Lancaster County Bible Church

Manheim, Pennsylvania

Daniel Bellay, Structural Option

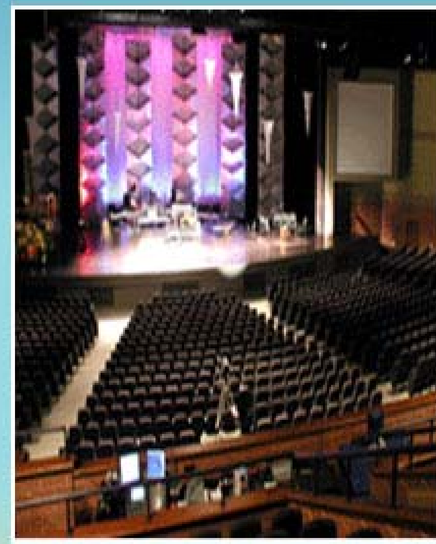
Prepared for: Dr. Richard A. Behr

Date of Submission: April 15, 2010

LANCASTER COUNTY BIBLE CHURCH Manhiem, PA

GENERAL BUILDING DATA:

- **Building Height**: 3 Stories With a Total Height of 48 Feet
- **Square Footage**: 200,000 Square Feet Total With 160,000 Square Feet of Additions
- **Occupancy**: Assembly, Business, Educational
- **First Addition Completion Date**: December 2003
- **Second Addition Completion Date:** August 2009
- **First Addition Cost**: $12,000,000
- **Second Addition Cost:** $14,000,000
- **Project Delivery:** Design-Bid-Build
- **Owner**: Pastor Davie Ashcraft
- **Architect/Engineer**: Mann Hughes Architecture
- **Site Engineers:** RGS Associates
- **CM:** Pelger Engineering and Construction Inc.



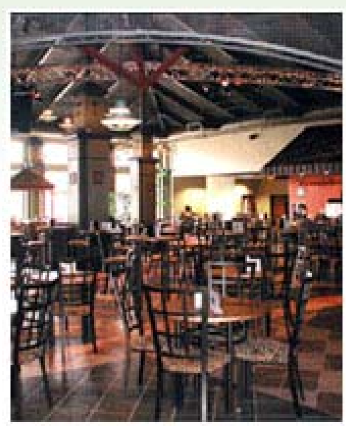
ARCHITECTURE:

Exterior
- Designed to Complement ExistingStructure
- Typical Facade is Stucco Panels ("Dryvit")

Interior
- 2500 Seat Auditorium for Worship Purposes
- Classrooms and Slides for the Youth
- Cafe Areas

STRUCTURE:

- One-Way Concrete Floor With 1.5" Metal Decking Reinforced with 10/10 Welded Wire Mesh
- Typical Slab Thickness = 4"
- Structural Steel Beams and Columns Typically W-Shaped
- K-Serier Metal Trusses Support Floor Loads
- 2.5" x 2.5" x 0.5" T.S. Cross Bracing With Connection Plates for Lateral Bracing
- Column Support is Provided by Spread Footings



http://www.engr.psu.edu/ae/thesis/portfolios/2010/dsb5019/index.html

Daniel S. Bellay Structural Option

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- Structural Engineer
- Architect

Pastor David Ashcraft Lancaster County Bible Church

- Owner

AE Faculty & Staff Pennsylvania State University

- Dr. Richard A. Behr, Advisor
- Professor Kevin Parfitt, Thesis Coordinator

Family & Friends

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Executive Summary:

This report will assess the current structural system of Lancaster County Bible Church and document any and all pertinent information relating to this thesis study of an alternate flooring system and its impact on the existing building systems. The objective of this report is to conduct a comprehensive redesign of the existing structural layout in an effort to engineer a more efficient structural design.

The existing structural system at Lancaster County Bible Church was designed using Allowable Stress Design method. One of the methods employed to engineer a more efficient structural layout was implanting Load and Resistance Factor Design in place of Allowable Stress Design method. The impact of implementing Load and Resistance Factor Design will be compared directly to the original Allowable Stress Design by re-designing the original structure using this alternate design method.

Previous analysis determined that a composite steel flooring system would yield a more efficient flooring system than the existing open web steel joist flooring system. Therefore, in an effort to optimize the structural system present in the building a composite steel flooring system was designed using Load and Resistance Factor Design method.

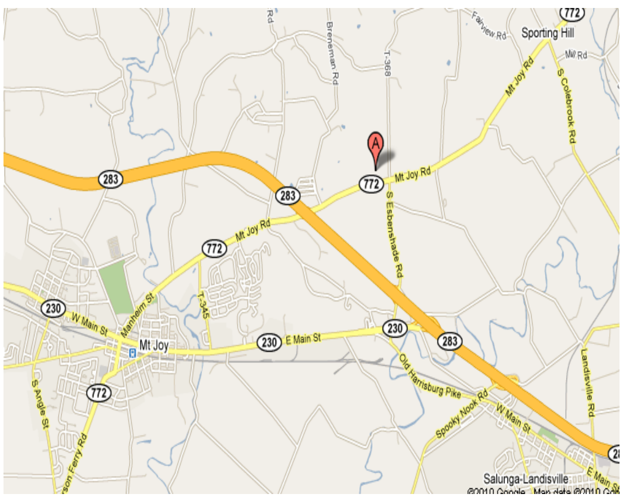
Employing the Load and Resistance Factor Design method coupled with a composite steel flooring system should yield a more efficient structural design than the original design. The re-design should use less steel than the existing structural layout resulting in a cost saving to the owner. In addition, the re-designed flooring system will result in a shorter floor depth and thusly increased ceiling height.

The architectural breadth focuses on the ceiling layout of the building. Using a composite steel flooring system in place of the existing open web steel joist flooring system will impact the existing ceiling layout by reducing complexity and decreasing the floor depth. The construction management breadth will focus on the cost impact of constructing a composite flooring system compared to the existing open web steel joist flooring system. Namely, will the cost saving from the reduction in structural steel from implementing the composite steel flooring system, offset the cost of constructing the alternative flooring system.

Introduction

LCBC (Lancaster County Bible Church) needed to expand its existing facility to accommodate the increased number of attendees at it Sunday service. The new expansion to LCBC would be focused towards the youth population and would include classrooms and youth performance areas. A three story, 78,000 square foot addition was designed by Mann Hughes Architecture. Construction began in May, 2008.

The new addition comprises three levels of multi-functional space. On the 100-level of the addition there is a large classroom and arcade areas for the younger children. Office spaces for the church's staff are the focus of the 200-level with executive offices for the pastor. In order to accommodate the needs of the adolescent population of LCBC a large performance and lounge area are provided on the 300 level. The 100-level, 200-level, and 300-level enjoy a 14'-0", 14'-0", and 15'-4" story height respectively. Total above grade height is 48'-0" to the top of the addition's parapet.



Land was not a restrictive component when the design of LCBC was made. Therefore the design of LCBC is a low profile sprawling structure with 100-level exhibiting a building footprint of 28,000 square feet. Successive levels step back from the 100-level's initial footprint giving the building its unique shape. Stucco panels were chosen as the exterior finish for the addition to complement the existing facilities façade.

Figure 1: Location Map of Lancaster County Bible Church

2392 Mount Joy Road, Manheim, PA 11754

Foundation

Various sized spread footings were designed to support column loads at LCBC. An F20, 2'x2'x12", is the smallest spread footing found at LCBC. Reinforcing for an F20 footing is provided by (3) #4 bars in each direction. Interior columns require the largest spread footing and exhibit F110's, 11'x11'x2'. Reinforcing for F110 is provided by (18) #7 bars in each direction. Typically spread footings are square however there are two rectangular footings, F 70x90 and F50x60. Load bearing masonry walls are supported by continuous spread footings that measure 24"x12". Horizontal reinforcing for the continuous footings is provided by (3) #4 bars. Vertical reinforcing is provided by #6 dowels with 4" hooks @ 8" O.C.

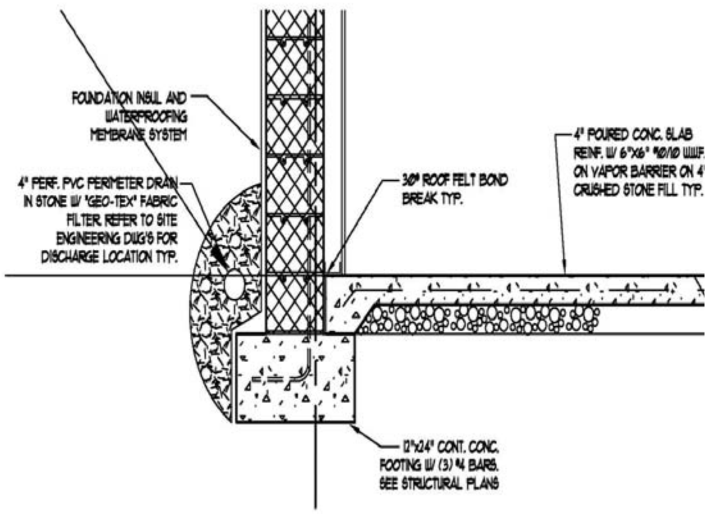


Figure 2: Typical Foundation Detail

Flooring System

Reinforced concrete on metal decking was selected as the primary flooring system for LCBC. A 4" concrete slab is reinforced with 6x6 10/10 welded wire mesh. 1 ½", 26 gauge metal deck provides additional strength for the concrete deck. This one-way floor system transfers gravity loads to supporting girders and columns. Concrete used be 3,000 psi strength.

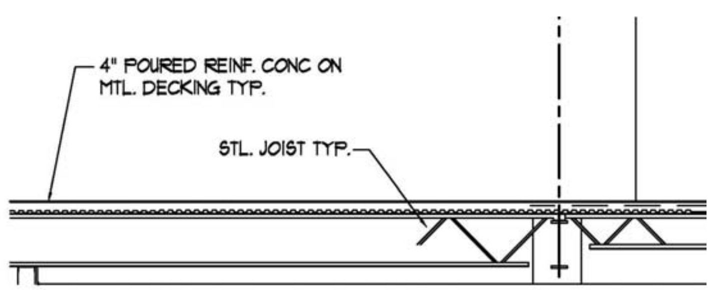


Figure 3: Typical Floor Framing

Framing for the flooring is provided by various open web steel joists. Longer spans at LCBC, typically 38'-4", demand 26K9 or 26K10 open web steel joists. Shorter spans, typically 25'-0", are typically supported by 18K4 open web steel joists. The 100-level flooring system is a slab on grade system. A 4” thick concrete slab is poured over a 6mm polyurethane vapor barrier. Underneath the vapor barrier on 4” of crushed stone on compacted earth.

Gravity System

Gravity loads at LCBC are resisted by a steel framing system. The majority of the columns are W-shaped with the exception of a few HSS 4x4x3/8 columns. Typically columns will start 7” below grade and continue to the roof level. There are a few columns that start on the 200-level but they are the minority. Column sizes vary depending on how many floors the column supports and if they are interior or perimeter columns. A W10x60 is the heaviest column at LCBC and a W8x31 is the lightest. Beams and girders are W-shaped and range from a W12x16 to a W30x108.

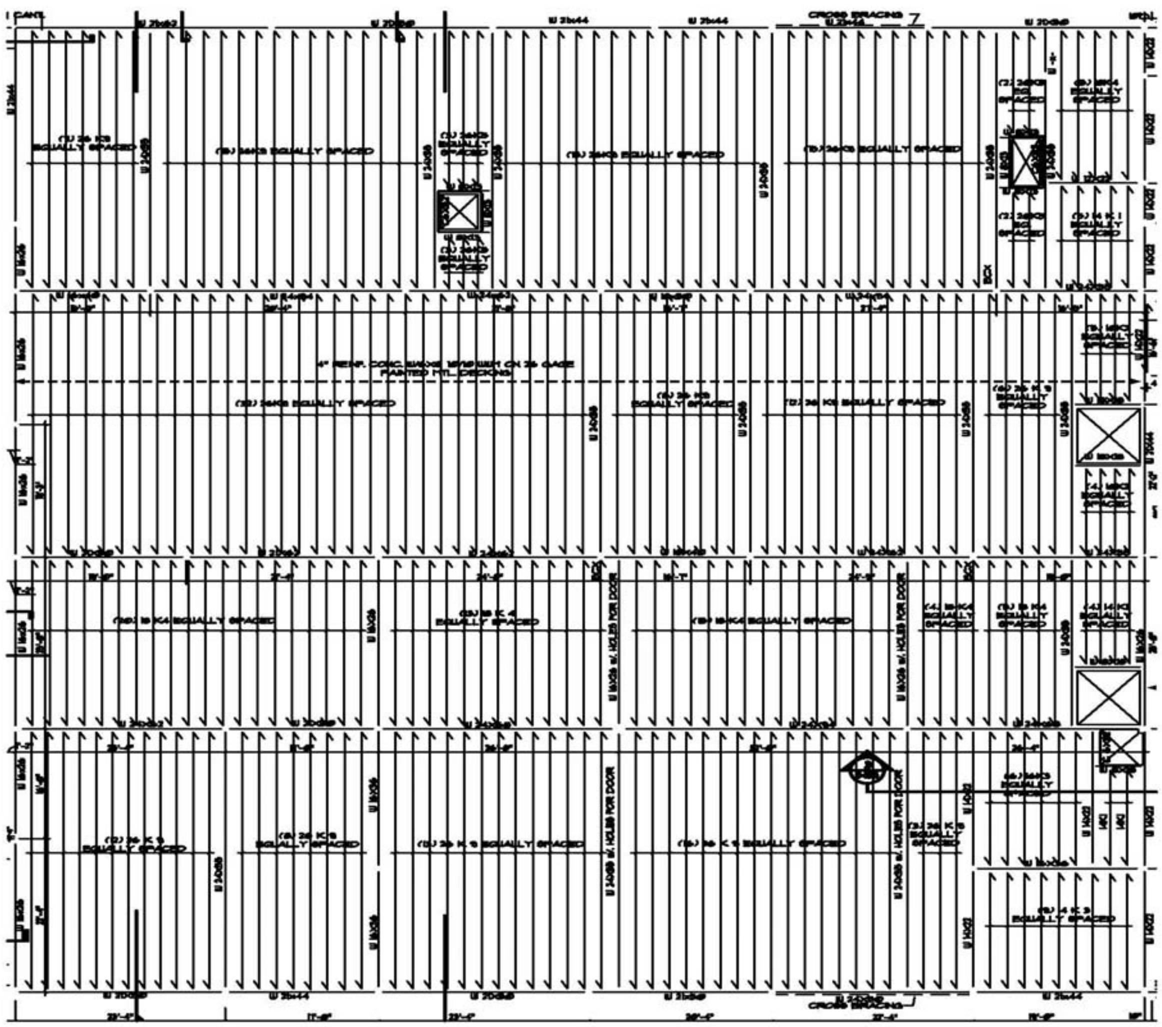
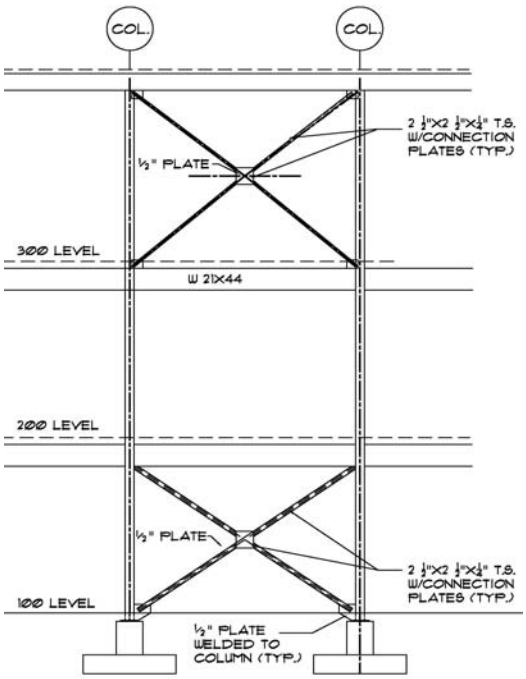


Figure 4: Typical Gravity System Layout

Lateral System

Lateral loads at LCBC are resisted by 5 braced frames. These 5 frames are all located on the perimeter column lines. The placement of the braced frames varies but is concentrated in the Southeast corner. Bracing is accomplished by welding (2) ½” steel plate to base of the column and (2) ½” steel plates the top of the same column. Then 2 ½” x 2 ½” tubular steel is welded to the steel plates in a cross arrangement. Lastly, a piece of ½” steel plate connects the cross bracing in the middle by means of welding.



