GOUVERNEUR HEALTHCARE SERVICES

NEW YORK, NY

TECHNICAL REPORT 1

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Structural

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EXECUTIVE SUMMARY

The Gouverneur Health Services was designed using the NYC Building Code. For the purpose of this report, existing conditions were analyzed using the loads provided in ASCE 7-05. Wind loads were analyzed using Chapter 6 of ASCE 7-05 and values were obtained that were significantly higher than the NYC Building Code. For the controlling case, wind in the East-West direction, a base shear of 671 kips was obtained and an overturning moment of 51100 ft-k was calculated. Seismic Loads were calculated using Chapter 12 of ASCE 7-05 with a base shear of 116 kips and an overturning moment of 10050ft-k, but these loads was unable to be compared to the loads specified in the NYC Building Code.

Spot checks were performed on selected members from a representative bay. Moment capacities for castellated beams were determined to be adequate for minimum loads, while other criteria was checked using a design aid from CMC Steel Products and was determined to be adequate. Capacities for regular W-shapes were adequate, which was expected. Their capacity is assumed to be controlled by lateral loads.

INTRODUCTION

The Gouverneur Health Services Modernization Project is an addition to an existing building and a renovation of the 35-year-old healthcare facility. The existing building is a 2-way flat plate floor construction with square and rectangular columns. An existing conditions survey revealed no shear-walls, so it can be assumed that lateral loads are resisting by the continuous frame construction of the flat plate slab. For the purpose of this technical report, and subsequent thesis project, only the addition will be investigated in further detail. Furthermore, portions of the addition that wrap around the existing building and tie into the existing structure will be neglected for this technical report.

The addition that will be the main focus of this thesis project consists of two distinct portions. The first portion is the 5-story ambulatory care facility. This facility is approximately 115’x175’ in plan, and sits on the western side of the site, connected to the existing building. The second portion is an expansion to the floor plan to the existing building in floors 6 through 13. It is roughly square, 50’x60’ in plan, and extends upwards from the ambulatory center on the western side of the existing building. The portions may be referred to as lower addition and upper addition, or ambulatory addition and tower addition, respectively. See Figures below.

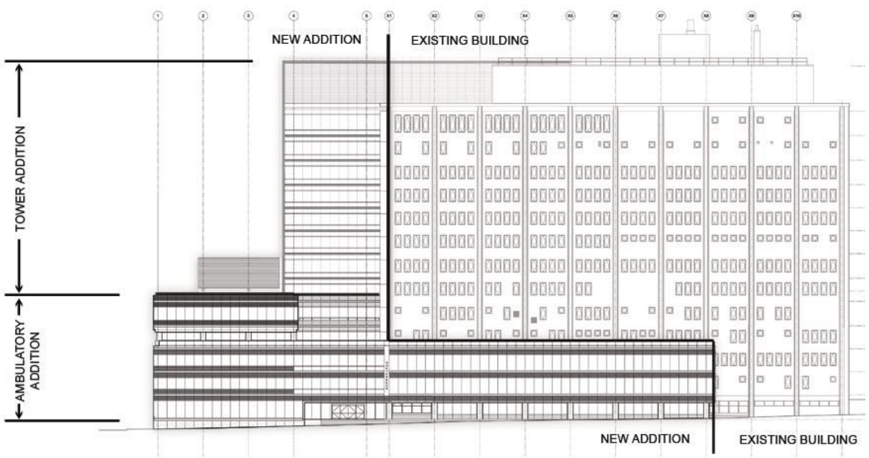

Fig 1. Gouverneur Layout Schematic

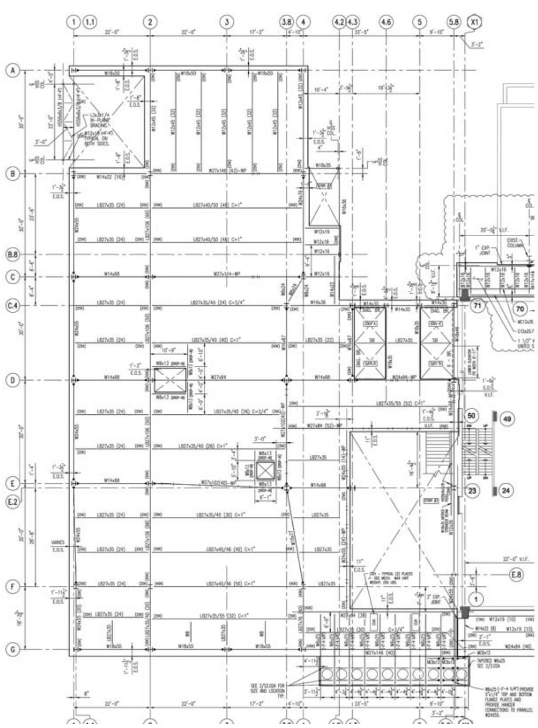

Fig 2. Typical Ambulatory Center Framing Plan

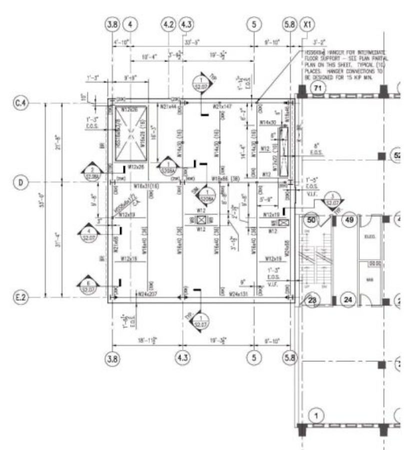

Fig 3. Typical Tower Addition Framing Plan

STRUCTURAL SYSTEM

Foundation

The Gouverneur Healthcare Facility bears on a pile foundation system, with 60-ton capacity, 12" piles. Pile caps vary from 35" to 54" thick with the number of piles ranging from 2 to 16 piles per cap. The footprint for the cellar is smaller than the extents of the overall building so the depths of the pile caps vary. The depths of the caps are either 4'-6" below datum if the columns terminate in the cellar, or 16'-9" above datum if the columns terminate on the first floor.

The piles support grade beams that span between 15' and 40'. Their sizes range from 4'-0" to 8'-3" deep with reinforcing bars from #8 to #12 bars. A structural, one-way slab-on-grade spans between grade beams to make up the cellar floor.

Floor System

The floor system for Gouverneur Healthcare Services is a composite system that utilizes castellated beams for all gravity beams in the ambulatory addition. A 4 ¼" slab rests on a 2" LOK floor composite deck, and is tied to the beam with 5" long, ¾" diameter shear studs. Typical bays are 30'-0" by 44'-0" and almost all beams are nominally 27" deep to accommodate mechanical systems. The tower addition uses traditional W-shapes in a composite floor system. Beams are W16's in areas where clearance for mechanical equipment is not an issue, and W14's where clearance is an issue.

Columns

Almost all columns in the Gouverneur Healthcare Services Building are W14 columns, regardless if it is a part of the lateral system or just a gravity column. Sizes range from W14x43 to W14x257, and are continuous from the foundation to the roof, with only column bearing on a transfer girder on the seventh floor. Columns are spliced on every other floor starting on the third floor. Base plates are typically 22" x 22" with bolts ranging in size from ¾" to 2".

Lateral System

Due to the vast use of glass curtain walls and irregular plan between floors, most of the lateral system in the Gouverneur Healthcare Services Building is moment resisting frames. For the interior moment frames, sizes are either W27's for long span beams or W14's for the shorter spans. Most beams in exterior moment frames are W18's and W24's. In the tower portion of the building, lateral loads are resisted by exterior moment frames in the East-West direction, and braced frames in the North-South direction, both concentric and eccentric. Most braced frames are continuous from the roof to the column termination at the foundation. But at the interface of the upper addition and the lower addition, where one frame is discontinuous, loads transfer into columns in the floor below, and redistribute through the structure.

Wind loads transfer from curtain wall system to floor diaphragm. Floor diaphragm is rigid compared to structure so loads transfer to lateral frames based off of relative stiffness.

