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October 8, 2003

STRUCTURAL TECHNICAL REPORT 1:

STRUCTURAL CONCEPTS AND EXISTING CONDITIONS REPORT

First Baptist Church of Doylestown Expansion Project
Doylestown, PA

Introduction/Summary

The Phase One Expansion of the First Baptist Church of Doylestown consists of a gymnasium on top of two classroom floors. It was designed with functionality and budget in mind. This is seen clearly when looking at the structural system; it is very simply laid out with a lot of repetition. The materials used also give an indication of this. Bearing block walls with steel framing are a good reliable and cheap way to construct a building, and so that is what was used. Also, because this was such a small project (around 25,000 SF), there wasn't much need for elaborate designs or innovative spans, everything was kept very straightforward.

The foundation system is a continuous footing around the perimeter of the exterior and interior foundation walls. The footing is 1'-0" deep, and has a minimum width of 2'-0". These are supported by concrete piers spaced at 20'-6" O.C. The piers rest on top of spread footings. This is to support the vertical load of the columns on the gymnasium level. They were designed with a soil bearing capacity of at least 3000 psf in mind. The main structural system is made up of interior and exterior bearing-block walls that rest directly on the foundation. The exterior wall is mostly 16" wide CMU. The interior walls are 8" CMU. Typical floor framing consists of 16K6 steel trusses spaced at 2' O.C. This is, of course, interrupted in the elevator shaft and the stair wells. The first and second floors were laid out and framed almost entirely the same. This was probably due to budget constraints and the need for simplicity in design. The floor is a one-way, poured 4" thick concrete slab on metal deck. Again, this is a very economic method of flooring for a simple steel-trussed system. The gymnasium structural system is different from the lower classroom floors. It is constructed using W18X16 steel columns. These columns are directly in line with the spread footings in the foundation. The roof is framed using ASTM specified A572 Grade 50 Steel. W12X16 beam frame into W18X36 roof girders. A rigid frame provides the building with its primary lateral support. The roofing material is asphalt shingles on metal deck.

Codes/Requirements

The primary code used on the church was BOCA 1996. BOCA 1996 also encompasses loading standards from ASCE 7-95. All materials were specified with ASTM standards.

AISC was used for the steel specifications. The AWS Structural Welding Code was used for welding standards. It was sprinkled to meet NFPA Requirements.

According to BOCA 1996:

Building Use Group:	E, Educational
	A-3, Assembly, Churches, Mixed Use
Building Construction Class:	2C

According to Pennsylvania Labor and Industry:

Building Occupancy Group:	B, Education
	A-1, Assembly

Structure/Framing:

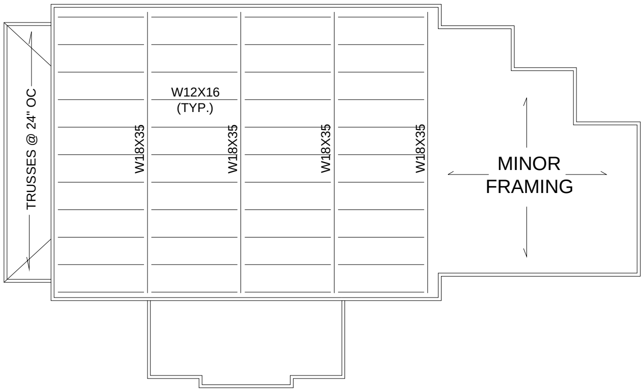
Overall Footprint – Rectangular and simple, approximately 8670 SF per footprint.
Typical Floor-to-Floor height – 11'-4"
Lateral System – A Rigid frame worked into the roof.
Elements –

