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AE 481W

Structural Technical Report 3

Lateral System Analysis and Confirmation Design

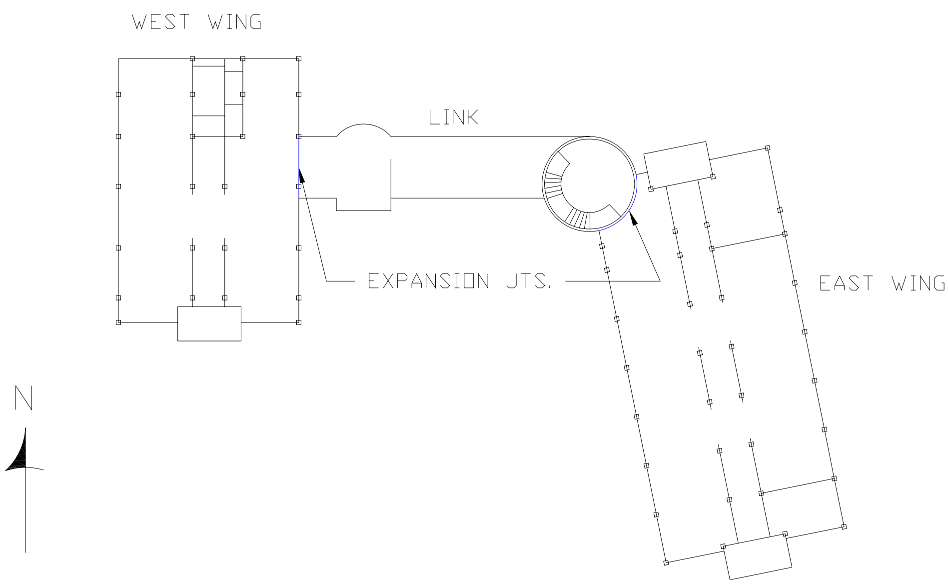
Executive Summary

The purpose of this report is to provide a detailed analysis of the existing design for the lateral force resisting system of the Koshland Integrated Natural Science Center, located at Haverford, PA. The first portion of this report includes the development and distribution of the seismic and wind loadings acting on the KINSC. The second portion of this report contains a detailed description of the lateral supporting system found in the KINSC. This section will provide an explanation of the lateral supporting elements that were used in the analysis. The next section of the report contains the analysis results concerning the lateral load resisting systems of the building. To obtain accurate results, the use of ETABS, an analysis and design software package, was incorporated. In ETABS, the building was modeled, lateral loads were applied, and the lateral forces on each lateral supporting element were outputted. Base shear, torsion, and overturning moments were found with the use of the program. A spot check of a typical shear wall was also conducted to ensure that the design of the shear walls is sufficient.

The existing structural framing system of the KINSC is predominately precast concrete. Some steel was incorporated in the roof framing. The lateral force resisting system is also precast concrete. It consists of a number of precast shear walls, with a few CMU block shear walls. The existence of two expansion joints separates the building into three sub-structures, able to be analyzed independently for lateral loads. The report that follows describes the analysis procedure of obtaining the lateral loads from ASCE7-02 and applying them to the structure. Though much of the derivation of the lateral loads is referenced from Technical Report 1, some factors and values have changed due to further investigation of the building. Also, in the report, the distribution of the lateral loads to the lateral resisting members is described in detail. Following the distribution, two methods of analyses are conducted and assessed. The first method being the ETABS program, the second being hand calculations. After the analysis results have been compared, a typical shear wall is checked for sufficient reinforcement against shear, and flexure, as well as overturning.

Lateral Load Development

Located in the KINSC are two, 2” expansion joints at different locations. The first expansion joint separates the West Wing of the building from the Link (the narrow corridor). The second expansion joint separates the Link from the East Wing. Therefore, the West Wing, the Link, and the East Wing all act independently when considering lateral loads. Separate analyses of the lateral loads were carried out for each one of these three sub-structures.



Wind

As conducted in Technical Report 1, the wind loads were calculated with the use of ASCE 7-02. Wind loads were calculated in both the N-S direction and the E-W direction for the East Wing, the Link, and the West Wing separately. The following tables display the derivation of the wind loads, and the wind forces on each of the three sub-structures in distributed loads of pounds per foot of width and in concentrated loads of kips.

Wind Analysis

Simplified Method - ASCE 7 - 02 Sec. 6.4

Wind Load Factors			
mean building height (must be < 60'):	h (ft.) =	53.19	
Basic Wind Speed:	V (mph) =	75	From General Notes on Plans
Building Category:	Category	III	Table 1-1
Importance Factor:	I =	1.15	Table 6-1
Exposure Category:	Category	B	Sec. 6.5.6
Ht.& Exposure Adjustment Coeff.:	λ =	1.178	Fig. 6-2; by interpolation

Zone	p_{s30}	
A	11.5	Horizontal Pressures
B	-5.9	
C	7.6	
D	-3.5	
E	-13.8	Vertical Pressure
F	-7.8	
G	-9.6	
H	-6.1	

$p_s = \lambda * I * p_{s30}$
I = 1.15
$p_{s30} = 11.5 - (-5.9)$
λ: see below

height	λ	I	$p_{total} = \lambda * I * p_{s30}$ (psf)
15	1.00	1.15	19.13
20	1.00	1.15	19.13
25	1.00	1.15	19.13
30	1.00	1.15	19.13
35	1.05	1.15	19.79
40	1.09	1.15	20.32
45	1.12	1.15	20.71
50	1.16	1.15	21.24
55	1.19	1.15	21.64
60	1.22	1.15	22.03

Forces on West Wing & East Wing due to Wind Loads

		kips (n-s)		kips (e-w)	
Level	plf	West	East	West	East
Roof	151.97	11.63	11.63	18.05	27.78
4th	277.88	21.26	21.26	33.01	50.80
3rd	252.44	19.31	19.31	29.99	46.15
2nd	248.69	19.02	19.02	29.54	45.46
1st	0	0	0	0	0
Basement	------	------	------	------	------
Base Shear		71.22	71.22	110.60	170.18

Forces on Link due to Wind Loads

		kips (n-s)	kips (e-w)
Level	plf	Link	Link
Roof	133.64	17.07	4.70
3rd	252.44	32.24	8.89
2nd	248.69	31.76	8.75
1st	0	0.00	0.00
Basement	------	------	------
Base Shear		81.06	22.34

Seismic

The seismic loads for the KINSC were calculated by the use of section 9 of ASCE 7-02. Originally, in Technical Report 1, the building was designated with a seismic use group of III, because it was considered an educational building with a capacity of over 500 people. However, since the building was analyzed in three smaller substructures, each structure was categorized as having a seismic use group of II. In addition, the response modification factors that were used in Technical Report 1 were incorrect and were changed accordingly. With the lateral resisting system being precast concrete shear walls and a few CMU shear walls, the R values were recorded as 5 and 2 respectively. The following tables display derivation of the seismic forces on the building, and the story shears in kips on the building due to seismic loads.