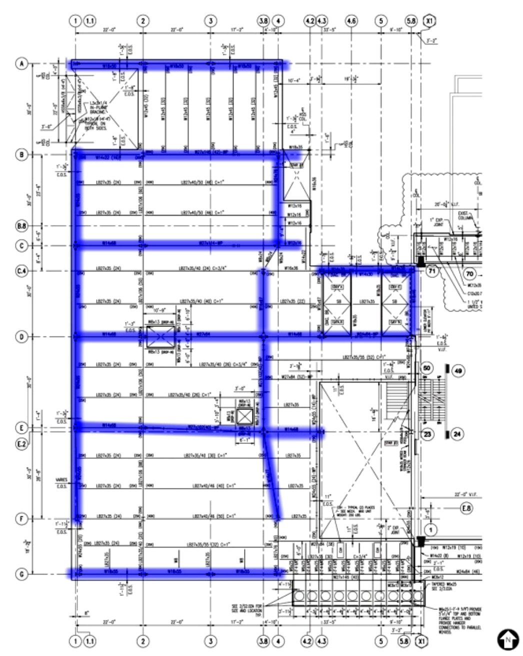

Fig 4. Typical Framing Plan Showing Moment Frames

MATERIALS

	ASTM	Min Strength
Concrete	ASTM	Min Strength
Structural slab-on-grade	-	3000 psi
Pile cap	-	4000 psi
Retaining walls	-	4000 psi
Interior Slabs	-	4000 psi
Reinforcing Steel	A615	60ksi
Structural Steel		
Structural Tubing	A500	46 ksi
Steel Pipe	A53	35 ksi
Rolled Shapes	A992	50 ksi
Other Rolled Plates	A36	36 ksi
Connection Bolts	A325	90 ksi
Anchor Bolts	A307	45 ksi

APPLICABLE CODES AND DESIGN REQUIREMENTS

Codes and References

The City of New York Building and Administrative Code
New York Electrical Code
All Applicable NFPA Codes
New York State Energy Code
AIA Guidelines for Design and Construction of Hospital and Health Care Facilities

Deflection Criteria

Floor Deflection L/240 Total and L/360 Live
Lateral Deflection
Total Drift 3 ½” (due to expansion joint between addition and existing building)
Story Drift H/400

DESIGN LOADS

Dead Load (psf)	
Floor Load	
3 1/4" LW concrete fill on 3" LOK-Floor	60
Ceiling	2
Floor Finish	2
Mech/Elect	10
Partitions	12
Steel Framing	13
TOTAL	99
	(psf)

Wall assemblies	
1. Metal Panel	25
2. Glass Curtainwall	15
GFRC	40
	(psf)

Dead Load (psf)	
Penthouse Roof	
Steel	8
Deck/Insulation	8
Mechanical	10
Membrane	2
Fire Proofing	2
TOTAL	30
	(psf)

Main Roof	
3 1/4" LW concrete fill on 3" LOK-Floor	60
Ceiling	3
Mech/Elect	14
Roofing/Insulation	9
TOTAL	86
	(psf)

Live Load (psf)		
Live Load	As Designed	As per ASCE7
Dormatory Floors	40	40
Lobby	100	100
Lounge	100	100
Corridor 1st Floor	100	100
Corridor above 1st	80	80
Stairs	100	100
Mechanical Rooms	150	-
Main Roof (Mech)	150	-

Fig 5. Design Load Tables

LATERAL LOADS

Wind Analysis

Lateral loads were calculated as per the provisions described in Chapter 6 of ASCE-7. Appendix A provides expanded tables used in the calculation of wind loads. Analysis was performed using a wind speed of 110mph, an importance factor of 1.15 and a directionality factor of 0.85.

Small irregularities in the footprint were neglected; however, the building was split into two zones vertically to account for the dramatic change in size. The change in dimensions between the ambulatory portion and the tower portion of the building was drastic enough to change gust factors, as evident in the jump in total pressure between the lower roof higher segments. Other portions of the building, including the mechanical screen wall on the lower roof, were neglected for the purpose of this technical assignment.

		Height	Kz,Kh	Windward p_z	Leeward p_h	Total	NYC Code	Total	Overturning
		(ft)		(psf)	(psf)	(psf)	(psf)	(kip)	(ft-k)
Zone 2	Parapet	***175.66***	1.16	52.72	-35.15	87.87	25.00	17.7	3080
	Upper Roof	***172.16***	1.15	38.92	-14.10	53.02	25.00	37.1	6164
		160.00	1.13	38.24	-14.10	52.34	25.00	60.3	9043
		140.00	1.09	37.04	-14.10	51.14	25.00	58.9	7658
		120.00	1.04	35.72	-14.10	49.82	25.00	57.4	6312
		100.00	0.99	34.22	-14.10	48.32	25.00	27.8	2644
		90.00	0.96	33.40	-14.10	47.50	25.00	27.4	2325
		80.00	0.93	32.50	-14.10	46.60	25.00	26.8	2013
		70.00	0.89	31.52	-14.10	45.62	25.00	26.3	1708
		60.00	0.85	30.43	-14.10	44.53	25.00	6.3	369
Zone 1	Lower Roof	***57.55***	0.84	26.64	-9.53	36.17	25.00	47.3	2544
		50.00	0.81	25.78	-9.53	35.31	25.00	61.2	2752
		40.00	0.76	24.49	-9.53	34.02	25.00	58.9	2062
		30.00	0.70	22.94	-9.53	32.47	25.00	28.1	773
		25.00	0.67	22.02	-9.53	31.55	25.00	27.3	615
		20.00	0.62	20.96	-9.53	30.49	25.00	26.4	462
		15.00	0.57	19.69	-9.53	29.22	25.00	75.9	569
						Base Shear	334.9	**671.2**	**51094**
							(kip)	(kip)	(ft-k)

	Story
Zone 2	parapet
	upper roof
	main roof
	13
	12
	11
	10
	9
	8
	7
Zone 1	6
	5
	4
	3
	2
	Ground

Story Force	Story Shear
(kip)	(kip)
17.7	17.7
33.4	51.1
51.1	102.3
35.6	137.8
35.3	173.1
33.3	206.4
31.9	238.3
30.7	269.1
31.4	300.5
30.7	331.2
52.1	383.3
71.1	454.4
65.7	520.1
61.4	581.5
59.2	640.7
30.3	671.0
671.0	Total

Fig 6. Wind East-West Tables

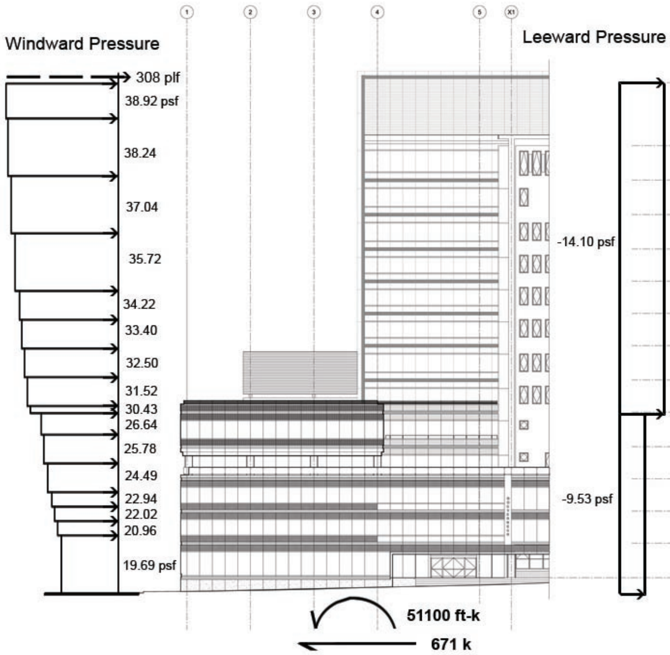

Fig 7. Wind East-West Pressures

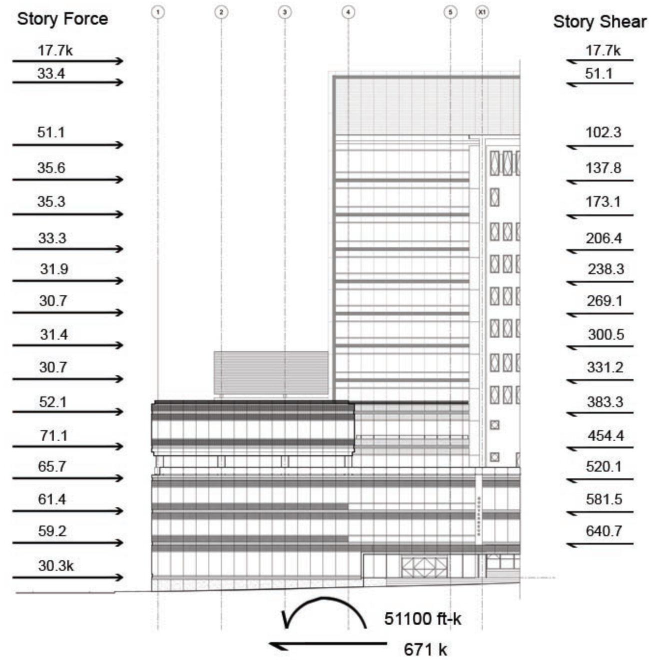

Fig 8. Wind East-West Story Forces

Wind in the East-West direction was determined to be the controlling case for all lateral loads. The base shear was calculated to be 671k and the overturning moment was calculated to be 51100ft-k. These values appear to be high but could be due to the importance factor and the high wind-speed conditions of New York City.

