American Eagle Outfitters
Quantum II Corporate Headquarters

Technical Report #1:
Structural Concepts & Existing Conditions

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Executive Summary

The Quantum II office building was built as a part of the commercial development The Southside Works in Pittsburgh Pennsylvania. The 6 story 186,000 square foot office suite was later purchased, and is currently being fitted-out by the American Eagle Outfitters Corporation. Being that the structure was built without a contracted tenant the designers took steps to make it versatile and attractive to business. The building is conveniently located just outside the confusion of the city where there is more space and parking. The structure has a contemporary shape and look utilizing a brick and glass curtain wall. The building fits in well with its other modern neighbors in the new district. The engineers strived to keep as many options for fit-out as open as possible. To achieve this they utilized composite slab floor decks, large bays, and moment frame connections.

This report gives a look at conditions and criteria of design, as well as a review of structural concepts. The report contains outlines of loads live, dead, snow and lateral. It describes foundations, framing, flooring, and lateral load resistance. A brief analysis of existing structural systems is also provided to express that current design is adequate.

Codes and requirements

The structure was design according to the 2003 International Building Code (IBC). The calculations of this report were performed according to the procedures of ASCE7-02, from where IBC 2003 adopted much if its provisions

Loads

Gravity loads

Dead Loads

The following loads were derived from information in the building's structural mechanical and fit-out plans.

Dead Load (psf)	
Typ. Floor Slab	57
Roof Slab	57
Exterior Walls	20
Mechanical	10
Miscelaneous	5

Live Loads

The following loads were taken from ASCE7-02 with the exception of the floor load which has been enlarged to account for the variability of fit-out. Loads are factored appropriately according to ASEC7-02 when designing structural members.

Live Loads (psf)	
Roof	30
All Floors	100
Stairs and Ramps	100
Balconies	100

Snow Loads

The following load comes from the ASCE7-02 calculation for a fully exposed category III building in exposure B, See appendix section A for full calculations.

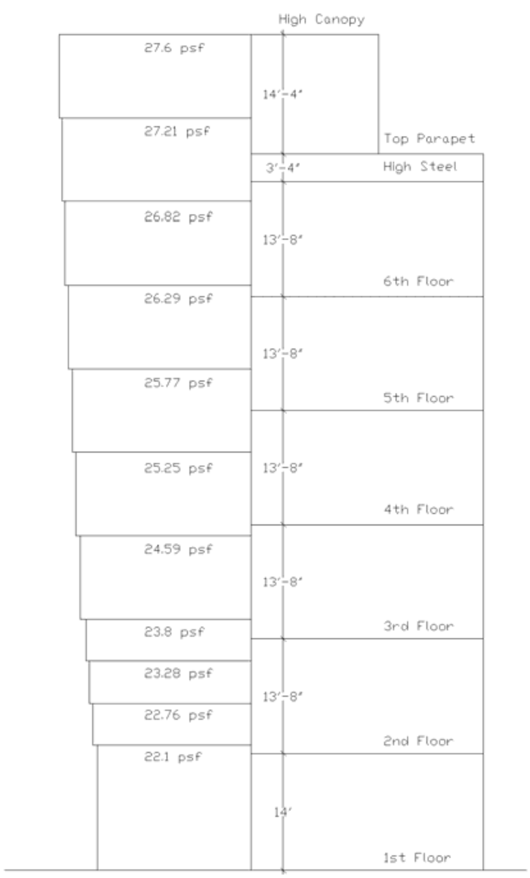
Flat Roof Snow Load (P_f) = 20.8 psf

Lateral loads

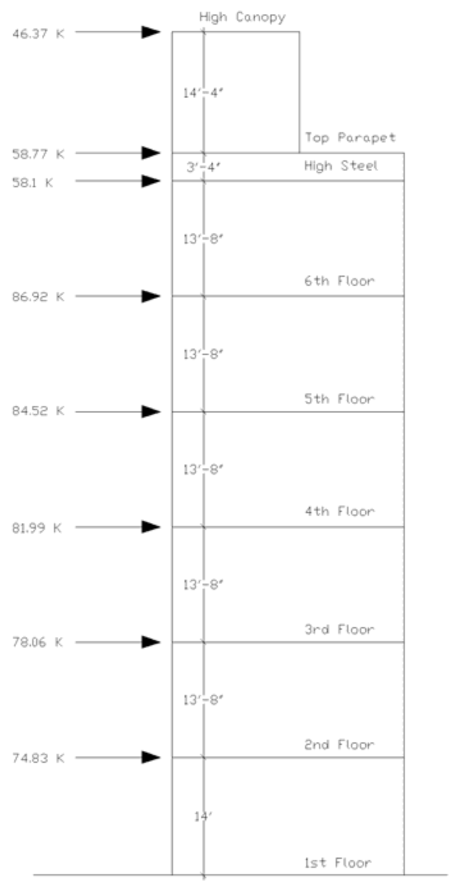
All Lateral Loads, wind and seismic, were developed with ASCE7-02. Below are results of the calculations, full calculations are in the appendix sections B and C.