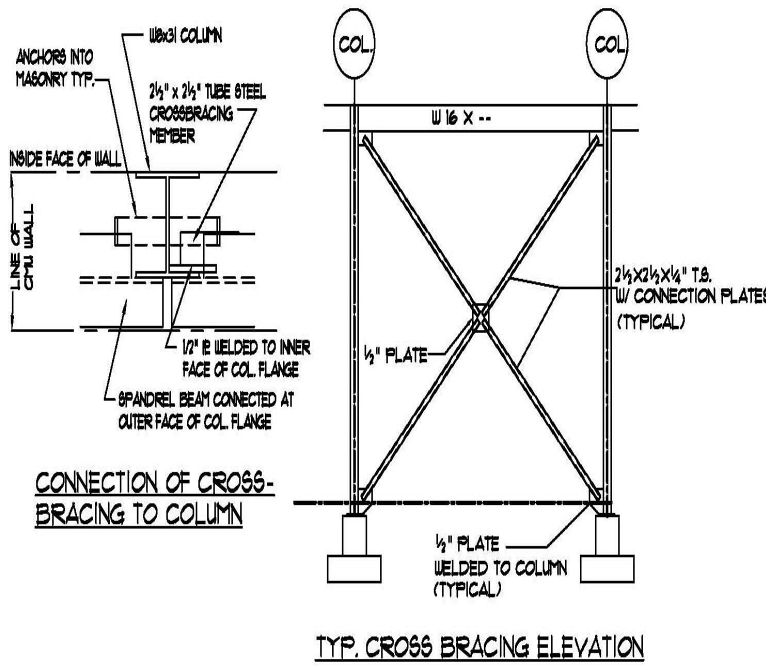


Figure 4: Typical Later Frame

Figure 5: Typical Cross-Bracing Detail

Roofing

Two different flat roofing systems are implemented at Lancaster County Bible Church. The first flat roof system uses three-inch rigid insulation supported by 1 ½” metal decking. A single ply roofing membrane provides moisture protection. Tectum “E” structural roofing panels are used above the youth performance area. The panels are 6-inces thick and are constructed of: OSB sheathing, EPS insulation, and substrate.

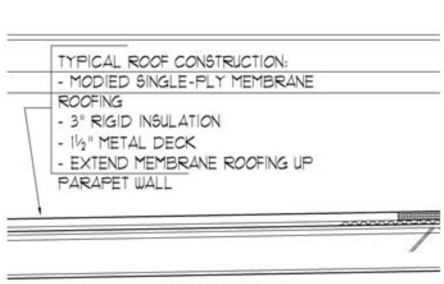


Figure 6: Typical Roof Detail

Building Envelope

The predominate façade of Lancaster County Bible Church is stucco. A ¾"prefabricated stucco panel called EIFS is installed on top of 5/8" dense glass. A vapor barrier provides moisture protection. 6" metal studs placed 16" on center provide support for the building's façade. R-19 batt insulation provides thermal resistance for the wall construction. Gypsum board is used for the interior finish.

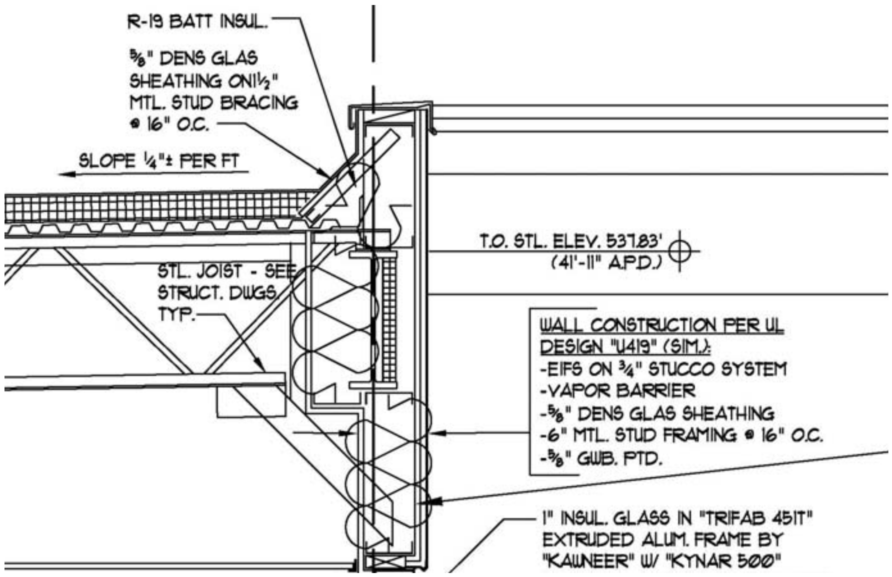


Figure 7: Typical Wall Section

Design Codes & Standards

IBC 2006

AISC Specification for Structural Steel Buildings

AISC Manual of Steel Construction – Allowable Stress Design, 9th Addition

AISC Manual of Steel Construction – Load and Resistance Factor Design, 13th Edition

Vulcraft Steel Joist and Steel Girders 2009

ACI Building Code Requirements for Reinforced Concrete, ACI 318-05

IBC 2000

American Society of Civil Engineers (ASCE) 7-05

Load Combinations

Design load combinations are in accordance with the 2006 International Building Code, Section 1605.2.1 and ASCE 7-05, Ch. 2.

Basic Combinations	
1.	1.4(D +F)
2.	1.2(D + F + T) + 1.6(L + H) + 0.5(L_r or S or R)
3.	1.2D + 1.6(L_r or S or R) + (L or 0.8W)
4.	1.2D + 1.6W + L +0.5(L_r or S or R)
5.	1.2D + 1.0E + L +0.2S
6.	0.9D + 1.6W + 1.6H
7.	0.9D + 1.0E + 1.6H

Figure 8: Typical Load Combinations

Design Loads

Gravity Loads (Dead & Live Loads):

Live Loads	
Area	Design Load (psf)
Corridor	100
Office	100
Stairs	60
Storage Rooms	80
Roof	30
Dead Loads	
Description	Design Load (psf)
Floor Dead Load	50
Partitions	20
Framing	8
Ceilings	3
Mechanical Ductwork	3

Figure 9: Design Load Table

Wind Loads

WIND LOADS (LATERAL)
1. IBC 2000 Basic Wind Speed, V = 100 mph Exposure Category = C Importance Factor, I = 1.0

Figure 10: Wind Load Table

Wind loads were calculated in accordance to ASCE 7-05 Chapter 6. North-South direction and East-West direction were determined using analytical method two. The East-West face of the building is broader than that of the North-South direction resulting in larger wind forces present there.

Seismic Calculations

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

Figure 11: Seismic Load Table

Seismic loads on Lancaster County Bible Church are calculated according to IBC Chapter 6. The seismic flowcharts located in this portion of the code detail the calculations used to determine lateral forces that are produce during a seismic event. Lancaster County Bible Church is a steel framed structure thusly it is light, 333.3 Kips, compared to a similar sized concrete building. In addition, Manheim Pennsylvania is not a seismic area further reducing seismic forces.

Depth Study: Structural Optimization Through the Use of Load and Resistance Factor Design and Composite Steel

Problem Statement: The existing flooring structural system at Lancaster County Bible Church consists of a 4” concrete floor on 1 ½” metal deck. Floor framing consists of open web steel joist typically sized at 26K9. Five braced frames on the perimeter of the structure resist lateral forces and keep the structures deflections within code limits. Completing the required technical reports for this senior thesis proved the existing structure is adequate for resisting the calculated gravity, wind, and seismic loads. For the purpose of this senior thesis the existing structure will be re-designed for the purpose of providing a more efficient structural system.

Solution: Composite steel construction will be employed to design a new flooring system using the Load and Resistance Factor Design method. Initial calculations will be done by hand. Staad.Pro 2007, a structural analysis program, will be used to check the re-designed structure for is ability to resist design loads. The re-designed structure will be compared to the original structure on the basis of pounds of steel required for construction.

Introduction: The dominate building materials in the Manheim region are concrete and steel. Ideally the re-design of Lancaster County Bible Church’s structure would incorporate these two materials. Additionally, the existing architectural layout would remain as undisturbed as possible to honor the tenants programming requirements. Typical bay sizing in the existing structure is 38’-4” x 32’-0” and re-design of Lancaster County Bible Church utilized the same bay sizing and layout. Keeping the original structural layout allowed the existing façade, roofing, architectural layout, and foundation systems to remain undisturbed.

Composite steel was chosen for the re-design of the flooring system for efficiency reasons. A composite steel member uses the strength of the concrete floor that it supports to further its ability to resist bending moments. The original design incorporates a 4” concrete floor on 1 ½” metal deck supported open web steel joist. Concrete in the original design is used to transfer floor loads to the gravity. However, the concrete is not used to resist bending moments caused by floor loads resulting in the need for larger floor beams. By utilizing concretes compressive strength a lighter more efficient design will result.

Allowable Stress Design was chosen as the design method for Lancaster County Bible Church’s structure. While Allowable Stress Design is a proven design method Load and Resistance Factor Design will typically produce a more efficient structure. Load and Resistance Factor Design takes building loads and compares them to a member’s strength. In contrast Allowable Stress Design compares building loads to a member’s allowable values which are less than the member’s full strength. Additionally, Load and Resistance Factor Design employs higher factors of safety on unpredictable building forces, such as live loads. Predictable loads, such as dead loads, receive lower factors of safety. The result is Load and Resistance Factor Method is more efficient when building dead loads are roughly larger than 25% of the total service load.

The goal of the structural re-design is to replace the current open web joist flooring with composite steel using 3” metal deck and a 4 ½” normal weight concrete slab on metal deck. All calculations will be done using the Load and Resistance Factor Design method. The overall re-design will be compared to the existing structure. Results from the re-design will be based upon efficiency, architectural impacts, performance, cost, and scheduling.

Load and Resistance Factor Design:

The first step in designing a more efficient structure for Lancaster County Bible Church was to implement Load and Resistance Factor Design in place of Allowable Stress Design. In order to determine the effect of using Load and Resistance Factor Design the original building was re-designed using the existing flooring system. Doing this allows for a direct comparison of the two design methods.

Using the original loading it was determined that a 26K9 joist was needed to span the 38’-4” distance between bays. This is the same joist that was specified in original design. This is contributed to fact that Vulcraft designs all of joist using Allowable Stress Design. A constant multiplier of 0.6734 is used to convert Load and Resistance Factor Design load calculations to tabulated values that Vulcraft calculated. Therefore no reduction was made for joists and the original structural layout was kept.

Floor beams are used to connect columns together throughout the structure and these members were the next to be designed. For the 38’-4” span a W 24x55 was calculated for the floor beams in the interior of the structure and a W 21x44 beam was selected for the exterior of the structure. Both beams were an exact match for the existing structure. The 25’-0” span floor beams in the existing structure are sized at W 16x26 for the interior and exterior of the structure. While the re-design yielded an exact match for the interior floor beam the exterior floor beam was specified as a W 14x22.

There are two different types of interior girders in the design. The first type of floor girder supports a 25’-0” span and a 38’-4” span while a second type of girder supports two, 38’-4” spans. In the existing structure the girders are specified as a W 30x99 and a W 30x108 respectively. Exterior girders on the original structure are specified as W 24x62. My calculations concluded that an exterior girder sized at W 21x55 would be sufficient to resist gravity loads. Additionally, interior girders sized ate W 30x90 (supporting (1) 38’-4” span and (1) 25’-0” span) and a W 30x99 (supporting (2) 38’-4” spans) would be adequate to support floor loads.

Designing roofing members was the next step in the Load and Resistance Factor Design re-design. The interior roof beam was the first member to be calculated. Results for the roof beams mimic the first and second floor results. The re-design specifies a W 21x44 for the 38'-4" span and a W 14x34 for the 25'-0" span. Both of these members are identical to the original interior roof beams. Exterior roof beams were also identical to the original design with a W 16x26 for the 38'-4" span and a W 14x22 for the 25'-0" span.

The existing roof girders at Lancaster County Bible Church were specified as W 21x55 for interior girders and W 21x44 for exterior girders. Through the implementation Load and Resistance Factor Design a W 21x48 for interior girders and a W 21x44 for exterior girders were found to be adequate for resisting roof loads. Results from the re-design are tabulated in Appendix A.

Using Load and Resistance Factor Design on the existing structure at Lancaster County Bible which was designed using Allowable Stress Design yielded a reduction of 13,584 pounds of structural steel. If the weight reduction of 13,584 pounds is compared to the entire flooring system weight of 445,994 pounds of structural steel, a reduction of 3.04 percent results. However, because of the inability to reduce the weight of the open web steel joists the 13,584 pound reduction is produced from the wide flange flooring members. Therefore, the weight reduction is compared to the existing wide flange members exclusively. Doing so will provide a percent reduction of 6.05.

Columns were neglected for the re-design due to their low proportion of weight in comparison to the weight of the entire structure. Twelve percent of the entire structures weight is attributed from columns. Calculations for the reduction of steel in the flooring system yielded an average savings of about six percent. Therefore, if the columns were re-designed using Load and Resistance Factor Design method it could be hypothesized that a reduction of about 3,000 pounds of structural steel would result. This is an insignificant when compared to the building entire structural system. Due to the fact that the weight of the structure was reduced it was assumed that existing column layout would be sufficient to resist gravity loads. Each column was spot checked using LRFD to ensure that it could resist gravity loads. These spot checks can be found in Appendix A.

Composite Steel Design:

Introduction: Composite steel design has proven to be efficient structural system because of its ability to utilize the strength of the concrete slab that it supports. In composite steel shear studs are welded to supporting steel member and concrete is then poured around these shear studs. This forces the steel and concrete to act together. As load is applied to the steel member it begins to deflect. When the steel member deflects it begins to pull the concrete slab down forcing the concrete into compression. Having the concrete slab and the steel structure working together to resist bending moment is beneficial because it translates into a more efficient structure.

Proposed Solution: Selecting the appropriate metal deck is the first step in designing a composite steel floor. A 3" 16 gauge metal deck manufactured by United Steel Deck was chosen for this design. The 3" decking is capable of spanning up to 12.02' un-shored. Dividing the typical bay size of 38'-4" x 32'-0" into three equal sizes yields an un-shored length of 10'-8" which can be adequately resisted by the 3" metal deck. The thickness of the concrete slab for the composite steel re-design is 4 ½". This is 3 ½" thicker than the existing concrete slab which. A composite re-design of the Lancaster County Bible Church roof was completed. However, the resulting design was impractical. Therefore, the LRFD designed roof was implemented in the final composite re-design. Results of the composite steel re-design can be found in Appendix B.

A one-way direct comparison of the two structural systems would not be an effective analysis procedure. Therefore, results from the composite steel re-design will be compared on the following bases; weight of wide flange members, weight of steel joists, weight of steel decking, and weight of shear studs. Results from the composite steel re-design will be compared to the Load and Resistance Factor Design re-design. The Load and Resistance Factor Design method was used to design the composite steel flooring system. Therefore, results of the composite steel re-design must be compared to the Load and Resistance Factor Design re-design of Lancaster County Bible Church to ensure accurate results.

On the bases of the total weight of wide flange structural steel needed the open web design is the favorable design method. This conclusion is founded upon the fact that the composite steel design requires 1.50 times as much wide flange steel. However, this comparison is bias and does not include the weight of the open web members. For example; a composite steel flooring system uses wide flange structural steel member in floor beams, girders, and columns. In contrast an open web steel joist flooring system relies on wide flange structural members to act as girders and columns only and uses steel joist to transfer floor loads to these wide flange girders.

