wall and columns located on bayline F from baylines 9 to 16. Both models include the footings for columns and shear walls. As discussed above, the vertical soil springs representing soil properties under each footing were provided by Thurber Engineering.

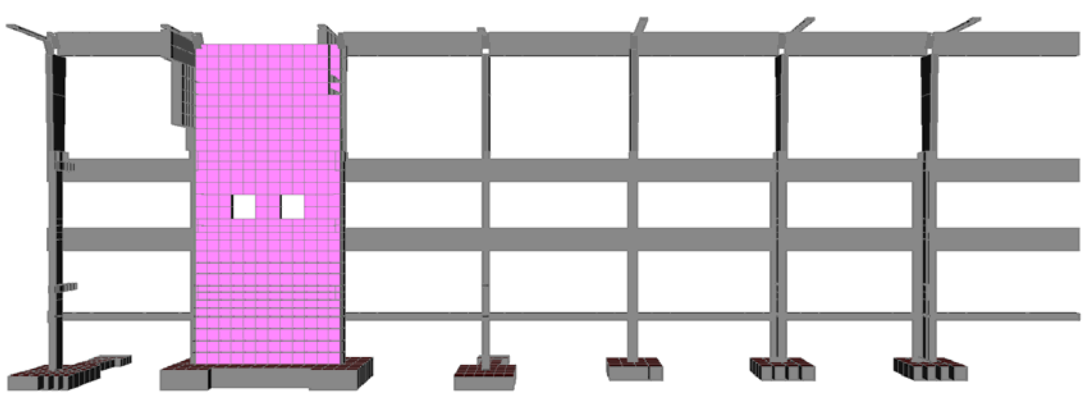

Figure 5: Elevation views of the BC Place radial (top) and circumferential (bottom) frames.

For both frames the nonlinear axial-moment interaction of beams and columns were computed excluding the material resistance factors for steel and concrete. The strength of the rebar material was considered as 400 MPa. The compressive strength of concrete was varied between 25 and 40 MPa for various elements.

Input parameters for the load-deformation response of the structural elements were adopted from FEMA 356. A strain-hardening slope of 2% of the elastic slope was considered for the post-yield load-deformation response of the frame elements. The interface of the frames and ground was modeled as a series of discrete spring elements depending on the elements of the soil underneath each support location. It is noted that the beam-column joints and the shear behaviour of the frames and walls were assumed to behave in a linear manner. This was later checked for representative elements to ensure the assumption of linearity is valid.

The pushover analysis was carried out for two different loading patterns:

1. The first loading pattern was similar to the profile of the story shear inertia forces consistent with the story shear distribution calculated according to the first mode response. This loading pattern is a good approximation for most of the low- to mid-height structures where the effect of the first mode is dominant.
2. The second pattern selected was a uniform distribution consisting of lateral forces at each level proportional to the total mass at each level (uniform acceleration method).

All pushover analysis cases started after the application of the dead load case including 50% of the live load.

Pushover Analysis Results (Radial Frame)

Fig. 6 shows the displaced shape of the radial frame at the end of pushover analysis together with the load-displacement plots for the loading patterns described above. The color spectrum at the bottom of the figure indicates the extent of nonlinear action. The purple color (far left) indicates start of nonlinear behaviour, the dark blue color indicates limited yielding in the region for immediate occupancy, the light blue is the life safety zone and the green color is the collapse prevention zone while the yellow color is near collapse region.

As the frame was pushed towards the centre of the bowl, the first yielding occurred in the raker beam between levels 4 and 5. Subsequently, the beams in levels 2 to 4 developed plastic hinging near the ends (lateral displacement of 100 to 150 mm at the fifth level). At a lateral displacement of about 200 mm the first sign of column yielding was observed at the base of 4th level column on bayline E and top of column on bayline C. The demand in beams at the 2nd level exceeded the collapse prevention criterion at a lateral displacement of about 280 mm. This was followed by further hinging of the columns on bayline F at a lateral displacement of 300 to 350 mm.

