THE PENNSYLVANIA STATE UNIVERSITY
SCHREYER HONORS COLLEGE

DEPARTMENT OF ARCHITECTURAL ENGNIEERING

THE STRUCTURAL REDESIGN OF THE LiUNA EXPANSION BUILDING UTILZING CROSS-LAMINATED TIMBER AND ENGINEERED WOOD PRODUCTS AND ASSOCIATED PRECEDENT STUDIES, FIRE PROTECTION DESIGN, AND CONSTRUCTABILITY ANALYSIS

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A thesis
submitted in partial fulfillment
of the requirements
for a baccalaureate degree
in Architectural Engineering
with honors in Architectural Engineering

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ABSTRACT

The LiUNA Headquarters Expansion features a rectangular structural grid on a 5,300 SF floor plan and nine occupied stories. The building information is on the following page. For the purposes of this thesis, the expansion structure was considered as a stand-alone structure. The structural re-design investigated changing the structural framing system to heavy timber with glulam beams and columns with Cross-Laminated Timber (CLT) floor deck as the primary structural components. The original floor plan was changed to reduce structural depth to an acceptable limit and reduce bay sizes more typical of those found in timber construction. In order to preserve the original architectural intent of an open floor plan and unrestricted views through both the North and South curtainwall, a lateral force resisting system of wood moment frames were designed using bolted moment connections and A36 steel plates.

Since heavy timber structural design is not a traditional focus of structural engineering, the additional views, drawings, and images were created using a 3D model in Sketch Up to help those unfamiliar with timber engineering to quickly understand the proposed structural system.

The mechanical breath focuses on fire protection systems for timber. A dual-part solution is presented that includes passive protection via the charring and encapsulation methods and mechanical protection via wet sprinkler systems. The construction management breath identifies key tasks in the timber erecting process, required skills, means, and methods, and potential cross-over points in the steel and prefabricated construction processes. Those crossover points would allow tradespersons from alternative industries to learn the timber erection process quickly while allowing for quick growth of the industry in new markets.

Laborer's International Union of North American Headquarters Renovation and Expansion

905 16th Street N.W. Washington D.C

Building Statistics

Project Type: Occupied Building Renovation & Building Addition

Occupancy: Office, Street-Level Retail, & Parking Garage

Size: 9 Stories & Penthouse

-Existing Structure

-79,181 SF Office Space

-29,792 SF Parking

-4880 SF Mechanical

-New Structure

-49,078 SF Office Space

-11,742 SF Parking

-5300 SF Mechanical

Project Contract: $33 Million; Guaranteed Maximum Price

Project Team

Owner: Laborer's International Union of North America

General Contractor: James G. Davis Construction

Architect: Gensler

Geotechnical Engineer: ECS Mid-Atlantic LLC

Civil Engineer: Wiles Mensch Co.

Structural Engineer: Thornton Tomasetti

MEP Engineer: GHT Limited

Lighting Consultant: SBLD Studio

Elevator Consultant: Lerch Bates, Inc.

Project Information

Architecture: The existing building was built in 1958-1959 and features a 4" limestone veneer façade in the federal-style architecture of the neighborhood. The addition is designed to attain LEED Silver certification. Floor to ceiling glass curtainwall allows for unrestricted views through the open-office plan while the MEP and egress cores are stored within the existing building.

Construction Management The constricted site restricts laydown area to one lane of adjacent I-Street protected by Jersey barriers. The lower parking level is 18' laterally from the Blue/Orange Metro line, thereby requiring that excavation support be designed to the WMATA's *Adjacent Construction Project Manual* requirements for sheeting and shoring rakers and/or slurry walls.

Electrical & Lighting : 120 V, 3-phase power services the building. Parking garage fixtures use 32W T8 fluorescent lamps with electronic zero-degree ballasts. interior lighting is provided by fluorescent lamps.

Mechanical : A factory prefabricated mechanical penthouse will be delivered to site for placement on the roof. The dedicated outdoor air system features a chiller plant with 2 chilled water pumps, evaporators, and compressors while the boiler plant features 3 natural gas boilers and 3 hot water pumps. A direct drive FANWALL system supplies 21,000 CFM and works in tandem with a 21,000 CFM heat recover wheel.

