Technical Report 1

Existing Conditions

STEPS Building
Lehigh University's Asa Packer Campus
Bethlehem, PA

JADOT MARCHMAN-MOOSMAN | STRUCTURAL OPTION

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1. Executive Summary

The purpose of this report is to develop and communicate an understanding the structural system of a building as part of the Penn State Architectural Engineering Department's Capstone Project, also known as Senior Thesis. The building used for this report was the STEPS Building, located on the Lehigh University Campus in Bethlehem, PA.

The report begins with a description of the building structural system. A concrete slab on composite metal deck transfers floor load to wide-flange steel beams. The beams take advantage of composite action with the concrete topping for added strength. Wide-flange steel columns transfer gravity loads to concrete foundation piers. The foundation piers are tied into shallow reinforced concrete footings that ultimately transfer building loads to the ground.

Information and details needed to compute the gravity load requirements of representative members were determined and tabulated. Seismic and wind load inputs were also determined for use in a future analysis of the lateral load resisting system.

Using this information the adequacy of the steel deck and slab was confirmed. A typical beam and column were then re-designed for gravity loading, and the resulting member was compared to the as-built design. In both cases, the existing member had greater capacity than the designed member, and possible reasons for the discrepancy were discussed.

Supporting calculations are also included in appendices to the report.

2. Building Introduction

The Science, Technology, Environment, Policy, Society (STEPS) Building was completed in 2010 as the primary home for the STEPS program at Lehigh University in Bethlehem, PA. The STEPS program aims to bring social sciences, engineering, and hard science activities into spatial proximity to encourage academic collaboration. As a result, the plan contains a mixture of classroom spaces, inter-disciplinary research and teaching laboratories, and faculty offices arranged to integrate the various functions and disciplines.

The four-story "B" wing and five-story "C" wing are steel-frame structures running north-south along the west edge of the site. Flexible moment connections at all column-beam connections provide lateral stability, allowing for an open floor plan well-suited to laboratory, classroom, and graduate office use. A normal weight concrete slab on 3" composite steel deck transfers floor loads to composite beams and girders.

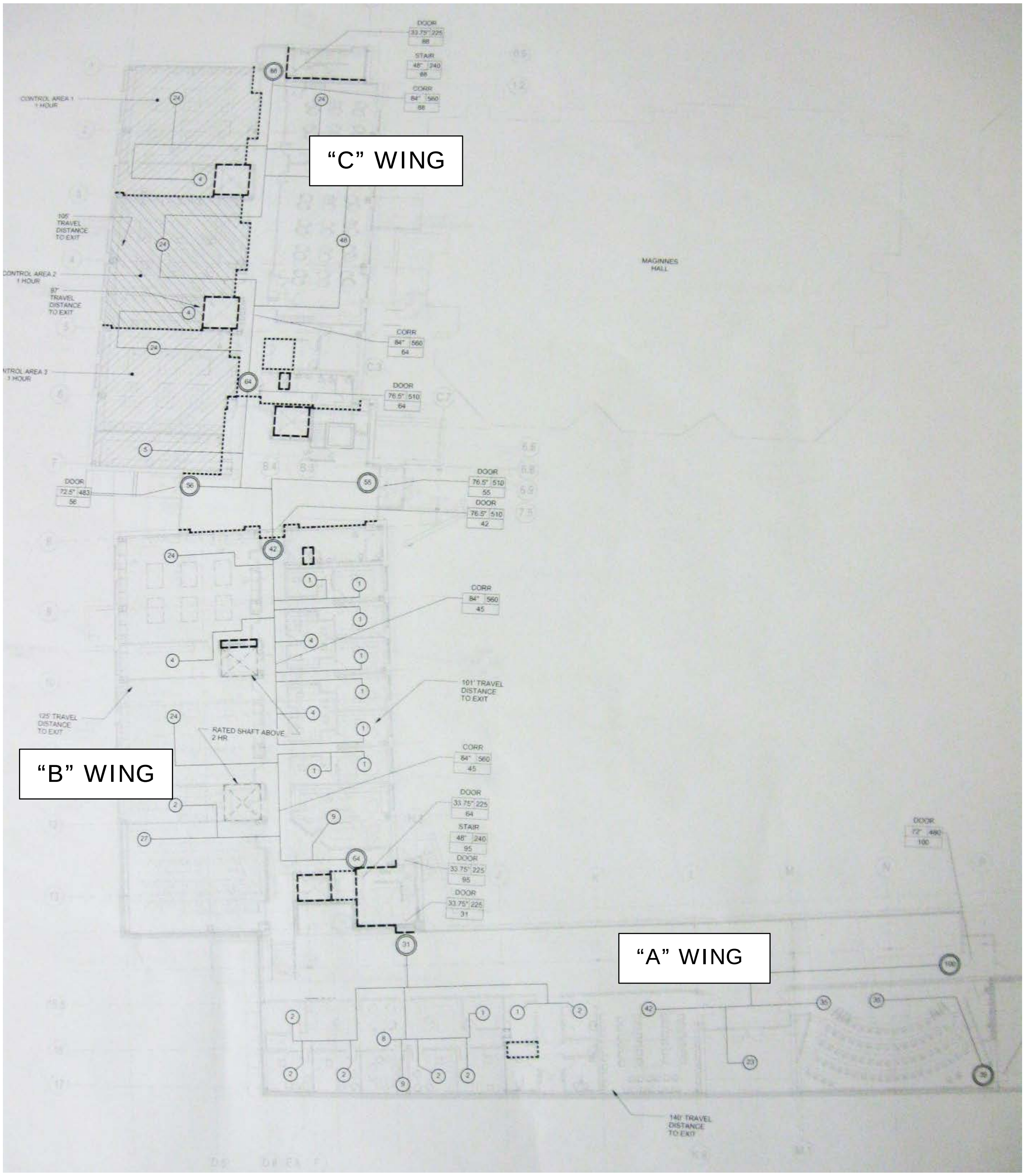
The longitudinal facades are primarily a highly-insulated brick assembly with punch-out style ribbon windows. The transverse facades are almost entirely high-efficiency glazing with rectangular HSS framing, housing student study areas and stairwells.

An atrium with student lounge areas and stairs connects the "B" and "C" wings. For analysis purposes, both wings act together as one structure because the load resisting system continues uninterrupted through the atrium area, and the size of the atrium opening relative to the full diaphragm does not constitute a significant horizontal irregularity that would compromise diaphragm rigidity.

The low-rise "A" wing, which is not investigated in this report, is a one-story steel-frame structure running east to west along the south edge of the site. Its primary features are a 70-seat lecture hall, 12"-deep green roof, extensive glazing, and laminated wood finishing.

The STEPS Building has received LEED Gold certification from the US Green Building Council (USGBC). Sustainable features (including a partial green roof; sunshading and high-efficiency glazing; and custom-sized mechanical systems) were incorporated from the onset of the project to physically embody the STEPS program's forward-looking mission of "collaboration, innovation and scholarship in the areas of science, technology, environment, policy and society."

Figure 1.1 | Site Layout



3. Description of Structural System Components

3.1 Floor System

A composite floor system comprised of a concrete slab with composite steel deck resting on steel framing supports design loads on all above-grade floors in the “B” and “C” wings. Basement floor loads are transferred directly to the soil by a slab-on-grade. In the longitudinal direction, typical girders span 21'-4” and support one transverse beam at mid-span. Transverse beams span from 36'-11” to 42'-8”.

3” 18-gauge composite deck is oriented longitudinally for a clear span of 10'-8”, with the exception of the two bays at the south end of the “B” wing where the deck is oriented transversely. The composite deck is topped with a 4-1/2” normal weight concrete topping, for a total thickness of 7-1/2”, and reinforced with 6”x6” W2.9 X W2.9 welded wire fabric situated 0-3/4” from the top of the slab.

Wide-flange members support the slab-deck floor system and are designed as simply-supported members due to the properties of the flexible moment connections at the columns (see “Lateral System”). Typical sizes for transverse beams are W24x55 and W24x76, with some local variations. Typical longitudinal girders are W21x44. Studs are employed to transfer flexure-induced shear from the slab to the beams and girders, with most beams having between 28 and 36 studs depending on span.

3.2 Vertical Members

Gravity and lateral loads are carried to the foundation by wide-flange columns oriented for strong-axis bending in the transverse direction due to larger surface area and resulting wind loads. Typical bays arranged with three longitudinal column lines, with one at each edge and one near mid-span.

Typical sizes for the main bearing columns in the lateral support system range from W14x90 to W14x132 on levels 3 to 5, and range from W14x109 to W14x192 on the lower floors. Sizes of other columns vary widely by location and purpose. Column lifts are typically three levels – top of pier to level 3, and level 3 to roof level – except on the upper levels of the shorter “B” wing, where lifts are two levels.

3.3 Foundation

Load transfer to bearing soil is provided by shallow reinforced concrete footings. A 2007 geotechnical analysis performed by Schnabel Engineering's West Chester, PA office determined that the existing subgrade material on site had sufficient bearing capacity to support building loads.

Column loads are transferred via base plates to reinforced concrete piers tied into the footings. Exterior columns bear on square footings, with most ranging from 11'-0” to 16'-0” square and 1'-6” to 2'-0” in depth. The interior column line is supported by a mat foundation 18'-0” wide and 3'-0” deep extending the length of the building in the longitudinal direction.

Exterior reinforced concrete foundation walls are supported by strip footings ranging from 2'-0” to 6'-0” in width and 1'-0” to 2'-0” in depth. Foundation walls and piers supporting exterior columns are integrated and cast as one piece. Likewise, the strip footings supporting the foundation walls are integrated with the square footings supporting the exterior columns.

3.4 Roof System

Roof loads are supported by 3” 16-gauge roof deck with a normal weight concrete topping. The topping thickness ranges from 0-1/4” to 4-1/2” to accommodate a 1/4”:1' slope for drainage, for a total slab thickness of 3-1/4” to 7-1/2”. The roof levels are framed very similarly to the floors described above, with typical members in snow-load governed roof areas sized from W24x55 to W24x68.

The roof framing system also supports mechanical equipment in rooftop penthouses, as well as the weight of penthouse square HSS framing and gravity loads transferred from the penthouse roof. The floor system in the mechanical areas matches that of lower floors, with heavier W27x84 shapes.

3.5 Lateral System

Lateral load resistance in both the longitudinal and transverse directions is provided by flexible moment connections at all beam to column connections. The moment frames are continuous to grade, transferring resulting shear and moment to the foundation. Flexible moment connections are sized to resist lateral forces only, and beams are designed as simply-supported members because the moment connections do not have excess capacity to transfer gravity moments to the columns under design lateral loads. Beam webs are connected with angles on each side sized to resist full shear resulting from gravity load. Beam top and bottom flanges are connected with angles to resist moment generated by the lateral load.

Penthouse lateral loads are supported by flexible moment connections at the high roof level in the transverse direction, and by single-angle braced frames designed for tension only in the longitudinal direction. Lateral loads are then transmitted through rigid connections to horizontal roof framing members connected to their supporting columns with flexible moment connections. These beams (typically W27x102) are larger than adjacent members (typically W24x68 or W27x84) to accommodate the additional moment generated by the lateral load.

4. Design Codes

Lehigh University is located within the jurisdiction the City of Bethlehem, which enforces standards as laid out in Pennsylvania Uniform Construction Code (PUCC). The PUCC is modeled on the work of the International Code Council (ICC) and is reviewed and updated triennially. As of the completion of design in 2008, the PUCC 2006 revision was in effect, with key model code components including:

2006 International Building Code,

2006 International Fire Code (only as referenced in IBC 2006),

2006 International Electrical Code,

2006 International Mechanical Code,

2006 International Fuel Gas Code,

and local amendments and requirements as provided for by ordinance.

By reference, the PUCC 2006 also incorporates:

Minimum Design Loads for Buildings and Other Structures (ASCE 7-05),

Building Code Requirements for Structural Concrete (ACI 318-05),

Building Code Requirements for Masonry Structures (ACI 530-05),

AISC Manual of Steel Construction (13th Edition),

and various other requirements specific to individual trades.

The primary codes employed in this report are ASCE 7-05 and the AISC Manual of Steel Construction.

5. Materials

This section provides a list of the major construction materials typically used throughout the existing design for the structural system.