AMOUNT OF WIDE FLANGE STRUCTURAL STEEL USED	
Composite Steel Flooring System: **412,517 Pounds**	Open Web Steel Joist Flooring System: **275,100 Pounds**

Figure 12: Total Weight of W-Shaped Members

The next area of comparison is weight of steel joist. As previously stated an open web steel joist flooring system relies upon steel joist to carry floor loads to supporting girders. Unlike wide flange members, which are produced from one type of steel, steel joists consist of different types of steels. While the price difference between different grades of structural steel is minimal the impact must still be noted. This fact makes it necessary for steel joists to be analyzed separately from wide flanged members.

TOTAL AMOUNT STRUCTURAL STEEL USED	
Composite Steel Flooring System: **412,517 Pounds (Wide Flange)** **37,478 Pounds (Open Web Steel Joist)** **Total: 450,000 Pounds**	Open Web Steel Joist Flooring System: **275,100 Pounds (Wide Flange)** **228,774 Pounds (Open Web Steel Joist)** **Total: 503,874 Pounds**

Figure 13: Total Weight Structural Steel (W-Shaped and Steel Joists)

While the amounts of structural steel cannot be compared directly it becomes evident that the composite steel flooring system requires less structural steel. In the case of a direct comparison it could be argued that the composite steel flooring system requires 12.0 percent less structural steel.

The weight of metal decking required is the next area of comparison. In the existing structure floor joists were spaced at 2'-0" on average. Due to the short distance between floor supports a 1 ½", 26 gauge metal deck was all that was require to resist floor loads. However, the distance between floor supports in the composite steel design is 10'-8". In order to support this span a 3", 16 gauge metal deck was required. This difference in span resulted in the metal deck for the open web flooring system to weigh 1.44 pounds per square foot while the composite steel metal deck weighs 3.58 pounds per square foot. Nearly two times (by weight) as much metal deck is needed to construct the composite flooring system when compared to the existing structure.

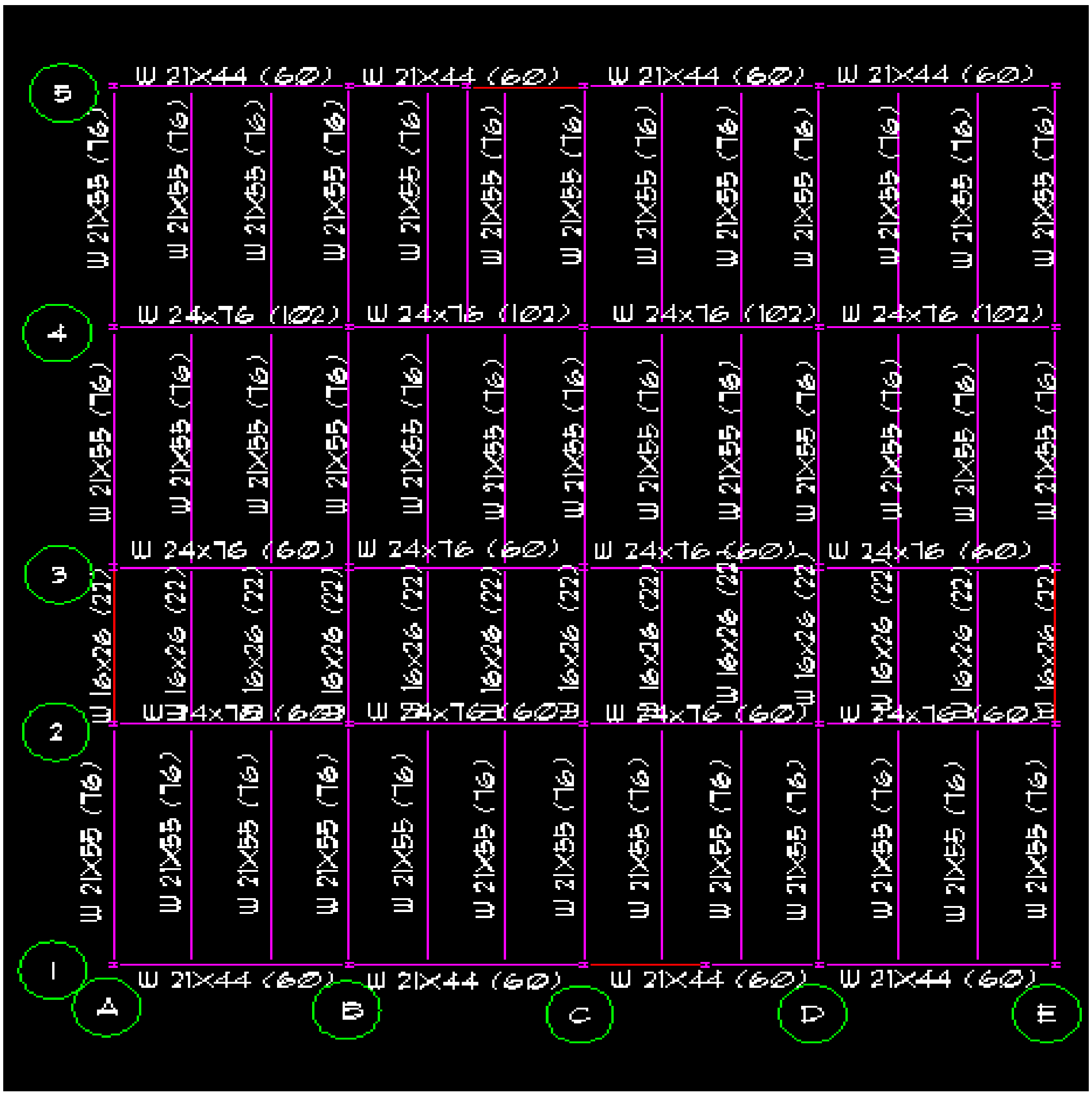


Figure 14: Typical Composite Floor Plan 200 & 300 Levels

AMOUNT OF METAL DECK REQUIRED	
Composite Steel Flooring System: **154,112 Pounds**	Open Web Steel Joist Flooring System: **77,415 Pounds**

Figure 15: Weight of Metal Deck Required

The weight of shear studs is the next area of comparison. Shear studs used in composite steel design allow structural steel members and concrete elements to behave in unison. Such behavior is achieved by welding shear studs to structural steel members then encasing the shear studs in concrete. When structural steel members begin to deflect from gravity loads the shear studs that are welded to the steel members transfer this deflection to concrete elements. Thusly, concrete that is used in composite designs will go into compression when steel members begin to deflect. Therefore, a composite steel beam must first compress a concrete element before it can deflect. By compressing concrete elements composite steel designs can use lighter steel members when compared to a similar non-composite design. However, this reduction in structural steel does come at a price. The composite re-design used a total of 11,420 shear studs. Shear studs for the re-design of Lancaster County Bible Church were specified as 5-inches long, ¾" in diameter, and have a weight of ten-pounds. The final design needed 9,236 shear studs resulting in an additional 92,360 pounds of steel. The process of welding shear studs to wide flange members is laborious as well and a more in depth cost analysis will be done in the construction breadth.

AMOUNT OF SHEAR STUDS REQUIRED	
Composite Steel Flooring System: **9,236 Shear Studs** **92,360 Pounds**	Open Web Steel Joist Flooring System: **0 Pounds**

Figure 16: Weight of Shear Studs

TOTAL AMOUNT STEEL REQUIRED	
Composite Steel Flooring System: **412,517 Pounds (Wide Flange)** **37,478 (Open Web Steel Joist)** **154,112 Pounds (Metal Decking)** **92,360 Pounds (Shear Studs)** **Total: 696,467 Pounds**	Open Web Steel Joist Flooring System: **275,100 Pounds (Wide Flange)** **228,774 Pounds (Open Web Steel Joist)** **77,415 Pounds (Metal Decking)** **Total: 518,286 Pounds**

Figure 17: Total Amount of Steel Required

It must be noted that the increased floor load from the thicker concrete slab yielded higher column loads. This increased load required a re-design of the columns. All calculations for the column re-design along with a structural layout can be found in Appendix B.

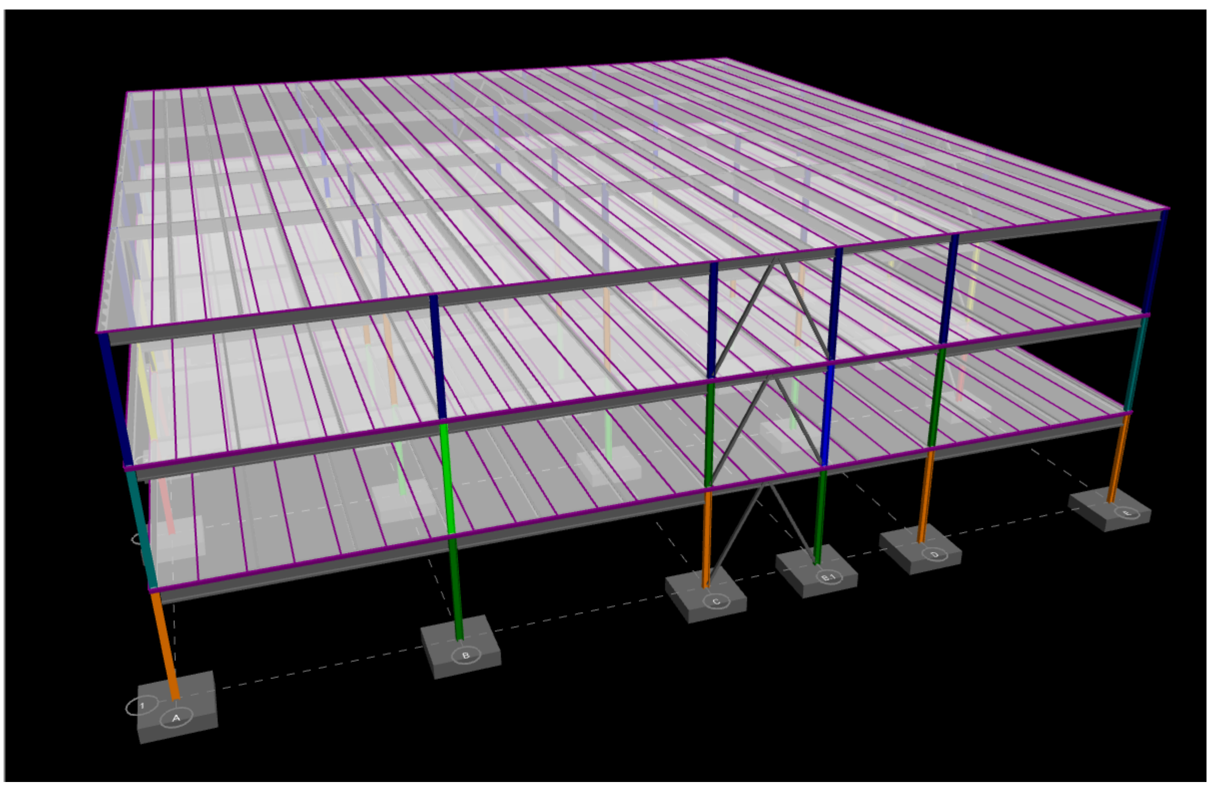


Figure 18: RAM 3-D Computer Model

Lateral System Optimization:

The existing lateral system at Lancaster County Bible County Church was found to be sufficient to resist lateral loads. However, the existing system created large torsion forces because the center of rigidity is located on the exterior of the building approximately sixty feet from the center of mass. The objective for the lateral system optimization is to move the center of rigidity closer to the center of mass to reduce torsion forces on the building. To achieve this goal a computer model was created in RAM and a new lateral layout was made. In an effort to not disturb the existing architectural layout lateral frames were placed on the exterior of the structure. Wind and seismic loads were determined by the computer software, RAM, after the building's parameters were entered into the software. Lateral bracing was sized at 5 x 5 x 3/8" tubular steel. RAM Frame was used to determine the lateral deflection caused by wind loads.

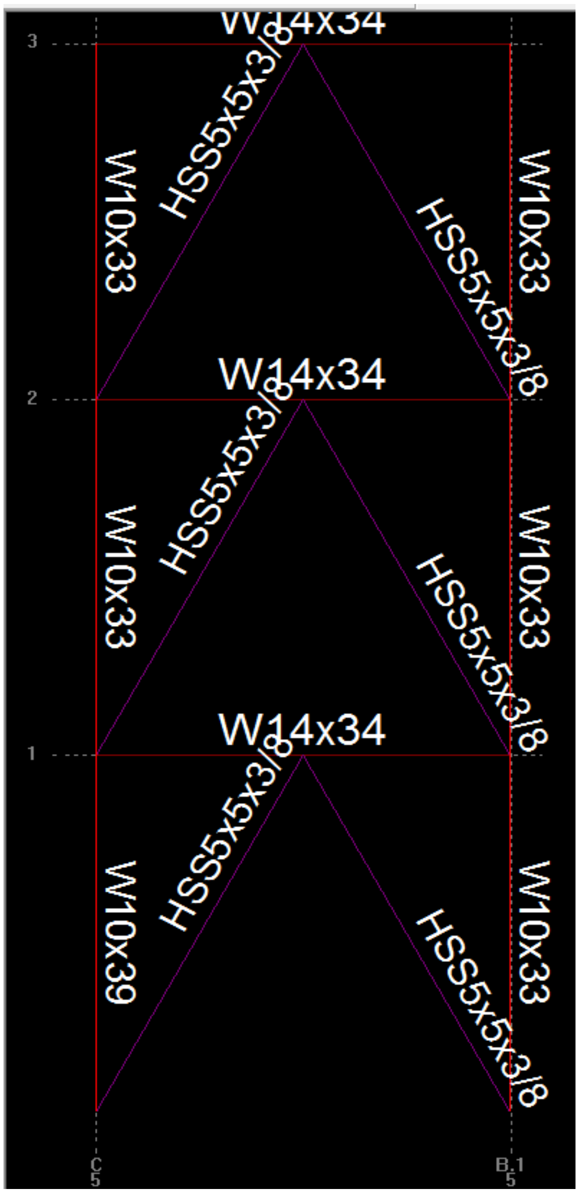
Wind Drift From E-W Wind Force						
Story	Story Height iches	Story Drift (inches)	Allowable Story Drift Δwind = H / 400 (in.)	Total Drift (inches)	Allowable Total Drift Δwind = H/400	Serviceability Check Actual < Allowable
Roof	522	0.0524	0.435	0.2173	1.305	Okay
3	348	0.0815	0.435	0.1648	0.870	Okay
2	174	0.0833	0.435	0.0833	0.435	Okay

Figure 19: Frame Deflections Caused By E-W Wind

Wind Drift From N-S Wind Force						
Story	Story Height iches	Story Drift (inches)	Allowable Story Drift Δwind = H / 400 (in.)	Total Drift (inches)	Allowable Total Drift Δwind = H/400	Serviceability Check Actual < Allowable
Roof	522	0.0209	0.435	0.1175	1.305	Okay
3	348	0.0421	0.435	0.0966	0.870	Okay
2	174	0.0545	0.435	0.0545	0.435	Okay

Figure 20: Frame Deflections Caused By N-S Wind

Implementing RAM structural software proved that the HSS 5 x 5 x 3/8" bracing members provided sufficient strength to resist lateral loads. The deflections of the braced frame are well below the deflections allowed by ASCE 7-05/IBC 2006. Below is an elevation view of the two types of braced frames that were employed in the composite steel re-design. RAM results can be found Appendix B.



