2004 ARCHITECTURAL ENGINEERING THESIS

GLUED-LAMINATED TIMBER
ARCHITECTURAL REDESIGN

COUNTRY MUSIC HALL OF FAME AND MUSEUM

NASHVILLE, TENNESSEE

PENNSYLVANIA STATE UNIVERSITY
ARCHITECTURAL STRUCTURAL ENGINEERING
SPRING 2004

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COUNTRY MUSIC HALL OF FAME AND MUSEUM

222 Fifth Avenue South & Demonbreun ? Nashville ? Tennessee ? 37203

General

- 40,000 square-foot exhibit
- 11,000-square-foot conservatory
- 5,500 square-foot museum shop
- 2-story Archive and Library
- Ford Theater
- Full service restaurant
- XM Radio studio

Structural System

- 3 Buildings: Hall of Fame, Museum, & Conservatory
- Museum is a Steel Composite Structure
- Maximum Live Load: 350PSF for Records & Archives
- Diagonal & Chevron Bracing for Lateral Loads
- The Hall of Fame building and Cadillac Fin are architectural concrete with intentional oversized columns inside to give grandness
- Repetitive Forms Used for Symmetry
- Foundation Piers Range From 6'-24' Due to Variations in Bedrock Depth

Mechanical

- Field Assembled, Draw-Through, Variable/Constant Volume System
- 6 Zoned Air Handling Units (137,410 CFM)
- 15% Min O.A. & 100% Maximum O.A.
- 100% Min O.A. for the Conservatory
- 4-Electric Humidifiers (200-400 lb/hr) to protect Exhibits and Hall of Fame
- 550,500 BTU/HR Preheat Coil to control inconsistent temperature variations in the glass-enclosed Conservatory
- 123,000 Gallon Hot Water Heater
- Pre- & Post Filters (35% & 95%, respectively)

Architecture

Conservatory

- The heavy steel frame is inspired by the railroads and bridges that connected the small towns where country music came to life.
- A symbolic steam follows the descending monumental stair represents the streams of the Appalachia and into the Mississippi Delta.

Museum

- The form of the museum inspires the country stores that feature large façades and signage but are really intimate spaces where people come to socialize and exchange information.
- The musical reference of the giant keyboard formed by the series of vertical windows positioned like ebony keys across the dominant, curved front façade.
- The tail fin of a '57 Chevy is inspired in the dramatic end of the concrete wall that rises above the street corner.

Hall of Fame

- The cylindrical shape is based on the water towers that nourished steam engines and grain silos dotting rural landscapes.
- A replica of the WSM tower pierces the rotunda roof.
- Four concentric circles representing the 78-, 45- and 33-rpm records and the compact disc create that stair stepping roof.

Construction

- Pre Construction December 1995
- Construction June 1999
- Finish May 2001
- Total Cost: $37,000,000

Electrical

- 3 Phase, 4 Wire, 277/480 Volts Wye & 120/208 Volts Wye
- 277/480 Volt Emergency Standby Engine Generator
- Illuminations: Incandescent, T8 Fluorescents, Cold Cathodes, Halogens, Metal Halides, & Neon

OWNER: COUNTRY MUSIC FOUNDATION
ARCHITECT: TUCK-HINTON ARCHITECTS
STRUCTURAL: EMC STRUCTURAL ENGINEERS
MEP: I.C. THOMASSON
CM: AM CONSTRUCTORS

DAVID L. CLARK II
STRUCTURAL EMPHASIS

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I would like to thank my project sponsor, facility, and outside consultants for providing resources and the opportunity to use this building for my senior thesis project.

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COUNTRY MUSIC HALL OF FAME AND MUSEUM

"A Bass Clef,...

a Cadillac fin, and. . .

an Old Country Church"

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Executive Summary

Tuck Hinton Architects knew from the beginning that the new *Country Music Hall of Fame & Museum* was about more than country music and the stars that brought it to life. The design firm wanted in this commission to celebrate country music's origins, inspirations and people. They wanted it to be a place of discovery and meaning as well as a historical archive. "We spent a great deal of time and effort investigating the roots and origins of this indigenous American musical art form," said Seab Tuck III, principal-in-charge for Tuck Hinton Architects. “Country music is about relationships, so we interviewed many individuals to hopefully discover its essential qualities and to also identify its special, unique images." Those interviews led the firm to study general stores, country churches, railroads, bridges, silos and even prisons. They researched pickup trucks, vintage cars, musical instruments, classic country songs and anything that might have inspired musicians. Country music's symbolism uncovered in this research became the foundation for the architectural design of the new $37 million facility in downtown Nashville. The facility is three distinct functions and building forms that combine into a cohesive whole: the Conservatory, a large introductory space full of natural light; the Museum, which chronicles the music, past and present; and the Hall of Fame, which honors the music's heroes and heroines. There is even more country imagery in the design of the curved concrete wall which anchors the building's front facade. "The curved wall embraces the city, and it's in the shape of a bass clef," Tuck said. "The openings in the wall represent the keys of a piano, but also give the appearance of prison bars. And the wall flares out at the end in a manner reminiscent of the tail fin of a 1957 Chevy.

Tuck Hinton Architects designed a form for the museum that would be inspired by the country stores that feature large façades and signage but are really intimate spaces where people come to socialize and exchange information. The depth work focused in this report intends to further strengthen and embrace the museum’s interior country general store appearance by partially replacing the exposed steel structural with naturally stained Southern Pine glued-laminated timbers. Glued-laminated timbers, abbreviated glu-lam, is sought after to bring out the warmth and the strength of the structure as well as it’s advantages in it’s strength to weight, durability, fire resistance, and chemical resistance. In typical cases glued-laminated timbers can be 20 % lighter than structural steel and 600 % than a reinforced concrete member. In addition the higher thermal insulation characteristics of timber and the charcoal layer that forms on it ensure that the interior of a fire exposed member remains cool and structurally sound over the design period. In addition glued-laminated behaves as a single piece throughout its exposure to fire because of the high resistance of laminating adhesives to fire temperatures.

1.0 Building Overview

1.1 Building Codes

The principle building code used for this building is the Standard Building Code 1994 Edition. Other design codes and material strengths are listed below.

Reinforced Concrete (ACI 318-95)

Piers and Walls	4,000 psi
Beams, Elevated Slabs and Columns	4,000 psi
All Other Concrete	3,000 psi

Architectural Exposed Cast In Place Concrete

Walls	4,000 psi
Columns	4,000 psi

Steel (9th Edition AISC ASD & LRFD Second Edition)

Structural Steel	ASTM A-36/572 Grade 50
Steel Plates & Angles	ASTM A-36
Reinforcing Steel	ASTM A-615 Grade 60
Framing Connections	ASTM A-325-N
Anchor Bolts	ASTM A-307

Masonry (ASTM C-476)

Fig 1.1 Museum At Night (Tuck Hinton Architects)

1.2 Structure

The Country Music Hall of Fame & Museum is three distinct buildings that combine into a cohesive whole: the *Museum*, which chronicles the music, past and present; the *Conservatory*, a large introductory space full of natural light; and the *Hall of Fame (H.O.F.)*, which honors the music's heroes and heroines.

The one-hundred thirty five thousand square foot, four-story *Country Music Hall of Fame* at 5th Avenue South and Demonbreun rest on one-hundred thirty-one drilled concrete reinforced caissons socketed into bedrock some twenty-feet feet below the surface. A series of grade beams rest on these caissons with steel columns and floors on top of the grade beams.

1.2.1 Museum

The museum is the dominant structure of the three buildings. This structure's most significant features are the giant keyboard formed by the series of vertical windows positioned like ebony keys across the dominant, concrete curved front façade with the end of the wall rising above the street corner like the tail fin of a 1957 Chevrolet automobile (*Fig 1.2*). The museum also has a two-story glass vault that contains a million piece musical archive most of it being vintage –78 rpm records.

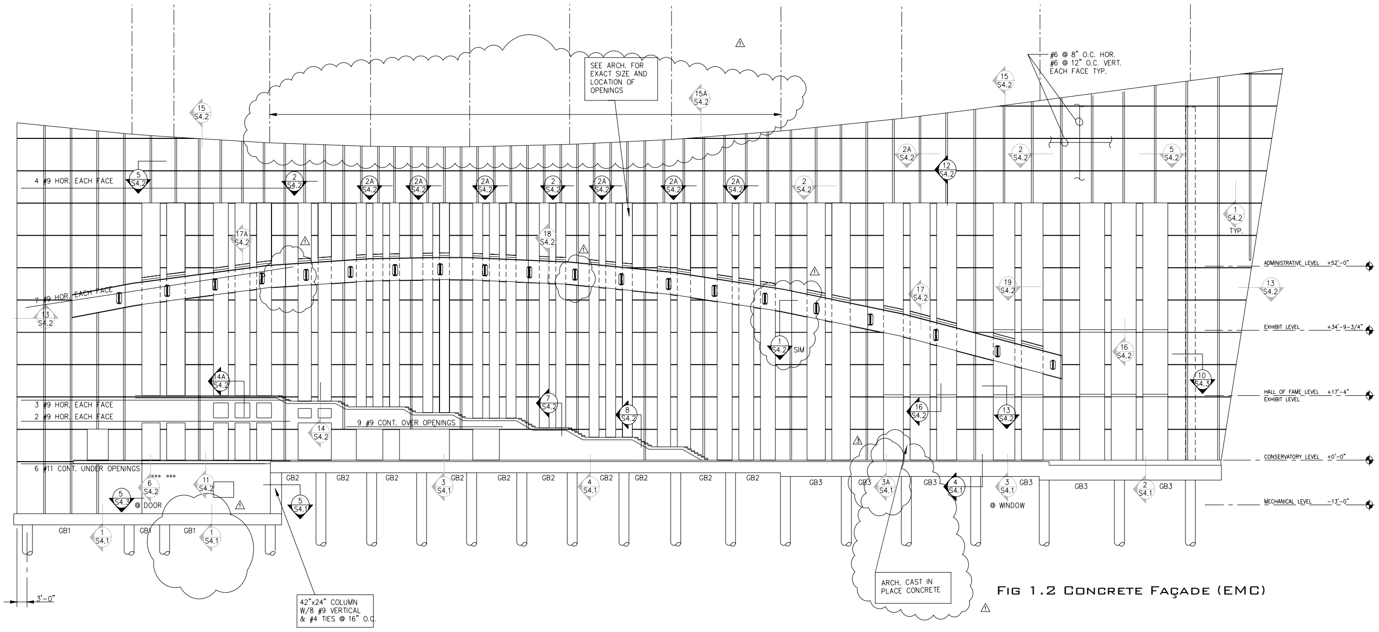


Fig 1.2 Concrete Façade (EMC)

The museum itself is a four story (average floor to floor height of 17 ft) composite steel frame building with ten 30 ft grids in the left to right direction and eight 20 ft grids in the front to rear direction. The rear, left, and right side of the building and the core uses W10 & W14 columns (with the exception of the rear eccentrically load W40×199 columns to support a cantilevered walkway) to transfer the axial loads to the grade beams and caissons. Only the right-rear corner has a ninety degree angle, the left-rear side is tapered at a sixteen degree angle from the rear wall and the front corners are different in themselves.

The front side of the building is a load-bearing architectural cast in place concrete wall that takes place of wide flange columns that would carry the axial forces on the exterior front side (*Fig 1.2*). The concrete wall is thirty inches thick with a maximum height of one-hundred four feet and has a two-hundred feet radii curve with an eighty-one and a quarter degree sweep. At the right-front corner the concrete wall ends at a forty-five degree angle from the right side. The concrete wall rests on 20-42 in & 48 in diameter caissons and 42”× (30”,36”,& 48”) grade beams (*Fig 1.3*).

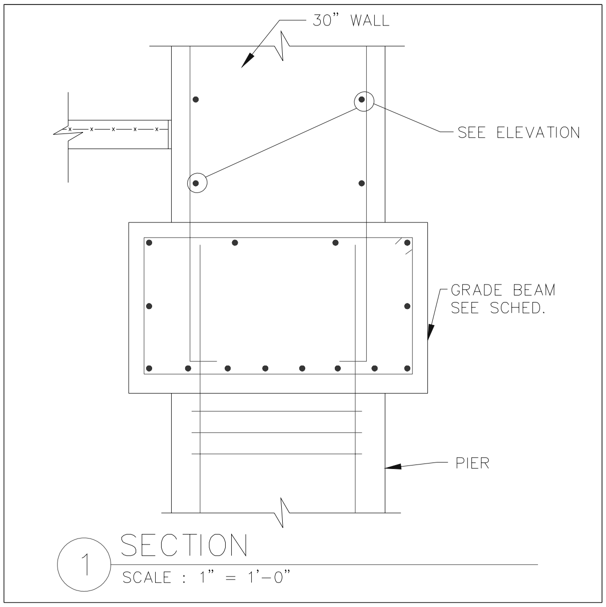


Fig 1.3 Typical Caisson & Grade Beam w/ Front Façade Wall (EMC)

The connection of the steel girders and beams to the concrete wall are attached with plates which are fasten on to the wall with anchor bolts. A cumbersome task for this project was the alignment of these girders and beams into this concrete wall. To support the steel frame glass roof of the conservatory which will be discussed in the next section, a continuous 26”×72” curved reinforced concrete beam was formed within the mid-part of the wall. The W16×135 conservatory roof girders are attached to this concrete beam by 3/4”×12”×38” and 3/8”×4”×27” plates which were anchored with 8-3/4” diameter anchor bolts into this concrete beam (*Fig 1.4*). To support the museum’s multi-story girders on the opposite side, embedded 3/4”×11”×3’ plates with 20-3/4”×10” studs were used in place (*Fig 1.5*). The metal decking was constrained with a 15×33.9” channel bars that run the entire length of the concrete wall for each floor.

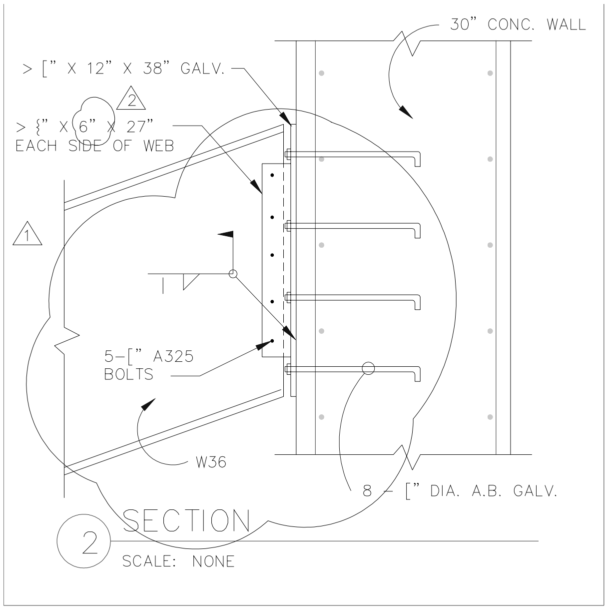


Fig 1.4 Conservatory Roof Connection (EMC)

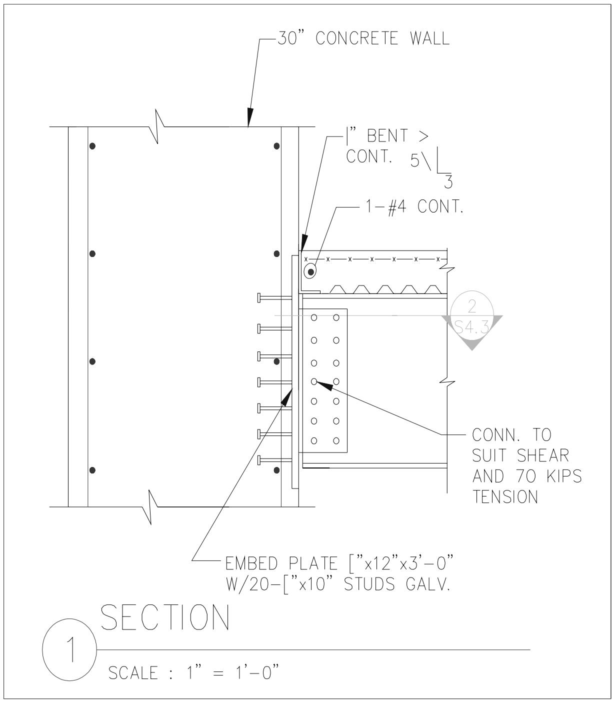


Fig 1.5 Museum Connection into Front Façade (EMC)

The museum rests on 30 in & 36 in

diameter caissons and a wide size of grade beams. As mentioned the columns are mostly W10s & W16s, while girders range from W18s, W21s, W24s, & W27s (W36×194s for the 3rd floor 20' cantilevered walkway), beams in the majority are W16s and the 2-1/2 : 12 mono-sloped roof has a wide depth variation of open web joists.

The main level has a 4 in concrete slab reinforced with WWF 6×6-W1.4×W1.4, while the upper floors are 5-1/2 in light weigh concrete slab over 2 in 20 gauge composite floor deck with WWF 6×6-W1.4×W1.4 and the roof is a 1-1/2 in gauge wide rib galvanized metal roof deck.

Eight inch CMU blocks were used as interior partitions while the exterior walls are framed with steel studs and Tennessee limestone cladding.