		Height	Kz,Kh	Windward p_z	Leeward p_h	Total	NYC Code	Total	Overturning
		(ft)		(psf)	(psf)	(psf)	(psf)	(kip)	(ft-k)
Zone 2	Parapet	***175.66***	1.16	52.72	-35.15	87.87	25.00	17.7	3080
	Upper Roof	***172.16***	1.15	39.18	-14.27	53.45	25.00	33.4	5545
		160.00	1.13	38.50	-14.27	52.77	25.00	54.2	8134
		140.00	1.09	37.30	-14.27	51.56	25.00	53.0	6888
		120.00	1.04	35.96	-14.27	50.23	25.00	51.6	5678
		100.00	0.99	34.45	-14.27	48.72	25.00	25.0	2378
		90.00	0.96	33.62	-14.27	47.89	25.00	24.6	2091
		80.00	0.93	32.72	-14.27	46.98	25.00	24.1	1810
		70.00	0.89	30.30	-11.03	41.33	25.00	21.2	1380
		60.00	0.85	29.20	-11.03	40.24	25.00	5.1	297
Zone 1	Lower Roof	***57.55***	0.84	28.26	-7.51	35.77	25.00	31.0	1664
		50.00	0.81	27.34	-7.51	34.85	25.00	39.9	1797
		40.00	0.76	25.95	-7.51	33.46	25.00	38.3	1342
		30.00	0.70	24.29	-7.51	31.79	25.00	18.2	501
		25.00	0.67	23.30	-7.51	30.81	25.00	17.7	397
		20.00	0.62	22.16	-7.51	29.67	25.00	17.0	297
		15.00	0.57	20.80	-7.51	28.30	25.00	48.6	365
						Base Shear	316.6	**520.7**	**43646**
							(kip)	(kip)	(ft-k)

	Floor	Story Force (kip)	Story Shear (kip)
Zone 2	parapet	15.8	15.8
	upper roof	30.1	45.9
	main roof	46.0	91.8
	13	32.0	123.9
	12	31.7	155.6
	11	29.9	185.5
	10	28.7	214.2
	9	27.7	241.9
	8	28.3	270.2
	7	26.0	296.2
Zone 1	6	36.3	332.5
	5	46.4	378.9
	4	42.7	421.6
	3	39.6	461.3
	2	38.0	499.3
	Ground	19.4	518.7
		518.7	Total

Fig 9. Wind North-South Tables

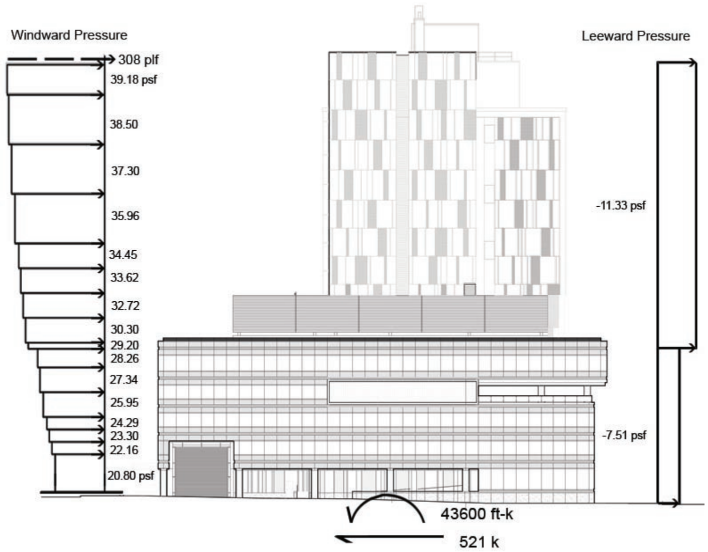

Fig 10. Wind North-South Pressures

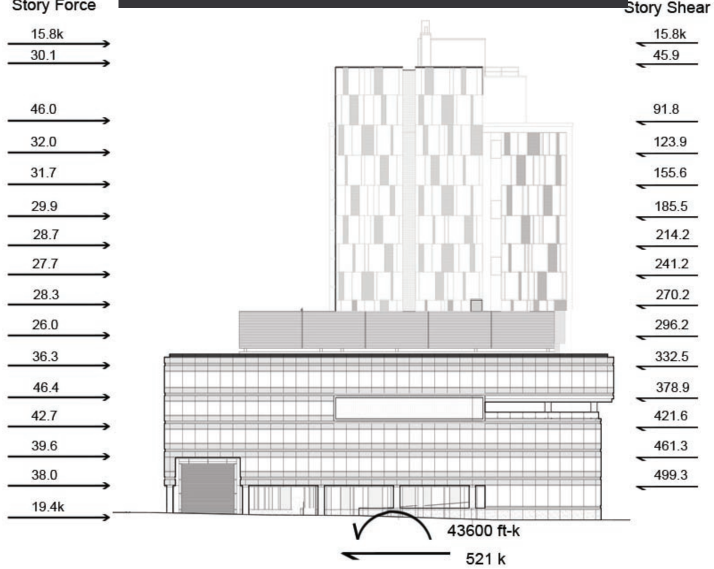

Fig 11. Wind North-South Story Forces

For wind in the North-South direction, the base shear is calculated to be 521k and the overturning moment is 43600ft-k. Values for wind loads in this direction are smaller due the change in dimensions from one face to the other – the area loaded by the wind pressure is smaller.

Seismic Analysis

Seismic forces were calculated using the Equivalent Lateral Force Method as described in ASCE-7, and response coefficients were determined by inputting the site latitude and longitude into the USGS Earthquake Ground Motion Parameter Application. All seismic coefficients used can be found in the appendix.

Seismic forces did not control the lateral design by a large margin. The base shear was calculated to be 115.7k and the overturning moment was determined to be 10050ft-k, significantly less than the controlling wind condition. Forces were not able to be compared to the NYC Building Code, although it would be safe to assume

Story	Floor Height	Floor Weight	$w_ih^k_i$	C_{vx}	Story Force	Story Shear	Moment Contribution
Upper Roof	174.2	134.344	23397	0.039	4.517	4.517	786.722
Main Roof	152.26	367.8	56009	0.094	10.814	15.331	1646.526
13	140.3	343.4	48178	0.081	9.302	24.633	1304.898
12	128.30	343.4	44064	0.074	8.508	33.141	1091.553
11	116.3	343.4	39950	0.067	7.713	40.854	897.239
10	105.13	343.4	36104	0.061	6.971	47.825	732.807
9	93.9	343.4	32258	0.054	6.228	54.053	585.004
8	82.73	343.4	28413	0.048	5.486	59.539	453.832
7	70.8	343.4	24299	0.041	4.691	64.231	331.918
6	59.55	1438.5	85664	0.144	16.539	80.770	984.958
5	47.6	1450.3	68994	0.116	13.321	94.091	633.713
4	36.38	1447.4	52649	0.088	10.165	104.256	369.760
3	25.2	1447.4	36441	0.061	7.036	111.292	177.143
2	13.98	1450.3	20274	0.034	3.914	115.206	54.719
Ground	2.0	1380.5	2761	0.005	0.533	115.739	1.066
	(ft)	(kip)			115.74		10051.9
					Base Shear		**Overturning**

Fig 12. Seismic Design Table

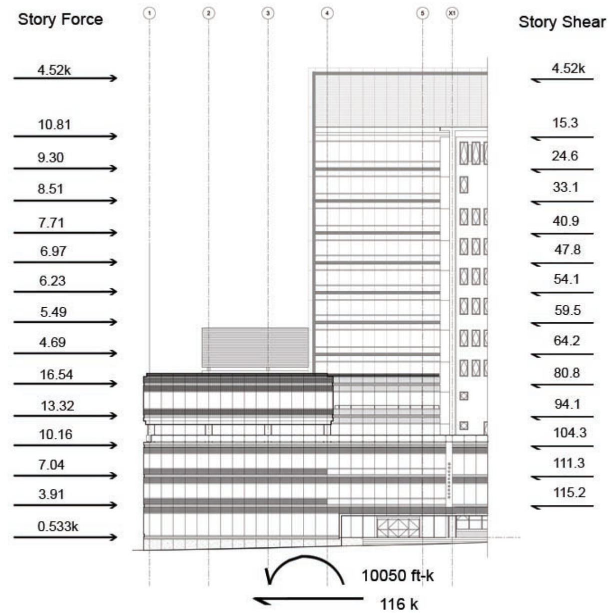

Fig 13. Seismic Story Forces

Spot Checks

Sport checks were performed on a representative bay to analyze the strength and serviceability of the members. Composite castellated beams were analyzed by modeling an approximate cross section and using stress relationships to find the full plastic moment capacities. To be conservative, the web of the member was neglected and the live load was not reduced unless unexpected values were obtained. Further analysis was performed using a CMC Steel Products design tool in order to determine capacities due to other failure modes and deflection limits.

W-shapes that were part of the lateral system were only analyzed in order to investigate moment capacity since gravity loading was expected to not control. All members were stronger by a margin that was large enough to make it obvious that lateral loads controlled the design of these members and it was not necessary to continue analysis.

CONCLUSIONS

The design solutions used in the Gouverneur Healthcare Services Modernization project reflect the need to match floor-to-floor heights of the existing concrete building. For a variety of reasons, the project team determined steel framing to be the most desirable structural system. A unique floor framing was developed as is evident in the extensive use of deep, castellated beams to allow for the MEP systems. In this technical report, the existing conditions of the addition to the existing building were investigated. Gravity loads were determined using ASCE 7-05 and lateral loads for wind and seismic conditions were calculated using chapter 6 and 12 respectively. Then, spot checks were completed to investigate the design.

Lateral loads were controlled by wind in the East-West direction. Without design checks for the main wind force resisting system, it was hard to gauge the accuracy of the calculations. However, upon comparing the values obtained using the analytical method to the current NYC Building Code, certain issues were evident; the loads acquired using ASCE 7 are significantly higher than the values specified by the code used in the original design of the building. Two possible conclusions for this disparity are that either mistakes were made in the analysis of the structure, or the analytical method is conservative in New York City. One way to determine the validity of the latter conclusion is to perform a wind tunnel test, which is outside the possible scope of this thesis report. The analytical method will be investigated in further detail in future technical reports to determine if mistakes were made in some of the simplifying assumptions used in the calculations.

