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PSU AE STRUCTURAL OPTION

Lancaster County Bible Church

Manheim, PA

Technical Report #1

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Executive Summary:

This technical report investigates the existing conditions of Lancaster County Bible Church. Critical structural components of the structure, lateral bracing systems, columns, floor, and foundations will be investigated. Images of the construction documents will be provided to aid in orientation within the building and allow for a more comprehensive understanding of Lancaster County Bible Church structural systems. I will begin with a brief description of Lancaster County Bible Church and the structural systems that can be found in the building. Preceding the description I will be a discussing codes and the materials used to construct Lancaster County Bible Church. The latter portion of this report is reserved for displaying my results of a seismic and wind analysis and the conclusion that I drew from my structural analysis of Lancaster County Bible Church. Appendices are attached to show detailed calculations of my results.

Building Description:

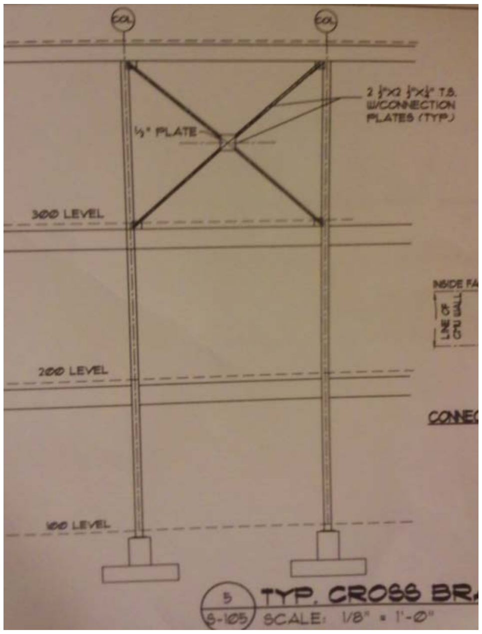
Lancaster County Bible Church was originally constructed in 1988 to serve the religious needs of the local populous. Since 1988 the church has experience tremendous growth in its weekend service attendance which facilitated the need for expansion to the original structure. A 77,833 square foot addition including a 2400 seat auditorium was added to the church in 2003. Work began shortly after the completion of this addition on a 75,586 square foot addition which is now nearly complete. The focus of this report is on the structural systems of the two additions to the existing facility.

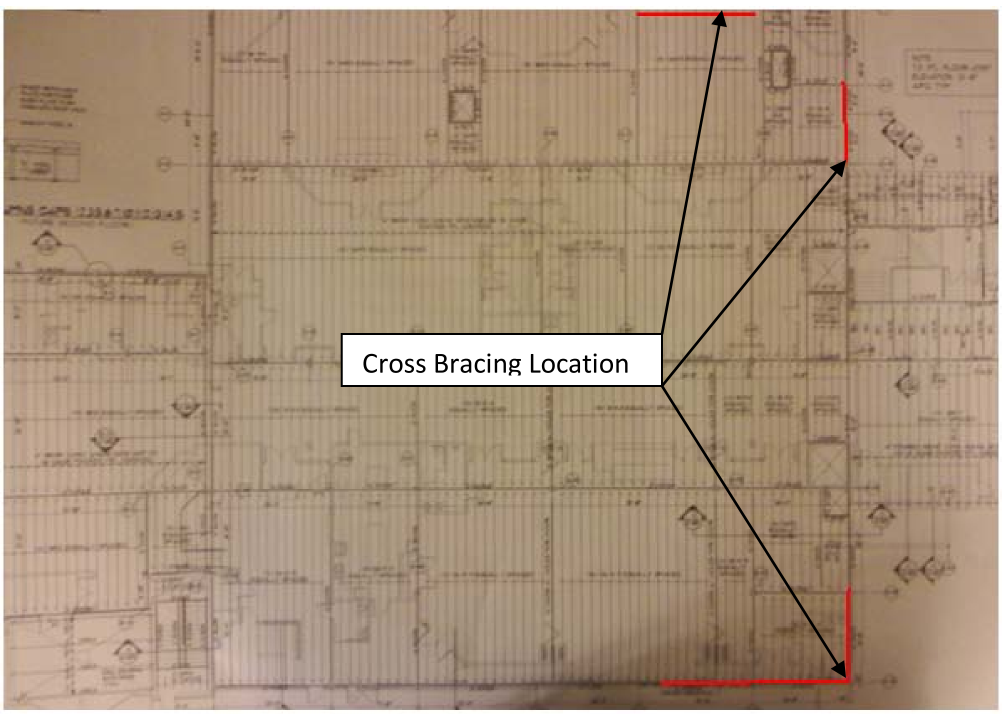
Structural System Discussion:

