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Comparison of Commonly-Used Pushover and Modal Pushover Analysis of Granville Bridge

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ABSTRACT

In current bridge engineering practice, the inelastic seismic demands of a bridge are usually estimated using response spectrum analysis (RSA) of bridges with reduced stiffnesses for realistic displacement demands. Non-linear static (pushover) analyses of each bent is then conducted separately to obtain post-elastic deformation effects in local components. As an extension to the pushover approach in use since the 1990's, a Modal Pushover Analysis (MPA) approach has been explored for the seismic assessment of building structures. In this study, RSA and MPA approaches are used in the seismic assessment of the Granville Bridge North Approach in Vancouver. Results show that the MPA approach results in an improved estimation of seismic demands by incorporating modal contribution and progressive collapse of the bridge into a 3D simulation. For the bridge studied in this project, the RSA approach resulted in conservative demands for small earthquakes and unconservative demands for larger earthquakes (e.g. 2% probability of occurrence in 50 years) in comparison to the MPA.

Keywords: Granville Bridge, Multi-Modal Pushover Analysis (MPA), Performance-Based Design (PBD), Seismic Assessment, Inelastic Demand.

INTRODUCTION

In Pushover Analysis (PA) practice, the displacement demands are usually estimated using Response Spectrum Analysis (RSA) of a bridge by reducing the initial stiffness of sub-structure to the secant stiffness at yield to account for the inelastic behavior [1, 2, 3, 4, 5]. However, this approach is a simplified treatment of the bridge response that relies on the elastic modal contributions.

As an extension of the pushover approach in use since the 1980's, a Modal Pushover Analysis (MPA) was presented by Chopra and Goel in [6], and has become an increasingly popular tool in the analysis of buildings owing to its conceptual simplicity while capturing both inelastic behavior and higher mode effects. Hence, it is an attractive analysis tool that can provide improved response estimates compared to PA without resorting to a more rigorous Nonlinear Time History Analysis (NTHA).

The MPA approach has been shown to give reliable estimates of peak inelastic response of structures, when compared to NTHA response. In addition, MPA has also been shown to be as accurate at estimating peak response well into the inelastic range, as RSA has been at estimating the elastic response. Despite these benefits, MPA has been slower to gain acceptance in the analysis of bridges, even for the analysis of more complex and irregular bridges where inelasticity can significantly affect the response.

The Granville Bridge in Vancouver is a highly irregular bridge that is composed of four distinctly-different structural systems. It has been rehabilitated and seismically retrofitted since its original construction in 1950, as part of which the main steel truss spans that have been base isolated from the seismically massive supporting piers. All of these differing systems and retrofit works make the bridges articulation complex and its predicted seismic response similarly difficult to estimate. This project involves a seismic assessment of the bridge and the design of a seismic instrumentation system, as part of which both MPA and PA were used to analyze the Northern approach spans of the bridge. The analysis results of the two procedures are compared, and their usefulness and ease of implementation from the perspective of Engineering Consulting practice are presented.

GRANVILLE BRIDGE DESCRIPTION

Granville Bridge was originally constructed circa 1950, and comprises numerous concrete approach and deck truss spans on the Granville Street alignment, with cast-in-place concrete spans for the connecting on- and off-ramps. As the scope of this assignment is limited to the North concrete approach spans, the bridge description presented is focused on those elements. Figure 1 below shows the Granville North Approach configuration.