- Basement and First Floor – CMU block bearing wall with reinforcement on continuous footing - 2' minimum width, 1' depth (*IVANY* Foundation Wall also used). Framed using 16K6 trusses spaced at 2' OC typical. One way, poured 4" slab (w/ 6X6, 10/10 WWF) over 6 mil. Poly V.B. on 4" crushed stone in the basement floor. One way, poured 4" slab on 1 ½" metal deck on the first floor. Exterior CMU's are 16", interior are 8". Concrete piers rest of spread footings under the vertical line of the gymnasium columns. The spread footings are square, inconsistent in depth (12"-18")
- Top (Second/Gymnasium) Floor – W18X60 Steel columns, W12X16 beams (spaced at 6' OC typical) framing into W18X35 girders (spaced at 20'-6" OC typical). Still a one way poured 4" slab.
- Roof – Asphalt shingles on 1 ½" metal deck
- Special – Metal studs (W12X16) frame four large dormers in the roof of the gymnasium to allow in daylight.
- Reinforcement – The continuous footing is reinforced with 2 #4 bars. In the 4'X4'X12" spread footing, 7 - #4 bars each way. In the 4'-6"X4'-6"X14" spread footing, 8 - #4 bars each way. In the 5'-6"X5'-6"X16" spread footing, 8 - #5 bars each way. In the 6'X6'X18" spread footing, 10 - #5 bars each way.
- Concrete - assumed to be 145 pcf, 4 ksi compressive strength
- Masonry – assumed to be 125 pcf and grouted at 24" OC

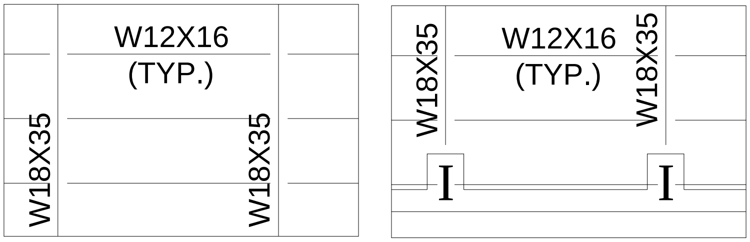
Questions/Issues – the drawings don't readily call out the materials used. Therefore while spot checking and calculating, many assumptions were made. ASD design method is assumed to have been used; this would result in bigger members. The roof

framing plan mentions rigid frame, this is assumed to be the lateral system, however little is know about this particular case.

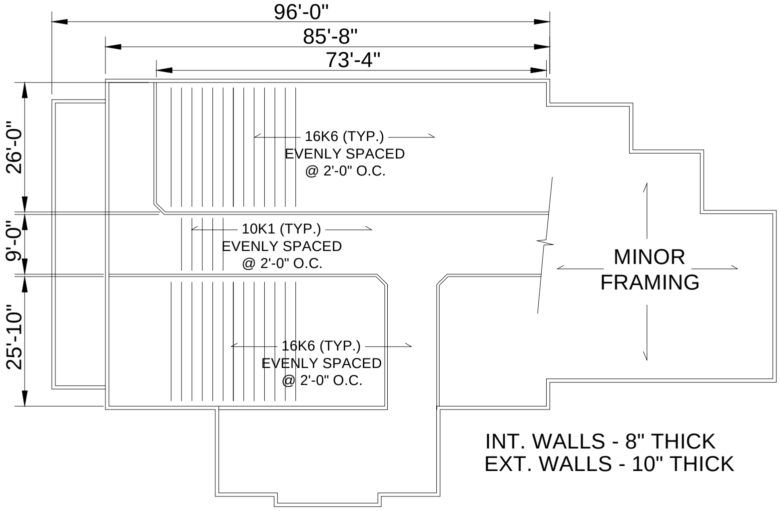
These are representations of the typical form the structural system takes in this project. Again, repetition is a readily observed aspect of this project. These are representations, and are not drawn to scale. Projects drawings are available upon request.