The museum has four shear walls built into the elevator shafts and four exterior lateral braced frames (*Fig 1.6*). The longitudinal (*9 & 19 in Fig 1.6*) and tangential braced frames (*10 & 20 in Fig 1.6*) are connected in a "T" at the rear sides of the building, each

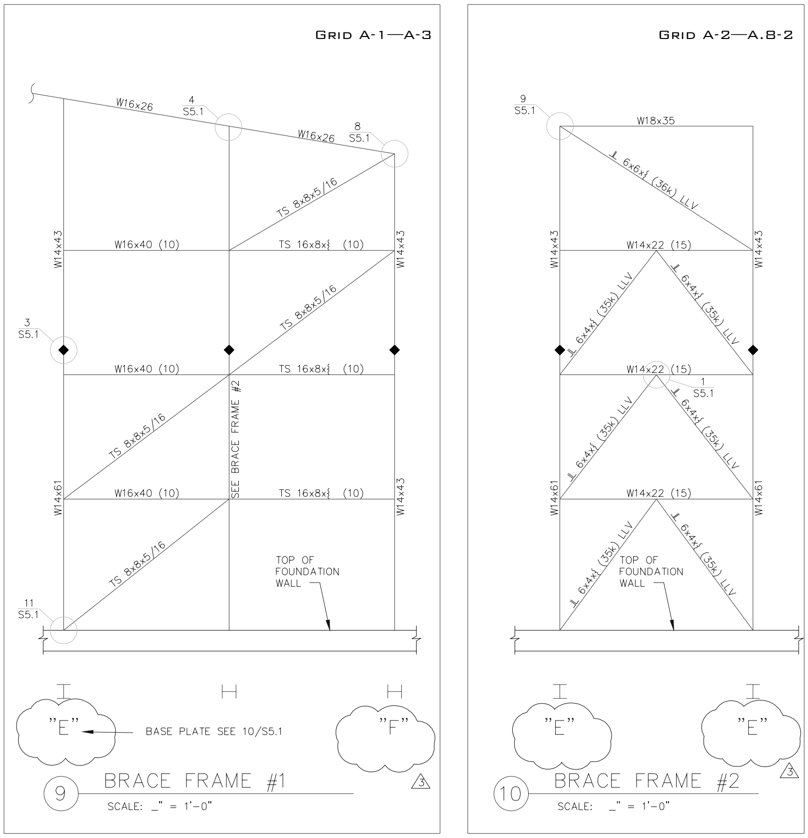
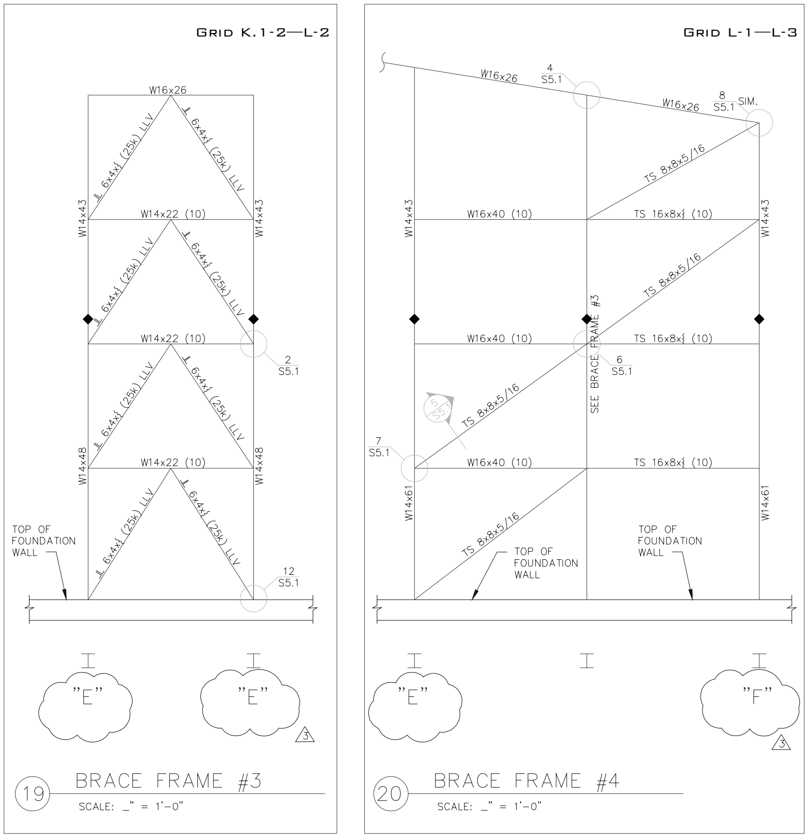


Fig 1.6 Building's Lateral Braced Frames (9 & 19 Upper, 10 & 20 Lower, Grid Coordination Given, See Appendix A for Strucutral Floor Plan) (EMC)



set on opposite rear corners. The longitudinal shear wall consist of two bays and the tangential one bay.

1.2.2 Conservatory

The 11,000-square-foot Conservatory architecturally anchors the museum's entrance. The conservatory is a simple steel glass enclosed structure with W10×22 columns, W16×135 roof girders, and W18×35 purlins.

1.2.3 Hall of Fame

The *Hall of Fame* was set apart from the museum rather than buried within it. The 2-story ninety-four foot diameter drum encloses the *Hall of Fame* and a two-hundred twenty seat *Ford* theater directly below it. The entire structure is constructed with architectural cast-in place concrete and a steel-topped roof. The original design was an entire steel structure with masonry cladding however due to cost architectural concrete was used instead.

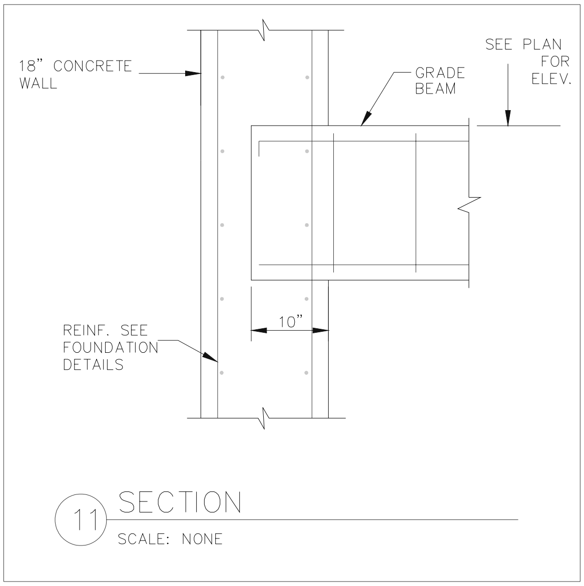


Fig 1.7 Caisson & Grade Beam w/ Outer Drum Wall (EMC)

The outer shell of the ninety-four foot drum is an eighteen inch thick, fifty-two foot high concrete reinforced circular wall with a foundation of sixteen 30 in diameter caissons and 30"×30" grade beam (*Fig 1.7*). Inside fourteen-28"×32" concrete columns equally segmented thirty-one feet from the center of the drum support the second level, the *Hall of Fame* (*Fig 1.8*). These columns rests on a 30"×30" grade beam directly above eight 36 in diameter equally segmented caissons.

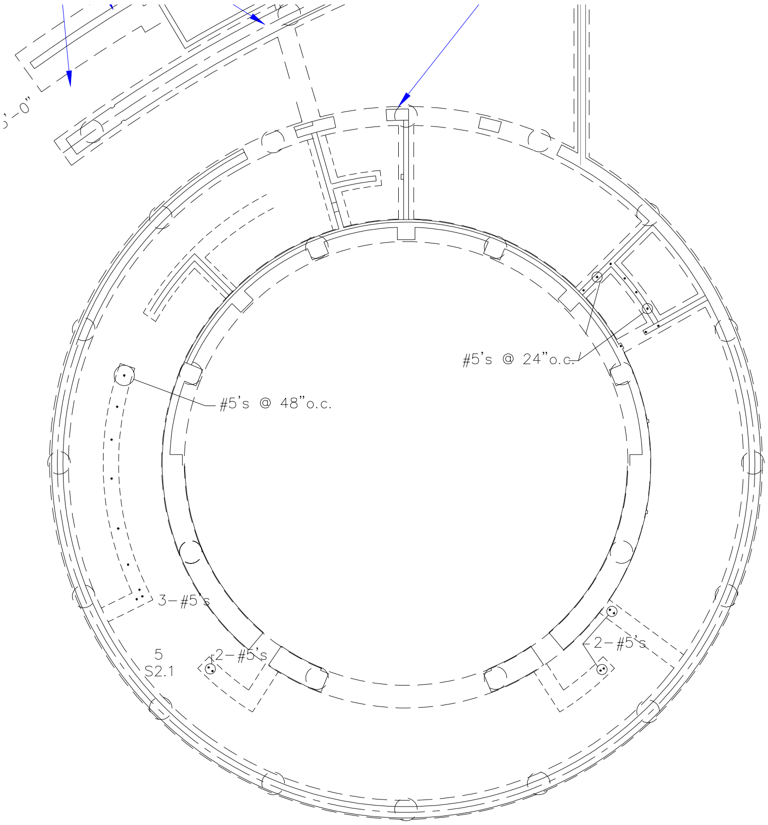


Fig 1.8 First Floor HOF Structural Layout (EMC)

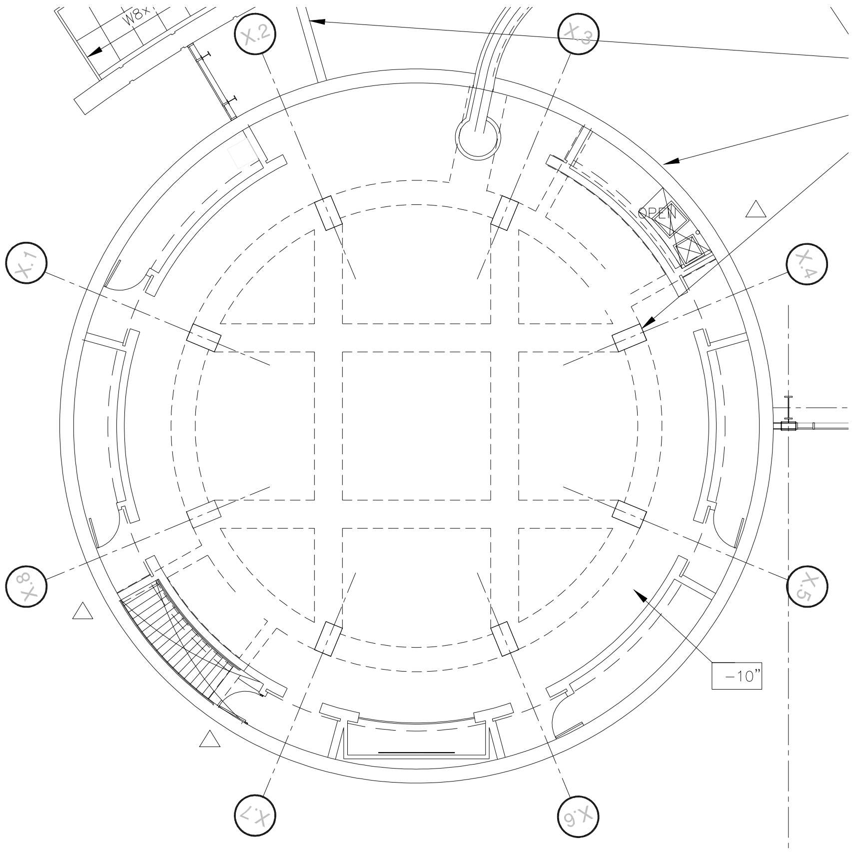


Fig 1.9 Second Floor H.O.F. Structural Layout (EMC)

The second level floor is supported by 32”×31” circumferential beam, that constrains the 14 columns, and four encompassed 44”×31” girders (each pair transversed) that connect into the intermittent eightcolumns that are directly above the caissons. (The design resembles an encompassed “tic-tac-toe board”) *(Fig 1.9)*. The circumferential beam is reinforced with top and bottom 6-#9 bars and the four girders with 4-#8 top bars and 10-#11 bottom bars *(Fig 1.10)*.

The second level floor is a comprised of a 9 in structural slab with a 4 in regular slab. The eight inner columns above the 36 in caissons that connect into the four girders extends beyond the

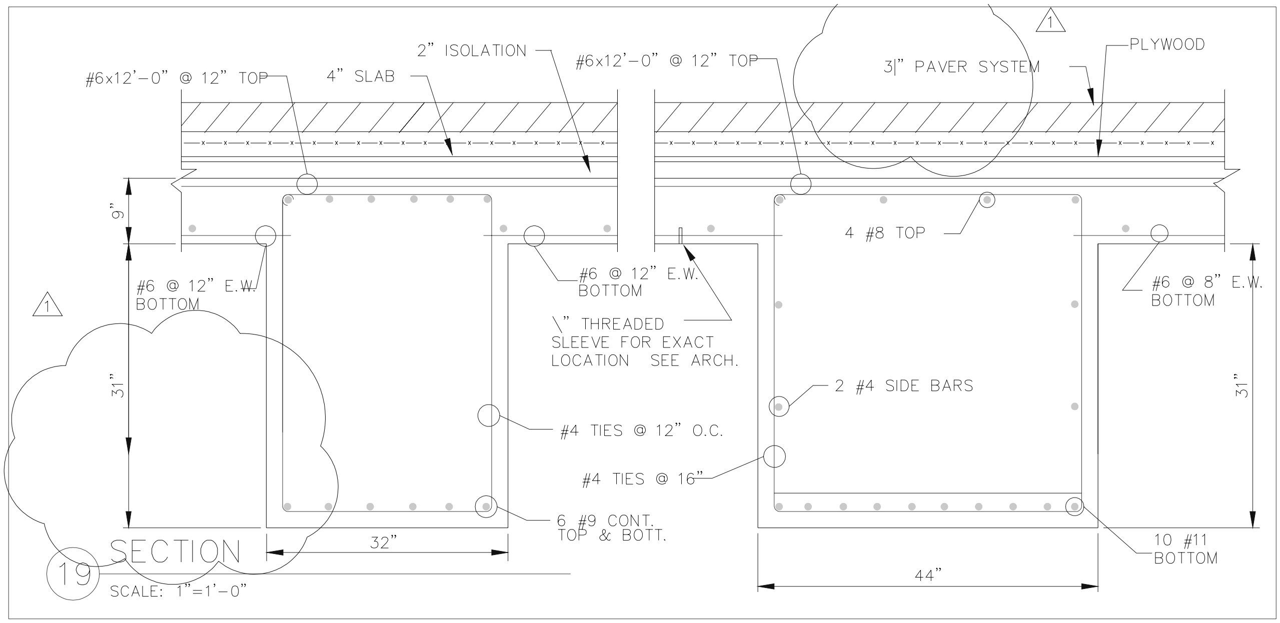


Fig 1.10 Section View of Second Floor Structure (EMC)

second level to four stair stepping steel framed roofs. The four roofs, constructed from wide flange beams, are supported by those eight columns. Each roof is framed as an octagon shape with the roof joists converging from the center of the drum in 48 equal segments, except for the upper roof, which is in 24 equal segments (all of the frames look like spider webs) *(Fig 1.11)*.

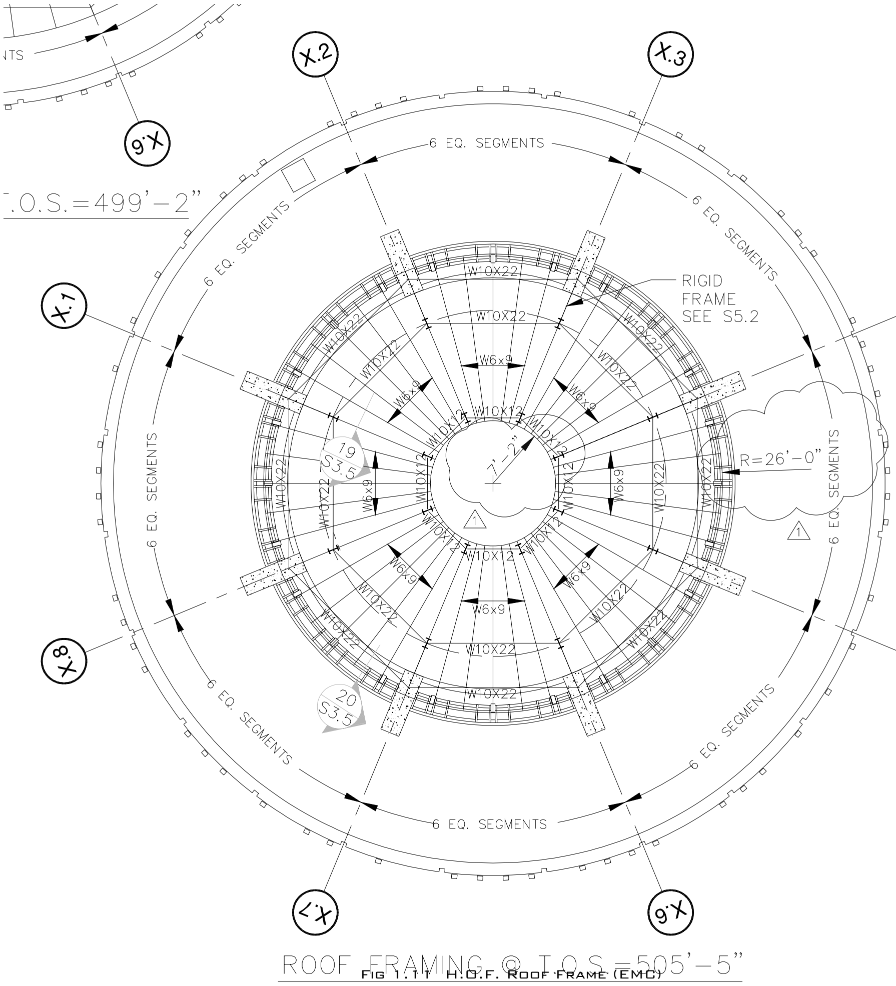


Fig 1.11 H.O.F. Roof Frame (EMC)

2.0 Depth Structural Proposal

2.1 Building Codes

Before redesigning the museum's structure with glued-laminated timbers, the proposed design must be confirmed with a current code to establish if the building can be safely framed with heavy timber. The museum was designed with the Southern Building Code (SBC) 1994 and is being redesigned with the new International Building Code 2000 (IBC). The museum has several diverse occupancies and building materials. IBC 2000 classifies the proposed structure as an Assembly Type III with Type IV heavy timber construction. The museum is a four story structure with an average height of 76 feet and a maximum floor area of 47,670 square feet. The number of floors and square footage exceeds IBC 2000 requirements for A-III, Type IV construction and is rejected. None the less, by incorporating sprinklers and frontage, which the building is so equipped, IBC 2000 grants an additional floor and 275 % square footage thus now approving the museum to be constructed as A-III, Type IV construction. IBC 2000 dictates minimum requirements and the city of Nashville, TN, may have a superior hierarchy of codes, but for simplicity, this report will use IBC 2000. Knowing that IBC 2000 will consent in allowing the museum to be designed as heavy timber, an assessment now can be made as what structural members will enhance the interior of the museum to the desired effect upon redesigning to glued-laminated timbers. It has been determined that the *Hall of Fame* will obtain a redesigned glued-laminated framed roof and the museum will obtain glued-laminated roof trusses and a glued-laminated floor frame. The timber design will follow the NDS 2001 Edition and allowable strength design (ASD) guidelines. Table 2.1 on the following page compares Type I SBC 1994 and Type IV IBC 2000 for this building.