Spot checks were performed on the composite, castellated beams to briefly investigate their capacity. The cross section was simplified into rectangles in order to calculate the plastic moment capacity through (the rule) tension=compression and plastic stress distribution. The web was neglected to be conservative and simplify the analysis. Through the analysis, it became evident that bending either controlled member design, or a shape was chosen to satisfy the depth requirements. This was the case of the LB27x46 found in the representative bay, the lightest 27” deep castellated beam provided by CMC Steel Products. When necessary, further analysis of the castellated beams was performed using the design guide provided by CMC. W-shapes in the moment-frame, lateral system were analyzed by investigating their moment capacity. Strengths exceeded the required gravity loading significantly, even while conservatively assuming a simply supported beam while the beams were actually fully restrained. This proved that lateral loads controlled the design and shear and deflection calculations were deemed unnecessary to perform. Along the same lines, a preliminary gravity analysis was conducted on a typical column. Analysis was conservative in that the live load was not reduced, so it became evident that the lateral loads controlled the design of the column as expected.

APPENDIX A LIST OF FIGURES

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	Wind E-W Zone 1		Wind E-W Zone 2	
B	173.23		57.59	
L	114.58		51.38	
h	57.55		172.16	
□	0.01		0.01	
$n_1=22.2/H^{0.8}$	0.3611		0.3611	
$g_Q=g_V=$	3.4		3.4	
$g_R=$	3.94		3.94	
$\bar{z}=$	34.53		103.30	
$I_z=$	0.2977		0.2480	
$L_z=$	324.87		468.10	
Q=	0.8144		0.8444	
$V_z=$	73.43		96.57	
$N_1=$	1.60		1.75	
$R_n=$	0.1016		0.0964	
$R_h=$	0.4949	□= 1.302	0.2808	□= 2.961
$R_B=$	0.2226	□= 3.919	0.5702	□= 0.991
$R_L=$	0.1086	□= 8.677	0.2810	□= 2.959
R=	0.8066		1.0110	
$G_f=$	1.0651		1.167	
$GC_{pi}=$	0.18		0.18	
	-0.18		-0.18	
	Windward	Leeward	Windward	Leeward
Cp	0.80	-0.50	Cp 0.80	-0.50
L/B=	0.66		L/B= 0.89	

Wind E-W

		Height	Kz,Kh	q_z	q_h	$q_zG_fC_p$	$q_hG_fC_p$	$q_h(GC_{pi})$	Windward p_z	Leeward p_h	Total	NYC Code	Total	Overturning
		(ft)							(psf)	(psf)	(psf)	(psf)	(kip)	(ft-k)
Zone 2	Parapet	175.66	1.16	35.15	35.15	52.72	-35.15	0.00	52.72	-35.15	87.87	25.00	17.7	3080
	Upper Roof	172.16	1.15	34.95	34.95	32.62	-20.39	6.29	38.92	-14.10	53.02	25.00	37.1	6164
		160.00	1.13	34.22	34.95	31.95	-20.39	6.29	38.24	-14.10	52.34	25.00	60.3	9043
		140.00	1.09	32.94	34.95	30.75	-20.39	6.29	37.04	-14.10	51.14	25.00	58.9	7658
		120.00	1.04	31.52	34.95	29.43	-20.39	6.29	35.72	-14.10	49.82	25.00	57.4	6312
		100.00	0.99	29.92	34.95	27.93	-20.39	6.29	34.22	-14.10	48.32	25.00	27.8	2644
		90.00	0.96	29.04	34.95	27.11	-20.39	6.29	33.40	-14.10	47.50	25.00	27.4	2325
		80.00	0.93	28.07	34.95	26.21	-20.39	6.29	32.50	-14.10	46.60	25.00	26.8	2013
		70.00	0.89	27.02	34.95	25.23	-20.39	6.29	31.52	-14.10	45.62	25.00	26.3	1708
		60.00	0.85	25.86	34.95	24.14	-20.39	6.29	30.43	-14.10	44.53	25.00	6.3	369
Zone 1	Lower Roof	57.55	0.84	25.55	27.02	21.77	-14.39	4.86	26.64	-9.53	36.17	25.00	47.3	2544
		50.00	0.81	24.55	27.02	20.92	-14.39	4.86	25.78	-9.53	35.31	25.00	61.2	2752
		40.00	0.76	23.03	27.02	19.62	-14.39	4.86	24.49	-9.53	34.02	25.00	58.9	2062
		30.00	0.70	21.21	27.02	18.08	-14.39	4.86	22.94	-9.53	32.47	25.00	28.1	773
		25.00	0.67	20.14	27.02	17.16	-14.39	4.86	22.02	-9.53	31.55	25.00	27.3	615
		20.00	0.62	18.89	27.02	16.10	-14.39	4.86	20.96	-9.53	30.49	25.00	26.4	462
		15.00	0.57	17.40	27.02	14.83	-14.39	4.86	19.69	-9.53	29.22	25.00	75.9	569
											Base Shear	334.9	**671.2**	**51094**
												(kip)	(kip)	(ft-k)

	Story	Floor Elev.	Elev. above datum	Story Height	Story Force	Story Shear
		(ft)	(ft)	(ft)	(kip)	(kip)
Zone 2	parapet	175.66	196.41	3.50	17.7	17.7
	upper roof	172.16	192.91	21.90	33.4	51.1
	main roof	150.26	171.01	11.98	51.1	102.3
	13	138.28	159.03	11.98	35.6	137.8
	12	126.30	147.05	11.98	35.3	173.1
	11	114.32	135.07	11.20	33.3	206.4
	10	103.13	123.88	11.20	31.9	238.3
	9	91.93	112.68	11.20	30.7	269.1
	8	80.73	101.48	11.98	31.4	300.5
	7	68.75	89.50	11.20	30.7	331.2
Zone 1	6	57.55	78.30	11.98	52.1	383.3
	5	45.57	66.32	11.20	71.1	454.4
	4	34.38	55.13	11.20	65.7	520.1
	3	23.18	43.93	11.20	61.4	581.5
	2	11.98	32.73	11.98	59.2	640.7
	Ground	0.00	20.75	0.00	30.3	671.0
		Datum	20.75		**671.0**	Total

	Wind N-S Zone 1		Wind N-S Zone 2	
B	114.58		51.38	
L	173.23		57.59	
h	57.55		172.16	
□	0.01		0.01	
$n_1=22.2/H^{0.8}$	0.3611		0.3611	
$g_Q=g_V=$	3.4		3.4	
$g_R=$	3.94		3.94	
$\bar{z}=$	34.53		103.30	
$I_z=$	0.2977		0.2480	
$L_z=$	324.87		468.10	
Q=	0.8385		0.8465	
$V_z=$	73.43		96.57	
$N_1=$	1.60		1.75	
$R_n=$	0.1016		0.0964	
$R_h=$	0.4949	□= 1.302	0.2808	□= 2.961
$R_B=$	0.3118	□= 2.592	0.6007	□= 0.884
$R_L=$	0.0733	□= 13.119	0.2561	□= 3.316
R=	0.9408		1.0284	
$G_f=$	1.1445		1.177	
$GC_{pi}=$	0.18		0.18	
	-0.18		-0.18	
	Windward	Leeward	Windward	Leeward
Cp	0.80	-0.40	0.80	-0.50
L/B=	1.51		1.12	

Wind N-S

		Height	Kz,Kh	q_z	q_h	$q_zG_fC_p$	$q_hG_fC_p$	$q_h(GC_{pi})$	Windward p_z	Leeward p_h	Total	NYC Code	Total	Overturning
		(ft)							(psf)	(psf)	(psf)	(psf)	(kip)	(ft-k)
Zone 2	Parapet	175.66	1.16	35.15	35.15	52.72	-35.15	0.00	52.72	-35.15	87.87	25.00	17.7	3080
	Upper Roof	172.16	1.15	34.95	34.95	32.89	-20.56	6.29	39.18	-14.27	53.45	25.00	33.4	5545
		160.00	1.13	34.22	34.95	32.21	-20.56	6.29	38.50	-14.27	52.77	25.00	54.2	8134
		140.00	1.09	32.94	34.95	31.01	-20.56	6.29	37.30	-14.27	51.56	25.00	53.0	6888
		120.00	1.04	31.52	34.95	29.67	-20.56	6.29	35.96	-14.27	50.23	25.00	51.6	5678
		100.00	0.99	29.92	34.95	28.16	-20.56	6.29	34.45	-14.27	48.72	25.00	25.0	2378
		90.00	0.96	29.04	34.95	27.33	-20.56	6.29	33.62	-14.27	47.89	25.00	24.6	2091
		80.00	0.93	28.07	34.95	26.42	-20.56	6.29	32.72	-14.27	46.98	25.00	24.1	1810
		70.00	0.89	27.02	27.02	25.44	-15.90	4.86	30.30	-11.03	41.33	25.00	21.2	1380
		60.00	0.85	25.86	27.02	24.34	-15.90	4.86	29.20	-11.03	40.24	25.00	5.1	297
Zone 1	Lower Roof	57.55	0.84	25.55	27.02	23.40	-12.37	4.86	28.26	-7.51	35.77	25.00	31.0	1664
		50.00	0.81	24.55	27.02	22.48	-12.37	4.86	27.34	-7.51	34.85	25.00	39.9	1797
		40.00	0.76	23.03	27.02	21.09	-12.37	4.86	25.95	-7.51	33.46	25.00	38.3	1342
		30.00	0.70	21.21	27.02	19.42	-12.37	4.86	24.29	-7.51	31.79	25.00	18.2	501
		25.00	0.67	20.14	27.02	18.44	-12.37	4.86	23.30	-7.51	30.81	25.00	17.7	397
		20.00	0.62	18.89	27.02	17.30	-12.37	4.86	22.16	-7.51	29.67	25.00	17.0	297
		15.00	0.57	17.40	27.02	15.93	-12.37	4.86	20.80	-7.51	28.30	25.00	48.6	365
											Base Shear	316.6	**520.7**	**43646**
												(kip)	(kip)	(ft-k)