TYPICAL ROOF FRAMING PLAN
NOT TO SCALE



DETAIL VEIW FRAMING PLAN
NOT TO SCALE



TYPICAL FLOOR FRAMING PLAN
NOT TO SCALE

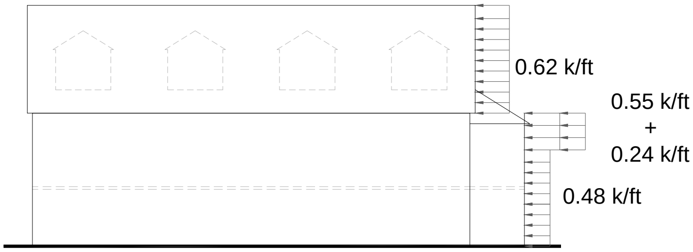
Loading:

GRAVITY LOADING (based on BOCA 1996 and ASCE 7-95):

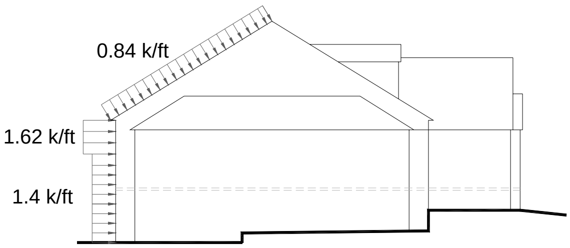
Dead:	20 psf, Roof 76 psf, Floor 51 psf, Interior Partitions 75 psf, Exterior Walls 3350 kip, Total Building DL *See Appendix A for calculations*
Live:	100 psf, Gymnasiums 40 psf, Classrooms 100 psf, Corridors 100 psf, Stairs
Snow:	22 psf *See Appendix B for calculations*

LATERAL LOADING (based on BOCA 1996):

Wind:	See Charts below, Should be the controlling lateral Load (drawings not to scale) *See Appendix C for calculations*



WIND LOADS -
WEST (LONGITUDINAL) VIEW



WIND LOADS -
SOUTH (TRANSVERSE) VIEW

Seismic:	174.14 kips, Seems high, Wind should control in the Philadelphia region *See Appendix D for calculations*
Soil Bearing Capacity	3000 psf as specified on the drawings

LATERAL ANALYSIS

A lateral analysis was not yet made on this structure because so little is known of the system. The only information obtained from the drawings is that a rigid frame is incorporated with the girders, therefore, a moment frame of some type is assumed. If a preliminary analysis were to be done, tributary areas identical to those of the girders could probably be assumed. Perhaps the portal method may be used with a calculation for overturn. Further research is recommended to get comfortably acquainted with the system such that a proper check can be made.

Spot Check of Members

Typical members were spot checked using *EnerCalc*, a structural design program available online. It checks members using member characteristics such as size, length, and strength combined with the appropriate loads given in kip/ft.

Beams
I checked a typical roof beam (W12X16) using its corresponding tributary width, the roof dead load (including the beam self weight), and snow live loads obtained from BOCA 1996 and ASCE 7-95. The allowable design moment in this case was about 25 ft-kips above the actual moment. This seems like an over-design; however there are many factors that could have caused such a differential. One possible reason for the discrepancy is that I may have underestimated the design loads associated with beams. More on this will be discussed at the end of this section. (Calculations available upon request) Deflection was also acceptable using these approximations, on the order of l/600.

Girders
I checked a typical girder (W18X35) using its corresponding tributary width, the roof dead loads (with the girder self-weight added), and the live loads obtained from BOCA 1996 and ASCE 7-95. The allowable moment on this calculation was about 25 ft-kips above the actual moment again. The same reasons apply as to why it could have turned out this way. More will be discussed on this at the end of this section. (Calculations available upon request) Deflection was also acceptable using these approximations, on the order of l/500.

Columns
I checked typical column sizes (W18X60) using their corresponding tributary widths, the roof dead loads (with the girder wt, and the column self-wt. added), and the live loads obtained from BOCA 1996 and ASCE 7-95. The column seemed grossly over-designed. This could be due to the fact that I under-estimated the loads to be carried by the column. The wall sections and drawings didn't provide specific information when assigning labels to material; therefore I had to make many assumptions that were overly conservative in order to maintain a well-planned design. (Calculations available upon request)

Overall, it seems like the numbers I used were overly conservative. There could be a few reasons for this. The first is that Mann-Hughes, in their structural design, typically designs very conservatively for reasons of safety, repetition of member sizes, and overall cost. Another could be that I under-estimated the design loads the member would need to carry. As stated before, the drawings and specifications didn't specify the materials used to a detailed level - this is another possible cause for the discrepancy.