The maximum uplift and downward displacements for the footing on bayline F are about 26 mm corresponding to a bearing pressure of about 650 kPa. The corresponding column axial, shear and bending moment are 4200 kN in compression, 1600 kN and 8200 kN.m, respectively.

It is noted that assuming only dead load and no live load contribution resulted in maximum footing uplift of about 40 mm.

As shown in Fig. 6, the load deformation response of the frame for the two different loading patterns shows similar trend. The elastic limit of the frame is reached at a base shear of about 2000 kN with a corresponding level 5 lateral displacement of about 75 mm. The total base shears attracted by the frame at the end of pushover analyses were in the order of about 2.2 to 2.6 times that of the elastic limit. The maximum lateral displacement, however, is about five times that of the elastic limit (yield displacement). This is mainly attributed to the effect of foundation flexibility/rocking resulting in additional lateral displacement at the top of the frame.

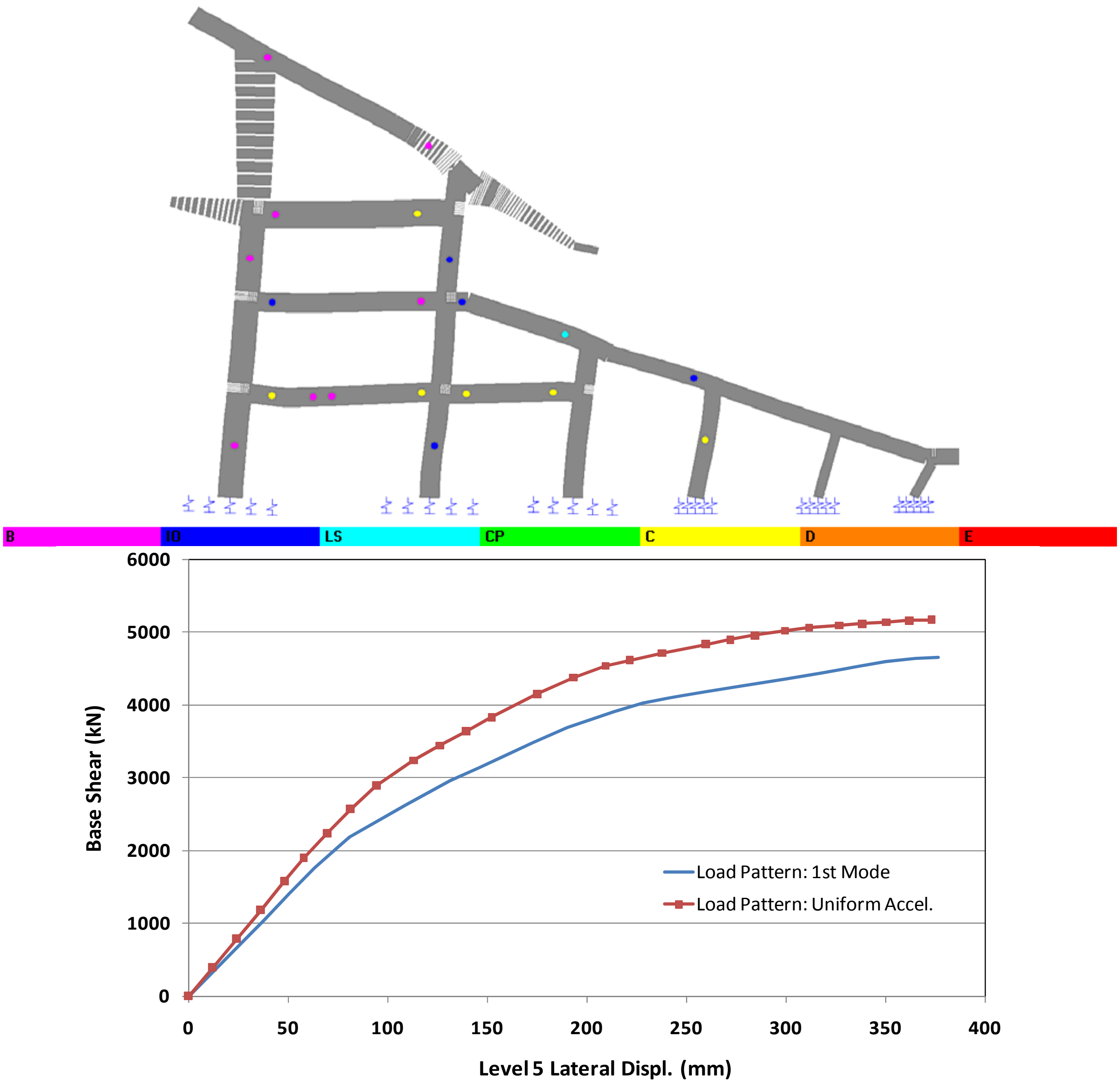

Figure 6: Extent of nonlinearity and plastic hinge formation at the end of pushover analysis (top) and load-displacement response of radial frame (bottom).

Pushover Analysis Results (Circumferential Frame)

Fig. 7 shows the displaced shape of the circumferential frame at the end of pushover analysis together with the load-displacement plots for the loading patterns described above. The color spectrum at the bottom of the figure indicates the extent of nonlinear action. The purple color (far left) indicates start of nonlinear behaviour, the dark blue color indicates limited yielding in the region for immediate occupancy, the light blue is the life safety zone, and the green color is the collapse prevention zone while the yellow color is near collapse region.