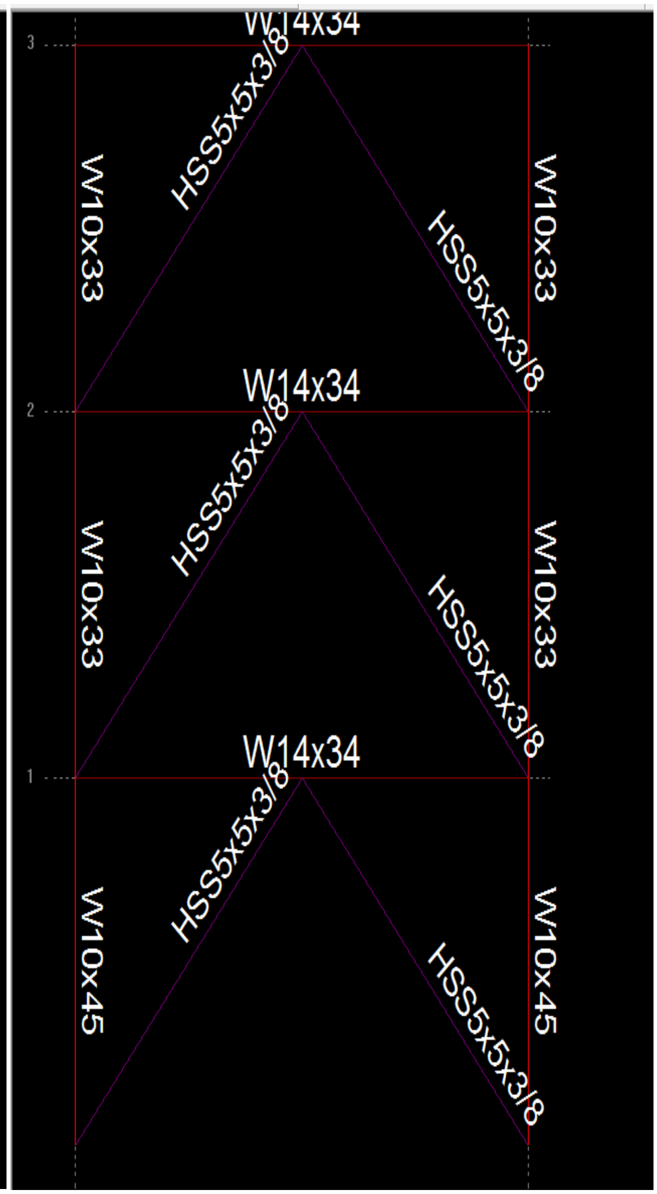


Figure 21: Typical E-W Braced Frame

Figure 22: Typical N-S Braced Frame

Summary: For the purpose of this thesis it can be determined that re-designing the existing flooring system at Lancaster County Bible Church using composite steel will not yield a more efficient structural system. Initial analysis indicated that a significant reduction in structural steel would result if the existing flooring system was replaced with a composite steel system. However, a more in-depth analysis proved that a composite steel design requires a heavier metal deck and numerous shear studs. The amount of additional steel needed, after the structural steel is erected, for a composite steel flooring system is greater than the existing steel joist system. However, an in depth cost analysis covered in the construction breadth showed that the composite steel re-design would actually cost about $15,000.00 less than the existing structure. This savings is less than one percent of the total structures cost rendering negligible. Due to the large bay sizing of 32’-0” x 38’-4” the composite design failed to make a significant impact. A composite design is most effective when bay sizing is smaller say 20’-0” x 20’-0”.

Construction Breadth

Introduction: Transforming the existing open web steel joist flooring system to a composite steel flooring system will produce an assortment of construction management issues. A cost analysis will be performed to determine the relative cost of the composite steel design. Results from the cost analysis of the composite steel design will be compared to the cost of the existing structure.

Existing System: Open web steel joist flooring system was selected as the primary flooring system for Lancaster County Bible Church for many practical reasons. Steel joists are easily erected, resulting in a flooring system that can be installed quickly and efficiently. The open webs in steel joists allow duct work, electrical wires, and other building systems to pass through these structural elements. Steel joists are produced in a factory where quality of construction is kept very high. Additionally, a factory environment allows for a short lead time and higher efficiency than competing field assembled options.

Proposed System: The newly designed composite steel system is considerably fast and easy to erect. Composite steel member are detailed and manufactured in the same factory that produces the structures other steel components such as the columns. This is beneficial because additional entities are not required for the erection of the structure. The existing structural layout requires 576 steel joists to adequately resist floor loads. Managing all of these steel members on a jobsite is a difficult task that can lead to many problems. However, the composite steel framing systems requires over 9,000 shear studs. Additionally, the composite system will need almost twice as much concrete to be poured. Using RSMeans Heavy Construction data an estimated cost was determined. Each step of the cost estimate along with its construction schedule impact is tabulated below. Calculations can be found in Appendix C.

Cost Analysis	
OPEN WEB STEEL JOIST	COMPOSITE STEEL
Concrete 442 Cu. Yds. $26.50 Cost Place One Cu. Yard With a Daily Output of 160 Cu. Yrds./Day it will take **2.77 Days** **Total**: $11,713	**Concrete** 830 Cu. Yds $26.50 Cost Place One Cu. Yard With a Daily Output of 160 Cu. Yrds./Day it will take **5.21 Days** **Total**: $22,000
No Shear Studs Required	**Shear Studs** 9,236 Shear Studs $2.58 Per Stud With a Daily Output of 930 Studs/Day it will take $23,828 , **10 Days**
Wide Flange Member Erection Cost Column Cost: $97,580 , 0.94 Days Beam Cost: $406,112 , 3.43 Days Joist Cost: $343,161 , 11.43 Days	**Wide Flange Member Erection Cost** Column Cost: $166,690 , 1.12 Days Beam Cost: $588,402 , 5.91 Days Joist Cost: $56,217 , 1.84 Days
Metal Decking Floors: $121,497 , 9.28 Days Roof: $60,748 , 4.64 Days	**Metal Decking** Floors: $229,376 , 13.27 Days Roof: $60,748 , 4.64 Days
Grand Total **$1,162,308.00 and 32.49 Days**	**Grand Total** **$1,147,261.00 and 32.71 Days**

Figure 23: Flooring System Cost Comparison

Summary: When all of the different variables of the two different structural systems are compared on a cost analysis it becomes clear that there is very little difference between the two structural systems. In the end the time and cost of placing hundreds of open web joists is more costly in both money and time than the composite steel re-design. Though the difference between the two structural systems is essentially negligible it is surprising that the composite design's cost was not substantially more than the existing structure due to the complex construction process.

Architectural Breadth:

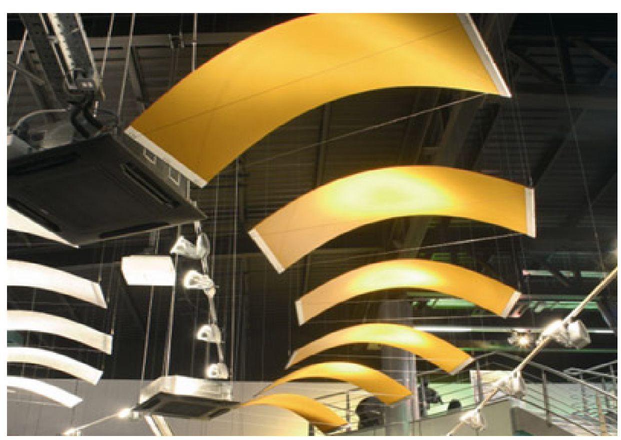
Introduction: The architectural breadth of this thesis will focus on the impact that a composite steel re-design will have on the existing ceiling layout. A solution will be proposed in this breadth to deal with the architectural ramifications caused by the composite steel re-design.

Existing System: Lancaster County Bible Church's existing ceiling is an exposed ceiling. Steel joists receive a coat of paint while ductwork and wiring is left exposed with no finish. The existing ceiling relies upon the complexity of the steel joists to provide a unique look.

Figure 24: Existing Ceiling

Figure 25: Existing Ceiling

Proposed System: The newly designed composite steel system requires two in-fill beams in the typical bay. This is a significant reduction in flooring members from the existing open web steel joist flooring system that specified fifteen joists per bay. If left exposed the composite steel re-design would provide a simplified ceiling that



may be viewed as boring. Suspended ceiling panels from Armstrong's Infusion collection would be installed to break up the large areas of untouched ceiling caused by the composite steel design. Below is the proposed ceiling re-design with a reflected ceiling plan. The large spaces utilized the largest of the Armstrong ceiling panels measuring 4'-0" x 10'-0". The more narrow areas, such as corridors, used the smallest panels which measure 2'-0" x 5'-0". Storage spaces and bathrooms were left unaltered.

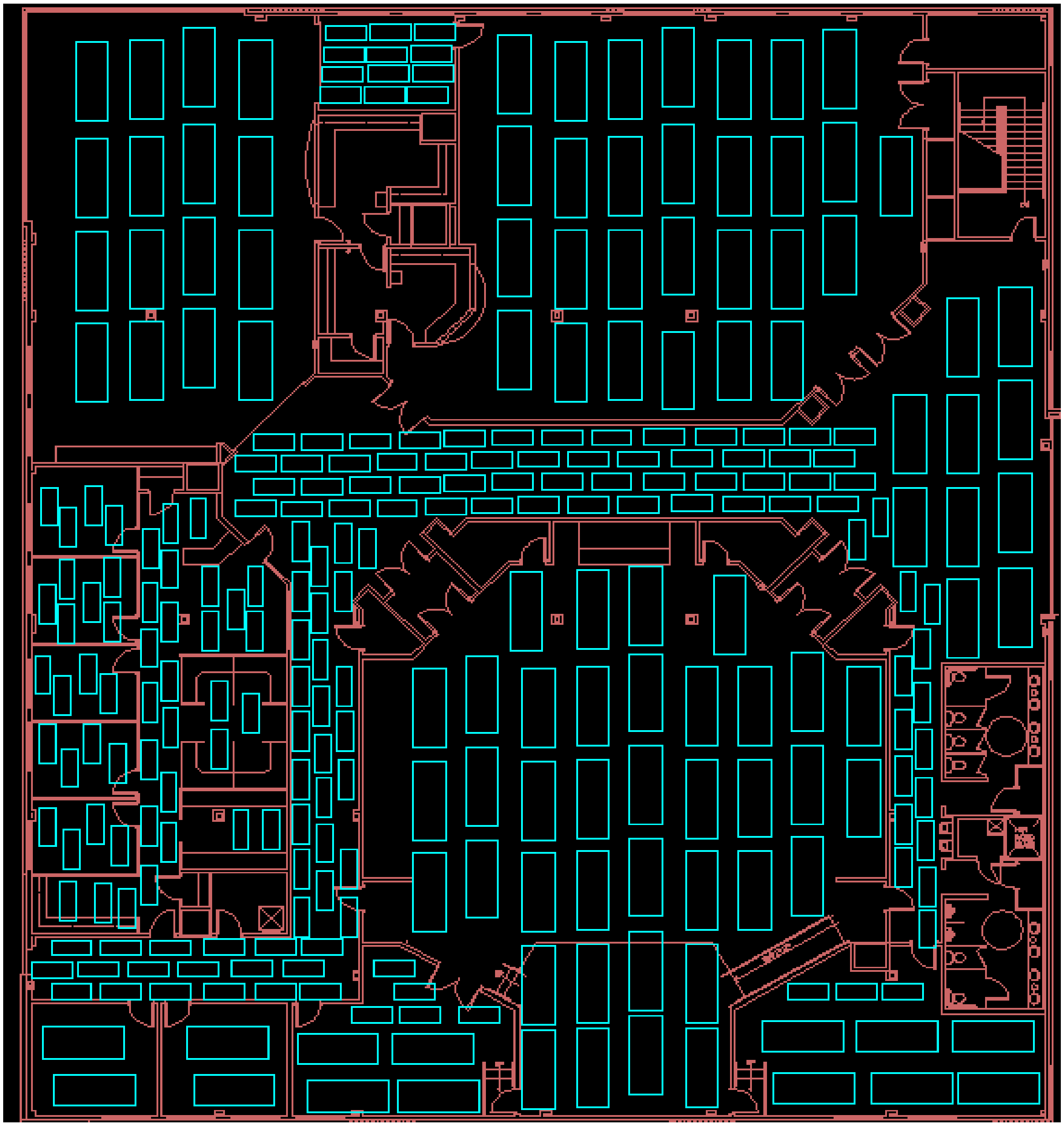


Figure 27: Reflected Ceiling Plan of Proposed Ceiling

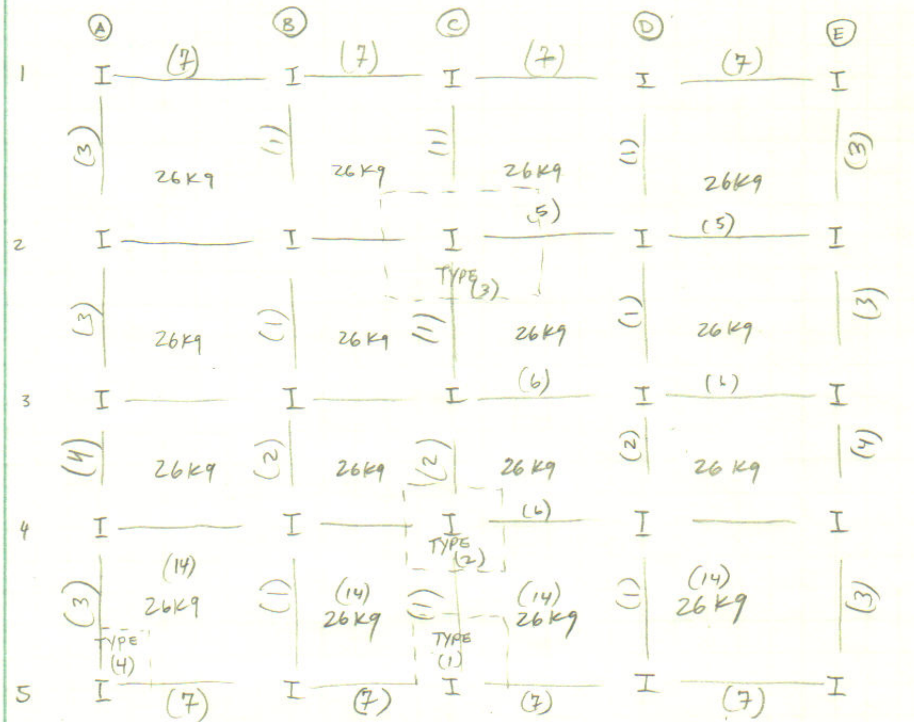
Summary: Using suspended ceiling panels provides many benefits that a more substantial ceiling system, such as a gypsum wallboard ceiling, cannot offer. The suspended ceiling panels are cheap and easy to install. Additionally, the panels will not affect the mechanical and lighting layout. Implementing suspended ceiling panels to the composite steel re-design will add variety and depth to the ceiling layout and fit in well with the existing architectural of the building.

Conclusion

The main goal of this senior thesis report was to design an alternate composite steel flooring system, using Load and Resistance Factor Design method, which would be more efficient than the existing open web steel joist flooring system, which was designed using the Allowable Stress Design method. Depth studies of the major structural impacts were conducted and determined that a composite steel re-design failed to be more efficient than the existing steel joist system. However, the Load and Resistance Factor Design method proved to be a more efficient design method than Allowable Stress Design method for this application. Breadth studies were conducted on the construction management aspects and architectural aspects of the composite steel re-design. It was determined that the impacts of the composite steel re-design on the construction management were essentially negligible and the impact to the existing ceiling was minimal.

APPENDIX A – Load and Resistance Factor Design

LCBC | OPEN WEB LRFD RE-DESIGN | PAGE (1)



- 200 & 300 LEVELS

BEAMS & GIRDERS

(1) W 24x55

(2) W 16x26

(3) W 21x44

(4) W 14x22

(5) W 30x99

(6) W 30x90

(7) W 21x55

(8) (14) 26K9 FLOOR JOISTS

COL

TYPE (1) : W10x49

TYPE (2 & 3) : W10x60

TYPE (4) : W8x48

ROOF

(1) W21x44 : 9, 345

(2) W14x34 : 3, 75

(3) W14x22 : 6, 230

(4) W16x26 : 2, 50

(5 & 6) W21x55 : 12, 384

(7) W21x44 : 8, 256

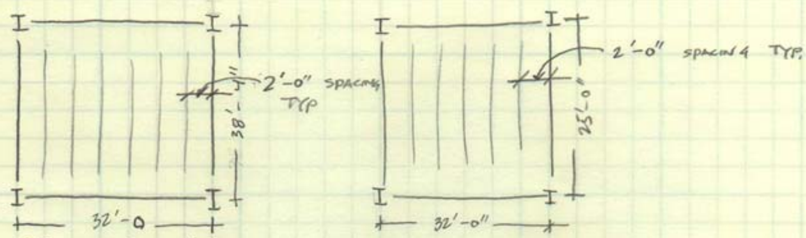
(8) (6) 26K9 ROOF JOISTS : 96, 3072

12.2#/ft

METAL DECKING

26 GUAGE

DESIGN NON-COMPOSITE OPEN WEB STEEL JOIST METAL DECK



TYP INT BAY (1)

$f'_c = 3{,}000$ psi N.W.C. (150#/ft³)

A992 STEEL BEAMS

4" CONC.