Further Investigation

Soil Capacity
The soil capacity was assumed to be 3000 psf, however, some water table issues arose after construction and the elevator shaft filled with water. A thorough report of the incident is needed to aid in a further understanding of the soil design issues.

Mechanical Systems
Because the mechanical contract was subbed out, a complete set of documents is needed to further refine the loads and placement of mechanical systems in the building.

Lateral System
Further research is needed on the method of lateral resistance used in the church. Perhaps the portal method may be used with a calculation for overturn.

APPENDIX A, DEAD LOAD CALCULATIONS

Based on ASCE 7-95 (referenced in BOCA 1996), and the United Steel Deck (USD) Design Manual. Assumptions* made

DL's:

ROOF = deck + 2x12 + insulation + 5/8" plywood + shingles + steel
= 3 + 5 + 3 + 0.4(5) + 2 + 5
= 20 psf

SNOW = **22 psf**, *SEE APPENDIX B FOR CALCULATIONS*

FLOOR = conc./deck + steel + misc. DL + MEP
= 55 + 5 + 10 + 6
= 76 psf

INTERIOR PARITITIONS = 8" CMU **= 51 psf**

EXTERIOR WALL = 12" CMU
= 75 psf

TRIBUTARY AREAS:

ROOF = 7485 SF

FLOOR 1 = 8,670 SF

FLOOR 2 = 8,670 SF

PERIMETER = 446'-8"

FLOOR-TO-FLOOR HEIGHT = 11'-4"

TOTAL BUILDING DEAD LOAD:

W = 20(7485) + 21.95(7485) +2(76(8670)) + 2(51(8670)) + 2(75(446.67*11.33)) = 3350141.415 lb = **3275 kip**

**The deck was not directly called out on the project drawings, therefore a USD 1.5" x 6" B-Lok Deck was assumed. A concrete weight of 145 pcf was used. The density of concrete in the CMU's was not specified, therefore, a density of 125 pcf and a grout spacing of 24" OC. were assumed to be conservative. The exterior walls are not 12" CMU's for the entire perimeter of the building, however, this was assumed to be the case in order to maintain a conservative estimate. The steel and insulation load values were based upon an example of such calculations prepared by a concrete professor at The Pennsylvania State University.*

APPENDIX B, SNOW LOAD CALCULATIONS

ROOF SNOW LOAD

Based on BOCA 1996, Section 1608

C_e	I	P_g (psf)	P_f (psf)	C_s	P_s (psf)
0.7	1.1	30	23.10	0.95	21.95

$P_s = C_s P_f$

All Factors determined using BOCA 1996, Section 1608

Snow Drift was not an issue for this design, because the roof was entirely gabled and provided no such areas for drift.

APPENDIX C, WIND LOAD CALCULATIONS

WEST FACE WIND LOADS

(Perpendicular to Main Ridge) Based on BOCA 1996, Section 1609

Level	Trib. Ht.	P_v (psf)	I	K_z	K_h	G_h	WW C_p	LW C_p	(GC_{pi})	P_{WW} (psf)	P_{LW} (psf)
1	18'-4"	16.4	1.15	0.37	0.37	1.65	0.8	-0.5	0.25	7.47	-7.50
2	7'-0”	16.4	1.15	0.46	0.46	1.54	0.8	-0.5	0.25	8.52	-8.85
ROOF	20'-6'"	16.4	1.15	0.54	0.54	1.48	-0.1	-0.7	0.25	-4.05	-13.10

V = 80 mph, Exposure B, L/B = 85.67/112 = 0.76, h = 35.43', Tributary Width = 93'-4"