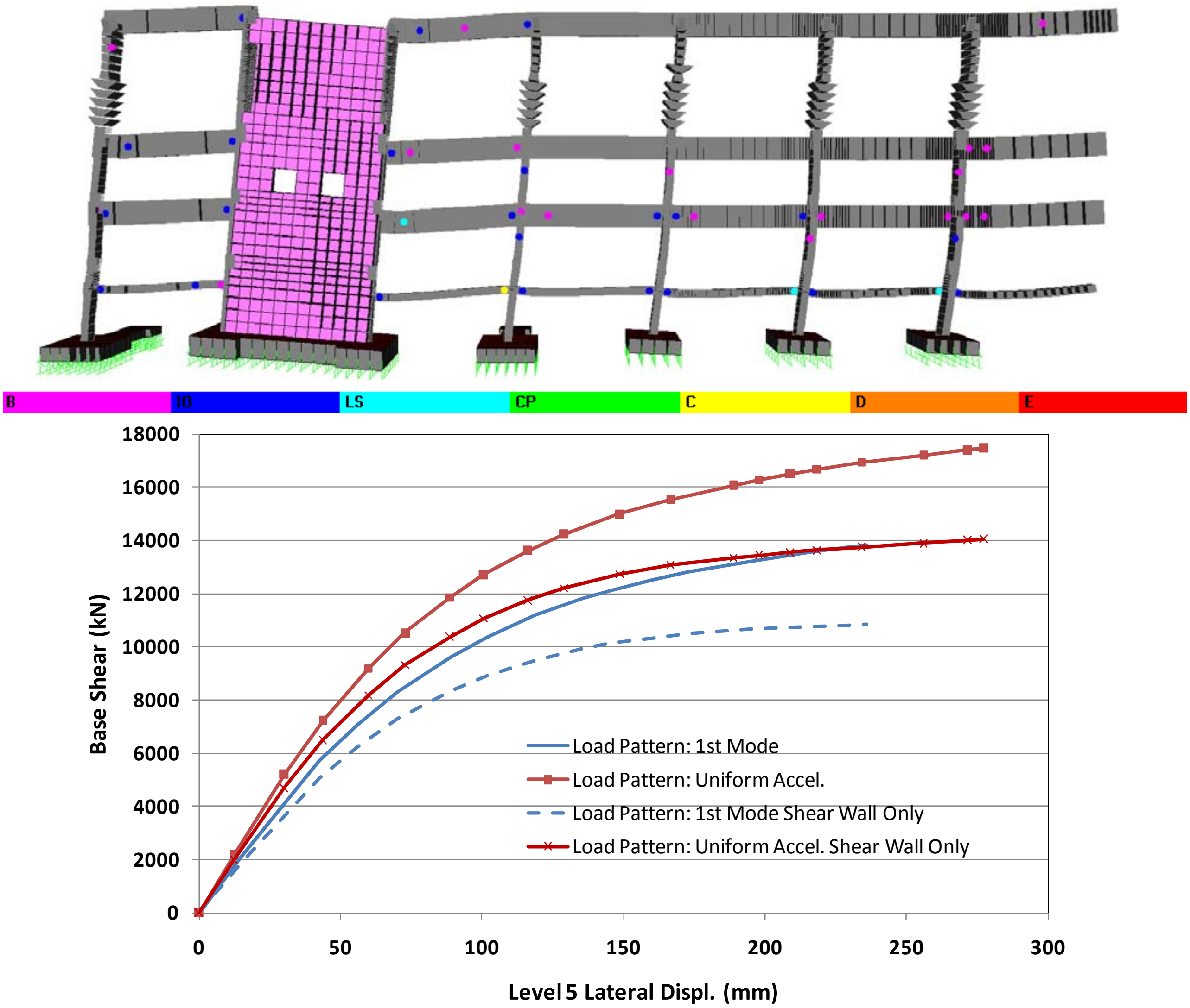

Figure 7: Extent of nonlinearity and plastic hinge formation at the end of pushover analysis (top) and load-displacement response of circumferential frame (bottom).

The first yielding occurred in the beams adjacent to the shear wall at levels 3 and 4 at a lateral displacement of about 45 mm. Subsequently, the beams in levels 2 and 5 adjacent to the shear wall developed plastic hinging near the ends at a lateral displacement of about 70 mm at the fifth level. At a lateral displacement of about 120 to 140 mm the first sign of column yielding was observed at the base of 5th and 4th level columns adjacent to the shear wall. The analysis terminated at a lateral displacement of about 250 mm where the 2nd level beams exceeded the collapse prevention criterion.

The maximum uplift and downward displacements for the footing under shear wall were about 40 mm and 85 mm, respectively, for 1st mode loading and 55 mm and 95 mm for uniform acceleration loading corresponding to a bearing pressure of about 900 to 1000 kPa. The spread footings under columns did not experience uplift with a maximum downward displacement of about 40 to 50 mm.

The maximum axial, shear and overturning moment demand for the shear wall are about 31000 kN in compression, 14000 kN and 190,000 kN-m, respectively, for uniform acceleration loading pattern. It can be observed from the load-displacement plot that the shear wall attracts 80% of the total base shear while the rest on input shear demand is distributed among the remaining five columns.

Conclusion

The results indicate the significance of soil-structure interaction and the effect of foundation rocking and soil yielding on the overall response of the building (e.g. reducing the base shear demand by about half of that assuming a fixed base). This behaviour would also result in increased displacement response of the superstructure and permanent foundation displacements. Further analysis is required to verify results, and additional analysis should consider a linked bowl structure to mitigate potential pounding between bowl segments that would largely result from the increased displacement associated with rocking foundations. Linking the bowl would have the added benefit of more evenly distributing lateral forces amongst the frame and wall elements throughout the building, and is currently being reviewed in a separate analysis to further consider the rocking foundation methodology.

Acknowledgments

B.C. Pavilion Corporation retained GENIVAR, in conjunction with EQ-Tec Engineering and Thurber Engineering Ltd. to carry out a seismic study of the BC Place Stadium.

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