Table 2.1 (Code Assessment: SBC 1994 & IBC 2000 for A-III, Type IV Heavy Timber Construction)

Governing Codes	**SBC 1994**	**IBC 2000**	**IBC Modifications**	**NFPA**	**NOTES**	
Occupancy Classification						
Primary Groups	A-1	A-3				
Other Groups	B (Business)	B				
	M (Mercantile)	M				
	S (Storage)	S-2				
Building Height						
Max Height Allowed	80	65	85		**Sec 504.2**	
Average Height	76					
Max Number of Stories	No Limit	3	4		**Sec 504.2**	
Actual Number of Stories	4				**Sec 506**	
Building Area					*506.3*	
Max Sq Ft Allowed	Unlimited	15,000 / floor	+ 41,250 / floor		Is	200.0%
Designed SQ FT					*506.2*	
Basement	4,824	15,000			F/P	1
First Floor	47,670	15,000	56,250 *(506.2)*		W/30	1
Second Floor	33,103	15,000	45000 *(506.3)*		If	75.0%
Third Floor	21,701	15,000	45000 *(506.3)*			
Fourth Floor	23,285	15,000	45000 *(506.3)*			
Zoning Requirements						
Present Zoning	CF					
Site Set-Backs						
Front	10					
Side	0					
Rear	20 = W					
Max Floor Area Ratio	5					
Fire Limits						
A. Building is in a municipal						
B. Fire Limit restrictions						
Construction Type	Type II, Sprinklered					
Travel Distance	250	250 *(1004.2.4)*	X	200		

The following are the proposed changes for the three spaces:

2.2 Hall of Fame

FIG 2.1 Hall of Fame (Tuck Hinton Architects)

The Hall of Fame is a dramatic concrete, stone and steel-topped structure that is positioned close to the street in front of the museum. Its round form sets an egalitarian tone where all of the inductees to the Hall of Fame stand on equal footing. Natural light in the rotunda comes from above through windows that are patterned after the original museum building. Four concentric circles representing the 78-, 45- and 33-rpm records and the compact disc together create the shape of the Hall of Fame's stair-stepping roof. . A replica of the *W.S.M.* (*We Shield Millions*) tower, which has broadcasted the *Grand Ole Opry* to the nation for more than sixty years and established country music as a radio staple, pierces the rotunda roof and hangs from the 60-foot ceiling of the round hall. The proposal for the *Hall of Fame* is to redesign the stair steeping roof members with either exposed dimensional rough saw or glued-laminated timbers.

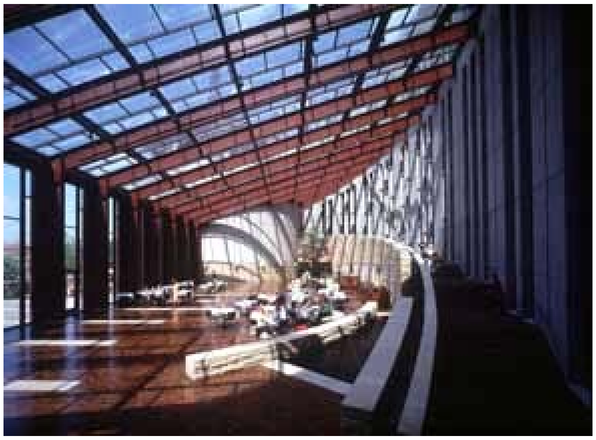


Fig 2.2 Upper: Interior of the Conservatory
Fig 2.3 Middle: Exterior of the Museum)
Fig 2.4 Lower: Interior of the Museum seen from the 3rd Floor
(Tuck Hinton Architects)

2.3 Conservatory

The 11,000 square foot Conservatory architecturally anchors the museum's entrance, seen in the upper right picture (Fig 2.2). The conservatory is a steel frame of columns, rafters, and purlins, curtained with glass. The architects drew inspiration for its heavy steel frame from railroads and bridges that connect the small towns where country music came to life. Because of the steel's symbolization there will be no changes to this structure. This will remain the only exposed steel structure in the facility seen by visitors.

2.4 Museum

This building's first and most obvious musical reference is the giant keyboard formed by the series of vertical windows positioned like ebony keys across the dominant, curved front façade (Fig 2.3). The tail fin of a '57 Chevy inspired the dramatic end of the concrete wall that rises above the street corner. The structural emphasis for the museum will be redesigning the museum's interior structural steel composite frame to glued-laminated timbers.

The museum's existing frame is a steel composite structure with braced lateral frames. The proposed design is to keep the existing second floor steel frame and all of the exterior steel columns and build the upper, interior frames with structural glued-laminated timbers. The first floor contains non-public spaces and combustible areas (mechanical rooms and a commercial kitchen). The second floor's concrete slab will be kept as a fire barrier between the first and upper floors. The second and third floors are public exhibit spaces and the topmost floor are administrative offices. The two upper most floors have an exposed steel structure and this is what will be redesigned with glued-laminated timbers. In summary the museum will be a one story steel frame composite structure with the steel exterior columns and lateral frames extending to the roof and an upper two-story glued-laminated interior frame sheltered by a glued-laminated heavy timber truss roof.

Most of the interior columns are non-continuous from floor to floor due to overhanging beams and the short length won't be a concern for the design of glued-laminates. The exterior columns are continuous and extend from the foundation to the roof with an intermediate splice part way and the side exterior columns are in lateral braced frames. To achieve the rear columns with glued-laminates, the column sizes will be considerable greater than the steel columns and slenderness and thermal and moisture movement will have a major involvement. In the side exterior columns, lateral braced frames are possible with wood using diagonal tension rods or shear walls. But more important is story shear transfer at the second floor. If the upper exterior columns are glued-laminates, the wood to steel connection at the second floor would be a pinned connection and this type of connection would need to be as strong as a continuous piece of steel or steel splices. To achieve story shear transfer would require a heavily reinforce connection which can be very costly. Thus the exterior columns will be kept to steel as originally designed and the interior columns will be redesigned with glued-laminates. As for what is being achieved in the museum, only one side of the exterior steel column will be seen, and by masquerading the exposed side with a dimensional board will conceal the steel. In between the exterior and interior columns, the steel studs with gypsum board will be kept unchanged .

The interior girders and joists will be designed using normal glued-laminates. In areas of high tension experimental fiberglass reinforced plastic glued-laminates will be assessed. Fiberglass reinforced plastic glued-laminates, or FiRPs, are manufactured with one or more thin layers of a fiber reinforced panel. The reinforcing consists of high-strength fibers embedded in a matrix and are strategitically placed between certain laminations to increase beam strength and stiffness. Three types of reinforcing are currently approved for use in FIRP beams: Aramid, Carbon, and Fiberglass.

The addition of reinforcing generally permits a reduction in beam width reducing the volume of wood fiber used. The FiRPS will be designed under accordance with ICC Evaluation Service, Inc. published supplement, PFC 5100, Division 06-Wood & Plastic, reissued May 1, 2003. One important concern when redesigning the floor system is that the framing layout is to keep the original design to avoid any new MEP coordination.

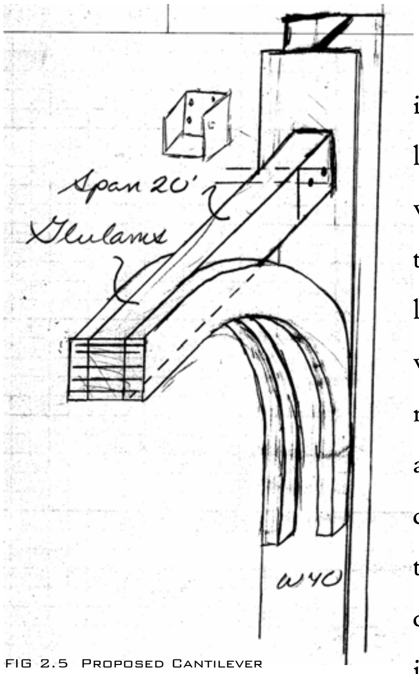


FIG 2.5 Proposed Cantilever

FIG 2.6 An Arch Bridge (O.D.O.T.)

One of the structural impediments in the museum is the cantilevered walkway located on the third floor. The cantilevered walkway has a twenty foot span with a thirty feet spacing and a 100 psf short-term load. Glued-laminated timber don't perform well as cantilevers chiefly due to the moment connection required, so other alternatives had to be devised. Alternatives were to sustain the cantilever at the free end with either a column or a tensed rod but there is a strong desire to preserve the drama of the cantilever and other alternatives were sought after. The proposed design (*Fig 2.5*) is influenced by the construction of arch bridges (*Fig 2.6*). The curved glued-laminates will carry the bending to bearing into the steel column while the horizontal piece will act as beam-column resisting tension and bending carried over. Most arch bridges start in two phases, they start at the ends and join in the middle where it is joined. If half an arch can support itself alone then this would seem viable for the cantilever walkway in the building. If the design works the only disquiet is clearance height for the floor below which is not an issue. Floor to floor height is approximately 17 to 18 feet and the curvature of glued-laminates will be governed by a Southern Pine minimum radius (18 feet) which will be sought after to reduce clearance restrictions.

2.5 Roofs

The last redesign in the museum's is the 2-1/2 : 12 mono-sloped roof. The mono-sloped roof is supported by a series of forty-eight inch open web joists which are intended to be replaced with glued-laminated timber trusses. The loads that exist on the roof are: short-term load, wind loads (including uplift) and mechanical loads (duct work and wenches). The mechanical wenches will be excluded from the design, due to only a visual observation and unavailable documented information.

2.6 Decking

The floor decking will be a tongue and groove decking acting as formwork with light weight concrete for stiffness and a laminated decking for the wearing surface (Fig 2.7). The roof decking will be existing with the exception of laminated roof decking (Fig 2.8).

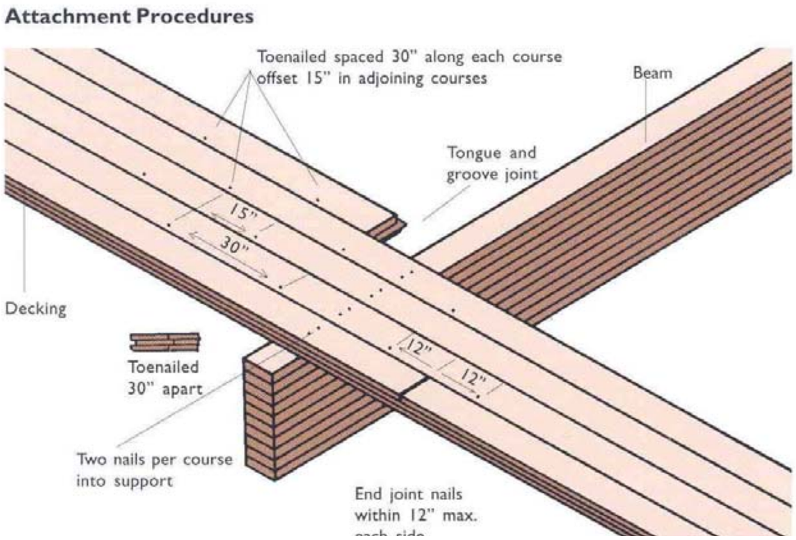


FIG 2.7 Tongue & Groove Decking (Rigidply Rafters)

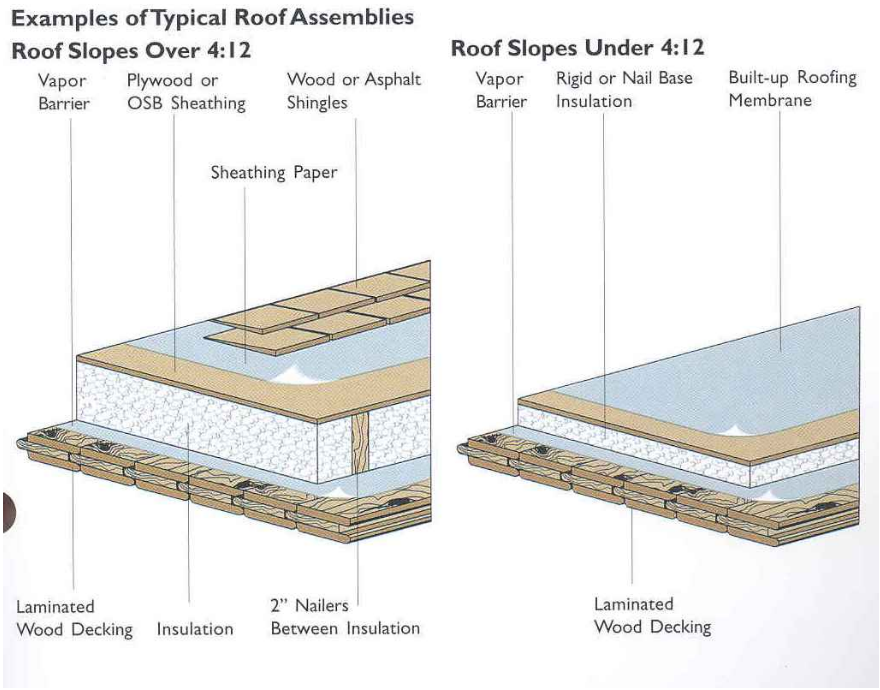


FIG 2.8 Laminated Roof Decking (Rigidply Rafters)

3.0 Breadth Disciplinary Design

3.1 Construction Management (Chapter 8)

The CM breadth consists of a cost comparison between the existing structure and the new proposed structure. The cost for redesigning part of the museum with glued-laminates will greatly increase the costs with materials and having a second structural erecting crew.

3.2 Lighting (Chapter 9)

The lighting breadth consists of the addition of sky lights on the museum's roof to provided natural light to the fourth floor office spaces. The principal disquiet with day lighting office spaces is glare. Shading will be recommended.

The use of skylights has grown in recent years, both because they enliven building interiors and because they can save energy dollars through daylighting. Sky lighting can be a solid asset for buildings, and satisfying for building designers, occupants, and owners. Skylights can make a number of major contributions to the built environment since they:

- Provide excellent lighting conditions to the interior of buildings
- Reduce the use of electric lighting, to save energy and reduce peak electric loads when combined with photo controls.
- Satisfy human needs for contact with the outdoors
- Increase safety and security with highly reliable daytime lighting.
- Provide emergency smoke vents.

3.3 Mechanical (Chapter 10)

The mechanical breadth consists of the installment of thermal ice storage units to save utility cost during on-peak utility time. To reduce the museum's on-peak electricity demand and cost, a thermal ice system is being proposed to help reduce energy costs by allowing the energy-intensive electrically driven cooling equipment to operate during off-peak utility hours when electricity rates are at their lowest. Ice storage generates ice at night during off-peak utility hours and stores it to cool a building the next day during utility's on-peak demand time. The California Energy Commission estimates an average of 12 % fewer kWh are used with ice thermal storage. With a low cost of operation and the potential for the lowest first cost, ice storage offers an energy-saving technology to accommodate new changes and trends in the electric power industry.

3.4 Architectural (Chapter 11)

The architectural breadth consists of the new redesign of the museum using exposed glued-laminated timbers. Examples of architectural glued-laminated structures are provided in chapter 11.

4.0 Glued-laminated Timber Overview

Glued wood structural members are manufactured in a variety of configurations. Structural composite lumber, or SCL, consist of small pieces of wood glued together to form sizes that are common to that of solid sawn lumber. SCL was developed in response to the increasing demand for high quality and unrestrained sized lumber at a time when it became difficult to obtain this type of lumber naturally from a forest. There are several types of SCL products manufactured. The first product is laminated veneer lumber or LVL and is manufactured by laminating veneer with all the plies parallel to the length. The second product, depending on the component material, is laminated strand lumber (LSL), parallel strand lumber (PSL), and oriented strand lumber (OSL). This product consists of strands of wood or strips, such as aspen or other underutilized or juvenile species, of veneer glued under high pressure and temperatures. The third product is glued-laminated timber (glulam for short) which consists of two or more layers of lumber in which the grain of all the layers is oriented parallel to the length of the member. And finally SCL can also include lumber that is glued to panel products, such as box beams and I-beams, and structural sandwich construction.

Structural composite lumber is a growing segment of the engineered wood products industry. It is used as a replacement for lumber in various applications and in the manufacture of other engineered wood products, such as prefabricated wood I-joists, which take advantage of engineering design values that can be greater than those commonly assigned to sawn lumber.