Wind

z (ft)	Kz (T6-3)	qz	P (psf) Windward	P (psf) Side	P (psf) Leeward EastWast	P (psf) Total
0-15	0.57	11.554	10.916	-9.983	-11.186	22.1020
20	0.62	12.567	11.571	-10.556	-11.186	22.7571
25	0.66	13.378	12.095	-11.014	-11.186	23.2812
30	0.70	14.189	12.619	-11.473	-11.186	23.8053
40	0.76	15.405	13.405	-12.161	-11.186	24.5914
50	0.81	16.418	14.061	-12.734	-11.186	25.2466
60	0.85	17.229	14.585	-13.193	-11.186	25.7706
70	0.89	18.040	15.109	-13.651	-11.186	26.2947
80	0.93	18.851	15.633	-14.110	-11.186	26.8188
90	0.96	19.459	16.026	-14.454	-11.186	27.2119
100	0.99	20.067	16.419	-14.798	-11.186	27.6049



Wind Load Distribution

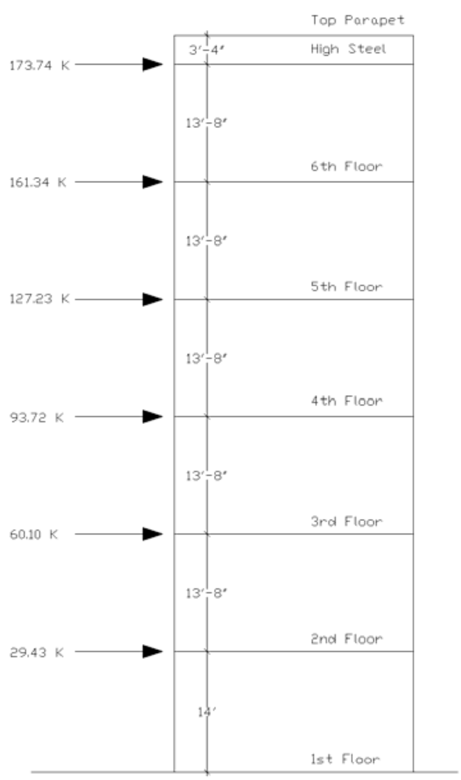


Wind Load Per Floor

Seismic

Level	Wx (K)	hx (ft)	Wxhx^1.07	Cvx	Fx (K)	Mo (ft-K)
Roof	2077	82.333	397485.39	0.258455	167.0806	13756.24
6	2442	68.666	389225.72	0.253084	163.6087	11234.35
5	2442	55	306955.86	0.19959	129.027	7096.486
4	2442	41.333	226113.02	0.147024	95.04523	3928.504
3	2442	27.666	147153.46	0.095683	61.85506	1711.282
2	2442	14	70997.794	0.046164	29.84349	417.8089
1	1760	0	0	0	0	0
SUM:			1537931.2	1	**646.46**	**38144.68**

V = 646.46 K



Seismic Loads

Description of Structural System

Foundation

The main foundation element is concrete piles. Column sits on a pile caps covering varying numbers of piles. Concrete grade beams run along the perimeter. All foundational elements are made of 3ksi concrete.

Framing

The structure is comprised of conventional steel framing, made with a combination of A36 and A572-50 grade steel components. Large bays, approximately 30’ x 30’ have been created in order to keep the floor plan as clear as possible to increase possibilities in fit-out. Each bay is filled with two beams spaced 10 feet apart.

Slabs

Each 31,000 square foot story of the structure consists of a composite floor deck of concrete poured over metal decking. 3” 20 gauge metal deck sits under 3” of 4ksi concrete. This system can span longer distances that competing floor systems and is utilized here to assist in increasing the bay size and opening the floor plan.

Lateral Resisting System

Moment frames used to resist the wind and seismic loads described above. This utilization avoided blocking bays with alternate methods of lateral support, mainly cross bracing or shear walls. This was, again to keep floor plan as open as possible for tenant fit-out.