Seismic Information		East & West Wings	Link
Building Location		Haverford, PA	Haverford, PA
# of stories		4	3
inner story ht.		13	13
Bldg. height		53	39
Seismic Use Group		II	II
Importance Factor		1.15	1.15
Site Classification		B	B
0.2s Acceleration		0.35	0.35
1.0s Acceleration		0.08	0.08
Site Class Factor:			
	Fa	1.00	1.00
	Fv	1.00	1.00
Adjusted Accelerations			0.35
	S_{ms}	0.35	
	S_{m1}	0.077	0.077
Spectral Response Accelerations			
	S_{DS}	0.233	0.233
	S_{D1}	0.051	0.051
Seismic Design Category		**B**	**B**

Seismic Analysis

East Wing

Vertical Distribution of Seismic Forces

$k_{N\text{-}S} = 1 + (T_{N\text{-}S} - 0.5)/(2.5 - 0.5) =$ 0.946

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
			-	0.000	-		-
Roof	1297	53	55,579	0.161	71	-	3,757
4	4455	39	142,772	0.413	182	71	7,101
3	4455	26	97,271	0.281	124	253	3,225
2	4455	13	50,476	0.146	64	377	837
1						441	
	Σ = 14661		Σ = 346098	Σ = 1.000	Σ = 441		Σ = 14921

Link

Vertical Distribution of Seismic Forces

$k_{E\text{-}W} = 1 + (T_{E\text{-}W}\ 0.5)/(2.5 - 0.5) =$ 0.906

Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
			-	0.000	-		-
Roof	561	39	15,516	0.242	89	-	3,461
3	1652	26	31,633	0.494	181	89	4,704
2	1652	13	16,881	0.264	97	270	1,255
1						-	
	Σ = 3866		Σ = 64030	Σ = 1.000	Σ = 366		Σ = 9421

West Wing

Vertical Distribution of Seismic Forces

$k_{N\text{-}S} = 1 + (T_{N\text{-}S} - 0.5)/(2.5 - 0.5) =$ 0.946

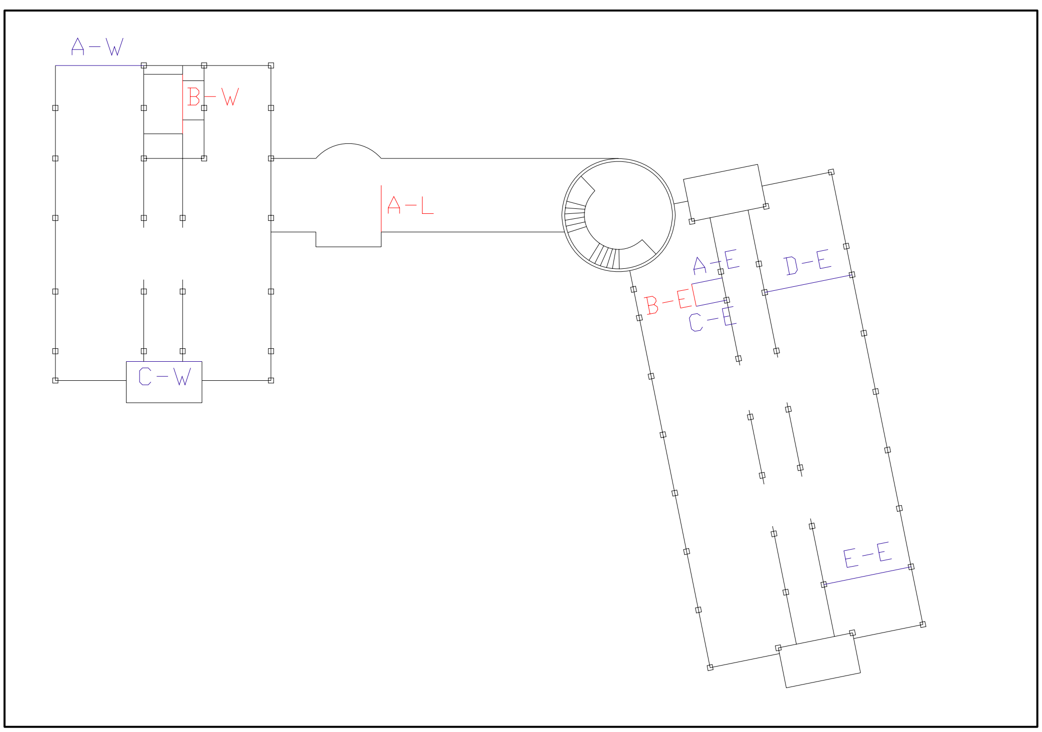
Level, x	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x	V_x	M_x
	(kips)	(ft)			(kips)	(kips)	(ft-kips)
			-	0.000	-		-
Roof	892	53	38,203	0.164	49	-	2,578
4	2992	39	95,895	0.411	122	49	4,762
3	2992	26	65,334	0.280	83	171	2,163
2	2992	13	33,903	0.145	43	254	561
1						297	
	Σ = 9868		Σ = 233335	Σ = 1.000	Σ = 297		Σ = 10064

From the information provided in the tables above, it is apparent that the seismic loading on the East Wing, the Link, and the West Wing controls over all wind loading in both directions. In fact, for the West and East Wings, seismic loading controls in the East-to-West direction due to building geometry. Seismic controls in the North-to-South direction for the Link for the same reason. Although both wind and seismic were tested in the ETABS analysis, for simplicity, only the seismic results will be outputted in this report.