Structural: The below-ground parking garage is supported by 9" two-way slabs that sit upon concrete piers and foundation walls. At locations where the new structure meets the existing building, #5 rebar dowels are drilled and epoxied 12" O.C., eliminating the use of an expansion joint. Steel framing and composite deck provide gravity support while steel moment frames provide lateral resistance in the East-West direction, preserving unobstructed views through occupied floors. The prefabricated penthouse and adjacent green roof are supported by a cast-in-place 9" NWC slab.

Image Courtesy of James G. Davis Construction

LiUNA!

Laborers' International Union of North America

Josh Jaskowiak
Structural Option
jmj5373.wix.com/josh-Jaskowiak-cpep

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The author would like to thank all of the many people involved with this thesis:

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Kevin Parfitt, Dr. Ryan Solnoksy, and Dr. Walter Schneider for their instruction, guidance, and assistance with this thesis and the education necessary to complete it. A special thanks to Dr. Thomas Boothby for providing the initial support for this endeavor and pushing me to go beyond the scope of my initial considerations.

Davis Construction including Paul Athanas, Patrick Cotter, Jonathan Dougherty, Greg Ghent, Stephen Ghent, Jonathan Dougherty, Bill Moyer, and Lauren Zegowitz for their continued support, insight, and help in securing the LiUNA Expansion Building as my senior thesis project and affording me my first opportunity in the building construction industry.

The American Wood Council, especially Kenneth Bland, for providing a forum to discuss the possibilities of structural timber construction and the fire protection of wood structures.

All of the following industry professionals who provided guidance along the way including but not limited to Tedd Benson, Kevin Chamberlin, Jim DeSteffano, Richard Mistrick, Paul Parfitt, Alex Schreyer, Jessie Smith, Heather Sustersic, and Mark Taylor.

All my friends and family for their continued support, understanding, and encouragement.

DEDICATION

To my parents, for their continued support over the years. Thank you for giving me a solid foundation on which I could build.

Chapter 1: Building Introduction and Existing Conditions

1.1 Building Information

1.1.1 Building Background, Usage, and Occupancy

Located at 16th and I Streets in the Federal District Neighborhood of Washington D.C., the Laborer's International Union of North America (LiUNA) Building and recently completed expansion building serves at the headquarters of the Laborer's Union for both its American and Canadian contingencies. The mission of LiUNA, also known as the Laborer's Union, is to represent unionized construction workers, trades workers, mail carriers, and those involved in the construction and maintenance of infrastructure.

The original LiUNA Headquarters was built in 1959 using a steel frame, steel deck, and featured a two-story parking garage beneath the building and a two-story mechanical penthouse above the 8th floor. From the beginning, the LiUNA Headquarters Building, also known as the Moreschi Building, was intended to be the LiUNA headquarters and be occupied by offices.

To match the original design, the headquarters addition is also a steel frame with a two-story parking deck beneath grade, a mechanical penthouse above the 9th floor, and a screened-in mechanical screen wall above the penthouse. A curtainwall façade will stretch around the north and south faces of the LiUNA addition; the existing building and the existing neighbor structure abut the East and West faces.

The project contract structure is that of a Guaranteed Maximum Price of $33 million after several changes. The scope of the project includes the complete interior renovation of the existing structure and the replacement of all windows and window encasements in the original structure in addition to the new structure. All nine stories are designated as office space featuring an open-office layout, which requires that open sight lines be maintained throughout the floor plan. The mechanical and elevator core located within the original structure provides mechanical, electrical, and fire protection service for the existing and expanded structure. The original building core also includes the means of egress, all elevators, and bathrooms.

On the following page, Figure 1-1 shows one of the site plans from the construction documents and is included for reference. The original LiUNA Building, the Expansion Building, and the surrounding lots are shown.

