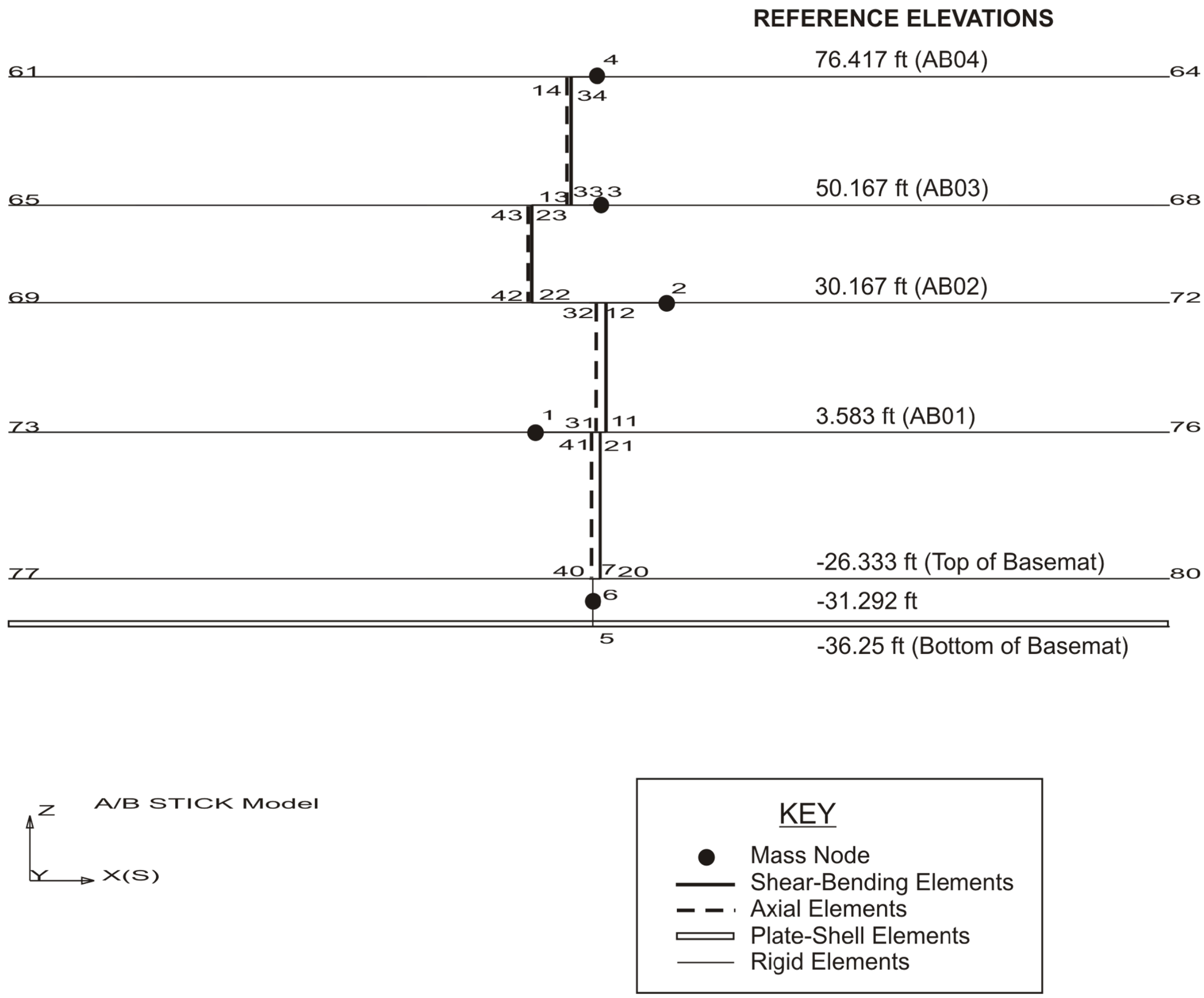


Figure 2.5-1 SASSI Lumped Mass Stick Model of the A/B in the XZ plane

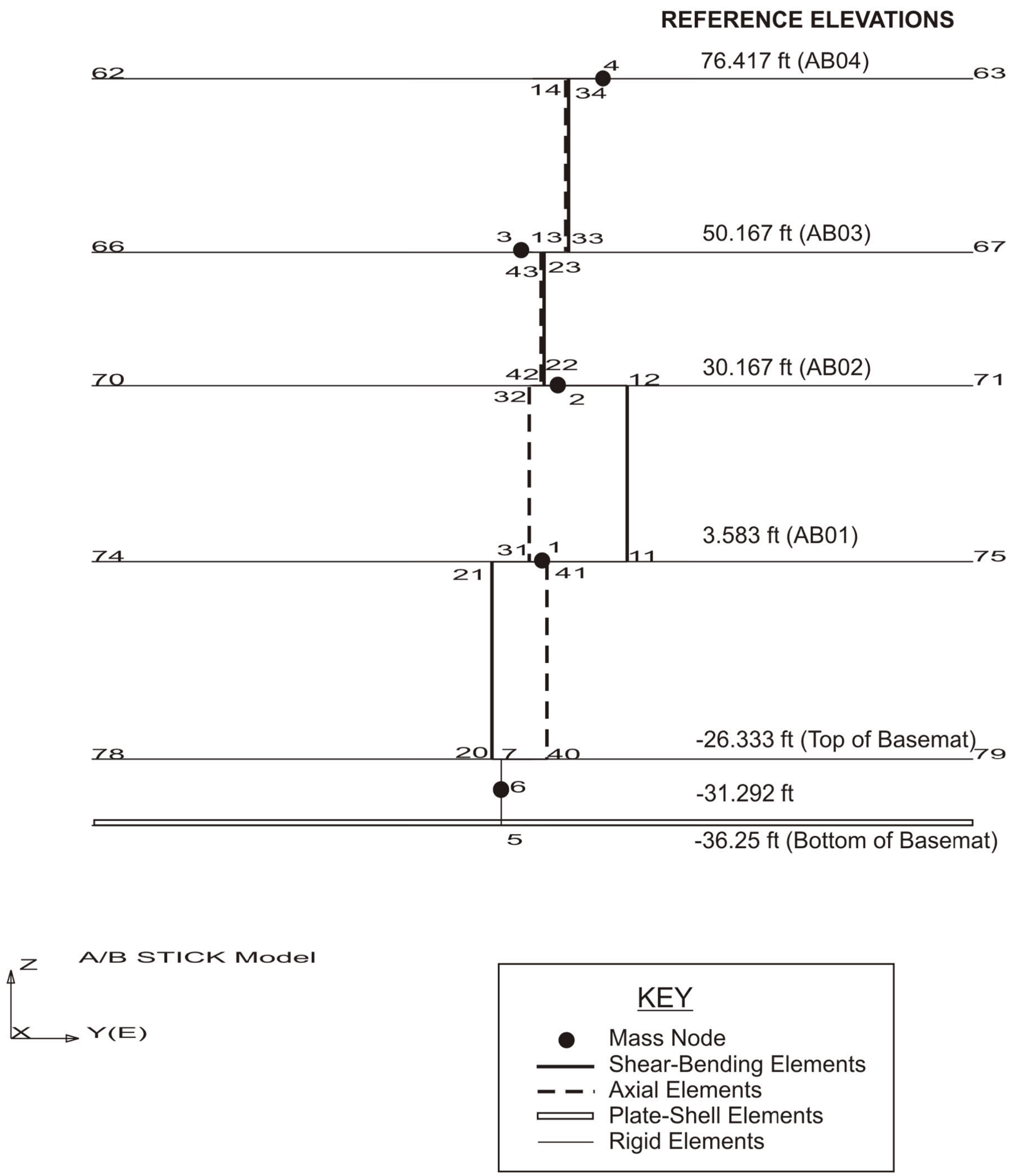


Figure 2.5-2 SASSI Lumped Mass Stick Model of the A/B in the YZ plane

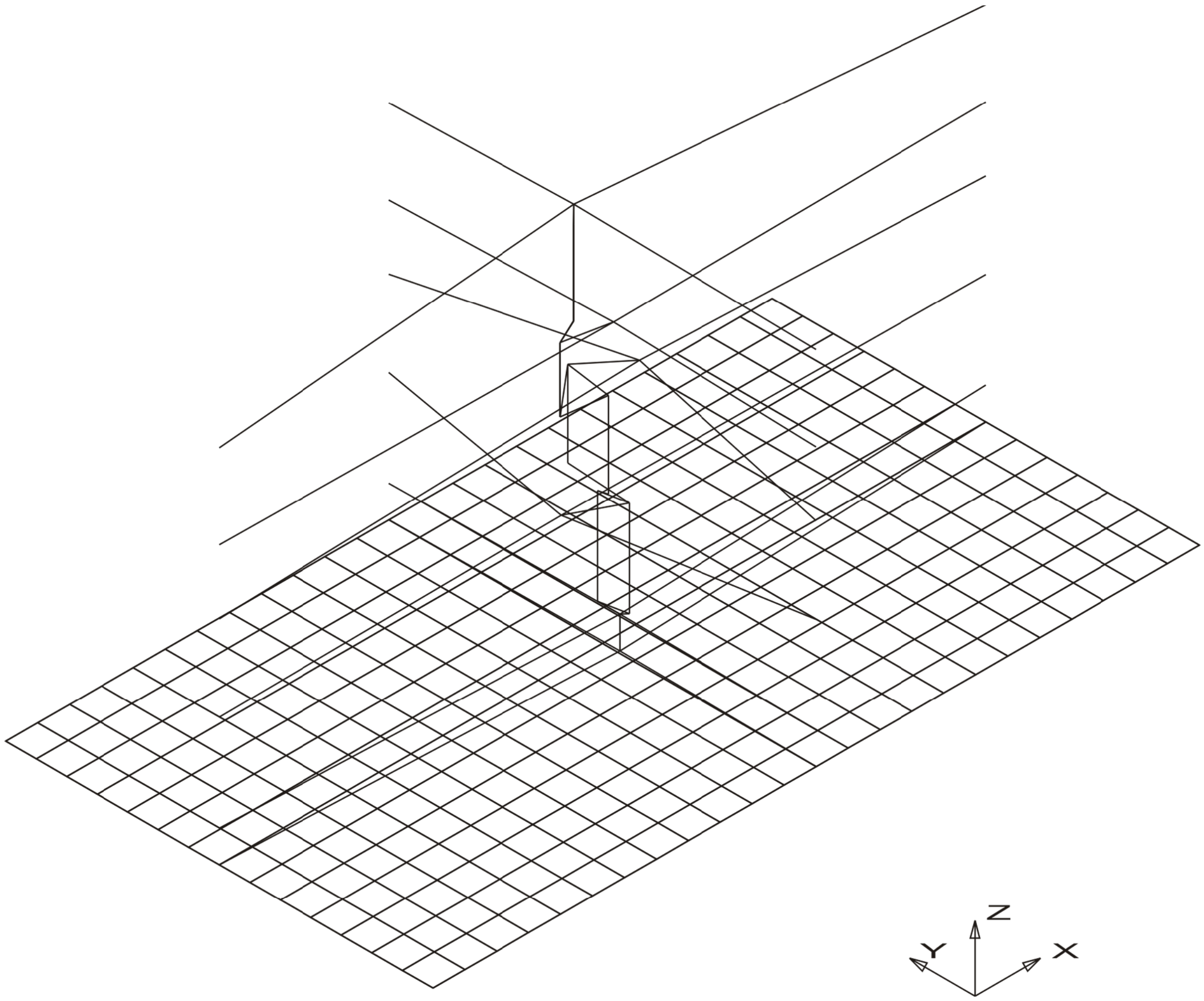


Figure 2.5-3 Isometric View of SASSI Lumped Mass Stick Model of A/B

3.0 VALIDATION OF A/B DYNAMIC MODELS

3.1 1g Static Verification Analysis Results

The mass and stiffness properties of both the Dynamic and Detailed FE models are compared to ensure the validity of the models. The seismic mass associated with each major floor elevation is calculated for both of the FE models and then compared with each other in Table 3.1-1 and Table 3.1-2.

ANSYS static analyses of both FE models are performed by establishing fixed boundary conditions at the base of the models and applying 1g static load to all masses in the structure. The results of these 1-g static analyses for deflections at selected nodes are compared with each other in Figures 3.1-1 through 3.1-8 which confirms that both models are consistent and properly represent the mass and stiffness properties of the A/B.

Table 3.1-1 Mass Distribution of FE Models

Location	Weight (10^3 kips) (1) Dynamic FE Model	Weight (10^3 kips) (2) Detailed FE Model	(1)/(2) Ratio
Fourth Floor / Roof	22.29	21.65	103%
Third Floor	33.94	31.42	108%
Second Floor	42.40	39.79	107%
Ground Floor	59.13	57.73	102%
Base Mat	83.62	81.32	103%

Table 3.1-2 Mass Properties of FE Models

Model	Mass Center Coordinates (ft)			Mass Inertia about Center of Mass (kip-ft^2)		
	Xc	Yc	Zc	IXX	IYY	IZZ
(1) Dynamic FE Model	5.231	5.095	9.993	2.16E+07	4.74E+06	5.09E+06
(2) Detailed FE Model	5.274	5.092	9.996	2.05E+07	4.54E+06	4.83E+06
Ratio (1)/(2)				105%	105%	105%