Lateral Loads Distribution

When distributing lateral loads that are acting on a building, the rule of thumb is that the load will follow the path of the greatest stiffness of the elements that make up the lateral system in the structure. Therefore, whatever lateral supporting members have greater stiffnesses, or rigidities, a larger portion of the lateral load will be distributed to those members. This method of distribution by rigidity was used in the lateral load analysis of the KINSC.

The lateral system found in the KINSC is entirely comprised of shear walls, mainly precast concrete. However, a few CMU shear walls are located throughout the building. In the West Wing, there are two 8" precast shear walls that run in the E-W direction, along with one 8" CMU block shear wall that runs in the N-S direction. Located in the Link is a single 10" CMU block shear wall spanning in the N-S direction. Finally, in the East Wing, two 8" precast concrete shear walls are located on the east side of the sub-structure running in the E-W direction. In addition, the elevator core, which is constructed of 8" CMU blocks, acts as shear walls in both directions. All shear walls located in the KINSC span from the ground floor to the 4th floor. The reinforcement of all shear walls is unknown. Below is a typical floor plan identifying the different shear walls and their locations.



General Shear Wall Information

Shearwall	Location	span	ht.	t	Direction
A-W	West Wing	31'-6"	52'	8"	E-W
B-W	West Wing	10'-9-1/4"	52'	8"	N-S
C-W	West Wing	26'-8"	52'	8"	E-W
A-L	Link	16'-5-1/2"	52'	10"	N-S
A-E	East Wing	11'-0"	52'	8"	E-W
B-E	East Wing	8'-0"	52'	8"	N-S
C-E	East Wing	11'-0"	52'	8"	E-W
D-E	East Wing	31'-5"	52'	8"	E-W
E-E	East Wing	31'-5"	52'	8"	E-W

To correctly distribute the controlling seismic loads acting on the building to the appropriate shear walls, the stiffnesses of each shear wall had to be calculated. To perform this task, two methods were used. The first method involved the use of the design program software, RAM Advance. Each shear wall was constructed in the RAM program and given the appropriate material properties and geometry. Next a 1 kip force was applied in the lateral direction at each story of the shear wall at a time. This provided a deflection at each floor due to that 1 kip lateral force. From these deflections, the relative stiffness of each shear wall could be found by the equation, $k = 1/\Delta$.

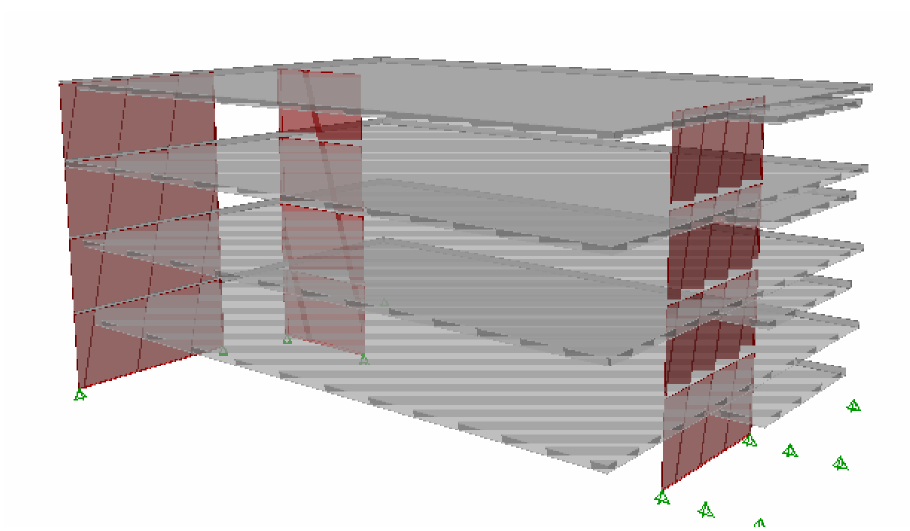
The second method used to find stiffnesses of each shear wall was based around the equation, $k = (E*t)/(4*(h/L)^3+(3*(h/L))$. The required variables for this equation were the shear wall height, length, and thickness, which are all known values. The resulting stiffness values from both methods proved to be similar. However, the stiffness values from the first method seemed to be the more conservative numbers, and were, therefore, used in this analysis.

Finding the stiffness of each shear wall is only the first step in distributing the lateral loads properly. Next, the distribution proportion was calculated. To perform this task, the sum of the stiffnesses of all shear walls running in each direction was calculated. Therefore, the sum of stiffnesses for walls running E-W was recorded, and then the same for walls running N-S was recorded. This task was performed three different times, once for each sub-structure. From this point, the distribution proportions for each shear wall were found by dividing the stiffness of each wall by the sum of shear wall stiffnesses in the corresponding directions. Once the distribution proportions were calculated, the appropriate story shears were multiplied by the distribution proportions. This computation resulted in the distributed shears on each shear wall. For an example of this procedure, refer to the ***Lateral Load Analysis*** section of the report. In addition, with the use of ETABS, the distribution of the lateral forces was computed and executed automatically. This will be discussed in the following section as well.

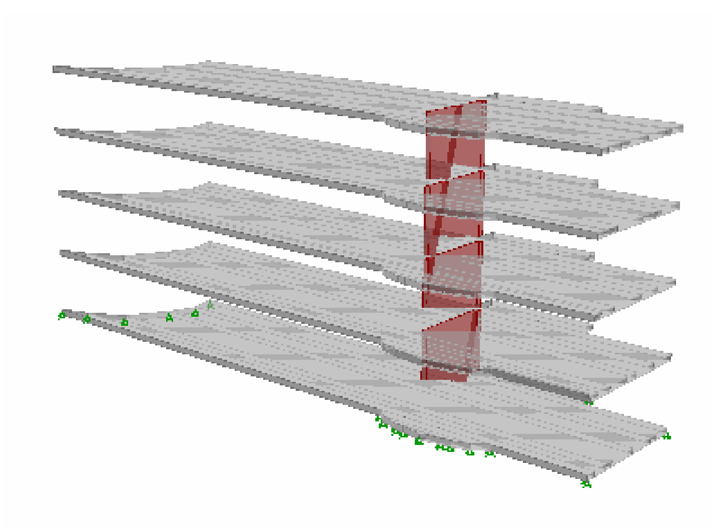
Lateral Load Analysis

For the analysis of the lateral system in the KINSC, the use of ETABS was heavily incorporated. Each sub-structure was accurately modeled in ETABS as a separate structure under lateral loading. One of the advantageous features of the software is the ability to look at the floors of the building strictly as rigid diaphragms against lateral loading. With this option, any columns, beams, or other structural members not included in the lateral system of the building, did not need to be inputted into the program. They could simply be omitted. Therefore, the floor slabs and shear walls were the only elements that were necessary to model in the program. Material properties as well as geometric properties were inputted for both the slabs and the shear walls. Next, the floor diaphragms were drawn, followed by the shear walls and any openings in the slab, such as the elevator core. All floors of the KINSC are typical in slab and shear wall size and spans. The following images are basic renderings from ETABS of the lateral systems of each sub-structure.