Glued-laminated timbers was first used in Europe in the construction of an auditorium in Basel, Switzerland in 1893 A.D.. It was patented as the “Hertzer System” and used non-waterproof adhesives which limited it self to dry use conditions. Improvements in adhesives during the Great World War stimulated the use of glued-laminated timbers in aircraft and building frames. During the second Great World War, the development of synthetic resin adhesive that were waterproof made it possible for glued-laminated timbers to be used extensively for every practical application. Structures that would not have been feasible using only sawn-timber members have proved practical and successful using glued-laminates. Glued-laminates were originally developed to produce curved members such as large arches. Later they were used to solve the problems faced in obtaining sawn timbers of both large size and good quality. Glued-laminates have the following advantages over sawn timbers:

a. Greater strength
b. Larger cross section
c. Longer spans
d. Freedom from warping and checking
e. Diversity of shapes
f. Varying section properties
g. Camber
h. Fire resistance
i. Economy

4.1 Grades and Combinations

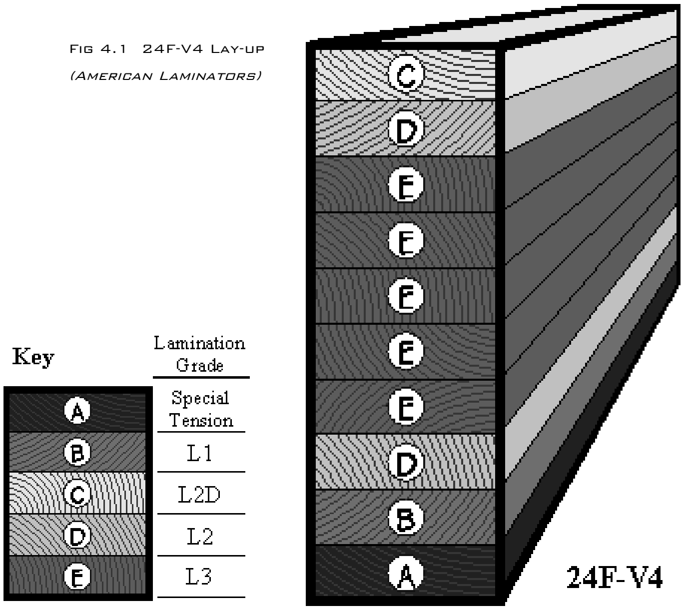


Fig 4.1 24F-V4 Lay-up
(American Laminators)

Wood for glued-laminated laminations are graded under special rules. Grading may be either visual or by machine. Visual grading is based on the occurrence and spacing of defects such as knots and cross grains. Groupings of laminations by strength are specified by the AITC specifications. These groupings are called combinations and are designated by terms such as combination 24F-V4. The numerals preceding the first letter indicate the allowable flexural stress – the 24, for example, meaning 2,400 psi. The letter “V” indicates visual grading of the individual lamination; a letter “E” is used to show machine grading. The “4” of the combination number indicates the fourth combination in the series that has the 2,200 psi bending strength (Fig 4.1).

Douglas Fir-Larch, Southern Pine, Hem-Fir, and Spruce-Pine-Fir (SPF) are commonly used for glued-laminate in the United States. Nearly any species can be used for glued-laminated timber, provided its mechanical and physical properties are suitable and it can be properly glued. Industry standards cover many softwoods and hardwoods, and procedures are in place for including other species

4.2 Adhesives

Two types of glue are permitted in the fabrication of glued-laminated members: (1) dry-use adhesive (casein glue) and (2) wet-use adhesives (usually phenol-resorcinol-base, resorcinol-base, or melamine-base adhesives). Both types of glue are capable of producing joints which have horizontal shear capabilities in excess of the capacity of the wood itself.

5.0 Design Loads

5.1 Load Combinations

Almost every building is exerted by both in-plane (gravity) and out-of-plane (lateral) forces. The forces that were used in this report are listed in Table 5.1, with their duration factors. Since the building weight was reduced with the replacement of glued-laminates seismic forces were not calculated based on this assumption. The lateral frames in the building were not altered to glued-laminates and wind loads were only used for sizing the roof trusses. The duration factors included in the table are used in allowable stress design because design stresses are based on an assumed 10-year loading period (called normal loading). If duration of loading, either continuously or cumulatively, is expected to exceed 10 years, design stresses are reduced 10 %.

Table 5.1 Load Types & Durations

Load Type	Duration Factor (C_D)
Long-Term (D)	0.9
Floor Short-Term (LL)	1.0
Roof Short-Term (LL)	1.25
Snow (SL)	1.6
Wind (WL)	1.6

In order to determine the critical load combination and associated duration-of-load factor, each load combination is divided by the applicable duration-of-load factors. The highest value determines the critical combination of loads.

$D/0.9$

$(D+LL)/1.25$

$(D+LL+SL)/1.15$

$(D+LL+WL)/1.33$

$(D+LL+SL/2+WL)/1.33$

$(D+LL+SL+WL/2)/1.33$

5.2 In-plane Loadings

5.2.1 Long-Term Loads

Of all the loads a designer must consider, long-term loads are the one that can be determined the most precisely, and it generally remains constant for the life of the structure. Since the concern is design wood members, it will be necessary to estimate the weight of the wood member. From the average specific gravity, the average weight per cubic foot for oven dry wood may be computed.

Since glued-laminated wood will contain approximately 16 % moisture, the computed weight per cubic foot must be adjusted upward to account for moisture content. The average specific gravity (SG) for Southern Pine (SP) based on oven-dry weight is 0.55 (SDIW Table C.1). Knowing the SG for SP, the average unit weight for a glued-laminated member, corrected for 16 % moisture, can be determined by (0.55) × (62.4 pcf) × (1.16 %) = 39.8 pcf. Knowing the unit weight of a SP wood member the linear weight can be calculated by the equation (39.8 pcf) × (AREA/144"/').

Ex. 2×4 SP → (39.8 pcf) × ((1.5")(3.5") / 144"/') = 1.5 plf

Other long-term loads that are accounted for this project are 3 in decking (8 psf), lightweight concrete topping (3/4") (6.5 psf), joists (glued-laminated) (10 psf), mechanical allowances (4 psf), and partition allowances (20 psf).

5.2.2 Floor Short-Term Loads

The short-term loads for the building's floors are specified by building codes and vary according to building use. Short-term loads defined by code for this building are 100 psf for assembly spaces, 50 psf for office spaces, and 350 psf for archives and libraries. Member sizes are determined by deflection are usually limited by either short-term loads only or by short term plus long-term loads magnified by creep, which ever governs, in accordance with Table 5.2. For floor loads glued-laminated fiber stresses are recommended at either 2,200 or 2,400 psi.

Table 5.2 - Deflection Limitations for Uses Where Increased Floor Stiffness is Desired (See 5.7 for K)

Use Classification	Applied Loads Only	Applied Loads + K(Dead
Floor Beams		
Commercial, Office,		
Floor joist, spans to		
LL < 60 psf	l/480	l/360
60 psf < LL < 80 psf	l/480	l/360
LL > 80 psf	l/420	l/300
Girders, spans to 36 ft^2		
LL < 60 psf	l/480[c]	l/360
60 psf < LL < 80 psf	l/420[c]	l/300
LL < 80 psf	l/360[c]	l/240

[C] Based on reduction short-term load as permitted by code

A full specified short-term load is not normally present. It is even less frequent that large floor areas receive a full design short-term load simultaneously on each unit of area. Building codes consider this improbability by allowing the designer to use a reduced value for the short-term load in designing members that receive short-term loads from large areas. As follows:

1. The total reduction may not exceed 40 % for members receiving load from one level only, nor 60 % for other members.
2. The total reduction may not exceed R (percent), where

   $R = 23.1\ (1 + D/L)$
3. $R = r\ (A - 150)$

where r = reduction rate equal to 0.08 percent per square foot of tributary floor area over 150 ft^2

A = tributary floor area of member under consideration

D = floor long-term load (D)

L = tabulated floor short-term load (L)

5.2.3 Roof Short-Term Loads

Even in areas where snow is not expected, some short-term load must be considered in designing members that support a roof. This short-term load is based on the realization that the weights of construction supplies and personnel must be carried by the roof members. The snow load for the museum's location is less than the minimum roof short-term load therefore the design load for the roofs will be the construction live load.

Full specified short-term load is not normally present. It is even less frequent that large roof areas receive full design short-term load simultaneously on each unit of area. Building codes consider this improbability by allowing the designer to use a reduced value of short-term load in designing members that receive short-term load from large areas. Because the tributary areas for the roof truss are large it is assumed that the largest reduction possible is 60 %, therefore design loads for the roofs will be 12 psf.

5.3 Out-of-plane Loads

5.3.1 Wind

The exterior steel frame contains the lateral braced frame which resists both the longitudinal and transverse wind, see appendix 3. Since the exterior frame was not change the lateral system is left untouched no changes were need in building's lateral frame. However since the roof has been completely redesign, by replacing the open web joists with heavy timber trusses, wind loads had to be computed to determined lateral forces on the trusses. Wind loads for the roof were determined by ASCE7-98 section 6.5.12.4.2 Components and Cladding with Buildings > 60 ft. Design wind pressures on components and cladding for all buildings with $h > 60$ ft (18.3 m) were determined from the following equation:

$$p = q(GC_p) - q_i\,(GC_{pi})\ (lb/ft\ )\ (N/m\ ) \quad (ASCE\ 7\text{-}98\ Eq.\ 6.8)$$

The external pressure coefficient, GC_p, was determined from Figure 6-8. With a roof area greater than 1,000 ft^2, GC_p equal – 0.9. Because GC_p is negative, q will equal q_h (18.92 lb/ft^2) at the mean roof height (81 ft). The internal pressure coefficient, GC_{pi}, was determined from Table 6-7. For enclosed buildings GC_{pi} equal ± 0.18. Applying the known variables, the design wind pressure on the components and cladding, p = - 13.62, - 20.43 (lb/ft^2). Applying this pressure to each joint on the top chord of the truss with the designated tributary area determined the uplift force perpendicular to the roof.

5.3.2 Seismic

The approximately weight of wood and steel is 35 pcf and 490 pcf, respectively. By removing the interior steel frame in place of glued-laminates, the weight of the building decreases and which in turn decreases story shear. By decreasing story shear, the original seismic design are adequate for the modifications since the weight of the building has been decreased and can justified by the equation below.

$$Story\ Shear = W \times Cs \quad (ASCE\ 7\text{-}98\ \ Eq.\ 9.5.3.2\text{-}1)$$

5.4 Creep

Because wood will creep under long-term loading, one cannot simply calculate short-term- and long-term load deflection by mechanics formulas and expect the result to be the maximum over the life of the beam. To account for creep over the long-term, the immediate deflection due to long-term loads is magnified. Assuming that all short-term loads are short-term loads, the NDS recommends separating the effects of the short-term (short-term) loads and long-term (long-term) loads, calculating the total deflection over the long-term as follows:

$$\Delta_{Total} = \Delta_{Short\text{-}term} + (1+K)\ \Delta_{Long\text{-}term}$$

This equation assumes that the additional deflection due to creep will equal one-half the initial, immediate long-term-load deflection. The creep factor (K) is taken as 1.0 and 0.5 for unseasoned and seasoned wood, respectively.

5.5 Camber

Fabricated members, such as glue-laminates and wood trusses, are typically built with curvature into the member at the time of manufacture. This built in curvature is known as camber, and it opposes the deflection under gravity loads to provide a more pleasing visual condition. Camber for this report is determined the by the produce of the long-term deflection and the mutiple 1.5.

5.6 Fire Safing

Untreated wood is flammable, yet wood structures are not necessarily more easily destroyed by fire than buildings of other materials. The most important step in minimizing fire danger is to protect the building contents against becoming ignited. Beyond this, there are two main categories of protection for the building itself: (1) appropriate design details to prevent ignition and the spread of fire from one part of the building to another and (2) protection or treatment of the wood to make ignition harder and retard burning. Using formulas published by the Council of American Building Officials, the designer may calculate the required size of members necessary for fire resistance. The formulas give the fire resistance rating in minutes for timber beams and columns with smaller dimensions not less than 6 in. nominal. The formulas are:

	For beams-	For columns –
Exposed to fire on all four surfaces	$2.54\ Z_b\ (4 - 2b/d)$	$2.54\ Z_d\ (3 - d/b)$
Exposed to fire on only three surfaces	$2.54\ Z_b\ (4 - b/d)$	$2.54\ Z_d\ (3 - b/2d)$

b = the horizontal dimension of a beam cross section, or the larger side dimension of a column
d = the vertical depth of a beam, or the small side dimension of a column.
Z = a "load" factor that depends n the percent of allowable load actually carried by the member and read from a graph.

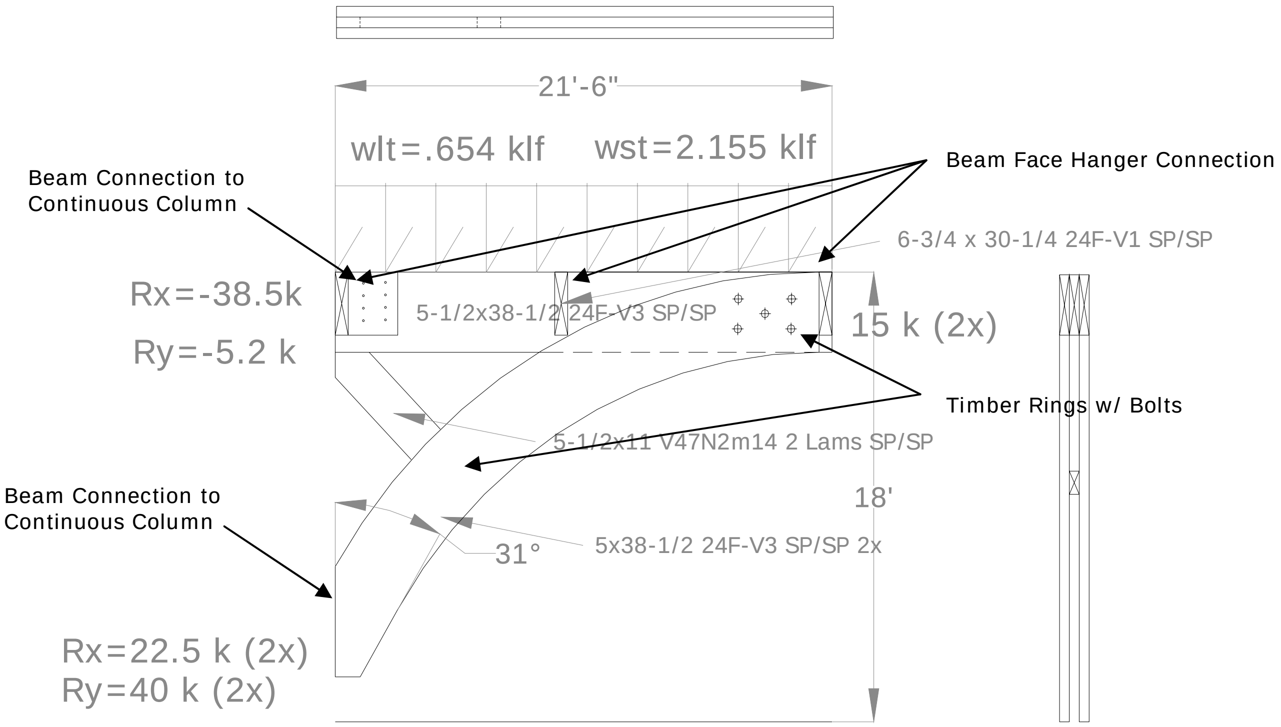
6.0 Structural Design

6.1 Manufacturer

Rigidply Rafters Inc, a manufacturer of glue-laminated timbers and a member of the American Institute of Timber Construction was chosen as the supplier for all of the glued-laminated timbers in this report. *Rigidply Rafters Inc*, is capable of producing glued-laminates up to 14-1/4" wide and supply Southern Pine visual graded 24F-V3 and 24F-V5 's. *Rigidply Rafters Inc*, is located near Lancaster, Pennsylvania, and was chosen for their close approximation for visitations. I was fortune to visit the company on two occasions for their help in compiling the data in this report. A manufacturer closer to Nashville, TN, would be the glued-laminated supplier for this museum.

6.2 Third Floor Cantilever Truss

Fig 6.1 Third Floor Canti-



The third floor has six cantilevered steel girders spaced at 30 ft 0 in with a span of 21 ft 6 in. To design a cantilever member using glued-laminate would not be a viable choice. The required short-term load moment connection for a cantilever necessitating a 100 psf short-term load would be approximately 137 foot-kips which is beyond the capacity of any wood connection and to control the deflection at the free end would require an enormous glued-laminated size beyond the specifications of the NDS as we will see in the fourth floor design.

Alternatives

After several variations, the best alternative design is to use the existing cantilevered member with two curved knee braces on both sides, Fig 6.1 (The methodology was mentioned in the proposal section of this report). The knee braces are curved to allow height clearance from below. This truss is constructed from four components: the cantilevered member (top chord), two curved knee braces, and a kick back brace to prevent the knee brace from "kicking" back. The W40×199 web width (bf) is 15.8 in. Keeping within that width, the total width of the truss must be less than the bf. Therefore the top chord glued-laminated member will be 5-1/8 in thick and the two curved glued-laminates will be 5 in thick, allowing approximately one half inch for connections.

Top Chord

The top chord spans 21 ft 6 in with three equally spaced point loads applied by the joists. The connections between the chord and curved knee braces are analyzed as a pin connection and located at the right end of the chord making the member a simple span beam for straightforwardness. The maximum shear is 30 kips, the maximum moment is 169 foot-kips, and the axial tension is 76 kips. With continuous lateral support on top side beam stability defaults to unity and the positive moment volume factor applies.

Angled Knee Brace (Curved)

Glued-laminates are produced by softening the wood to permit easy bending. The American Institute of Timber Construction, AITC, specification recommends curvature limits. For laminations of 2-in nominal thickness (1-1/2 in actual), the minimum radius of curvature (measured to the inner surface of the lamination) shall not be less than 18 ft 0 in for southern pine lumber. This is the minimum radii, and in no case should the ratio of lamination thickness to inside face radius of curvature, t/R_i, exceed 1/125 for southern pine.

Within the curved glued-laminated knee brace, there are negative moments, shear, and axial compression. The floor to ceiling height below is 18 ft 0 in and coincidently the minimum curvature for a southern pine glued-laminate is 18 ft 0 in. The radius that is used is 21 ft 6 in, the same length as the top chord.

Curvature limit

AITC's specifications limit the minimum radius of curvature, R_i, (measured to the inner surface of the lamination) for southern pine to be no less than 18 ft 0 in. In no case should the ratio of lamination thickness to inside face radius of curvature, t/R_i, exceed 1 /100 for southern pine. When the bending moment is in the direction tending to decrease curvature (increase the radius) the radial stress shall not exceed the allowable radial tension design value perpendicular to grain. If the radial stress is less than 15 psi, then tensile reinforcement is not needed.

When the bending moment is in the direction tending to increase curvature (decrease the radius) the radial stress shall not exceed the allowable compression design value perpendicular to grain.