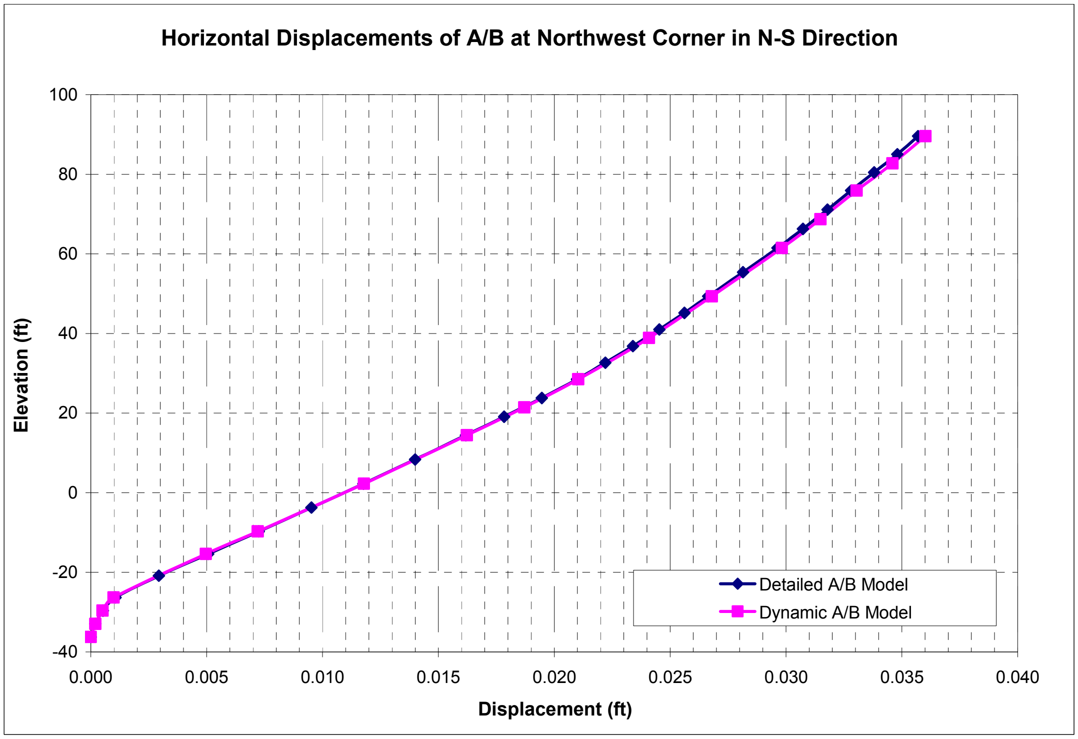


Figure 3.1-1 N-S 1g Static Analysis of FE Models at NE Corner

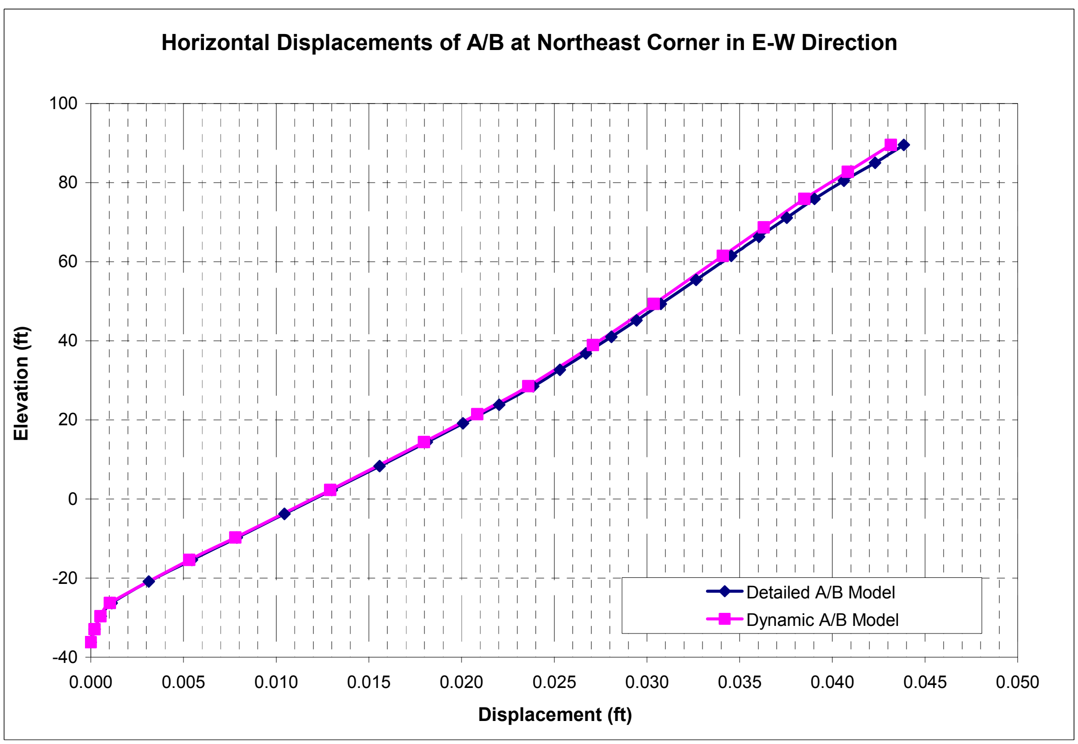


Figure 3.1-2 E-W 1g Static Analysis of FE Models at NE Corner

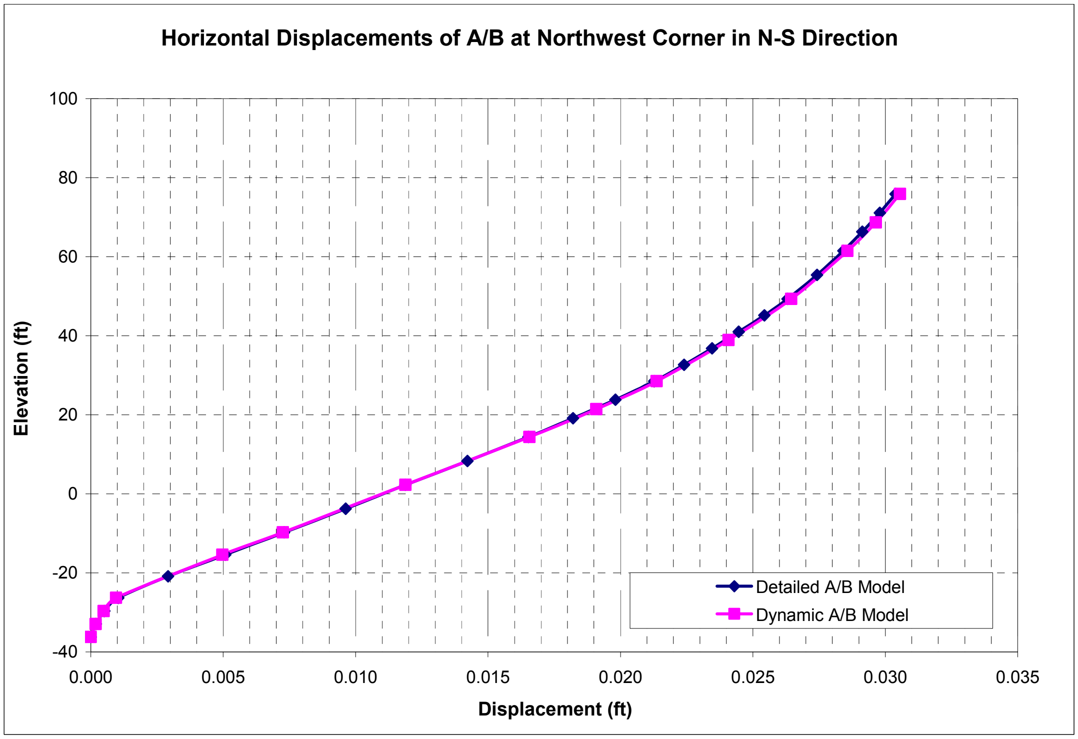


Figure 3.1-3 N-S 1g Static Analysis of FE Models at NW Corner

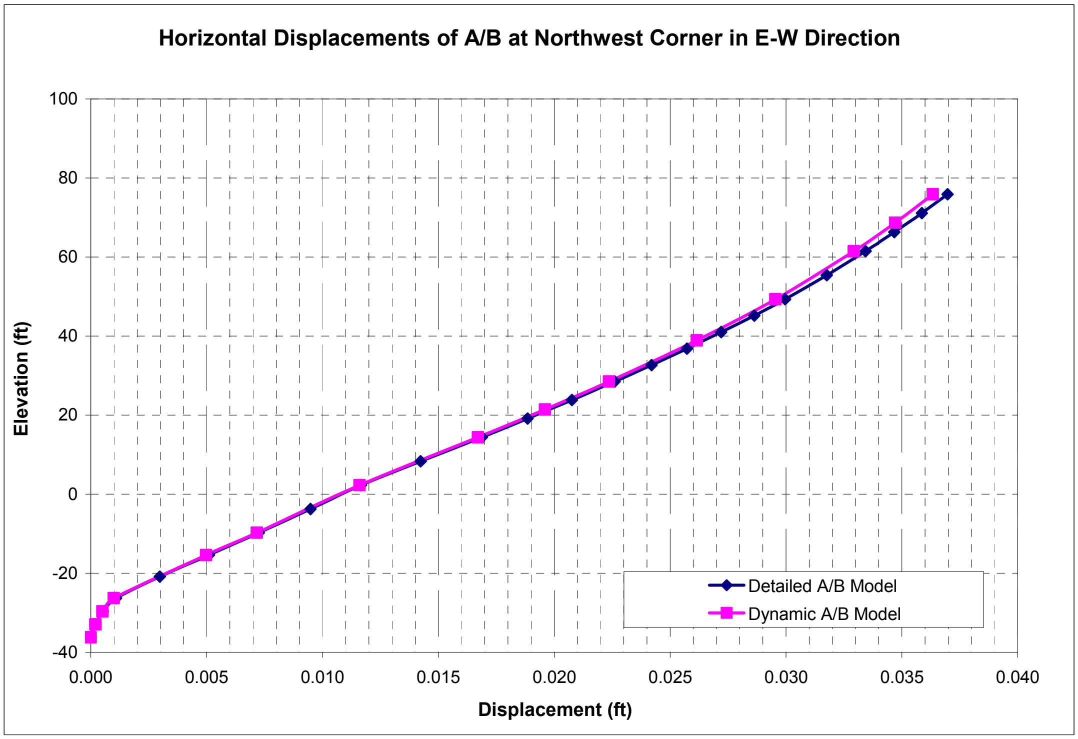


Figure 3.1-4 E-W 1g Static Analysis of FE Models at NW Corner

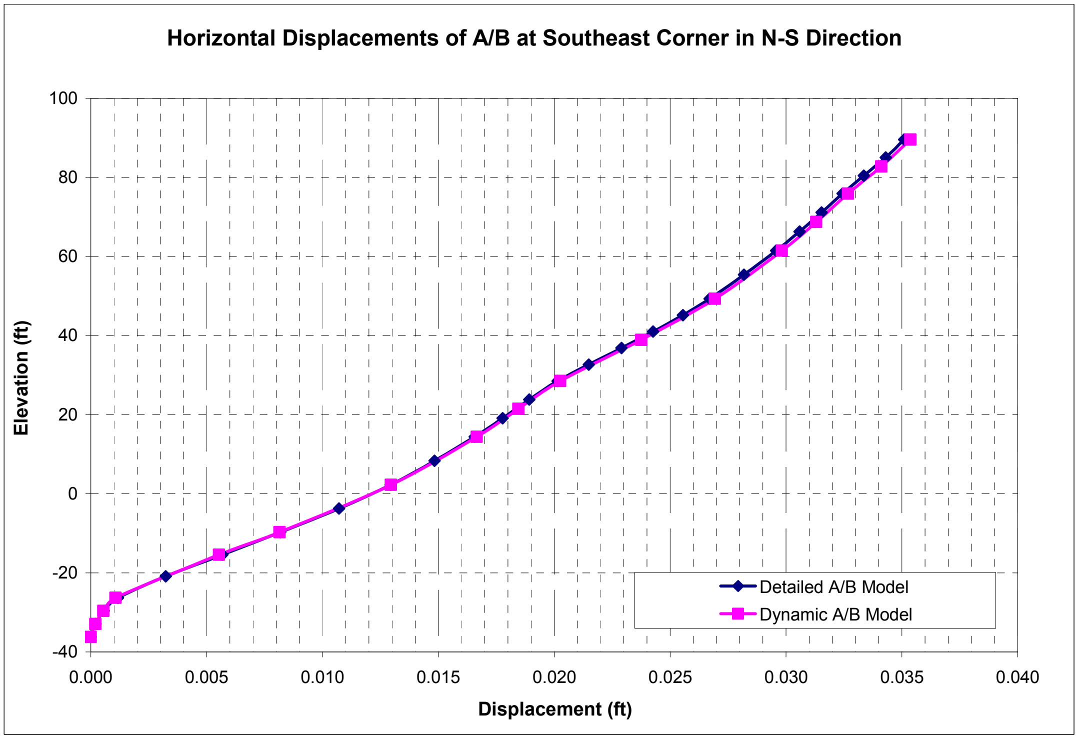


Figure 3.1-5 N-S 1g Static Analysis of FE Models at SE Corner

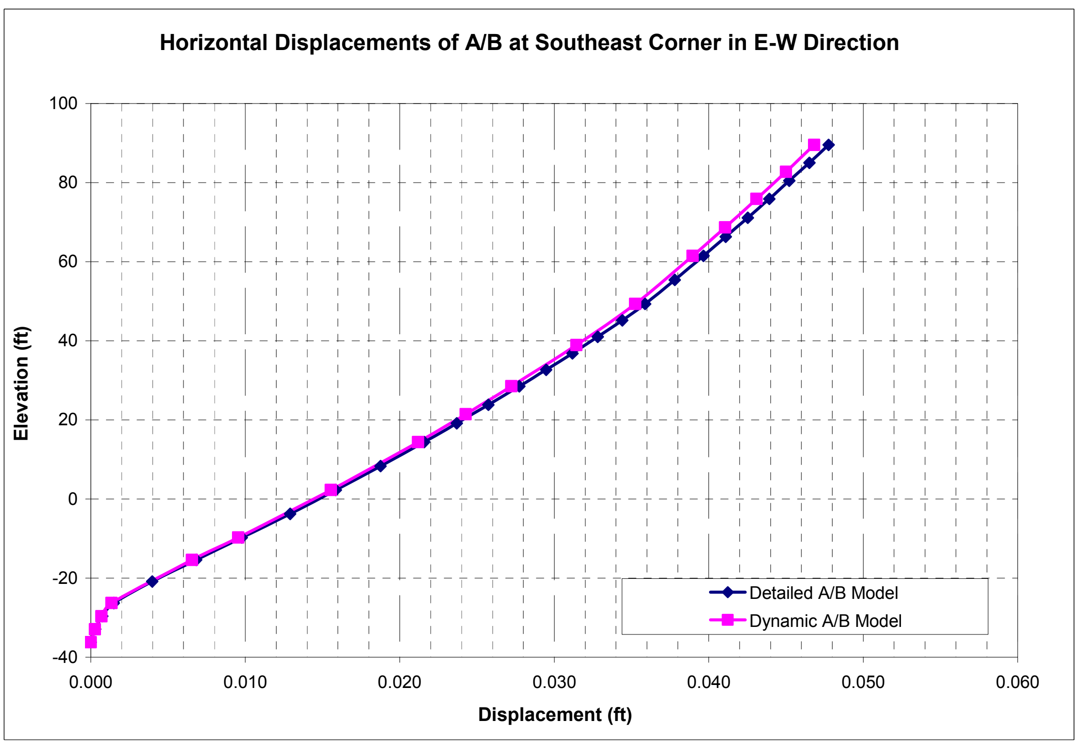


Figure 3.1-6 E-W 1g Static Analysis of FE Models at SE Corner

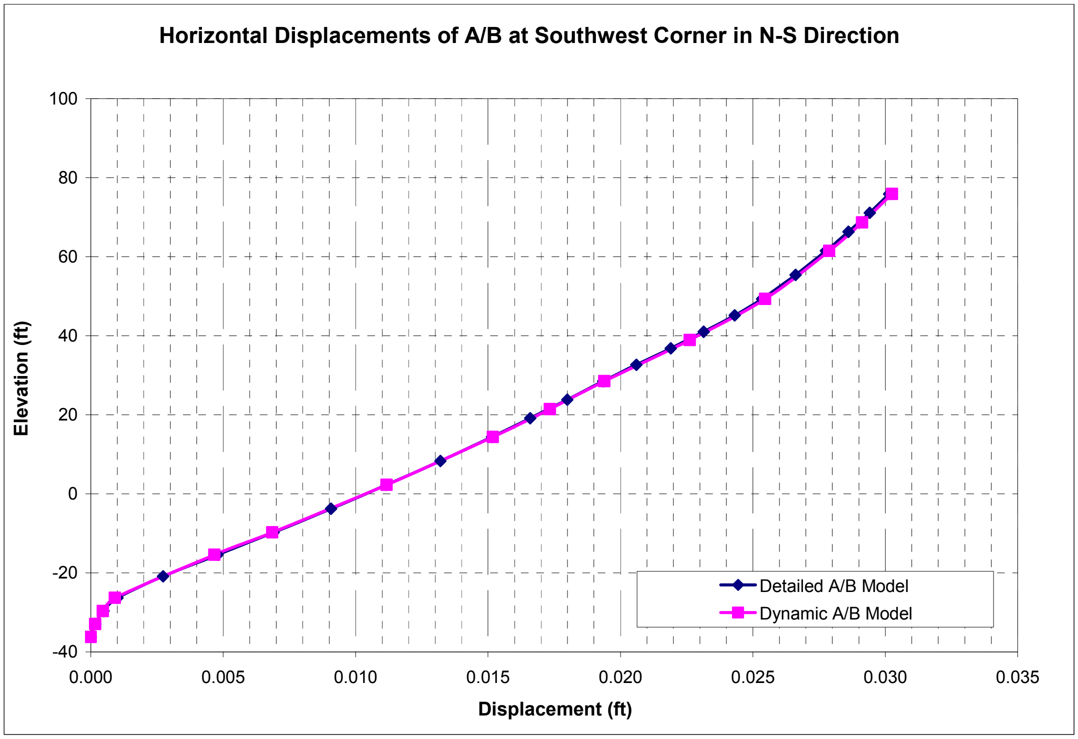


Figure 3.1-7 N-S 1g Static Analysis of FE Models at SW Corner

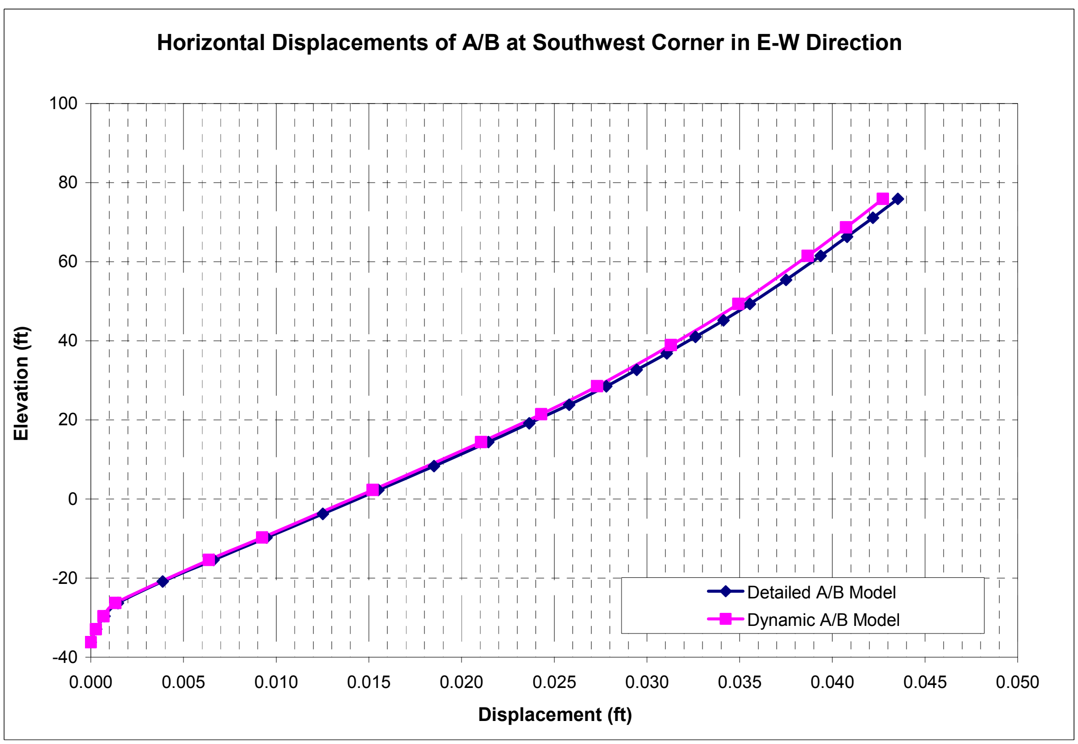


Figure 3.1-8 E-W 1g Static Analysis of FE Models at SW Corner

3.2 Modal Analysis Results and Mesh Size Verification

Modal analyses of both the Dynamic and Detailed FE models are performed in ANSYS with fixed boundary conditions established at the base of the A/B basemat. Table 3.2-1 summarizes the results of the modal analyses of both FE models for the dominant natural frequencies in both horizontal N-S and E-W directions and the vertical direction. The results of the modal analyses for cumulative mass participation are plotted as function of frequency in Figure 3.2-1, Figure 3.2-2 and Figure 3.2-3 for the N-S, E-W and vertical directions. The conformity of the dominant-mode frequencies obtained from the modal analyses of both FE models and the cumulative mass fraction plots indicates the models properly capture the dynamic behavior of the A/B and that no further refinement of the mesh is required for the two dynamic FE models.

Table 3.2-1 Frequencies of Dominant Modes (Hz)

North-South (X) direction		
Mode	Frequency (Hz)	Period (sec)
Detailed A/B Model		
2	5.678	0.1761
11	9.946	0.1006
13	10.133	0.0987
20	11.138	0.0898
24	11.708	0.0854
Dynamic A/B Model		
2	5.650	0.1770
12	9.830	0.1017
21	10.882	0.0919
33	11.909	0.0840
35	12.028	0.0831

East-West (Y) direction		
Mode	Frequency (Hz)	Period (sec)
Detailed A/B Model		
1	4.703	0.2126
5	8.997	0.1112
6	9.202	0.1087
44	13.102	0.0763
56	14.065	0.0711
Dynamic A/B Model		
1	4.738	0.2111
9	9.136	0.1095
51	13.203	0.0757
59	13.776	0.0726
62	13.947	0.0717

Vertical (Z) direction		
Mode	Frequency (Hz)	Period (sec)
Detailed A/B Model		
13	10.133	0.0987
14	10.591	0.0944
20	11.138	0.0898
23	11.647	0.0859
35	12.494	0.0800
Dynamic A/B Model		
11	9.697	0.1031
13	9.895	0.1011
19	10.591	0.0944
26	11.318	0.0884
35	12.028	0.0831

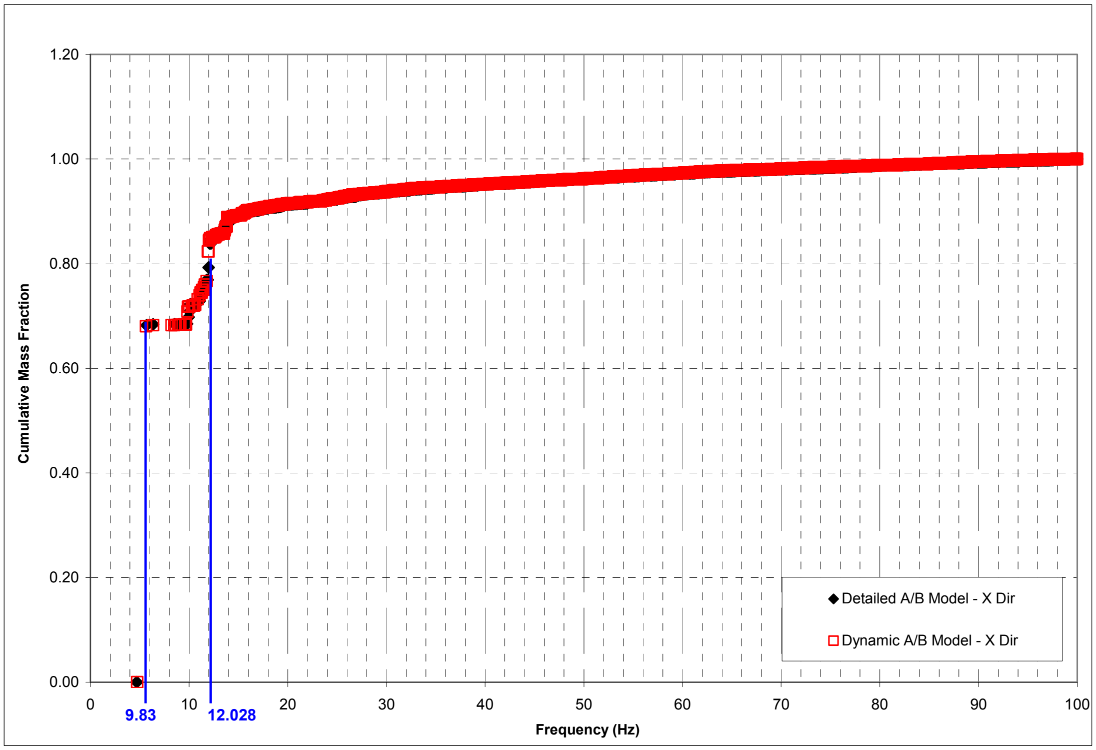


Figure 3.2-1 N-S Mass Participation Dynamic Properties of A/B Models

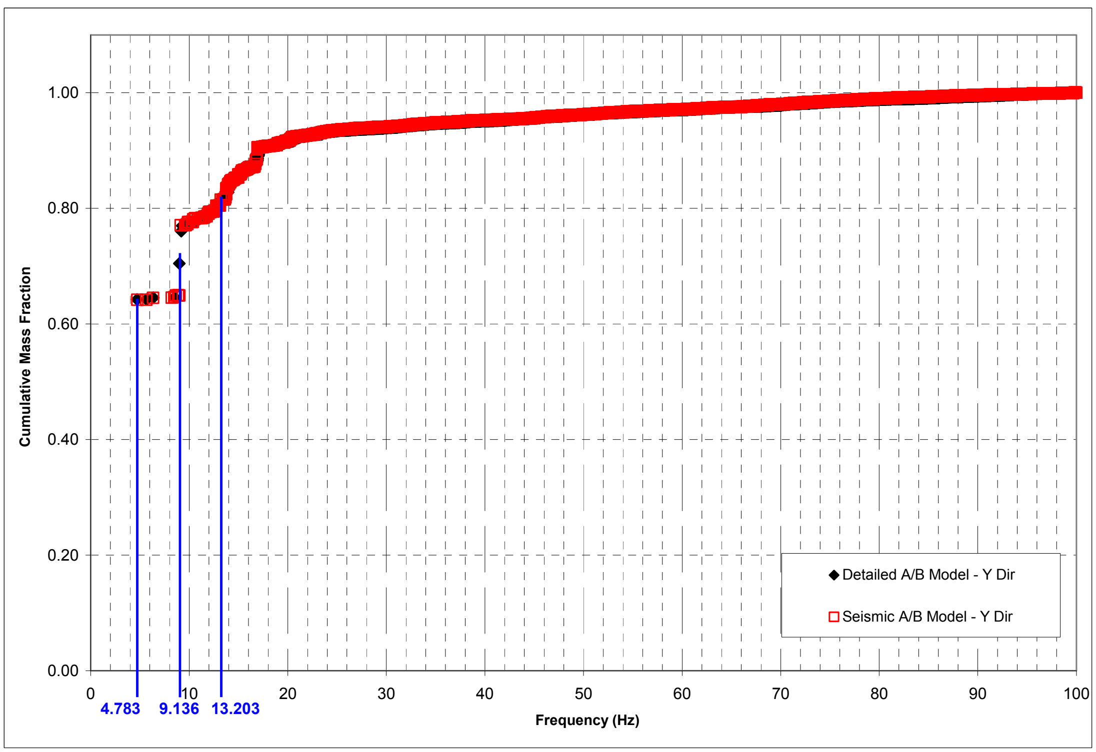


Figure 3.2-2 E-W Mass Participation Dynamic Properties of A/B Models

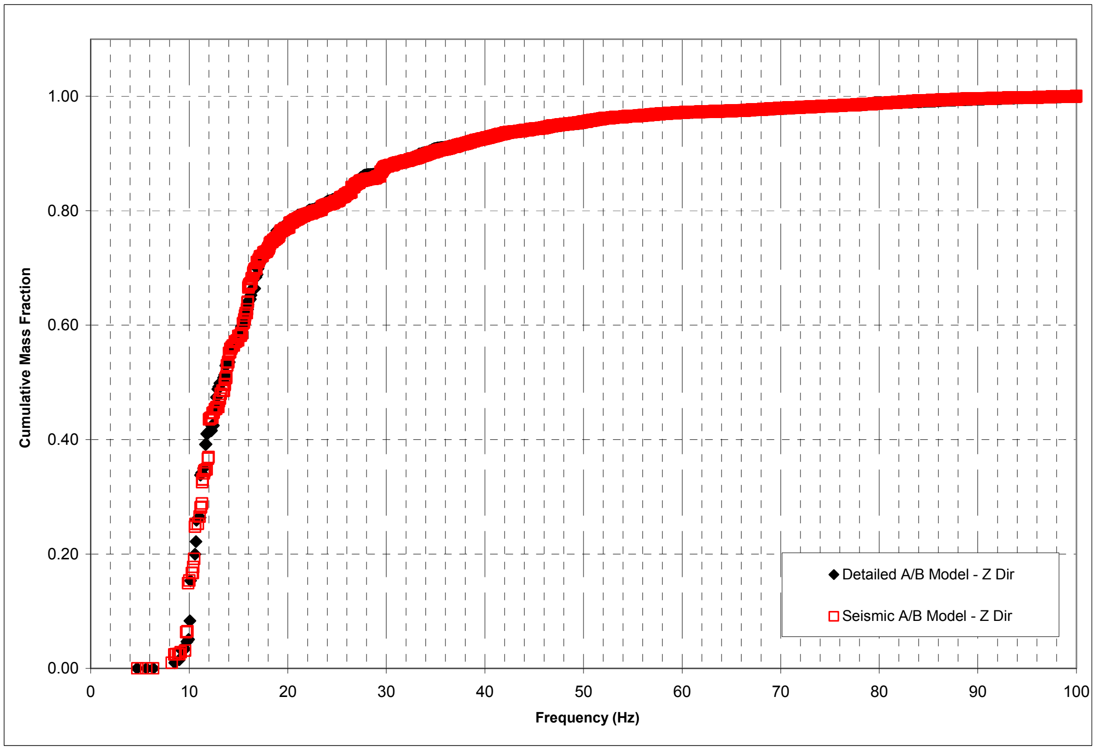


Figure 3.2-3 Vertical Mass Participation Dynamic Properties of A/B Models

3.3 Validation of Model Translation from ANSYS to SASSI

An SSI analysis is performed in ACS SASSI with the dynamic FE model resting on the surface of an elastic half-space with high stiffness that approximates fixed-base conditions. Figure 3.3-1 and Figure 3.3-2 present the results of the validation SASSI analyses for acceleration transfer functions at selected locations. These figures show that the peak amplifications of the transfer functions occur at or close to the values of the natural frequencies of the dominant modes shown in Table 3.2-1 and Figure 3.2-1 and Figure 3.2-2, which indicates that the translation of the A/B dynamic FE model into SASSI format is accurate.