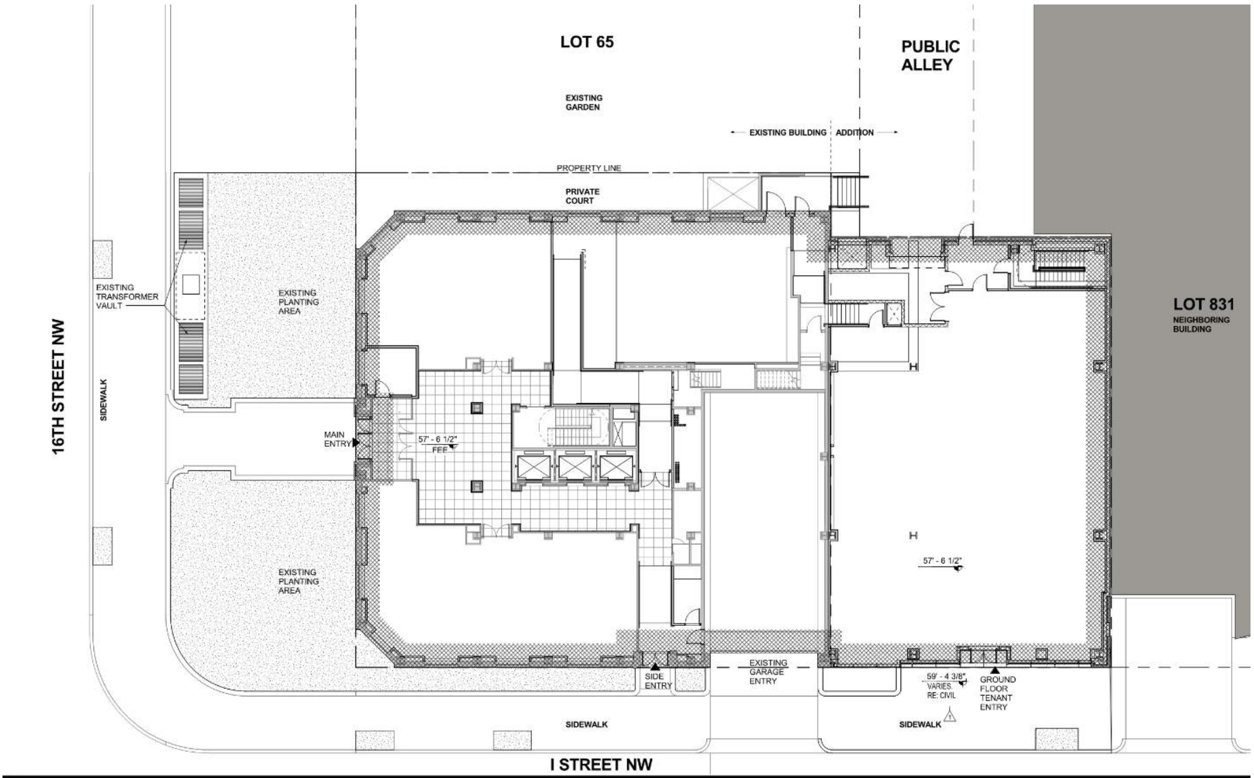


Figure 1-1: Site Plan

1.2 Building Loads

1.2.1 Gravity Design Loads

The loads in Table 1-1 were listed in the original design documents as being used by the design team. In the structural redesign, the assumed gravity and live loads changed significantly at various locations due to the change in building material and assumed live loads. In Chapter 3, the assumed loads for the gravity redesign will be introduced.

Table 1-1: Original Gravity Design Loads

Loading Plan Schedule		
Description	SDL (PSF)	LL (PSF)
Parking	10	50
Retail	50	100
Office	10	100
8” Green Roof	100	30*
Prefabricated Mechanical Roof PH	75	180*
Roof – Ballast and Pavers	50	30*
Roof Terrace – Ballast and Pavers	50	100*
Office Incl. New 3” LWC Slab on Insulation	55	100
Office Incl. 1 ½” Stone Tile Finish	35	100
Mechanical Room	10	150 or Actual Wt.
Parking	10	50
Storage	10	125
* Indicates Roof Condition. Snow Load of 25 PSF + Drift per DC Building Code Also Considered for Design		

In the structural redesign, the 8” green roof was eliminated along with all additions to the penthouse. Since the structural wood design is intended to emphasize sustainable design, the green roof was eliminated to reduce loads and minimize column sizes. The office floor loads were assumed to be 50 PSF with an additional 15 PSF for partitions. The original 100 PSF was used to ensure that the floor would not be affected by vibrations. Since the floor spans will be less and floor vibrations were considered in the design of the new floor system, a reduction in floor loads was deemed appropriate. The penthouse deck (the roof) was redesigned with a load of 180 PSF to account for self-weight and 150 PSF of mechanical equipment. Since the mechanical equipment is integral to occupancy of the building, it was considered as dead load.