26 GAGE METAL DECKING

LOADING

LIVE: 100 psf x 2'-0" = 200 plf (0.2 klf)

DEAD: 25 psf (SDL)

4/12 x 150 = 50 psf

75 psf

2' = 176 plf (0.176 klf)

FACTORED LOADS: 1.2(176) + 1.6(200) = 531 plf

531 / (1.65 x 0.9) = 357.7 plf ∴ USE 26K9

* ORIGINAL DESIGN CALLS FOR 26K9 ∴ MY CALCULATIONS AGREE WITH ORIGINAL DESIGN. BECAUSE OF THIS I WILL USE ORIGINAL JOIST LAYOUT

TRY W 21 x 48

$$\Delta_L = \frac{5(1.01)(32)^4 1728}{(29{,}000)(384)(459)} = 0.857'' < 1.07'' \therefore \text{ ok } \checkmark$$

SHEAR: $\phi V_n = 217^k > 93^k$

(1) 38.33' SPAN & (1) 25' SPAN

$$W_u = \frac{(28)(29.85') \times (0.086)}{32'} = 2.25^k/ft$$

$$M_u \frac{2.25(32)^2}{8} = 288'^k$$

USE W 21 x 48

ASSUME EXTERIOR GIRDERS RECIEVE 1/2 LOAD

INT. GIRDERS

$\phi M_u = 184.2'^k$

TRY W14X34 $\phi M_p = 205'^k$

$$\Delta_L = \frac{5(1.01)(32)^4 1728}{(29000)(384)(340)} = 2.42'' > 1.07'' \therefore \text{ NO GOOD}$$

$$\therefore 1.07 = \frac{5(1.01)(32)^4(1728)}{(29000)(384)(x)} \Rightarrow x \geq 768 \text{ in}^4$$

USE W 21X44

BEAM DESIGN EXT. 38'-4" SPAN

ASSUME 1/2 MOMENT OF INT. BEAM ∴

$M_u = 48.7^{'k}$

TRY W 21X44

$$\Delta L_L = \frac{5(0.32)(38.33)^4 1728}{384(29{,}000)(843)} = 0.64" < 1.92" \therefore \text{ ok}$$

25'-0" SPAN BEAMS

$$M_u = \frac{(0.53)(25)^2}{8} = 41.4^{'k}$$

TRY W16X26

$$\Delta_{L_L} = \frac{5(0.32)(25)^4 1728}{384(29{,}000)(301)} = 0.32" < 0.83" \therefore \text{ ok}$$

EXT $M_u = 1/2 \text{ INT} = 20.7^{'k}$

TRY W14X22

$$\Delta_L = \frac{5(0.32)(25)^4 1728}{384(29{,}000)(199)} = 0.49" < 0.83" \therefore \text{ ok}$$

BEAM DESIGN

- TYPICAL INTERIOR BAY (1)

$W_u = 0.53$ klf

$$M_u = \frac{(0.53)(38.33)^2}{8} = 97.3^{'k}$$

USE W24 x55

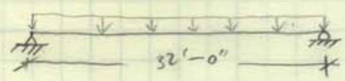
$$\Delta_{L_L} = \frac{5(0.32)(38.33)^4(1728)}{384(29{,}000)(1350)} = 0.397'' \qquad L/360 = 1.92''$$

$0.397'' < 1.92''$ ∴ ok ✓

SHEAR

$$\frac{0.53 \times 38.33}{2} = 10.2^k < \phi V_n = 251^k$$

GIRDER DESIGN



$$W_u = \frac{(30')(38.33') \times ((1.6 \times 100) + (1.2 \times 75))}{(32')(1000^{\#}/kip)} = 8.98^k/ft$$

$$M_u = \frac{(8.98)(32)^2}{8} = 1150^{'k}$$

TRY W30 x 99 $\qquad \phi M_p = 1170^{'k}$

$$\Delta_{L_L} = \frac{5(5.42)(32)^4(1728)}{(29000)(384)(3990)} = 1.05'' < L/360 = 1.07''$$

SHEAR

$\phi V_n = 463 > (8.98)(32) = 289^k$ ∴ ok ✓

ROOF DESIGN

DEAD = 15 psf
LIVE = 30 psf
WIND = 25 psf

$W_u = 1.2(15) + 1.6(30) + 0.8(25) = 86 \text{ psf}$

BEAM DESIGN

25' SPAN

$W_u = (86) \times 4.0' = \dfrac{(0.344 \text{ klf})(25)^2}{8} = 27.1'^{K}$

USE W16x26

$\Delta L_L = \dfrac{5(0.120)(25)^4 1728}{384(29{,}000)(340)} = 0.107'' < 0.833'' = L/360$

SHEAR OK ✓ $\phi V_n = 120^K$

38.33' SPAN

$W_u = \dfrac{0.344 \text{ klf } (38.33)^2}{8} = 63.2'^{K}$

USE W21x44

$\Delta L_L = \dfrac{5(0.120)(38.33)^4 1728}{384(29{,}000)(843)} = 0.238'' < 1.28'' = L/30$

GIRDER DESIGN

(2) 38.33' SPANS

$W_u = \dfrac{(28')(38.33') \times (0.086)}{(32')} = 2.88 \text{ K/ft}$

$M_u = \dfrac{(2.88)(32)^2}{8} = 368.6'^{K}$

OPEN

Page (2)

COL TYPE (1) : C-5

$T_A = 613$ #

LOADING : 200 ; 300 = 250 psf

ROOF : 66psf

TOTAL = 316 psf

$P_u = 194^K$

$M_u = 34$ k·ft

W10X 49

$KL = 14.5(1.71) = \underline{24}'$

$P_x = 3.94$

$b_x = 5.19$

$194(3.94) + 34(5.19) = 0.978 < 1.0 \therefore$ ok

TYPE (2) : C-4

$T_A = 1{,}013 \times 316 = 320^K = P_u$

$M_u = 67.8$

W10X60

$P_x = 2.75$

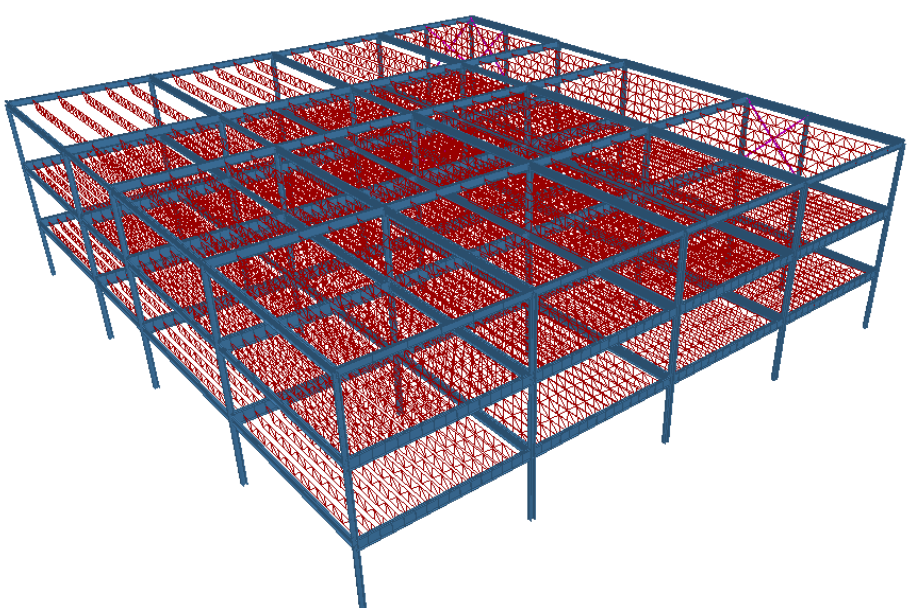
$b_x = 3.39$

$320(2.75) + 67.8(3.39) = 0.952 < 1.0 \therefore$ ok

* ORIGINAL COLUMN LAYOUT IS IDENTICAL TO LRFD RE-DESIGN

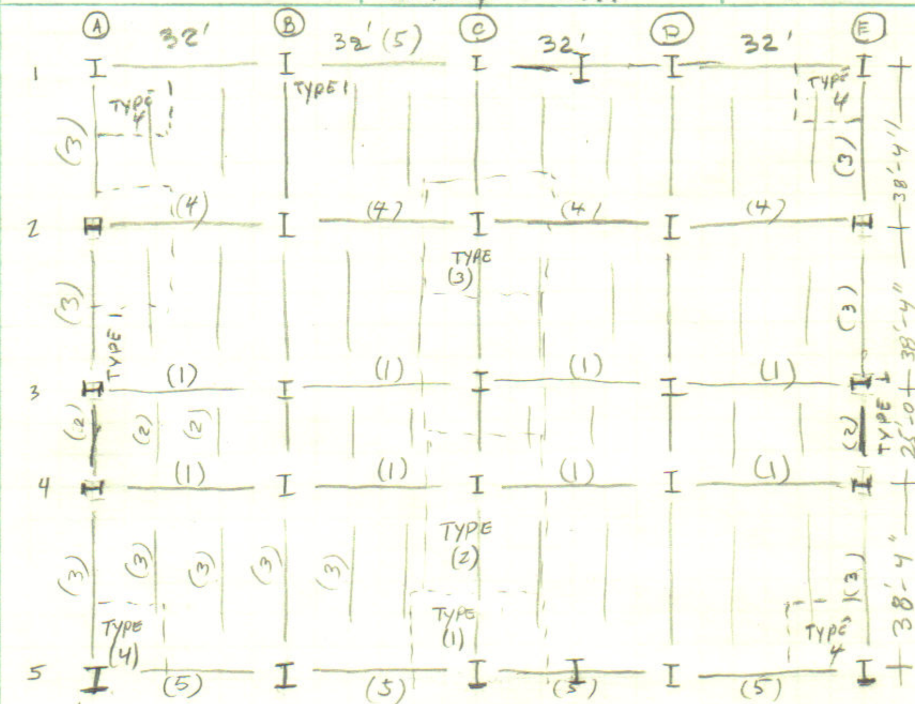
Member Description	Original Design (ASD)	Re-Design (LRFD)	Difference (Pounds/ Ft.)	Total Linear Feet	Steel Reduction (Pounds)
1st Floor Int. Beam (38'-4") Span	W 24x55	W 24x55	0	345	0
1st Floor Ext. Beam (38'-4") Span	W 21x44	W 21x44	0	230	0
1st Floor Int. Beam (25'-0") Span	W 16x26	W 16x26	0	75	0
1st Floor Ext. Beam (25'-0") Span	W 16x26	W 14x22	4	50	200
1st Floor Int. Girder (1) short span, (1) Long span	W 30x99	W 30x90	9	256	2304
1st Floor Int. Girder (2) Long spans	W 30x108	W 30x99	9	128	1152
1st Floor Ext. Girder	W 24x62	W 21x55	7	256	1792
1st Floor Long Span Joist (38'-4")	26 K9	26 K9	0	6900	0
1st Floor Short Span Joist (25'-0")	18 K4	18 K4	0	1500	0
2nd Floor Int. Beam (38'-4") Span	W 24x55	W 24x55	0	345	0
2nd Floor Ext. Beam (38'-4") Span	W 21x44	W 21x44	0	230	0
2nd Floor Int. Beam (25'-0") Span	W 16x26	W 16x26	0	75	0
2nd Floor Ext. Beam (25'-0") Span	W 16x26	W 14x22	4	50	200
2nd Floor Int. Girder (1) short span, (1) Long span	W 30x99	W 30x90	9	256	2304
2nd Floor Int. Girder (2) Long spans	W 30x108	W 30x99	9	128	1152
2nd Floor Ext. Girder	W 24x62	W 21x55	7	256	1792
2nd Floor Long Span Joist (38'-4")	26 K9	26 K9	0	6900	0
2nd Floor Short Span Joist (25'-0")	18 K4	18 K4	0	1500	0
Int. Roof Beam (38'-4") Span	W 21x44	W 21x44	0	345	0

Ext. Roof Beam (38'-4") Span	W 16x26	W 16x26	0	230	0
Int. Roof Beam (25'-0") Span	W 16x26	W 16x26	0	75	0
Ext. Roof Beam(25'-0") Span	W 14x34	W 14x34	0	50	0
Roof Int. Girder (1) short span, (1) Long span	W 21x55	W 21x48	7	256	1792
Roof Int. Girder (2) Long spans	W 21x55	W 21x48	7	128	896
Roof Ext. Girder	W 21x44	W 21x44	0	256	0
Roof Long Span Joist (38'-4")	26 K9	26 K9	0	2300	0
Roof Short Span Joist (25'-0")	18 K4	18 K4	0	500	0
Total Reduction (Pounds) : 13,584					



APPENDIX B – Composite Steel Re-Design

LCBC | COMPOSITE DESIGN | Page (1)



100 & 200 LEVEL

(1) W 24x76 (60) - 8 : 256 LF, 960

(2) W 16x 26 (22) - 13 : 325 LF, 572

(3) W 21x55 (76) - 39 : 1495 LF, 5,928

(4) W 24x76 (102) - 4 : 128 LF, 816

(5) W 21 x 44 (60) - 8 : 256 LF, 960

COL. (14'-6" HEIGHT)

TYPE (1) : W14x90
TYPE (2) : W 14x90 } (18) TOTAL
TYPE (3) : W14x109 (3)
TYPE (4) : W12x65 (4)

ROOF

(3) W14x34 (26)

(2) W 14x22 (18)

(4) W 24x55 (36)

METAL DECKING

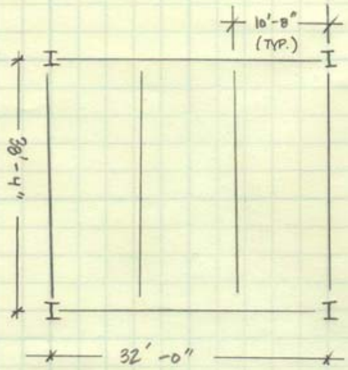
3" USD
16 GAUGE

SHEAR STUDS

5"- 65 KSI

COMPOSITE STEEL WITH METAL DECK

2ND FLOOR BAY TYPE (I)
(38'-4" x 32'-0")



ASSUME UNSHORED CONSTRUCTION
$f'c = 3{,}000$ psi NWC ($150^{\#}/FT^3$)
A992 STEEL BEAMS & GIRDERS ($F_y = 50$ ksi)
3/4"ϕ x 5" STUDS ($F_u = 65$ ksi)
16 GUAGE METAL DECKING
3" LOK FLOOR BY USD
4 1/2" N.W.C. FLOOR
7 1/2" TOTAL SLAB THICKNESS

MAX UNSHORED LENGTH 3 SPANS = 12.02' > 10'-8" ∴ OK ✓

MAX UNIFORM L.L. 11'-0" SPAN = 380 psf > 1.6 (100) = 160 psf ∴ OK ✓

BEAM DESIGN

2^{ND} FLOOR LOADING

LIVE : 100 psf x 10.67' / 1000 = 1.067

$\boxed{LL = 1.1 \text{ KLF}}$

DEAD : 25 psf (SUPER IMPOSED)
76.5 psf (DECK & SLAB)
13 psf (ADD'L 1" CONC. DURING CONSTRUCTION LEVELING)
Σ: 114.5 psf x 10.67' / 1000 = 1.222
+0.060 (BM. SELF WEIGHT ASSUMED DUE TO HIGH LOADS)
1.28

$\boxed{DL = 1.3 \text{ KLF}}$

FACTORED LOADS: 1.2 (1.1) + 1.6 (1.3) = W_u

$\boxed{W_u = 3.4 \text{ KLF}}$

$$M_u = \frac{WL^2}{8} = \frac{(3.4)(38.33)^2}{8} = 624.4\ '^{K}$$

$\boxed{M_u = 625'^{K}}$

ASSUME DEFLECTION CONTROLS : TRY W 21x 55

1. CHECK FLEXURE BEFORE CONCRETE CURES (NO LL / NO SDL)

$W_u = 1.4(1.03) = 1.44$

$W_u = 1.44$ KLF $\rightarrow M_u = \dfrac{(1.44)(38.33)^2}{8} = 265$ 'K

W 21 x 55

$\phi M_p = 473$ 'K $>$ 265 'K $\therefore$ OK ✓

2. AFTER CONCRETE CURES

$M_u = 625$ 'K

$b_{eff\,MIN} = \begin{cases} \frac{1}{4} \text{ SPAN} & \frac{38.33(12)}{4} = 115'' \\ \text{BM. SPACING} & = 10.67'(12) = 128'' \end{cases} \Rightarrow b_{eff} = 115''$