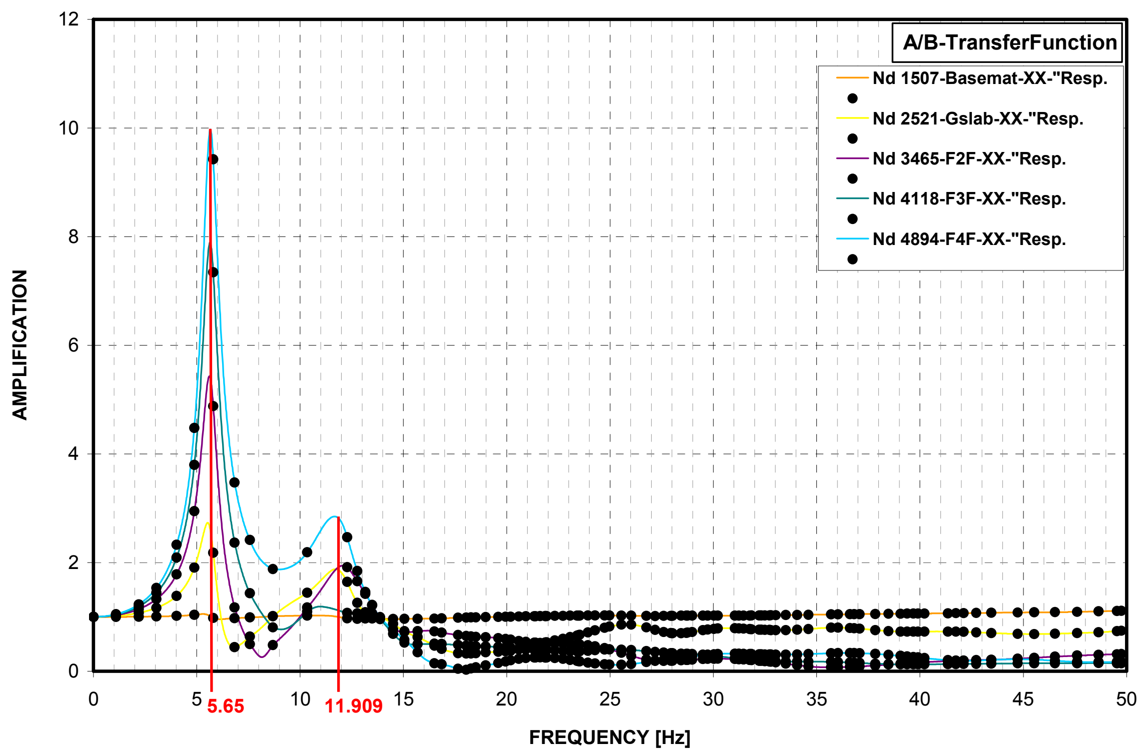


Figure 3.3-1 N-S Transfer Functions, Dynamic FE Model

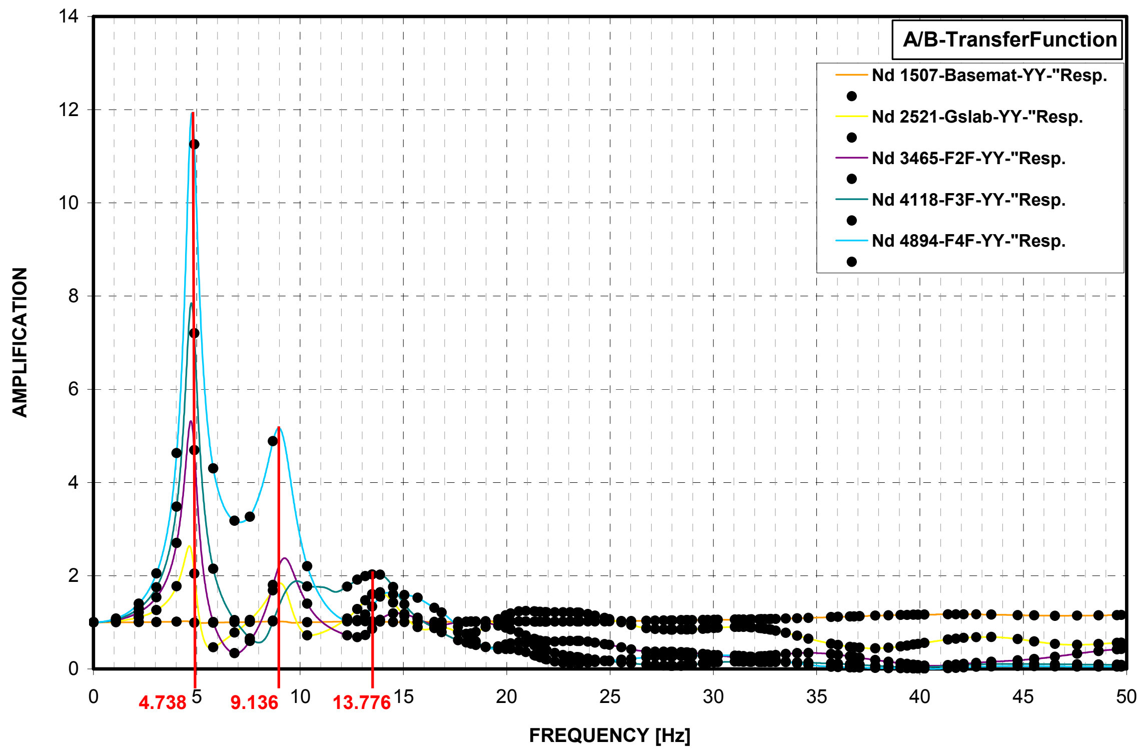


Figure 3.3-2 E-W Transfer Functions, Dynamic FE Mode

3.4 Validation of Lumped Mass Stick Model

Section 3.4 was not completely updated in revision 1 and may be out of date. This data will be updated in future revisions of this report when the applicable analysis is complete.

In order to verify that the simplified lumped-mass stick model developed as described in Section 2.5 is dynamically equivalent to the dynamic FE model of the A/B, the fixed-base dynamic properties of the lumped-mass stick model are correlated with the fixed-base dynamic properties of the SASSI FE models. The correlation is made by comparing the horizontal (NS and EW) seismic response transfer functions computed from equivalent fixed-base SSI analyses of the lumped-mass stick model and the SASSI FE models resting on the surface of very stiff elastic half-space using SASSI. These transfer functions computed from the lumped-mass stick model at selected locations of A/B are compared with the corresponding transfer functions computed from the dynamic FE model. Figure 3.4-1 and Figure 3.4-2, respectively, compare the transfer function amplitudes computed for the horizontal NS and EW seismic responses at the roof elevation (El. 75.9 ft). As indicated from these figures, the correlation between the dynamic properties of the fixed-base lumped-mass stick model and that of the fixed-base FE model are reasonable for the fundamental-mode responses.

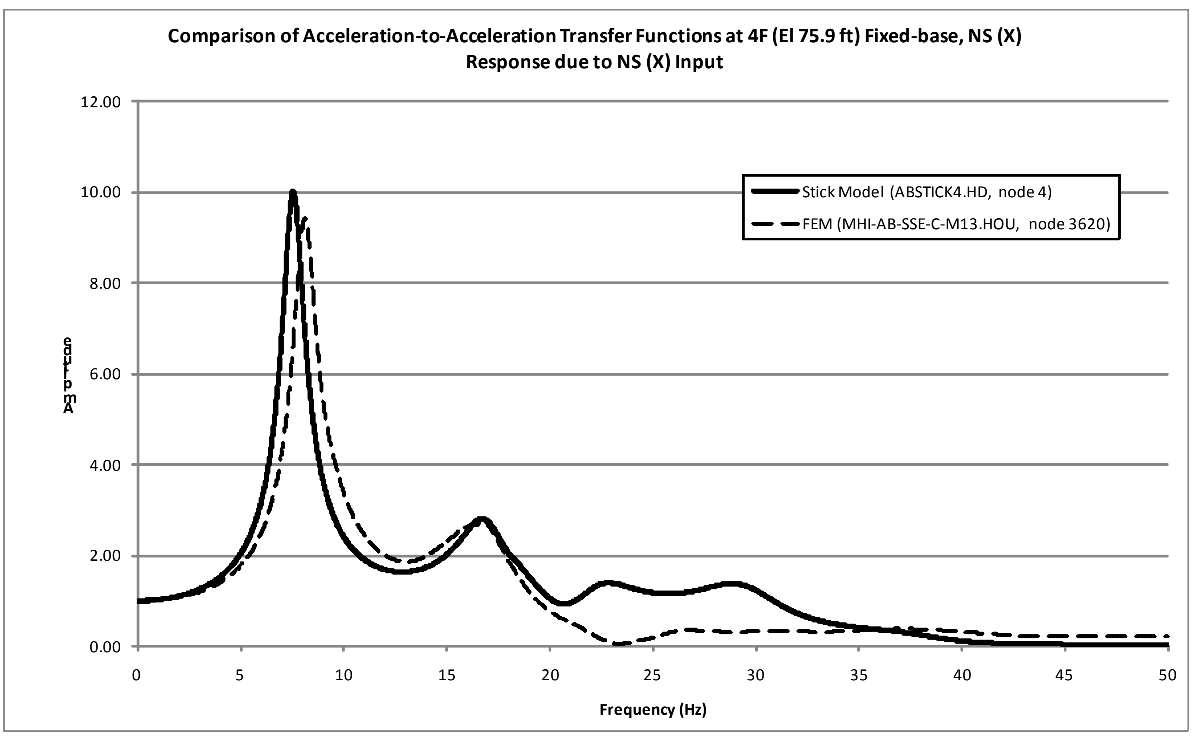


Figure 3.4-1 Transfer Function Amplitudes for NS Response of Roof

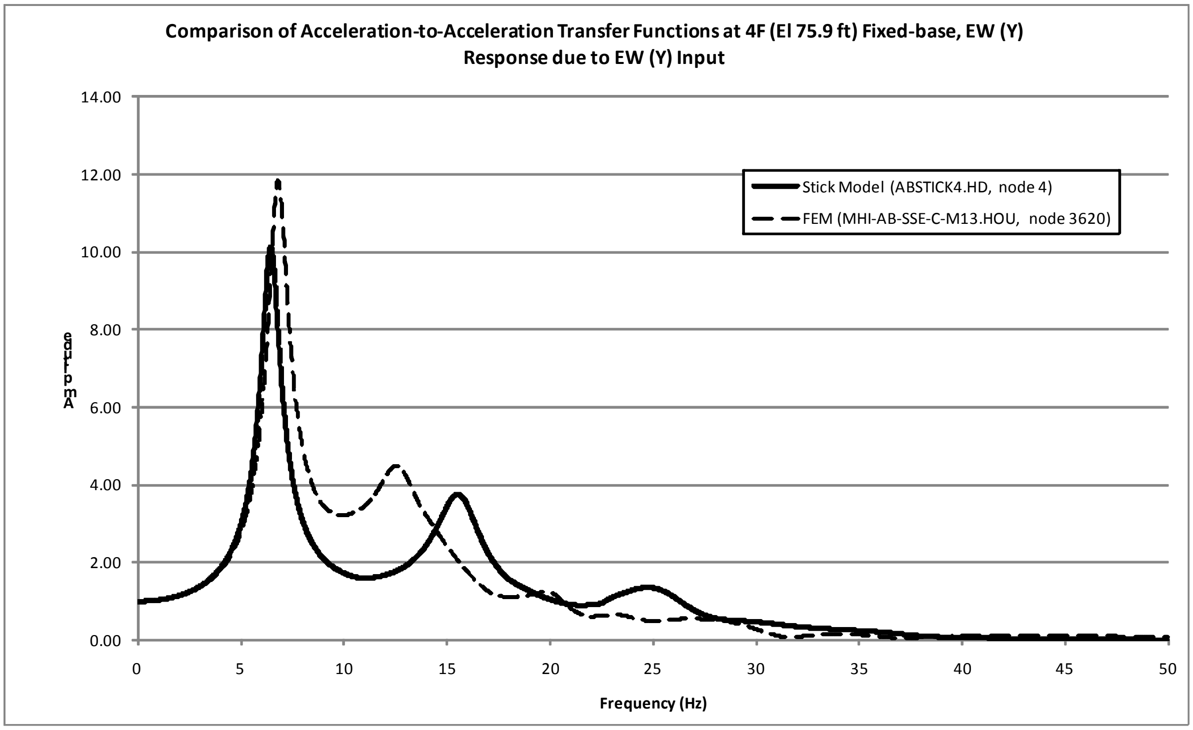


Figure 3.4-2 Transfer Function Amplitudes for EW Response of Roof

4.0 SOIL STRUCTURE INTERACTION ANALYSIS

4.1 Methodology

Seismic SSI analyses of the A/B are performed using ACS SASSI for the eight generic site profiles described in MHI TR MUAP-10006 (Reference 8.2). Consistent with the site-independent SSI analyses performed for the R/B Complex and PS/B, the SSI analyses of the A/B considers a surface supported foundation and neglects the effect of the approximate 40 ft embedment. The soil profile is modeled to a depth of at least twice the dimension of the A/B foundation. Two sets of SSI analyses are performed using the Detailed FE Model and lumped-mass stick model. Reduced (cracked concrete) stiffness and SSE structural damping properties are assigned to both of the models as described in Section 2.3.1.

The SSI analyses of the Dynamic FE Model provide the critical maximum seismic response parameters, such as the maximum absolute accelerations, maximum displacements relative to structural base, and in-structure response spectra at the top of basemat, The in-structure response spectra (ISRS) computed at the top of the basemat for all eight generic site profiles provide the basis for development of the seismic acceleration input spectra for the response spectrum analysis of the detailed FE model described in Section 5.2. The results of maximum relative displacements are used for evaluation of the gap between the A/B and the adjacent buildings.

The SSI analyses of the A/B lumped mass stick model provide time histories of the stick member forces and moments and time history of the basemat accelerations. These results serve as input for evaluation of the seismic stability of the A/B.

Table 4.2-1 provides a summary of the dynamic models, site profiles, number of frequencies of analyses and cut-off frequency of analyses used for the different SSI analyses presented in this report. The horizontal size of the FE mesh of the basemat is also presented in the table together with the maximum frequency of the waves that can be transmitted through the soil-foundation interface based on the criterion that the basemat FE size is not more than 20% of the minimum wave length. The cut-off frequencies of the SSI analyses of softer soil conditions are lower than the required 50 Hz since these analyses provide results that envelope the design parameters that are governed by lower frequency responses, such as maximum displacements. The responses at higher frequencies are enveloped by results obtained from analyses of stiffer soil conditions which are run for frequencies up to 50Hz. Therefore, the SSI analyses of the eight generic profiles provide results that envelope the response of the A/B up to frequency of 50 Hz.

4.2 Results of Lumped Mass Stick Model SSI Analyses

Section 4.2 was not completely updated in revision 1 and may be out of date. This data will be updated in future revisions of this report when the applicable analysis is complete.

The SASSI model used in the first-stage SSI analyses of the lumped-mass stick model is shown in Figure 2.5.3. In the model, the stick model is rigidly connected to the basemat modeled as flat shell finite elements. The base of the stick model where it connects to the FE model of the basemat is linked to the center of each perimeter wall that is structurally connected to the basemat. To obtain seismic response values at the extreme edge locations of each floor or roof, mass less rigid arms modeled using mass less rigid beam elements are used to connect the nodes at the extreme edge locations to the CM node of each floor or roof. The CSDRS-compatible acceleration time-histories are used as the seismic input. SASSI analyses are performed with each individual direction of seismic input separately and the 3-D seismic responses of the lumped-mass stick model are calculated at the foundation base. Co-directional maximum seismic responses generated from the seismic inputs in the three directions are combined on time-history square-root-of-the-sum-of-squares (SRSS) basis.

The results obtained from the first-stage SASSI analyses of the lumped-mass stick model are summarized below.

4.2.1 Maximum Forces and Moments

The combined maximum seismic response axial, NS and EW shear forces and the maximum torsional and bending moments about the NS and EW axes obtained from SRSS combinations of the maximum seismic responses generated from the three individual directions (horizontal NS and EW and vertical) of seismic input for all eight generic site profile cases considered are shown in Figures 4.2-1 through 4.2-6.

From the results shown in these figures, one can identify that the maximum global structure seismic response forces and moments obtained from the SSI analyses of the eight generic site profiles are dominated (controlled) by the site profiles 900-100, 900-200, and 2032-100.

4.2.2 Maximum Relative Displacements

Figure 4.2-7 presents the combined maximum NS, EW and vertical displacements of the A/B structure relative to the bottom center of the basemat. The maximum values presented in Figure 4.2-7are envelopes of the maximum response values obtained at the center of mass and four extreme edge nodes on each floor elevation. The vertical displacements at the basemat are not zero because they include the envelope of the edge nodes of the basemat. At the edge nodes, there are vertical displacements caused by rocking of the basemat due to horizontal input. As indicated in this Figure 4.2-7, the maximum relative displacements of the structure are controlled by the softest generic site profiles, namely, 270-200 and 270-500.

4.2.3 Maximum Absolute Accelerations

The combined maximum seismic response absolute accelerations in the horizontal NS and EW and vertical directions obtained are summarized in Figure 4.2-8. Similar to the maximum relative displacements presented above, the maximum absolute acceleration values presented in Figure 4.2-8 are envelopes of the maximum response values obtained at the center of mass and four extreme edge nodes on each floor elevations. The results shown in Figure 4.2-8 indicate that the maximum acceleration response values are controlled by the generic site profiles 900-100 and 2032-100.

4.2.4 Basement In-Structure Response Spectra

In-structure (acceleration) response spectra (ISRS) are computed for the response motions at the basemat center and four extreme-edge nodes. Figure 4.2-9, Figure 4.2-10, and Figure 4.2-11, respectively, shows the 7%-damped horizontal NS and EW, and vertical ISRS computed that are envelopes of responses at the five nodes on the basemat for all eight generic site profile cases.

4.2.5 Seismic Horizontal Sliding Shear and Vertical Force Demands

The combined maximum seismic horizontal NS and EW sliding shear forces and the vertical forces computed at the bottom of the basemat for all eight generic site profile cases are summarized in Table 4.2-2. In this table, the time instances at which the maximum forces occur are also identified.

Table 4.2-1 Matrix of SSI Analysis of A/B

Structural Model	Basemat FE Mesh Size (ft)	Generic Soil Profile	Max. Wave Passage Freq. (Hz)	Total Number of Freq. of Analyses	Cut-off Freq. of Analyses (Hz)
Lumped Mass Stick Model	10.0	270-200	26.0	67	50
	10.0	270-500	24.8	69	50
	10.0	560-100	31.8	66	50
	10.0	560-200	31.0	69	50
	10.0	560-500	34.0	68	50
	10.0	900-100	68.1	71	50
	10.0	900-200	64.7	71	50
	10.0	2032-100	142.5	70	50
Dynamic FE Model	8.92 (max)	270-500	27.9	45	50
	13.17 (max)	900-100	51.7	45	50
	13.17 (max)	900-200	49.2	45	50
	13.17 (max)	2032-100	108.2	45	50

Table 4.2-2 A/B Stick Model – Maximum Seismic Sliding Shear Demand (kips)

Generic Site Profile	Direction		
	X (NS) Sliding Shear (time of maximum in seconds)	Y (EW) Sliding Shear (time of maximum in seconds)	Z (Vertical) Base Force (time of maximum in seconds)
270-200	93,200 (8.390)	96,700 (7.865)	82,200 (9.035)
270-500	87,300 (8.395)	93,500 (7.865)	76,000 (7.850)
560-100	102,600 (9.280)	100,100 (8.365)	94,300 (8.730)
560-200	88,900 (10.095)	98,800 (8.545)	94,300 (7.845)
560-500	90,800 (10.085)	90,200 (7.850)	91,900 (7.840)
900-100	122,300 (10.765)	112,200 (7.830)	114,500 (8.700)
900-200	118,800 (10.770)	115,700 (7.840)	92,800 (9.015)
2032-100	126,300 (7.340)	100,600 (8.310)	91,600 (8.640)
Envelope Value Site Profile Case	126,300 2032-100	115,700 900-200	114,500 900-100