1.2.2 Lateral Design Loads

The original design used ASCE 7-05 to determine both the wind and seismic design loads. In the structural redesign, the lateral load calculations were re-run using ASCE 7-10 guidelines. Wind was found to control in both cases.

Tables 1-2 and 1-3 are the load calculations of wind forces according to ASCE 7-05. Tables 1-4 and 1-5 represent the East-West and North-South base shears.

Table 1-2: East-West Wind Pressures According to ASCE 7-05

East-West Wind								
Gust Factor in E-W	1.78							
							Net Pressure (PSF)	
Location	**z (ft)**	**q_z or q_h (PSF)**	**C_p**	**q_zGC_p (PSF)**	**GC_{pi}**	**q_hGC_{pi} (PSF)**	**$q_zGCp-q_h(+GC_{pi})$**	**$q_zGCp-q_h(-GC_{pi})$**
Windward	0.00	10.05	0.80	14.31	0.18	3.21	11.11	17.52
	13.34	10.05	0.80	14.31	0.18	3.21	11.11	17.52
	24.28	11.46	0.80	16.32	0.18	3.21	13.11	19.52
	35.22	12.87	0.80	18.33	0.18	3.21	15.12	21.53
	46.16	13.93	0.80	19.84	0.18	3.21	16.63	23.04
	57.10	14.81	0.80	21.09	0.18	3.21	17.88	24.30
	68.04	15.51	0.80	22.09	0.18	3.21	18.88	25.29
	78.98	16.40	0.80	23.35	0.18	3.21	20.15	26.56
	90.92	16.92	0.80	24.09	0.18	3.21	20.89	27.30
	107.92	17.81	0.80	25.36	0.18	3.21	22.16	28.57
Leeward	All	17.81	-0.42	-13.31	0.18	3.21	-16.52	-10.11
Side	All	17.81	-0.70	-22.19	0.18	3.21	-25.40	-18.99
Parapet (WW)	108.92	17.81			1.50			26.72
Parapet (LW)	108.92	17.81			-1.00			-17.81
Mechancial PH (WW)	123.92	18.51			1.50			27.77
Mechancial PH (LW)	123.92	18.51			-1.00			-18.51
Roof 0' to 54'	107.92	17.81	-1.30	-41.21	0.18	3.21	-44.42	-38.01
Roof 54' - 95'	107.92	17.81	-0.70	-22.19	0.18	3.21	-25.40	-18.99

Table 1-3: North-South Wind Pressure Calculations According to ASCE 7-05

North-South Wind								
Gust Factor in N-S	1.92							
							Net Pressure (PSF)	
Location	**z (ft)**	**q_z or q_h (PSF)**	**C_p**	**q_zGC_p (PSF)**	**GC_{pi}**	**q_hGC_{pi} (PSF)**	**$q_zGCp-q_h(+GC_{pi})$**	**$q_zGCp-q_h(-GC_{pi})$**
Windward	0.00	10.05	0.80	15.44	0.18	3.21	12.23	18.64
	13.34	10.05	0.80	15.44	0.18	3.21	12.23	18.64
	24.28	11.46	0.80	17.60	0.18	3.21	14.40	20.81
	35.22	12.87	0.80	19.77	0.18	3.21	16.56	22.97
	46.16	13.93	0.80	21.40	0.18	3.21	18.19	24.60
	57.10	14.81	0.80	22.75	0.18	3.21	19.54	25.95
	68.04	15.51	0.80	23.82	0.18	3.21	20.62	27.03
	78.98	16.40	0.80	25.19	0.18	3.21	21.98	28.40
	90.92	16.92	0.80	25.99	0.18	3.21	22.78	29.19
	107.92	17.81	0.80	27.36	0.18	3.21	24.15	30.56
Leeward	All	17.81	-0.50	-17.10	0.18	3.21	-20.30	-13.89
Side	All	17.81	-0.70	-23.94	0.18	3.21	-27.14	-20.73
Parapet (WW)	108.92	17.81			1.50			26.72
Parapet (LW)	108.92	17.81			-1.00			-17.81
Mechancial PH (WW)	123.92	18.51			1.50			27.77
Mechancial PH (LW)	123.92	18.51			-1.00			-18.51
Roof 0' to 54'	107.92	17.81	-1.30	-44.45	0.18	3.21	-47.66	-41.25
Roof 54' - 95'	107.92	17.81	-0.70	-23.94	0.18	3.21	-27.14	-20.73