	Floor	Floor Elev.	Elev. above datum	Story Height	Story Force	Story Shear
		(ft)	(ft)	(ft)	(kip)	(kip)
Zone 2	parapet	175.66	196.41	3.50	15.8	15.8
	upper roof	172.16	192.91	21.90	30.1	45.9
	main roof	150.26	171.01	11.98	46.0	91.8
	13	138.28	159.03	11.98	32.0	123.9
	12	126.30	147.05	11.98	31.7	155.6
	11	114.32	135.07	11.20	29.9	185.5
	10	103.13	123.88	11.20	28.7	214.2
	9	91.93	112.68	11.20	27.7	241.9
	8	80.73	101.48	11.98	28.3	270.2
	7	68.75	89.50	11.20	26.0	296.2
Zone 1	6	57.55	78.30	11.98	36.3	332.5
	5	45.57	66.32	11.20	46.4	378.9
	4	34.38	55.13	11.20	42.7	421.6
	3	23.18	43.93	11.20	39.6	461.3
	2	11.98	32.73	11.98	38.0	499.3
	Ground	0.00	20.75	0.00	19.4	518.7
		Datum	20.75		**518.7**	Total

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Seismic Loading

General Information

				SOURCE	SECTION/TABLE PAGE
	Occupancy Type		IV	ASCE7-05	Table 1-1 (page 3)
	Occupancy Importance Factor		1.15		
	Site Class		B	Seismic Tool	
	Seismic Design Category		B	ASCE 7-05	Table 11.6-1 116
	Height Above Grade [ft]	h_n	172.16		
	Short Period Spectral Response	S_S	0.363	Seismic Tool	
	Spectral Response at 1 Second	S_1	0.070	Seismic Tool	
	Maximum Short Period Spectral Reponse	S_{MS}	0.363	Seismic Tool	
	Maximum Spectral Reponse at 1 Second	S_{M1}	0.070	Seismic Tool	
	Design Short Period Spectral Response	S_{DS}	0.242	Seismic Tool	
	Design Spectral Response at 1 Second	S_{D1}	0.047	Seismic Tool	
	Period Parameter 1	C_t	0.028	ASCE 7-05	Table 12.8-2 129
	Period Parameter 2	x	0.8	ASCE 7-05	Table 12.8-2 129
	Response Modification Coefficient	R	3.5	ASCE 7-05	Section 12.8.1.1 120 > 129
Ta=Ct*hn^x	Approximate Fundamental Period	T_a	1.721	ASCE 7-05	Section 12.8.2.1 129
Can use Ta in lieu of T	Fundamental Period	T		ASCE 7-05	Section 12.8.2
	Long-Period Transition Period	T_L	6.000	ASCE 7-05	see Section 11.4 > 229
	Short-Period Transition Period	T_S	0.194		
Cs = Sds/(R/I)	Seismic Response Coefficient	C_S	0.080	ASCE 7-05	Section 12.8 129
Cs = Sd1/(T*R/I)	Maximum Required Cs Value	$C_{S.max}$	0.009	ASCE 7-05	Section 12.8.1.1 129
	Max Cs per ASCE7-12.8.1.1	C_S	0.01		
	Effective Weight	**W**	11520.62377		
	Base Shear	V	115.21	ASCE 7-05	Section 12.8 129
	Overturning Moment	M	10051.9		

Story	Floor Height	Floor Weight	$w_i h_i^k$	C_{vx}	Story Force	Story Shear	Moment Contribution
Upper Roof	174.2	134.344	23397	0.039	4.517	4.517	786.722
Main Roof	152.26	367.8	56009	0.094	10.814	15.331	1646.526
13	140.3	343.4	48178	0.081	9.302	24.633	1304.898
12	128.30	343.4	44064	0.074	8.508	33.141	1091.553
11	116.3	343.4	39950	0.067	7.713	40.854	897.239
10	105.13	343.4	36104	0.061	6.971	47.825	732.807
9	93.9	343.4	32258	0.054	6.228	54.053	585.004
8	82.73	343.4	28413	0.048	5.486	59.539	453.832
7	70.8	343.4	24299	0.041	4.691	64.231	331.918
6	59.55	1438.5	85664	0.144	16.539	80.770	984.958
5	47.6	1450.3	68994	0.116	13.321	94.091	633.713
4	36.38	1447.4	52649	0.088	10.165	104.256	369.760
3	25.2	1447.4	36441	0.061	7.036	111.292	177.143
2	13.98	1450.3	20274	0.034	3.914	115.206	54.719
Ground	2.0	1380.5	2761	0.005	0.533	115.739	1.066
	(ft)	(kip)			115.74		10051.9
					Base Shear		**Overturning**

1st Floor		
Floor		
	area	13500
	weight	99
Wall		
	length	490
total	height	5.99
assembly1	height	0.00
	weight	25
assembly2	height	5.99
	weight	15
	TOTAL	1380.527

2nd Floor		
Floor		
	area	13500
	weight	99
Wall		
	length	490
total	height	11.59
assembly1	height	5.83
	weight	25
assembly2	height	5.76
	weight	15
	TOTAL	1450.27

3rd Floor		
Floor		
	area	13500
	weight	99
Wall		
	length	490
total	height	11.2
assembly1	height	5.83
	weight	25
assembly2	height	5.37
	weight	15
	TOTAL	1447.403

4th Floor		
Floor		
	area	13500
	weight	99
Wall		
	length	490
total	height	11.2
assembly1	height	5.83
	weight	25
assembly2	height	5.37
	weight	15
	TOTAL	1447.403

5th Floor		
Floor		
	area	13500
	weight	99
Wall		
	length	490
total	height	11.59
assembly1	height	5.83
	weight	25
assembly2	height	5.76
	weight	15
	TOTAL	1450.27

6th Floor		
Floor		
	area	13500
	weight	99
Wall	below	
total	length1	490
total	height1	5.99
assembly1	height	5.83
	weight	25
assembly2	height	0.16
	weight	15
Wall	above	
total	height	5.99
curtainwall	length	115.16
assembly1	height	5.83
	weight	25
assembly2	height	0.16
	weight	15
GFRC	length	51.38
	height	5.99
	weight	40
	TOTAL	1438.47

7th Floor		
Floor		
	area	2958.46
	weight	99
Wall		
total	height	11.59
curtainwall	length	115.16
assembly1	height	5.83
	weight	25
assembly2	height	5.76
	weight	15
GFRC	length	51.38
	height	11.59
	weight	40
	TOTAL	343.442

8th Floor		
	TOTAL	343.442
9thFloor		
	TOTAL	343.442
10th Floor		
	TOTAL	343.442
11th Floor		
	TOTAL	343.442
12th Floor		
	TOTAL	343.442
13th Floor		
	TOTAL	343.442

Main Roof		
Floor		
	area	2958.46
	weight	99
Wall	below	
total	height	5.99
curtainwall	length	115.16
assembly1	height	5.83
	weight	25
assembly2	height	0.16
	weight	15
GFRC	length	51.38
	height	5.99
	weight	40
Wall	above	
total	length	166.54
total	height	10.95
	weight	25
	Total	367.85

Main Roof		
Floor		
	area	2958.46
	weight	30
Wall		
total	length	166.54
total	height	10.95
	weight	25
	Total	134.34

Total Building Weight: (kips)

11521

Typical Story Weight of Steel

count				**length**	**Steel Beam Weight**
5	W	18 x 50	@	22.0 =	5500
11	W	12 x 45	@	30.0 =	14850
1	W	12 x 45	@	18.2 =	820.3125
1	W	14 x 22	@	22.0 =	484
1	W	27 x 146	@	44.0 =	6424
4	LB	27 x 46	@	44.0 =	8096
5	LB	27 x 46	@	39.0 =	8970
1	W	27 x 114	@	44.0 =	5016
1	W	27 x 94	@	39.0 =	3666
1	LB	27 x 50	@	44.0 =	2200
10	LB	27 x 35	@	22.0 =	7700
3	W	14 x 48	@	22.0 =	3168
4	LB	27 x 106	@	30.0 =	12720
1	W	24 x 55	@	18.2 =	1002.604
3	LB	27 x 35	@	18.2 =	1914.063
1	w	24 x 76	@	30.0 =	2280
1	w	16 x 67	@	30.0 =	2010
1	w	16 x 89	@	30.0 =	2670
1	w	16 x 77	@	30.0 =	2310
5	lb	27 x 35	@	15.5 =	2712.5
1	w	27 x 114	@	15.5 =	1767
1	w	14 x 68	@	15.5 =	1054
2	lb	27 x 35	@	48.1 =	3365.833
2	lb	27 x 76	@	48.1 =	7308.667
1	w	24 x 55	@	48.1 =	2644.583
1	lb	27 x 56	@	15.0 =	840
1	lb	27 x 56	@	35.0 =	1960
1	w	21 x 44	@	18.2	802.0833
1	w	12 x 26	@	25	650
1	w	14 x 68	@	30.0	2040
1	w	18 x 46	@	21.83333	1004.333
					117950 k
					Column Weight
18	w	14 x 120	**@**	11	23760
3	w	14 x 90	**@**	11	2970
1	w	14 x 193	**@**	11	2123
1	w	14 x 257	**@**	11	2827
1	w	14 x 132	**@**	11	1452
2	w	14 x 145	**@**	11	3190
2	w	14 x 159	**@**	11	3498
2	w	14 x 176	**@**	11	3872
1	w	14 x 211	**@**	11	2321
1	w	14 x 342	**@**	11	3762
1	w	14 x 109	**@**	11	1199
					50974 k
				floor area:	13500 sf
				Steel area Load:	**12.51 psf**