WINDWARD WALLS -- $P = P_v I[K_z G_h C_p - K_h(GC_{pi})]$

LEEWARD WALLS & ROOF -- $P = P_v I[K_h G_h C_p - K_h(GC_{pi})]$

SOUTH FACE WIND LOADS

(Parallel to Main Ridge) Based on BOCA 1996, Section 1609

Level	Trib. Ht.	P_v (psf)	I	K_z	K_h	G_h	WW C_p	LW C_p	(GC_{pi})	P_{WW} (psf)
1	18'-4"	16.4	1.15	0.37	0.37	1.65	0.8	N/A	0.25	7.47
2	7'-0”	16.4	1.15	0.46	0.46	1.54	0.8	N/A	0.25	8.52
ROOF	20'-6"	16.4	1.15	0.54	0.54	1.48	0.8	N/A	0.25	9.51

V = 80 mph, Exposure B, L/B = 112/85.67 = 1.31, h = 35.43', Tributary Width = 64'-10"

WINDWARD WALL -- $P = P_v I[K_z G_h C_p - K_h(GC_{pi})]$

MINOR SOUTH FACE WIND LOADS

(Parallel to Main Ridge) Based on BOCA 1996, Section 1609

Level	Trib. Ht.	P_v (psf)	I	K_z	K_h	G_h	WW C_p	LW C_p	(GC_{pi})	P_{WW} (psf)	P_{LW} (psf)
1	14'-6"	16.4	1.15	0.37	0.37	1.65	0.8	-0.3	0.25	7.47	-5.20
ROOF	14'-0"	16.4	1.15	0.46	0.46	1.54	-0.1	-0.7	0.25	-3.50	-11.52

V = 80 mph, Exposure B, L/B = 43.3/19 = 2.28, h = 25.53', Tributary width = 19'-0"

WINDWARD WALL -- $P = P_v I[K_z G_h C_p - K_h(GC_{pi})]$

LEEWARD WALLS & ROOF -- $P = P_v I[K_h G_h C_p - K_h(GC_{pi})]$

APPENDIX C, WIND CALCULATIONS CONTINUED…

WEST FACE LOADS,

Level 1	(7.50 + 7.47)93'-4" = 1397.15 plf = **1.4 k/ft**
Level 2	(8.85 + 8.52)93'-4" = 1621.14 plf = **1.62 k/ft**
Roof	(13.10 + -4.05)93'-4" = 844.64 plf = **0.84 k/ft**

SOUTH FACE LOADS,

Level 1	(7.47)64'-10" = 484.31 plf = **0.48 k/ft**
Level 2	(8.52)64'-10" = 552.38 plf = **0.55 k/ft**
Roof	(9.51)64'-10" = 616.57 plf = **0.62 k/ft**

MINOR FACE LOADS,

Level 1	(5.20 + 7.47)19'-0" = 240.73 plf = **0.24 k/ft**
Roof	(11.52 + -3.50)19'-0" = 152.38 plf = **0.15 k/ft**

APPENDIX D, SEISMIC CALCULATIONS

SEISMIC BASE SHEAR

Based on BOCA 1996, Section 1610.4

A_v	S	R	C_a	C_T	h_n (ft)	T_a (sec)	T (sec)	C_s	W (kip)	V (kip)
0.1	2.0	4.5	1.7	0.035	11.33	0.22	0.37	0.10	3275	340.47

$V = C_s W$
All Factors determined using BOCA 1996, Section 1610.4

SEISMIC LATERAL FORCES BY LEVEL

Based on BOCA 1996, Section 1610.5

LEVEL	V (kip)	w_x (kip)	h_x (ft)	k	C_{vx}	F_x (kip)
1	340.47	658.92	11.33	1	0.50	170.24
2	340.47	658.92	11.33	1	0.50	170.24

$F_x = C_{vx} V$
All Factors determined using BOCA 1996, Section 1610.5