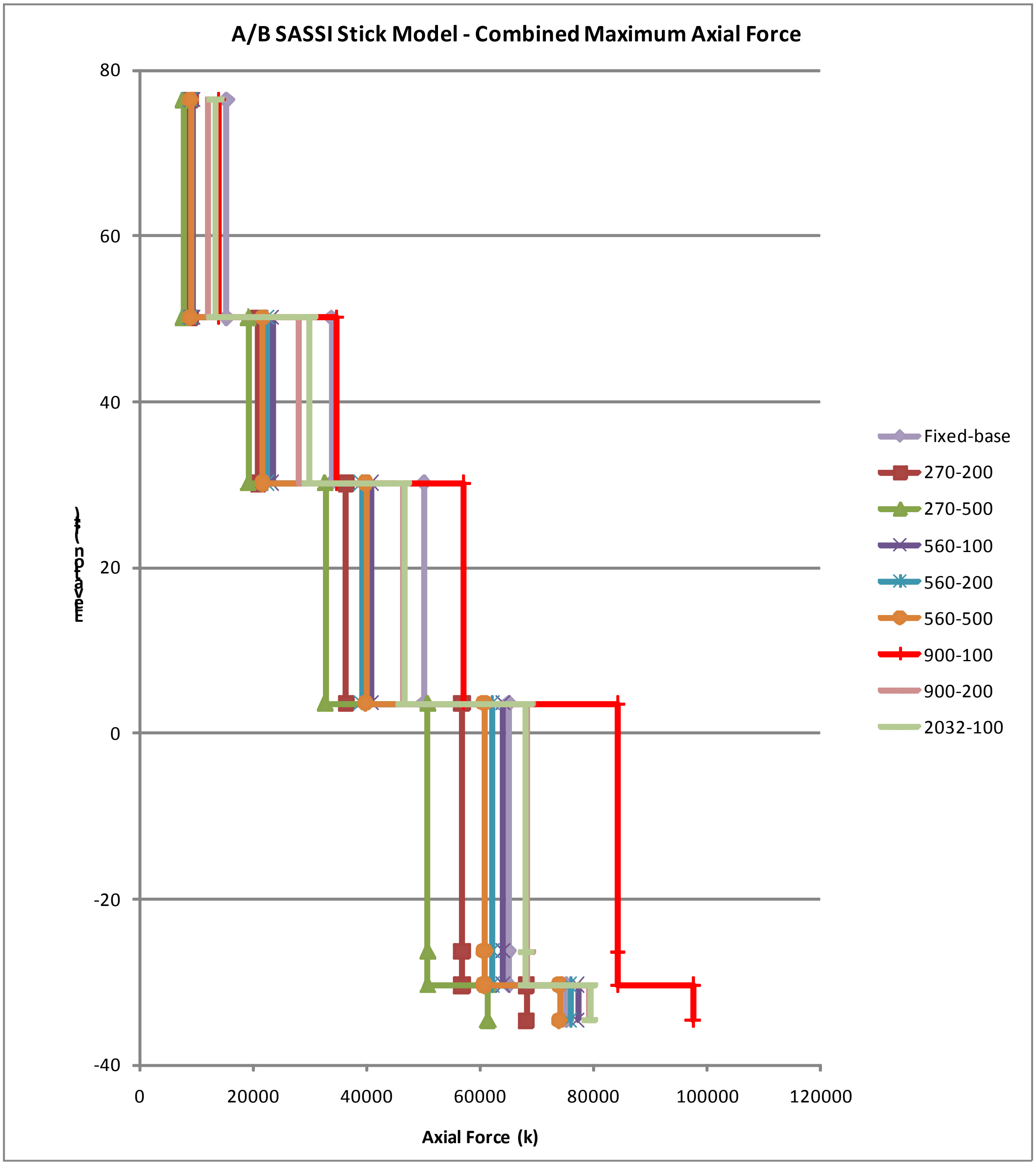


Figure 4.2-1 Stick Model – Max. Axial Force

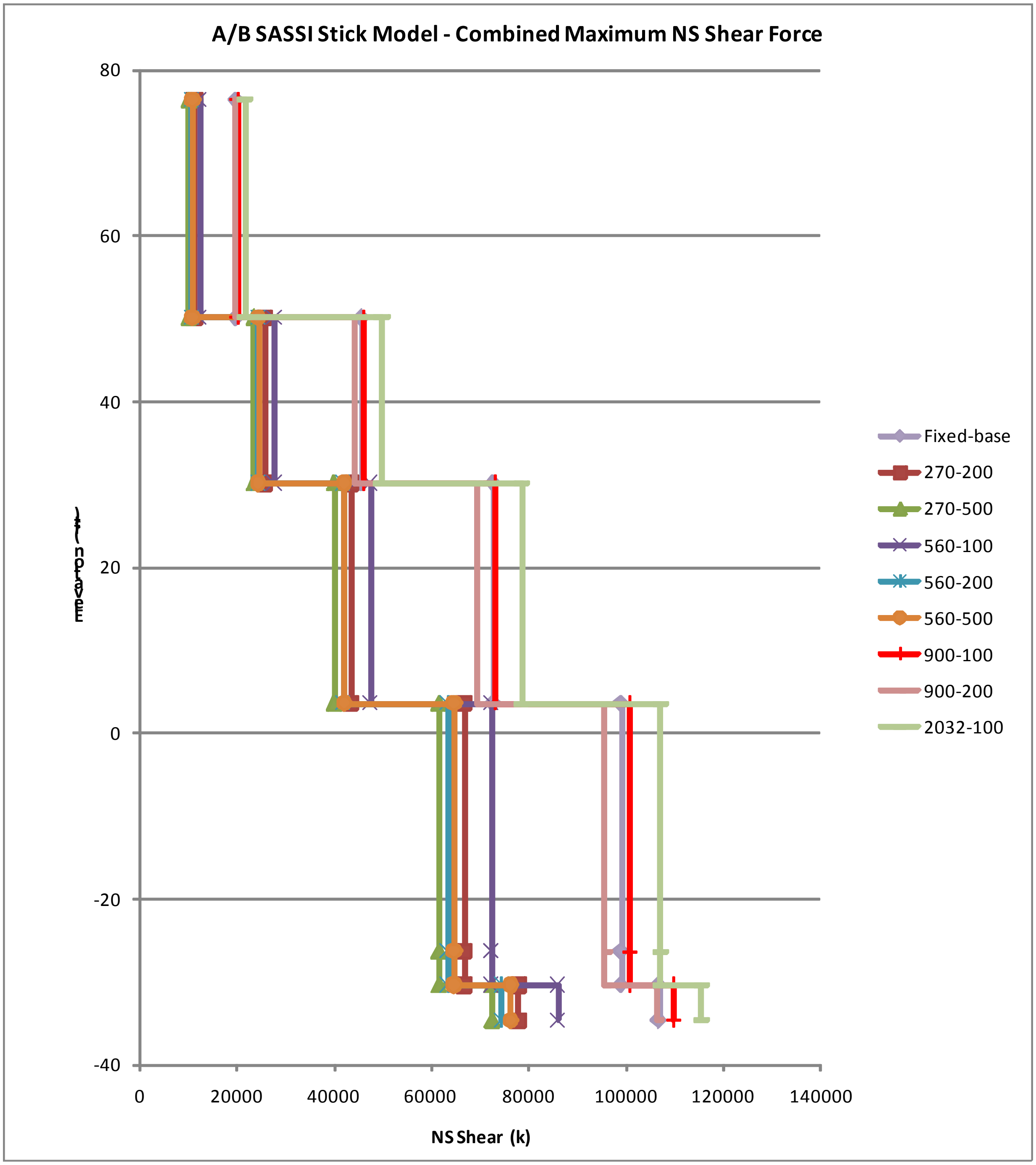


Figure 4.2-2 Stick Model – Max. N-S Shear Force

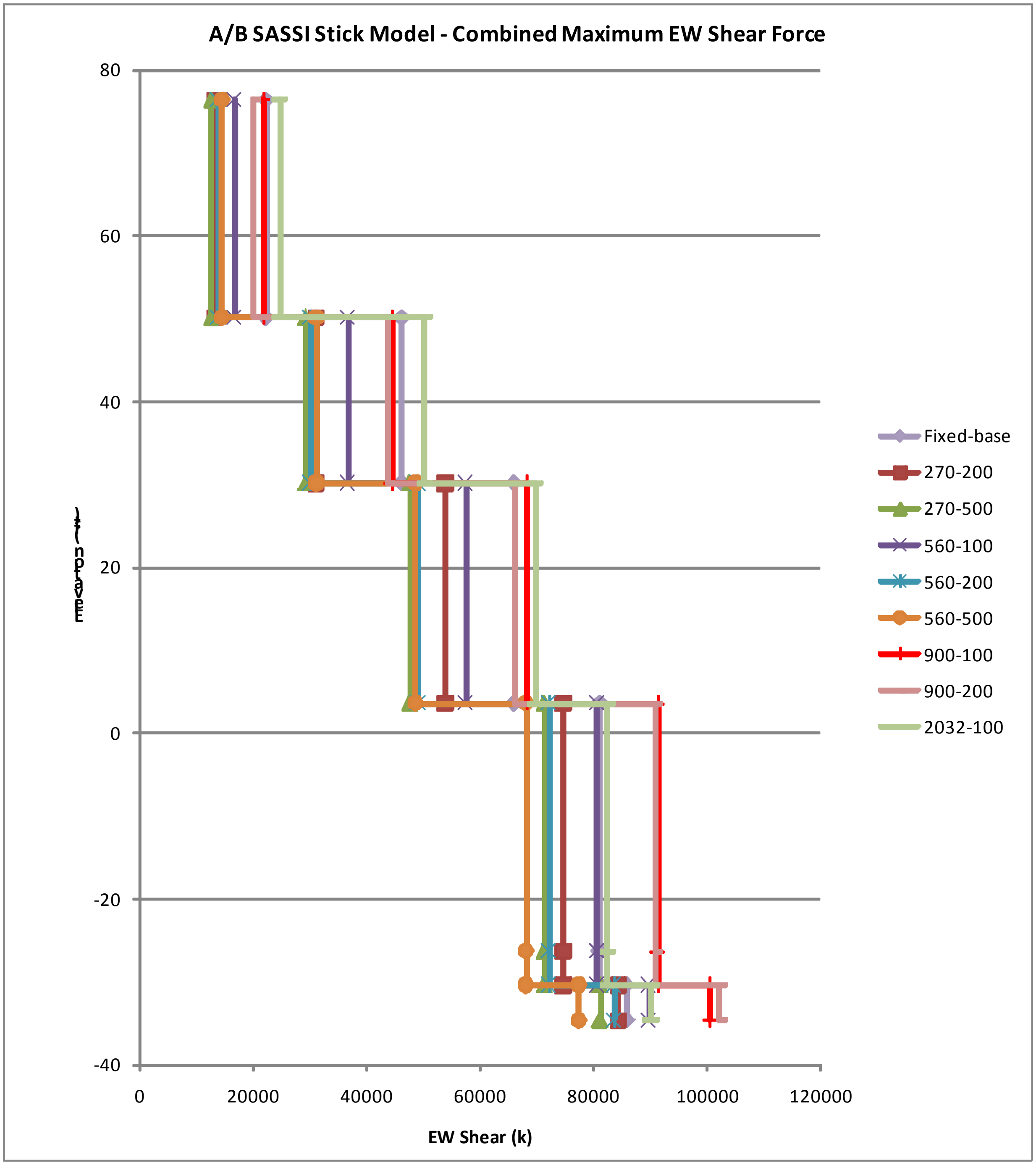


Figure 4.2-3 Stick Model – Max. E-W Shear Force

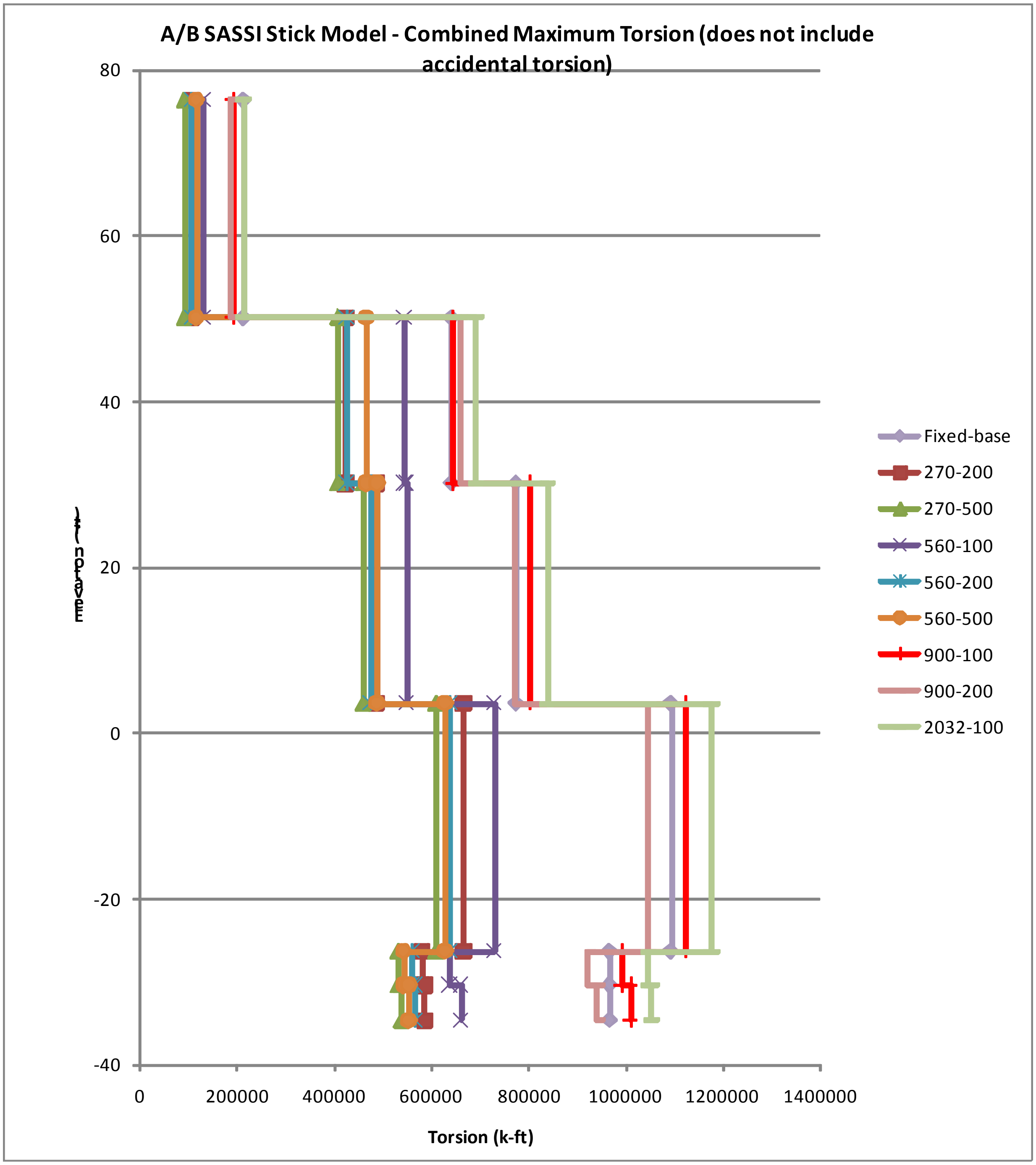


Figure 4.2-4 Stick Model – Max. Torsion Moment

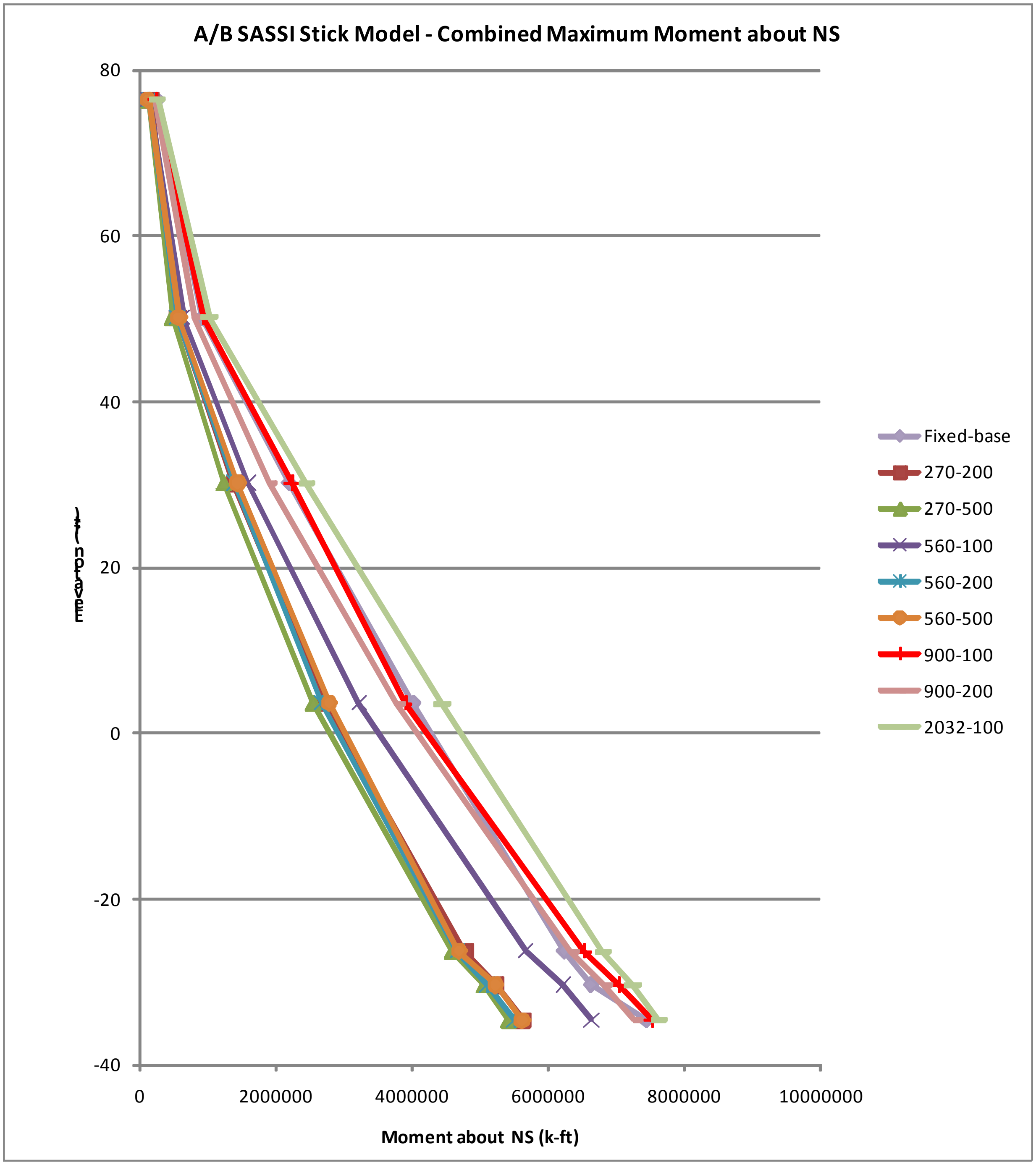


Figure 4.2-5 Stick Model – Max. Moment about N-S axis

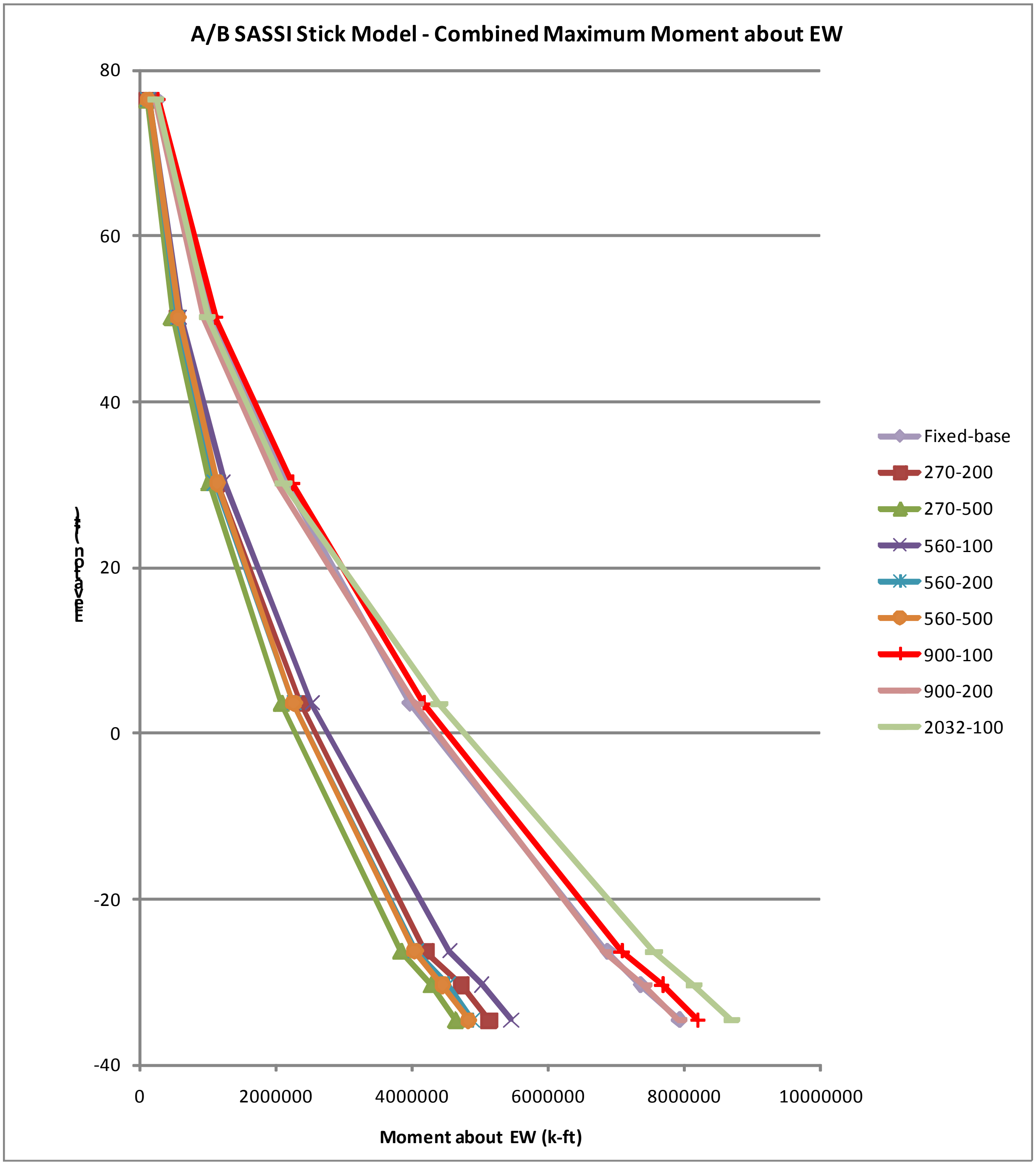


Figure 4.2-6 Stick Model – Max. Moment about E-W axis

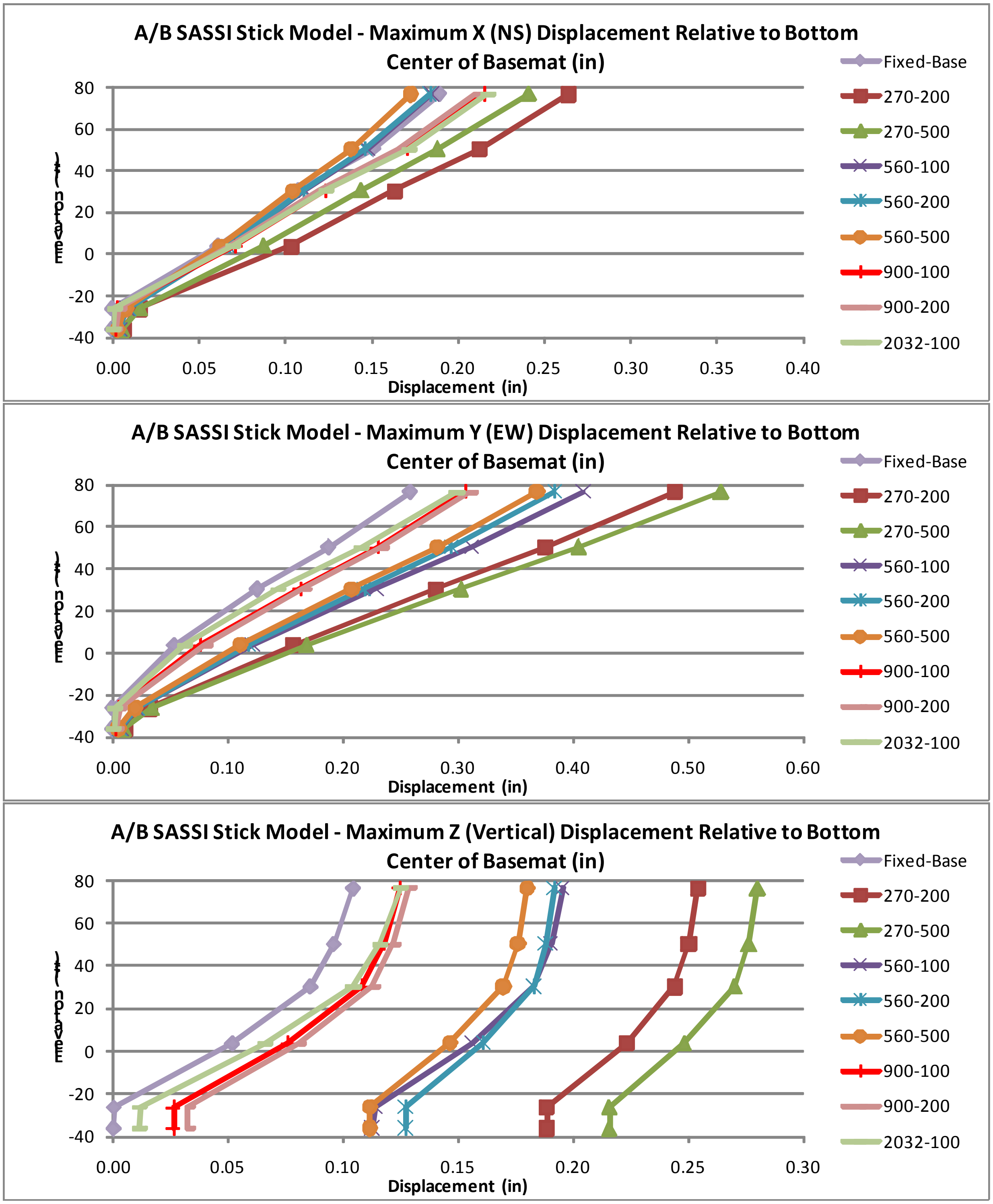


Figure 4.2-7 Stick Model – Max. Displacements Relative to Basemat Bottom Center

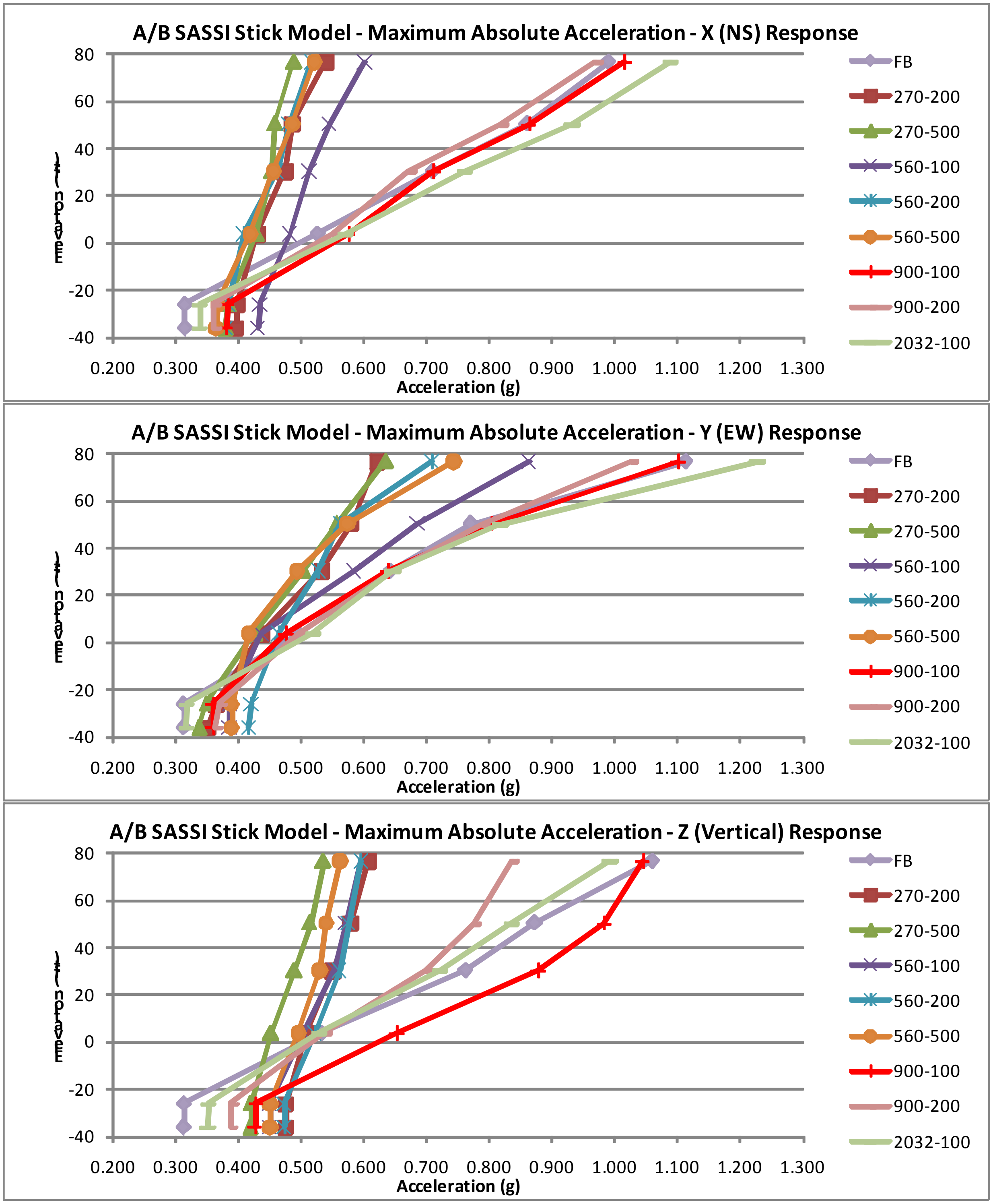


Figure 4.2-8 Stick Model – Max. Absolute Accelerations

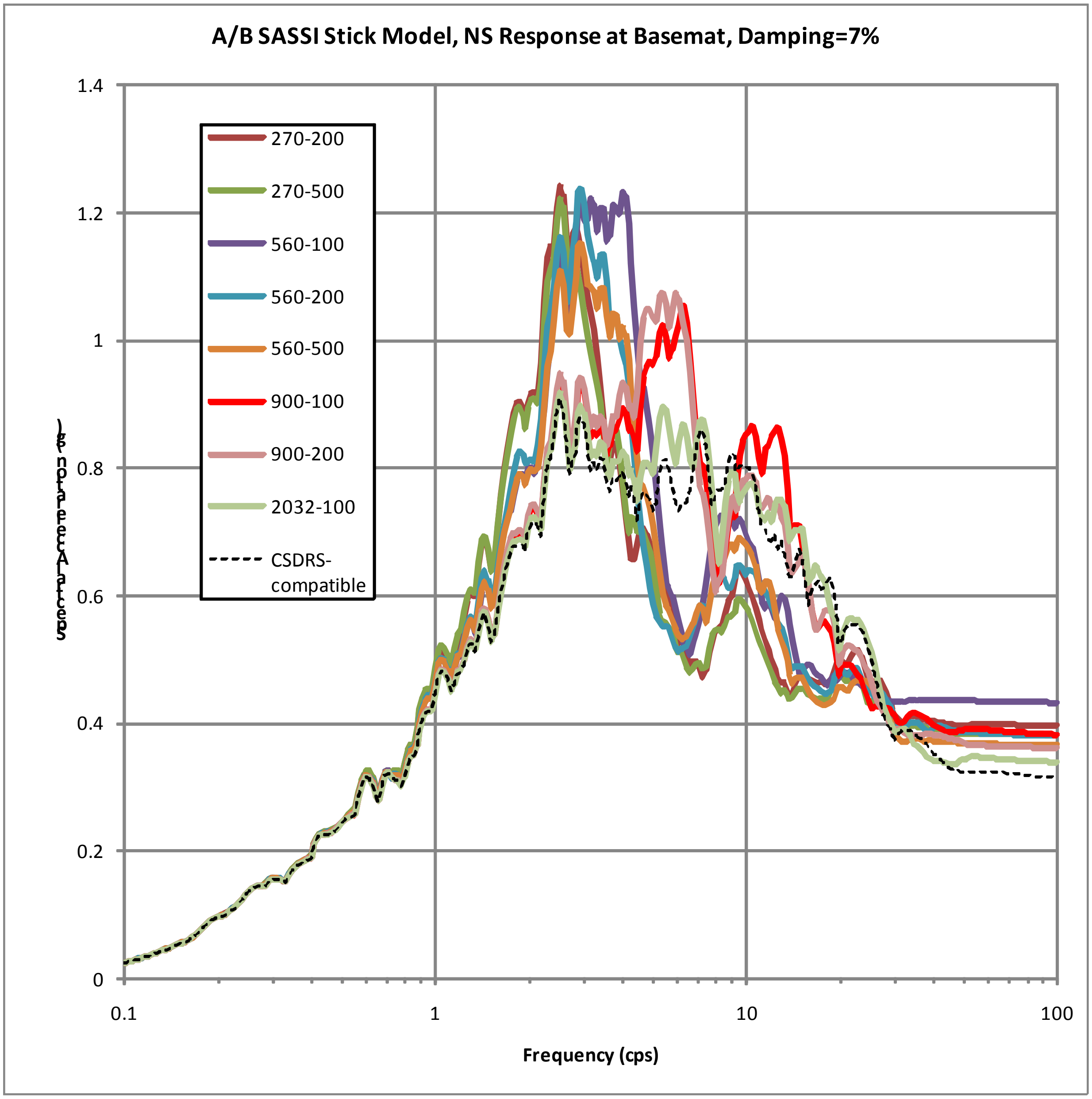


Figure 4.2-9 7%-Damped ISRS at Basemat – N-S Direction

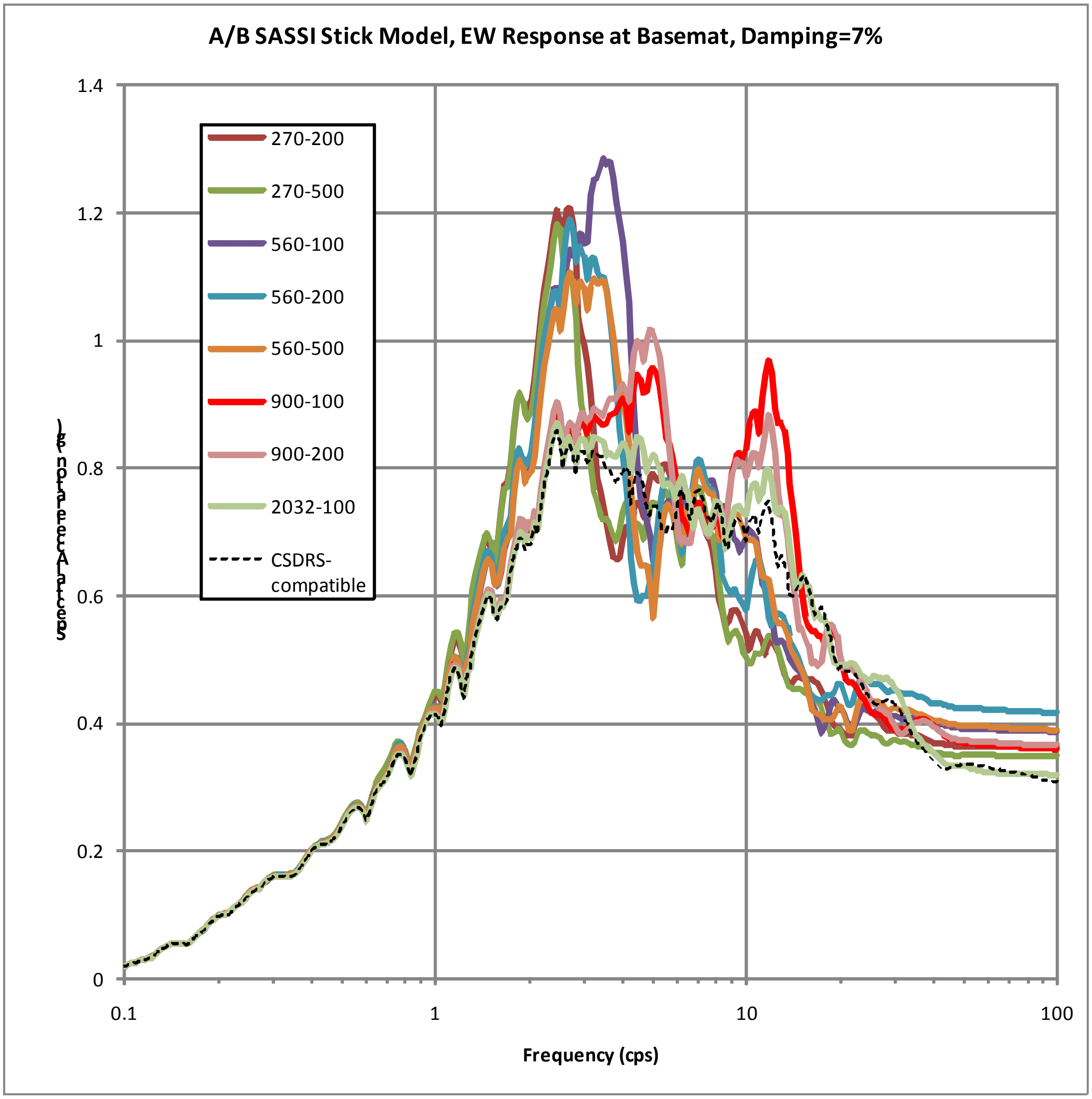


Figure 4.2-10 7%-Damped ISRS at Basemat – E-W Direction

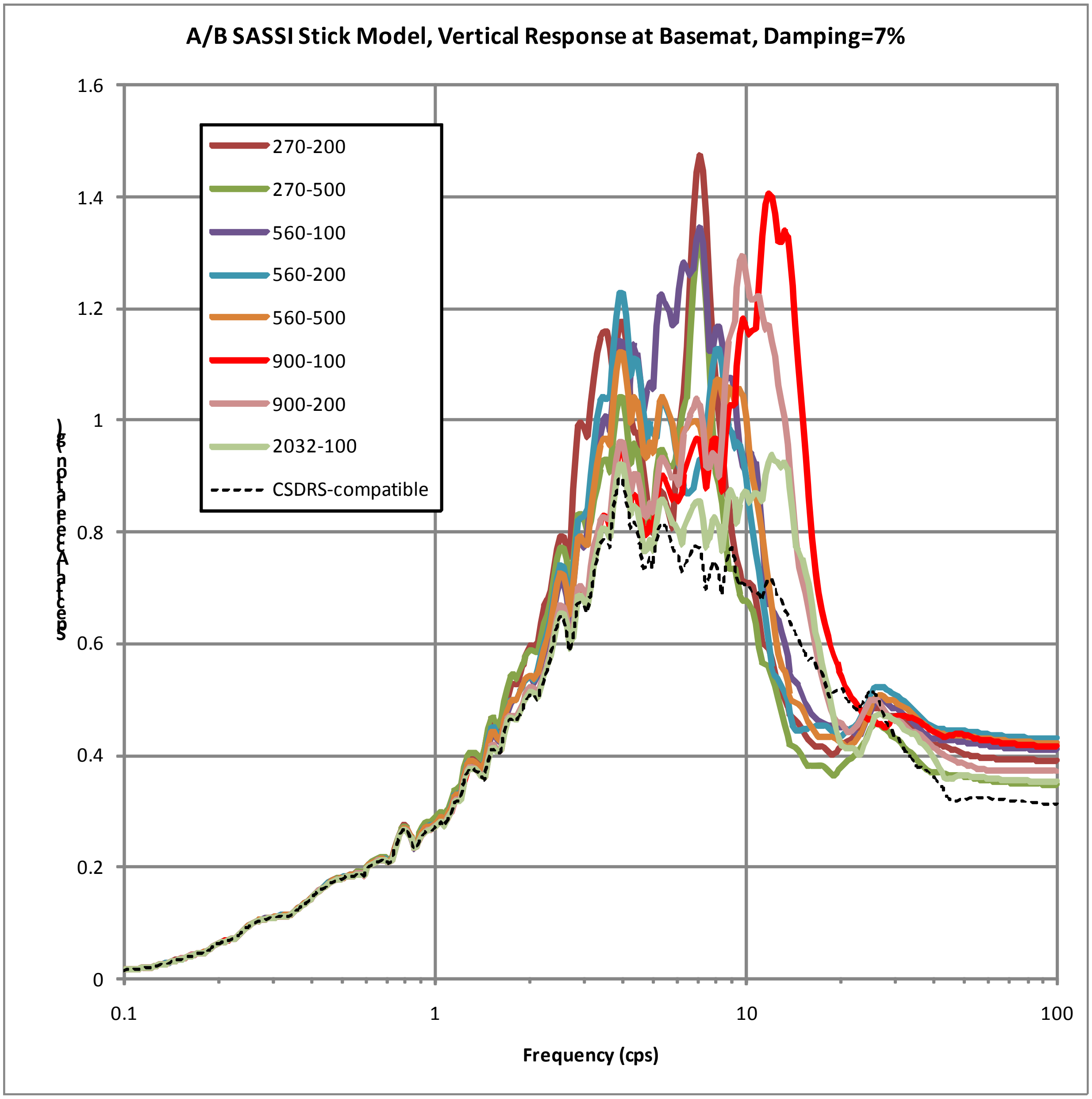


Figure 4.2-11 7%-Damped ISRS at Basemat – Vertical Direction

4.3 Results of Dynamic FE Model SSI Analyses

Section 4.3 was not updated in revision 1 and may be out of date. This data will be updated in future revisions of this report when the applicable analysis is complete.

From the critical maximum seismic response results obtained from the first-stage SSI analyses performed using the lumped-mass beam-stick model presented above, up to four critical generic site profiles that control the critical maximum seismic response values are identified. These four site profiles identified are (a) 270-500, (b) 900-100, (c) 900-200, and (d) 2032-100.

The site profiles 900-100, 900-200, and 2032-100 are selected based upon the maximum forces and moments (Figures 4.2-1 through 4.2-6) and maximum absolute accelerations (Figure 4.2-8). The maximum displacements relative to the basemat bottom center (Figure 4.2-7) show that site profile 270-200 controls in the North-South direction and site profile 270-500 controls in the East-West and vertical directions. Since the displacements of the A/B will ultimately be used to evaluate the seismic gap between the A/B and the R/B and the A/B and the Ac/B, the 270-500 site profile was chosen for the second-stage analysis because the R/B is on the East side of the A/B and the Ac/B is on the West side of the A/B. The results of the SSI analyses also indicate that the displacements are largest in the East-West direction.