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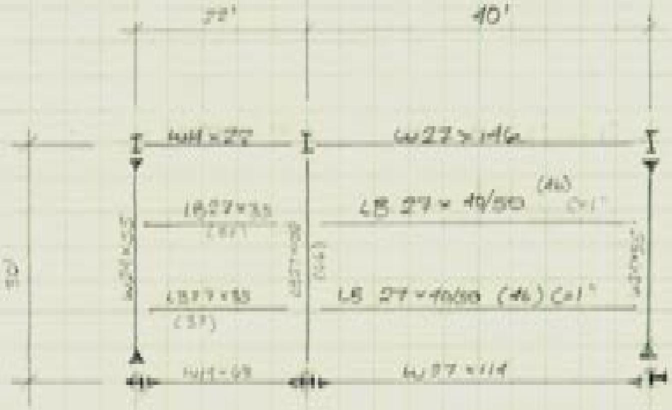
Spot Check – Gravity Analysis

Representative Bay – 3rd Floor



– Live loads NOT reduced
– self weight neglected

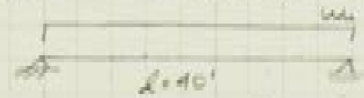
LOADS:

w_D = 86 psf DNI. S.W.

w_L = 100 psf

↳ framing typical of whole floor, w/ all types of loading

— Analyze LB 27×46 w/ 52 steel studs ¾" × 5" studs

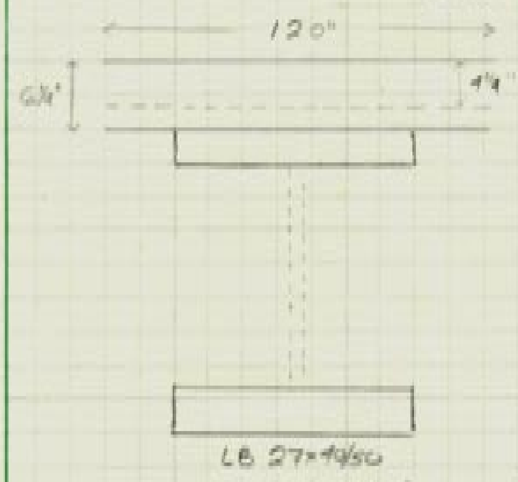


Note: web is neglected in A_{se} calculations

$w_u = 1.2w_D + 1.6w_L$ = 263 psf ←

$M_u = \frac{w\ell^2}{8} = (263 \times 10)(40)^2/8$

= 526 ft-k



$b_{eff} \le$ spacing/2 = $\frac{10' \times 12}{2}$ = 60" ← controls

span/8 = $\frac{40 \times 12}{8}$ = 60"

d = 27.39"

top b_f = 6.02, t_f = 0.525

b_f = 7.495, t_f = 0.570 bot

q_n = 21.2 AISC MANUAL TABLE 3-21

Q_n = 21.2(46/2) = 488 k

NOTE: neglect web to be conservative
– web openings will effect strength but will be able to transfer shear

Scott M. Rabold

Tension Capacity Steel / Compression Capacity Concrete

$T_s = A_s f_y = [(6.02)(0.525) + (7.495)(0.570)](50)$

$= 372\ k \longleftarrow$ controls

$C_c = 0.85 f'_c\, bt = 0.85(4)(120)(4.25)$

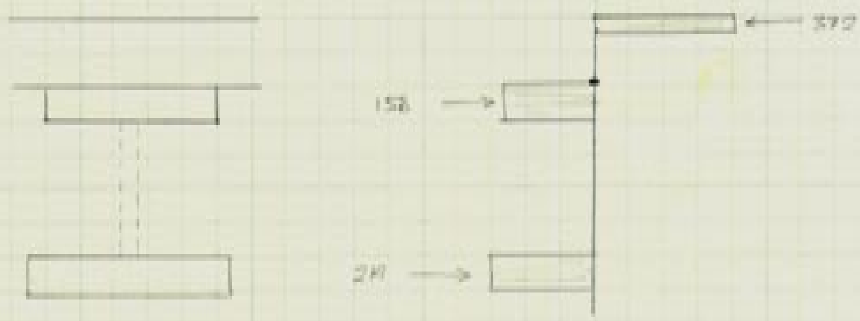
$= 1734\ k$

$t_c = 291 / 0.85 f'_c b = 291 / 0.85(4)(120)$

$= 0.713''$

$T_{s_{bot}} = A_s f_y = (7.495)(0.570)(50) = 214\ k$

$T_{s\,top} = A_s f_y = (6.02)(0.525)(50) = 158\ k$



take M about T.O. Steel

$M_n = [214(27.39 - \tfrac{1}{2}(0.57)) + 158(\tfrac{1}{2}\,0.525) + 372(6.25 - \tfrac{1}{2}(0.713))] / 12$

$= 670\ ft\text{-}k$

$\phi M_n = 0.9(670) = 603\ ft\text{-}k$

$\phi M_n = 603 > M_u = 526 \quad \therefore$ OK

SmartBeam™

Composite Castellated Design Program

CASTELLATED BEAM INFORMATION

Job Name	Sample Project	
Beam Mark #	lb	
Span	40.000	ft
Spac. Left	10.000	ft
Spac. Right	10.000	ft
Mat. Strength-Fy	50	ksi
Round Duct Diam.	15.036	in
Duct W x H	8.500 in	14.882 in
Castellated Beam	CB27X40/50	
Root Beams (T/B)	W18X40	W18X50
d	17.9	17.99
bf	6.015	7.495
tf	0.525	0.57
tw	0.315	0.355

CASTELLATION PARAMETERS:

e	7.000	in
b	5.500	in
dt	4.250	in
S	25.000	in
dg	27.390	in
phi	59.787	deg
ho	18.890	in
wo	18.000	in

CMC Steel Products

LOADING INFORMATION

Uniform Distributed Loads

Live Load	1000	plf	Pre-comp %	0%
Dead Load	860	plf	Pre-comp %	80%

Concentrated Point Loads

Load # (#)	Magnitude (kips)	Dist from Lft. End (ft)	Percent DL (%)	Percent Pre-Comp.
P1	0.00	10.00	0%	0%
P2	0.00	20.00	0%	0%
P3	0.00	0.00	0%	0%
P4	0.00	0.00	0%	0%

COMPOSITE INFORMATION

Concrete & Deck:		Shear Studs:	
conc. strength - fc' (psi)	4000	stud dia. (in)	3/4"
conc. wt. - wc (pcf)	115	stud ht. (in)	5
conc. above deck - tc (in)	4 1/4	studs per rib	1
rib height - hr (in)	2	composite %	100%
rib width - wr (in)	2	Stud Spacing:	
		N=44,Uniformly Dist.	

RESULTS

Failure Mode	Interaction	Status
Bending	**0.811**	<=1.0 OK!!
Web Post	**0.984**	<=1.0 OK!!
Shear	**0.920**	<=1.0 OK!!
Concrete	**0.252**	<=1.0 OK!!
Pre-Comp.	0.625	<=1.0 OK!!
Overall	**0.984**	**<=1.0 OK!!**

Pre-Composite Deflec.	0.930"	=L/516
Live Load Deflection	0.414"	=L/1158

WARNINGS

EXPAND'D. SXN. PROP'S

Avg. wt.	45.0	plf
Anet	9.987	in^2
Agross	16.317	in^2
Ix net	1643.92	in^4
Ix gross	1838.70	in^4
Sx net	107.22	in^3
Sx gross	124.19	in^3
rx min	10.62	in
Iy	29.55	in^4
Sy	9.83	in^3

COMPOSITE SXN. PROP'S

n	11.76	
beffec.	120.00	in
Actr	43.376	in^2
N.A. ht.	28.05	In Deck
Itr	4792.17	in^4
Ieffec.	4792.17	in^3
Sxconc	856.71	in^3
Sxsteel	170.87	in^3

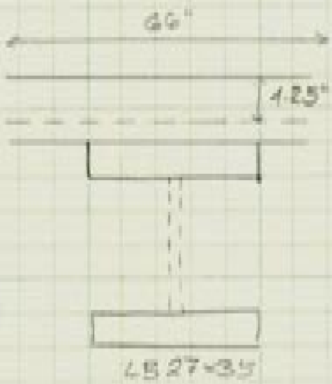
CONSTRUCTION BRIDGING

End Connection type	Double clip
Min. No. Of Bridging Rows	0
Max. Bridging. Spacing (ft)	41