Analysis

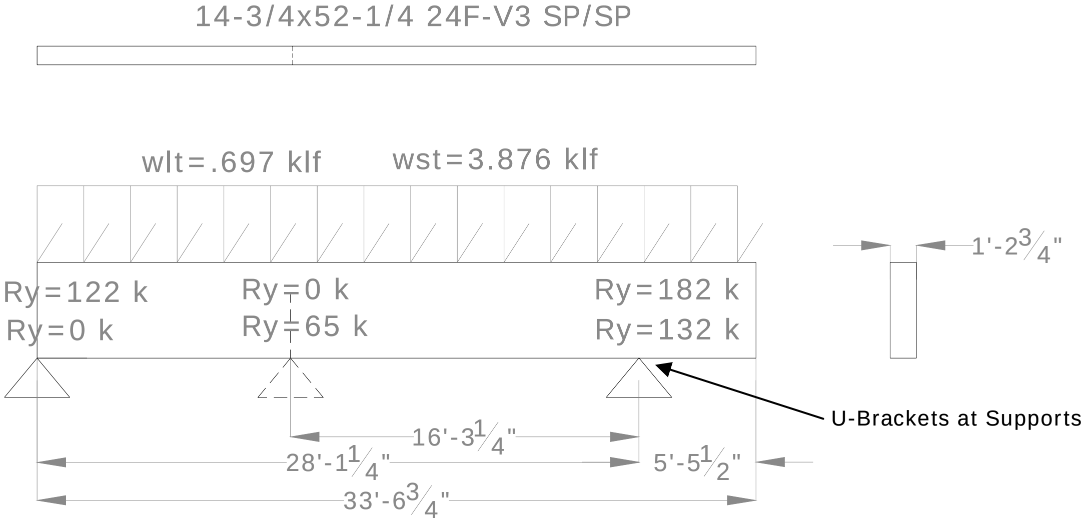
Because the design is hyper-static, the structural analysis was performed in *StaddPro 2002*. All material properties in the software were adjusted for southern lumber. The tributary area is 645 ft^{2}. Long-term loads of the members are calculated by the software while the long-term loads of the joists were added. Short-term load is 100 psf with a 28 % short-term load reduction based on the ratio of long-term and short-term load. The short-term and joists long-term loads were applied as point loads to the top chord at the connection points. Inputting the loads, the short-term plus creep deflections at the free end of the top chord is 0.792 in, approximately equal to l/D of 326. This deflection limit is slightly less than 360, however this is including creep and judged acceptable. Sizes and grades for the members were governed by deflection and are accordingly in table 6.1. Joists selection were also governed by deflection, and listed in table 6.1. Connections details are covered in the connection section.

Table 6.1 Design Results: Third Floor Cantilever Braced with Curved Knee Braces

Desig-naton	Size	$F_{bxt/t}$ (psi)	$F_{bxc/t}$ (psi)	F_v (psi)	F_c (psi)	F_t (psi)	$F_{c\perp(C)}$ (psi)	E_x (psi)	Camber (in)
Top Chord	5-1/8”×38-½” 24F-V3	2,400	1,950	270	1650	1150	740	1,800,000	
Curved Knee Brace	5”×38-½” 24F-V3	2,400	1,950	270	1650	1150	740	1,800,000	0
Kick Back	5-½”×11” V47N2M14	1400	1600	270	1150 (2 or 3 Lams)	1200	650	1,400,000	0
Joist	6-¾”×30-¼” 24F-V3	2,400	1,950	270	1650	1150	740	1,800,000	0.25

6.2 Third Floor Archive/Library Girders

Fig 6.2 Third Floor Archive/Library Girder



The third floor has six girders spaced at 30 ft 0 in ranging in length from 16 ft to 26 ft with a constant 5 ft 0 in overhang. The structural impediment is the large short term loading required. The girders support the museum's archive and library, requiring a 350 psf short-term load. There were two alternative designs for this particular member. Alternative one was to reduce the spacing to 15 ft. This alternative would require a 12-3/4 × 44 size. Alternative two was to maintain the existing 30 ft spacing and use a larger size, a 14-¼ × 52-¼. Alternative one is 1.5 in narrower and 8.25 in shallower. With a floor to ceiling height of 18 ft 0 in there is adequate clearance for deep members. Alternative one would use twice the members, 12, with a total wood area of 6,732 in^2. Alternative two would use half the number with a total wood area of 4,467 in^2, 33 % less material, so alternative two is judged the best alternative in material savings. In addition, by maintaining the spacing of the girders a 30 ft 0 in, this reduces either the number columns or a transfer girder.

In order to maintain the 30 ft spacing, the width of the glued-laminates exceeded the maximum NDS width of 10.5 in. Size widths are governed by the machinery, particularly the planar. A nearby glued-laminate manufacture, *Rigidply Rafters Inc.*, is cable of producing maximum widths of 14-¼ consisting of a 2"×8" and 2"×6" butt jointed with randomly staggering the joint.

Analysis

The analysis was perform with a mechanics formula. The tributary area is 1,160 ft^2. Long-term loads included the member itself and the joists. Short-term load are 350 psf with a 26 % short-term load reduction based on the ratio of the long-term and short-term loads. The short-term and joists long-term loads were applied as uniformly distributed to the top chord due to the multiple closed spaced joists. Applying the loads, the greatest deflection limit is the short-term load on the overhang with a deflection of 0.18 in, approximately equal to l/Δ of 362 with a limit of 360. Other deflection controls are listed in the table 6.2. Sizes and grade for the member is governed by deflection and is accordingly. Joists selection was also governed by deflection. Connections details are covered in the connection section.

Fire Rating: > 60 minutes

Table 6.2: Design Results: Third Floor Archive/Library Girder

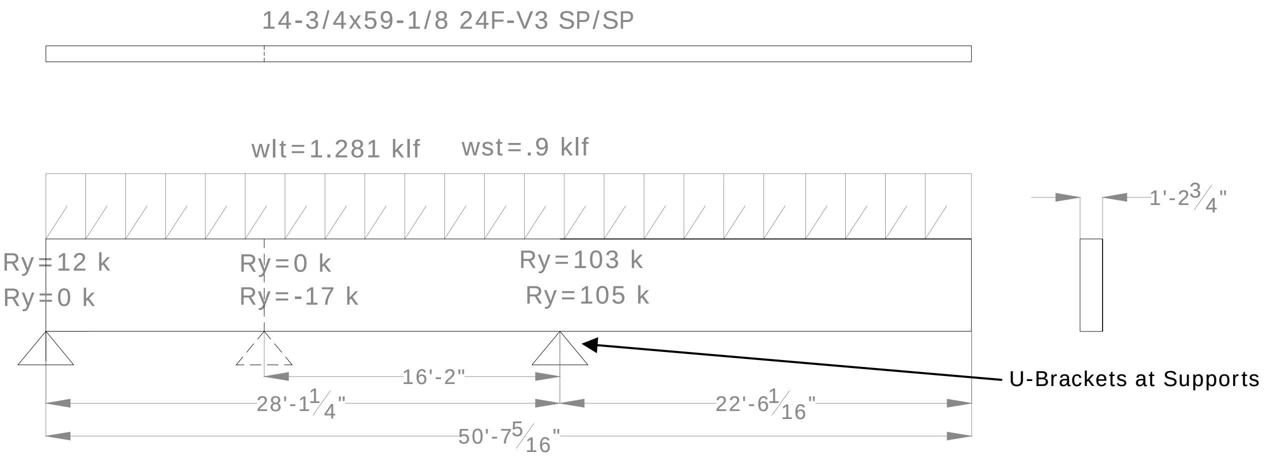
Desig-naton	Size	$F_{bxt/t}$ (psi)	$F_{bxc/t}$ (psi)	F_v (psi)	F_c (psi)	F_t (psi)	$F_{c\perp(C)}$ (psi)	E_x (psi)	Cam-ber (in)
Girders	14-1/4”×52-1/4”	2,400	1,950	270	1650	1150	740	1,800,000	1/16
Joists	6-¾”×30-¼”	2,400	1,950	270	1650	1150	740	1,800,000	1/8

Table 6.3: Deflection Results: Third Floor Archive/Library Girder

Beam			
Δ_{ST}	0.32	$\Delta_{KCR\text{-}LT}+\Delta_{ST}$	0.40
L/Δ_{ST}	1,051 > 360	$L/[(\Delta_{KCR\text{-}LT}+\Delta_{ST}]$	835 > 240
Overhang			
Δ_{ST}	-0.18	$\Delta_{KCR\text{-}LT}+\Delta_{ST}$	-0.23
L/Δ_{ST}	-362 > 360	$L/[(\Delta_{KCR\text{-}LT}+\Delta_{ST}]$	-287 > 240

6.3 Fourth Floor Girders

Fig 6.3 Fourth Floor Girders



The fourth floor has six girders spaced at 30 ft 0 in ranging in length from 16 ft to 26 ft with a constant 22 ft 6 in overhang. The structural impediment is the long overhang seen Fig 6.3. These girders resist the loads from the museum's executive offices and necessitate by code a 50 psf short-term load. There were six alternatives to increase the stiffness of the overhang and control deflection. The first two alternatives are the same for third floor girders in section 6.2. The third alternative is to use knee braces, either straight or curved, to reduce the overhang length. Because of obstacles below, the glass wall and architectural displays, the braces would interfere with the architecture. Alternative four was to experiment with fiber reinforce plastic glued-laminates. As mentioned earlier in the report, FiRPs do not increase stiffness significantly and reasoned unsuitable. Alternative five is to support the overhang with a tension rod to the roof. This would relieve the stress in the member however this does not erase the stresses. The stresses would instead transfer into the roof structure thus increasing their size. Since the girders will be in sight, and appearance is the motive, alternative two is chosen to keep the members spaced at 30 ft and use large size glued-laminates. Thus the required size is 14-¼ x 59-1/8. In addition, by spacing the girders a 30 ft 0 in, this reduces either the number of additional columns or a transfer girder. The sixth alternative, a flitch girder was considered but was overruled in keeping the honesty in the structure.

In order to keep the 30 ft spacing and have stiffness for the overhang, the width of the glued-laminate had to exceed the NDS standard widths of 10.5 in and be sized to 14-3/4 in width. As mentioned previously, size widths are governed by the machinery, particularly the planar. *Rigidply Rafters Inc.* is able to produce a maximum width of 14-¼ by joining a 2"×8" and 2"×6" butt jointed and randomly staggering the joint.

Analysis

The analysis was perform with a mechanics formula. The tributary area is 1,500 ft^2. Long-term loads included the member itself and the joists. Short-term load are 50 psf with a 40 % short-term load reduction based on the maximum allowed. The short-term and joists long-term loads were applied as uniformly to the top chord because of the multiple closed spaced joists. Applying the loads, the greatest deflection limit is the short-term load and creep on the overhang with a deflection of 0.74 in., approximately equal to l/Δ of 363 with a limit of 360. Other deflection controls are listed in the table 6.4. Sizes and grade for the member is governed by deflection and is accordingly. Joists selection was also governed by deflection. Connections details are covered in the connection section.

Table 6.4 Design Reults: Fourth Floor Girder w/ Overhang

Desig-naton	Size	$F_{bxt/t}$ (psi)	$F_{bxc/t}$ (psi)	F_v (psi)	F_c (psi)	F_t (psi)	$F_{c\perp(C)}$ (psi)	E_x (psi)	Camber (in)
Girders	14-½ × 59-1/8	2,400	1,950	270	1650	1150	740	1,800,000	1/16
Joists	6-¾”×27-½”	2,400	1,950	270	1650	1150	740	1,800,000	1/8

Table 6.5 Deflection Results: Fourth Floor Girder w/ Overhang

Beam			
Δ_{ST}	0.0	$\Delta_{KCR\text{-}LT}+\Delta_{ST}$	-0.02
L/Δ_{ST}	-68,602	$L/[(\Delta_{KCR\text{-}LT}+\Delta_{ST}]$	-21,003 > 360
Overhang			
Δ_{ST}	0.23	$\Delta_{KCR\text{-}LT}+\Delta_{ST}$	0.74
L/Δ_{ST}	1187 > 480	$L/[(\Delta_{KCR\text{-}LT}+\Delta_{ST}]$	363 > 360

6.4 Roof Trusses

There are six W40×199 columns at the rear of the building. To employ their full capacity they will be used to support the glued-laminated roof trusses which will frame the 2-1/2 : 12 mono-sloped roof (*Fig 6.4*). The two illustrated trusses are located on column lines J8 & J, respectively, in respect to the structural plans in Appendix A. Truss J is truncated due to the middle rear of the building having an inset. The six W40×199 columns are spaced at 30 ft thus spacing the truss at 30 ft. There will be nine truss, seven which will be truncated. The truss members sizes for J & J8 are 24F-V3 southern pine 8-1/2" × 23-3/8" and 10-1/2" × 23-3/8" respectively. The truss design is a Warren truss with the diagonal webs positioned in a defined way to carry compression only. The archetype designs had webs compressed and tensed. The tensed webs were tensed upwards to 400 k. A typical shear plate can resist approximately 5 kips, thus 80 shear plates would be needed which is implausible. With all the webs compressed, as illustrated in *Fig 6.4*, the greatest compressed force is 300 k which is within the bearing capacity of the webs at 500 k. Perpendicular compression for the bottom and top chord by the webs is another concern. The webs connect into the bottom and top chords, bearing at an angle to grain. The bearing stress is somewhere between parallel and perpendicular compression and is determined by *Hankinson Formula*. The bearing angle capacity is greater than the compressive forces that are exerted by the webs to the top and bottom cords and ruled satisfactory.

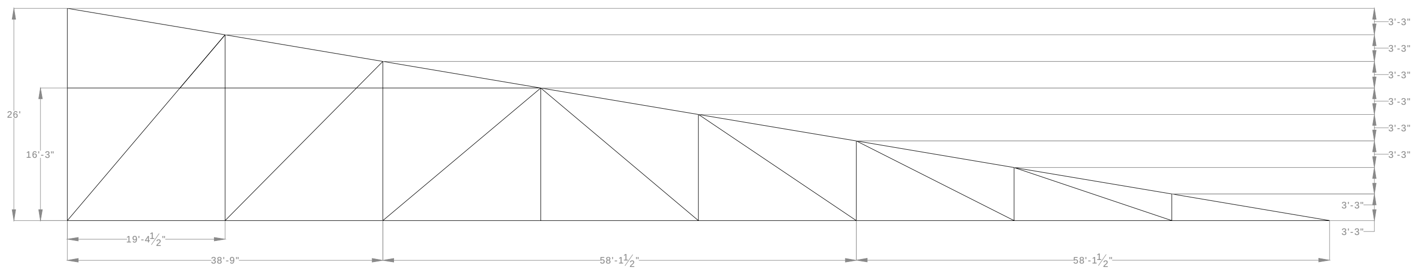
Transporting this trusses to site is another issue. The truss are to large to truck on to site and would very time consuming to assemble on site. Figure 6.5 illustrates how the truss will be shipped to minimize costs. Length wise, the top and bottom chords would be built with continuous members, up to lengths of 60 ft, allowing up to three bays to be fully assembled. Height wise, the truss is segment in the middle. The top pieces are known as cap trusses. A typical tractor trailer is 53 ft long and special arrangements and designated routes can be devised to bring these trusses on site by truck.

The purlins will be spaced eight feet on center and specified as 24F-V3 6-3/4x27-1/2 SP/SP.

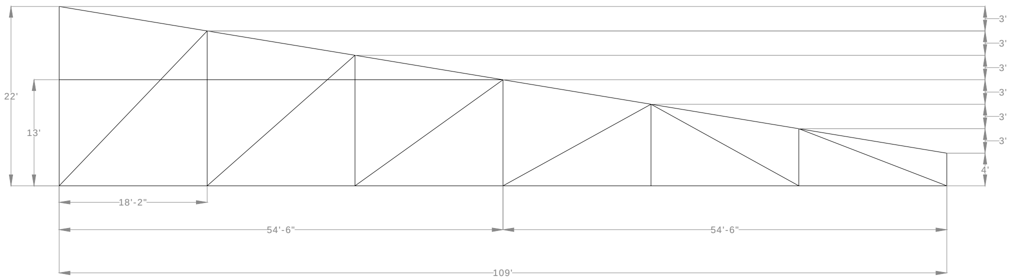
6.5 F.I.R.P.

Fiberglass Reinforced Plastic (FIRP) glued-laminates are manufactured with one or more thin layers of a fiber reinforced panel. The reinforcing consists of high-strength fibers embedded in a matrix and are strategitically placed between certain laminaotions to increase beam strength and stiffness. Three types of reinforcing are currently approved for use in FIRP beams: Aramid, Carbon, and Fiberglass. After running through several calculations the reinforced plastics increase moment

Fig 6.4 Mono-Sloped Trusses J-8 & J



Truss Located at Column Line B & J.8 C



Truss Located at Column Line D, E, F, G, J, & H

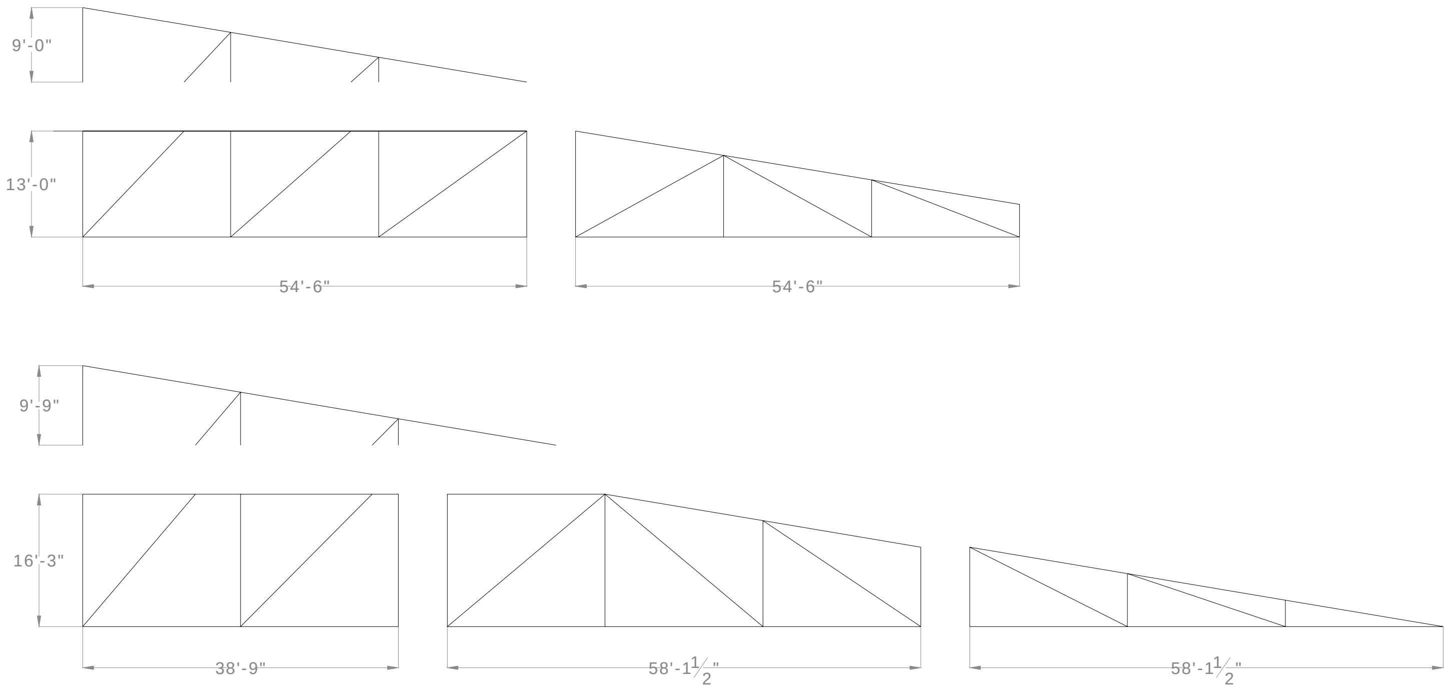


Fig 6.5 Mono-Sloped Trusses J-8 & J—Segmented for Shipment

capacity but do not increase stiffness significantly. All of the designs are controlled by deflection and FIRPS increase stiffness by only 10 %.