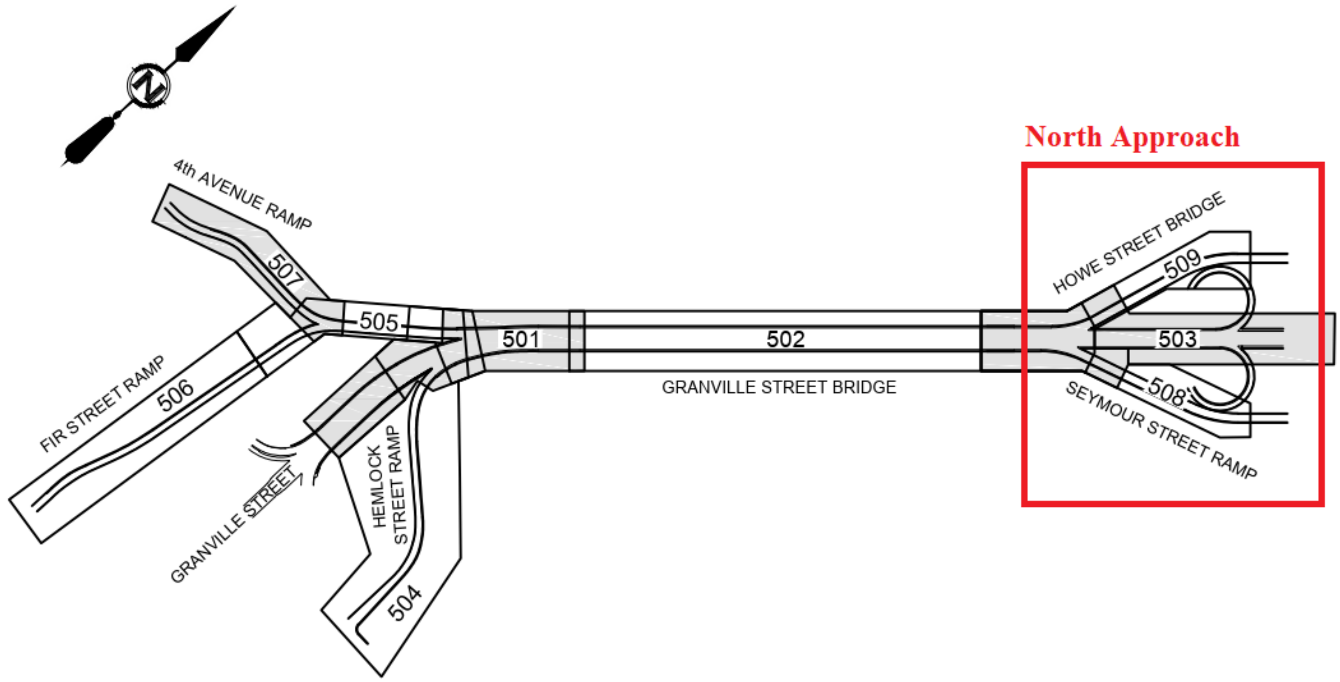

Figure 1: Granville Bridge Plan Configuration

Figure 2 shows representative photos of the Howe and Seymour Ramps. The Howe Ramp is about 163 m in length, spanning from Pier N25 to N34. The ramp comprises nine spans, which are approximately 18.3 m in length. The spans are reinforced concrete girders, and are cast monolithically with single-column concrete piers. Every second pier, there is a transverse joint running from the top of deck to the top of footing. The Seymour Ramp spans from Pier N49 to N59, and is about 182 m in length. Its configuration is similar to the Howe Ramp, comprising 10 spans of about 18 m, a similar cross section and two-span continuous 'split pier' articulation. The Granville Approach spans from Pier N8 to N22, and is about 297 m in length. It comprises 14 spans which are approximately 22 m in length, except for the spans between N20 and N22, which are about 15 m long. In all three approaches of North ramp, the pier columns vary significantly in height from about 9 m to more than 25 m.

Figure 2: Granville Bridge North Approach Typical Pier:
a) Howe and Seymour Ramp; b) Granville Ramp

Granville Bridge is classified as a "Lifeline Route" bridge as per the Canadian Highway Bridge Design Code (CHBDC) classification [7]. Figure 3 shows the site-specific spectra for the seismic assessment of Granville Bridge North approach at different seismic levels as per the CHBDC requirements.