FOR FULL COMPOSITE ACTION $C_{MIN} = \begin{cases} A_s F_y = 16.2(50) = 810^k \\ 0.85 f'_c A_c = 0.85(3)[115(7.5-3)] = 1320^k \end{cases}$

$\therefore C = 810^k \quad a = \dfrac{C}{0.85 f'_c b} = \dfrac{810^k}{0.85(3)(115)} = 2.76''$

$y = \dfrac{d}{2} + t - \dfrac{a}{2} = \dfrac{20.75''}{2} + 7.5'' - \dfrac{2.76}{2} = 16.50''$

DESIGN STRENGTH:

$\phi M_n = 0.90(810^k)(16.5''/12) = 1002.4$ 'K $>$ 625 'K $= M_u$ $\therefore$ OK ✓

3. CHECK SHEAR

$V_u = \dfrac{wL}{2} = \dfrac{(3.4)(38.33)}{2} = 65.2^k < 234^k = \phi V_n$ $\therefore$ OK ✓

SHEAR CONNECTORS

$C = V_n = 810^k$

MAX DIA $= 2t_f = 2(0.522") = 1.044"$ OR $3/4"$

$$\text{REDUX FACTOR} = \frac{0.85}{\sqrt{N_r}}\left(\frac{W_r}{h_r}\right)\left[\left(\frac{H_s}{h_r}\right) - 1.0\right] \le 1.0$$

$N_r = 1.0$ (ONE STUD PER SECTION)
$W_r = 6.0$ IN (RIB WIDTH)
$h_r = 3.0$ IN (DECK HEIGHT)
$H_s = 5.0$ IN (STUD LENGTH)

$$= \frac{0.85}{\sqrt{1.0}}\left(\frac{6.0}{3.0}\right)\left[\left(\frac{5.0}{3.0}\right) - 1.0\right] = 1.13 > 1.0 \therefore \text{ NO REDUX}$$

$$Q_{n_{MIN}} = \begin{cases} 0.5 A_{sc}\sqrt{f'c \cdot E_c} = 0.5(0.4418)\sqrt{3(3321)} = 22.047^k \\ A_{sc} F_u = 0.4418(65) = 28.716^k \end{cases}$$

$Q_n = 22.05^k$

STUDS REQ'D FOR 1/2 SPAN

$$N_1 = \frac{V_n}{Q_n} = \frac{810}{22.05} = 36.75 \approx 38 \text{ STUDS PER HALF BEAM}$$

76 TOTAL

W 21 x55 ; $I = 1140 \text{ in}^3$ $\frac{L}{360} = 1.28"$ (LIVE)

DEFLECTIONS:

LIVE LOAD DEFLECTIONS:

$I_{tr} \Rightarrow b/n$ $n = \frac{E_s}{E_c} = \frac{29{,}000}{3{,}321} = 8.73$

$b/n = \frac{115}{8.73} = 13.173$ in

$I_{EFF} = I_s + \sqrt{\Sigma Q_n / C_f}\,(I_{tr} - I_s)$

$= 1140 + \sqrt{1.017}\,(4412 - 1140) = 4439.7$

$I_{EFF} = \boxed{4,440 \text{ in}^4}$

LIVE LOAD DEFLECTION

$\Delta_{LL} = \dfrac{5 W_L L^4}{384 (E) I_{EFF}} = \dfrac{5 (1.1)(38.33)^4 1728}{384 (27{,}000)(4440)} = 0.415''$

$0.415'' < 1.28'' \therefore$ ok ✓

GIRDER DESIGN (COMPOSITE)

3" METAL DECK (PARALLEL (∥) TO GIRDER)
4.5" SLAB (f'_c = 3,000 psi)
P_u @ 1/3 PTS.

LOADING → L_L = 100 psf
D_L = 25 psf (SDL)
55 plf (21x55)
76.5 psf (DECK & SLAB)
13 psf (ADD. 1" NWC)

BM TRIB = 10.67'

LIVE = 100 (38.33 x 10.67)/1000 = 40.9^k $\boxed{\text{LIVE} = 41^k}$

DEAD = (25 + 76.5 + 13) (38.33 x 10.67)/1000 = 46.8
+ 55 x (38.33) /1000 = + 1.05

$\boxed{\text{DEAD} = 48^k}$

$P_U = 1.2(48) + 1.6(41) = 123.2^k$

$\boxed{P_u = 123^k}$

$M_u = P_a = 123 \times (10'\text{-}8'') = 1312.4^{'k}$ AISC STEEL MANUAL

$\boxed{M_u = 1{,}312^{'k}}$

ASSUME $a = 2''$ ⇒ $Y_2 = 7.5 - 2''/2 = 6.5$

TRY W 24 x 76 (TFL)

- $\phi M_n = 1550\ ^{'K}$
- $\Sigma Q_n = 1240\ ^{K}$

$$a = \frac{1120}{0.85(3)(32 \times 12)} = 1.26'' < 2'' \therefore \text{CONSERVATIVE ok}$$

$$\frac{1120}{22.05} = 50.8 \rightsquigarrow 51 \text{ STUDS PER HALF SPAN (102 TOTAL)}$$

CHECK GIRDER DEFLECTION

$$\Delta_{LL} \leq \frac{L}{360} = \frac{(32)(12)}{360} = 1.07''$$

$$\Delta = \frac{PA}{24EI}(3l^2 - 4a^2) = \frac{(1720)(41)(10.67)(3(32)^2 - 4(10.67)^2)}{24(29{,}000)(5190)} = 0.55''$$

$0.55 < 1.07'' \therefore$ ok ✓

ASSUME EXTERIOR GIRDER GETS ½ LOAD → $M_u = \frac{1{,}312}{2} = 661\ ^{'K}$

ASSUME $a = 2''$ ⇒ $Y_2 = 7.5 - 2''/2 = 6.5''$

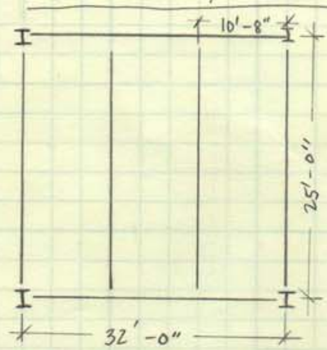
TRY W 21x44 (TFL)

- $\phi M_n = 819$
- $\Sigma Q_n = 649$

$$a = \frac{649}{0.85(3)(32 \times 12)} = 0.66'' < 2'' \checkmark \text{ ok}$$

$$\#\text{STUDS} = \frac{649}{22.05} = 29.4 \approx 30 \text{ STUDS PER } 1/2 \text{ SPAN } (60 \text{ TOTAL})$$

DESIGN BEAM & GIRDER FOR 25' SPAN



$f'_c = 3{,}000$ psi NWC (150#/ft³)

A992 STEEL BEAMS & GIRDERS ($F_y = 50$ ksi)

3/4" φ x 5" STUDS ($F_u = 65$ ksi)

3" LOK FLOOR BY USD

4½" N.W.C FLOOR

7½" TOTAL SLAB THICKNESS

MAX UNSHORED LENGTH 3 SPANS = 12.02' > 10' - 8" ∴ ok ✓

MAX UNIFORM L.L. 11'-0" SPAN = 380 psf > 1.6 (100) = 160 psf ∴ ok ✓

BEAM DESIGN

2ND FLOOR LOADING

LIVE : 100 psf x 10.67' /1000 = 1.067 L_L = 1.1 KLF

DEAD : 1.3 KLF

FACTORED : W_u = 3.4 KLF

$$M_u = \frac{(3.4)(25)^2}{8} = 265'^k$$

ASSUME a = 1" → $Y_2 = 7.5 - 1/2 = 7.0"$

$Y_1 = 4$

TRY 16 x 26

- $\phi M_n = 352'^k$
- $\Sigma Q_n = 242^k$

$\phi M_p = 166'^k$

$$a = \frac{\Sigma Q_n}{0.85 f'_c \cdot b} = \frac{242}{.85(3)(25 \times 12)} = 0.316 < 1.0 \therefore ok$$

$$\frac{243}{22.05} = 11.0 \text{ STUDS PER HALF } (22 \text{ TOTAL})$$

CHECK BEAM UNDER CONSTRUCTION LOADS

$W_D = 76.5$ psf ASSUME 15 psf L_L DURING CONSTRUCTION

$W_u = 1.2(76.5) + 1.6(15) = 115.8 \text{ psf} \times 10.67' = 1.24 \text{ klf}$

$M_u = \dfrac{1.24(25)^2}{8} = 96.9^{'k} < 166^{'k} = \phi M_p \therefore$ ok ✓

CHECK L_L DEFLECTION

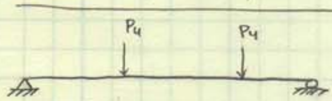
$\Delta_{LL} < \dfrac{L}{360} = \dfrac{25 \cdot 12}{360} = 0.83''$

$\Delta_{LL} = \dfrac{5}{384}\dfrac{wL^4}{EI} = \dfrac{5(1.1)(25)^4\,1728}{384\,(29{,}000)(955)} = 0.349'' < 0.83'' \therefore$ ok ✓

CHECK SHEAR

$V_u = \dfrac{wL}{2} = \dfrac{3.4(25)}{2} = 42.5^k < 106^k \therefore$ ok ✓

DESIGN GIRDER FOR 38.33 & 25' SPANS



$P_u = \left(\dfrac{25'}{2}\right)(3.4) + \left(\dfrac{38.33}{2}\right)(3.4) = 107.7^k$

$M_u = Pa = 107.7 \times 10.67 = 1149.16^{'k}$ CLOSE ENOUGH TO $1313^{'k}$

$\therefore$ TO SIMPLIFY & UNIFY DESIGN USE W 24 X 76

$a = 2'' \Rightarrow Y_2 = 6.5$ $Y_1 = 4$

- $\phi M_n = 1320$
- $\Sigma Q_n = 660$

$a = \dfrac{660}{(.85)(3)(32 \times 12)} = 0.67'' < 2.0'' \therefore$ ok ✓

$\dfrac{660}{22.05} = 29.9 \approx 30$ STUDS PER HALF (60 TOTAL)

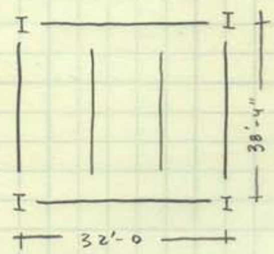
ROOF BEAM DESIGN : USE ORIGINAL DESIGN LOADING

DEAD : 15 PSF
LIVE : 30 psf
WIND : 25 psf

CONTROLLING LOAD COMBINATION:

$1.2D + 1.6L_r + 0.8W$

BAY #1 :



USING 3" LOK-FLOOR

12.02' MAX UNSHORED LENGTH

USE ORIGINAL SPACING 10'-8"

DEAD

15 psf × 10.67'/1000 = 0.16 KLF

$DL = 0.16$ KLF

LIVE

30 psf × 10.67 /1000 = $L_L = 0.320$ KLF

WIND

25 psf × 10.67 /1000 = $W_L = 0.267$ KLF

FACTORED LOAD = $1.2(0.16) + 1.6(0.320) + 0.8(0.267) = 0.918$ KLF

$$M_u = \frac{WL^2}{8} = \frac{(0.92)(38.33)^2}{8} = 167^{1K}$$

FROM AISC STEEL MANUAL: USE W21 x 44

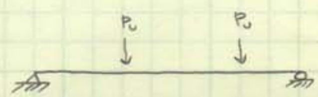
FOR 25' SPAN:

$$M_u = \frac{(0.92)(25)^2}{8} = 72^{'K}$$

USE W 21 x 44

GIRDER DESIGN

INTERIOR GIRDER (1):
TWO 38.33' SPANS



$P_u = 0.92 \times 38.33' = 35.3^K$

$M_u = Pa \Rightarrow M_u = (10.67')(35.3) = 376.7^{'K}$

USE W 33 X118

INTERIOR GIRDER (2)
(1) 38.33' SPAN
(1) 25' SPAN

$P_u = 0.92 \times \left(\frac{38.33}{2} + \frac{25}{2} \right) = 29.1^K$

$M_u = (10.67)(29.1) = 310^{'K}$

USE W 30 X 108

EXTERIOR GIRDER: ASSUME SEES $\frac{1}{2}(376.7) = 189^{'K}$

USE W 21 x 44

CHECK DEFLECTIONS OF BEAMS

38.33' SPAN : $\frac{5 w_L L^4}{384 EI} = \frac{5(0.32)(38.33)^4(1728)}{384(29{,}000)(843)} = 0.637''$

$0.226'' < L/360 \Rightarrow (38.33)(12)/360 = 1.28''$ ∴ ok ✓

25' SPAN : $\frac{5(0.32)(25)^4(1728)}{384(29{,}000)(843)} = 0.115''$

$L/360 = 0.83''$ $0.115'' < 0.83''$ ∴ ok ✓

CHECK GIRDER DEFLECTIONS

— INT. GIRDER (1)

$\Delta L_L = 360 = \frac{(32)(12)}{360} = 1.07''$

$\Delta L_L = \frac{PA}{24EI}(3l^2 - 4a^2) = \frac{(1728)(3(32)^2 - 4(10.67)^2)(10.67)(12.27)}{24(29{,}000)(843)} = 1.00''$

$1.00'' < 1.07''$ ∴ ok ✓

— INT. GIRDER (2)

$\Delta_L = \frac{1728(3(32)^2 - 4(10.67)^2)(10.67)(10.14)}{24(29{,}000)(4470)} = 0.833''$

$0.833'' < 1.07''$ ∴ ok ✓

— EXT. GIRDERS

2616

$\Delta L_L = \frac{1728(3(32)^2 - 4(10.67)^2)(10.67)(6.14)}{24(29{,}000)(2370)} = 0.0168$

$0.0168'' < 1.07''$ ∴ ok ✓

ROOF DESIGN COMPOSITE

38.33' SPAN

DEAD : 1.1 KLF

LIVE : 0.320 KLF

WIND 0.267 KLF

$1.2(1.1) + 1.6(0.320) + 0.8(0.267) = 2.05^{KLF}$

VS,

$1.4D = 1.4(1.1) = 1.54$ KLF

USE 2.05 KLF

$M_u = \frac{wL^2}{8} = \frac{(2.05)(38.33)^2}{8} = 376.5^{1K}$

ASSUME $a = 1"$ $Y_2 = 7.5 - 1/2 = 7.0"$

$Y_1 = 4$

TRY W 14X 34

- $\phi M_n = 401^{1K}$
- $\Sigma Q_n = 270$
- $\phi M_p = 205^{1K}$

$a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{270}{0.85(3)(38.3 \times 12)} = 0.23" < 1.0"$ ∴ ok

$\frac{270}{22.05} = 12.3$ STUDS PER BEAM = (26 TOTAL)

CHECK L_L DEFLECTION

$\Delta_L = \frac{5(0.32)(38.33)^4 1728}{384(29000)(1030)} = 0.52" < 1.28"$ ∴ ok ✓

25'-0 BEAM

$M_u = \dfrac{(2.05)(25)^2}{8} = 160.2'^{k}$

Assume: $a = 1''$ $Y_2 = 7.0$
$Y_1 = 4$

TRY W14 x 22

$\phi M_n = 271$
$\Sigma Q_n = 199$
$\phi M_p = 125$

$a = \dfrac{\Sigma Q_n}{0.85(f'_c)b} = \dfrac{199}{0.85(3)(25 \times 12)} = 0.260'' < 1.0''$ ∴ ok ✓

$\dfrac{199}{22.05} = 9.03$ (10) STUDS TOTAL

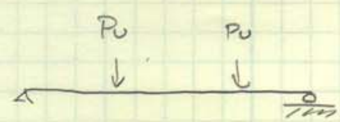
CHECK DEFLECTION (L_L)