6.6 Columns

The columns that will support the glued-laminate floor can be either steel columns enfolded in wood veneer or glued-laminated columns. Steel columns are favored for their high strength capacity but in keeping with the honesty of the structure, glue laminated columns will be used on the second and third floor (there are no columns on the fourth floor). To verify if a glued-laminated column is adequate to carry the heaviest axial loads from a girder reaction, the largest vertical load is located underneath the third floor archive/library supporting the third and fourth floor. The size required is a SP V49 N1M16 4 LAMS 12-3/4" x 12-3/4" (V is visually graded; 49 is the combination number, N1 is number one grade lumber, M is medium density grain, and 16 is the slope of grain, in which it can not exceed 1 : 16). This size can be assembled using a 2x8 and 2x6 edged butted (The depth of a 2x12 is only 11-1/4). The prominent disparity between glued-laminated columns and beams is the edge gluing required for the columns. Edge gluing the columns controls lateral buckling. Knowing a 12-3/4 x 12-3/4 is capable of supporting the 3rd floor archive/library and fourth floor office spaces, all other columns will be ample for glued-laminates.

6.7 Hall of Fame Roof

Part of the proposal was to redesign the Hall of Fame's stair-stepped roof with either exposed sawn lumber or glued-laminated timbers. At the time of the proposal it was unaware that the entire roof structure was a rigid frame with a compression ring at the top most roof. Knee braces or a complete redesign with joints would be the only solution, and with the additional weight of an ornamental radio tower at the center, the frame has been decided to leave as is.

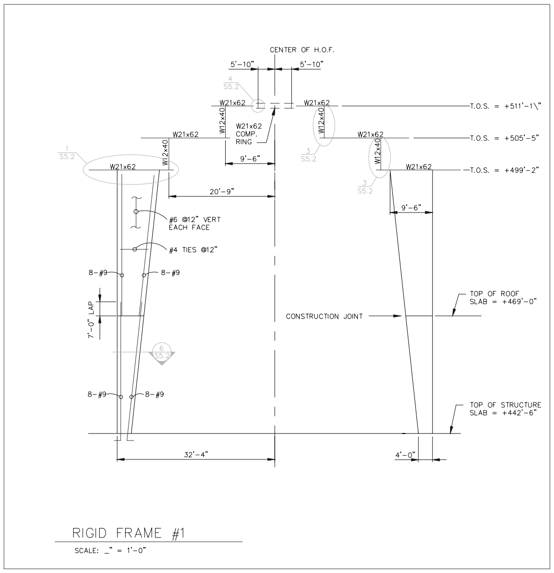


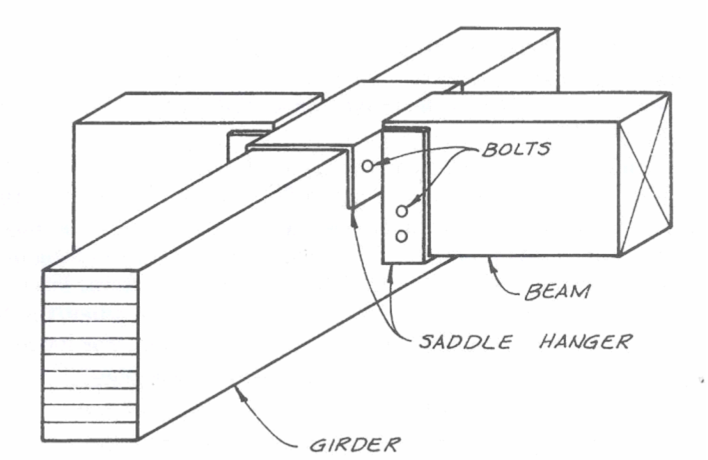
Fig 6.6 Hall of Fame Stair Steep Roof

7.0 Timber Connections

All of the connections for the glued-laminate frame will be edge bearing connections. Regardless of the bearing load or the perpendicular compressive stress, any connection can be achieved by the size of the seat area of each connector.

7.1 Beam Saddle

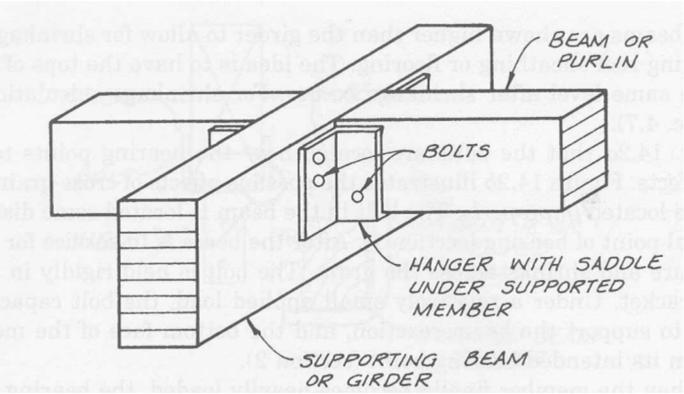
Fig 7.1 Beam Saddle Detail



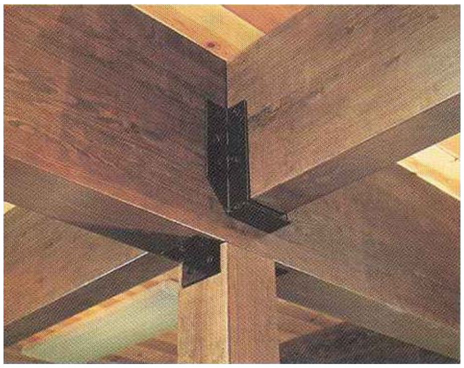
Beam saddle connections are the preferred means of transferring beam reactions to girders. The beam reaction is transferred by bearing perpendicular to the grain on the bottom of the hanger. The load on the hanger is then transferred by bearing perpendicular to the grain through the top of the saddle to the girder. This type of connection is recommended for larger loads.

7.2 Beam Face Hanger Connection

Fig 7.2 Beam Face Hanger Detail



When the reaction of the beam or purlin is relatively small, a cheaper alternative to the beam saddle is a hanger can be bolted to the face of the girder. The bolts in the main supporting beam or girder should be placed in the upper half of the member, but not too close to the top of the beam where extreme fiber-bending stresses are maximized.



7.3 U-brackets

U-brackets are fabricated from bent plates when the bracket does not cantilever a long distance beyond the width of the column. For longer U-brackets, the vertical plates will probably be welded to a thicker base plate. It is important that the wood member be fully seated on the bearing surface before the bolts are installed. Without such precaution the squared corners of the beam could initially rest on the inside radius of a bent plate, and the member may not fully seat until it is loaded heavily in service.

7.4 Beam Connection to Continuous Column

When the design is such that the bending member does not rest on the top of a column, wall, or pilaster, but frames into another member, several methods can be used to support the ends. The preferred method is to transfer the end reaction by bearing perpendicular to grain.

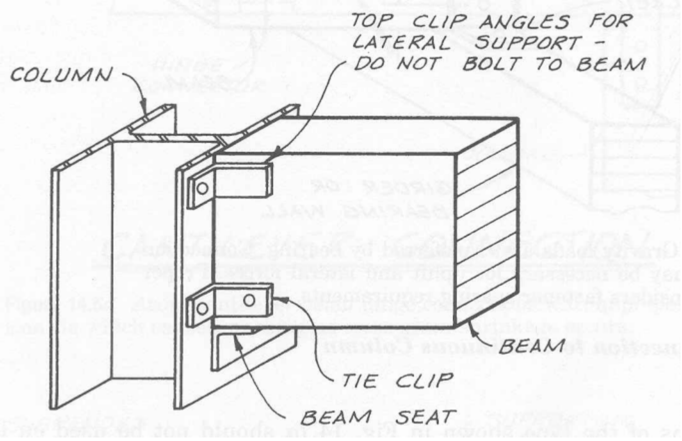


Fig 7.3 Beam Connection to Continuous Column Detail

1. Vertical beam reaction is transferred by bearing perpendicular to grain on the beam seat angle.
2. Positive connection is made between the beam and the column with fasteners in the tie clip.
3. Clip angle at top provides lateral stability to beam, but it not attached to the top of the beam with fasteners. In other words, the clip angle prevents rotation perpendicular to the plane of the beam, and it thereby braces the member against lateral torsional buckling. This provides a stable member that will not split the end.

7.5 Timber Ring & Shear Plate

Timber rings and shear plates are connectors that are installed in precut grooves in wood members. The connectors provide a large bearing surface to resist shearing-type forces in a wood connection. A bolt or lag bolt is required through the center of the timber ring or shear plate to hold the assembly together.

Timber rings are only for wood-to-wood connections because the steel ring fits into a groove cut into the mating surfaces of the members being connected.

Shear plates can be used for wood-to-metal connections because the shear plate is flush with the surface of the wood. Shear plates may also be used for wood-to-wood connections, but a shear plate is required in each wood member.

The allowable design values for timber rings and shear plates are higher than those for bolts or lag bolts. However, timber ring and shear plate connectors require special fabrication equipment, and their use is limited by fabrication costs. Timber rings and shear plate connectors are more widely used in glued-laminated arches and heavy timber trusses.

Split rings are available in 2-1/2- and 4-in diameters, and shear plates are available in 2-5/8- and 4-in diameters. Both split rings and shear plates were used in the connection details of this building. Shear rings are used with the connection hardware mentioned previous in this chapter. In the illustration below, eight shear plates are used in conjunction with the connection hardware to fasten the top chord to the continuous beam and 10 timber rings (5 on each side) are used to fasten the third floor cantilever 3-ply truss.

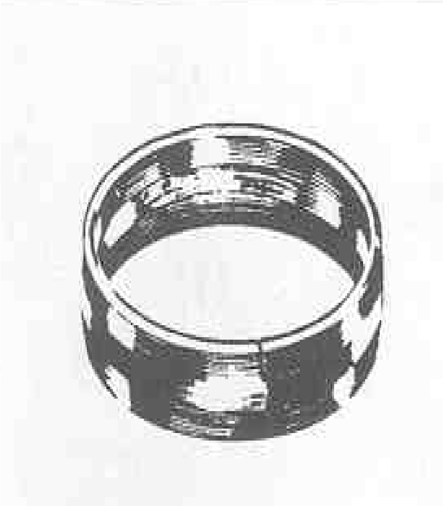


Fig 7.4 Timber Ring (NDS)

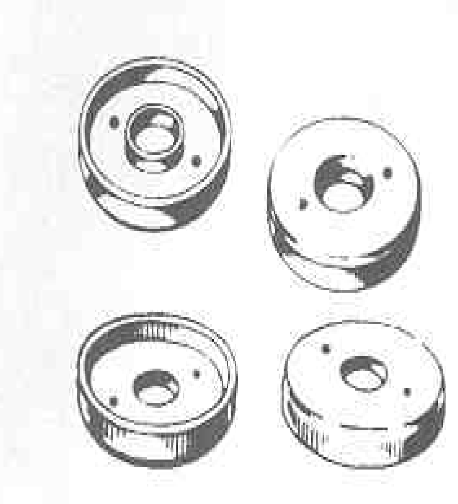


Fig 7.5 Shear Plate (NDS)

7.6 Double Gusset Plates

Members of heavy timber trusses are usually connected by bolts passing through the members and two steel gusset plates, on each face of the members. When the bearing strength of the bolt on the wood is not adequate, bolts may be supplemented by shear plates embedded in each face of the timber members. Advantages of the truss with double plane gusset plates are:

1. Design flexibility
2. Less caper entry skill is needed, members do not have to be notched or precise angles cut.
3. Double-gusseted trusses can be of very attractive appearance.

8.0 Cost Analysis & Comparison

The CM breadth will be a cost comparison between the existing structure and the new proposed structure. The cost of the redesigning part of the museum with glued-laminates will greatly increase the costs with materials and having a second erecting team.

8.1 Steel Deduction

Steel cost was performed using *RS Means 2004*, and deducted by linear footage of steel. Appendix C outlines the individual costs of each steel wide flange shape. Table 8.1 summarizes the cost for each floor.

Table 8.1 Steel Deduction Costs

Description	Cost ($)
Third Floor	153,000
Fourth Floor	137,000
Open Web Joists	56,500
Total	**$350,000**

8.2 Glued-Laminated Addition

The following costs were compiled based on cost data provide by *Rigidply Rafters*. Table 8.2 lists unit cost for each description. *RS Means 2004* prices glued-laminates using typical members and framing methods. The cost data is only an approximation based on an partial structural design and time constraints. These unit values are typical values used in project pricings for *Rigidply Rafters*.

Table 8.2 Glue-laminated timber Cost Addition

Description	Unit Cost ($/SF)	Quantity	Cost ($)
Assembly/Office	$21/SF	45,000 SF	945,000
Archive/Library	$12/SF	5,000 SF	60,000
Trusses	$150/LF	800 LF	120,000
Column	$30/LF	810 LF	25,000
Edge Gluing	+20% (Column)		5,000
Material Cost	+15% (Total)		175,000
Discount	20%		25,000
TOTAL			**$1,300,000**

9.0 Day Lighting Cost/Benefit Analysis

The lighting breadth will be the addition of sky lights on the museum's roof to provided natural light to the fourth floor office spaces.

The use of skylights has grown in recent years, both because they enliven building interiors and because they can save energy dollars through daylighting. Sky lighting can be a solid asset for buildings, and satisfying for building designers, occupants, and owners. Skylights can make a number of major contributions to the built environment since they:

- Provide excellent lighting conditions to the interior of buildings
- Reduce the use of electric lighting, to save energy and reduce peak electric loads when combined with photo controls.
- Satisfy human needs for contact with the outdoors
- Increase safety and security with highly reliable daytime lighting.
- Provide emergency smoke vents.

9.1 Glazing Types

Table 9.1 illustrates the different types of glazing that can be chosen for the skylights (Highlighted Selected)

Type	Layers	Color	VT	SHGC	LSG	U (Flush/Site) Metal w/ Thermal Break
Glass	Single-Glazed	Clear	0.89	0.82	1.09	1.250
		Bronze	0.55	0.64	0.87	
		Green	0.74	0.59	1.25	
	Double-Glazed	Clear	0.78	0.70	1.11	0.650
		Bronze	0.48	0.51	0.94	
		Green	0.66	0.47	1.40	
	Double-Glazed Low-E	Clear	0.72	0.57	1.25	0.580
		Bronze	0.45	0.39	1.15	
		Green	0.61	0.39	1.56	
	Triple-Glazed Low-E	Clear	0.70	0.53	1.32	0.410
		Bronze	0.42	0.37	1.14	
		Green	0.61	0.38	1.61	

Double-glazed low E glass was chosen for our sky lights and based on the following reasons.

It's high visible transmittance (VT) allows the use of smaller skylights, thus a smaller aperture and the fewer heating and cooling losses.

It's low solar heat gain coefficient (SHGC), or the shading coefficient (SC). The lower the SHGC the better the skylight glazing material will be at preventing unnecessary heat gain from the skylights. Also the hotter the climate, this being Nashville, the more important the solar heat gain coefficient (SHGC) becomes.

It's LSG ratio, a high ratio will increase lighting savings while reducing cooling losses.

It's U-value wasn't a dominate concern, Nashville doesn't experience enough cold climate days, but a yet, low-E was chosen.

From the selections of different types of glazing, the only superior glass was triple glazed and was not chosen because of cost in is not used commonly in the United States.

Other advantages were the high ceiling heights (30 ft). High ceiling heights allow larger and/or father apart skylights which can achieve uniform illumination levels, as well as a less concerns for visual quality. The general rule of thumb is to space skylights at 1.0 to 1.5 times ceiling height (center-to-center in both directions). This rule of thumb works out well. The ceiling height is 30 ft, the roof truss are spaced 30', so a sky light can installed mid-way between each truss, 30 ft center to center.

9.2 Savings

The Lawrence Berkeley National Library provides design aides in calculating the annual day lighting energy savings ($) with the use of nomographs. Nomographs are a quick and simplified tool for cost/benefit analysis, they are a graphic way to present a formula that has several variables. Rather than doing the calculation mathematically, a nomograph user can "walk" through diagrams. Nomographs will not deliver a guaranteed answer about cost-effectiveness, but with good design details, energy costs, and owner's investment criteria, good data results can be used to discuss with the building's owner. The nomograph worksheet can found in appendix B. Total cost savings per year is approximated to $3,600 per year in just lighting.

9.3 Glazing Area

The average useful window ratio is 50 %. The glazing area is the product of the useful window ratio and the perimeter area of the room. The fourth floor perimeter area is roughly 15,000 SF. Applying the general rule, glazing area is approximated at 7.500 SF.