Structural Analysis

Beam

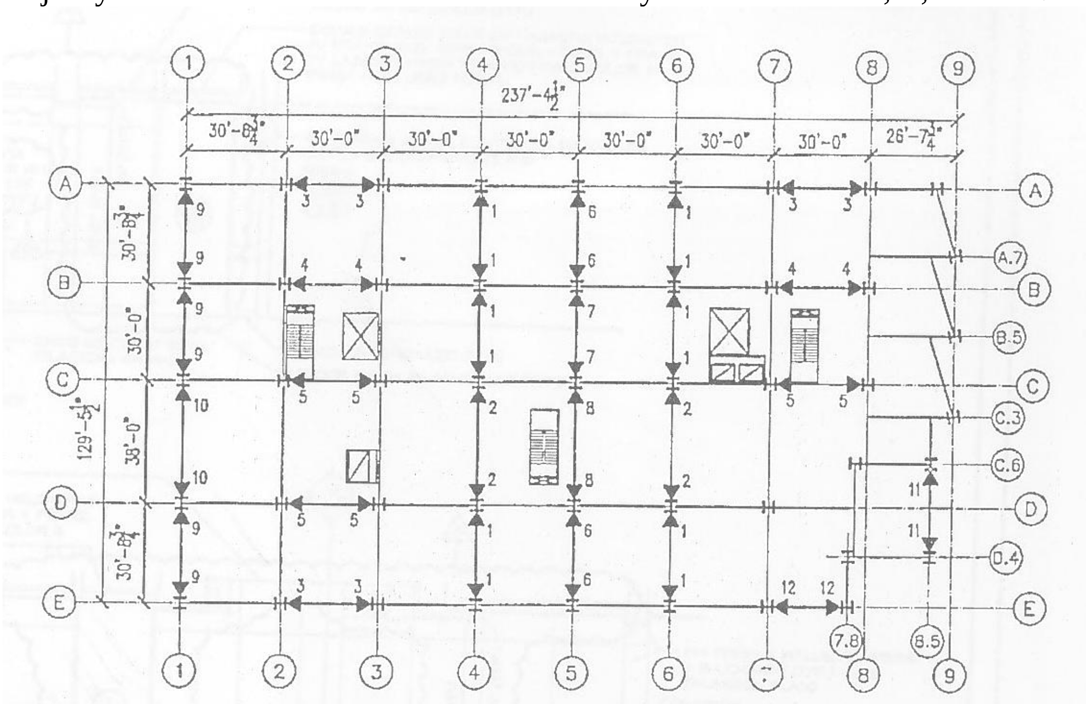
As stated before a composite floor deck was utilized to increase beam span and spacing thus opening the floor plan to more possibilities. A calculation, located in appendix section D, details that the moment capacity of a standard beam is larger then the maximum applied moment from the structural loads, thus verifying the existing design as sufficient. All loads were appropriately factored per ASCE7-02

Column

Column spot check was performed on an interior column (B-4, see plan below) at ground level. The calculations, located in appendix section E, verify that the column specified is capable of supporting the floor loads above it. All loads were appropriately factored per ASCE7-02

Lateral

To resist the lateral forces of wind and seismic shown above the structure utilizes moment frame connections. A plan of the moment connections is shown below. With North pointing to the right we can see that most of the East-West lateral movement(where the most extreme cases occur) will be taken up by frames on beam lines 1,4,5 and 6. The majority of North-South movement will be held by frames in lines A, B, C and E.



PLAN - MOMENT CONNECTIONS

Conclusion

The building does a fine job of providing a pliable space for a tenant. This report made a basic analysis of the loads applied to the steel structure and expressed the systems adequacy. Future, more in depth analysis of foundations as well as the lateral system may prove interesting. Possible alternatives, especially in the lateral system, could be explored.

APPENDIX

Appendix A: Snow Load

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Snow Load

F 7-1: $P_g = 30$ psf

T 7-4: $I = 1.1$ (Category III)

T 7-3: $C_T = 1.0$

T 7-2: $C_e = 0.9$ (Exposure B) (fully exposed)

eq 7-1: $P_f = 0.7\, C_e \cdot C_T \cdot I \cdot P_g$

$P_f = 0.7 \cdot 0.9 \cdot 1.0 \cdot 1.1 \cdot 30$ psf

$P_f = 20.79$ psf $\Rightarrow$ 20.8 psf

Appendix B: Wind Load

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Wind Load $\quad h > 60'$

T 1.1: Occupancy Category III

T 6.1: $I = 1.15$ (non hurricane Region)