Table 1-4: East-West Base Shear Calculations

East-West Base Shear Calculations				
Trib H	**Trib L**	**Max Load**	**Story Force**	
(ft)	**(ft)**	**(PSF)**	**(Lbs)**	**(K)**
16.00	60.00	28.57	27424.55	27.42
8.50	60.00	27.30	13922.94	13.92
20.44	60.00	26.56	32572.45	32.57
10.94	60.00	25.29	16601.70	16.60
10.94	60.00	25.29	16601.70	16.60
10.94	60.00	25.29	16601.70	16.60
10.94	60.00	23.04	15124.85	15.12
10.94	60.00	21.53	14134.05	14.13
10.94	60.00	19.52	12816.10	12.82
12.12	60.00	17.52	12738.36	12.74
		Base Shear (Kips)		**178.54**

Table 1-5: North-South Base Shear Calculations According to ASCE 7-05

North-South Base Shear Calculations				
Trib H	**Trib L**	**Max Load**	**Story Force**	
(ft)	**(ft)**	**(PSF)**	**(Lbs)**	**(K)**
16.00	95.00	27.77	42202.80	42.20
8.50	95.00	30.56	24678.78	24.68
20.44	95.00	29.19	56690.70	56.69
10.94	95.00	28.40	29512.17	29.51
10.94	95.00	27.03	28091.41	28.09
10.94	95.00	25.95	26973.95	26.97
10.94	95.00	24.60	25569.15	25.57
10.94	95.00	22.97	23877.00	23.88
10.94	95.00	20.81	21626.13	21.63
12.12	95.00	18.64	21465.09	21.47
		Base Shear (Kips)		**300.69**

Tables 1-6 and 1-7 show the resultant seismic base shear and vertical distribution of seismic forces in the North-South and East-West directions, respectively. The wind loads controlled the original design.

Table 1-6: Vertical Distribution of Seismic Forces in the North-South Direction According to ASCE 7-05

Vertical Distribution of Seismic Forces

	Base Shear		V=CsW	Cs= 0.01		V= 39.7842
		East-West				
		k=	1.79		$C_{vx} = w_x h_x$^k/Σ$w_x h_x$^k	Shear Story Force
Floor	H (Ft)	h_x^k	w_x	$w_x h_x$^k		$F_x=C_{vx}V$
PF Mech PH						
Penthouse	107.92	4357.65	1456.92	6348749.21	0.67	26.75602103
Green Roof						0
Mech PH						0
9	90.92	3206.27	282.55	905932.18	0.10	3.817939506
8	78.98	2492.04	282.55	704126.96	0.07	2.967456287
7	68.04	1908.31	282.40	538905.73	0.06	2.27115176
6	57.1	1394.37	282.40	393770.64	0.04	1.659497808
5	46.16	952.87	282.40	269091.81	0.03	1.134054253
4	35.22	587.16	282.40	165812.64	0.02	0.69879693
3	24.28	301.71	282.40	85203.97	0.01	0.359081632
2	13.34	103.28	276.15	28521.61	0.00	0.120200795
1	0	0.00	268.25	0.00	0.00	0
			Σ$w_x h_x$^k	9440114.7		

Table 1-7 Vertical Distribution of Seismic Forces in the East-West Direction According to ASCE 7-05