The second-stage SSI analyses are performed using SASSI with the dynamic FE models (9-ft mesh and 13-ft mesh for the basemat) of the A/B structure supported on the surface of the four critical generic site profiles identified above as the SSI models. These SASSI models are shown in Figure 2.3-1 through Figure 2.3-3 for the 9-ft-mesh model, which is used for the critical 270-500 site profile case, and in Figure 2.3-4 through Figure 2.3-12 for the 13-ft-mesh model, which is used for the critical 900-100, 900-200, and 2032-100 site profile cases.

Using these refined SASSI FE models, SSI responses in terms of the maximum seismic response displacements relative to the structural base (at the bottom center of the basemat) and relative to the free-field ground surface where the seismic input motion is prescribed, are extracted. Figure 4.3-1 presents the results of maximum seismic displacements relative to the free-field ground motion and Figure 4.3-2 presents the results of the maximum seismic response displacements relative to the structural base. These results are obtained from the SASSI analyses performed using the dynamic FE models of A/B for the four critical generic site profiles identified.

Comparison of the relative displacements with respect to the structural base shown in Figure 4.3-2 obtained from the SASSI analyses of the FE models for the four controlling site profile cases with the corresponding relative displacements shown in Figure 4.2-7 obtained from the SASSI analyses of the lumped-mass stick model shows that the maximum displacement response values obtained from the FE models are quite consistent with, but slightly higher than, the corresponding results obtained from the lumped-mass stick model. As with the stick model results, the vertical displacements at the base have a large rocking component (vertical displacement at the basemat edge due to horizontal input). Figure 4.3-1 shows that the maximum displacement relative to the free-field ground surface is 0.92 inches in the East-West direction.

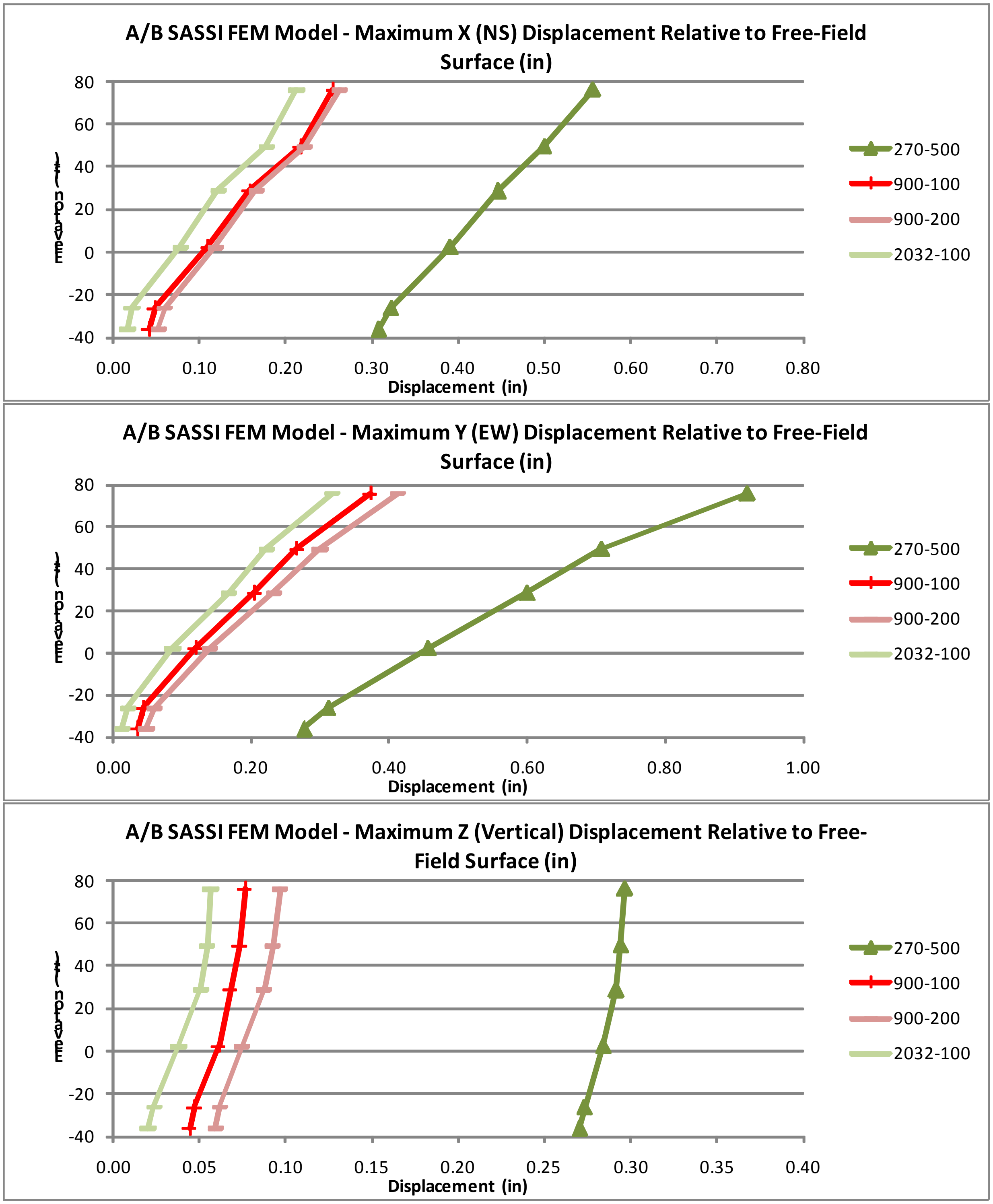


Figure 4.3-1 Dynamic FE Model – Max. Displacements Relative to Free-Field Surface

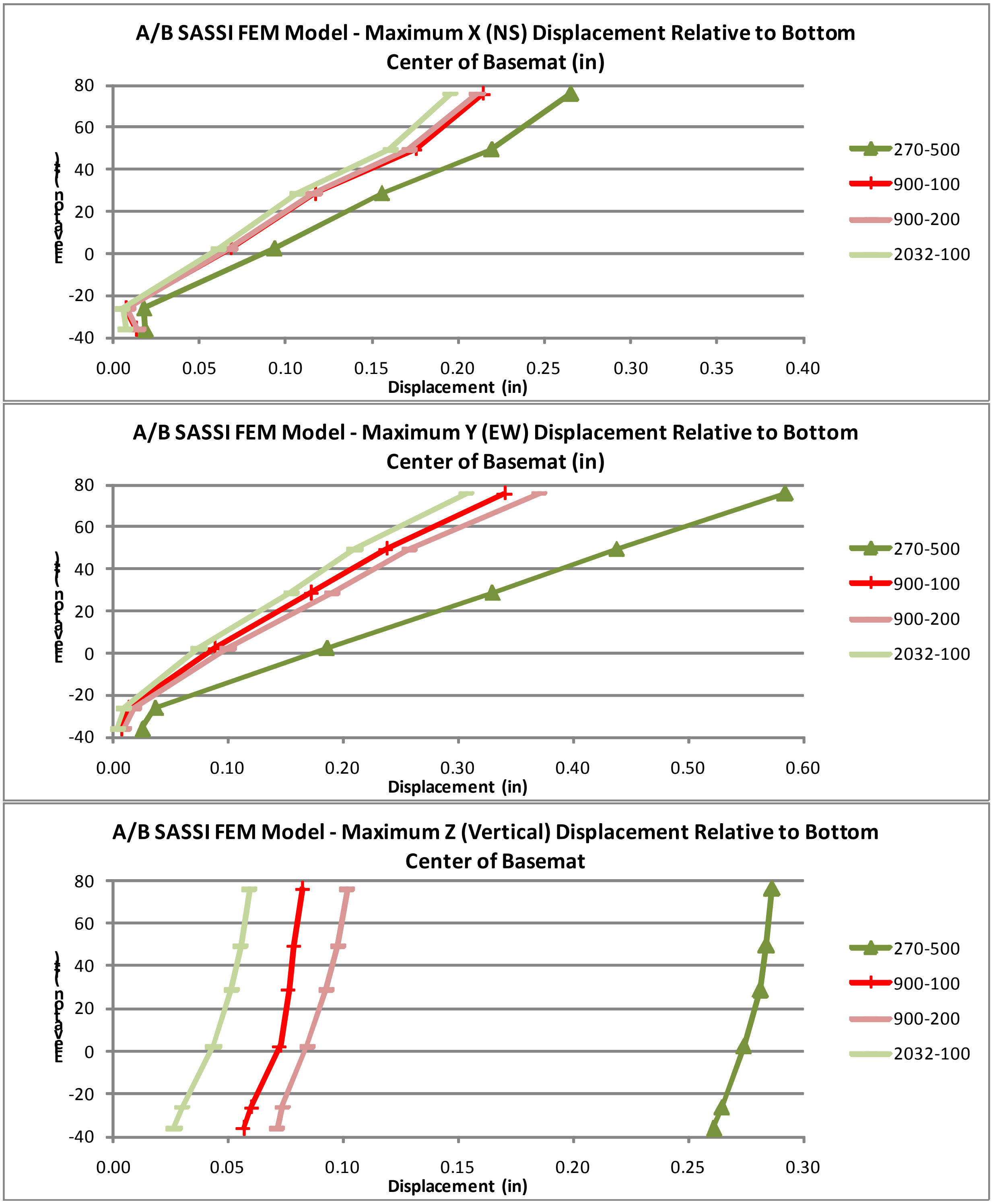


Figure 4.3-2 Dynamic FE Model – Max. Displacements Relative to Basemat Bottom Center

5.0 STATIC AND DYNAMIC ANALYSIS OF DETAILED FE STRUCTURAL MODEL OF A/B

Section 5 was not updated in revision 1 and may be out of date. This data will be updated in future revisions of this report when the applicable analysis is complete.