Table 5.1 | Typical Construction Materials for Existing Design

Material	Standard, Strength, and/or Grade
Structural Steel	
W & WT Shapes	ASTM A992 Grade 50
Channels, Angles, & Plates	ASTM A-36
Steel Tubing (Round, Square, & Rectangular)	ASTM
Steel Pipe	ASTM A-53, Grade B
Stainless Steel	ASTM A240 Type 304
Connection Bolts (0-3/4” minimum diameter)	ASTM A325/A490
Shear Studs (0-3/4” round)	ASTM A496
Reinforced Concrete	
Structural Concrete (Footings, Piers, Walls, Slabs)	f'c = 4000 PSI, Normal Weight
Deformed Bars	ASTM A-615 Grade 60
Welded Reinforcing Steel	ASTM A-706 Grade 60
Welded Wire Fabric	ASTM A-185
Metal Deck	
Floors	3” 18 Ga. Galvanized Composite Deck
Roof	3” 16 Ga. Type “NS” Galvanized Roof Deck
Masonry	
CMUs	f'm = 1500 psi
Grout	f'c = 2000 psi

6. Design Gravity Loads

6.1 Floor Live Load

Table 6.1.1 | Code, Existing, and Design Floor Live Load Values

Occupancy	ASCE 7-05 Load (Tables 4-1/C4-1)	Existing Design As Noted on Drawings	Design Load Used for Typical Floors
Offices	50 PSF + 15 PSF (PTN)	50 PSF	125 PSF
Classrooms	40 PSF	40 PSF	
Laboratories	100 PSF	100 PSF	
Laboratory Storage	125 PSF	125 PSF	
Corridors at Ground Level	100 PSF	100 PSF	
Corridors Above Ground Level	80 PSF	80 PSF	
Lobbies	100 PSF	100 PSF	

6.2 Floor Dead Load

Table 6.2.1 | Calculation of Design Floor Dead Load

Item	Dimension	Unit Weight	Load
3” 18 Ga. Composite Deck			2.84 PSF
4-1/2” NW Concrete Topping	0.485 CF/SF	145 PCF	70.3 PSF
Framing Self-Weight Allowance			5 PSF
MEP Allowance			10 PSF
Ceiling Allowance			5 PSF
Misc Finishes Allowance			2.5 PSF
		Total:	**96 PSF**

6.3 Roof Live Load

Table 6.3.1 | Code, Existing, and Design Roof Live Load Value

Occupancy	ASCE 7-05 Load (Tables 4-1/C4-1)	Existing Design As Noted on Drawings	Design Load Used for Typical Floors
Roof	20 PSF	N/A	20 PSF
		Total:	**20 PSF**

6.4 Roof Dead Load

Table 6.4.1 | Calculation of Design Roof Dead Load

Item	Dimension	Unit Weight	Load
3" 16 Ga. Type NS Roof Deck			2.46 PSF
3" NW Concrete Topping (Avg)	0.290 CF/SF	145 PCF	42 PSF
Framing Self-Weight Allowance			4 PSF
Roofing Material			12 PSF
		Total:	**62.5 PSF**

6.5 Roof Snow Load

The uniform roof snow load and snow drift surcharge were determined using the procedure provided in Chapter 7 of ASCE 7-05.

Intermediate hand calculations showing the determination of all factors and loads are included in Appendix A.1.

6.5.1 *Uniform Roof Snow Load*

Note: *A discrepancy exists between the design roof snow load of 21 PSF and the calculated value of 22 PSF that can be attributed to Building Type II and a resulting importance factor of I=1.0 being used for the existing design.*

Table 6.5.1.1 | Uniform Roof Snow Design Factors and Load

Design Factor	ASCE 7-05 Reference	Design Value
Ground Snow Load (pg)	Figure 7-1	30 PSF
Roof Exposure	Table 7-2	Fully Exposed
Exposure Type	Section 6.5.6.2	B
Exposure Factor (Ce)	Table 7-2	0.9
Thermal Factor (Ct)	Table 7-3	1.0
Building Type	Table 1-1	III
Importance Factor (I)	Table 7-4	1.1
Calculated Flat Roof Snow Load (pf)	Equation 7-1	21 PSF
Alternative Minimum Snow Load (pf,min)	Section 7.2	22 PSF
Design Flat Roof Snow Load (pf)	Section 7.2	**22 PSF**

6.5.2 *Snow Drift Surcharge*

As a representative case, the East side of the "C" wing penthouse was selected for a sample calculation. This location was selected because it provided the greatest distance from the obstruction (penthouse) to the edge of the roof for a substantial drift to develop, and because it is within the tributary area of the column selected for a spot check.

It is important to note that there are several other locations where a drift calculation would be required. All sides of the penthouses on both the "C" and "B" wings (especially the East side of the "B" wing); the cooling towers on the "C" wing roof; and, significantly, the area between the "B" wing penthouse and the change in elevation to the "C" wing roof where the two resulting drifts could overlap and lead to significant accumulations.

Table 6.5.2.1 | Snow Drift Surcharge Design Factors and Load

Design Factor	ASCE 7-05 Reference	Design Value
Ground Snow Load (pg)	Figure 7-1	30 PSF
Snow Density (γ)	Equation 7-3	**17.9 PCF**
Design Flat Roof Snow Load (pf)	Section 7.2	22 PSF
Height of Balanced Snow Load (hb)	Section 7.7.1	1.28'
Clear Height Above Balanced Snow Load (hc)	Section 7.7.1	15.0'
Roof Length Upwind (lu)	Figure 7-8	45.5'
Snow Drift Height (hd)	Figure 7-9	**2.36'**
Snow Drift Width (w)	Section 7.7.1	**9.44'**

6.6 Penthouse Live Load

Table 6.6.1 | Calculation of Design Penthouse Live Load

Occupancy	ASCE 7-05 Load (Tables 4-1/C4-1)	Existing Design As Noted on Drawings	Design Load Used for Typical Floors
Mechanical Equipment Rooms	200 PSF	N/A	200 PSF
		Total:	**200 PSF**

6.7 Penthouse Dead Load

Table 6.7.1 | Calculation of Design Penthouse Dead Load

Item	Dimension	Unit Weight	Load
3" 18 Ga. Composite Deck			2.84 PSF
4-1/2" NW Concrete Topping	0.485 CF/SF	145 PCF	70.3 PSF
Framing Self-Weight Allowance			5 PSF
MEP Allowance			10 PSF
Ceiling Allowance			5 PSF
Misc Finishes Allowance			2.5 PSF
		Total:	**96 PSF**

6.8 Brick Veneer Façade Dead Load

Table 6.8.1 | Calculation of Design Brick Veneer Façade Dead Load

Item	Dimension	Unit Weight	Load
Brick Veneer	10'-3" per level	35 PSF	357.8 PLF
2" Rigid Insulation	10'-3" per level	3 PSF	30.7 PLF
Cold-form Steel Framing & Ins.	10'-3" per level	6 PSF	61.3 PLF
Gypsum Board	10'-3" per level	2 PSF	20.5 PLF
Window glass, frame, and sash (per ASCE 7-05 Table C3-1)	5'-1" per level	8 PSF	40.8 PLF
		Total:	**510.6 PLF**

6.9 Glass Curtainwall Dead Load

Table 6.9.1 | Calculation of Design Glass Curtainwall Dead Load

Item	Dimension	Unit Weight	Load
Window glass, frame, and sash (per ASCE 7-05 Table C3-1)	15'-4" per level	8 PSF	122.4 PLF
		Total:	**122.4 PLF**

6.10 Penthouse Wall Dead Load

Table 6.10.1 | Calculation of Design Penthouse Wall Dead Load

Item	Dimension	Unit Weight	Load
Metal Wall Panel System	16'-4” per level	5 PSF	81.7 PLF
Cold-form Steel Framing	16'-4” per level	7 PSF	114.3 PLF
Bracing Allowance	16'-4” per level	3 PSF	49 PLF
		Total:	**246 PLF**

7. Wind Pressures

Design wind pressures were determined using the Analytical Procedure provided in Chapter 6 of ASCE 7-05. The Fundamental Natural Frequency (n1) of the building was determined to be 0.68 Hz in the transverse (E/W) direction and 0.56 Hz in the longitudinal (N/S) direction by Eq. C6-19 (ASCE 7-05). Since both values are less than 1 Hz, the building is considered flexible, and provisions related to flexible buildings apply.

Intermediate hand calculations showing the determination of all factors and pressures are included in Appendix A.2.

Table 7.1 | Wind Pressure Design Factors

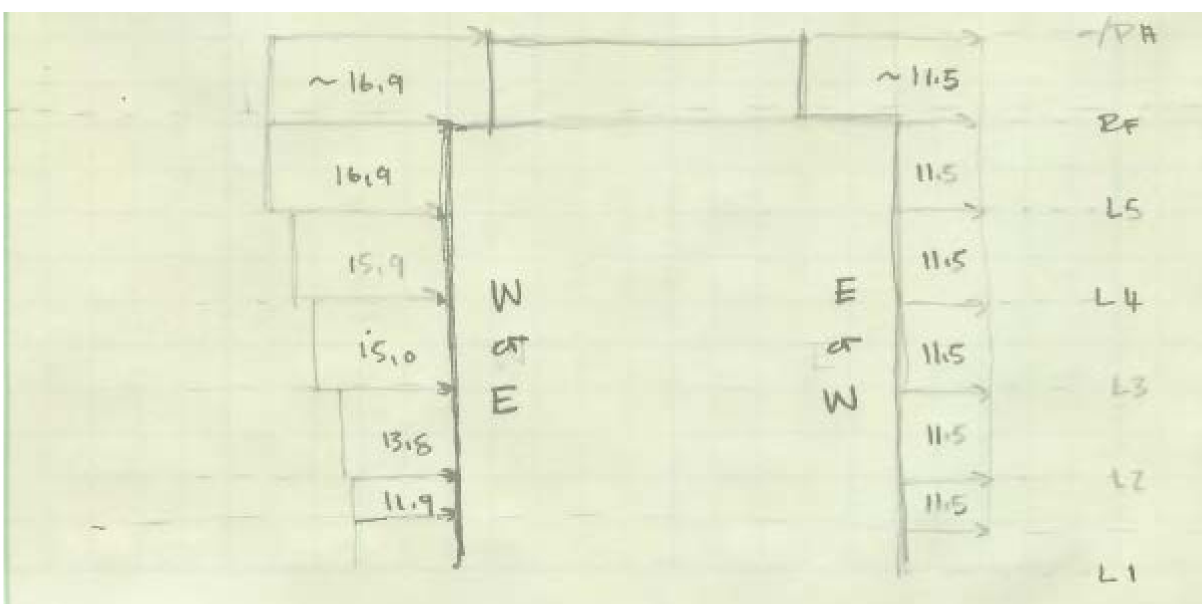
Design Factor	ASCE 7-05 Reference	E/W Value	N/S Value
Design Wind Speed (V)	Figure 6-1C	90 mph	
Building Type	Table 1-1	III	
Importance Factor (I)	Table 6-1	1.15	
Exposure Type	Section 6.5.6.2	B	
Fundamental Natural Frequency (n1)	Equation C6-19	0.68 Hz	0.56 Hz
Equivalent Height (z)	Section 6.5.8	46.8'	60'
Integral Length Scale of Turbulence (Lz)	Equation 6-7	360'	390'
Intensity of Turbulence (Iz)	Equation 6-4	0.23	0.22
Mean Hourly Wind Speed (Vz)	Equation 6-14	64.7 ft/sec	69.0 ft/sec
Reduced Frequency (N1)	Equation 6-12	3.78 Hz	3.16 Hz
Damping Ratio (**Beta**)	Commentary p. 294	0.01	
Background Response (Q)	Equation 6-6	0.79	0.85
Resonant Response Factor (R)	Equation 6-10	0.0238	0.0506
Gust Effect Factor (Gf)	Equation 6-8	0.877	0.914

In the transverse direction, the building is roughly symmetrical, but the site slopes from north to south, creating variation in roof height above grade. The Mean Roof Height (h) of 86'-2" was established using level 1 as the average ground level. Because there is no significant difference between the East and West facades, one set of calculations was completed to determine Velocity Pressures (qz) and Wind Pressures (p).