10.0 Thermal Ice Storage Units

10.1 Recommendation

To reduce the museum's on-peak electricity demand and cost, a thermal ice system is being proposed to help reduce energy costs by allowing the energy-intensive electrically driven cooling equipment to operate during off-peak utility hours when electricity rates are at their lowest. Ice storage generates ice at night during off-peak utility hours and stores it to cool a building the next day during utility's on-peak demand time. The California Energy Commission estimates an average of 12 % fewer kWh are used with ice thermal storage. With a low cost of operation and the potential for the lowest first cost, ice storage offers an energy-saving technology to accommodate new changes and trends in the electric power industry.

10.2 About the Technology

There are many different types of thermal storage systems representing different combinations of storage media, charging mechanisms and discharging mechanisms. The two basic operating strategies are full storage and partial storage. Full storage systems build enough ice during the night to serve the entire on-peak cooling requirement. This strategy, which is demand- or usage-charge driven, shifts the largest amount of electrical demand and results in low operating costs. However, due to larger storage requirements, full storage systems have a higher upfront cost. The ice storage systems can be broken down into four different types of ice-making: ice harvesting, ice-on-coil, ice slurry, and encapsulated ice options. Ice harvesting systems form ice on coils and periodically release the ice into a storage tank that contains a mixture of ice and water. Ice-on-coil forms ice on heat transfer surface, generically referred to as a "coil". There are several variations of this system. Ice slurry systems produce small particles of ice within a solution of glycol and water, resulting in a slushy mixture than can be pumped or dropped into a storage tank. The encapsulated ice option consist of water contained in plastic containers surrounded by coolant, all contained within a tank or other storage vessel. Subfreezing coolant from a chiller is circulated through the storage tank and pas the plastic containers, freezing the ice.

10.3 Materials

Ice thermal storage systems will decrease supply water temperature and use smaller components than traditional cooling systems. The size of the chillers and cooling towers required for an ice system is significantly reduced compared to conventional chillers and cooling towers. Pump and pipe sizes are also reduced in an ice storage system as well as condenser water pipe sizes and head pressure falls. The only physical constraint for ice storage is physical space in the building. First cost comparison is listed below in Table 10.1

Table 10.1 First Cost Comparison for Ice Thermal Storage Systems

First Cost Comparison	
Ice Thermal Storage Savings by Com-	
Chillers/Cooling Towers	$73,500
Distribution Piping	$60,000
Electrical	$28,000
Condenser Piping	$8,000
Pumps	$3,000
Additional Ice Thermal Storage Components	
Ice Thermal Storage Units	$105,500
Heat Exchanger	$20,000
Concrete Slab	$7,200
Ethylene Glycol	$4,900
Total	$137,600
Net First Cost Savings:	$34,900

Sample First Cost Comparison for a 400 ton system

10.4 Case Study

Thermal storage has been lately receiving a lot of attention in the Federal sector. With the Federal sector representing nearly 4 % of commercial building floor space and 5 % of commercial building energy use, the estimate potential annual savings for using thermal storage in the Federal sector is $50 million. An internal melt ice on coil energy storage system was installed in a federal office building in Pittsburgh, PA. The William S. Moorhead Federal Building is a 23-story, 788,000 square foot structure constructed in 1963. Installation of the cool storage system reduced the size and cost of the new chillers and also resulted in reduced demand charges by minimizing chiller operation during peak demand period. During a typical summer weekday, the ice storage system is charged from 6:00 p.m. until 6:00 a.m. the following morning. Storage is discharged from noon until 4:00 p.m. The chillers are operated as needed during the other hours of the day to directly meet the building cooling load. On relatively mild summer days and when cooling during late spring and early fall months, the chillers don't need to be run during the peak demand period at all. The lower chilled water deshort-termry temperature possible with an ice storage system also makes it possible to consider the benefits of cold air distribution in an anticipated future replacement of the building's airside systems. In the ice making role, the chillers are de-rated from 600 to 461 tons. Two charts below illustrate storage system operation – design day and energy reduction versus historical for the Moorhead Building.

11.0 Architectural

Tuck Hinton Architects designed a form for the main museum that would be inspired by the country stores that feature large façades and signage but are really intimate spaces where people come to socialize and exchange information. The depth work for this report intends to further strengthen and embrace the museum's interior country general store appearance by replacing the exposed steel structural with natural stained Southern Pine glued-laminated timbers. Glued-laminated timbers, known as glulam for short, is sought after to bring out the warmth and the strength of the structure as well as it's advantages in it's strength to weight, durability, fire resistance, and chemical resistance. In typical cases glued-laminated timbers can be 20 % lighter than structural steel and 600 % than a reinforced concrete member. In addition the higher thermal insulation characteristics of timber and the charcoal layer that forms on it ensure that the interior of a fire exposed member remains cool and structurally sound over the design period. In addition glued-laminated behaves as a single piece throughout its exposure to fire because of the high resistance of laminating adhesives to fire temperatures.

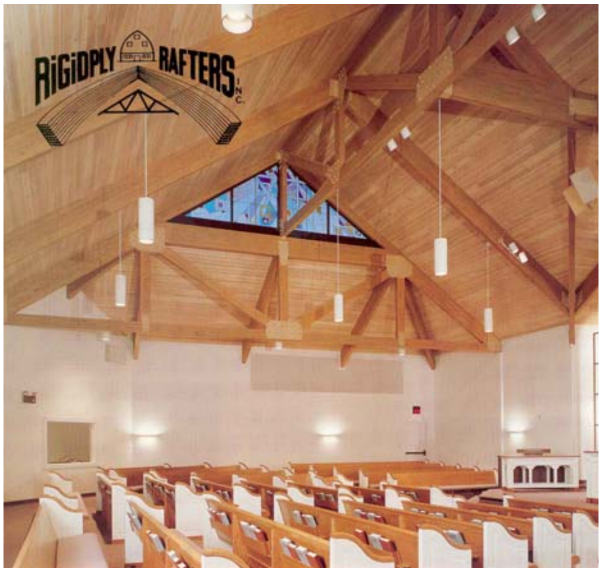


FIG 11.1 The Use of Heavy Timber Trusses (Rigidply Rafters)

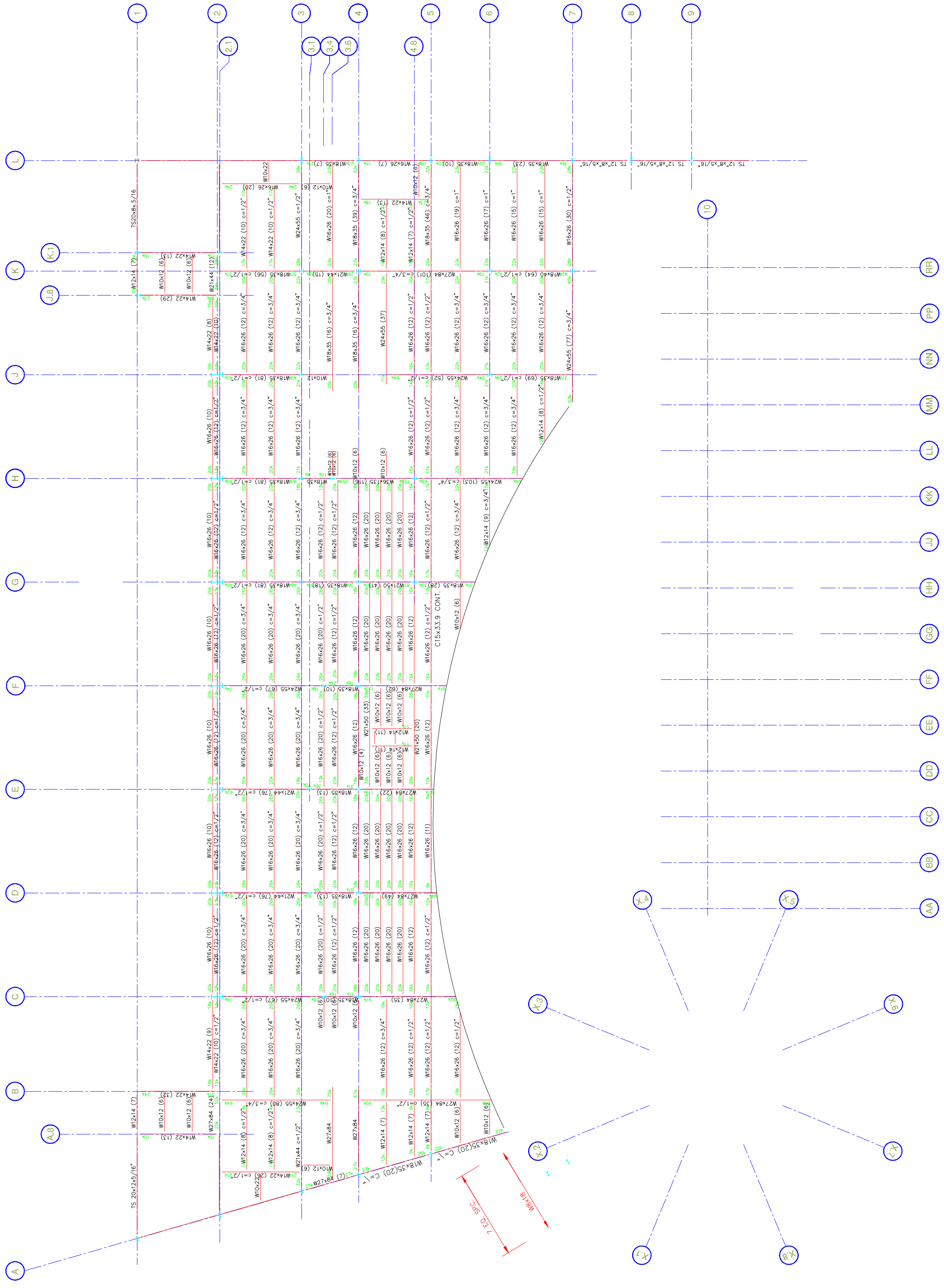
Fig 11.2 The Use of Glued-Laminated Beams (Rigidply Rafters)