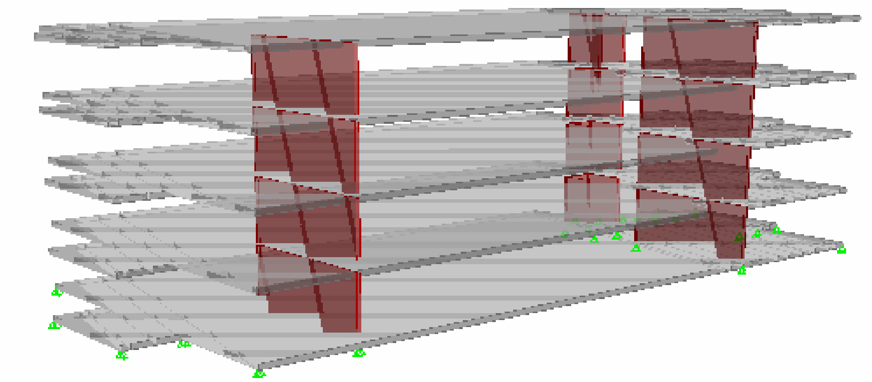
West Wing



Link



East Wing



The next task was to define and apply the lateral loads acting on the sub-structures. Since the option to focus strictly on lateral loads was utilized, dead loads and live loads were not incorporated in the analysis of the KINSC. Therefore, only wind loads in the x and y directions, as well as seismic loads in the x and y directions were defined and applied. Due to the omission of the structural framing that was not part of the lateral system, the program was not analyzing the seismic loads on the building with the correct building weight. With this being the case, user-defined loads from the ***Lateral Load Development*** section of this report were inputted for seismic. Both wind and seismic loads were applied about the center of rigidity of each substructure, which was calculated automatically in ETABS. Refer to Appendix C for an example display showing the auto-calculation of the center of rigidity. After all of the program editing was completed, the analysis was run for both wind and seismic loads on the sub-structures. Once the analysis had been run, the program was able to output values such as story shear, torsion, moments, and story drift for each shear wall that was analyzed. The tables found in Appendix A display these output values for all of the shear walls for the controlling seismic loads.

In addition to running an analysis through ETABS, an analysis was performed by manually finding the relative stiffnesses of each shear wall of the three separate sub-structures. For simplicity, this method of lateral analysis was conducted on the West Wing shear walls running in the E-W direction. This analysis of the West Wing shear walls will act as a representation for the entire building. This procedure was performed as a check against the results from the ETABS software. As stated in the previous section of this report, the relative stiffnesses of each shear wall were found by two different methods. The more conservative value was used. Next the distribution proportions were calculated. Finally the story shears were then distributed accordingly to the shear walls based on the distribution proportions. The following tables display the procedure, and ultimately, the story shears on the E-W shear walls of the West Wing.

story shears

E-W	(kips)
Roof	
4	49
3	171
2	254
1	297

Shear Wall A-W

Location	qty.	span	ht.	t	Direction	f'c (psi)	E (ksi)
West Wing	1	31'-6"	52'	8"	E-W	5000	4030

k_{eq} = (E*t)/(4*(h/L)^3+(3*(h/L))

Floor	Δ	k = 1/Δ	k_{eq}	Σk (E-W)	prop.	shear (kips)
1st	0.00006747	14821.4	21220.3	28595.5	0.518312	**153.94**
2nd	0.000159	6289.3	6823.3	11096.99	0.566757	**143.96**
3rd	0.000356	2808.9	2851.6	4746.884	0.591735	**101.19**
4th	0.000737	1356.9	1405	2261.059	0.600117	**29.41**

Shear Wall C-W

Location	qty.	span	ht.	t	Direction	f'c (psi)	E (ksi)
West Wing	1	26'-8"	52'	8"	E-W	5000	4030

k_{eq} = (E*t)/(4*(h/L)^3+(3*(h/L))

Floor	Δ	k = 1/Δ	k_{eq}	Σk (E-W)	prop.	shear (kips)
	0.0000726	13774.1	16739.4	28595.5	0.481688	**143.06**
2nd	0.000208	4807.692	4861.3	11096.99	0.433243	**110.04**
3rd	0.000516	1937.984	1907.7	4746.884	0.408265	**69.81**
4th	0.001106	904.1591	139.1	2261.059	0.399883	**19.59**

When the story shears in the E-W, West Wing shear walls from the tables above were compared with the ETABS output values found in Appendix A, there were noticeable discrepancies. Reasons for the significant difference in values may be due to the fact that the analysis by hand did not account for torsional shear, only direct shear. In addition, the ETABS analysis incorporates the ground floor shear walls in the design, while the hand analysis does not. The first floor of the KINSC is located at grade only for some sections of the building. At other sections, the grade slopes down to the ground elevation. The hand analysis does not take this into consideration. It is assumed that the first floor elevation is consistently located at grade. Also, the difference in shear values may ultimately be due to errors in the program output, as either a result of the program itself, or from human error regarding the input values.

Lateral Member Check

After analyzing the lateral loads on the building, a spot check of a typical shear wall was conducted. The typical shear wall that was selected for the lateral check was shear wall A-W. The purpose of the check was to verify that the reinforcing found in the wall is sufficient to support the shear and flexure carried by the wall, as well as to check the overturning moment. However, the existing reinforcement of the shear walls in the KINSC is unknown. Therefore, reinforcement was designed for the shear wall based on the loads from the analysis results. It is considered that the designed reinforcement for the shear wall is a realistic possibility for the existing reinforcement.