Table 7.2 | Design Wind Pressure by Level (Transverse Direction)

Level	Height	kz	qz	Pz (windward)	Ph (leeward)	Ptot
G	(below ground)	--	--	--	--	--
1	0'-0"	0.57	11.5	11.8	-11.5	23.3
2	15'-4"	0.58	11.7	11.9	-11.5	23.5
3	30'-8"	0.71	14.4	13.8	-11.5	25.9
4	46'-0"	0.79	16	15	-11.5	27.5
RF / 5	60'-8"	0.85	17.2	15.9	-11.5	28.7
RF / PH	77'-0"	0.92	18.6	16.9	-11.5	30.1

Figure 7.1 | Design Wind Pressure by Level (Transverse Direction)



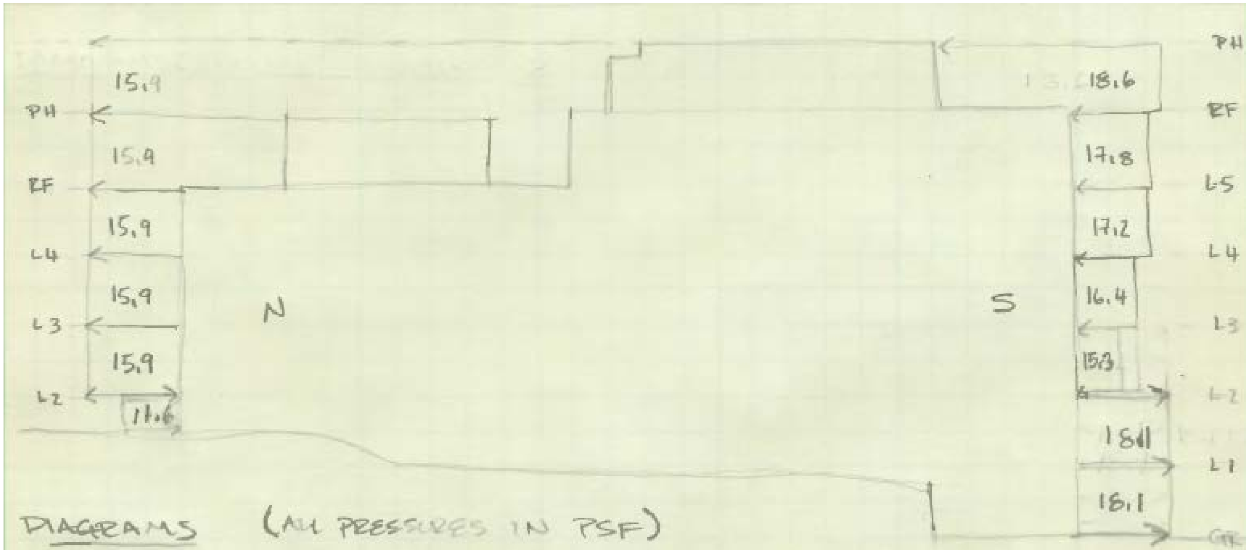
In the longitudinal direction, there is a significant difference in height between the north and south facades, with the south facade being 32' taller. Wind pressure factors were calculated using the south facade Mean Roof Height (h) of 100' to generate conservative results. Velocity Pressures (qz) and Wind Pressures (p) were then calculated once assuming wind from the north and once assuming wind from the south to determine the worst-case loading scenario for each story. From level G (below grade) to level 2 (9'-4”) measured from the base of the north facade, wind from the north created larger pressures, primarily resulting from leeward pressure on exposed south facade from level G to level 2. From level 3 (46'-8”) to the “C” wing Penthouse Roof (108'-4”) measured from the base of the south facade, wind from the south created larger pressures, resulting from the greater height of the south facade. The greatest absolute total pressure combinations from each analysis were then combined to generate the wost-case values for story shear.

Table 7.3 | Design Wind Pressure by Level (Longitudinal Direction)

Level (“C” / “B” wings)	Height	kz	qz	Pz (windward)	Ph (leeward)	Ptot
G*	(below grade)	--	--	--	-18.1	18.1
1*	(below grade)	--	--	--	-18.1	18.1
2*	9'-4”	0.57	11.5	11.6	-18.1	29.7
3	46'-8'	0.79	16.0	15.3	-15.9	31.2
4	62'-0”	0.86	17.5	16.4	-15.9	32.3
RF / 5	77'-4”	0.92	18.6	17.2	-15.9	33.1
PH / RF	92'-0”	0.96	19.5	17.8	-15.9	33.7
-- / PH	108'-4”	1.01	20.5	18.6	-15.9	34.5

** Dimensions and values for these levels are based on the north facade. All other dimensions and values are based on south facade. See Appendix [X] for complete values for each facade.*

Figure 7.2 | Design Wind Pressure by Level (Longitudinal Direction)



8. Seismic Loads

Design seismic loads were determined using the Equivalent Lateral Force procedure provided in Chapters 11 and 12 of ASCE 7-05. The design values for story shear generated by the procedure ensure that the lateral system is capable of handling the shear and moment resulting from seismic motion, taking into account both site and building properties.

Intermediate hand calculations showing the determination of all factors and loads are included in Appendix A.3.

8.1 Design Factors

Identical design factors were used in the longitudinal and transverse directions because the lateral system in both directions is the same. In lieu of the significantly more extensive analysis needed to determine the actual fundamental period of the building, the approximate fundamental period described in ASCE 7-05 Section 12.8.2.1 was determined, as permitted by Section 12.8.2.

Table 8.1.1 | Seismic Load Design Factors

Design Factor	ASCE 7-05 Reference	Value
Short-period Spectral Response Acceleration Parameter (Ss)	(USGS/Existing)	0.291
One-second Spectral Response Acceleration Parameter (S1)	(USGS/Existing)	0.081
Site Class	(USGS/Existing)	C
Short-period Site Coefficient (Fa)	Table 11.4-1	1.2
Long-period Site Coefficient (Fv)	Table 11.4-2	1.7
Adjusted MCE Short-period Spectral Response Acceleration Parameter (SMs)	Equation 11.4-1	0.349

Adjusted MCE One-second Spectral Response Acceleration Parameter (SM1)	Equation 11.4-2	0.138
Design Short-period Spectral Response Acceleration Parameter (SMs)	Equation 11.4-3	0.233
Design One-second Spectral Response Acceleration Parameter (SM1)	Equation 11.4-4	0.0918
Maximum Height from Base (hn)	n/a	108.3'
Approximate Period Parameter (Ct)	Table 12.8-2	0.028
Approximate Period Parameter (x)	Table 12.8-2	0.8
Approximate Fundamental Period (Ta)	Equation 12.8-7	1.19 Hz
Building Type	Table 1-1	III
Importance Factor (I)	Table 11.5-1	1.25
Seismic Design Category	Table 11.6-2	B
Response Modification Coefficient ®	Table 12.2-1	3.0
System Overstrength Factor (**omega)**	Table 12.2-1	3.0
Deflection Amplification Factor (Cd)	Table 12.2-1	3.0
Flexible Diaphragm Condition	Section 12.3.1	Rigid
Long-period Transition Period (TL)	Figure 22-15	6
Seismic Response Coefficient (Cs)	Equation 12.8-3	0.0321

8.2 Effective Seismic Weight

The effective seismic weight throughout the building was calculated using typical floor, roof, facade, and penthouse wall values determined in the "Design Gravity Loads" portion of this report. Additional loads were considered per Section 12.7.2. Partition weight was not included due to the previous assumption that all floor areas were designed for live load in excess of 80 PSF. The mechanical penthouse live load of 200 PSF was included because the mechanical equipment is permanent. Roof snow load was not included because the ground snow load is not in excess of 30 PSF.

Table 8.1.1 | Effective Seismic Weight by Level

Level	**Floor Area (96 PSF)**	**Roof Area (62.5 PSF)**	**Penthouse Floor Area (296 PSF)**	**Brick Veneer Facade Perimeter (510.6 PLF)**	**Glass Curtainwall Perimeter (122.4 PLF)**	**Penthouse Wall Perimeter (246 PLF)**	**Effective Seismic Weight**
7 (PH-C)		4497 SF					281.06k
6(RF-C/PH-B)		7894 SF	4497 SF			288.7'	1895.08k
5 (RF-B)	10832 SF	9375 SF	1557 SF	421.3'		161.3'	2341.01k
4	21814 SF			589.7'	89.5'		2406.21k
3	21814 SF			589.7'	89.5'		2406.21k
2	21814 SF			589.7'	89.5'		2406.21k
1	21814 SF			589.7'	89.5'		2406.21k
Total	**98088 SF**	**21766 SF**	**6054 SF**	**2780'**	**382'**	**450'**	**14141.9k**

8.3 Design Seismic Loads

The seismic base shear (V) was determined to be 453.9 kips, and the overturning moment at the base was determined to be 34250 ft-kips. The actual seismic base shear used for the existing design is not known, but this value falls within the range determined for similarly sized buildings in design guides and Technical Reports from prior years.

Table 8.1.1 | Design Seismic Load by Level

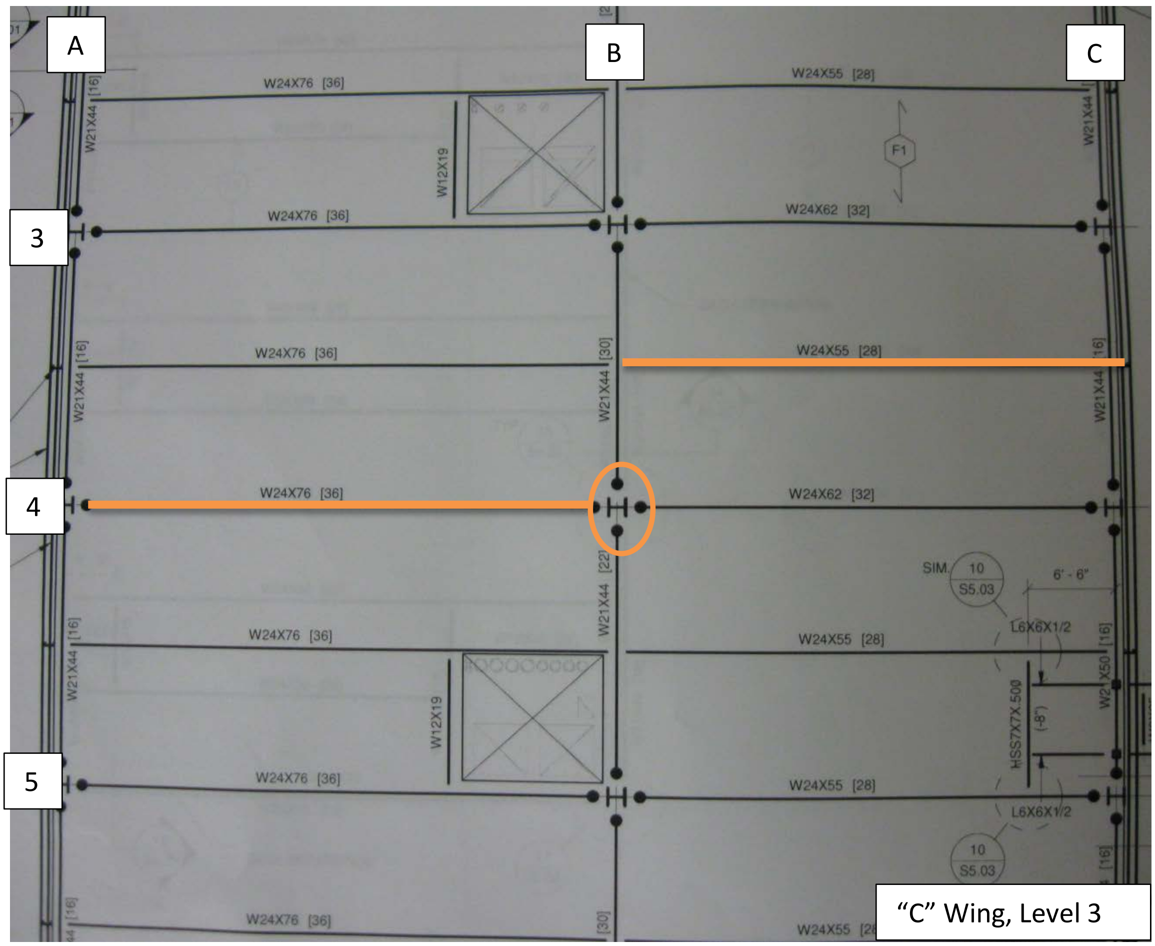
Level	Effective Seismic Weight (wx)	Height from Base (hx)	wxhx^k	Vertical Distribution Factor (Cvx)	Lateral Seismic Force (Fx)	Seismic Design Story Shear (Vx)	Overturning Moment
7 (PH)	281.06k	108'-4”	3298551.4	0.06542042	29.698k	29.7k	3217.3ft-k
6 (RF/PH)	1895.08k	93'-0”	16390546.9	0.32507488	147.57k	177.3k	13724ft-k
5 (RF)	2341.01k	76'-8”	13759936.5	0.27290179	123.89k	301.2k	9498.2ft-k
4	2406.21k	61'-4”	9051627.3	0.17952157	81.495k	382.7k	4998.3ft-k
3	2406.21k	46'-0”	5091540.4	0.10098088	45.841k	428.5k	2108.7ft-k
2	2406.21k	30'-8”	2262905.8	0.04488037	20.374k	448.9k	624.67ft-k
1	2406.21k	15'-4”	565726.7	0.01122010	5.093k	454.0k*	78.091ft-k
Total	**14141.9k**		**50420835**	**0.993 ~ 1.0**			**34250ft-k**

***Calculated Seismic Base Shear = **453.9k**

9. Gravity Member Spot Checks

Designs for a span of composite metal deck and slab, a transverse composite beam, and a wide-flange column were each checked against strength and serviceability requirements and compared to members at the same locations in the existing design.