$\Delta_L = \dfrac{5(0.32)(25)^4(1728)}{384(29{,}000)(674)} = 0.144'' < 0.833''$

∴ ok ✓

GIRDER DESIGN

(2) 38.33' SPANS



$P_u = (38.33)(2.05) = 78.6^{k}$

$M_u = Pa = 78.6(10.67') = 838.4'^{k}$

TRY W 24x55

$a = 2''$ $Y_2 = 6.5$ $Y_1 = 4$

$\phi M_n = 998$
$\Sigma Q_n = 545$
$\phi M_p = 503$

$a = \dfrac{545}{0.85(3)(32 \times 12)} = 0.56'' < 2.0''$ ∴ ok ✓

CHECK DEFLECTION L_L

$$\Delta_L = \frac{5(0.32)(32)^4(1728)}{384(29{,}000)(1650)} = 0.071'' < 1.07'' \therefore \text{ok}$$

$\frac{545}{22.05} = 25 \therefore$ 50 STUDS TOTAL

– (1) 38.33 SPAN

(1) 25' SPAN

$$P_u = \left(\frac{25}{2}\right)2.05 + \left(\frac{38.33}{2}\right)2.05 = 65.0^k$$

$$M_u = Pa = 65 \times 10.67 = 693^{1k}$$

TRY W21X48 $a = 1''$ $Y_2 = 7.0$ $Y_1 = 3$

$\phi M_n = 824$
$\Sigma Q_n = 619$
$\phi M_p = 461$

$$a = \frac{619}{0.85(3)(32\times12)} = 0.632 < 1.0'' \therefore \text{ok}$$

$\frac{619}{22.05} = 28 \therefore$ 56 STUDS TOTAL

CHECK DEFLECTION L_L

$$\Delta_L = \frac{5(0.32)(32)^4(1728)}{384(29{,}000)(2780)} = 0.10'' < 1.07'' \therefore \text{ok}$$

ASSUME EXT. GIRDER GET 1/2 MOMENT 38.33 GIRDER

$M_u = 49.2^{1k}$

TRY W18x35 assume $a = 1''$ $Y_2 = 7.0''$ $Y_1 = 3$

$\phi M_n = 544$
$\Sigma Q_n = 387$
$\phi M_p = 249^{1k}$

$$a = \frac{387}{0.85(3)(32\times12)} = 0.40'' < 1.07'' \therefore \text{ok}$$

CHECK DEFLECTION

$$\Delta_L = \frac{5(0.32)(32)^4(1728)}{384(29{,}000)(1620)} = 0.16'' < 1.07'' \therefore ok$$

$\frac{387}{22.05} \sim 18$ (36) TOTAL STUDS

Member Description	Re-Design (LRFD)	Number of Shear Studs (Per Beam)	Total Linear Feet	Total Amount Steel (Pounds)
1st Floor Int. Girder (1) short span, (1) Long span	W 24x76	60	256	19,456
1st Floor Int. Girder (2) Long spans	W 24x76	102	128	9,728
1st Floor Ext. Girder	W 21x44	60	256	11,264
1st Floor Long Span Beam (38’-4”)	W 21x55	76	1495	82,225
1st Floor Short Span Beam (25’-0”)	W 16x26	22	325	5,200
2nd Floor Int. Girder (1) short span, (1) Long span	W 24x76	60	256	19,456
2nd Floor Int. Girder (2) Long spans	W 24x76	102	128	9,728
2nd Floor Long Span Beam (38’-4”)	W 21x55	76	1495	82,225
2nd Floor Short Span Beam (25’-0”)	W 16x26	22	325	5,200
2nd Floor Ext. Girder	W 21x44	60	256	11,264
Int. Roof Beam 38’-4” Span	W 14x34	26	1495	50,830
Int. Roof Beam 25’-0” Span	W 14x22	18	325	7,150
Roof Int. Girder (1) short span, (1) Long span	W 21X48	56	256	12,288
Roof Int. Girder (2) Long spans	W 24x55	50	128	7,040
Roof Ext. Girder	W 18x35	36	256	8,960
	Total Number Shear Studs	11,420	Total Steel (Pounds)	342,014

LCBC | COL. RE-DESIGN | PAGE (2)

TYPE (1) : C-5

$T_A = 32'\text{-}0 \times \frac{38'\text{-}4''}{2} = 613 \text{ SQ. FT.}$

Roof $L_L = 30$ psf $D_L = 15$ psf

300 $L_L = 100$ psf $D_L = 114$ psf

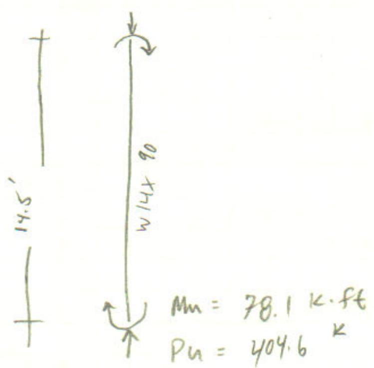
200 $L_L = 100$ psf $D_L = 114$ psf

TOTAL : $L_L = 230$ + $D_L = 243$ psf

$P_u = 1.2(D) + 1.6(L) = 660$ psf

COLUMN LOAD = 660 psf × 613 ft^2 = 404.6^k

$M_u = 1.6(w) = 1.6(48.8) = 78.1^{k \cdot ft}$



AISC MANUAL

Table 6-1 $KL = 14.5\,(r_x/r_y) = 24.0$

$p_x = 1.31 \times 10^{-3}$

$b_x = 1.74 \times 10^{-3}$ $404.6(1.31 \times 10^{-3}) + 78.1(1.74 \times 10^{-3}) = 0.67$

$0.67 < 1.0$ ∴ OK ✓

comp | page (3)

TYPE (2) : C-4

$T_A = 32'-0'' \times \left(\frac{25}{2} + \frac{38.33}{2}\right) = 1.013 \times 660\,psf$

$P_u = 669^K$

$M_u = 1.6(42) = 67.8$

$\boxed{W14 \times 90}$: $P_x = 1.31 \times 10^{-3}$ $\quad KL = 14.5/(1.66) = 24'-0''$

$b_x = 1.74 \times 10^{-3}$

$669(1.31 \times 10^{-3}) + 67.8(1.74 \times 10^{-3}) = 0.984 < 1.0 \therefore ok$

TYPE (3) : C-2

$T_A = 38'-4 \times 32 = 1,227\,\phi \times 660 = 809.6^K$

$M_u = 1.6(42) = 67.8$

$\boxed{W14 \times 109}$

$P_x = 1.07 \times 10^{-3}$ $\quad 809.6(1.07 \times 10^{-3}) + 67.8(1.39 \times 10^{-3}) = 0.961 < 1.0 \therefore ok$

$b_x = 1.39 \times 10^{-3}$

TYPE (4) A - 5

$T_A = \frac{38.33'}{2} \times \frac{32'}{2} = 306.7\,\phi \times 660 = 203^K$

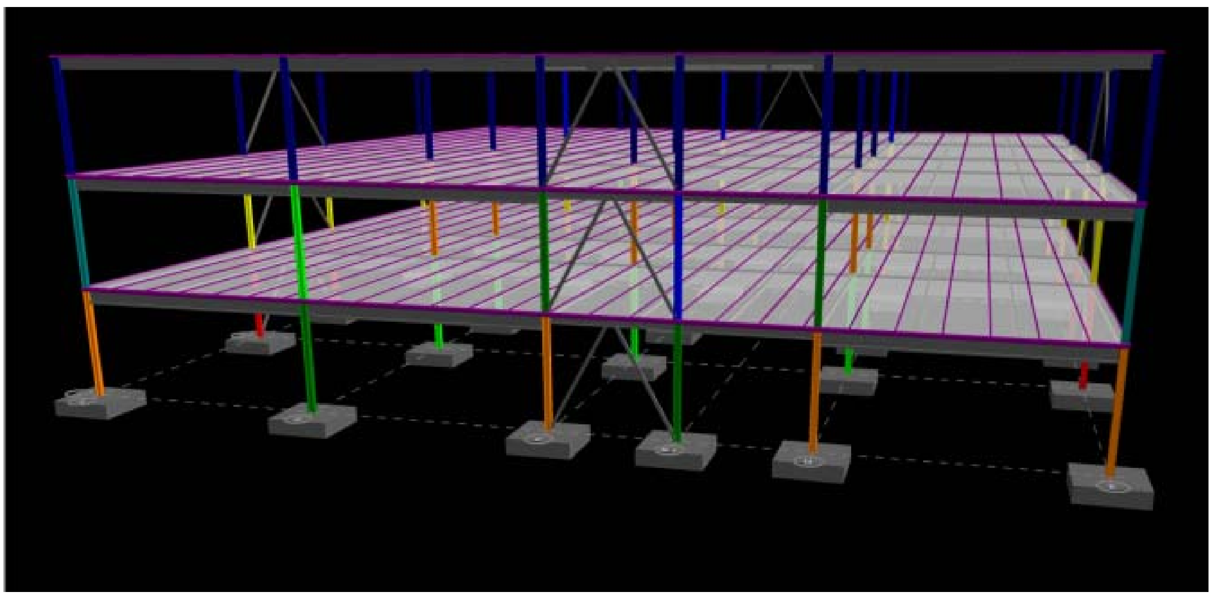
$M_u = 1.6(42) = 67.2^{k\text{-}ft}$

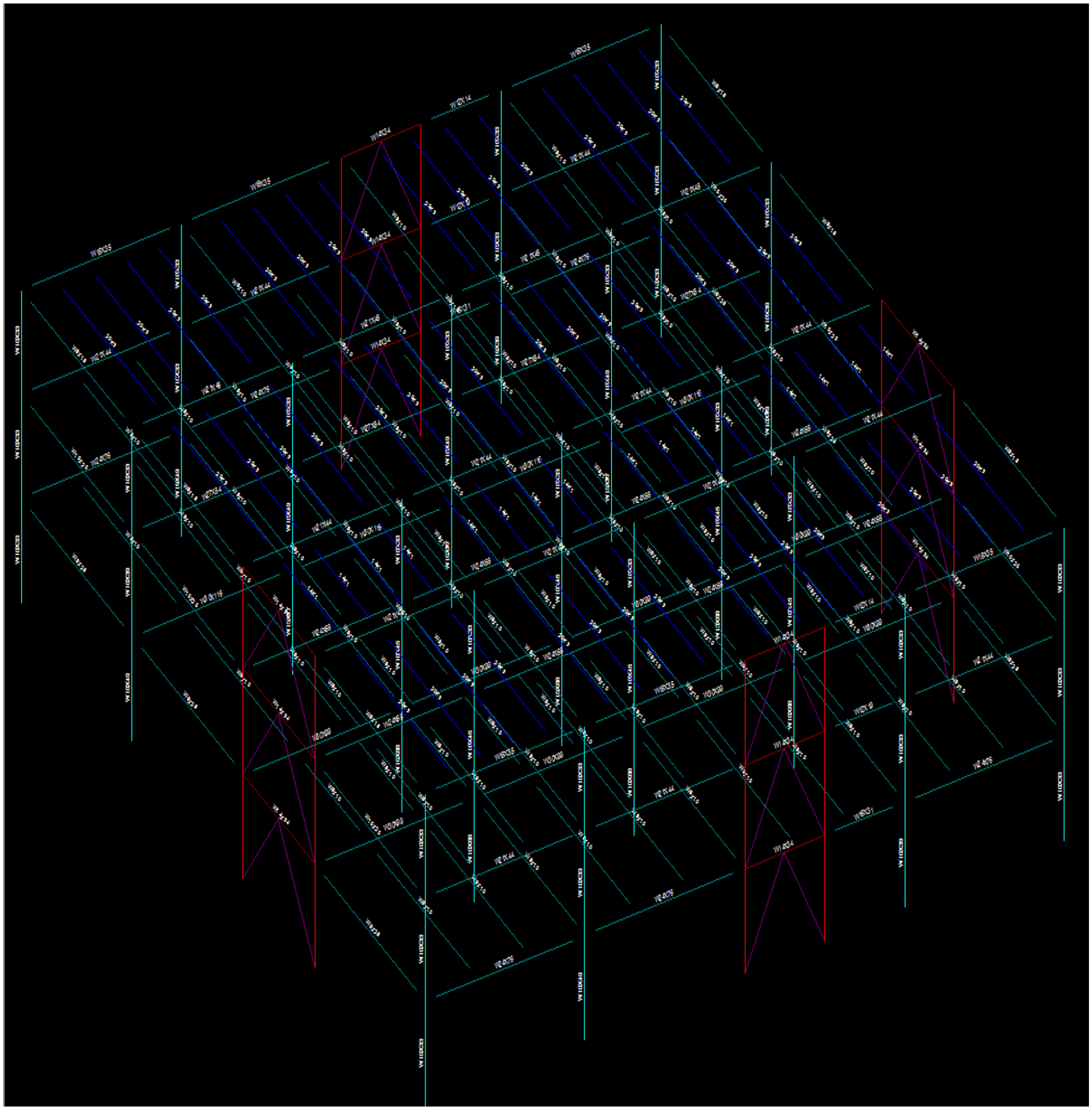
$W12 \times 65 \quad r_x/r_y = 1.75 \quad KL = 14.5'(1.75) = 26'$

$P_x = 2.54 \times 10^{-3}$

$b_x = 3.17 \times 10^{-3}$ $\quad 203(2.54 \times 10^{-3}) + 67.2(3.17 \times 10^{-3}) = 0.729$

$0.729 < 1.0 \therefore ok$ $\quad \boxed{\text{USE} \quad W12 \times 65}$





Story Displacements

RAM Frame v14.03.01.00
DataBase: Thesis_Model
Building Code: IBC

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CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes   Scale Factor: 1.00
Ground Level: Base
Mesh Criteria :
Max. Allowed Distance between Nodes (ft) : 4.00
Merge Node Tolerance (in) : 0.0100

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Sp	PosRoofLiveLoad	RAMUSER
W1	Wind	Wind_IBC06_1_X
W2	Wind	Wind_IBC06_1_Y
W3	Wind	Wind_IBC06_2_X+E
W4	Wind	Wind_IBC06_2_X-E
W5	Wind	Wind_IBC06_2_Y+E
W6	Wind	Wind_IBC06_2_Y-E
W7	Wind	Wind_IBC06_3_X+Y
W8	Wind	Wind_IBC06_3_X-Y
W9	Wind	Wind_IBC06_4_X+Y_CW
W10	Wind	Wind_IBC06_4_X+Y_CCW
W11	Wind	Wind_IBC06_4_X-Y_CW
W12	Wind	Wind_IBC06_4_X-Y_CCW

Note: Story displacements for semirigid diaphragms are reported at their mass centers.