9/26/2008

— LB 27×35   32 studs   3/4" × 5"

$b_{eff} \le \begin{cases} \frac{spacing}{2} = 60' \\ \frac{span}{8} = \frac{22\times 12}{8} = 33" \end{cases}$ ← controls



$d = 26.9"$
$t_f = 0.125"$
$b_f = 6"$

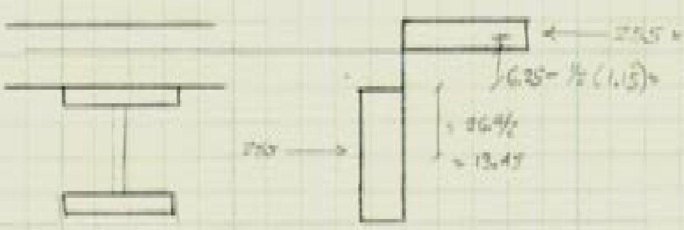
$M_u = wl^2/8 = (263\times 10)(22)^2/8$
$= 159$ ft-k

Steel / Conc. CAPACITY

$T_s = A_s F_y = 2(6)(0.125)(50)$
$= 255$ k ← controls

$C_c = 0.85 f'_c\, b\, t = 0.85(4)(66)(4.25)$
$= 954$ k

$Q_n = 16(21.2) = 339$ $t_{req} = 255/0.85(4)(66)$
$= 1.15"$



$M_n = 255\,(13.45 + 5.675) = 488$ ft-k

$\phi M_n = 0.9(488) = 439$ ft-k > M_u = ft-k ∴ OK in Bending

— Bending does not control
 — see CMC excel

— well overdesigned for strength
— lightest LB27 — used for depth restrictions

SmartBeam™

Composite Castellated Design Program

CASTELLATED BEAM INFORMATION

Job Name	Sample Project	
Beam Mark #	lb	
Span	22.000	ft
Spac. Left	10.000	ft
Spac. Right	10.000	ft
Mat. Strength-Fy	50	ksi
Round Duct Diam.	14.950	in
Duct W x H	8.500 in	14.555 in
Castellated Beam	CB27X35	
Root Beams (T/B)	W18X35	W18X35
d	17.7	17.7
bf	6	6
tf	0.425	0.425
tw	0.3	0.3

CASTELLATION PARAMETERS:

e	7.000	in
b	5.500	in
dt	4.250	in
S	25.000	in
dg	26.900	in
phi	59.128	deg
ho	18.400	in
wo	18.000	in

CMC Steel Products

LOADING INFORMATION

Uniform Distributed Loads

Live Load	1000	plf	Pre-comp %	0%
Dead Load	860	plf	Pre-comp %	80%

Concentrated Point Loads

Load # (#)	Magnitude (kips)	Dist from Lft. End (ft)	Percent DL (%)	Percent Pre-Comp.
P1	0.00	0.00	0%	0%
P2	0.00	0.00	0%	0%
P3	0.00	0.00	0%	0%
P4	0.00	0.00	0%	0%

COMPOSITE INFORMATION

Concrete & Deck:		Shear Studs:	
conc. strength - fc' (psi)	4000	stud dia. (in)	3/4"
conc. wt. - wc (pcf)	115	stud ht. (in)	5
conc. above deck - tc (in)	4 1/2	studs per rib	1
rib height - hr (in)	1 1/2	composite %	100%
rib width - wr (in)	1 1/2	Stud Spacing:	
		N=34,Uniformly Dist.	

RESULTS

Failure Mode	Interaction	Status
Bending	**0.380**	<=1.0 OK!!
Web Post	**0.561**	<=1.0 OK!!
Shear	**0.545**	<=1.0 OK!!
Concrete	**0.132**	<=1.0 OK!!
Pre-Comp.	0.314	<=1.0 OK!!
Overall	**0.561**	**<=1.0 OK!!**

Pre-Composite Deflec.	0.130"	=L/2035
Live Load Deflection	0.062"	=L/4283

WARNINGS

EXPAND'D. SXN. PROP'S

Avg. wt.	35.0	plf
Anet	7.472	in^2
Agross	12.992	in^2
Ix net	1192.93	in^4
Ix gross	1348.67	in^4
Sx net	88.69	in^3
Sx gross	100.27	in^3
rx min	10.19	in
Iy	15.33	in^4
Sy	5.11	in^3

COMPOSITE SXN. PROP'S

n	11.76	
beffec.	66.00	in
Actr	25.260	in^2
N.A. ht.	26.91	In Deck
Itr	2948.84	in^4
Ieffec.	2948.84	in^3
Sxconc	492.55	in^3
Sxsteel	109.57	in^3

CONSTRUCTION BRIDGING

End Connection type	Double clip
Min. No. Of Bridging Rows	0
Max. Bridging. Spacing (ft)	36

9/25/2008

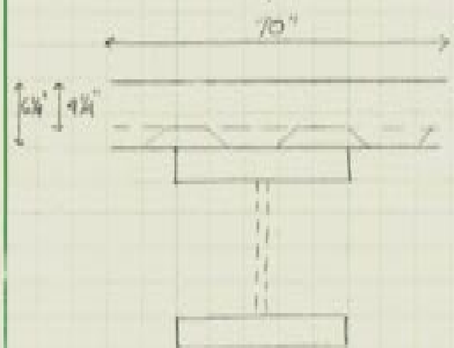
— LB 27 × 106

$P = 263\,psf \times (10)(11+20) = 81.53\ k$

$M_n = P \times 10' = (81.53)(10)$

$= 815\ ft\text{-}k$

$b_{eff_1} \le \left| \begin{array}{l} \frac{22'}{2} = 132' \\ \frac{30'}{8} = 45'' \leftarrow \end{array}\right.$ $b_{eff_2} \le \left| \begin{array}{l} \frac{40}{2} = 240'' \\ 45'' \leftarrow \end{array}\right.$



LB 27 × 106

$d = 28.095''$

$b_f = 11.2''$

$t_f = 0.94''$

$q_n = 18.3$

$Q_n = 18.3 \times 33 = 604\ k$

$T_s = A_s f_y = 2(11.2)(0.94)(50)$

$= 1053\ k$

$C_c = 0.85 f'_c\, b t = 0.85(4)(90)(4\,1/4)$

$= 1301\ k$

— shear studs control

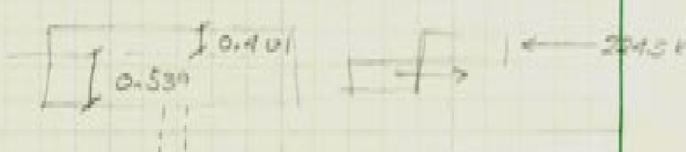
Compression Steel

$1053 - x = 604 + x$

$C_{s_{top}} = x = 224.5\ k$

t_{s_c} $224.5 = 11.2(t)(50)$

$t = 0.401''$



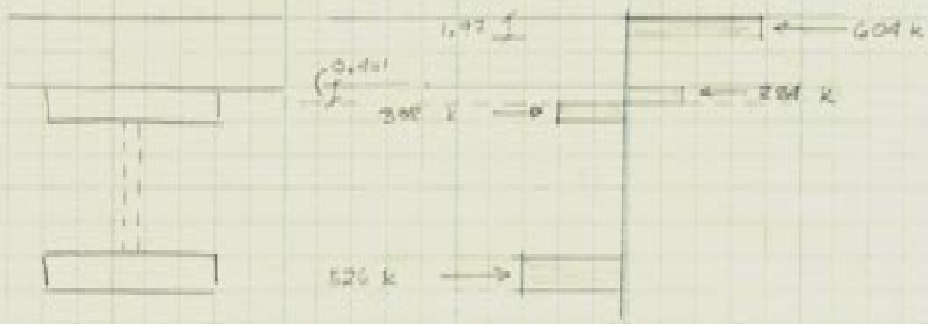
$T_{s_{top}}$ $A_s f_y = (0.539)(11.2)(50)$

$= 301.8$

$T_{s_{Bot}}$ $A_s f_y = (0.94)(11.2)(50) = 526.4$

M_n — $t_{conc} = 604 / 0.85(4)(90)$

$= 1.97"$



Moment about T.O. steel

$$M_n = \left[604\left(6\tfrac{1}{4} - \tfrac{1}{2}(1.97)\right) - 224\left(\tfrac{1}{2}(0.141)\right) + 302\left(0.91 - \tfrac{1}{2}(0.539)\right) + 526\left(28.095 - \tfrac{1}{2}(0.91)\right)\right] / 12$$

$= 1497$ ft-k

$\phi M_n = 1347$ ft-k — analysis is off, see CMC sheet

— then RAM beam output for reduced LL

— then CMC sheet for accurate output.