Figure 9.1 | Location of Spot-Checked Members



9.1 Composite Metal Deck and Slab

With the design loads determined in the "Design Gravity Loads section of this report, the total superimposed load on the slab is 125 PSF live load plus 20 PSF miscellaneous dead load, for a total of 145 PSF. Using Vulcraft 3VLI18 as representative, the 3" composite metal deck with 4-1/2" normal weight concrete topping can support a superimposed live load of approximately 210 PSF with a conservative 11'-0" clear span. The deck is also suitable for unshored construction, with the 12'-0" maximum unshored clear span exceeding the design span of 10'-8".

Based on this check, the deck and slab specified in the existing design is suitable for the design loads.

9.2 Composite Beam

The beam selected for design spans transversely between columns A4 and B4 and supports a biology laboratory. The initial check was performed using the design live load of 100 PSF for laboratory occupancy, which was subsequently reduced to 75 PSF per ASCE 7-05 Section 4.8. The design dead load was determined in the "Design Gravity Loads" section of this report.

The results of this check are shown in Table 9.2.1.

Table 9.2.1 | Comparison of Trial Member to Existing Design

Design Loads D = 96 PSF L = 75 PSF 1.2D + 1.6L = 235.2 PSF	**Beam Size**	**Bare Beam Flexure Capacity**	**Bare Beam Moment of Inertia**	**Composite Beam Design Strength**	**Composite Lower-Bound Moment of Inertia**
Required		306 ft-k	1004 in4	556 ft-k	1354 in4
Trial Member	W21x55 [24]	473 ft-k	1140 in4	695 ft-k	2110 in4
Existing Design	W24x76 [36]	750 ft-k	2100 in4	1230 ft-k	4480 in4
Ratio of Existing / Trial	1.38 [1.5]	1.58	1.84	1.77	2.12

The most apparent reason for the discrepancy would be underestimation of the design live load. Considering that institutions typically plan for a much longer building life cycle than commercial owners, it is reasonable to assume that the system was designed for maximum flexibility. From this reasoning, the highest design floor live load of 125 PSF was used. Because this live load is greater than 100 PSF, it could not be reduced per ASCE 7-05 Section 4.8. To isolate variables, the design live load remained unchanged.

The results of this check are shown in Table 9.2.2.

Table 9.2.2 | Comparison of Trial Member to Existing Design

Design Loads D = 96 PSF L = 125 PSF 1.2D + 1.6L = 315.2 PSF	**Beam Size**	**Bare Beam Flexure Capacity**	**Bare Beam Moment of Inertia**	**Composite Beam Design Strength**	**Composite Lower-Bound Moment of Inertia**
Required		306 ft-k	1004 in4	745 ft-k	2332 in4
Trial Member	W24x55 [24]	503 ft-k	1350 in4	865 ft-k	2500 in4
Existing Design	W24x76 [36]	750 ft-k	2100 in4	1230 ft-k	4480 in4
Ratio of Existing/Trial	1.38 [1.5]	1.49	1.55	1.42	1.79

To troubleshoot this result, a second location was then checked. The new member also spans transversely, but on the opposite side of the building between columns B4 and C4. The beam supports graduate student offices, rests on longitudinal girders, and does not participate in the flexible moment frame system.

Selecting a beam that is not framed into columns and in a different occupancy area was hoped to determine whether the member size mis-match was driven either by 1) an unaccounted-for aspect of the lateral system, or 2) additional strength or serviceability requirements in the area of the first member. If the trial member were substantially oversized, it would suggest that the former is true, and the live load assumption was a false lead. If the trial member were close to the existing design, it would suggest that the latter is true and the live load assumption was appropriate. If the trial member were undersized by a ratio similar to that of the second trial member, it would suggest that the same unknown load conditions exist throughout the building.

The results of this check are shown in Table 9.2.3.

Table 9.2.3 | Comparison of Trial Member to Existing Design

Design Loads D = 96 PSF L = 125 PSF 1.2D + 1.6L = 315.2 PSF	**Beam Size**	**Bare Beam Flexure Capacity**	**Bare Beam Moment of Inertia**	**Composite Beam Design Strength**	**Composite Lower-Bound Moment of Inertia**
Required		233 ft-k	671 in4	569 ft-k	1672 in4
Trial Member	W21x48 [22]	401 ft-k	959 in4	597 ft-k	1810 in4
Existing Design	W24x55 [26]	473 ft-k	1350 in4	852 ft-k	2910 in4
Ratio of Existing/Trial	1.14 [1.18]	1.17	1.41	1.42	1.61

Surprisingly, applying a live load in excess of twice the office occupancy load to this member did result in a trial size that is still less than the existing design, but even closer than the corresponding trial size in the laboratory occupancy area. This suggests the possibility that the increased beam sizes throughout the building were selected to provide increase diaphragm stiffness in order to meet serviceability requirements like vibration control and increased sensitivity to live load deflection in a laboratory environment.

Hand calculations showing the determination of required capacity and the selection of the members are included in Appendix A.4.

9.3 Column Gravity Check

The column selected for design was B4 mid-height between levels 3 and 4, an interior column adjacent to the beams used for the check above. All columns used in the existing design have a depth of 14”, and that restriction was also used to design the trial member.

Gravity loads accumulate from floor and roof areas within the tributary area of the column. These loads include floor dead and live loads; roof dead, live, uniform snow, and snow drift loads; and penthouse dead and live loads. The design loads used were determined in the “Design Gravity Loads” section of this report, and are calculated in detail, with live load reductions as permitted, in Appendix A.5.

These loads are totaled in Table 9.3.1.

Table 9.3.1 | Accumulated Gravity Loads in Column B4

Level	Dead Load	Reduced Live Load	Snow Load
Penthouse Roof	30.3k	5.82k	11.2k
Roof Level	71.0k	90.9k	6.84k
Level 5	80.9k	84.3k	
Level 4	80.9k	84.3k	
Total	**263.1k**	**265.3k**	**18.1k**

In determining the factor G needed to enter the nomograph (AISC Figure C-C2.4), the girder length was doubled following the procedure outlined in Geschwinder/Desque 2005. This method of modeling flexible moment connections assumes that the far end of each girder is pinned, reflecting the limited capacity of flexible moment connections to support girder moments. Because each connection is designed to resist lateral loads only, it has no remaining capacity to absorb moment and stiffen the column under design lateral loading. This is also the reason horizontal members are designed as simply supported beams.

The column, like all other major columns in the building, is part of the lateral load resisting system. Because the analysis required to determine the design lateral load for the frame is beyond the scope of this report, and the beams do not transmit any floor load flexure to the column, the trial member was designed for gravity load only.

The results of this check are shown in Table 9.3.2.

Table 9.3.2 | Comparison of Trial Member to Existing Design

Design Loads D = 96 PSF L = 265.3 kips S = 18.1 kips 1.2D + 1.6L 0.5S = 749.25 kips	**Column Size**	**Column Axial Capacity w/ (KL)eff = 21'**	**Column Moment of Intertia**
Required		750 ft-k	
Trial Member	W14x90	848 ft-k	999 in4
Existing Design	W14x193	1925 ft-k	2400 in4
Ratio of Existing/Trial	2.14	2.27	2.4

The extreme difference between the trial member and existing design reflects the design for gravity load only, and demonstrates that lateral loads and second order effects will govern column design due to the flexible moment frame system.

Hand calculations showing the determination of required capacity and the selection of a member are included in Appendix A.5.

A.1 Design Snow Load Calculations

TECH REPORT #1	CALCULATIONS	SNOW LOADS	1/

FLAT ROOF SNOW LOAD

• DETERMINE GROUND SNOW LOAD (P_g)

BY MAP, $P_g = 30$ PSF (FIG 7-1)

• DETERMINE EXPOSURE FACTOR (C_e)

LARGE MECHANICAL EQUIPMENT ∴ FULLY EXPOSED

EXPOSURE/TERRAIN TYPE - TYPE B
(DETERMINED IN WIND ANALYSIS)

∴ $C_e = 0.9$ (TABLE 7-2)

• DETERMINE THERMAL FACTOR (C_t)

DOES NOT MEET EXCEPTION REQUIREMENTS

∴ $C_t = 1.0$ (TABLE 7-3)

• DETERMINE SNOW LOAD IMPORTANCE FACTOR (I)

CATEGORY III ∴ $I = 1.1$ (TABLE 7-4)

• ALTERNATIVE MINIMUM SNOW LOAD ($P_{f,min}$)

$P_g = 30$ PSF > 20 PSF

∴ USE $P_{f,min} = (20)(I) = (20)(1.1) = 22$ PSF (§ 7.2)

• FLAT ROOF SNOW LOAD (P_f)

$P_f = 0.7 C_e C_t I P_g \geq P_{f,min}$ (EQ. 7-1)

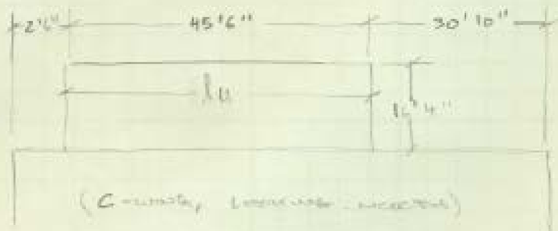
$= (0.7)(0.9)(1.0)(1.1)(30) = 21 \leq 22$

∴ $P_f = 22$ PSF

TECH REPORT #1	CALCULATIONS	SNOW LOADS	2/

SNOW DRIFT LOAD EXAMPLE

- WILL CHECK EAST SIDE OF C-WING PENTHOUSE AS AN EXAMPLE DUE TO IMPACT ON COLUMN SELECTED FOR ANALYSIS.