Vertical Distribution of Seismic Forces

	Base Shear		V=CsW	Cs= 0.02		V= 79.5684
		North-South				
		k=	1.09		$C_{vx} = w_x h_x$^k/Σ$w_x h_x$^k	Shear Story Force
Floor	H (Ft)	h_x^k	w_x	$w_x h_x$^k		$F_x=C_{vx}V$
PF Mech PH						
Penthouse	107.92	164.47	1456.92	239616.55	0.59	46.68119022
Green Roof						0
Mech PH						0
9	90.92	136.44	282.55	38550.86	0.09	7.510332966
8	78.98	117.03	282.55	33066.56	0.08	6.441901607
7	68.04	99.47	282.40	28091.68	0.07	5.472715719
6	57.1	82.17	282.40	23205.88	0.06	4.520881825
5	46.16	65.17	282.40	18404.09	0.05	3.585415726
4	35.22	48.53	282.40	13704.56	0.03	2.669870459
3	24.28	32.35	282.40	9136.62	0.02	1.779962532
2	13.34	16.84	276.15	4651.20	0.01	0.906128951
1	0	0.00	268.25	0.00	0.00	0
			Σ$w_x h_x$^k	408428.01		

For the purposes of the structural redesign, the lateral loads were found using ASCE 7-10. Wind loads were found to control. Shown below in Tables 1-8 and 1-9 are the resultant story forces in each direction for the controlling wind load case. The story forces in the far right ASD column were used for the design of the elements and for evaluating serviceability requirements.

Table 1-8: North-South Story Forces According to ASCE 7-10

	East-West Wind Story Force Calculator (For Moment Frames)									
Floor	Trib H	Trib W	WW	LW	Total	Story Force LRFD			Story Force ASD	
	(ft).	(ft)	PSF	PSF	PSF	Lbs	K		K	K
PH	16	89.33	45.33	30.22	75.55	107982.1	107.9821		64.78926	
	8.5	89.33	34.88	21.8	56.68	43037.41	43.03741	151.0195	25.82244	90.61171
9	14.47	89.33	33.16	21.8	54.96	71041.58	71.04158	71.04158	42.62495	42.62495
8	11.44	89.33	32.12	21.8	53.92	55102.75	55.10275	55.10275	33.06165	33.06165
7	10.94	89.33	30.4	21.8	52.2	51013.5	51.0135	51.0135	30.6081	30.6081
6	10.94	89.33	29.02	21.8	50.82	49664.87	49.66487	49.66487	29.79892	29.79892
5	10.94	89.33	27.92	21.8	49.72	48589.87	48.58987	48.58987	29.15392	29.15392
4	10.94	89.33	25.21	21.8	47.01	45941.47	45.94147	45.94147	27.56488	27.56488
3	10.94	89.33	22.45	21.8	44.25	43244.21	43.24421	43.24421	25.94652	25.94652
2	12.12	89.33	19.68	21.8	41.48	44909.55	44.90955	44.90955	26.94573	26.94573

Table 1-9: East-West Story Forces According to ASCE 7-10

	North-South Wind Story Force Calculator (For Braced Frames)									
Floor	Trib H	Trib W	WW	LW	Total	Story Force LRFD			Story Force ASD	
	(ft).	(ft)	PSF	PSF	PSF	Lbs	K		K	K
PH	16	60.08	45.33	30.22	75.55	72624.7	72.6247		43.57482	
	8.5	60.08	44.42	27.76	72.18	36860.88	36.86088		22.11653	65.69135
9	14.47	60.08	42.22	27.76	69.98	60837.64	60.83764		36.50259	36.50259
8	11.44	60.08	40.9	27.76	68.66	47191.06	47.19106		28.31464	28.31464
7	10.94	60.08	38.7	27.76	66.46	43682.51	43.68251		26.20951	26.20951
6	10.94	60.08	36.95	27.76	64.71	42532.28	42.53228		25.51937	25.51937
5	10.94	60.08	34.75	27.76	62.51	41086.27	41.08627		24.65176	24.65176
4	10.94	60.08	32.1	27.76	59.86	39344.49	39.34449		23.6067	23.6067
3	10.94	60.08	28.59	27.76	56.35	37037.46	37.03746		22.22247	22.22247
2	12.12	60.08	25.06	27.76	52.82	38461.92	38.46192		23.07715	23.07715

1.3 <u>Existing Building Structure</u>

1.3.1 Existing Below-Grade Layout

The original LiUNA Headquarters Building and the addition both feature two floors of bellow-grade parking supported by a concrete structure. The original parking garage layout is shown in Figure 1-2. Please note that the architectural drawing, not the structural, is shown below.