Level: Roof, Diaph: 1

Center of Mass (ft): (63.98, 69.90)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.02844	-0.00001	0.00000
Lp	-0.02952	-0.00001	0.00000
Sp	-0.00557	-0.00000	0.00000
W1	0.21727	0.00001	0.00000
W2	0.00000	0.11752	0.00000
W3	0.16296	0.00001	0.00000
W4	0.16294	0.00000	0.00000
W5	-0.00001	0.08813	0.00000
W6	0.00002	0.08814	0.00000
W7	0.16296	0.08814	0.00000
W8	0.16295	-0.08814	0.00000
W9	0.12223	0.06612	0.00000
W10	0.12220	0.06610	0.00000
W11	0.12222	-0.06610	0.00000

Story Displacements

RAM Frame v14.03.01.00
DataBase: Thesis_Model
Building Code: IBC

Page 2/2
12/30/10 15:15:59

W12	0.12220	-0.06611	0.00000

Level: 3rd, Diaph: 1

Center of Mass (ft): (64.00, 69.99)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.01009	-0.00000	-0.00000
Lp	-0.01073	-0.00000	-0.00000
Sp	-0.00154	-0.00000	-0.00000
W1	0.16484	0.00000	-0.00000
W2	0.00000	0.09657	-0.00000
W3	0.12368	0.00000	-0.00002
W4	0.12358	-0.00000	0.00002
W5	-0.00004	0.07242	0.00002
W6	0.00004	0.07243	-0.00002
W7	0.12363	0.07243	-0.00000
W8	0.12363	-0.07243	-0.00000
W9	0.09279	0.05433	-0.00003
W10	0.09265	0.05432	0.00003
W11	0.09279	-0.05432	-0.00003
W12	0.09265	-0.05433	0.00003

Level: 2nd, Diaph: 1

Center of Mass (ft): (63.98, 69.98)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.00031	-0.00000	-0.00000
Lp	-0.00031	-0.00000	-0.00000
Sp	-0.00005	-0.00000	-0.00000
W1	0.08330	0.00000	-0.00000
W2	0.00000	0.05448	-0.00000
W3	0.06254	0.00001	-0.00001
W4	0.06241	-0.00001	0.00001
W5	-0.00005	0.04086	0.00001
W6	0.00005	0.04087	-0.00001
W7	0.06248	0.04086	-0.00000
W8	0.06248	-0.04086	-0.00000
W9	0.04694	0.03066	-0.00001
W10	0.04677	0.03064	0.00001
W11	0.04694	-0.03064	-0.00001
W12	0.04677	-0.03065	0.00001

APPENDIX C – Construction Breadth

LCBC	COMPOSITE RE-DISIGN	CONSTRUCTION PAGE(1)

(1) CONCRETE
- 17,920 ⌀ PER FLOOR
- 7 1/2" SLAB
- $(\frac{7.5}{12})(17,920)$ = 11,200 c.f. per floor = 414.8 CY ≈ 415 C.Y.

TOTAL CONCRETE = 2 FLRS · 415 = 830 C.Y.

$26.50 PER YARD x 830 YDRS = $22,000.00

DAILY OUTPUT: 160 YARDS/PER DAY TIME INSTALL = 5.2 DAYS

(2) TOTAL SHEAR STUDS

76 X 39 BMS = 2,964
\+ 60 X 8 BMS = 480
\+ 102 X 4 BMS = 408
\+ 60 X 8 BMS = 480

4,332 X 2 FLOORS = 8,664 STUDS

COST PER STUD $2.58 DAILY OUTPUT: 930 STUDS

ROOF:

36 X 20 BMS = 720
\+ 26 X 39 BMS = 1,014
\+ 18 X 13 BMS = 234

1,968

TOTAL BUILDING STUDS = 10,632

COST INSTALL = 27,430.56 TIME INSTALL = 11.43 DAYS

(3) METAL DECK

3", 16 GUAGE $8.65 S.F.

53,760 S.F. x $8.65 = $465,024.00

LCBC | COMP CONSTRUCTION | PAGE (2)

COLUMNS

14'-6" PER FLOOR x 3 = 43.5' TOTAL COL. LENGTH

(18) W14X90 => 18 x 43.5 = 783 L.F.

MATERIAL COST : 1.65 x 783 x 90 = 116,275.50

LABOR, EQUIPMENT, O&P = 22 L.F. x 783 = 17,226

CAMPAD

THESIS — WEIGHT CALCS

COMPOSITE:

COLUMNS

TYPE	TOTAL NO. BUILDING	TOTAL HEIGHT	TOTAL WEIGHT #'s	
W 14X90	18	42'-6" (765) 980@160	68,850	$122,400
W 14 X109	3	42'-6" (128) 970@180	13,897	$23,040
W12X65	4	42'-6" (170) 1000@125	11,050	$21,250
			93,797	

BEAMS 100 & 200 LEVEL

TYPE	TOTAL LINEAR FEET	TOTAL WEIGHT	Days	TOTAL # SHEAR STUDS
W24 X76	768 (1110 @ 130) 99,840	58,368	0.69	1776
W 16X26	650 (1000 @ 53) 34,450	16,900	0.65	572
W 21 X55	2,990 (1064 @ 100) 299,000	164,450	2.81	5,928
W 21 X44	512 (1064 @ 87.50)	22,528	0.48	960
	478,090	262,246	4.63	9,236

BEAMS ROOF

TYPE	TOTAL LIN FEET		TOTAL WEIGHT	Days to erect
W 21X44	345	(1064 @ 87.5)	15,180	0.33
W 14X34	75	(810 @ 69)	2,550	0.10
W 14X22	230	(990 @ 50)	5,060	0.20
W 16X26	50	(1000 @ 53)	1,300	0.05
W 21 X55	384	(1064 @ 100)	21,120	0.36
W 21X44	256	(1064 @ 87.5)	11,264	0.24
		$110,372	56,474	1.28 days

ROOF JOIST

TYPE	TOTAL LIN FEET	TOTAL WEIGHT
26K9, (12.24/ft)	3,072	37,478

TOTAL W-SHAPED WEIGHT

412,517

TOTAL SHEAR STUD WEIGHT

92,360

TOTAL JOIST WEIGHT

37,478

OPEN WEB TOTAL WEIGHT

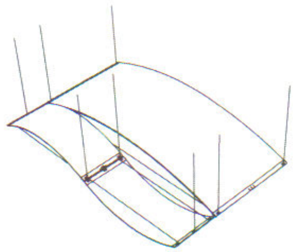
W-SHAPE		JOISTS		METAL DECK		
275,100	+	228,774	+	77,414	=	581,286 #'s

CONCRETE	AREA PER FLOOR	NO. FLRS	TOTAL
4" FLOOR	17,920	2	11,945 CU. FT.
			442 CU. YDS

APPENDIX D – Architectural Breadth

Ways You Can Suspend Linked 2′ x 5′ and 4′ x 10′ Canopies

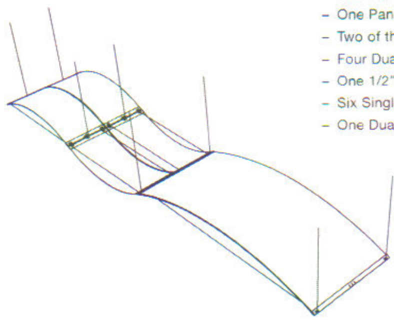
To Create: Two Small Canopies and a Large Canopy Linked Side-to-Side



You need:
- Two Panels (2′ x 5′ installed with a 60° arc)
- One Panel (4′ x 10′ installed with a 50° arc)
- Three of the same Hanging Kits (7004 or 7005, depending on length)
- Two Dual Canopy Hanging Kits (7041)
- Two of the same Spacing Kits (choose from 7042, 7043 or 7007)
- Two Single Hinge Linking Kits (7044)

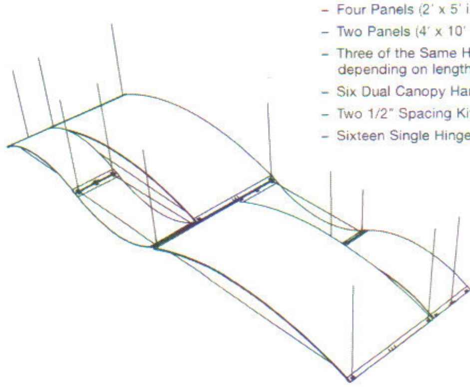
NOTE:
You must use 60° arc for the 2′ x 5′ canopies and a 50° arc for 4′ x 10′ canopies in this configuration.

To Create: Four Small Canopies and a Large Canopy Linked End-to-End



You need:
- Four Panels (either 2′ x 5′ or 2′ x 6′)
- One Panel (4′ x 10′)
- Two of the Same Hanging Kits (7004 or 7005, depending on length)
- Four Dual Canopy Hanging Kits (7041)
- One 1/2" Spacing Kit (7043)
- Six Single Hinge Linking Kits (7044)
- One Dual Hinge 1/2" Linking Kit (7046)

To Create: Four Small Canopies and Two Large Canopies in This Configuration



You need:
- Four Panels (2′ x 5′ installed with a 60° arc)
- Two Panels (4′ x 10′ installed with a 50° arc)
- Three of the Same Hanging Kits (7004 or 7005, depending on length)
- Six Dual Canopy Hanging Kits (7041)
- Two 1/2" Spacing Kits (7043)
- Sixteen Single Hinge Linking Kits (7044)

NOTE:
You must use 60° arc for the 2′ x 5′ canopies and a 50° arc for 4′ x 10′ canopies in this configuration.

Important: There is a different assembly process for these types of configurations. See Section 11 of the Installation Instructions (LA-297055) for details.

Please Recycle.

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1 877 276 7876
armstrong.com/infusions
CS-3908-208

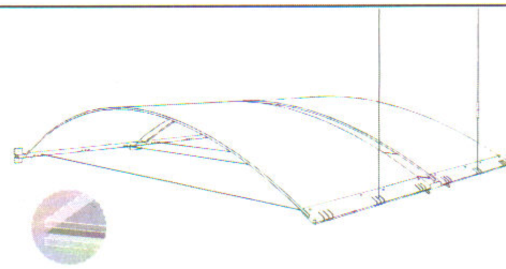
U.S. Patents Pending,
including U.S. Publication No. 2004/0182022

Armstrong®

INFUSIONS® Accent Canopies

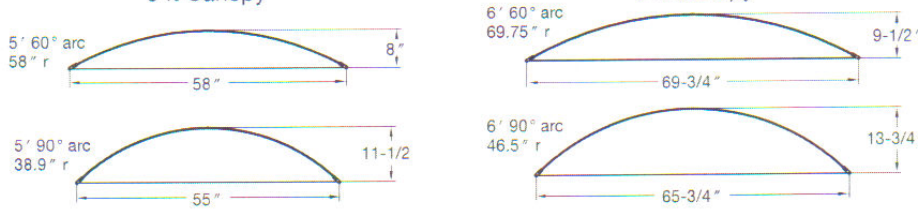
2′ x 5′ and 2′ x 6′ Panel Kits

KIT CONTENTS:
- Flat Infusions panel (1 or 2) with nominal 2′ Aluminum Extrusions (2/panel) attached
- Cables to form arcs (60° or 90°)



Infusions solids and patterns are typically made by infusing fabric of varying color, texture and translucency between layers of polycarbonate resin.

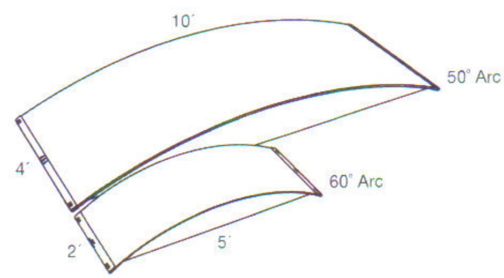
5 ft Canopy — 6 ft Canopy



4′ x 10′ Panel Kits

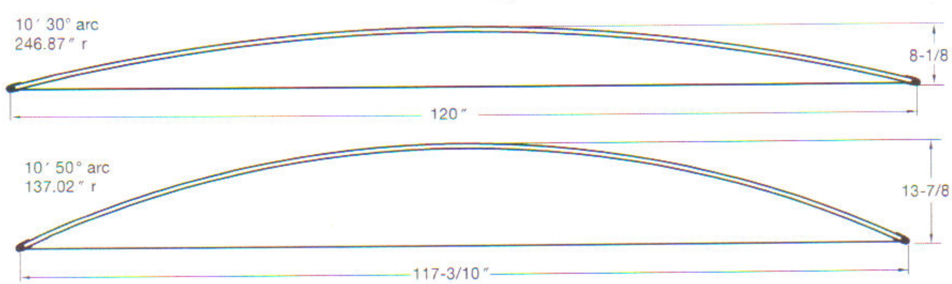
KIT CONTENTS:
- Flat Infusions panel (1 or 2) with nominal 4′ Aluminum Extrusions (2/panel) attached
- Cables to form arcs (30° or 50°)

NOTE: Panel extrusions for nominal 4′ x 10′ panels are 48-1/2″ long. Suspension points for these panels are 44-1/2″ apart.



The elevations would be different with these two different size canopies linked side-by-side.

10 ft Canopy



NOTE:
Refer to Infusions data page CS-3910 for all canopy item numbers and color options.

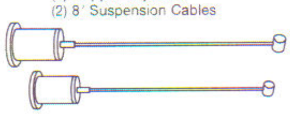
Accessory Kits

Hanging Kits

ITEM # KIT CONTENTS

7004 – Standard 8′ Hanging Kit

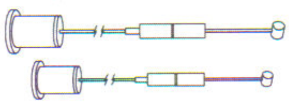
(2) Gripper Structure Anchors
(2) Gripper Adjusters
(2) 8′ Suspension Cables



7005 – Extended Hanging Kit – 16′

Allows for extended drops from deck and bottom end adjustment of height at panel.

(2) Gripper Structure Anchors
(2) Gripper Anchor Caps
(2) Upper Cables (16′)
(2) Gripper Bottom End Assemblies
(2) Bottom End Cable Adjusters



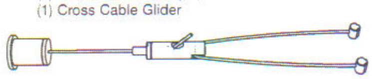
7010 – Extended Hanging Kit – 30′

Same as 7005 above except 30′ cables (2).

7041 – Dual Canopy Hanging Kit

Links two canopies end-to-end with one yoked wire to minimize visible wires.

(1) Gripper Structure Anchor
(1) Gripper Anchor Cap
(1) Upper Cable (16′)
(1) Two-ended Cable (8″)
(1) Cross Cable Glider



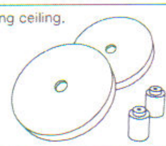
Wall and Ceiling Kits

ITEM # KIT CONTENTS

7006 – Escutcheon Kit

Used when hanging canopy below an existing ceiling.

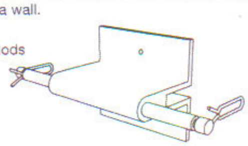
(2) Collars with Set Screws
(2) 2″ Escutcheons



7008 – Wall Attachment Kit

Anchors canopies side-by-side to a wall.

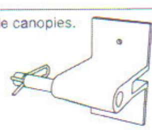
(2) Locking Clips
(2) 2-1/4″ Linking Rods
(1) 3″ Wall Bracket



7009 – Wall End Attachment Kit

Used at ends when linking single or multiple canopies.

(1) Locking Clip
(1) 2-1/4″ Linking Rod
(1) 1-1/2″ Wall Bracket



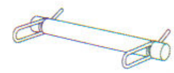
Linking Kits

ITEM # KIT CONTENTS

7042 – Flush Spacing Kit

Links two canopies side-by-side with flush spacing.

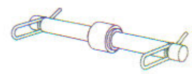
(2) Locking Clips
(1) Linking Rod



7043 – 1/2″ Spacing Kit

Links two canopies side-by-side with 1/2″ spacing.

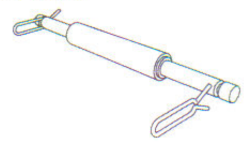
(2) Locking Clips
(1) 1/2″ Spacer
(1) Linking Rod



7007 – 3″ Spacing Kit

Links two canopies side-by-side with 3″ spacing.

(2) Locking Clips
(1) Linking Rod
(1) 3″ Spacer



7044 – Single Hinge Linking Kit

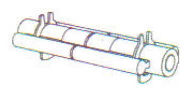
Links two canopies end-to-end. Two kits typically needed.

(1) Left Hinge
(1) Right Hinge
(1) Hinge Rod
(1) Circle Clip

7045 – Dual Hinge Flush Linking Kit

Links four canopies together end-to-end and flush side-to-side.

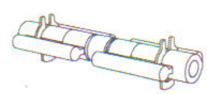
(2) Left Hinges
(2) Right Hinges
(1) Hinge Rod
(2) Circle Clips



7046 – Dual Hinge 1/2″ Linking Kit

Links four canopies together end-to-end and with 1/2″ spacing side-to-side.

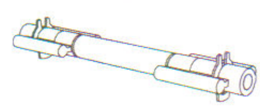
(2) Left Hinges
(2) Right Hinges
(1) Hinge Rod
(1) 1/2″ Spacer
(2) Circle Clips



7047 – Dual Hinge 3″ Linking Kit

Links four canopies together end-to-end and with 3″ spacing side-to-side.

(2) Left Hinges
(2) Right Hinges
(1) Hinge Rod
(1) 3″ Spacer
(2) Circle Clips



NOTE:

Accessory kits are compatible with all canopy sizes.

If you need assistance identifying what and how many accessory kits are needed for your project, please contact TechLine at 1 877 ARMSTRONG.

END OF SENIOR THESIS