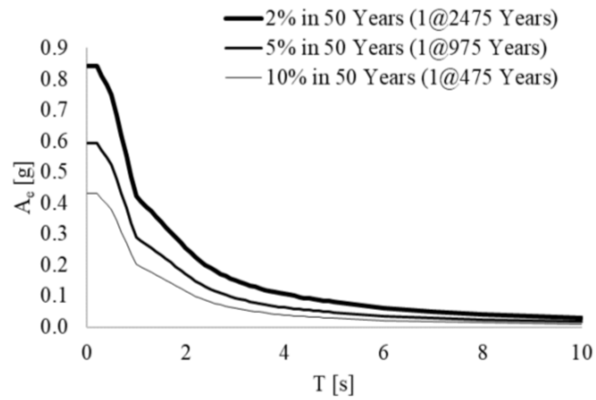

Figure 3: 5%–damped site-specific spectra for different seismic hazards

MPA APPROACH

The modal equation of motion for an inelastic bridge subjected to seismic ground acceleration, $\ddot{u}_g(t)$, is expressed by a set of *N* coupled equations as

$$\ddot{D}_n + 2\zeta_n\omega_n\dot{D}_n + \frac{F_{sn}}{L_n} = -\ddot{u}_g(t)(\mathrm{t}) \tag{1}$$

where,

$$F_{Sn} = \boldsymbol{\phi}_{\boldsymbol{n}}^{\boldsymbol{T}}\boldsymbol{f}_{\boldsymbol{s}}\big(\boldsymbol{D}, \mathrm{sign}\dot{\boldsymbol{D}}\big)(\mathrm{t}) \tag{2}$$

$$L_n = \Gamma_n M_n \tag{3}$$

D_n, ω_n, ζ_n, M_n, and Γ_n represent inelastic modal displacement, modal (angular) frequency, modal damping ratio, modal mass and modal participation factor of the n^{th} equivalent SDF respectively. In Eq. (3), $\boldsymbol{\phi}_{\boldsymbol{n}}$ denotes n^{th} natural vibration mode and $\boldsymbol{D}$ is the vector of D_n displacements. Solving Eq. (1) for each mode, MPA assumes that the lateral forces ($\boldsymbol{f}_{\boldsymbol{s}}$) are correlated to only one modal displacement, i.e. D_n. Therefore, Eq. (3) is written as

$$F_{Sn} = \boldsymbol{\phi}_{\boldsymbol{n}}^{\boldsymbol{T}}\boldsymbol{f}_{\boldsymbol{s}}\big(D_n, \mathrm{sign}\dot{D}_n\big) \tag{4}$$

In the MPA approach, the bridge is statically analysed under the modal static push forces and the relationship given in Eq. (4) is obtained for each mode. Then, Eq. (1) is solved for *N* inelastic SDFs and modal responses are combined using modal combination rules, e.g. SRSS, CQC, etc. F_{Sn} and D_n in Eq. (4) are related to the modal base shear, V_{bn}, and the displacement of a control joint, $\underline{u}_{cn}$, through the following equations

$$\begin{cases} F_{Sn} = \frac{V_{bn}}{\Gamma_n} \\ D_n = \frac{u_{cn}}{\Gamma_n \phi_{cn}} \end{cases} \tag{5}$$

In this study, the Capacity Spectrum Method (CSM) [8, 9] is used to solve inelastic modal equations. In the CSM, the inelastic spectral acceleration and displacement can be defined as

$$\begin{cases} A = \frac{A_e}{R} \\ D = CD_e \end{cases} \tag{6}$$

where, A_e and D_e are the elastic spectral acceleration and displacement respectively. Herein, the inelastic deformation ratio of site class “C” is estimated using the following equation from [10]

$$C = 1 + \left[\frac{1}{50\left(\frac{T}{0.85}\right)^{1.8}} + 0.0182\right](R - 1) \tag{7}$$

In the above equations, R and T denote the response reduction ratio and period respectively.

COMPUTER SIMULATION

A three-dimensional finite element (FE) model of the Granville Bridge North Approach was developed in MIDAS platform (Figure 4).