The PCA Simplified Design-Reinforced Concrete Buildings of Moderate Size and Height guide was referenced for the lateral member check. The shear strength in the wall was checked first. This check resulted in the minimum reinforcing in the shear wall at all floors, for both vertical and horizontal reinforcement. Secondly, the flexural strength of shear wall A-W was assessed. The flexural loads on the shear wall required an increase in only the reinforcement at the first floor. The results of reinforcement in the shear wall due to shear and flexural strength can be viewed in the table below. Finally, the overturning moment was checked. It was concluded that the shear wall in combination with the adjacent column are sufficient to resist the overturning moment caused by the controlling seismic load acting on the shear wall. Refer to Appendix B for all member check calculations

West Wing Shear Wall, A-W

Floor	Vertical Reinforcement	Horizontal Reinforcement
1st	# 5 bars @ 10" o.c.	#4 bars @10" o.c.
2nd	#4 bars @ 10" o.c.	#4 bars @10" o.c.
3rd	#4 bars @ 10" o.c.	#4 bars @10" o.c.
4th	#3 bars @ 10" o.c.	#4 bars @12" o.c.

Conclusion

The analysis results of this report indicate that further investigation is necessary for the lateral force resisting system of the KINSC. Although values were obtained through two different methods of lateral analysis, the difference in the values was significant. The reasons for the discrepancies are not certain, but should be found with further review of the lateral system. The methods of analysis did, however, provide a vast understanding of the existing lateral force resisting system of the building. Also, with the story shears that resulted from the analysis, though possibly inaccurate, a reasonable layout for steel reinforcement was obtained for the shear wall that was spot checked. In conclusion, realistic values resulted from this lateral load analysis, however further investigation should provide a more accurate and valid outcome.

Appendix A

ETAB Output Values for All Shear Walls

West Wing Shear Wall Output Values

Story	Pier	Load	Loc	P (kips)	V2 (kips)	V3 (kips)	T (in-k)	M2 (in-k)	M3 (in-k)
STORY4	A-W	SEISX	Top	-0.22	21.47	-0.23	332.269	0	-32.302
STORY4	A-W	SEISX	Bottom	-8.34	21.47	-0.23	332.269	-35.377	3316.732
STORY3	A-W	SEISX	Top	-8.98	42.68	-0.41	310.208	-35.377	3220.865
STORY3	A-W	SEISX	Bottom	-17.1	42.68	-0.41	310.208	-99.666	9879.602
STORY2	A-W	SEISX	Top	-18.11	59.17	-0.88	245.977	-99.667	9729.286
STORY2	A-W	SEISX	Bottom	-26.23	59.17	-0.88	245.977	-236.758	18959.568
STORY1	A-W	SEISX	Top	-27.42	58.7	1.52	79.074	-236.758	18780.606
STORY1	A-W	SEISX	Bottom	-35.55	58.7	1.52	79.074	0	27938.534

Story	Pier	Load	Loc	P (kips)	V2 (kips)	V3 (kips)	T (in-k)	M2 (in-k)	M3 (in-k)
STORY4	B-W	SEISY	Top	0	169.74	0.07	70.743	0	0
STORY4	B-W	SEISY	Bottom	-27	169.74	0.07	70.743	10.234	26479.558
STORY3	B-W	SEISY	Top	-27	252.18	0.1	65.628	10.234	26479.558
STORY3	B-W	SEISY	Bottom	-54	252.18	0.1	65.628	26.205	65819.233
STORY2	B-W	SEISY	Top	-54	291.25	0.25	51.451	26.205	65819.233
STORY2	B-W	SEISY	Bottom	-81	291.25	0.25	51.451	65.745	111253.744
STORY1	B-W	SEISY	Top	-81	305.83	-0.42	18.124	65.745	111253.744
STORY1	B-W	SEISY	Bottom	-108	305.83	-0.42	18.124	0	158964

Story	Pier	Load	Loc	P (kips)	V2 (kips)	V3 (kips)	T (in-k)	M2 (in-k)	M3 (in-k)
STORY4	C-W	SEISX	Top	0	149.5	-0.04	355.083	0	0
STORY4	C-W	SEISX	Bottom	-8.67	149.5	-0.04	355.083	-6.871	23322.025
STORY3	C-W	SEISX	Top	-8.67	211.22	-0.08	332.08	-6.871	23322.025
STORY3	C-W	SEISX	Bottom	-17.33	211.22	-0.08	332.08	-20.058	56272.161
STORY2	C-W	SEISX	Top	-17.33	237.52	-0.16	264.972	-20.058	56272.161
STORY2	C-W	SEISX	Bottom	-26	237.52	-0.16	264.972	-44.848	93324.558
STORY1	C-W	SEISX	Top	-26	238.74	0.29	79.909	-44.848	93324.558
STORY1	C-W	SEISX	Bottom	-34.66	238.74	0.29	79.909	0	130567.976

Link Shear Wall Output Values

Story	Pier	Load	Loc	P (kips)	V2 (kips)	V3 (kips)	T (in-k)	M2 (in-k)	M3 (in-k)
STORY4	A - L	SEISY	Top	0	89.05	0.12	65355.832	0	0
STORY4	A - L	SEISY	Bottom	-26.74	89.05	0.12	65355.832	17.967	13891.344
STORY3	A - L	SEISY	Top	-26.74	269.97	-0.08	166300.579	17.967	13891.344
STORY3	A - L	SEISY	Bottom	-53.49	269.97	-0.08	166300.579	5.722	56006.227
STORY2	A - L	SEISY	Top	-53.49	366.98	-0.04	206235.205	5.722	56006.227
STORY2	A - L	SEISY	Bottom	-80.23	366.98	-0.04	206235.205	-0.34	113255.401
STORY1	A - L	SEISY	Top	-80.23	367	0	206527.101	-0.34	113255.401
STORY1	A - L	SEISY	Bottom	-106.97	367	0	206527.101	0	170508

East Wing Shear Wall Output Values

Story	Pier	Load	Loc	P (kips)	V2 (kips)	V3 (kips)	T (in-k)	M2 (in-k)	M3 (in-k)
STORY4	A-E	SEISX	Top	36.07	-28.97	-0.1	7.597	4.99	2163.116
STORY4	A-E	SEISX	Bottom	23.07	-28.97	-0.1	7.597	-9.931	-2356.43
STORY3	A-E	SEISX	Top	108.46	-33.5	-0.16	9.23	2.211	2764.012
STORY3	A-E	SEISX	Bottom	95.46	-33.5	-0.16	9.23	-23.179	-2461.95
STORY2	A-E	SEISX	Top	202.27	-41.85	-0.29	7.313	-6.236	3943.098
STORY2	A-E	SEISX	Bottom	189.27	-41.85	-0.29	7.313	-50.916	-2585.62
STORY1	A-E	SEISX	Top	293.48	-10.4	0.09	13.853	-37.772	3664.04
STORY1	A-E	SEISX	Bottom	280.48	-10.4	0.09	13.853	-23.865	2042.258