Foundations: Lancaster County Bible Church's foundations are comprised of different sized spread footings to support structural steel columns and continuous concrete footing s to support load bearing concrete block walls. These footings are designed to bear onto 4000 P.S.F. soil and vary in size from an F20 (2'-0" x 2'-0" x 1'-0") to an F110 (11'-0" x 11'-0" x 2'-0"). Reinforcing bar sizes in the spread footings vary from a #4 bar to a #7 bar with the larger spread footings getting larger reinforcing bars. (3) #4 reinforcing bars are used to reinforce the continuous concrete footings. Additional reinforcing is provided by #6 dowels with a 4" hook spaced at 8" O.C. Concrete used to construct the foundations must meet a minimum strength of 3000 psi. The bottom of all exterior footings must be constructed at a minimum of 36" below finish grade.

Gravity System: Lancaster County Bible Church's typical flooring system is a 4" concrete deck poured on top of 26 gauge 1 ½" metal decking. Slab reinforcing is achieved through 6 x 6, 10/10 welded wire mesh. The concrete slab is supported by steel joist that vary in size from 8K1 for short spans to 36 LH 12 for especially long spans. Metal trusses are also used to support the roof loads in the auditorium space. These metal trusses were specially designed and span 118'-0 and were appropriately sized at 72 DHL 16. Steel beams carry the floor truss load to columns which in turn distribute the floor loads to foundations. The column sizes vary throughout the structure and depend greatly upon the tributary area of the column and the columns overall height. Typically columns start at foundation level and continue three stories vertically until they reach the roof level. A few columns are only as long as the story height. Bay sizes vary throughout the structure with the most commonly sized bay being 31'-0" x 38'-0". However, being that Lancaster County Bible Church is a multi-purpose facility it exhibits mixed bay sizes that can be as small as 18'-0" x 19'-0" and as large as 190'-0" x 162'-0".

Lateral System: Structural steel bracing was used at Lancaster County Bible Church to resist lateral loads. 2 ½" x 2 ½" x ¼" T.S. steel is welded to a few select exterior columns. Flat steel plate is welded to the cross members to further increase the cross bracing strength. Cross bracing is typically placed in the corner of the building and the bracing alternates floors.





Code Review/Reference Material:

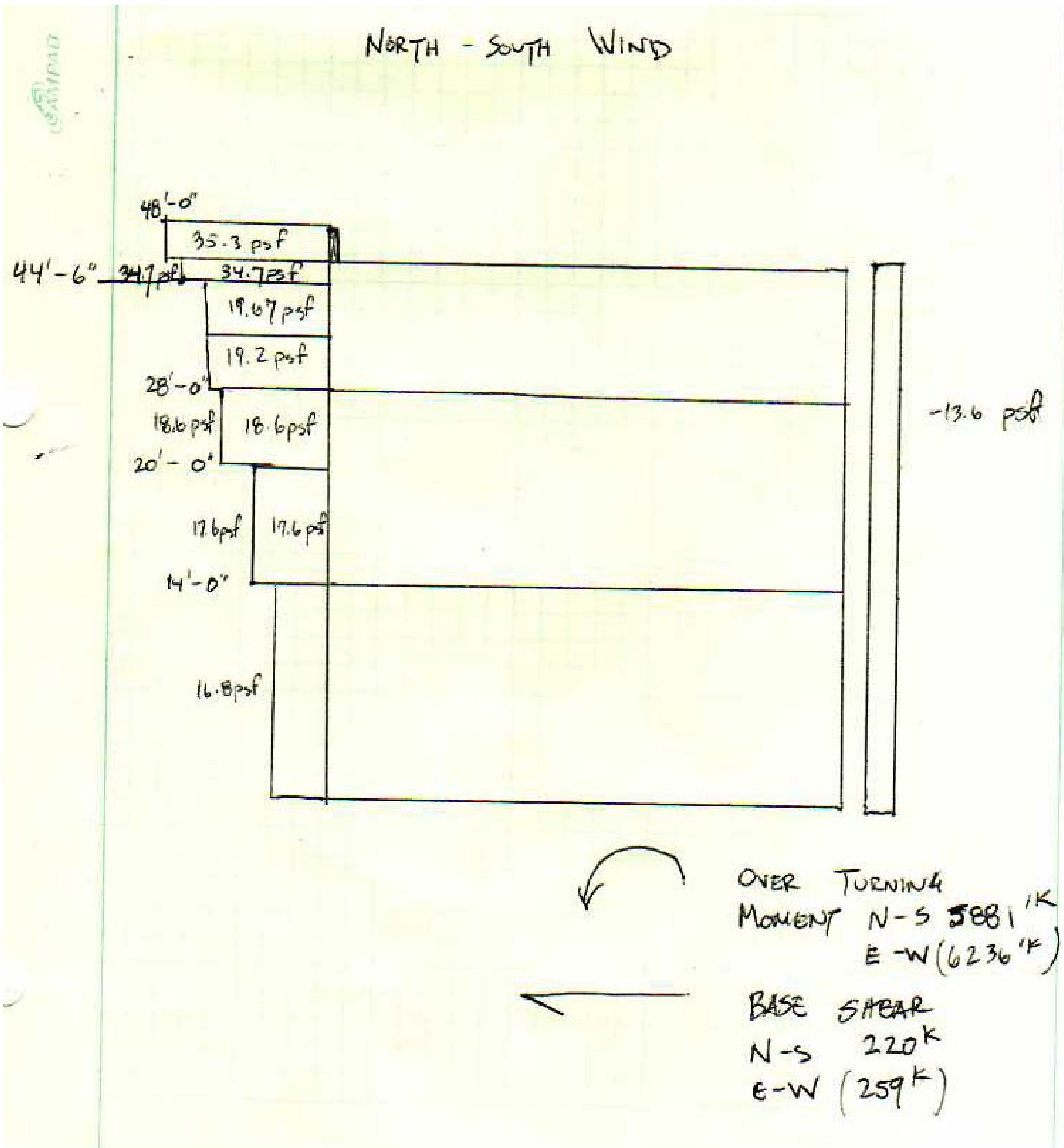
i. 2006 International Building Code
ii. (ACI 318-08) Building Code Requirements for Structural Concrete
iii. Allowable Steel Design, 13th Edition, American Institute of Steel Construction
iv. (ASCE - 07) Minimum design loads for Buildings and other Structures