F 6-1: $V_0 = 90$ mph

T 6-4: $k_d = 0.85$

eq. 6.3: $k_{zT} = (1 + k_1 \cdot k_2 k_3)^2 = 1$

$k_h = 0.945$ @ (mean roof height) = 85'

k_z = height @ specific point

eq. 6-15: $q_h = 0.00256\, k_h \cdot k_{zT} \cdot k_d \cdot V^2 \cdot I \left(\frac{lb}{ft^2}\right)$

$q_h = 0.00256\,(0.945)\; 1 \cdot 0.85\,(90\,\text{mph})^2\, 1.15 =$

$q_h = 19.15462$ psf

$q_z = 0.00256 \cdot k_z \cdot K_{zt} \cdot k_d\; V^2\; I$

$q_z = 0.00256\; k_z \cdot 1 \cdot 0.85\,(90\,\text{mph})^2\, 1.15 =$

$q_z = 20.26944 \cdot k_z$ psf

Chart q_z's

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Wind Load

T 6-2: (exp. B) $\quad c = 0.3 \quad \ell = 320' \quad \bar{\epsilon} = \frac{1}{3.0}$

eq 6-7: $\quad L_{\bar{z}} = \ell \left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320 \left(\frac{51}{33}\right)^{\frac{1}{3.0}} = 370$

$\bar{z} = 0.6 (85') = 51'$

eq 6-6: $\quad Q = \sqrt{1 / 1 + 0.63 \left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}} = \sqrt{1 / 1 + 0.63 \left(\frac{240' + 85'}{370'}\right)^{0.63}}$

$Q = 0.7954$

eq 6-5: $\quad I_{\bar{z}} = c \left(\frac{33}{\bar{z}}\right)^{\frac{1}{6}} = 0.3 \left(\frac{33}{51}\right)^{\frac{1}{6}} = 0.279$

eq 6-4:

$$G = 0.925 \left(\frac{1 + (1.7 \cdot 3.4)\ I_{\bar{z}}\ Q}{1 + (1.7 \cdot 3.4)\ I_{\bar{z}}}\right)$$

$$G = 0.925 \left(\frac{1 + (1.7 \cdot 3.4)\ 0.279 \cdot 0.7954}{1 + (1.7 \cdot 3.4)\ 0.279}\right) = 0.808$$

F 6-6: $\quad C_p = $

0.8 windward

-0.7 side

-0.25 NS leeward $\quad L/B = \frac{240'}{132'}$

-0.5 EW leeward $\quad L/B = \frac{132'}{240'}$

F 6-5: $\quad GC_{pi} = \pm 0.18$ (enclosed building)

eq 6-23: $\quad p = q (G \cdot C_p) - q_h (GC_{pi}) \qquad q = q_z$ (windward)

$= q_h$ (leeward & side)

wind $\quad p = q_z (0.808 \cdot 0.8) - 19.15(-0.18)$

side $\quad p = q_z (-0.7 \cdot 0.808) - 19.15 (0.18)$

EW lee $\quad p = 19.15 (-0.5 \cdot 0.808) - 19.15 (0.18)$

NS lee $\quad p = 19.15 (-0.25 \cdot 0.808) - 19.15 (0.18)$

Appendix C: Seismic Load

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Seismic Loading

T 9.1.3 : Seismic Use Group II (Occ. Cat. III)

T 9.1.4 : $I = 1.25$ (Seis. Use Grp III)

T 9.4.1.2 : Site Class D

F 9.4.1.1a : $S_S = 0.127$

F 9.4.1.1b : $S_1 = 0.054$

T 9.4.1.2a : $F_a = 1.6$

T 9.4.1.2b : $F_v = 2.4$

eq: 9.4.1.2.4-1 : $F_a \cdot S_s = 1.6 \cdot 0.127 = S_{MS} = 0.203$

eq: 9.4.1.2.4-2 : $F_v \cdot S_1 = 2.4 \cdot 0.054 = S_{M1} = 0.129$

eq: 9.4.1.2.5-1 : $\frac{2}{3} S_{MS} = \frac{2}{3}\, 0.203 = S_{DS} = 0.135$

eq: 9.4.1.2.5-2 : $\frac{2}{3} S_{M1} = \frac{2}{3}\, 0.129 = S_{D1} = 0.086$

T 9.4.2.1a : Seismic Design Cat. A

T 9.4.2.1b : Seismic Design Cat. B

T 9.5.2.2 ; $R = 3.5$ (Steel Moment Frame)