Story	Pier	Load	Loc	P (kips)	V2 (kips)	V3 (kips)	T (in-k)	M2 (in-k)	M3 (in-k)
STORY4	B-E	SEISY	Top	-5.24	-245.62	-0.02	0.439	0.616	17372.27
STORY4	B-E	SEISY	Bottom	-15.64	-245.62	-0.02	0.439	-1.844	-20944
STORY3	B-E	SEISY	Top	-32.44	-365.51	-0.04	-2.825	0.282	21325.23
STORY3	B-E	SEISY	Bottom	-42.84	-365.51	-0.04	-2.825	-5.697	-35694.2
STORY2	B-E	SEISY	Top	-66.27	-419.85	-0.04	-8.48	-2.8	18588.21
STORY2	B-E	SEISY	Bottom	-76.67	-419.85	-0.04	-8.48	-8.344	-46908.7
STORY1	B-E	SEISY	Top	-118	-446.43	-0.09	-4.142	-3.575	9606.301
STORY1	B-E	SEISY	Bottom	-128.4	-446.43	-0.09	-4.142	-17.098	-60037

Story	Pier	Load	Loc	P (kips)	V2 (kips)	V3 (kips)	T (in-k)	M2 (in-k)	M3 (in-k)
STORY4	C-E	SEISX	Top	-37.58	31.43	-0.1	7.597	4.99	-2253.66
STORY4	C-E	SEISX	Bottom	-50.58	31.43	-0.1	7.597	-9.931	2649.384
STORY3	C-E	SEISX	Top	-144.6	45.15	-0.16	9.23	2.211	-2987.6
STORY3	C-E	SEISX	Bottom	-157.6	45.15	-0.16	9.23	-23.179	4055.798
STORY2	C-E	SEISX	Top	-281.21	57.68	-0.29	7.313	-6.236	-3355.54
STORY2	C-E	SEISX	Bottom	-294.21	57.68	-0.29	7.313	-50.916	5643.02
STORY1	C-E	SEISX	Top	-429.79	45.78	0.09	13.853	-37.772	-2485.22
STORY1	C-E	SEISX	Bottom	-442.79	45.78	0.09	13.853	-23.865	4656.582

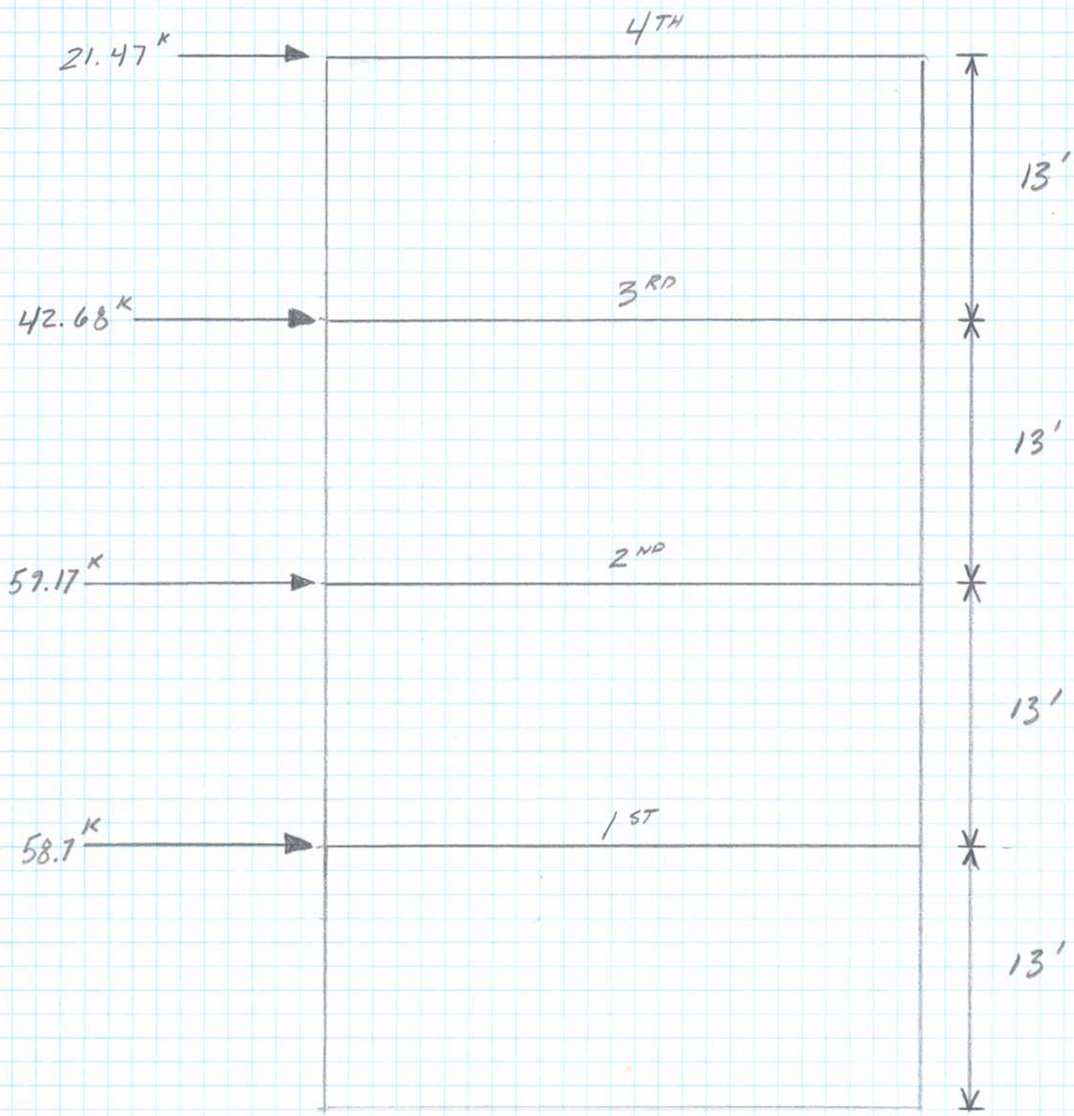
Story	Pier	Load	Loc	P (kips)	V2 (kips)	V3 (kips)	T (in-k)	M2 (in-k)	M3 (in-k)
STORY4	D-E	SEISX	Top	0	72.93	-0.12	14.616	0	0
STORY4	D-E	SEISX	Bottom	-40.84	72.93	-0.12	14.616	-19.072	11376.76
STORY3	D-E	SEISX	Top	-40.84	115.73	-0.2	14.052	-19.072	11376.76
STORY3	D-E	SEISX	Bottom	-81.68	115.73	-0.2	14.052	-49.566	29431.33
STORY2	D-E	SEISX	Top	-81.68	146.23	-0.88	10.96	-49.566	29431.33
STORY2	D-E	SEISX	Bottom	-122.52	146.23	-0.88	10.96	-186.699	52243.46
STORY1	D-E	SEISX	Top	-122.52	125.84	1.2	3.58	-186.699	52243.46
STORY1	D-E	SEISX	Bottom	-163.36	125.84	1.2	3.58	0	71874.03