APPENDICES

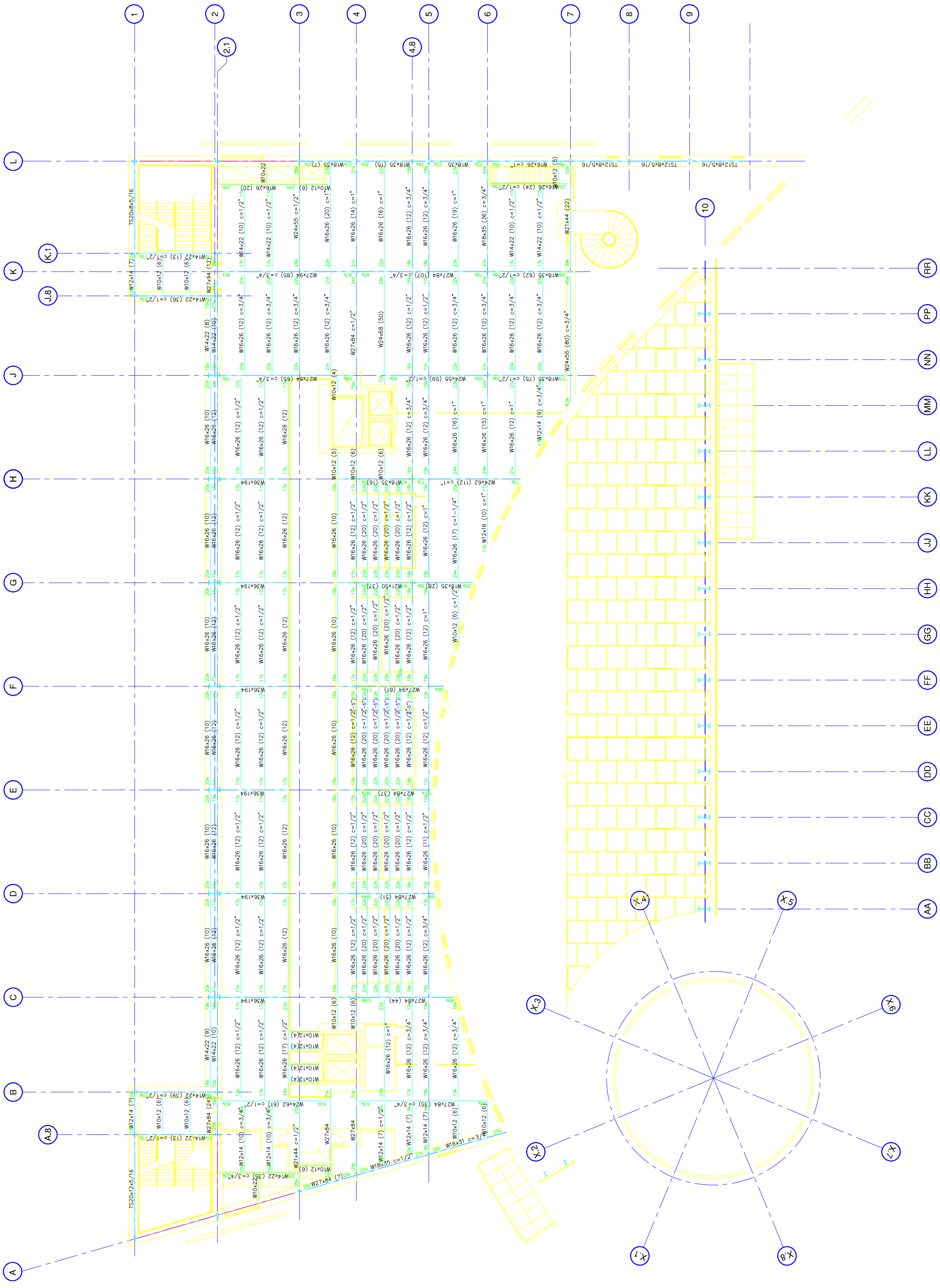
APPENDIX A

Existing Structural Floor Plans

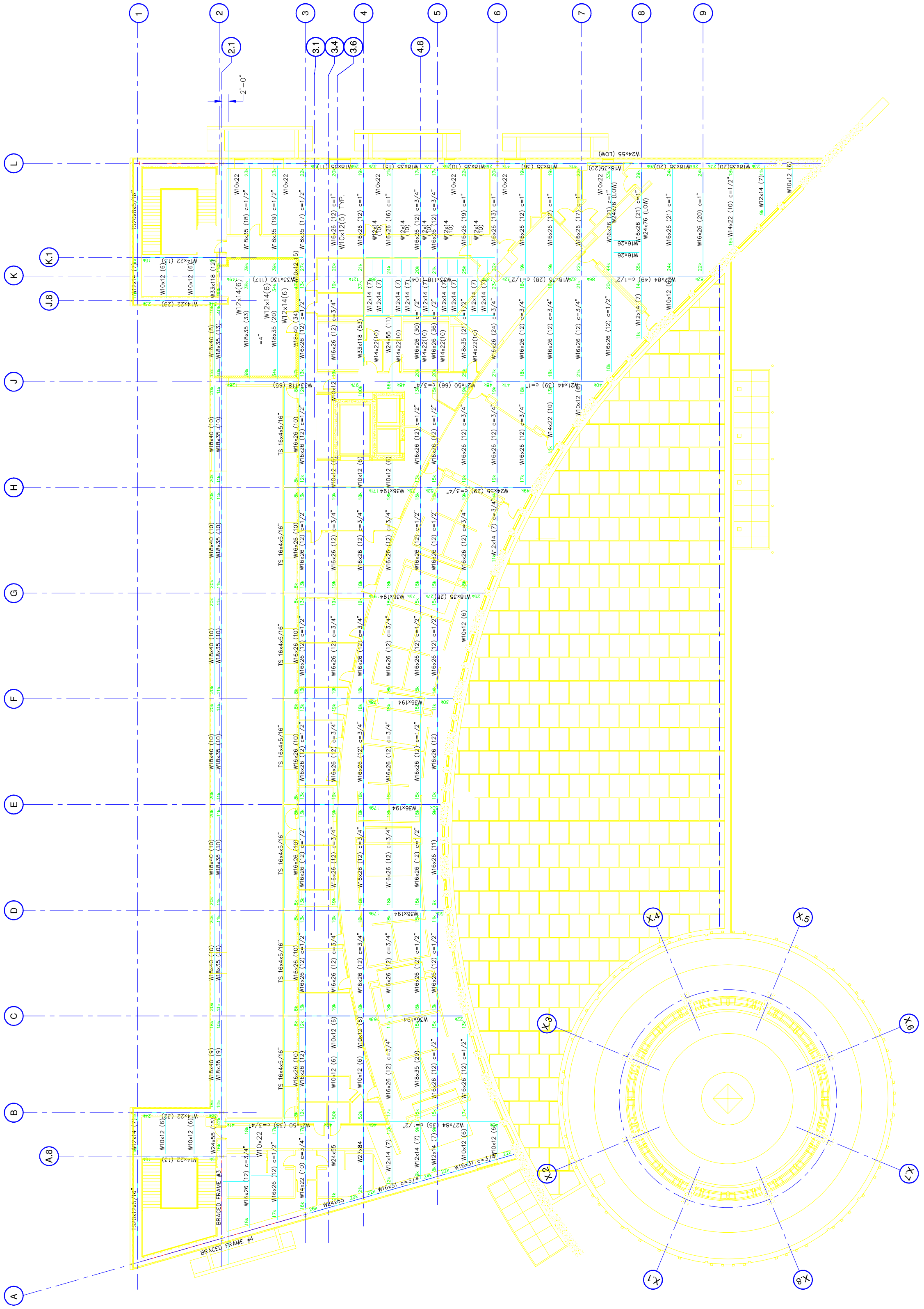
A.2 Second Floor Structural Plans



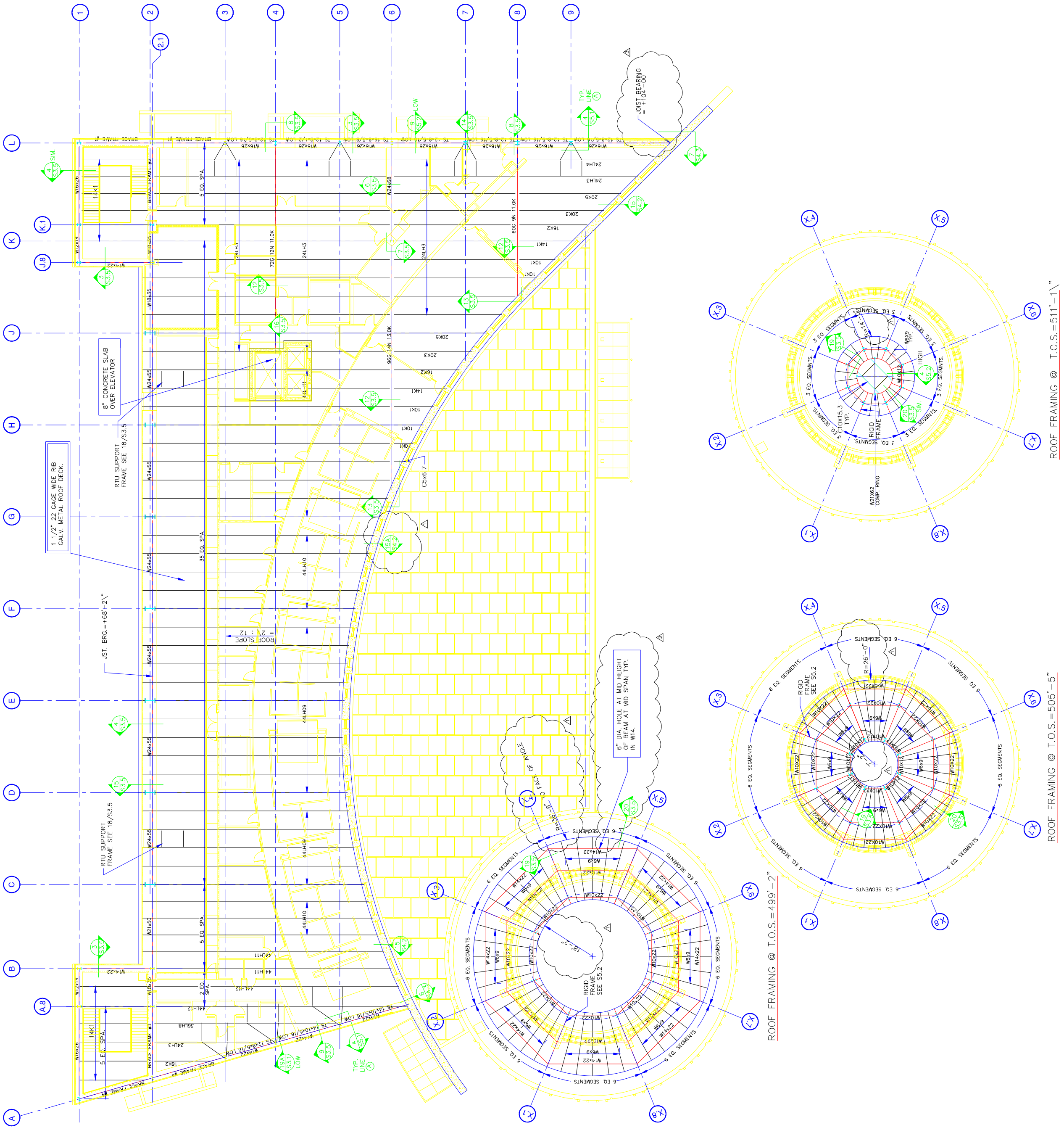
A.3 Third Floor Structural Plans



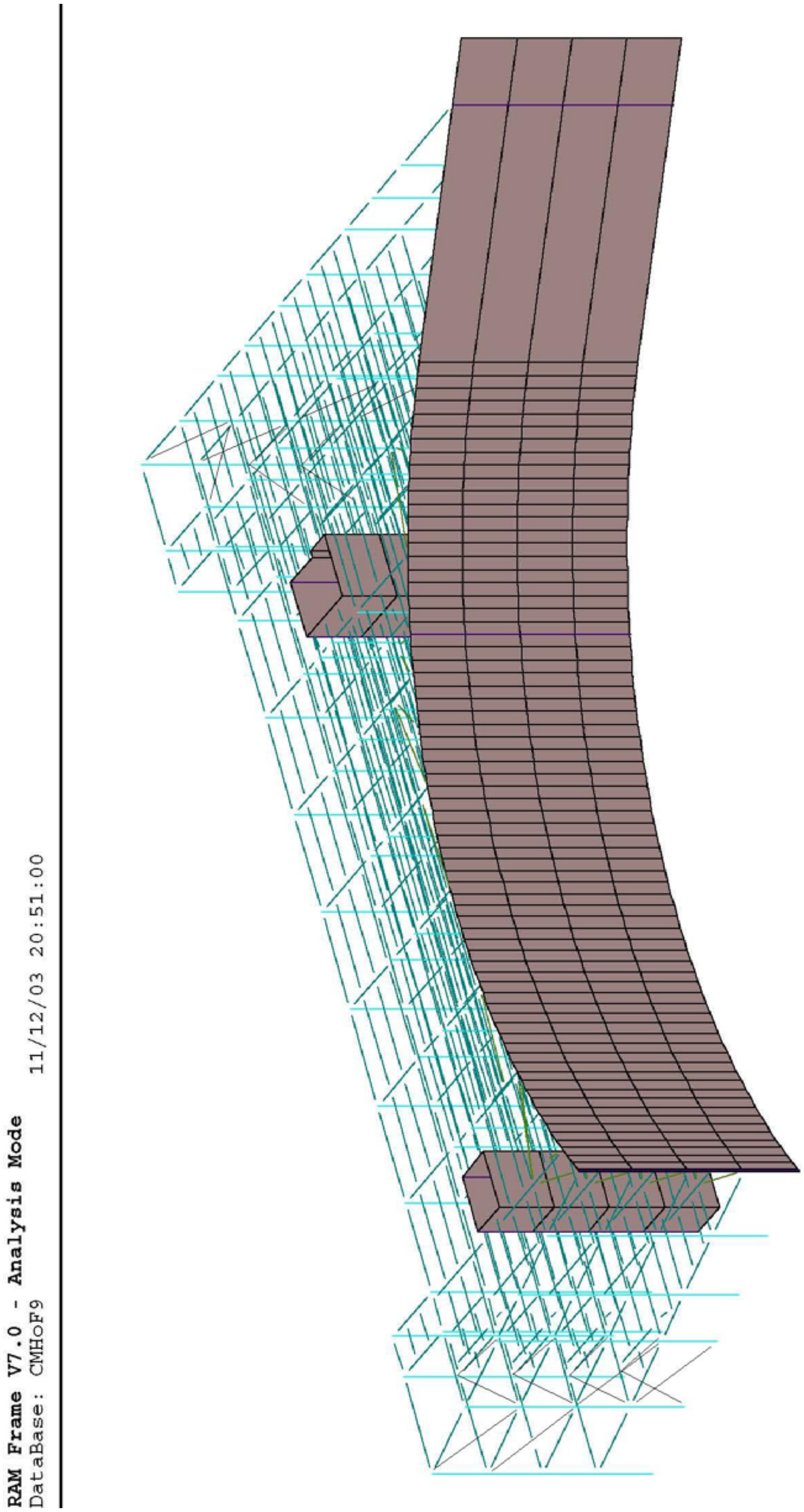
A.4 Fourth Floor Structural Plans



A.5 Roof Structural Plans



A.6 3-D Structural View



APPENDIX B

Daylighting Nomograph

B.2 Nomograph Summary

Assumptions

To build the nomograph on the next page assumptions and approximations had to be made.

The first step is to determine what percentage of occupied hours will find daylight available. The museum has latitudinal location of 36.07 degrees north. The typically daily schedule of occupancy was chosen from 8 am to 6 pm.

Step two is finding the percentage savings due to daylighting controls. Continuous dimming is assumed for lighting control. The choice is expensive, but is less disturbing to occupants and tends to deliver higher energy savings. The illumination level was assumed 50 foot candles for typical desk work. Double pane medium reflectance glazing was selected, have a glazing visible transmittance (VT) of 0.2.

Step three if finding the percentage of total floor area that can be daylighted. The layout is open-plan with low partitions with a ceiling height higher than 9 feet with a correspondingly high window head, therefore assuming a 20-foot zone. Daylight area percentage is calculated by the product of the zone depth and total perimeter length and then the quotient of the total floor area.

Step four is finding the potential energy and load savings from daylighting (percentage reduction over a non-daylighted building). Defaults were used for the dimming factor and potential energy savings and load savings were determined from a nomograph.

Step five is finding the lighting energy and cost savings due to daylighting. A value of 1.5 was chosen for lighting power densities for an office space.

B.3 Nomograph Worksheet

Item	Project
Latitude	36.07 N
Daily Occupancy Schedule	8 am—6 pm
Gross Area per Floor	20,000
Width to Length Ratio	1:2
Daylight Zone Depth	20
Lighting Control Type	Continuous
Illumination Level (fc)	50
Useful Window Ratio	0.50
Glazing VT	0.2
Useful Window Ratio x VT	0.13
Annual Hours of Occupancy	2,500
Installed Lighting Load (W/ft2)	1.5
Electricity Cost ($/kWh)	0.10
Gross Total Building Area (ft2)	20,000
Non-Lighting Electric Loads (W/ft2)	3.5
Peak Demand Rate ($/kWh-month)	11
Daylit Hours (%)	0.945
Control Effectiveness (%)	44
Dimming Factor (%)	80
Daylit Area (%)	0.91
Annual Energy Saving due to Daylight (%)	38
Daylight Peak Load Savings (%)	78
Non-Daylit Lighting Energy Consumption	4
Non-Daylit Lighting Energy Cost	0.40
Daylighting Energy Consumption Savings	1.8
Daylighting Energy Cost Savings	0.18
Annual Daylighting Energy Savings ($)	3,600
Non-Daylit Peak Demand (kW)	100
Non-Daylit Monthly Demand Charge	0.002
Non-Daylit Annual Demand Charge	0.025
Daylit Peak Demand Savings (kW)	25
Daylit Monthly Demand Savings	0.001
Daylit Annual Demand Savings	0.01
Total Annual Savings	0.19

APPENDIX C

Cost Sheets

C.2 Open Web Joists Takeoff

Roof

Steel Joist	w (lb/ LF)	# LF	Crew	Daily Out	Labor Hr	Unit	Mat	Labor	Equip	Total	Ttl O&P	City Index	Bare Cost	Weight (k)
10K1	5.00	46.00	E7	1,200.00	0.07	LF	2.10	2.43	1.31	5.84	8.15	96%	359.90	0.23
14K1	6.00	284.84	E7	1,500.00	0.05	LF	2.52	1.95	1.04	5.51	7.45	96%	2,037.15	1.71
16K2	6.30	62.17	E7	1,800.00	0.04	LF	2.65	1.62	0.87	5.14	6.80	96%	405.83	0.39
20K3	8.20	53.34	E7	2,000.00	0.04	LF	3.38	1.46	0.78	5.62	7.25	96%	371.25	0.44
20K5	8.20	63.00	E7	2,000.00	0.04	LF	3.38	1.46	0.78	5.62	7.25	96%	438.46	0.52
24LH3	13.00	1,465.75	E7	1,400.00	0.06	LF	6.05	2.09	1.12	9.26	11.65	96%	16,392.95	19.05
36LH8	21.00	59.30	E7	1,800.00	0.04	LF	9.75	1.62	0.87	12.24	14.65	96%	834.00	1.25
44LH09	22.00	1,017.75	E-7	2,200.00	0.04	LF	10.25	1.33	0.71	12.29	14.45	96%	14,118.23	22.39
44LH10	22.00	589.75	E-7	2,200.00	0.04	LF	10.25	1.33	0.71	12.29	14.45	96%	8,181.01	12.97
44LH11	22.00	801.00	E-7	2,200.00	0.04	LF	10.25	1.33	0.71	12.29	14.45	96%	11,111.47	17.62
44LH12	22.00	161.00	E-7	2,200.00	0.04	LF	10.25	1.33	0.71	12.29	14.45	96%	2,233.39	3.54
TOTAL													$56,483.64	80.11

C.3 4th Floor Steel Takeoff

4th Floor

Steel Members	w (lb/LF)	# LF	Crew	Daily Out	Labor Hr	Unit	Mat	Labor	Equip	Total	Ttl O&P	City Index	Cost	Weight (k)
W10x12	12.00	212.00	E2	600.00	0.09	LF	8.40	3.36	2.36	14.12	17.86	96%	3,634.87	2.54
W10x22	22.00	58.33	E2	600.00	0.09	LF	15.40	3.36	2.36	21.12	25.56	96%	1,431.28	1.28
W12x14	14.00	320.00	E2	880.00	0.06	LF	9.80	2.29	1.61	13.70	16.66	96%	5,116.92	4.48
W14x22	22.00	43.20	E2	990.00	0.06	LF	15.40	2.03	1.43	18.86	22.15	96%	918.71	0.95
W16x26	26.00	1,888.20	E2	1,000.00	0.06	LF	18.20	2.01	1.42	21.63	25.19	96%	45,653.31	49.09
W18x35	35.00	242.60	E2	960.00	0.08	LF	24.50	3.04	1.55	29.09	34.11	96%	7,942.98	8.49
W21x44	44.00	29.00	E2	1,064.00	0.08	LF	30.80	2.72	1.40	34.92	40.30	96%	1,121.86	1.28
w21x50	50.00	59.33	E2	1,064.00	0.08	LF	35.00	2.75	1.40	39.15	44.97	96%	2,561.37	2.97
W24x55	55.00	100.00	E2	1,110.00	0.07	LF	38.50	2.63	1.34	42.47	48.54	96%	4,659.76	5.50
W27x84	84.00	37.75	E2	1,190.00	0.07	LF	58.80	2.46	1.25	62.51	70.47	96%	2,553.67	3.17
W27x94	94.00	58.50	E2	1,190.00	0.07	LF	65.80	2.46	1.25	69.51	78.17	96%	4,389.77	5.50
W33x118	118.00	84.33	E2	1,176.00	0.07	LF	82.60	2.48	1.27	86.35	96.70	96%	7,828.79	9.95
W33x130	130.00	38.50	E2	1,134.00	0.07	LF	91.00	2.58	1.32	94.90	106.18	96%	3,924.32	5.01
W36x194	194.00	263.00	E2	1,125.00	0.07	LF	135.80	2.60	1.33	139.73	155.50	96%	39,261.76	51.02
Studs	# =	2,429.00									2.50		6,072.50	
TOTAL													$137,071.86	151.23

C.4 3rd Floor Steel Takeoff

3rd Floor

Steel Members	w (lb/LF)	# LF	Crew	Daily Out	Labor Hr	Unit	Mat	Labor	Equip	Total	Ttl O&P	City Index	Cost	Weight (k)
W10x12	12.00	129.50	E2	600.00	0.09	LF	8.40	3.36	2.36	14.12	17.86	96%	2,220.36	1.55
W10x22	22.00	7.50	E2	600.00	0.09	LF	15.40	3.36	2.36	21.12	25.56	96%	184.03	0.17
W12x14	14.00	105.00	E2	880.00	0.06	LF	9.80	2.29	1.61	13.70	16.66	96%	1,678.99	1.47
W14x22	22.00	118.00	E2	990.00	0.06	LF	15.40	2.03	1.43	18.86	22.15	96%	2,509.44	2.60
W16x26	26.00	3,689.00	E2	1,000.00	0.06	LF	18.20	2.01	1.42	21.63	25.19	96%	89,193.44	95.91
W18x35	35.00	110.75	E2	960.00	0.08	LF	24.50	3.04	1.55	29.09	34.11	96%	3,626.07	3.88
W21x44	44.00	54.25	E2	1,064.00	0.08	LF	30.80	2.72	1.40	34.92	40.30	96%	2,098.65	2.39
w21x50	50.00	20.50	E2	1,064.00	0.08	LF	35.00	2.75	1.40	39.15	44.97	96%	885.02	1.03
W24x55	55.00	94.50	E2	1,110.00	0.07	LF	38.50	2.63	1.34	42.47	48.54	96%	4,403.48	5.20
W24x62	62.00	61.42	E2	1,110.00	0.07	LF	43.40	2.63	1.34	47.37	53.93	96%	3,179.84	3.81
W24x68	68.00	32.00	E2	1,110.00	0.07	LF	47.60	2.63	1.34	51.57	58.55	96%	1,798.63	2.18
W27x84	84.00	196.90	E2	1,190.00	0.07	LF	58.80	2.46	1.25	62.51	70.47	96%	13,319.66	16.54
W27x94	94.00	69.25	E2	1,190.00	0.07	LF	65.80	2.46	1.25	69.51	78.17	96%	5,196.44	6.51
W36x194	194.00	129.00	E2	1,125.00	0.07	LF	135.80	2.60	1.33	139.73	155.50	96%	19,257.67	25.03
Studs	# =	1,607.00									2.50		4,017.50	
TOTAL													$153,569.20	168.24

APPENDIX D

Museum Photos

FRONT FAÇADE

CONSERVATORY

FORD THEATER

INTERIOR OF MUSEUM

SPIRAL STAIRCASE/GIFT SHOP

INTERIOR OF HALL OF FAME

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About the Author

David L. Clark II is currently a sixth-year undergraduate student at the Pennsylvania State University majoring in the study of Architectural Engineering. David has an extreme interest and reverence for structural design in the field of log and timber buildings. His plans are to graduate in May 2004, with an accredited A.B.E.T. Bachelor of Architectural Engineering, with a discipline in Structural Engineering and a Residential Building Industry Minor. David will have well over 200 college course credits, including non-required courses in residential design/development and wood products. He not only has a tremendous amount of education in structural design, but in construction management, electrical/illumination, M.E.P, and residential housing design as well. He has previously gained professional experience by interning with *Ibacos*, a residential research and engineering company, designing heating, air conditioning and duct layouts for large national residential home developers. In this coming May, David will gain his undergraduate degree and soon afterwards will enter the timber industry and incorporate his knowledge of structural design in the field. David has also received a passing score on the Fundamentals of Engineering Exam and wishes to continue on his path to receive a Professional Engineering License.

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