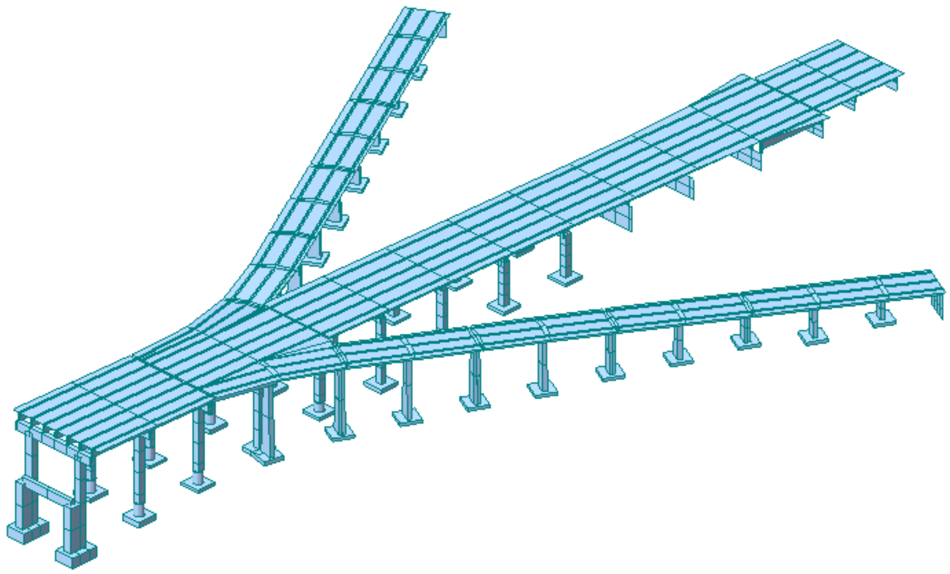

Figure 4. MIDAS model of the Granville Bridge North Approach

FE Model was used for two different types of analyses. First, the stiffness of structural components was reduced to the secant stiffness to yield (effective/cracked stiffness) and an RSA analysis was conducted to obtain inelastic seismic demands as the commonly used practice. Second, 108 plastic hinges were defined at both sides of the piers and cap beams and an MPA analysis was carried out as described earlier.

RESULTS AND DISCUSSIONS

In the MPA approach using CSM, the capacity of the inelastic SDFs are compared against the inelastic demand curves and the seismic demand for each mode is defined where the capacity and demand curves meet. Figure 5 and Figure 6 represent the CSM plots for mode 1 and Mode 2 respectively. Plots include demand curves for three different seismic events as per CHBDC requirements [7]. Figures 7 and 8 display the status of plastic hinge deformations in Mode 1 and Mode 2 for an earthquake with 2% probability of occurrence in 50 years.

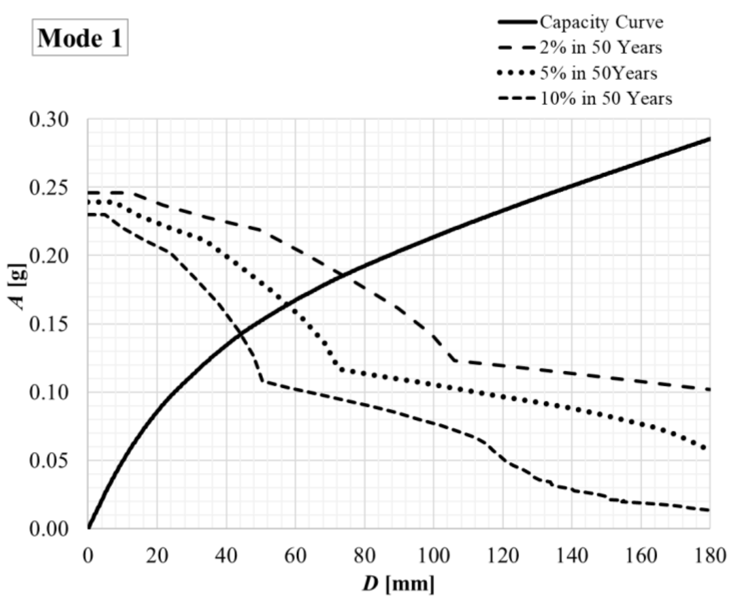

Figure 5. Spectral seismic demand of Mode 1 using CSM for different seismic events