Gravity Loads and Design Loads:

Loading	Design Value
Dead Load	
Floor Dead Load	50 psf
Partitions	20 psf
Framing	8 psf
Ceiling	3 psf
Mechanical Duct Work	3 psf
Roof Dead Load	15 psf
Live Load	
Corridor	100 psf
Stairs	100 psf
Office	60 psf
Storage Room	80 psf
Snow Load	
Pg	30 psf
Pf	17 psf

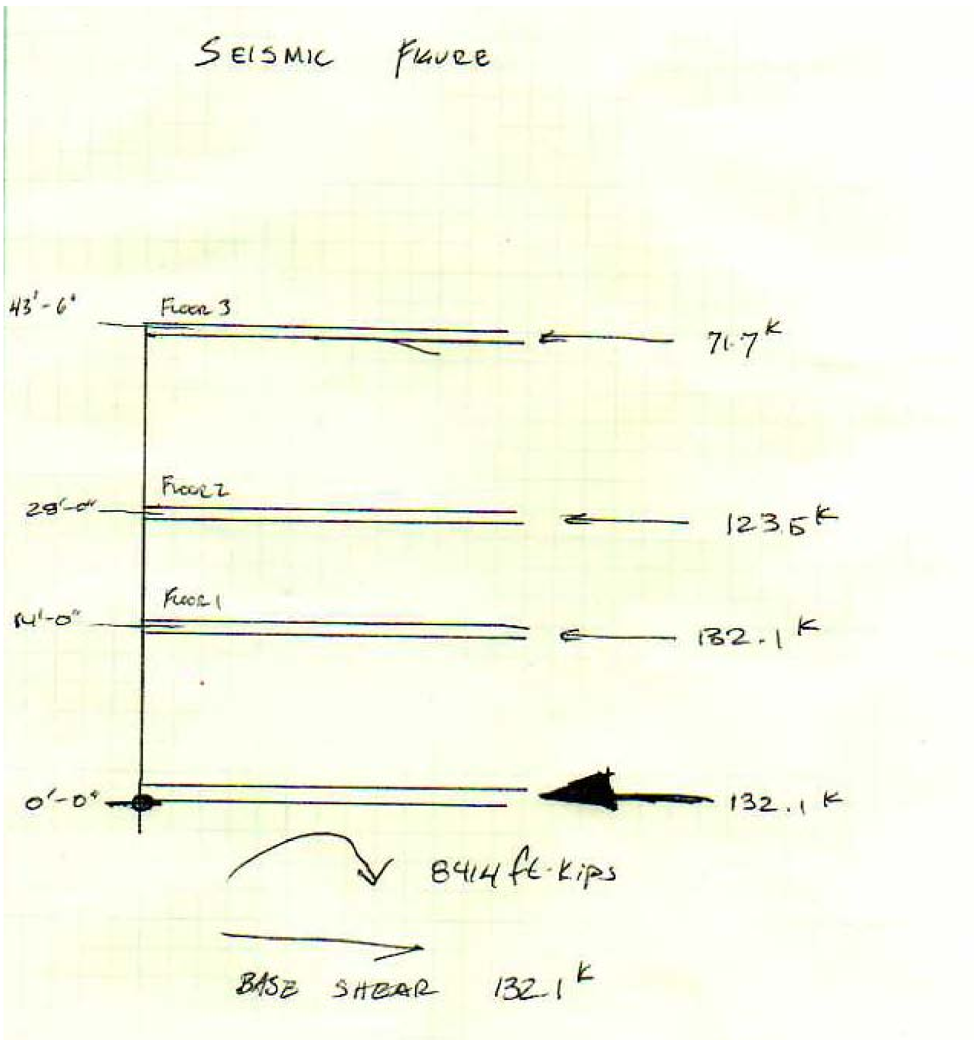
Wind Loading:

The shape of Lancaster Bible Church poses many problems for a basic wind analysis. This particular structure exhibits cantilevers, multiple roof elevations, and even a courtyard. It was assumed that the building was a perfect rectangle when I performed my analysis which is a stretch of the truth. Surely my wind loading calculations can be further exacted for Technical Report 2. An exposure category C and a site basic wind speed of 100 M.P.H. were used for my calculations. The figure below depicts the leeward and windward wind forces and the subsequent wind shear and wind moment. More detailed calculations can be found in appendix A.



Seismic Loading:

The seismic loads were calculated using the equivalent lateral force procedure in accordance with ASCE 7 -05. Seismic floor weight, the equivalent lateral forces per floor and additional seismic loading properties can be found in appendix B. The figure below illustrates the seismic forces that I found upon completion of my seismic investigation.



Spot Check:

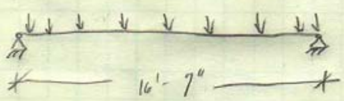
In order to better understand the structural system of Lancaster County Bible Church a spot check was performed on a typical girder. Checking the capacity of this member allows me to check my assumptions against the assumptions made by the designer. I decided to check the flexure capacity of a wide flange girder in the flooring system of the 200 level. I concluded that the beam was seeing 104.5 Ft-kips from floor loads which is well under its nominal moment capacity of 183.0 Ft-kips. Upon further investigation I found that the beam was very close to its nominal moment capacity under ASD design method.

SPOT CHECK

PARTIAL 200 LEVEL BEAM

W18 x50

TRUSS LOADS



$A_T =$

$\dfrac{38'-8"}{2} \times \dfrac{25'-6.5" + 2'-11.5" + 9}{2}$

$\dfrac{39'-4}{2} \times \dfrac{38'-8"}{2} = 19'-8" \times 19'-7" = 380\ ft^2$

* ASSUME UNIFORM LOAD

LIVE LOAD = 60 psf

DEAD LOAD = 50 psf

$1.2(D) + 1.6(L) = 0.156\ ksf$

$3.04\ KLF = w_u$

$M_n = \dfrac{wl^2}{8} = \dfrac{3.04\,(16.58)^2}{8} = 104.5'^k < \phi M_n\ 183'^k$

∴ ok ✓

Conclusion:

The completion of my first technical report brought up some interesting issues. It seems that there is no simple or practical method of estimating wind loads on this structure. Different roof elevations matched with angled building orientations make a wind analysis seem like a wind estimate. On the next technical assignment more research will need to be devoted to a more accurate wind analysis.

A spot check allowed me to estimate that the designer of this structure most likely used ASD design method in the place of LRFD design method. This opens that opportunity for a redesign of the buildings structural steel system in an attempt to achieve a lighter, cheaper building. More checks of the buildings structural members must be made before it is clear weather ASD was used.

Calculating the roof loads that are placed on the long span trusses in the auditorium will be a priority for the next technical report. Being that the auditorium portion of the church is only one level calculating wind and seismic was not practical. However, the loads that are being carried by the deep long span trusses is very substantial.

To my surprise it was the seismic forces that are the controlling factor at this church. Certainly I thought that Manheim Pennsylvania would emphasize wind gusts over the forces of a earthquake but according to my calculations it was the seismic forces that control this design. It will be interesting to see if a more detailed wind analysis will make the wind forces greater than the seismic.