T 9.5.5.3.2 : $C_T = 0.028$ $x = 0.8$ (Steel)

eq 9.5.5.3.2-1 : $T = C_T h_n^{\,x} = 0.028\,(85')^{0.8} = 0.9788$

$C_S = \frac{S_{DS}}{R/I} = \frac{0.135}{3.5/1.25} = 0.0482 \leftarrow$

$C_{s\,max} = \frac{S_{D1}}{T(R/I)} = \frac{0.086}{0.9788\left(\frac{3.5}{1.25}\right)} = 0.0314$

$k = 1 + \frac{0.632 - 0.5}{2} = 1.07$

$C_{s\,min} = 0.044 \cdot I \cdot S_{DS} = 0.044\,(1.25)\,0.135 = 0.0074$

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Seismic Loading

TYP Floor Area = $31{,}000\ ft^2$
TYP perimeter = 770 ft

Calc. Dead Load of Structure

Exterior wall: $20psf \times 13'\text{-}8'' \times 770\ ft = 210\ k$

TYP Floor: $(2\text{-}6) = 31{,}000 ft\ (57+10+5) psf \cdot \frac{1k}{1000p} + 210k = 2442\ k$

Roof: $31{,}000\ ft^2 \cdot (57+10) psf \cdot \frac{1k}{1000p} \pm 2077\ k$

1st floor $= \left(31{,}000 ft^2 \times 4'' \times \frac{1'}{12''} \cdot 150\ pcf\right) \frac{1k}{1000p} + 210k = 1760k$

$\Sigma W = 5(2442k) + 2077k + 1760k = \cancel{13{,}412k}\ 16{,}047k$

$\cancel{V = C_s \cdot W = 0.0482 \cdot 13412k} =$

$V = C_s \cdot W = 0.0482 \cdot 16047\ k = 773.46\ k$

Appendix D: Beam Spot Check

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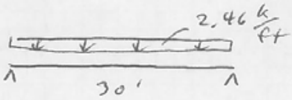
Beam Spot Check

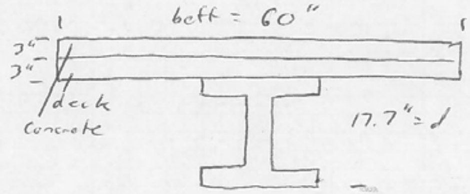
W18X75 $A = 10.3 in^2$ deck = 3" deck + 3" concrete
$f_y = 50 ksi$ $d = 17.7"$ 30' span 10' Spacing

load = 1.2D + 1.6L = 1.2(72) + 1.6(100) = 246.4 psf
w = 246.4 psf · 10' = 2.464 k/ft



$M_u = \frac{wL^2}{8} = \frac{(2.464 \frac{k}{ft})(30')^2}{8} = 277.2'k = M_u$

beff $\begin{cases} \frac{30'}{4} = 90" \\ \frac{10'}{2} = 60" \leftarrow \text{controls} \end{cases}$



T = C

10.3 in² · 50 ksi = 0.85 · 60" · 3" · 4 ksi
515 k < 612 k ok Tension controlled

515 k = 0.85 (60") · a · 4 ksi ⇒ 2.52"

$M_n = 515k \cdot \left(\frac{17.7"}{2}\right) + 515k\left(6" - \frac{2.52"}{2}\right) = 583.14 'k$

$\phi M_n = 0.9(583.14'k) = 524.82 'k$

$524.8'k = \phi M_n > M_u = 277.2'k$

Beam ok

Appendix E: Beam Spot Check

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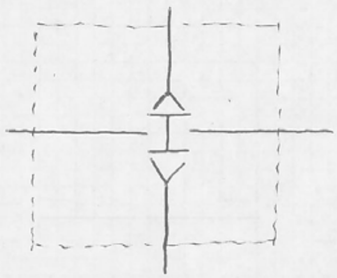
Column Check

Col. B-4



30' x 30' Trib area

Floor loads:

TYP: $1.2(72\,psf) + 1.6(100\,psf) = 246.4\,psf$

roof: $1.2(72\,psf) + 1.6(30\,psf) = 134.4\,psf$

Col. Loads.

$Typ = 246.4\,psf \cdot 30' \cdot 30' = 221.76\,k$

$Roof = 134.4\,psf \cdot 30' \cdot 30' = 120.96\,k$

Col. load @ ground level $= 5(221.76) + 120.96 = 1229.76\,k$

$\underline{P_u = 1229.76\,k}$

W 14X120

$kL = 14'$

fails in Y

AISC T: 4-2 $\quad \phi P_n = 1290\,k > 1229\,k = P_u$

Col. ok