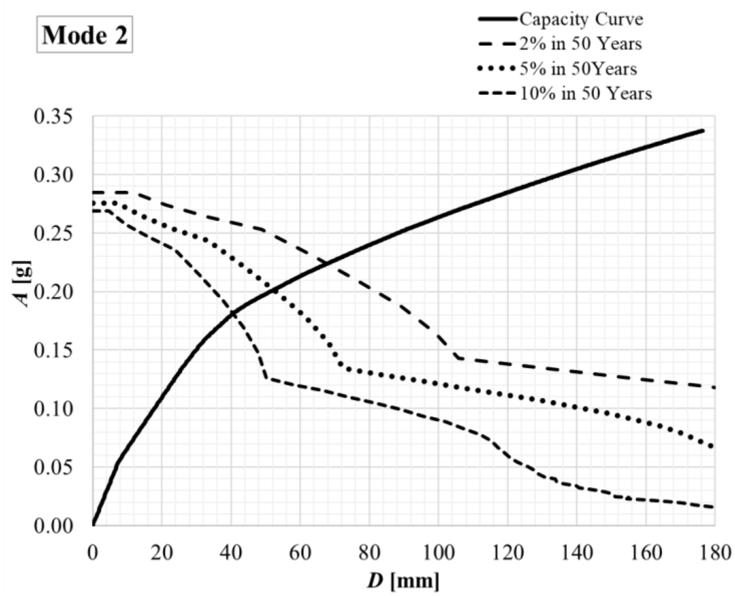

Figure 6. Spectral seismic demand of Mode 2 using CSM for different seismic events

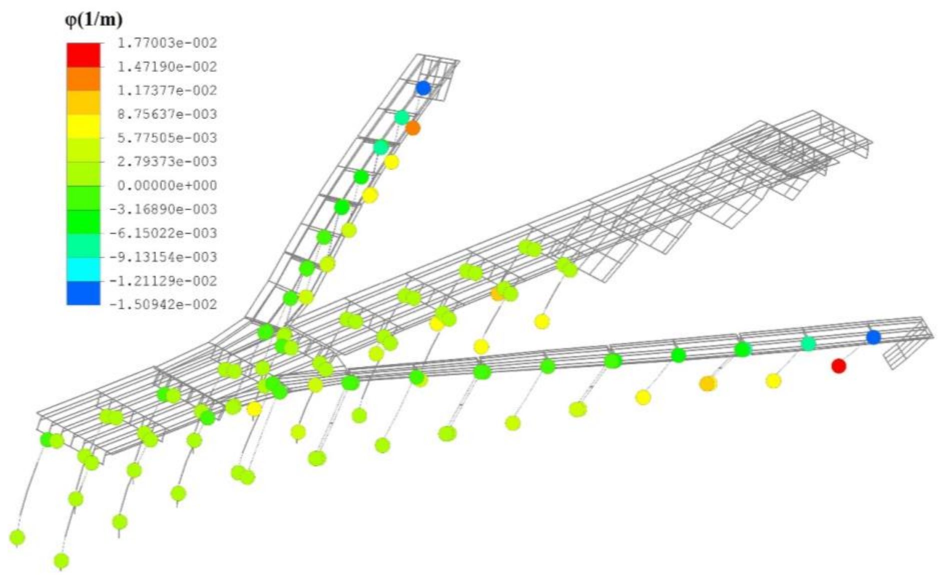

Figure 7. Plastic hinge deformations in Mode 1 for 2%-in-50-year earthquake

Table 1 to Table 3 compare the drift demand of bridge bents obtained by the MPA and RSA methods for different earthquake levels. The obtained results show that the MPA approach results in considerably lower seismic demands in both transverse and longitudinal directions for all bridge bents in all earthquake levels (up to 60%) except for the Bent 53 and Bent 54 in the biggest earthquake (2% probability of occurrence in 50 years). The reason behind these observations is that the seismic demand in MPA approach is defined based on the sequence of plastic hinge formations and progressive collapse of the bridge which are developed based on the modal contributions as it is pushed forward in accordance to a seismic demand level. Whereas, in the RSA approach, it is assumed that plastic hinges are formed everywhere in the bridge structure regardless of modal contribution and earthquake level. Hence, RSA approach generally results in overly conservative demands for small earthquakes and unconservative demands for bigger earthquakes by ignoring the effect of post-yield deformations which makes this approach less efficient especially for the seismic assessment of existing bridges.