Story	Pier	Load	Loc	P (kips)	V2 (kips)	V3 (kips)	T (in-k)	M2 (in-k)	M3 (in-k)
STORY4	E-E	SEISX	Top	0	172.57	-0.12	14.616	0	0
STORY4	E-E	SEISX	Bottom	-40.84	172.57	-0.12	14.616	-19.072	26921.65
STORY3	E-E	SEISX	Top	-40.84	242.09	-0.2	14.052	-19.072	26921.65
STORY3	E-E	SEISX	Bottom	-81.68	242.09	-0.2	14.052	-49.566	64687.51
STORY2	E-E	SEISX	Top	-81.68	270.13	-0.88	10.96	-49.566	64687.51
STORY2	E-E	SEISX	Bottom	-122.52	270.13	-0.88	10.96	-186.699	106827.7
STORY1	E-E	SEISX	Top	-122.52	270.94	1.2	3.58	-186.699	106827.7
STORY1	E-E	SEISX	Bottom	-163.36	270.94	1.2	3.58	0	149094

Appendix B

Shear Wall Spot Check

WEST WING - SHEAR WALL A-W



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SHEAR WALL SPOT CHECK

- CHECK SHEAR STRENGTH IN 1ST STORY

TOTAL SHEAR = 21.47 + 42.68 + 59.17 + 58.7 = 182.02

$V_u = 1.6(182.02) = 291.2^k$

$\phi V_c = 2\sqrt{f'_c}\, h d$, ACI 11.10 $\longrightarrow$ $d = 0.8\,l_w$

$= 2\sqrt{5000}\,(8'')(0.8)(31.5')(12''/')(\frac{1}{1000})$

$= 342.12^k$

$V_n = V_c + V_s$

$\phi V_c = 342.12^k > V_u = 291.2^k$ $\quad\phi V_n > V_u$

∴ Minimal STEEL is allowed

FOR 8" WALLS, use #4 bars @ 10" o.c.

- WHEN CHECKING SHEAR STRENGTH IN STORIES 2 THROUGH 4 V_u CONTINUES TO DECREASE WHILE ϕV_c REMAINS THE SAME. THEREFORE MINIMAL REINFORCEMENT IS ACCEPTABLE AT ALL FLOORS.

- VERTICAL BARS

FLOORS 1 – 3 $\longrightarrow$ USE #4 bars @ 10" o.c.

FLOOR 4 $\longrightarrow$ USE #3 bars @ 10" o.c.

- HORIZONTAL BARS

FLOORS 1–3 $\longrightarrow$ USE #4 bars @ 10" o.c.

FLOOR 4 $\longrightarrow$ USE #4 bars @ 12" o.c.

(VALUES TAKEN FROM TABLE 6-4 OF PCA SIMPLIFIED DESIGN – REINFORCED CONCRETE BUILDINGS OF MODERATE SIZE & HT.)

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CHECK FLEXURE DESIGN

$U = 0.9D + 1.6S$

NON-LOAD BEARING WALL ⇒ TRIBUTARY WIDTH = 0

$DL_{SELF\ WT.} = (150\,pcf)(8)(31.5')(1/12)(52') = 163.8^k$

1ST FLOOR: $P_u = 0.9(163.8^k) = 147.42^k$

$M_u = 1.6((21.47)(52) + (42.68)(39) + (59.17)(26) + (58.7)(13'))$

$= 8132^{'k}$

2ND FL: $P_u = 0.9(122.85) = 110.6^k$

$M_u = 1.6((21.47)(39) + (42.68)(26) + (59.17)(13))$

$= 4346^{'k}$

3RD FL: $P_u = 0.9(81.9) = 73.7^k$

$M_u = 1.6(1113.06) = 1781^{'k}$

4TH FL: $P_u = 0.9(40.95) = 36.9^k$

$M_u = 1.6(279.11) = 447^{'k}$

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MOMENT FROM REQ'D VERTICAL REINFORCEMENT FROM SHEAR.

STRENGTH OF MOMENT @ 1ST FLOOR: $\#4 \rightarrow A_s = 0.2 \, in^2$

$$A_{ST} = ((0.2)(1/10")(12"))(31.5') = 7.56 \, in^2$$

$$w = \left(\frac{A_{ST}}{l_w h}\right)\frac{f_y}{f'_c} = \left(\frac{7.56}{31.5(12)(8)}\right)\left(\frac{60}{5}\right) = 0.03$$

$$a = \frac{P_u}{l_w h f'_c} = \frac{147.42}{(31.5)(12)(8)(5)} = 0.00975$$

$$\frac{c}{l_w} = \frac{w + a}{2w + 0.85b_1} = \frac{0.03 + 0.00975}{2(0.03) + 0.85(0.8)} = 0.0537$$

$$\phi M_n = \phi\left(0.5 A_{ST} f_y l_w \left(1 + \frac{P_u}{A_{ST} f_y}\right)\left(1 - \frac{c}{l_w}\right)\right)$$

$$= 0.9\left(0.5(7.56)(60)(31.5)(12)\left(1 + \frac{147.42}{(7.56)(60)}\right)\left(1 - 0.0537\right)\right)$$

$$= 96743 \, "k \qquad = 8062 \, 'k$$

$$\phi M_n = 8062 \, 'k < M_u = 8132 \, 'k$$

INCREASE BAR SIZE $\longrightarrow$ USE #5 bars @ 10" o.c.