- DETERMINE SNOW DENSITY

$\gamma = 0.13 p_g + 14 \leq 30$ PCF (EQ 7-3)

$= (0.13)(30) + 14 = 17.9 \leq 30$

$\therefore \gamma = 17.9$ PCF

- DETERMINE HEIGHT OF BALANCED SNOW LOAD

$h_b = \frac{p_f}{\gamma} = \frac{23}{17.9} = 1.28'$ (§ 7.7.1)

- DETERMINE CLEAR HEIGHT ABOVE BALANCED SNOW LOAD

$h_c = h_{PENTHOUSE} - h_b = 16.3 - 1.28 = 15.0'$ (§ 7.7.1)

- DETERMINE SNOW DRIFT HEIGHT

LEEWARD

$h_d = 0.43 \sqrt[3]{l_u} \sqrt[4]{p_g + 10} - 1.5$ (FIG 7-9)

$= 0.43 \sqrt[3]{45.5} \sqrt[4]{30+10} - 1.5 = 2.36'$

WINDWARD

$h_d =$ MAX OF | $0.75 h_{d,LEEWARD} = (0.75)(2.36) = 1.77'$
CALC'D w/ $l_u = 30'10" = 1.69'$

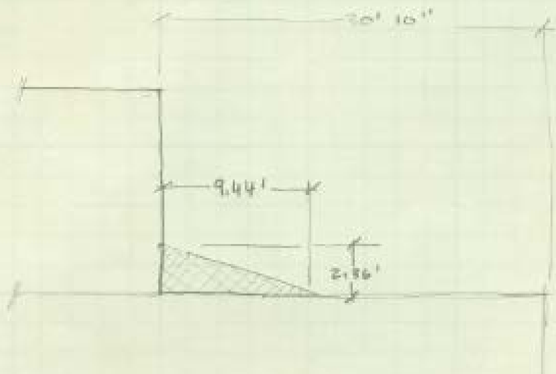
TECH REPORT #1 | CALCULATIONS | SNOW LOADS | 3/

* DETERMINE SNOW DRIFT WIDTH

$h_d = 2.36' < h_c = 13.0'$ (7.7.1)

$\therefore w = 4h_d = 4(2.36) = 9.44'$

* SKETCH OF DESIGN SNOW DRIFT



A.2 Design Wind Pressure Calculations

TECH REPORT #1	CALCULATIONS	WIND LOADS	1/

- USE ANALYTICAL PROCEDURE (ASCE 7-05 § 6.5)

DESIGN WIND SPEED (V)

$\boxed{V = 90 \text{ MPH}}$ (FIG 6-1C)

BUILDING TYPE

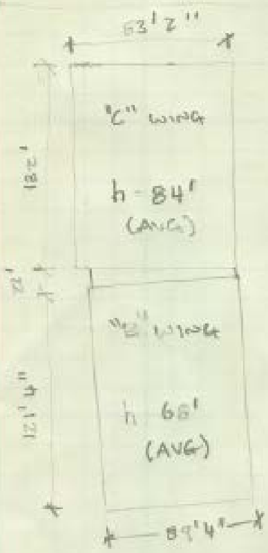
COLLEGE + EDUCATION USE
OCCUPANCY = 1490 > 500 } TYPE III (TABLE 1-1)

IMPORTANCE FACTOR (I)

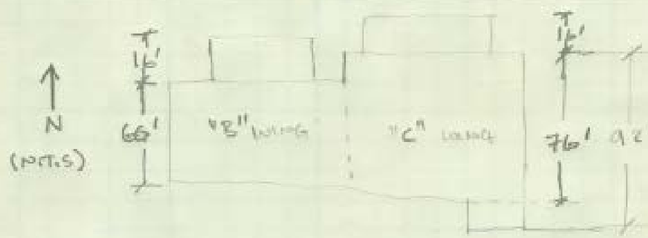
TYPE II ; V < 100 ∴ I = 1.15 (TABLE 6-1)

SURFACE ROUGHNESS / EXPOSURE TYPE

USE ROUGHNESS "B" (URBAN/SUBURBAN) (§ 6.5.6.2)



PLAN



EAST ELEVATION

E/W

$L = 86.2'$ $B = 275.3'$

$$h = \frac{(154)(84') + (121.3)(68')}{(154 + 121.3)} = 78'$$

* USED WEIGHTED AVERAGE TO SIMPLIFY CALCULATIONS

N/S

$L = 275.3'$ $B = 86.2'$

$h = 100'$

* USED NORTH END HT TO BE MORE CONSERVATIVE:

$h\uparrow \Rightarrow n_1 \downarrow \Rightarrow G_f \uparrow \Rightarrow p \uparrow$

TECH REPORT #1	CALCULATIONS	WIND LOADS	2/

FUNDAMENTAL NAT'L FREQ. (n_1)

STEEL MOMENT FRAME

$\therefore \quad n_1 = 22.2 / H^{0.8}$ (EQ C6-19)

EAST / WEST

$n_1 = \frac{22.2}{78^{0.8}} = 0.68\ Hz$

$n_1 < 1\ Hz \therefore$ FLEXIBLE

NORTH / SOUTH

$n_1 = \frac{22.2}{84^{0.8}} = 0.56\ Hz$

$n_1 < 1\ Hz \therefore$ FLEXIBLE

GUST EFFECT FACTOR

- DETERMINE EQUIVALENT HT $(\bar{z})$

	E/W	N/S	
$\bar{z} = 0.6h > 30'$	$\bar{z} = 46.8'$	$\bar{z} = 60.0'$	(6.5.8.1)

- DETERMINE INTEGRAL LENGTH SCALE OF TURBULENCE $(L_{\bar{z}})$

EXPOSURE B $\therefore$ $\ell = 320'$, $\bar{\epsilon} = 1/3.0$ (TABLE 6-2)

	E/W	N/S	
$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$	$L_{\bar{z}} = 360'$	$L_{\bar{z}} = 390'$	(EQ 6-7)

- DETERMINE INTENSITY OF TURBULENCE $(I_{\bar{z}})$

EXPOSURE B $\therefore$ $c = 0.3$ (TABLE 6-2)

	E/W	N/S	
$I_{\bar{z}} = c\left(\frac{10}{\bar{z}}\right)^{1/6}$	$I_{\bar{z}} = 0.232$	$I_{\bar{z}} = 0.222$	(EQ 6-5)

- DETERMINE MEAN HOURLY WIND SPEED $(\bar{V}_{\bar{z}})$

	E/W	N/S	
$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right)$	$\bar{V}_{\bar{z}} = 64.7$ ft/sec	$\bar{V}_{\bar{z}} = 69.0$ ft/sec	(EQ 6-14)

where $\bar{b} = 0.45$ (TABLE 6.2)
$\bar{\alpha} = 1/4.0$ (")

TECH REPORT #1	CALCULATIONS	WIND LOADS	3/

- DETERMINE REDUCED FREQUENCY (N_1)

	E/W	N/S	
$N_1 = \frac{n_1 L_z}{\bar{V}_z}$	$N_1 = 3.78\ Hz$	$N_1 = 3.16\ Hz$	(EQ 6-12)

- DETERMINE DAMPING RATIO (β)

STEEL FRAME ∴ $\beta = 0.01$ (PG 294)

- DETERMINE BACKGROUND RESPONSE (Q)

	E/W	N/S	
$Q = \sqrt{\left(1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}\right)^{-1}}$	$Q = 0.765$	$Q = 0.847$	(EQ. 6-6)

- DETERMINE RESONANT RESPONSE FACTOR (R)

	E/W	N/S	
$\eta_h = \frac{4.6 n_1 h}{\bar{V}_z}$	$\eta_h = 3.77$	$\eta_h = 3.73$	(EQ 6-13)
	$R_h = 0.230$	$R_h = 0.232$	(")
$\eta_B = \frac{4.6 n_1 B}{\bar{V}_z}$	$\eta_B = 13.3$	$\eta_B = 3.21$	(")
	$R_B = 0.0723$	$R_B = 0.263$	(")
$\eta_L = \frac{15.4 n_1 L}{\bar{V}_z}$	$\eta_L = 13.9$	$\eta_L = 34.4$	(")
	$R_L = 0.0719$	$R_L = 0.0266$	(")

* ABOVE $R_j = \frac{1}{\eta_j} - \frac{1}{2\eta_j^2}\left(1 - e^{-2\eta_j}\right)$ (")

	E/W	N/S	
$R_n = \frac{7.47 N_1}{(1+10.3 N_1)^{5/3}}$	$R_n = 0.0605$	$R_n = 0.0773$	(EQ 6-11)
$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$	$R = 0.0238$	$R = 0.0506$	(EQ 6-10)

TECH REPORT #1	CALCULATIONS	WIND LOADS	4/

• SOLVE FOR GUST EFFECT FACTOR

$$G_f = 0.925\left[\frac{1+1.7I_{\bar{z}}\sqrt{g_Q^2Q^2+g_R^2R^2}}{1+1.7g_v I_{\bar{z}}}\right]\ ;\ g_R = \sqrt{2\ln(3600n_1)} + \frac{0.577}{\sqrt{2\ln(3600n_1)}}$$

$g_Q = g_v = 3.4$ (6.5.8.2)

	E/W	N/S	
	$g_R = 4.09$	$g_R = 4.04$	(EQ 6-9)
	$G_f = \underline{0.877}$	$G_f = \underline{0.914}$	(EQ 6-6)

VELOCITY PRESSURE (q_z)

$K_{zt} = 1.0$ (6.5.7.1-2)

BUILDING ∴ $K_d = 0.85$ (TABLE 6-4)

$K_z = 2.01\left(\frac{15}{z_g}\right)^{\frac{2}{\alpha}} = 0.57$ FOR $z < 15'$, CASE 2 (TABLE 6-3)

$K_z = 2.01\left(\frac{z}{z_g}\right)^{\frac{2}{\alpha}}$ FOR $15' \leq z \leq z_g$, CASE 2 (TABLE 6-3)

where $z_g = 1200'$, $\alpha = 7.0$ (TABLE 6-2)

• SAMPLE CALCULATION OF q_z @ N/S MEAN ROOF HT, h

$h = 100'$ ∴ $K_z = 2.01(100/1200)^{2/7} = 0.98$

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$ (EQN 6-15)

$q_h = 0.00256(0.98)(1.0)(0.85)(90^2)(1.15)$

$= \underline{19.9\ \text{PSF}}$

• SAMPLE CALCULATION OF q_z @ E/W MEAN ROOF HT, h

$h = 76'$ ∴ $K_z = 2.01(76/1200)^{2/7} = 0.92$

$q_h = 0.00256(0.92)(1.0)(0.85)(90^2)(1.15)$

$= \underline{18.6\ \text{PSF}}$

* ALL VALUES CALCULATED + TABULATED ON FOLLOWING PAGE

TECH REPORT #1	CALCULATIONS	WIND LOADS	5/

EXTERNAL PRESSURE COEFFICIENTS (Cp)

	E/W	N/S	
L/B	86.2/275.3 = 0.31	275.3/86.2 = 3.19	
LEEWARD	$C_p = -0.50$	$C_p = 0.24$	(FIG 6-6)
WINDWARD	$C_p = 0.8$	$C_p = 0.8$	

WIND PRESSURE (P)

ENCLOSED BUILDING ∴ $GC_{pi} = \pm 0.18$

• SAMPLE CALCULATION OF E/W LEEWARD PRESSURE

$P = q_h G_f C_p - q_i (GC_{pi})$ (EQ 6-20

$P_h = (18.6)(0.877)(-0.5) - (18.6)(0.18)$

$= -11.5$ PSF

• SAMPLE CALCULATION OF N/S WINDWARD PRESSURE @ LEVEL 4

$z = 62'$ ∴ $q_z = 17.5$ PSF

$P_{62'} = (17.5)(0.914)(0.8) - (19.9)(-0.18)$

$= 16.4$ PSF

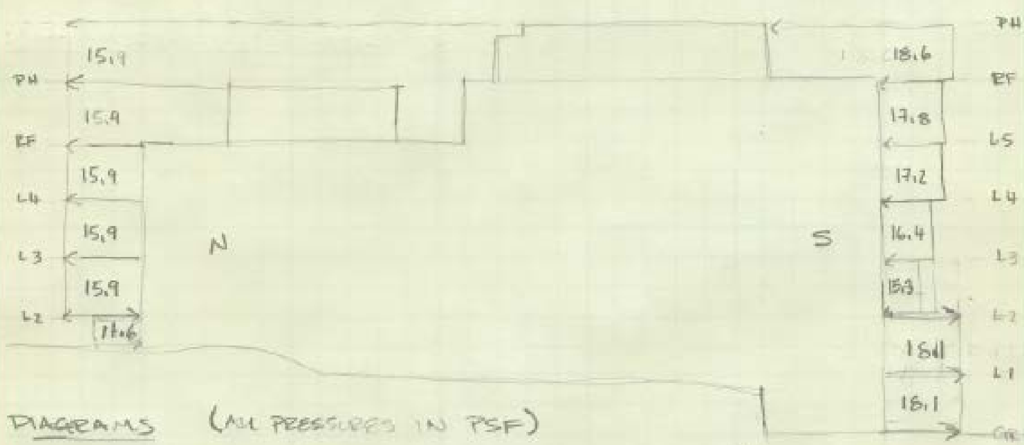
TABULATED VALUES FOR K_z, q_z (PSF), P_z (PSF)