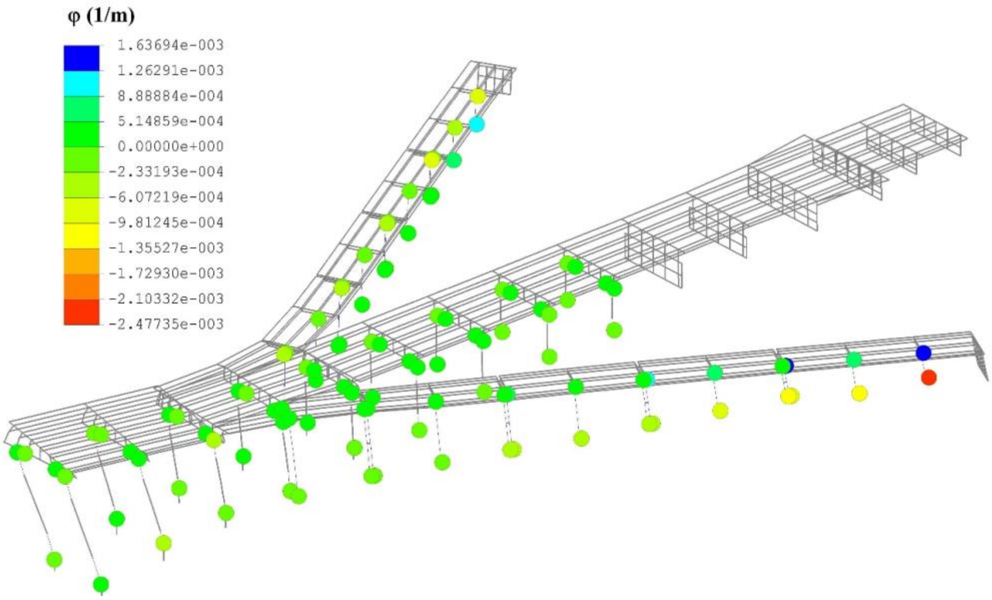

Figure 8. Plastic hinge deformations in Mode 2 for 2%-in-50-year earthquake

Table 1. Comparison of drift demands of RSA and MPA for ***2%-in-50-year earthquake***

Transverse | Longitudinal

Howe Ramp				**Granville Ramp**				**Seymour Ramp**			
Bent #	RSA mm	MPA mm	Diff. %	Bent #	RSA mm	MPA mm	Diff. %	Bent #	RSA mm	MPA mm	Diff. %
N25	74	57	**22**	**N8**	205	126	**39**	**N49**	76	56	**26**
	130	82	**37**		131	78	**40**		126	82	**35**
N26	85	58	**32**	**N9**	140	74	**47**	**N50**	89	57	**36**
	124	77	**38**		130	78	**40**		114	80	**30**
N27	87	54	**39**	**N10**	90	59	**35**	**N51**	90	58	**35**
	117	72	**38**		122	73	**40**		102	76	**26**
N28	79	44	**44**	**N11**	75	56	**26**	**N52**	58	56	**4.1**
	106	67	**37**		126	75	**41**		91	72	**21**
N29	68	34	**49**	**N12**	71	57	**21**	**N53**	30	47	**-59**
	98	62	**36**		110	68	**38**		85	69	**19**
N30	54	27	**50**	**N13**	51	37	**26**	**N54**	19	36	**-88**
	90	59	**34**		125	77	**39**		82	66	**19**
N31	43	22	**49**	**N14**	35	24	**32**	**N55**	77	24	**68**
	84	57	**32**		109	71	**35**		121	64	**47**
N32	31	18	**44**	**N15**	24	15	**38**	**N56**	95	16	**83**
	79	56	**29**		113	72	**36**		108	63	**42**
N33	21	13	**36**	**N16**	14	8.5	**41**	**N57**	77	13	**83**
	76	55	**27**		103	68	**34**		97	62	**35**
								N58	42	9.9	**76**
									88	62	**29**

CONCLUSIONS

As part of the seismic assessment of the Granville Bridge North Approach, the modal pushover analysis method was studied against commonly-used RSA and pushover analysis to assess the global and local seismic demands of bridges and their components for different seismic events. Results obtained showed that the MPA approach resulted in an improved estimation of inelastic seismic demands by using modal contribution and 3D progressive collapse of the bridge due to plastic hinge formation. However, the RSA approach resulted in conservative demands for smaller earthquakes (with 5% and 10% probability of exceedance in 50 years) and unconservative demands for a large earthquake (2% probability of exceedance in 50 years) by neglecting the effect of modal contribution, sequence of hinge formation and post-yield deformations.