A structural integrity evaluation is performed of the A/B based on structural demands calculated from ANSYS static and response spectrum analyses of the detailed FE model. The structural demands due to applicable static and seismic design loads and design load combinations on main representative shear walls and slabs of the building are checked against design allowables in order to evaluate the overall structural integrity of the A/B.

5.1 Loads and Load Combinations for Seismic Structural Integrity Evaluation of A/B

The structural integrity evaluation considers loads and load combinations associated with an SSE seismic event as listed below. All applicable A/B loads are applied to the detailed FE model.

5.1.1 Dead Loads (D)

Dead loads include the weight of structures (D1) such as slabs, roofs, decking, framing (beams, columns and walls), and the weight of permanently attached major equipment (D2) such as tanks, machinery, cranes, elevators, etc. MHI report N0-EHB0032, "Component Weight List Auxiliary Building" (Reference 8.6) provides the foot prints and weights of major pieces of equipment in the A/B.

5.1.2 Live Loads (L)

Live loads are the loads imposed by the use and occupancy of the building/structure.

Building Floor (L_1) MHI report N0-EHB0031, "Load Distribution Auxiliary Building" (Reference 8.5) provides live loads P2 (loads of piping, duct, tray, conduit, platform equipped to component and support) and live loads P3 (floor load during periodic inspection where positions and weights vary) and some P1 (component load) dead loads not listed in the component weight list (Reference 8.6) for the structural evaluation and design of buildings.

Roof Snow/Live (L_2) The roof is conservatively designed for uniform snow live load of 75 50 psf per Table 1 of the SDC (Reference 8.9). Consistent with DC/COL-ISG-7 (Reference 8.15), this load represents the 100-year snow pack maximum snow weight including the contributing portion of either extreme frozen winter precipitation event or extreme liquid winter precipitation event. The roof design live load is 40 psf. Roof live load is not added to roof snow load when evaluating the design load combinations.

5.1.3 Static Ground Water (F)

A static ground water pressure (F) acting on the structures during normal operation is included in the analysis. Table 2.0-1 of Chapter 2 of the DCD (Reference 8.3) identifies that the maximum groundwater level is 1 ft. below grade. However, to simplify the calculations, the groundwater level is conservatively taken as at grade level for determination of hydrostatic pressure.

5.1.4 Static Lateral Earth Load (H)

A static at rest earth pressure (H) acting on the structures during normal operation is included in the analysis. Soil properties are taken from MHI TR MUAP-10006 (Reference 8.2).

Table 2.0-1 of Chapter 2 of the DCD (Reference 8.3) identifies that the maximum groundwater level is 1 ft. below grade. However, to simplify the calculations, the groundwater level is conservatively taken as being at grade level for determination of hydrostatic pressure and static earth pressure.

5.1.5 SSE Loads (E_{ss})

SSE Loads are defined as the loads generated by the SSE specified for the plant. This includes dynamic soil pressures (E_{ds}) on exterior walls that contact with surrounding backfill and member forces (E_{ss}) derived from the RSA analyses. It also includes accidental torsion (E_t).

Dynamic soil pressures are taken from Table 4-12 of MHI TR MUAP-10006 (Reference 8.2). Member forces are derived from RSA analyses performed on the A/B detailed model per Section 5.2. The ISRS input for the RSA analyses considers SSI amplified effects. The ISRS is provided by the seismic soil-structure interaction (SSI) analysis per Section 4.0.

SRP Acceptance Criteria 1.A.iii of Section 3.7.2 of NUREG 800 (Reference 8.10) requires consideration of torsional, rocking and translation responses of site structures and their foundations. SRP Acceptance Criteria 11 of Section 3.7.2 also requires consideration of an accidental torsion that is based on an additional eccentricity of ±5% of the maximum building lateral dimension. Therefore, in addition to the earthquake forces derived from RSA analyses, the effects of accidental torsion is also considered. A torsion moment equal to the larger of the torsions resulting from the product of the base shears times 5% of the building dimension that is perpendicular to the direction of the base shear force is applied to the analytical model.

5.1.6 Other Applicable Load Cases

Liquid (F) Hydrostatic loads are based on the water volumes within the various pits during normal operations. No significant pits are located in the A/B.

Normal Pipe Reactions (R_o) The normal operating environment inside and outside the R/B is specified in Table 6 of the SDC (Reference 8.9). Since the change in temperature (ΔT) of the reference temperature of 70^o F and normal operating temperatures are not excessive, this load is not considered in the structural integrity evaluation.

Normal Operating Thermal (T_o) Pipe and equipment reactions during normal operation or shutdown conditions are based on the most critical transient or steady state condition. These loads produce primarily local effects and will not be addressed in structural integrity evaluation.

Accident Loads (P_a, T_a, R_a, Y_r, Y_j, Y_m) These loads will not be addressed in the structural integrity evaluation

5.1.7 Load Combinations

Concrete structures are designed in accordance with ACI-349 (Reference 8.16) and the provisions of RG 1.142 (Reference 8.14) where applicable, with the load combinations and load factors provided in Table 5.1-1. Load combinations #1, 4, and 8 are applicable for the structural integrity evaluation at this stage. Future revision of this report will includes evaluation for all the applicable load cases and combinations. Table 5.1-2 provides the load combinations considered in the ANSYS analysis and evaluation.

Table 5.1-1 Load Combinations and Load Factors for Concrete Structures

LOAD COMBINATIONS AND FACTORS(1),(2)												
ACI 349 Load Combination:		1	2	3	4	5	6	7	8	9	10	11
Load Type												
Dead	*D*	1.4	1.4	1.4	1.0	1.0	1.0	1.0	1.0	1.05	1.05	1.05
Liquid	*F*	1.4	1.4	1.4	1.0	1.0	1.0	1.0	1.0	1.05	1.05	1.05
Live	*L*	1.7	1.7	1.7	1.0	1.0	1.0	1.0	1.0	1.3	1.3	1.3
Earth	*H*	1.7	1.7	1.7	1.0	1.0	1.0	1.0	1.0	1.3	1.3	1.3
Design pressure	P_d											
Normal Pipe reactions	R_o	1.7	1.7	1.7	1.0	1.0				1.3	1.3	1.3
Normal thermal	T_o				1.0	1.0				1.2	1.2	1.2
Wind	*W*			1.7								1.3
OBE	E_{ob}		NA					NA			NA	
SSE	E_{ss}				1.0				1.0			
Tornado	W_t					1.0						
Accident pressure	P_a						1.4	1.15	1.0			
Accident thermal	T_a						1.0	1.0	1.0			
Accident thermal pipe reactions	R_a						1.0	1.0	1.0			
Pipe rupture reactions	Y_r							1.0	1.0			
Jet impingement	Y_j							1.0	1.0			
Pipe Impact	Y_m							1.0	1.0			

Notes:
1. Design per ACI-349 Strength Design Method for all load combinations.
2. Where any load reduces the effects of other loads, the corresponding coefficient for that load is taken as 0.9 if it can be

Table 5.1-2 A/B Structural Integrity Concrete Load Combinations

		L01	L02	L06	L07	L08	L12	L11	L13	L15	L16	L17	L18	L19	L20	L21	L22
		Dead Loads		Liquid Loads	Live Loads		Earth Pressure	SSE Dynamic Soil Pressure		SSE RSA Seismic Load						SSE Accidental Torsion	
		Self Weight	Equipment Loads	Ground Water	Floor LL Roof Snow/LL	Piping LL	Static	E-W	N-S	N-S +	N-S -	E-W +	E-W -	Vertical +	Vertical -	+	-
Load Combo		D_1	D_2	F_3	L_1	L_2	H_1	E_{ds1}	E_{ds1}	E_{ss1}	E_{ss2}	E_{ss3}	E_{ss4}	E_{ss5}	E_{ss6}	E_{t+}	E_{t-}
ABConcLC01	1.4D+1.4F+1.7L+1.7H	1.4	1.4	1.4	1.7		1.7										
ABConcLC02	1.4D+1.4F+1.7L+1.7H	1.4	1.4	1.4		1.7	1.7										
ABConcLC03	1.4D+1.4F+1.7L+1.7H	1.4	1.4	1.4			1.7										
ABConcLC04	1.4D+1.4F+1.7L+1.7H	1.4	1.4	1.4	1.7	1.7	1.7										
ABConcLC05	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4	1.0	1.0		0.4		0.4		1.0	
ABConcLC06	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4	1.0	1.0		0.4			0.4	1.0	
ABConcLC07	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0		1.0	1.0			0.4	0.4		1.0	
ABConcLC08	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0		1.0	1.0			0.4		0.4	1.0	
ABConcLC09	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4			1.0	0.4		0.4		1.0	
ABConcLC10	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4			1.0	0.4			0.4	1.0	
ABConcLC11	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0				1.0		0.4	0.4		1.0	
ABConcLC12	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0				1.0		0.4		0.4	1.0	
ABConcLC13	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	1.0	0.4	0.4		1.0		0.4		1.0	
ABConcLC14	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	1.0	0.4	0.4		1.0			0.4	1.0	
ABConcLC15	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	1.0			0.4	1.0		0.4		1.0	
ABConcLC16	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	1.0			0.4	1.0			0.4	1.0	
ABConcLC17	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0		0.4	0.4			1.0	0.4		1.0	
ABConcLC18	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0		0.4	0.4			1.0		0.4	1.0	
ABConcLC19	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0				0.4		1.0	0.4		1.0	
ABConcLC20	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0				0.4		1.0		0.4	1.0	
ABConcLC21	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4	0.4	0.4		0.4		1.0		1.0	
ABConcLC22	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0		0.4	0.4			0.4	1.0		1.0	
ABConcLC23	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4			0.4	0.4		1.0		1.0	
ABConcLC24	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0				0.4		0.4	1.0		1.0	
ABConcLC25	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4	0.4	0.4		0.4			1.0	1.0	
ABConcLC26	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0		0.4	0.4			0.4		1.0	1.0	
ABConcLC27	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4			0.4	0.4			1.0	1.0	
ABConcLC28	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0				0.4		0.4		1.0	1.0	
ABConcLC29	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4	1.0	1.0		0.4		0.4			1.0
ABConcLC30	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4	1.0	1.0		0.4			0.4		1.0
ABConcLC31	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0		1.0	1.0			0.4	0.4			1.0

Table 5.1-2 A/B Structural Integrity Concrete Load Combinations (continued)

		L01	L02	L06	L07	L08	L12	L11	L13	L15	L16	L17	L18	L19	L20	L21	L22
		Dead Loads		**Liquid Loads**	**Live Loads**		**Earth Pressure**	**SSE Dynamic Soil Pressure**		**SSE RSA Seismic Load**						**SSE Accidental Torsion**	
		Self Weight	Equipment Loads	Ground Water	Floor LL Roof Snow/LL	Piping LL	Static	E-W	N-S	N-S +	N-S -	E-W +	E-W -	Vertical +	Vertical -	+	-
Load Combo		D_1	D_2	F_3	L_1	L_2	H_1	E_{ds1}	E_{ds1}	E_{ss1}	E_{ss2}	E_{ss3}	E_{ss4}	E_{ss5}	E_{ss6}	E_{t+}	E_{t-}
ABConcLC32	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0		1.0	1.0			0.4		0.4		1.0
ABConcLC33	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4			1.0	0.4		0.4			1.0
ABConcLC34	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4			1.0	0.4			0.4		1.0
ABConcLC35	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0				1.0		0.4	0.4			1.0
ABConcLC36	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0				1.0		0.4		0.4		1.0
ABConcLC37	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	1.0	0.4	0.4		1.0		0.4			1.0
ABConcLC38	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	1.0	0.4	0.4		1.0			0.4		1.0
ABConcLC39	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	1.0			0.4	1.0		0.4			1.0
ABConcLC40	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	1.0			0.4	1.0			0.4		1.0
ABConcLC41	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0		0.4	0.4			1.0	0.4			1.0
ABConcLC42	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0		0.4	0.4			1.0		0.4		1.0
ABConcLC43	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0				0.4		1.0	0.4			1.0
ABConcLC44	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0				0.4		1.0		0.4		1.0
ABConcLC45	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4	0.4	0.4		0.4		1.0			1.0
ABConcLC46	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0		0.4	0.4			0.4	1.0			1.0
ABConcLC47	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4			0.4	0.4		1.0			1.0
ABConcLC48	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0				0.4		0.4	1.0			1.0
ABConcLC49	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4	0.4	0.4		0.4			1.0		1.0
ABConcLC50	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0		0.4	0.4			0.4		1.0		1.0
ABConcLC51	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0	0.4			0.4	0.4			1.0		1.0
ABConcLC52	1.0D+1.0F+1.0L+1.0H+1.0E_{ss}	1.0	1.0	1.0	1.0	1.0	1.0				0.4		0.4		1.0		1.0
ABConcLC53	1.05D+1.05F+1.3L+1.3H	1.05	1.05	1.05	1.3	1.3	1.3										

Notes:

D_1	Self Weight of Elements	E_{ds1}	Dynamic Soil Pressure Safe Shutdown Earthquake (E-W direction Earthquake)	E_{ss4}	Safe Shutdown Earthquake (E-W) Response (Negative)
D_2	Dead Load (Includes permanently attached equipment, tanks, machinery, cranes, elevators, etc.)	E_{ds2}	Dynamic Soil Pressure Safe Shutdown Earthquake (N-S direction Earthquake)	E_{ss5}	Safe Shutdown Earthquake (Vertical) Response (Positive)
F_3	Fluid Load (Lateral and upward pressure due to high water table)	E_{ss1}	Safe Shutdown Earthquake (N-S) Response (Positive)	E_{ss6}	Safe Shutdown Earthquake (Vertical) Response (Negative)
L_1	Live Load (Building floor loads)	E_{ss2}	Safe Shutdown Earthquake (N-S) Response (Negative)	E_{t+}	Accidental Torsion due to SSE (positive rotation)
L_2	Live Load (Piping Live Load)	E_{ss3}	Safe Shutdown Earthquake (E-W) Response (Positive)	E_{t-}	Accidental Torsion due to SSE (negative rotation)
H_1	Earth Pressure (Static)				

5.2 Response Spectrum Analysis Methodology

The seismic inertial forces for the structural integrity evaluation on structural members are obtained from the response spectrum analysis (RSA) of the detailed FE model with the effective seismic mass and fixed boundary conditions set at the base of A/B structure. An RSA is performed for each orthogonal seismic input motion (N-S, E-W, and Vertical). No uplift is assumed at the base and will be further confirmed/verified during the basic design of the basemat.

5.2.1 Determination of the Input Response Spectrum

In order to conservatively account for the amplification of the structural response due to soil structure interaction effects, the acceleration response spectra used as input for the response spectrum analysis (RSA) are obtained as follows:

Step 1: Perform SSI analyses on the A/B stick model described in Section 2.5 above for all eight generic soil profiles.

Step 2: The Acceleration Responses Spectrum (ARS) for 7% damping ratio at edges and center of top surface of basemat due to three directions of the earthquake are combined using the rule of Square Root of the Sum of Square (SRSS) to account for cross directional earthquake effects.

Step 3: Envelope SRSS combined ARS for 7% damping ratio at edges and center of basemat top surface for the eight soil profiles.

Step 4: Generate ISRS for 7% damping ratio at basemat top surface by applying 15% peak broadening and 5% increase in magnitude of the response to the enveloped ARS obtained from Step 3. The resulting ISRS in Figure 5.2-1 through Figure 5.2-3 are used as input base excitation spectra for the RSA analysis.

Step 5: ISRS generated at Step 4 are verified by SSI analyses performed on A/B dynamic FE model for the four critical soil cases as described in Section 4.3.

5.2.2 Boundary Conditions for Response Spectrum Analysis

The response spectrum analysis (RSA) considers a fixed-base condition by setting a fixed boundary condition at each node at the bottom of the basemat. The response spectrum analysis considers SSI affects through use of the broadened input acceleration response spectrum generated by SSI analysis described above. The fixed nodes do not allow the basemat to deflect and therefore not all applicable seismic forces are captured for the basemat slab. The response of the basemat is not considered in this structural integrity evaluation and will be evaluated during the finial basic design.

5.2.3 Combined Modal Responses: Lindley-Yow Method

The response from each individual mode is combined by Lindley-Yow method per Regulatory Position 1.3.2 of RG 1.92, Rev. 2 (Reference 8.19). The ISRS obtained from the SSI analyses are modified per Reg. 1.92 for modal combinations using Lindley-Yow method. Figure 5.2-4 through Figure 5.2-6 present the Lindley-Yow modified response spectra that are used as input for the response spectrum analysis (RSA). The periodic response portion of the Lindley-Yow method is implemented by using ANSYS “Grouping Method” and the rigid response portion is implemented by using "Static ZPA Method" per Regulatory Position 1.4.2 of RG 1.92, Rev. 2 (Reference 8.19). The directional effect from each direction is combined by 100-40-40 method.