	E/W					N/S, WIND FROM S				
LEVEL	HT	K_z	q_z	$P_z(W)$	$P_h(L)$	HT	K_z	q_z	$P_z(W)$	$P_h(L)$
GR	—	—	—	—	—	0'0"	0.57	11.5	12.0	—
L1	0'0"	0.57	11.5	11.8	-11.5	16'	0.59	11.9	12.3	—
L2	15'4"	0.58	11.7	11.9	-11.5	31'4"	0.71	14.4	14.1	-15.9
L3	30'8"	0.71	14.4	13.8	-11.5	46'8"	0.79	16.0	15.3	-15.9
L4	46'0"	0.79	16.0	15.0	-11.5	62'0"	0.86	17.5	16.4	-15.9
RF/L5	60'8"	0.85	17.2	15.9	-11.5	77'4"	0.92	18.6	17.2	-15.9
PH/RF	77'0"	0.92	18.6	16.9	-11.5	92'0"	0.96	19.5	17.8	-15.9
—/PH	—	—	—	—	—	105'4"	1.01	20.5	18.6	-15.9

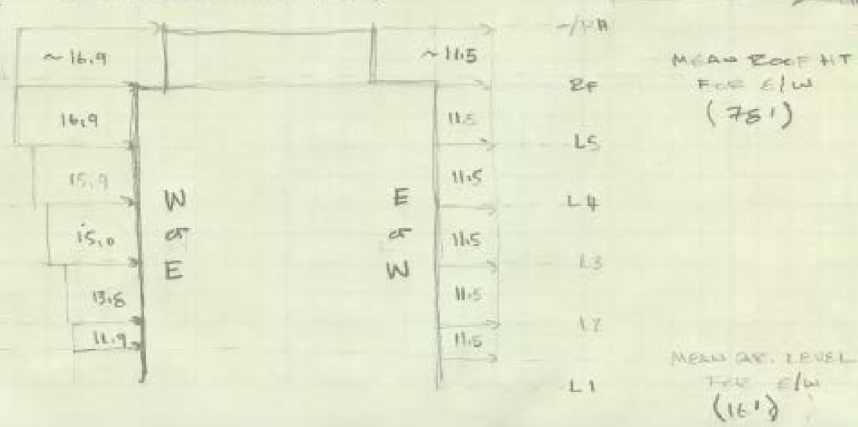
TECH REPORT #1 | CALCULATIONS | WIND LOADS | 6/

TABULATED VALUES, CONTINUED

LEVEL	N/S, WIND FROM N: HT	K_z	q_z	$P_z(W)$	$P_h(L)$	E/W P_{TOT}	N/S (S) P_{TOT}	N/S (N) P_{TOT}
GR	—	—	—	—	-16.1	—	12.0	16.1
L1	—	—	—	—	-16.1	23.3	12.3	16.1
L2	9'4"	0.57	11.5	11.6	-16.1	23.5	30	29.7
L3	24'8"	0.66	13.4	12.9	-16.1	25.9	31.8	31.0
L4	40'0"	0.76	15.4	14.4	-16.1	27.5	32.3	32.5
RF/L5	54'8"	0.83	16.5	15.4	-16.1	28.7	33.1	33.5
PH/RF	71'0"	0.96	18.2	16.5	-16.1	30.1	33.7	34.6
-/PH	—	—	—	—		—	34.5	—



DIAGRAMS (ALL PRESSURES IN PSF)



A.3 Design Seismic Load Calculations

TECH REPORT #1	CALCULATIONS	SEISMIC LOADS	1/6

SITE CLASSIFICATION + ACCELERATION

- FROM USGS ONLINE SEISMIC HAZARD CALCULATOR, IBC 2006

  $S_s = 0.261$ $\quad S_1 = 0.061$

  SITE CLASS = B

- AS DESIGNED

  $S_s = 0.291$ $\quad S_1 = 0.081$

  SITE CLASS = C

- FOR THIS CALCULATION, USE MORE CONSERVATIVE VALUES DETERMINED BY GEOTECHNICAL ENGINEER

MCE SPECTRAL RESPONSE ACCELERATIONS

- DETERMINE F_a

  SITE CLASS C, $S_s = 0.291$

  $\therefore F_a = 1.2$ (TABLE 11.4-1)

- DETERMINE F_v

  SITE CLASS C, $S_1 = 0.081$

  $\therefore F_v = 1.7$ (TABLE 11.4-2)

- DETERMINE S_{MS}

  $S_{MS} = F_a S_s = (1.2)(0.291)$ (EQ 11.4-1)

  $\therefore S_{MS} = 0.349$

- DETERMINE S_{M1}

  $S_{M1} = F_v S_1 = (1.7)(0.081)$ (EQ 11.4-2)

  $\therefore S_{M1} = 0.138$

TECH REPORT #1	CALCULATIONS	SEISMIC LOADS	2/6

DESIGN SPECTRAL ACCELERATION PARAMETERS

- DETERMINE S_{DS}

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.349)$ (EQ 11.4-3)

$\therefore S_{DS} = 0.233$

- DETERMINE S_{D1}

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.135)$ (EQ 11.4-4)

$\therefore S_{D1} = 0.0916$

APPROXIMATE FUNDAMENTAL PERIOD

$T_a = C_t h_n^x$ (EQ 12.8-7)

h_n = MAX HT FROM BASE = 105.3'

$\left.\begin{array}{l} C_t = 0.028 \\ x = 0.8 \end{array}\right\}$ FOR STL. MOMENT FRAME (TABLE 12.8-2)

$T_a = 0.028(105.3)^{0.8}$

$\therefore T_a = 1.19\ Hz$

OCCUPANCY CATEGORY

TOTAL OCCUPANCY > 500, COLLEGE

$\therefore$ CATEGORY III (TABLE 1-1)

IMPORTANCE FACTOR (I)

CATEGORY III $\longrightarrow$ I = 1.25 (TABLE 11.5-1)

SEISMIC DESIGN CATEGORY

$S_{DS} = 0.233$, OCC. CATEGORY = III (TABLE 11.6-1)

$\therefore$ CATEGORY = B

TECH REPORT #1	CALCULATIONS	SEISMIC LOADS	3/6

- CHECK CONDITION #1 FROM § 11.4.5

$T_a < 0.6\,T_S$

$T_S = \frac{S_{D1}}{S_{DS}} = \frac{0.0916}{0.1233} = 0.1393$

$T_a = 1.19 > 0.8T_S = 0.315$ ✗ NO GOOD

SO GO TO TABLE 11.6-2

$S_{D1} = 0.0981$, OCC. CATEGORY = III

S_D CATEGORY = B

RESPONSE MODIFICATION COEFFICIENT (R)

TYPE H → R = 3.0 (TABLE 12.2-1)

SYSTEM OVERSTRENGTH FACTOR (Ω_0)

TYPE H → Ω_0 = 3.0 (TABLE 12.2-1)

DEFLECTION AMPLIFICATION FACTOR (C_d)

TYPE H → C_d = 3.0 (TABLE 12.2-1)

FLEXIBLE DIAPHRAGM CONDITION

SLAB ON DECK ∴ RIGID DIAPHRAGM § 12.3.1

STRUCTURAL IRREGULARITIES

- HORIZONTAL IRREGULARITIES

TORSIONAL IRREGULARITY (TYPE 1a) COULD EXIST IF DRIFT ON TALLER C-WING EXCEEDS DRIFT ON SHORTER B-WING, BUT FULL LATERAL ANALYSIS IS NEEDED TO ESTABLISH CONDITION.

- VERTICAL IRREGULARITIES

VERTICAL GEOMETRIC IRREGULARITY (TYPE 3) EXISTS BETWEEN FLOORS 4 AND 5, BUT NOT APPLICABLE FOR SEISMIC DESIGN CATEGORY B.

TECH REPORT #1	CALCULATIONS	SEISMIC LOADS	4/6

SEISMIC RESPONSE COEFFICIENT (C_S)

- DETERMINE T_L

  BY FIGURE 22-15 → $T_L = 6$ (FIG. 22-15)

- DETERMINE UPPER LIMIT FOR C_S

  $T_a = 1.19 \leq T_L = 6$ (EQ. 12.8-3)

  $$\therefore C_S \leq \frac{S_{D1}}{T(R/I)} = \frac{0.0918}{(1.19)(3.0/1.25)} = 0.0321$$

- CHECK S_1 CRITERIA

  $S_1 = 0.046g < 0.6g$ (FIG 22-2)

  ∴ CONDITION DOES NOT APPLY

- SEISMIC RESPONSE COEFFICIENT

  $$C_S = \frac{S_{DS}}{R/I} = \frac{0.1233}{3.0/1.25} = 0.0970$$ (EQ 12.8-2)

  $C_S = 0.0970 \geq C_{S,MAX} = 0.0321$

  ∴ USE $C_S = 0.0321$

AREAS FOR EFFECTIVE SEISMIC WEIGHT

- USE DIMENSIONS ESTABLISHED DURING COLUMN SPOT CHECK AND PRELIMINARY SNOW DRIFT CALCULATIONS

  C WING FLOOR AREA = 121.3' x 89.3' = 10832 SF

  B WING FLOOR AREA = 132' x 83.1' = 10982 SF

  C WING PENTHOUSE AREA = 98'10" x 45'6" = 4497 SF

  B WING PENTHOUSE AREA = 32' x 48'8" = 1557 SF

  C WING ROOF AREA = 10832 − 4497 = 6334 SF

  B WING ROOF AREA = 10982 − 1557 = 9375 SF

TECH REPORT #1	CALCULATIONS	SEISMIC LOADS	5/6

PERIMITERS FOR EFFECTIVE SEISMIC WEIGHT

- BRICK VENEER FACADE PERIMETER

  (121.3')x2 + (132')x2 + 83.1' = 569.7'

- GLASS CURTAINWALL PERIMETER

  89.5'

- C WING PENTHOUSE WALL PERIMETER

  (95'10")x2 + (45'6")x2 = 282.7'

- B WING PENTHOUSE WALL PERIMETER

  (32'0")x2 + (48'8")x2 = 161.33'

SAMPLE CALCULATIONS

- SEISMIC BASE SHEAR (V)

  $V = C_s W = (0.0321)(14141.9\ K)$ (EQ 12.8-1)

  $\therefore V = 453.9\ K$

- VERTICAL DISTRIBUTION FACTOR (C_{vx})

  $C_{vx} = \dfrac{w_x h_x^2}{\sum w_i h_i^2}$ (EQ 12.8-12)

  AT LEVEL 3:

  $w_3 h_3^2 = (2406.21)(41.33)^2 = 5091540.4$

  (BY SPREADSHEET, $\sum w_i h_i^2 = 50420835$)

  $\therefore C_{v3} = \dfrac{5091540.4}{50420835} = 0.10098058$

- LATERAL SEISMIC FORCE (F_x)

  $F_x = C_{vx} V$ (EQ 12.8-11)

  AT LEVEL 3: $F_3 = (0.10098058)(453.9) = 45.84$ KIPS

TECH REPORT #1	CALCULATIONS	SEISMIC LOADS	6/6

- SEISMIC DESIGN STORY SHEAR (V_x)

AT LEVEL 3 :

$$V_3 = \sum_{i=7}^{3} F_i = 29.7 + 147.6 + 123.9 + 81.5 + 45.6$$

$$\therefore V_3 = 426.5 \text{ KIPS}$$

- OVERTURNING MOMENT (M_{OT})

AT LEVEL 3 :