$\#5 \rightarrow A_s = 0.31 \, in^2$

$$A_{ST} = (0.31)(1/10")(12")(31.5') = 11.72 \, in^2$$

$w = 0.0465$

$a = 0.00975$

$\frac{c}{l_w} = 0.076$

$$\phi M_n = 133694 \, "k = 11,141 \, 'k$$

$$\phi M_n = 11,141 \, 'k > M_u = 8,132 \, 'k \quad \therefore \text{ ok}$$

use #5 @ 10" o.c

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FOR MOMENT STRENGTHS @ FLOORS 2 → 4, FOLLOW SAME PROCEDURE

2ND FL: # 4 bars @ 10"

$\phi M_n = 7594$ 'k $>$ $M_u = 4346$ 'k ∴ ok

3RD FL: # 4 bars @ 10"

$\phi M_n = 7122$ 'k $>$ $M_u = 1781$ 'k ∴ ok

4TH FL: # 3 bars @ 10"

$\phi M_n = 3952$ 'k $>$ $M_u = 447$ 'k ∴ ok

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OVERTURNING MOMENT CHECK

$M_{O.T.} = 21.47^k(52') + 42.68^k(39') + 59.17^k(26') + 58.7^k(13')$
$= 5669.5^{'k}$

$M_{R\,WALL} = 150\,pcf\,(8/12)(31.5')(52')(31.5'/2)$
$= 2579.9$

COLUMN: TRIB AREA $= (31.5')(7.5') = 236.25\ ft^2$

$P_{COLUMN} = 236.25((4)(126)) = 119.07^k$

$$\frac{M_{OT} - M_R}{l_w} = \frac{5669.5 - 2579.9}{31.5'} = 98.1^k = T$$

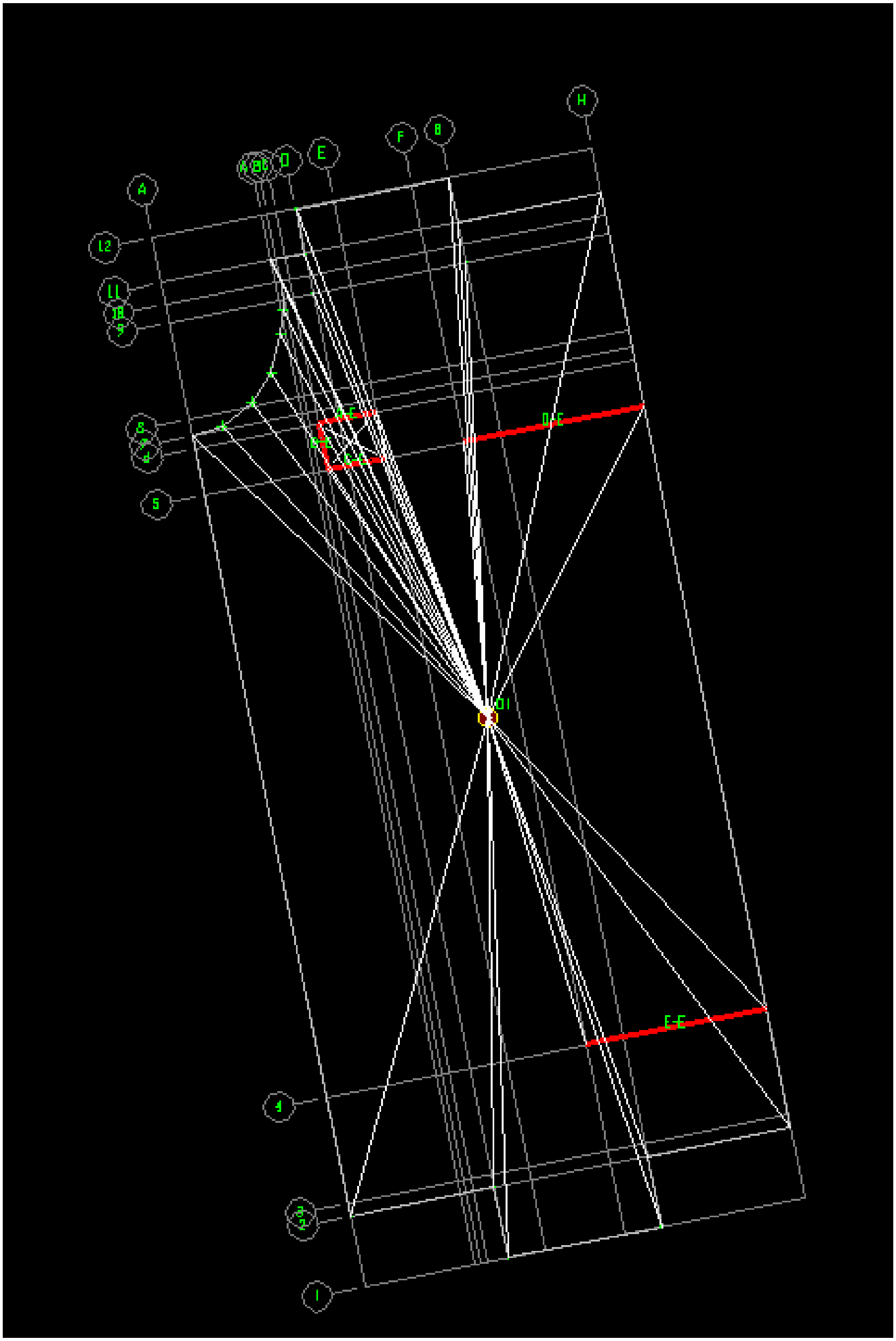
$P_{COLUMN} = 119.07^k > T = 98.1^k \therefore$ NO OVERTURNING

MAX COLUMN FORCE $= 119.07 + 98.1 = 217.17^k$

Appendix C

ETABS Display Screen Example
East Wing

Center of Rigidity - ETABS



Example Display of Shear Forces on East Wing Shear Walls - ETABS