*Table 2. Comparison of drift demands of RSA and MPA for **5%-in-50-year earthquake***

Transverse (blue rows) | Longitudinal (light rows)

Howe Ramp				**Granville Ramp**				**Seymour Ramp**			
Bent #	RSA mm	MPA mm	Diff. %	Bent #	RSA mm	MPA mm	Diff. %	Bent #	RSA mm	MPA mm	Diff. %
N25	51	46	**10**	**N8**	141	101	**28**	**N49**	52	44	**15**
	89	64	**28**		90	61	**32**		86	64	**26**
N26	59	47	**21**	**N9**	96	58	**40**	**N50**	54	46	**14**
	85	60	**29**		89	61	**32**		83	62	**26**
N27	61	43	**29**	**N10**	62	45	**27**	**N51**	62	47	**24**
	80	56	**30**		84	57	**33**		78	58	**25**
N28	55	36	**35**	**N11**	52	44	**15**	**N52**	66	45	**32**
	73	52	**29**		86	57	**34**		74	56	**25**
N29	47	28	**42**	**N12**	49	45	**8.8**	**N53**	62	38	**40**
	67	48	**28**		75	51	**32**		70	53	**25**
N30	37	21	**44**	**N13**	35	29	**18**	**N54**	53	28	**47**
	61	45	**26**		86	59	**32**		66	51	**24**
N31	30	17	**44**	**N14**	25	17	**29**	**N55**	40	19	**53**
	58	44	**24**		75	55	**27**		63	49	**22**
N32	22	13	**39**	**N15**	17	10	**37**	**N56**	29	13	**57**
	54	43	**21**		77	54	**30**		60	48	**20**
N33	15	10	**29**	**N16**	10	5.9	**41**	**N57**	21	9.6	**53**
	52	42	**18**		71	53	**25**		58	48	**18**
								N58	13	7.5	**43**
									56	47	**16**

*Table 3. Comparison of drift demands of RSA and MPA for **10%-in-50-year earthquake***

Transverse (blue rows) | Longitudinal (light rows)

Howe Ramp				**Granville Ramp**				**Seymour Ramp**			
Bent #	RSA mm	MPA mm	Diff. %	Bent #	RSA mm	MPA mm	Diff. %	Bent #	RSA mm	MPA mm	Diff. %
N25	36	29	**19**	**N8**	98	79	**20**	**N49**	37	29	**20**
	62	48	**22**		63	46	**26**		60	48	**20**
N26	42	30	**28**	**N9**	67	46	**31**	**N50**	38	30	**21**
	59	45	**24**		62	46	**25**		58	46	**20**
N27	43	28	**34**	**N10**	43	32	**26**	**N51**	44	31	**30**
	56	42	**25**		58	43	**26**		54	43	**20**
N28	39	23	**40**	**N11**	37	29	**21**	**N52**	47	30	**37**
	51	38	**24**		60	44	**27**		52	41	**21**
N29	34	18	**47**	**N12**	35	28	**19**	**N53**	44	25	**44**
	47	36	**23**		52	37	**30**		49	39	**20**
N30	27	14	**49**	**N13**	25	18	**27**	**N54**	38	19	**51**
	43	34	**21**		60	44	**26**		46	37	**19**
N31	21	11	**48**	**N14**	18	11	**35**	**N55**	29	12	**57**
	40	32	**19**		52	41	**21**		43	36	**17**
N32	16	8.9	**43**	**N15**	12	7.2	**40**	**N56**	21	8.2	**60**
	37	31	**16**		54	39	**27**		42	35	**16**
N33	10	6.8	**34**	**N16**	7.3	4.3	**41**	**N57**	15	6.4	**56**
	36	31	**14**		49	40	**19**		40	35	**14**
								N58	9.4	5	**46**
									39	35	**11**

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