SmartBeam™

Composite Castellated Design Program

CASTELLATED BEAM INFORMATION

Job Name	Sample Project	
Beam Mark #	lb	
Span	30.000	ft
Spac. Left	22.000	ft
Spac. Right	40.000	ft
Mat. Strength-Fy	50	ksi
Round Duct Diam.	15.793	in
Duct W x H	8.907 in	15.793 in
Castellated Beam	CB27X106	
Root Beams (T/B)	W18X106	W18X106
d	18.73	18.73
bf	11.2	11.2
tf	0.94	0.94
tw	0.59	0.59

CASTELLATION PARAMETERS:

e	8.000	in
b	5.407	in
dt	4.683	in
S	26.814	in
dg	28.095	in
phi	60.000	deg
ho	18.730	in
wo	18.814	in

CMC Steel Products

LOADING INFORMATION

Uniform Distributed Loads

Live Load	0	plf	Pre-comp %	0%
Dead Load	0	plf	Pre-comp %	80%

Concentrated Point Loads

Load # (#)	Magnitude (kips)	Dist from Lft. End (ft)	Percent DL (%)	Percent Pre-Comp.
P1	85.00	10.00	60%	0%
P2	85.00	20.00	60%	0%
P3	0.00	0.00	0%	0%
P4	0.00	0.00	0%	0%

COMPOSITE INFORMATION

Concrete & Deck:		Shear Studs:	
conc. strength - fc' (psi)	4000	stud dia. (in)	3/4"
conc. wt. - wc (pcf)	115	stud ht. (in)	5
conc. above deck - tc (in)	4 1/4	studs per rib	1
rib height - hr (in)	2	composite %	100%
rib width - wr (in)	2	Stud Spacing:	
		40 | 32 | 40	

RESULTS

Failure Mode	Interaction	Status
Bending	1.797	>1.0, NG!!
Web Post	1.118	>1.0, NG!!
Shear	**0.996**	<=1.0 OK!!
Concrete	**0.536**	<=1.0 OK!!
Pre-Comp.	0.024	<=1.0 OK!!
Overall	1.797	**>1.0, NG!!**

WARNINGS

Pre-Composite Deflec.	0.014"	=L/25504
Live Load Deflection	0.210"	=L/1712

EXPAND'D. SXN. PROP'S

Avg. wt.	106.0	plf
Anet	25.549	in^2
Agross	36.600	in^2
Ix net	4459.04	in^4
Ix gross	4782.10	in^4
Sx net	317.43	in^3
Sx gross	340.42	in^3
rx min	11.43	in
Iy	220.24	in^4
Sy	39.33	in^3

COMPOSITE SXN. PROP'S

n	11.76	
beffec.	90.00	in
Actr	32.532	in^2
N.A. ht.	24.23	In Steel
Itr	9233.90	in^4
Ieffec.	9233.90	in^3
Sxconc	912.54	in^3
Sxsteel	381.15	in^3

CONSTRUCTION BRIDGING

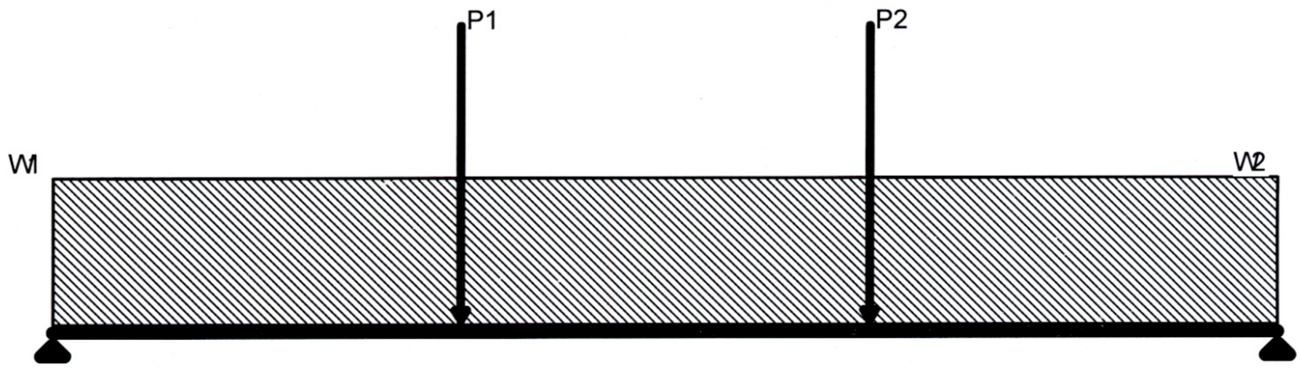
End Connection type	Double clip
Min. No. Of Bridging Rows	0
Max. Bridging. Spacing (ft)	57

9/26/2008

Load Diagram

RAM Steel v11.2
DataBase:
Building Code: IBC

Floor Type: 2nd_Floor **Beam Number = 340**
Span information (ft): I-End (-89.25,95.33) J-End (-89.25,125.33)



Load	Dist	DL	LL+	LL-	Max Tot
	ft	kips	kips	kips	kips
P1	10.000	29.757	21.874	0.000	51.631
P2	20.000	29.757	21.874	0.000	51.631
	ft	k/ft	k/ft	k/ft	k/ft
W1	0.000	0.106	0.000	0.000	0.106
W2	30.000	0.106	0.000	0.000	0.106

SmartBeam™

Composite Castellated Design Program

CASTELLATED BEAM INFORMATION

Job Name	Sample Project	
Beam Mark #	lb	
Span	30.000	ft
Spac. Left	22.000	ft
Spac. Right	40.000	ft
Mat. Strength-Fy	50	ksi
Round Duct Diam.	15.793	in
Duct W x H	8.907 in	15.793 in
Castellated Beam	CB27X106	
Root Beams (T/B)	W18X106	W18X106
d	18.73	18.73
bf	11.2	11.2
tf	0.94	0.94
tw	0.59	0.59

CASTELLATION PARAMETERS:

e	8.000	in
b	5.407	in
dt	4.683	in
S	26.814	in
dg	28.095	in
phi	60.000	deg
ho	18.730	in
wo	18.814	in

CMC Steel Products

LOADING INFORMATION

Uniform Distributed Loads

Live Load	0	plf	Pre-comp %	0%
Dead Load	0	plf	Pre-comp %	80%

Concentrated Point Loads

Load # (#)	Magnitude (kips)	Dist from Lft. End (ft)	Percent DL (%)	Percent Pre-Comp.
P1	51.63	10.00	60%	0%
P2	51.63	20.00	60%	0%
P3	0.00	0.00	0%	0%
P4	0.00	0.00	0%	0%

COMPOSITE INFORMATION

Concrete & Deck:		Shear Studs:	
conc. strength - fc' (psi)	4000	stud dia. (in)	3/4"
conc. wt. - wc (pcf)	115	stud ht. (in)	5
conc. above deck - tc (in)	4 1/4	studs per rib	1
rib height - hr (in)	2	composite %	100%
rib width - wr (in)	2	Stud Spacing:	
		40 | 32 | 40	

RESULTS

Failure Mode	Interaction	Status
Bending	**0.982**	<=1.0 OK!!
Web Post	**0.687**	<=1.0 OK!!
Shear	**0.612**	<=1.0 OK!!
Concrete	**0.328**	<=1.0 OK!!
Pre-Comp.	0.024	<=1.0 OK!!
Overall	**0.982**	**<=1.0 OK!!**

WARNINGS

Pre-Composite Deflec.	0.014"	=L/25504
Live Load Deflection	0.128"	=L/2819

EXPAND'D. SXN. PROP'S

Avg. wt.	106.0	plf
Anet	25.549	in^2
Agross	36.600	in^2
Ix net	4459.04	in^4
Ix gross	4782.10	in^4
Sx net	317.43	in^3
Sx gross	340.42	in^3
rx min	11.43	in
Iy	220.24	in^4
Sy	39.33	in^3

COMPOSITE SXN. PROP'S

n	11.76	
beffec.	90.00	in
Actr	32.532	in^2
N.A. ht.	24.23	In Steel
Itr	9233.90	in^4
Ieffec.	9233.90	in^3
Sxconc	912.54	in^3
Sxsteel	381.15	in^3

CONSTRUCTION BRIDGING

End Connection type	Double clip
Min. No. Of Bridging Rows	0
Max. Bridging. Spacing (ft)	57

9/26/2008

Check W27 × 114   L = 10'

Gravity ⟹ $M_u = 526$ ft-k $< \phi M_p = 856$ $\therefore$ ok

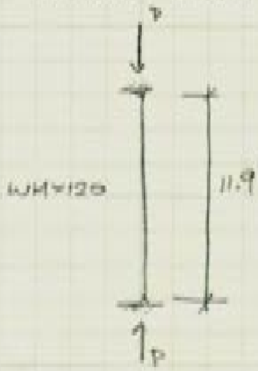
-sized for lateral

Check W14 × 68

Gravity ⟶ $M_u = 159$ ft-k $< \phi M_p = 287$ ft-k $\therefore$ ok

-sized for lateral

Column Axial load Column Below 2^{nd} FLOOR (C2)



Load $w_D = 99$ psf including slab
$w_L = 100$ psf unreduced

Factored

$$w_u = 1.6(100) + 1.2(99)$$
$$= 278.8 \text{ psf}$$

Trib Area per floor

$$A_{T_f} = 30 \times \left(\frac{22}{2} + \frac{40}{2}\right) = 930 \text{ sf}$$

TOTAL

$$A_{T_T} = 5 \text{ floors} \times 930 \text{ sf}$$
$$= 4650 \text{ sf}$$

$$P = w_u \times A_T$$
$$= 0.2788 \times 4650$$

TABLE 4-1 Steel Manual KL=12 $= 1296$ k

$\phi P_n = 1430$ k $> P_u = 1296$ k $\therefore$ OK

- expected due to member being part of lateral system. Analysis was conservative because live load was <u>not</u> reduced.