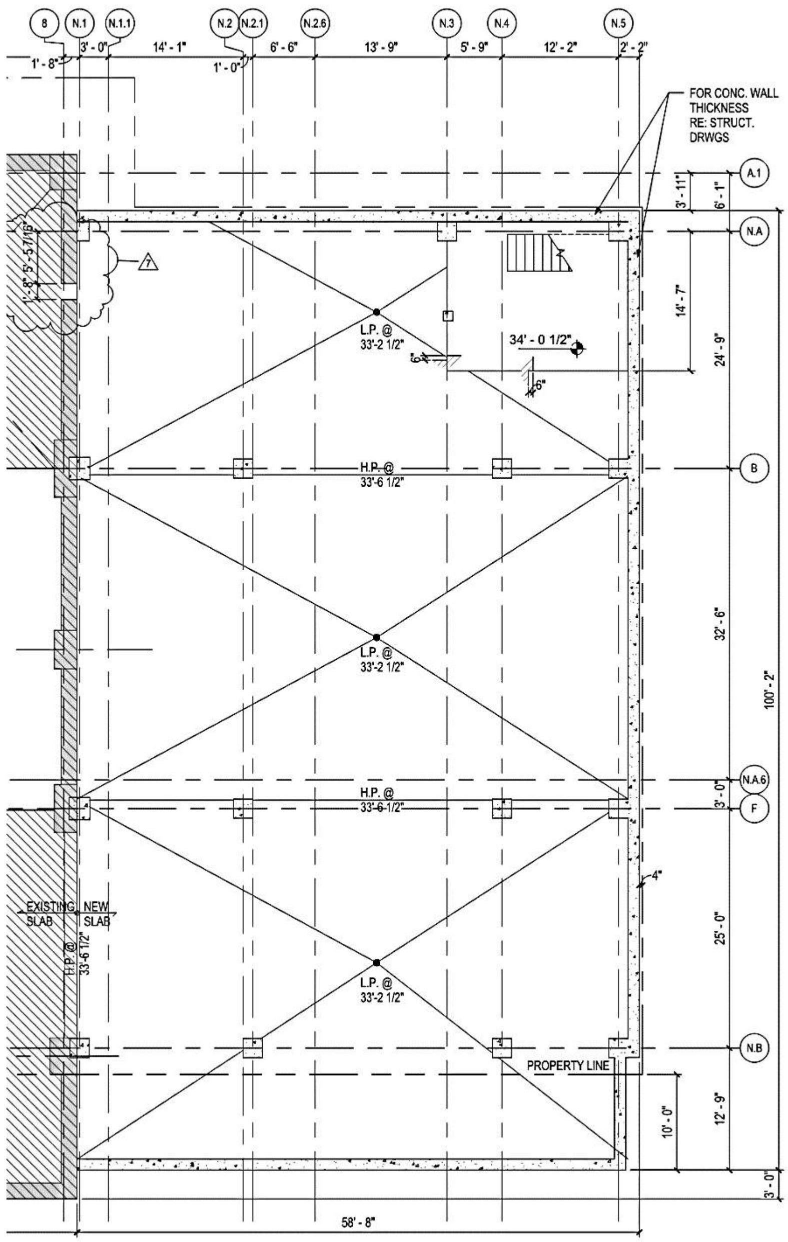


Figure 1-2: Original Below-Grade Parking Garage Layout

1.3.2 Existing Above-Grade Building Layout and Structural Grid

Below in Figure 1-3 is the existing above grade structural layout for the 4th floor. The 4th floor was the typical floor layout throughout the project.