MOMENT = FORCE × DISTANCE

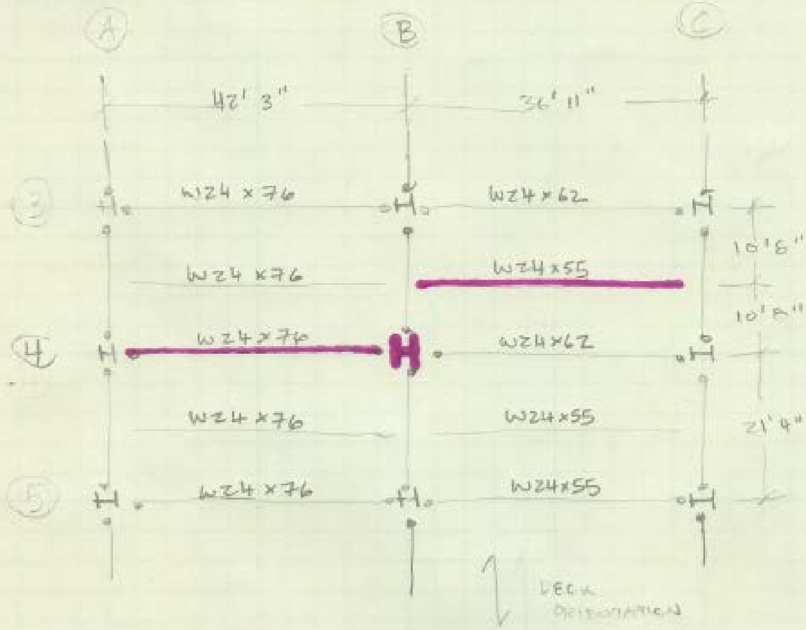
$$M_{OT,3} = V_3 \times h_3 = (426.5\text{ K})(46')$$

$$\therefore M_{OT,3} = 2106.7 \text{ ft·k}$$

A.4 Typical Beam Spot Check Calculations

TECH REPORT #1	CALCULATIONS	BEAM SPOT CHECK	1/8

LEVEL 3



FLOOR DEAD LOAD

DECK = 2.84 PSF

SLAB = $(0.485)(145)$ = 70.3 PSF

FRAMING SELF WT ALLOWANCE = 5 PSF

MEP ALLOWANCE = 10 PSF

CEILING ALLOWANCE = 5 PSF

MISC FINISHES = 2.5 PSF

TOTAL DL = 96 PSF

FLOOR LIVE LOAD

OCCUPANCY = LABORATORY $\therefore L_o = 100$ PSF (T 4-1)

$L_o = 100$ PSF ≤ 100 PSF $\therefore$ REDUCABLE

TECH REPORT #1	CALCULATIONS	BEAM SPOT CHECK	3/8

LIVE LOAD REDUCTION

- DETERMINE TRIBUTARY AREA (A_T)

$$A_T = (42.25)(10.6) = 447.8 \text{ SF}$$

- DETERMINE INFLUENCE AREA (A_I)

$$A_I = (42.25)(21.5) = 895.7 \text{ SF}$$

- DETERMINE LL ELEMENT FACTOR (K_{LL})

$$K_{LL} = \frac{A_I}{A_T} = \frac{895.7}{447.8} = 2.0 \qquad \text{(TABLE 4-2)}$$

- REDUCED LIVE LOAD (L)

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) = (100)\left(0.25 + \frac{15}{\sqrt{895.7}}\right) \qquad \text{(EQ 4-1)}$$

$$\therefore L = 0.75(100) = 75 \text{ PSF}$$

REQUIRED STRENGTH FOR DESIGN LOAD

- DETERMINE FACTORED UNIFORM LOAD (w)

$$1.2D + 1.6L = 1.2(96) + 1.6(75) = 235.2 \text{ PSF}$$

$$w = (235.2)(10.6) = 2.49 \text{ KLF}$$

- DESIGN MOMENT (M_u) AND SHEAR (V_u)

$$M_u = \frac{wl^2}{8} = \frac{(2.49)(42.25)^2}{8} = 555.6 \text{ FT-K}$$

$$V_u = \frac{wl}{2} = \frac{(2.49)(42.25)}{2} = 52.6 \text{ KIPS}$$

REQUIRED STRENGTH FOR UNSHORED CONST.

- DETERMINE GOVERNING LOAD CASE

$$w_u = \text{MAX OF} \begin{cases} 1.4D = 1.4(76+5)(10.6) = 1.20 \text{ KLF} \\ 1.2D + 1.6L = 1.2(76+5)(10.6) + 1.6(20)(10.6) = \underline{\underline{1.37}} \text{ KLF} \end{cases}$$

- REQUIRED STRENGTH

$$M_u = \frac{wl^2}{8} = \frac{(1.37)(42.25)^2}{8} = 305.7 \text{ FT-K}$$

TECH REPORT #1	CALCULATIONS	BEAM SPOT CHECK	3/8

REQUIRED MOMENT OF INERTIA FOR Δ_{WC}

- DETERMINE MAXIMUM DEFLECTION

$$L/240 = (42.25)(12)/240 = 2.11''$$

- DETERMINE UNIFORM LOAD

$$w_{WC} = (76+5)(10.6) = 0.858 \text{ KLF}$$

- REQUIRED MOMENT OF INERTIA (I_{REQD})

$$\Delta_{WC} = 2.11'' = \frac{5(0.858)(42.25)^4(12)^3}{(384)(29000)(I_{REQD})}$$

$$\therefore I_{REQD} = 1006 \text{ IN}^4$$

REQUIRED MOMENT OF INERTIA FOR Δ_{LL}

- DETERMINE MAXIMUM DEFLECTION

$$L/360 = (42.25)(12)/360 = 1.41''$$

- DETERMINE UNIFORM LOAD

$$w_{LL} = (75)(10.6) = 0.795 \text{ KLF}$$

- REQUIRED MOMENT OF INERTIA (I_{REQD})

$$\Delta_{LL} = 1.41'' = \frac{5(0.795)(42.25)^4(12)^3}{(384)(29000)(I_{REQD})}$$

$$\therefore I_{REQD} = 1354 \text{ IN}^4$$

EFFECTIVE WIDTH OF COMPOSITE BEAM

$$b' = \text{MIN OF} \begin{cases} \frac{1}{8}L = \frac{1}{8}(42.25) = 5.28' \\ \frac{1}{2} \text{ DIST TO ADJ} = \frac{1}{2}(10.3) = 5.15' \end{cases}$$

$$b_{eff} = (5.15 + 5.15)(12) = 123.6''$$

TECH REPORT #1	CALCULATIONS	BEAM SPOT CHECK	4/8

SELECT TRIAL MEMBER

- TRY W21 x 55

$I_x = 1140 > 1004 \text{ in}^4$ ✓ OK FOR Δ_{WC}

$\phi_b M_p = 473 > 305.7$ FT·K ✓ OK FOR UNSHORED CONST.

ASSUME $a = 1''$ ∴ $Y2 = 7.5 - 0.5 = 7''$ w/ $\Sigma Q_n = 203$ K

$\phi M_n = 695 > 555.6$ FT·K ✓ OK FOR M_u

$I_{LB} = 2110 > 1354 \text{ in}^4$ ✓ OK FOR Δ_{LL}

$\Sigma Q_n = 203$ KIPS ∴ $\frac{203}{17.2} = 11.8 \rightarrow 24$ STUDS / BM

CHECK $a = 1''$ ∴ $a = \frac{203}{(0.85)(125.6)(4)} = 0.48 < 1''$ ✓ OK

∴ USE W21 x 55 w/ 24 STUDS

COMPARE TO EXISTING MEMBER

- DESIGN USED W24 x 76 w/ 36 STUDS

$I_x = 2100 \text{ in}^4$ $\Sigma Q_n = 508$ KIPS

∴ NOT CLOSE → TRY AGAIN w/ L = 125 PSF

RE-EVALUATE REQUIRED STRENGTH FOR DESIGN LOAD

- DETERMINE FACTORED UNIFORM LOAD

$1.2D + 1.6L = 1.2(96) + 1.6(125) = 315.2$ PSF

$w = (315.2)(10.6) = 3.34$ KLF

- DESIGN MOMENT (M_u) AND SHEAR (V_u)

$M_u = \frac{wl^2}{8} = \frac{(3.34)(42.25)^2}{8} = 745$ FT·K

$V_u = \frac{wl}{2} = \frac{(3.34)(42.25)}{2} = 70.5$ KIPS

TECH REPORT #1	CALCULATIONS	BEAM SPOT CHECK	5/8

RE-EVALUATE REQ'D MOMENT OF INERTIA FOR Δ_{LL}

- MAXIMUM DEFLECTION = 1.41" (FROM ABOVE)
- DETERMINE UNIFORM LOAD

$w_{LL} = (125)(10.6) = 1.33$ KLF

- REQUIRED MOMENT OF INERTIA ($I_{REQ'D}$)

$$\Delta_{LL} = 1.41" = \frac{5(1.33)(42.25)^4(12)^3}{(384)(29000)(I_{REQ'D})}$$

$\therefore I_{REQ'D} = 2332$ IN^4

SELECT TRIAL MEMBER

- TRY W24x55

$I_x = 1350 > 1004$ IN^4 ✓ OK FOR Δ_{WC}

$\phi_b M_p = 503 > 305.7$ ✓ OK FOR UNSHORED CONST.

ASSUME $a = 1"$ $\therefore$ $Y2 = 7.5 - 0.5 = 7"$ W/ $\Sigma Q_n = 203$ K

$\phi M_n = 866 > 745$ FT·K ✓ OK FOR M_u

$I_{LB} = 2500 > 2332$ IN^4 ✓ OK FOR Δ_{LL}

$\Sigma Q_n = 203$ KIPS $\therefore$ $\frac{203}{17.2} = 11.8 \rightarrow 24$ STUDS / BM

CHECK $a = 1"$: $a = \frac{203}{(0.85)(125.6)(4)} = 0.46 < 1"$ ✓ OK

- COMPARE ECONOMY TO W21x48 W/ $\Sigma Q_n = 357$ KIPS

$\frac{357}{17.2} = 20.7 \rightarrow 42$ STUDS $\rightarrow$ WON'T FIT

$\frac{357}{14.6} = 24.4 \rightarrow 50$ STUDS, 2 PER RIB

AT 1 STUD = 10 # STEEL

W24x55 : $(55)(42.25) + (24)(10) = 2564$ # STL

W21x48 : $(48)(42.25) + (50)(10) = 2528$ # STL

ABOUT THE SAME $\therefore$ GO W/ LESS STUDS

TECH REPORT #1	CALCULATIONS	BEAM SPOT CHECK	6/8

- SELECT W 24×55 w/ 24 STUDS

CHECK SELF-WT ASSUMPTION

- ASSUMED 5 PSF

$$(5)(10.6) = 53 \text{ PLF} < 55 \text{ PLF} \quad \text{OK}$$

(CHECK ANOTHER BEAM ON OFFICE SIDE TO CHECK LL = 125 PSF)

DEAD LOAD

- SAME AS ABOVE → DL = 96 PSF

LIVE LOAD

- SAME AS ABOVE → LL = 125 PSF w/ NO REDUCTION

REQUIRED STRENGTH FOR DESIGN LOAD

- DETERMINE FACTORED UNIFORM LOAD

  SAME AS ABOVE → w = 3.34 KLF

- DESIGN MOMENT (M_u) AND SHEAR (V_u)

$$M_u = \frac{w l^2}{8} = \frac{(3.34)(36.9)^2}{8} = 566.5 \text{ FT}\cdot\text{K}$$

$$V_u = \frac{w l}{2} = \frac{(3.34)(36.9)}{2} = 61.6 \text{ KIPS}$$

REQUIRED STRENGTH FOR UNSHORED CONSTRUCTION

- DETERMINE GOVERNING LOAD CASE

  SAME AS ABOVE → w_u = 1.37 KLF

- REQUIRED STRENGTH

$$M_u = \frac{w l^2}{8} = \frac{(1.37)(36.9)^2}{8} = 233 \text{ FT}\cdot\text{K}$$

TECH REPORT #1	CALCULATIONS	BEAM SPOT CHECK	7/8

REQUIRED MOMENT OF INERTIA FOR Δ_{LC}

- DETERMINE MAXIMUM DEFLECTION

$$L/240 = (36.9)(12)/240 = 1.84''$$

- DETERMINE UNIFORM LOAD

$$w_{LC} = (76+5)(10.6) = 0.858 \text{ KLF}$$

- REQUIRED MOMENT OF INERTIA (I_{REQD})

$$\Delta_{LC} = 1.84'' = \frac{5(0.858)(36.9)^4(12)^3}{(384)(29000)(I_{REQD})}$$

$$\therefore I_{REQD} = 671 \text{ in}^4$$

REQUIRED MOMENT OF INERTIA FOR Δ_{LL}

- DETERMINE MAXIMUM DEFLECTION

$$L/360 = (36.9)(12)/360 = 1.23''$$

- DETERMINE UNIFORM LOAD

$$w_{LL} = (125)(10.6) = 1.33 \text{ KLF}$$

- REQUIRED MOMENT OF INERTIA (I_{REQD})

$$\Delta_{LL} = 1.23'' = \frac{5(1.33)(36.9)^4(12)^3}{(384)(29000)(I_{REQD})}$$

$$\therefore I_{REQD} = 1672 \text{ in}^4$$

EFFECTIVE WIDTH OF COMPOSITE BEAM

$$b' = \text{MIN OF} \left| \begin{array}{l} \frac{1}{8}L = \frac{1}{8}(36.9) = 4.61 \\ \frac{1}{2}\text{ DIST TO ADJ} = \frac{1}{2}(10.6) = 5.3' \end{array} \right.$$

$$b_{eff} = (4.61 + 4.61)(12) = 110.7''$$

TECH REPORT #1	CALCULATIONS	BEAM SPOT CHECK	8/8

SELECT TRIAL MEMBER

- TRY W21 × 48

$I_x = 959 > 671\ in^4$ ✓ OK FOR Δ_{LL}

$\phi_b M_p = 401 > 233\ in^4$ ✓ OK FOR UNBRACED CONST.