Appendix A: Wind

Wind Loading	Height (z) Feet	K_z	q_z	P_z (windward) psf	P_z (leeward) psf	Total Force (psf)	Lateral Force (Kips)	Overturning Moment (Ft.-kips)
	North-South Wind							
Parapet (top)	48'-0"	1.08	23.5	35.3	-23.5	58.8	26.1	1207
Parapet (bot.)	44'-6"	1.06	23.1	34.7	-23.1	57.8	33.0	1394
	40'-0"	1.04	22.7	19.67	-13.6	33.3	21.1	791
	35'-0"	1.01	22.0	19.2	-13.6	32.8	29.2	920
2nd Floor	28'-0"	0.97	21.1	18.6	-13.6	32.2	32.7	785
	20'-0"	0.90	19.6	17.6	-13.6	31.2	23.8	405
1st Floor	14'-0"	0.85	18.5	16.8	-13.6	30.4	54.1	379

Total Shear: 220 kips

Total Moment: 5881 ft-k

Wind Loading	Height (z) Feet	K_z	q_z	P_z (Windward) psf	P_z (Leeward) psf	Total Force (psf)	Lateral Force (Kips)	Overturning Moment
	East-West Wind							
Parapet (top)	48'-0"	1.08	23.5	35.3	-23.5	58.8	30.1	1392
Parapet (bot.)	44'-6"	1.06	23.1	34.7	-23.1	57.8	37.0	1563
	40'-0"	1.04	22.7	19.67	-10.2	29.9	21.2	795
	35'-0"	1.01	22.0	19.2	-10.2	29.4	29.2	920
2nd Floor	28'-0"	0.97	21.1	18.6	-10.2	28.8	32.8	787
	20'-0"	0.90	19.6	17.6	-10.2	27.8	23.7	403
1st Floor	14'-0"	0.85	18.5	16.8	-10.2	27.0	53.7	376

Total Shear: 259 kips

Total Moment: 6236 ft-k

Appendix B: Seismic

Seismic Loading Properties	Value	Source
Occupancy Category	I	Drawings
Seismic Importance Factor (I)	1.0	Drawings
Mapped Spectral Response Accelerations		
Short Period (S_S)	0.343	USGS Website
1-Second Period (S_s)	0.086	USGS Website
Site Class		
Spectral Response Coefficients		
Short Period (S_{DS})	0.229	USGS Website
1-Second Period (S_{D1})	0.057	USGS Website
Seismic Design Category (SDC)	C	Drawings
Response Modification Factor (R)	5	ASCE 7-05 Table12.2-1
Approx. Period of the Structure (T_a)		
h_n (ft.)	43'-4"	Drawings
C_t	0.020	ASCE 7-05 Table 12.8-2
X	0.75	ASCE 7-05 Table 12.8-2
T_a	0.346	ASCE 7-05 Eqn. 12.8-7
Long-Period Transition Period (T_L)	6	ASCE 7-05 Fig. 22-15
Seismic Response Coefficient (C_s)	0.033	ASCE 7-05 Eqn. 12.8-2
Exponent Related to the Structure(k)	0.923	ASCE 7-05 12.8.3

Floor	Area	Height Range (Feet)		Slab (psf)	Slab Kips	Partitions (psf)	Partitions Kips	Roof Dead (psf)	Roof Dead Kips	Floor Weight (Kips)
3	19,270	28'-0"	43'-4"	50	964	20	385	15	289	1638
2	25,303	14'-0"	28'-0"	50	1265	20	506	0	0	1771
1	27,869	0'-0"	14'-0"	0	0	20	557	0	0	557.4
Total	72,442									5180

Equivalent Lateral Force Procedure Loading

Floor	Height (Feet)	Weight (kips)	wh^k	Fx (kips)	Vx (kips)	Moment (k-ft)
3	43'-4"	1638	53,100	71.7	71.7	3107
2	28'-0"	1771	38,365	51.8	123.5	3458
1	14'-0	557	6,364	8.6	132.1	1849
Total		3966	97,830	132.1		8414

Appendix C: Snow

SNOW LOADS

$C_e = 1.0$ (TABLE 7-2)

$C_t = 1.0$ (TABLE 7-3)

$I = 0.8$ (TABLE 7-4)

$P_g = 30$ psf (Fig 7-1)

$$P_f = 0.7(C_e)(C_t)(I)(P_g) = 16.8 \text{ psf}$$