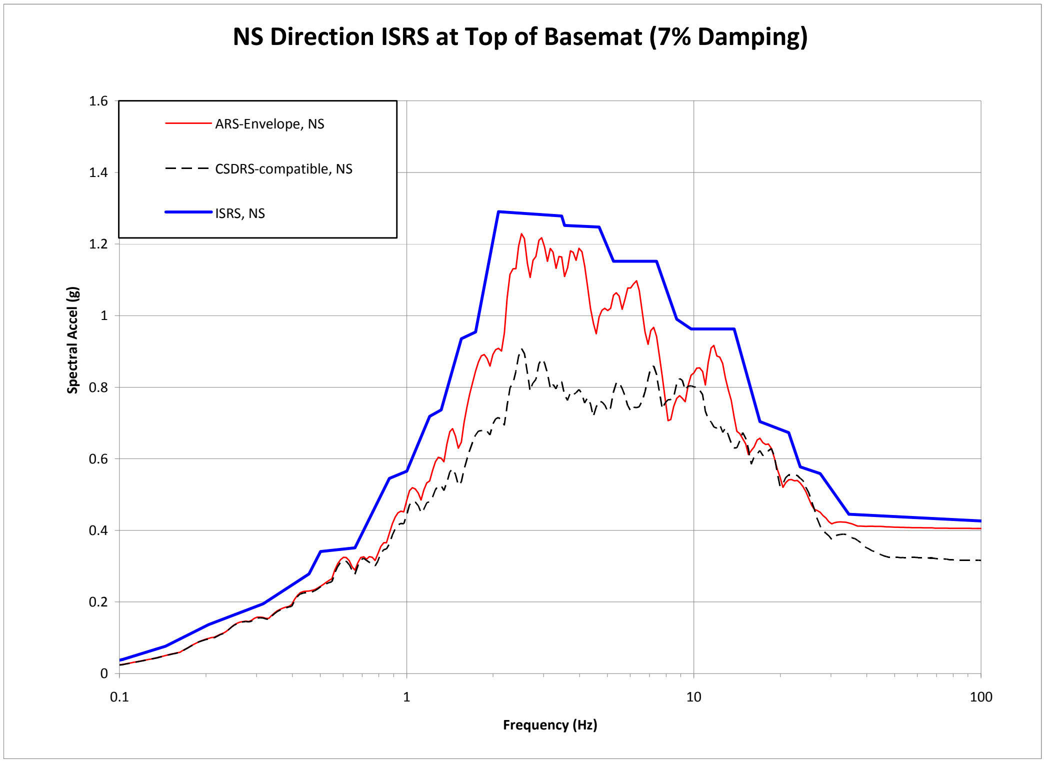


Figure 5.2-1 N-S 7% Damped ISRS at Top of A/B Basemat

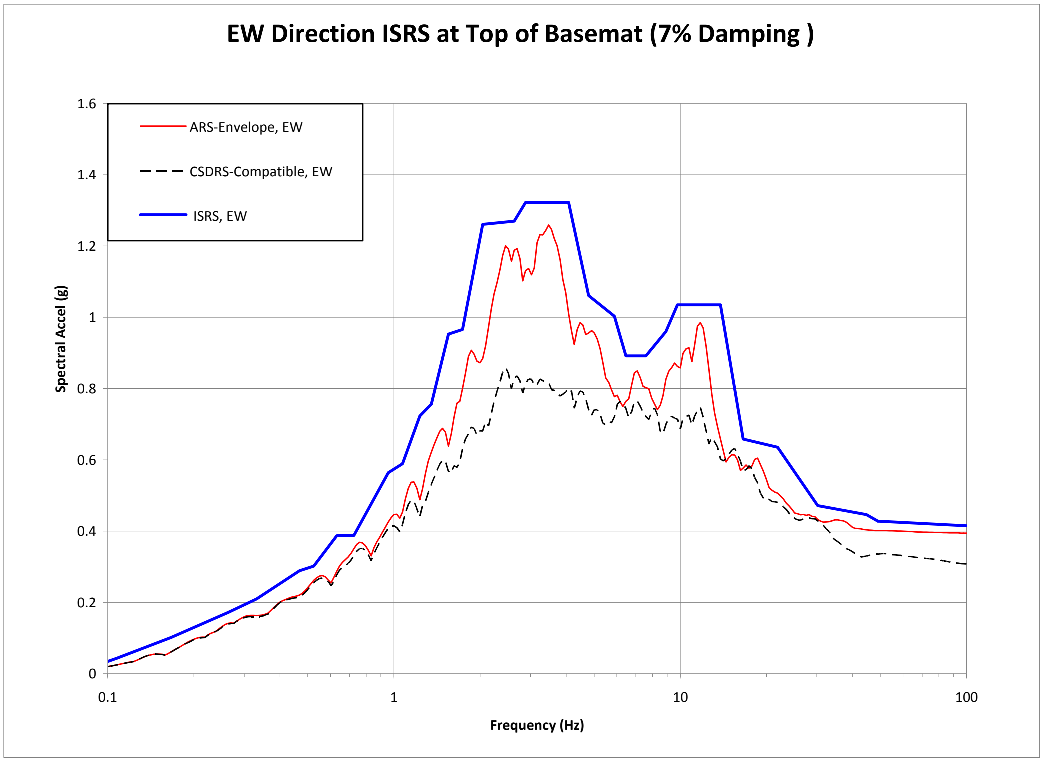


Figure 5.2-2 E-W 7% Damped ISRS at Top of A/B Basemat

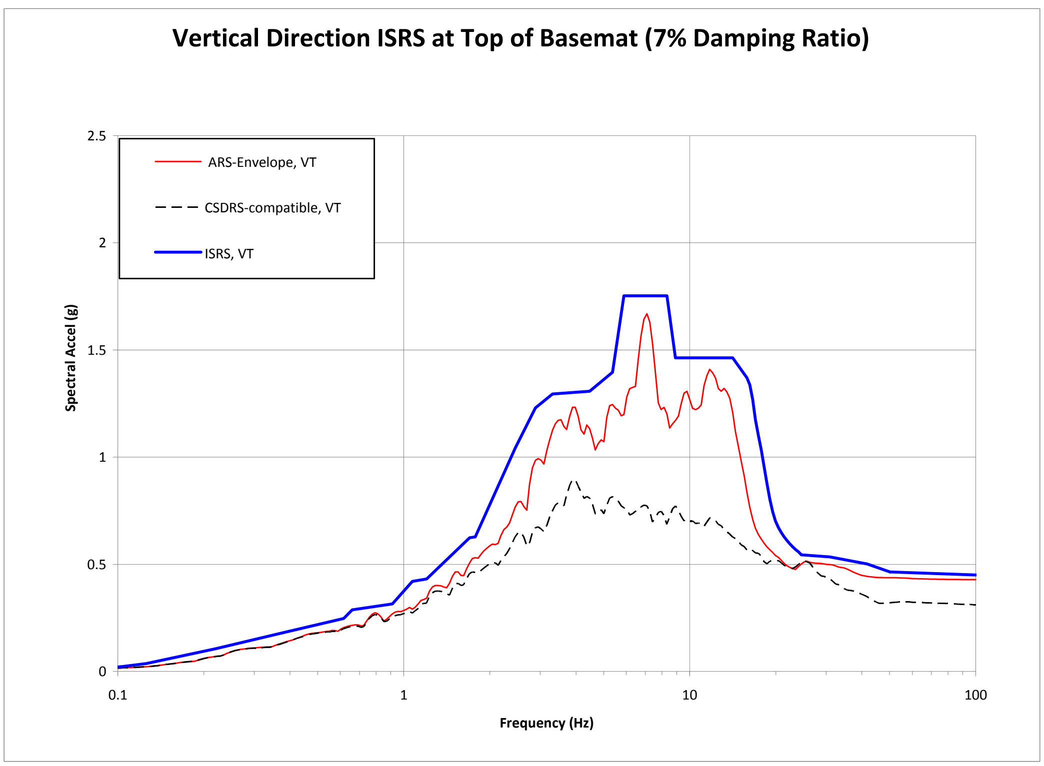


Figure 5.2-3 Vertical 7% Damped ISRS at Top of A/B Basemat

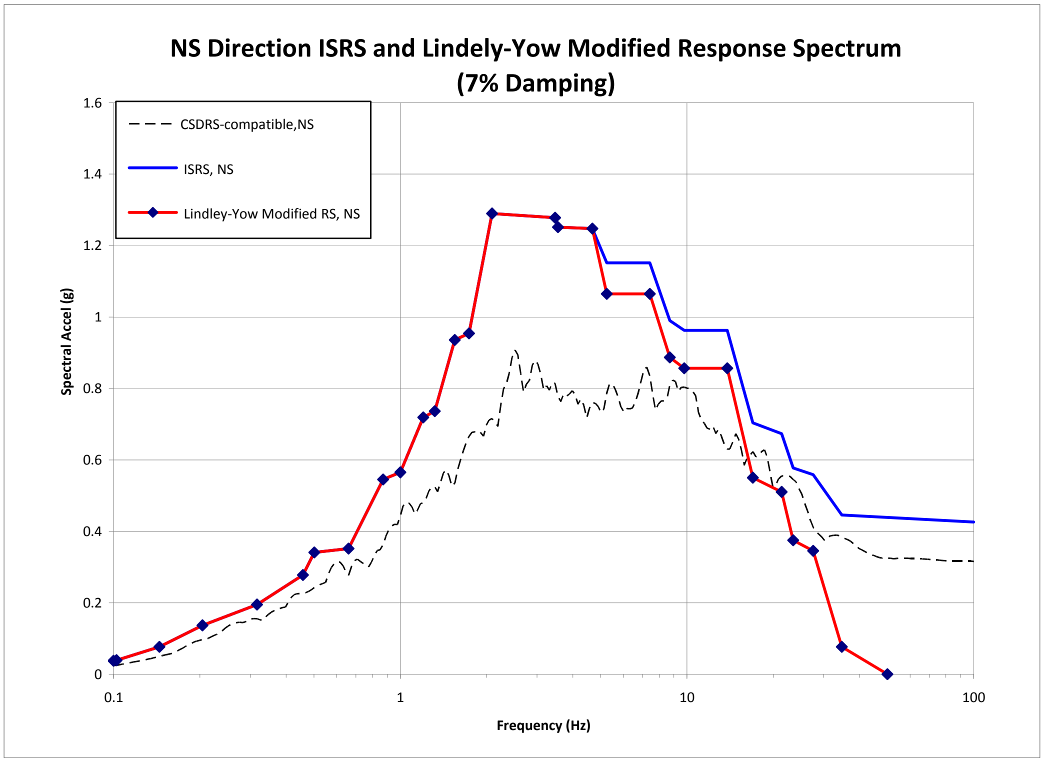


Figure 5.2-4 Modified N-S Response Spectrum for Response Spectrum Analysis

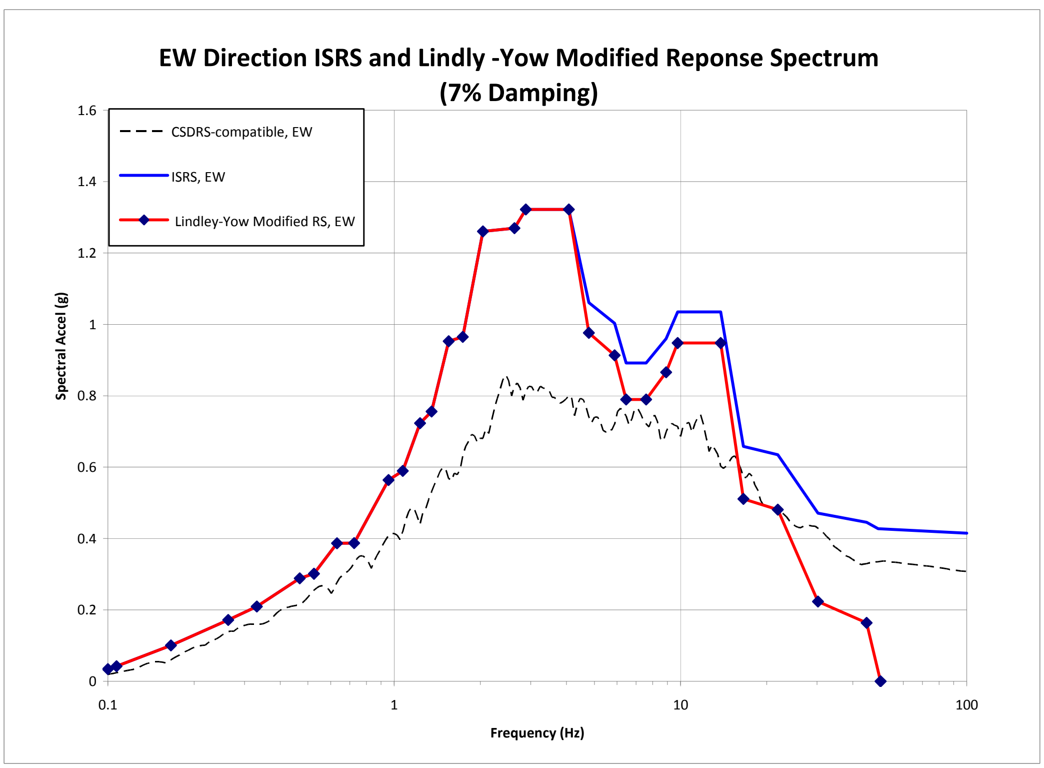


Figure 5.2-5 Modified E-W Response Spectrum for Response Spectrum Analysis

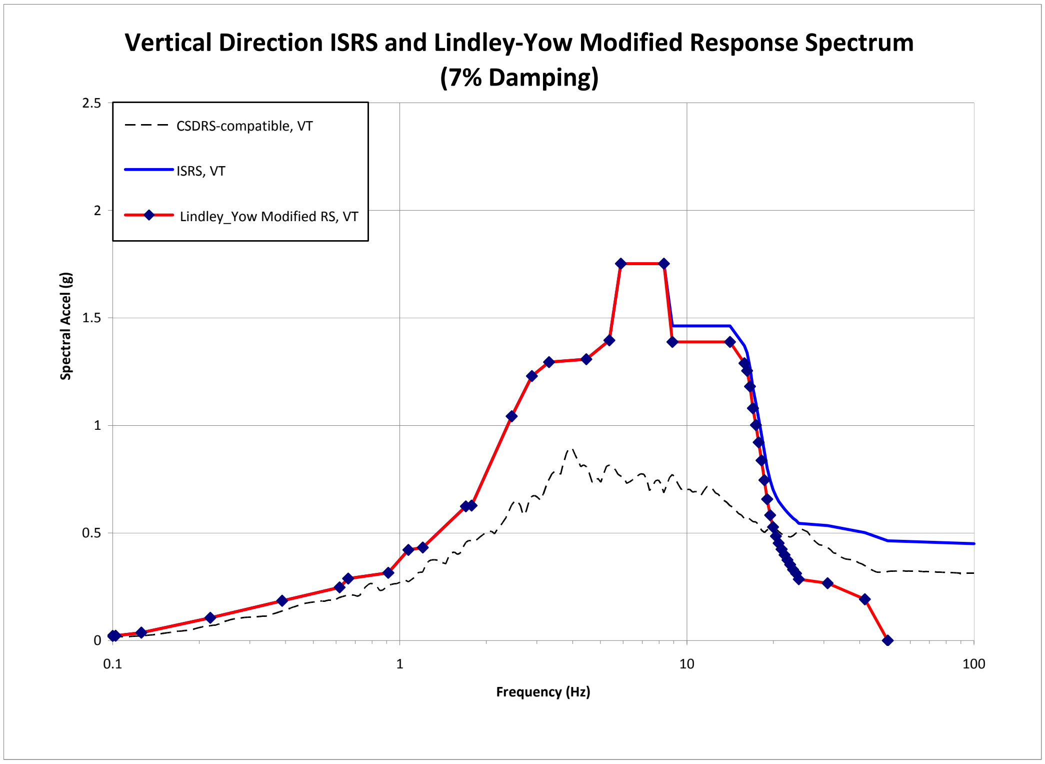


Figure 5.2-6 Modified Vertical Response Spectrum for Response Spectrum Analysis

Note: The Lindley-Yow's curve representing the periodic response only and needs to be combined with rigid response from static ZPA.

5.3 Static and Response Spectrum Analysis Results

5.3.1 Base Shear

The seismic forces for the structural evaluation from each response spectrum analysis (RSA) are applied to the structural members/element as seismic loads with separate load cases for each orthogonal direction: N-S, E-W, and vertical. The shear forces from each RSA analysis are summed to obtain the base shear in each direction. A summary of the total seismic base shear force at the top of the base mat from the effect of each directional response are combined by SRSS and the results are given in Table 5.3-1. The total base shears as shown on Table 5.3-1 are used to determine accidental torsions as applied forces in the static analysis.

5.3.2 Structural Demands of Main Structural Components

The structural demands for the main shear walls and slabs are determined for the load combinations described in Section 5.1.6, which include SSE loads. The applicable element forces are extracted from the static analysis for each shear wall and slab. The demand–to-capacity ratios are determined in Section 5.5.

Table 5.3-1 Seismic Base Shear Seismic Force

Base Shear at Top of Basemat By SRSS the RSA results in N-S, E-W and Vertical Direction (Kips)			
Base Shear direction	Seismic Input Direction	Base shear from source excitation alone	Base shear with combined directional effect by SRSS
N-S	N-S	128,467	134,908
	E-W	25,996	
	Vertical	25,204	
E-W	N-S	21,483	117,006
	E-W	110,347	
	Vertical	26,201	

5.4 Structural Integrity Evaluation Methodology

The structural integrity of several representative shear walls and slabs is evaluated to determine if their capacity is adequate to resist the seismic demands calculated from the static and response spectrum analyses of the detailed FE model. The structural demands and the geometry of these walls and slabs are input to concrete design programs“wall.exe” and “slab.exe”. The program calculates the member demand-to-capacity ratios and provides the necessary rebar arrangement to resist the demands. To facilitate the process, several additional ANSYS macros were developed. The programs are described in Appendix A.

5.5 Structural Integrity Evaluation Results

Exterior walls at column line 1A and the first floor slabs are selected as representative members for structural Integrity evaluation. Exterior wall segment along column line AA between column lines 1A and 6A, from elevation -26'-4" to elevation 2'-3" has been selected to check the shear capacity of the exterior walls. Table 5.5-1 provides the forces of selected elements from the ANSYS analysis for this typical wall.

The required reinforcement due to out-of-plane bending moments and in-plane forces are calculated by program "WALL" and is shown in Table 5.5-2. Similarly, the required reinforcement due to out-of-plane bending moments and out-of plane shear force in slabs are calculated by program "SLAB". These required reinforcements are calculated in element level and are defined as "Demands".

In order to demonstrate the results of the integrity evaluation, the above mentioned "Demands" need to be compared against the "Capacity" of the components. The capacity for each item of demands is defined below:

- Out-of-plane Bending Capacity for Walls and Slab:

  The capacity is based on the designer's selection of the reinforcement in each of the two orthogonal directions. This is really a "design capacity" instead of an absolute capacity. For the comparison with the demand, the basis of the capacity is stated in this report.

- Out-of-plane Shear Capacity for Walls and Slab:

  The capacity is based on concrete strength of the section cut without the consideration of shear reinforcement. It is essentially a "design capacity" as well since the capacity can be increased if the shear reinforcement is to be added by design engineer.

- In-plane Shear Capacity for Walls:

  The capacity is the maximum code allowed capacity for the wall section. For walls with the length of l_w, thickness of t, concrete strength of fc' and reduction factor of Φ (=0.6), the design strength allowed by ACI 349-01 is $\phi 10\sqrt{f_c^{'}} * (l_w * t)$ which includes the strength contributed by shear reinforcement. This is a "true capacity" not a "design capacity".

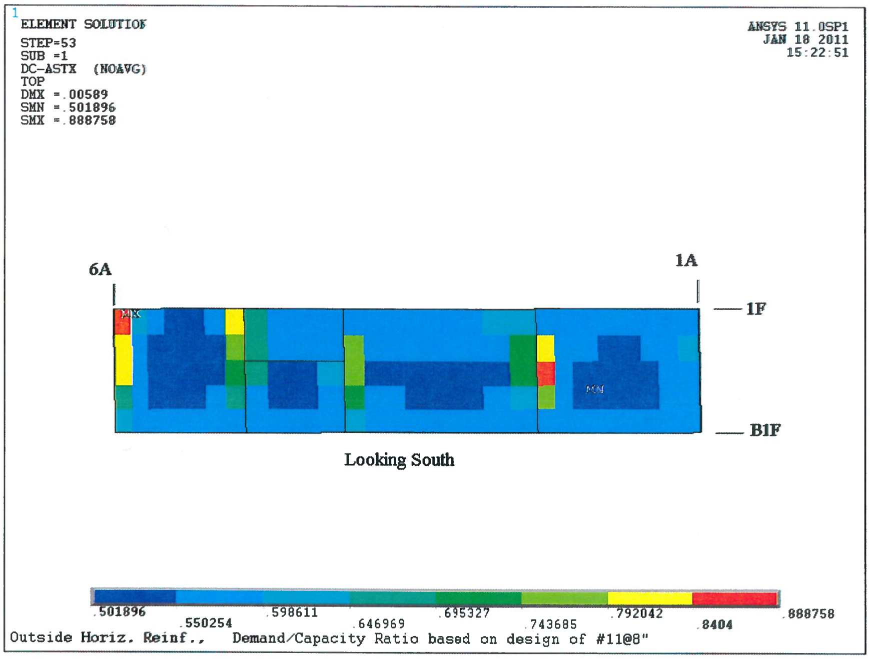
The demand/capacity ratio for in-plane shear, out-of plane shear (horizontal and vertical directions), out of plane bending (horizontal and vertical directions) are summarized in Table 5.5-3. The demand/capacity ratios are shown in the contour plots in Figures 5.5-1 to 5.5-8. All the DCRs are less than 1 with the exception of out-of-plane shear in horizontal cut in shear wall along Column Line AA (See Figure 5.5-4). In order to reduce the DCRs, there are a few options, including increasing wall thickness, adding boundary beams or adding shear reinforcement. A more extensive check of the remaining walls will be provided in the future revision of the report.

Table 5.5-1 Typical Wall Element Forces

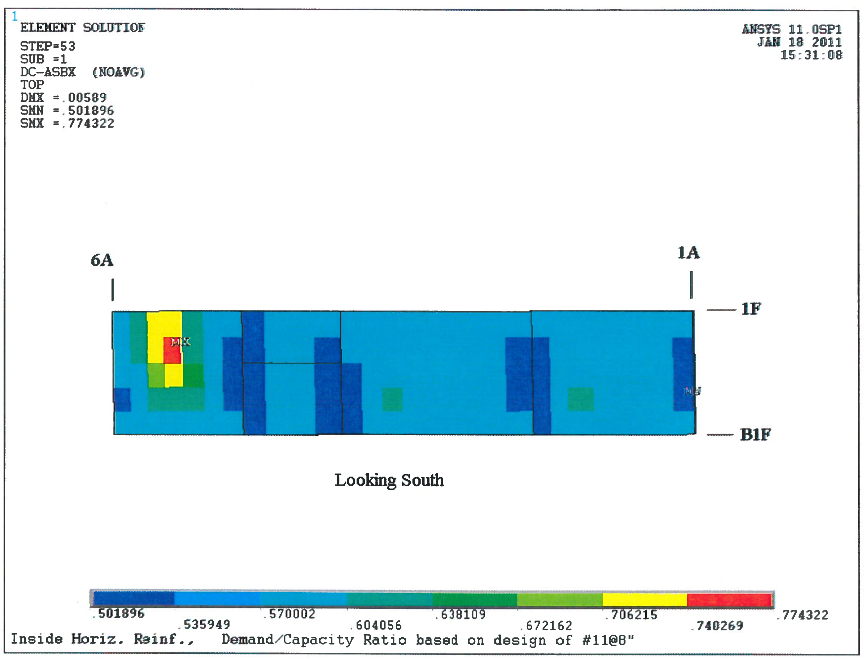
Element	Load Case	In-plane Base Shear (kips)	Nx (kip/ft)	Ny (kip/ft)	Nxy (kip/ft)	Mx (kip-ft/ft)	My (kip-ft/ft)	Mxy (kip-ft/ft)
3057	13	14613.4	4.11	9.19	112.41	193.24	-0.45	8.0
3410	37	12895	-2.99	213.0	99.19	38.65	38.14	61.06
4141	20	14080	-30.2	-270.1	108.3	-189.1	-132.8	-1.56
3411	29	9334	-10.6	165.1	71.8	0.46	-21.8	47.1

Table 5.5-2 Typical Wall Required Reinforcement

Element	Load Case	ASTX (in^2/ft)	ASBX (in^2/ft)	ASTY (in^2/ft)	ASBY (in^2/ft)
3057	13	1.417	0.576	0.768	0.768
3410	37	0.85	0.768	2.63	2.19
4141	20	0.576	1.06	1.304	1.75
3411	29	0.768	0.805	1.699	2.226