ASSUME $a = 1"$ ∴ $Y2 = 7.5 - 0.5 = 7"$ w/ $\Sigma Q_n = 177$ KIPS

$\phi M_n = 597 > 568.5$ FT·K ✓ OK FOR M_u

$I_{LB} = 1810 > 1672\ in^4$ ✓ OK FOR Δ_{LL}

$\Sigma Q_n = 177$ KIPS ∴ $\frac{177}{14.6} = 12.1 \rightarrow 26$ STUDS

CHECK $a = 1"$: $a = \frac{177}{(0.85)(4)(110.7)} = 0.47" < 1"$ ✓ OK.

∴ SELECT W21 × 48 w/ 26 STUDS

CHECK SELF-WT ASSUMPTION

- ASSUMED 5 PSF

$(5)(10.6) = 53$ PLF > 48 PLF ✓ OK

COMPARE TO EXISTING MEMBER

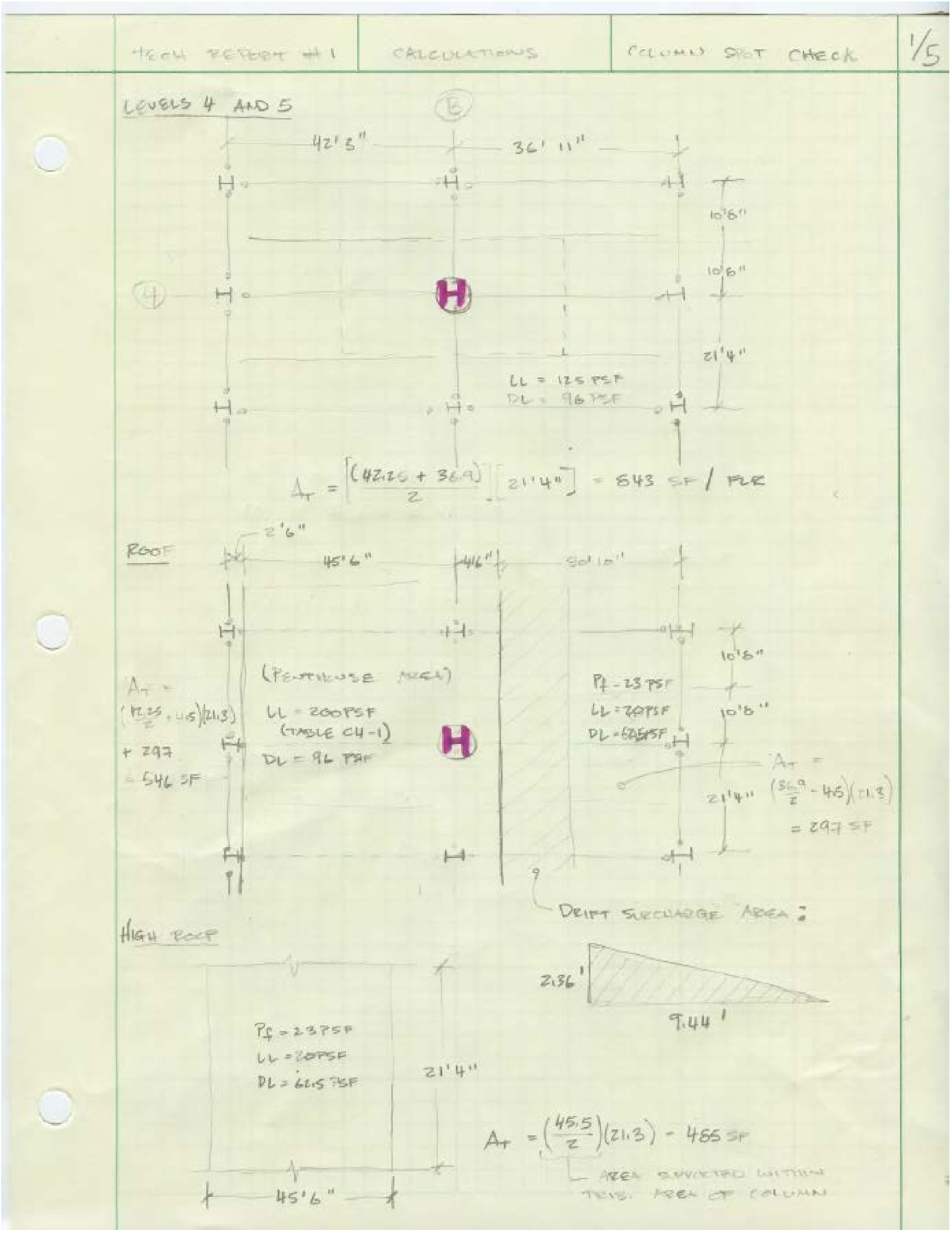
- DESIGN USED W24 × 55 w/ 28 STUDS

$I_x = 1350$ $\Sigma Q_n = 409$

∴ TRIAL MEMBER IS STILL UNDERSIZED

└ DISCUSS POSSIBLE REASONS IN REPORT

A.6 Typical Column Spot Check Calculations



TECH REPORT #1	CALCULATIONS	COLUMN SPOT CHECK	2/5

ROOF LIVE LOAD REDUCTION

- DETERMINE TRIBUTARY AREA OF ROOF

$A_T = 485 + 297 = 782$ SF

- DETERMINE REDUCTION FACTORS

$A_T = 782 > 600$ SF $\therefore R_1 = 0.6$ (§4.9.1)

FLAT ROOF $\therefore R_2 = 1.0$ (§4.9.1)

- REDUCED ROOF LIVE LOAD

$L_R = L_o R_1 R_2 = (20)(0.6)(1.0) = 12$ PSF ≥ 12 PSF (EQ. 4-2)

$\therefore L_R = 12$ PSF

FLOOR LIVE LOAD REDUCTION

$L_o = 125$ PSF > 100 PSF

$\therefore$ 20% REDUCTION ON 2+ FLOORS (§4.8.2)

$\therefore L = (125)(0.8) = 100$ PSF

ACCUMULATION OF LOADS

- HIGH ROOF ($A_T = 485$ SF)

$L_R = 12$ PSF $S = 22$ PSF

$D = 46.5$ PSF (SLAB + DECK)
$+ \ 10$ PSF (ROOFING MAT'L)
$+ \ 5$ PSF (FRAMING)
$= 62.5$ PSF

$P_{LL} = (12)(485) = 5.82$ K

$P_{SL} = (22)(485) = 11.2$ K

$P_{DL} = (62.5)(485) = 30.3$ K

TECH REPORT #1	CALCULATIONS	COLUMN SPOT CHECK	3/5

• ROOF LEVEL ($A_{PENTHOUSE}$ = 546 SF, A_{ROOF} = 297 SF)

L_R = 12 PSF L_{PH} = 200 PSF (NOT YET REDUCED)

D_R = 62.5 PSF D_{PH} = 96 PSF

S = 22 PSF

$P_{LL} = (12)(297) + (0.8)(200)(546) = 90.9$ K

$P_{SL} = (22)(297) + 4.25^{*} = 6.84$ K

$P_{DL} = (62.5)(297) + (96)(546) = 71.0$ K

* DRIFT SURCHARGE

$\left[\frac{1}{2}(2.36)(9.44)\right]\left[21.3\right]\left[\gamma_{snow} = 17.9\right] = 4.25$ KIPS

• FLOORS 4 AND 5 (A_T = 843 SF)

L = 100 PSF   D = 96 PSF

$P_{LL} = (100)(843) = 84.3$ K

$P_{DL} = (96)(843) = 80.9$ K

TOTALS

	DEAD	LIVE	SNOW	
HIGH ROOF	30.3	5.62	11.2	(KIPS)
ROOF LEVEL	71.0	90.9	6.84	(KIPS)
FLOOR 5	80.9	84.3	—	(KIPS)
FLOOR 4	80.9	84.3	—	(KIPS)
TOTAL	263.1	265.3	18.1	(KIPS)

LOAD COMBINATION

$P_u = 1.2D + 1.6L + 0.5S$

$= 1.2(263.1) + 1.6(265.3) + 0.5(18.1)$

$P_u = 749.25$ KIPS

TECH REPORT #1	CALCULATIONS	COLUMN SPOT CHECK	4/5

COLUMN EFFECTIVE LENGTH

- DETERMINE G FOR STRONG AXIS

FMC ∴ USE NOMOGRAPH WITH GIRDER LENGTH DOUBLED PER GESCHWINDER / DISQUE

TRIAL MEMBER BASED ON AXIAL LOAD

$P_u = 749.25\ K$, $KL = 16'$

∴ TRY W14×90 ($\phi_c P_n = 978\ K$, $I_x = 999\ in^4$, $I_y = 362\ in^4$)

$$G_x = \frac{\Sigma \frac{I_c}{L_c}}{\Sigma \frac{I_G}{2L_G}} = \frac{\left[\frac{999}{15.3}\right]\times 2}{\frac{1510}{(357)(2)}} = 5.3$$

- DETERMINE G FOR WEAK AXIS

APPROXIMATE I_G BASED ON I_{REQD} FOR $\Delta_{LL} \le \frac{L}{360}$

$L / 360 = (21.3)(12) / 360 = 0.71''$

$$\Delta_{LL} = 0.71 = \frac{P\ell^3}{48EI} = \frac{(61.6 + 70.5)(21.3)^3(12)^3}{(48)(29000)(I_{REQD})}$$

∴ USE $I_G = I_{REQD} \approx 2230\ in^4$

$$G_y = \frac{\Sigma \frac{I_c}{L_c}}{\Sigma \frac{I_G}{2L_G}} = \frac{\left[\frac{362}{15.3}\right]\times 2}{\frac{2230}{(21.3)(2)}} = 0.9$$

- DETERMINE EFFECTIVE LENGTH

$G_x = 5.3 \longrightarrow K_x = 2.2$ PER NOMOGRAPH

$G_y = 0.9 \longrightarrow K_y = 1.25$ PER NOMOGRAPH

W14×90 $\longrightarrow r_x / r_y = 1.66$

$$\frac{K_x}{r_x/r_y} = \frac{2.2}{1.66} = 1.33 > K_y = 1.25$$

∴ $(KL)_{EFF} = (1.33)(15.3) = 20.4' \longrightarrow$ USE 21'

TECH REPORT #1	CALCULATIONS	COLUMN SPOT CHECK	5/5

SELECT TRIAL MEMBER

- TRY W14 x 90

$(KL)_{EFF} = 21' \rightarrow \phi_c P_n = 848$ KIPS

$\phi_c P_n = 848 > 749.25$ KIPS

$\therefore$ SELECT W14 x 90 TO SATISFY GRAVITY LOAD

COMPARE TO EXISTING MEMBER

- DESIGN USED W14 x 193
- ADDITIONAL CAPACITY REFLECTS DESIGN FOR LATERAL LOAD, AS EXPECTED.