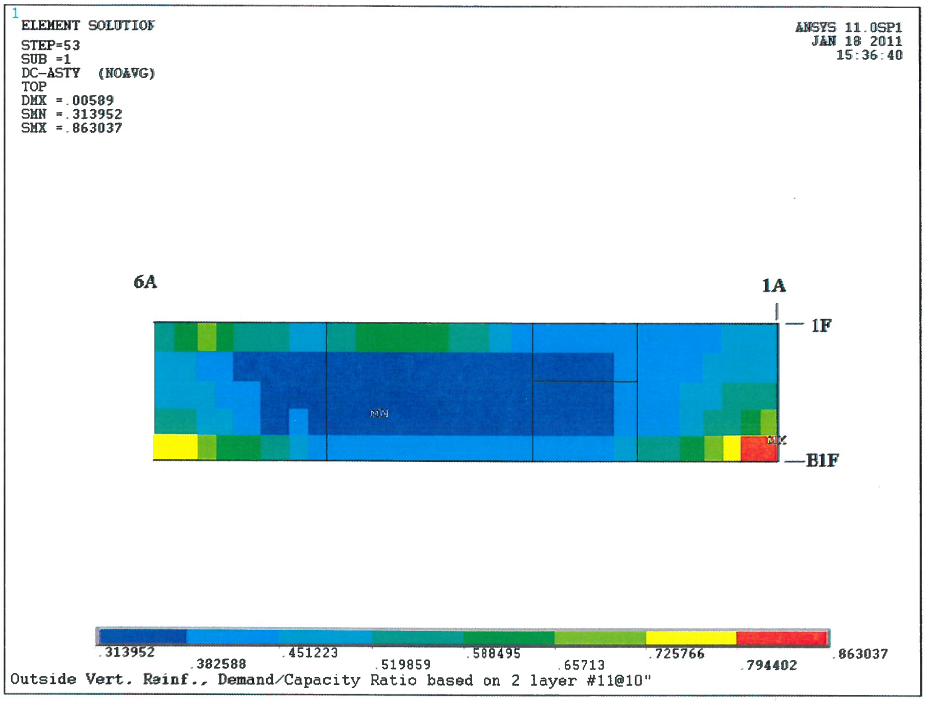


Outside Layer Moment Demand/Capacity for Walls in Column AA between B1F to 1F

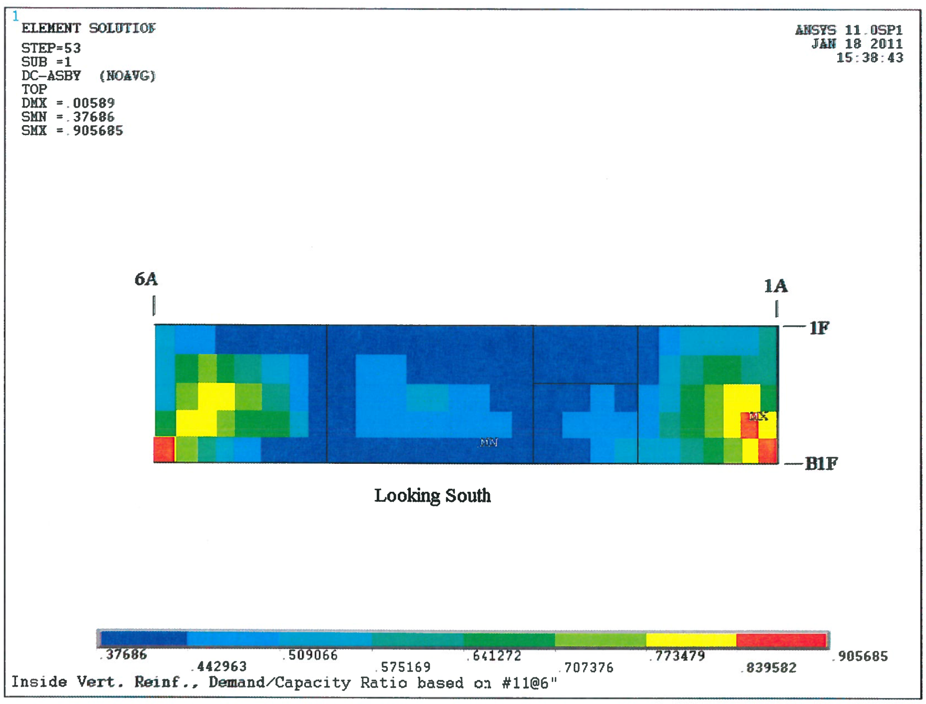


Inside Layer Moment Demand/Capacity for Walls in Column AA between B1F to 1F

Figure 5.5-1 Selected Exterior Wall: Vertical Cut Out-of-plane Moment DCR



Outside Layer Moment Demand/Capacity for Walls in Column AA between B1F to 1F



Inside Layer Moment Demand/Capacity for Walls in Column AA between B1F to 1F

Figure 5.5-2 Selected Exterior Wall: Horizontal Cut Out-of-plane Moment DCR

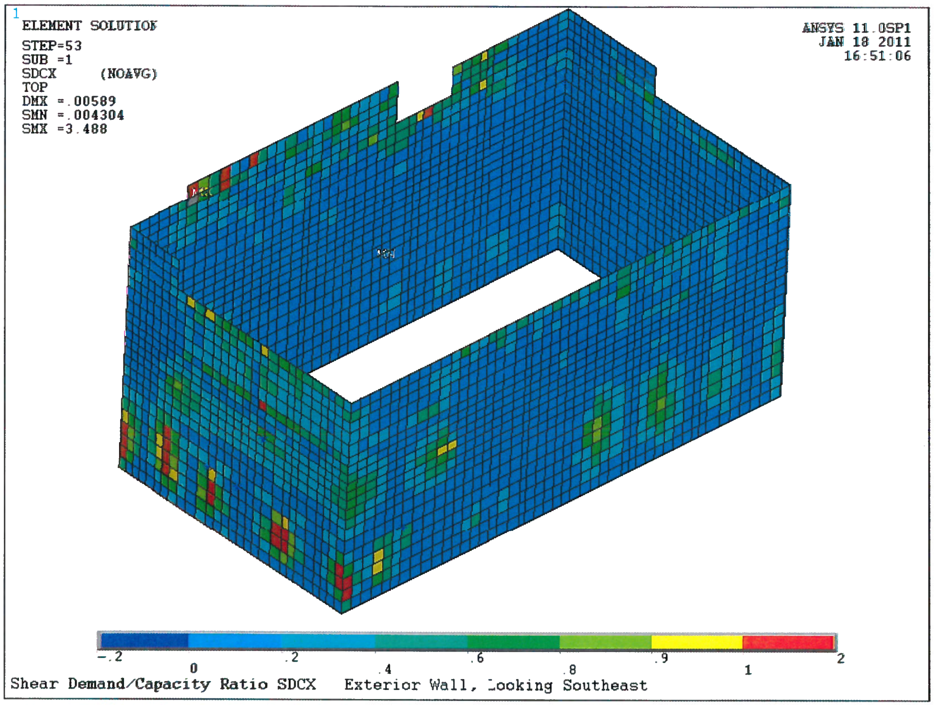


Figure 5.5-3 Exterior walls: Vertical Cut Out-of-plane Shear DCR

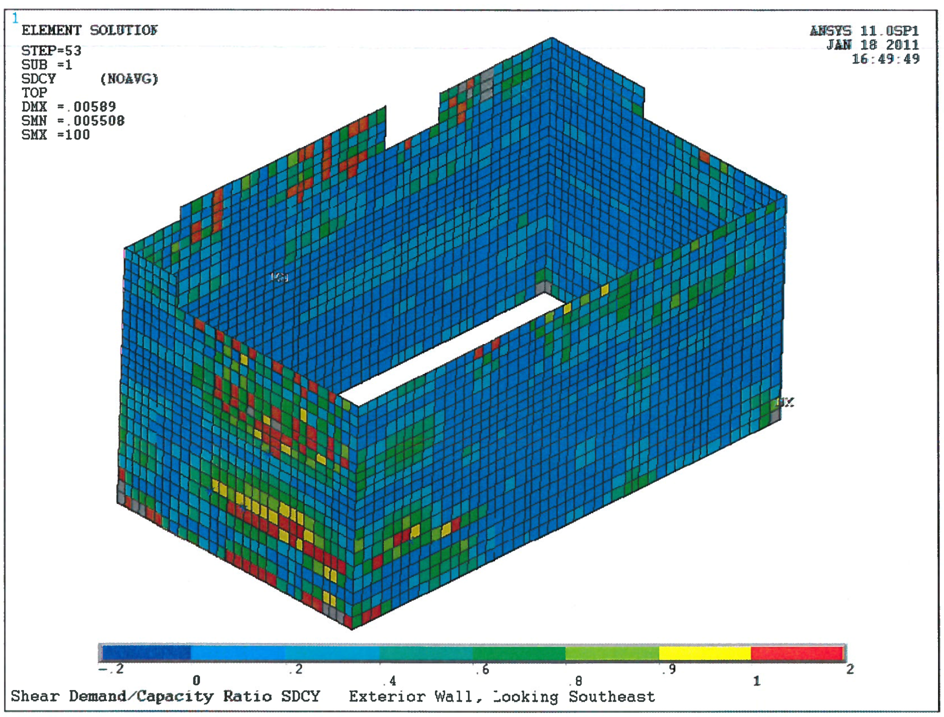
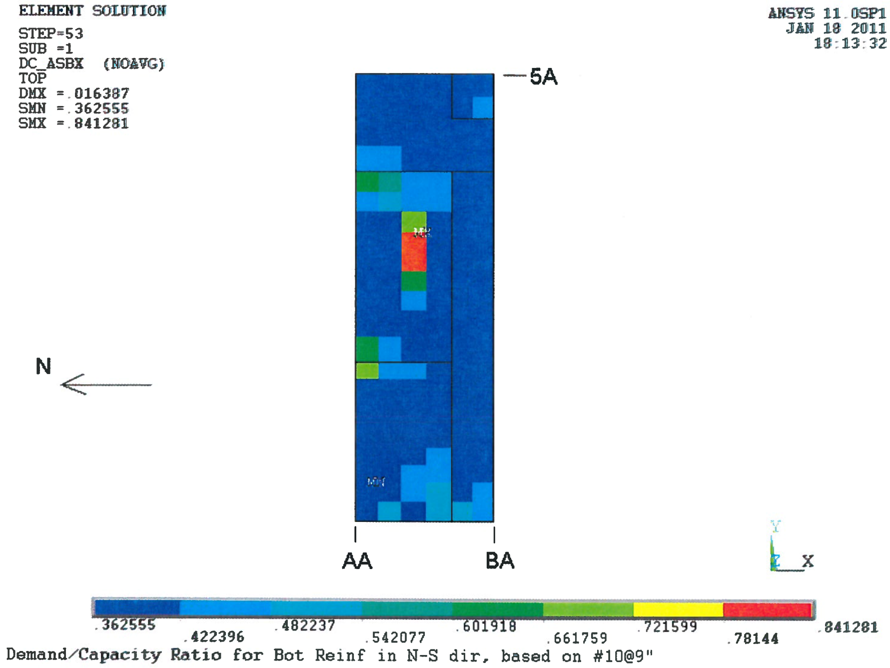
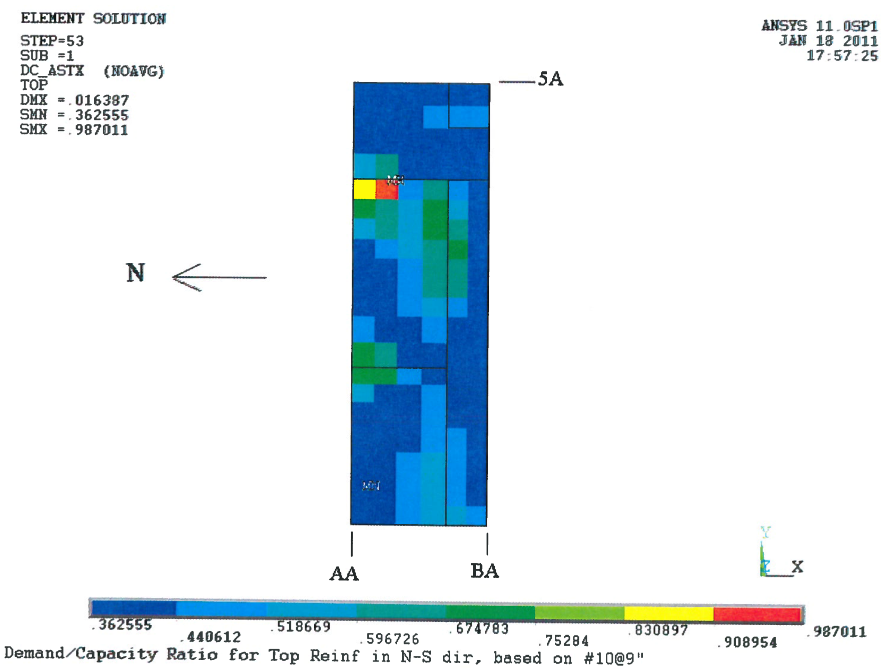


Figure 5.5-4 Exterior Walls: Horizontal Cut Out-of-plane Shear DCR

Note: All wall sections not within distance “d” from the interfacing interior walls or floor slabs (not shown) have D/C ratio less than 1. The shear capacity is based on the concrete strength, ΦV_n alone.

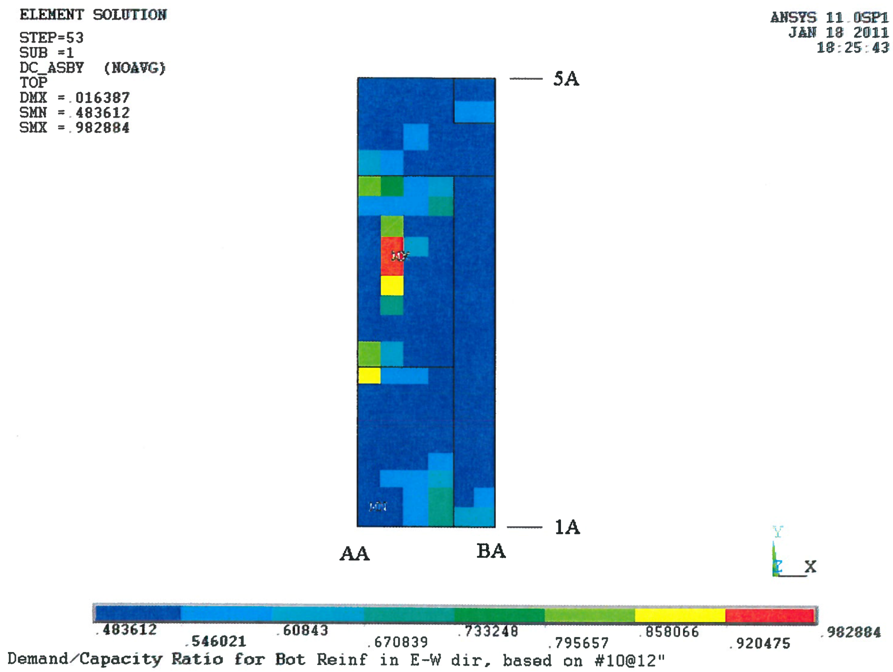


DCR for N-S Bottom Reinforcement at North End of 1st Floor Slab

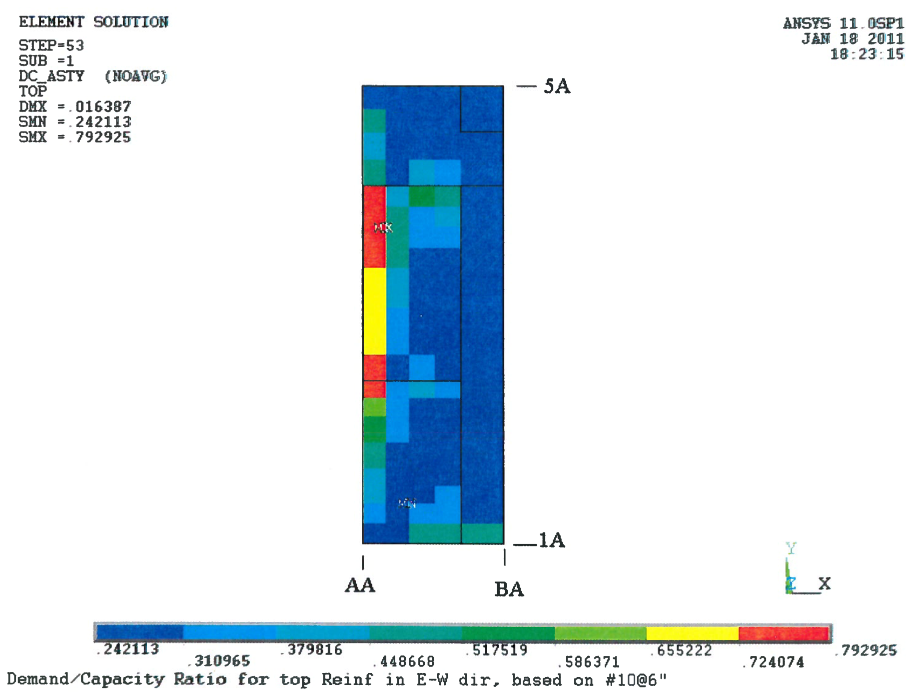


DCR for N-S Top Reinforcement at North End of 1st Floor Slab

Figure 5.5-5 Selected Ground Slab: E-W Direction Cut Out-of-plane Moment DCR



DCR for E-W Bottom Reinforcement at North End of 1st Floor Slab



DCR for E-W Top Reinforcement at North End of 1st Floor Slab

Figure 5.5-6 Selected Ground Slab: N-S Direction Cut Out-of-plane Moment DCR

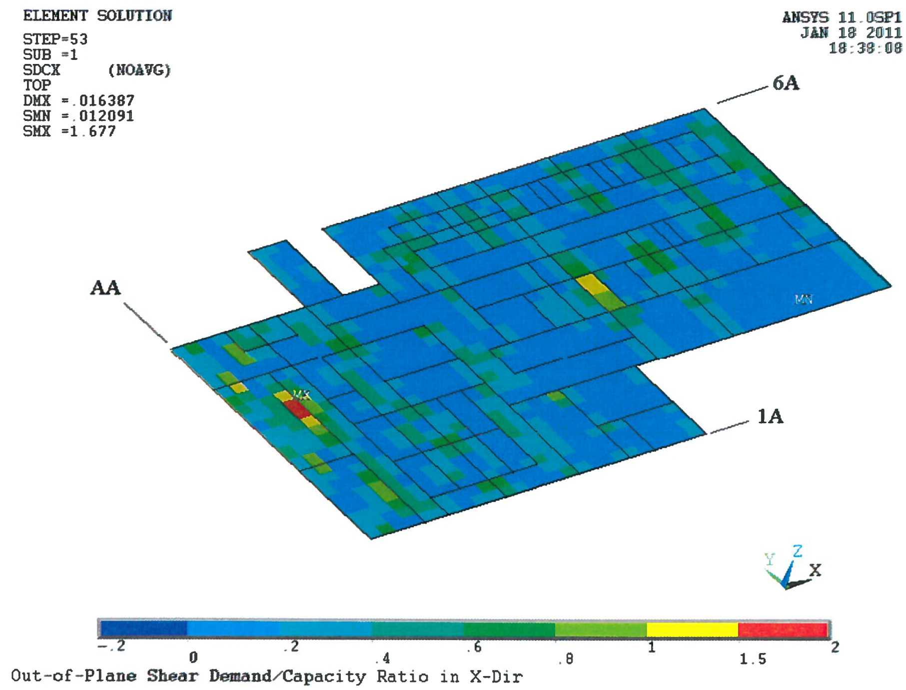


Figure 5.5-7 Ground Slabs: E-W Direction Cut Out-of-plane Shear DCR

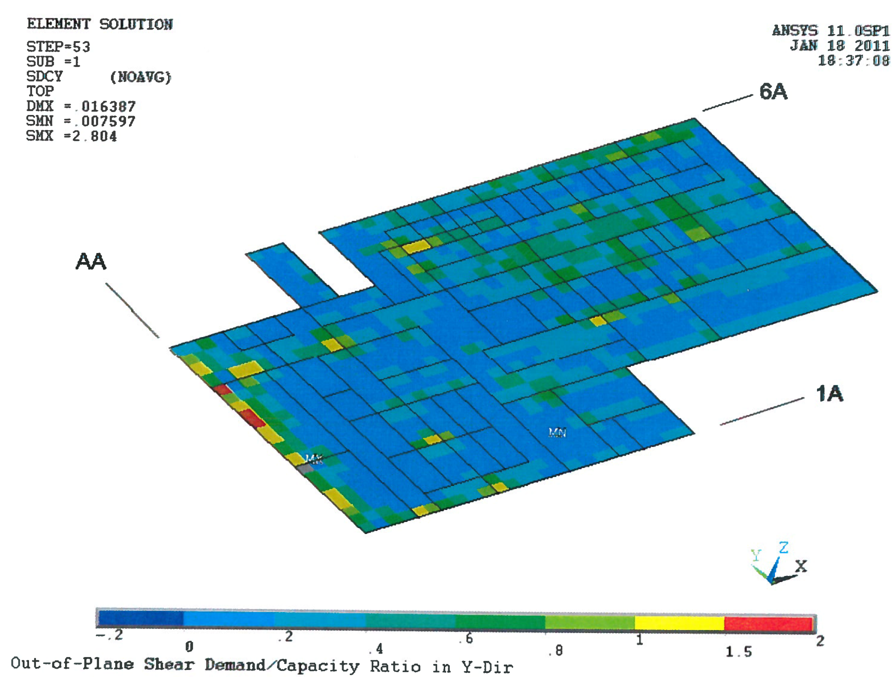


Figure 5.5-8 Ground Slabs: N-S Direction Cut Out-of-plane Shear DCR

Note: All slabs sections not within "d" distance from the face of the interfacing walls have DCR < 1.0
Shear capacity is based on concrete strength, ΦV_n, alone.

Table 5.5-3 Typical Structural Demands

Location of Components	Items	Demand/Capacity Ratio	Reference Figure
Wall in Column Line AA Between B1F and 1F	Wall In-plane-shear	0.617	NA
Wall in Column Line AA above and below 3rd floor	Wall out-of-plane shear (Horizontal)	1< DCR <2 for wall above and below 3rd floor	Figure 5.5.3
Wall in Column Line AA Between B1F and 1F	Wall out-of-plane shear (Vertical)	< 1 for all wall sections not within the "d" distance of the interfacing walls	Figure 5.5.4
Wall in Column Line AA Between B1F and 1F	Wall out-of-plane bending Vertical Cut	< 0.89	Figure 5.5.1
Wall in Column Line AA Between B1F and 1F	Wall out-of-plane bending Horizontal Cut	< 0.91	Figure 5.5.2
1st Floor Slab	E-W direction cut out-of-plane shear	<1.0 for slab not within the "d" distance from wall surface	Figure 5.5.7
1st Floor Slab	N-S direction cut out-of-plane shear	<1.0 for slab not within the "d" distance from wall surface	Figure 5.5.8
1st Floor Slab	E-W direction cut out-of-plane bending	< 0.99 for #10@9" top and bottom reinforcement	Figure 5.5.5
1st Floor Slab	N-S direction cut out-of-plane bending	<0.98 for #10@6" top and #10@12" for bottom	Figure 5.5.6

6.0 STABILITY EVALUATION OF A/B

6.1 Stability Evaluation Methodology

The sliding stability of the A/B is analyzed by reviewing the SSI response obtained from the site-independent SSI analyses of the A/B lumped mass stick model. ACS SASSI "stress" and "motion" modules provide time histories of the member forces and moments and time history of the basemat accelerations. Time histories are generated for each soil case for each seismic input motion direction for each response direction considering the global orientation of the stick member at the base of the A/B stick model. Horizontal seismic forces acting along the North-South and East-West directions of the A/B are then computed and combined to find the resultant driving force acting on the building. The resultant driving force is then compared with the resisting force to develop the factor of safety (FOS) of the A/B against sliding.

The seismic force acting on the A/B is generated at each time step by combining the member forces at the base of the A/B stick models with the inertia that is due to the mass of the basemat. At rest soil pressures acting along the North and East sides of the A/B foundation as well as a 500 psf surcharge load are added to obtain the total driving force. Section 3.6.3 of MUAP 10006 (R2) (Reference 8.2) provides the magnitude of the static at rest soil pressure. The resisting force is based on the friction resistance at foundation subgrade interface and is calculated by considering the weight of the building, the effects of buoyancy and the effect of vertical seismic response of the building at each time step.

Similar to sliding, the A/B overturning stability along each of its four edges (north, south, east and west) is also analyzed by reviewing the time histories of SSI responses of lumped mass stick model for each soil case for each direction of seismic input motion. Vertical and horizontal seismic forces are multiplied by their lever arm distance and added to the stick element end moments to determine the total overturning moment acting along each of the R/B four edges. The corresponding resisting moment due to weight of the building is reduced for the effect of buoyancy. The Factor of Safety (FOS) against overturning for each edge of A/B at each time step is calculated as the ratio of the restoring moment to the overturning moment.

7.0 CONCLUSION

Section 7 was not completely updated in revision 1 and may be out of date. This data will be updated in future revisions of this report when the applicable analysis is complete.

This report documents the methodology, the models and the results of the SSI analyses and structural integrity evaluation of the A/B to confirm that the structural integrity of the building is maintained under design earthquake excitation.

Sections 2.5 and 3.4, respectively, present the description and validation of the lumped-mass stick model and Dynamic FE Model used for SSI analyses. The detailed and dynamic FE models used for static and RSA are described and validated as presented in Sections 2.3,2.4, and 3.0.

The dynamic properties of the A/B model are presented in Section 3.2. The results of the SSI analyses of lumped-mass stick model for eight generic soil profiles and dynamic FE models for three selected critical soil profiles are presented in Section 4.2 and 4.3, respectively. The SSI analyses of A/B dynamic FE model yield maximum seismic horizontal displacements relative to the free field are on the order of 0.6" and 0.9" for NS and EW direction, respectively (Figure 4.3-1). TR MUAP-10006 Table 4-3 (Reference 8.2) shows that R/B EW direction displacement relative to the free field at node NW 03, which is at about same elevation as A/B roof level, is on the order of 1.1". NS direction displacement relative to free field at roof level of PS/B is on the order of 0.37" (Reference 8.2, Table 4-4). A/B is located at west to R/B and north to PS/B. The buildings are separated by 4" wide gap. Conservatively assuming that the two adjacent buildings move toward each other during an earthquake, the minimum net gap between A/B and R/B will be 4"-(0.9"+1.1") = 2.0", between A/B and PS/B will be 4"-(0.6"+0.37")=3.03". Therefore, the 4" gap is wide enough to accommodate earthquake induced deflections between A/B and R/B, A/B and PS/B. In future revision of this report, the maximum seismic displacements will be combined with non-seismic horizontal displacements that are due to possible tilting of A/B and effects of adjacent buildings and/or construction tolerances to check if the gaps between the A/B and adjacent structures are adequate.

Structural static and seismic demands are obtained by applying fixed boundary condition at bottom of basemat without consideration of uplift. The seismic base shear and vertical forces are presented in Section 5.3. The demand to capacity ratios (DCR) for selected members are presented in Section 5.5. The structural integrity evaluation of selected major structural components demonstrates the major structural components, namely, exterior walls and major floor slabs, have adequate thickness for an SSE event based on the analysis summarized in this report. The wall along Column Line AA may require additional shear reinforcement or increase of wall thickness locally to accommodate out-of-plane shear. This is to be determined during the detailed design of the building.

8.0 REFERENCES

Section 8 was not updated in revision 1 and may be out of date. This data will be updated in future revisions of this report when the applicable analysis is complete.

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