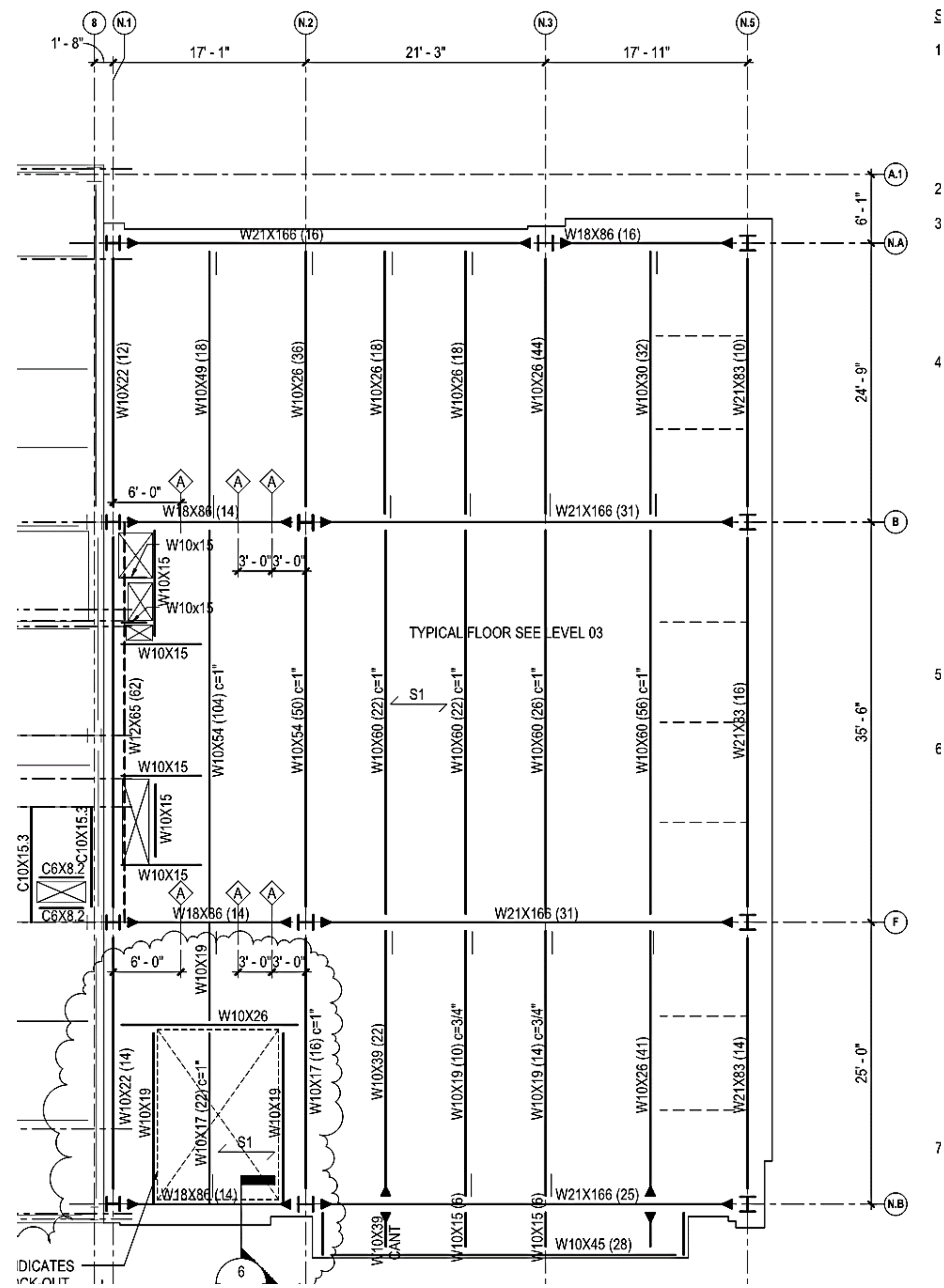


Figure 1-3: Original Above-Grade Structural Layout at 4th Floor

1.3.3 Existing Lateral System in E-W Direction - Moment Frames

In the original design, there were four steel moment frames spanning in the East-West direction. As previously mentioned, these moment frames were a critical component of the design. Since the lateral system could not impede the inter-story space, moment frames provided a continuous view from the back of the building through to the street. Below in Figure 1-4 is the design of moment frame 3, one of the two original interior moment frames.

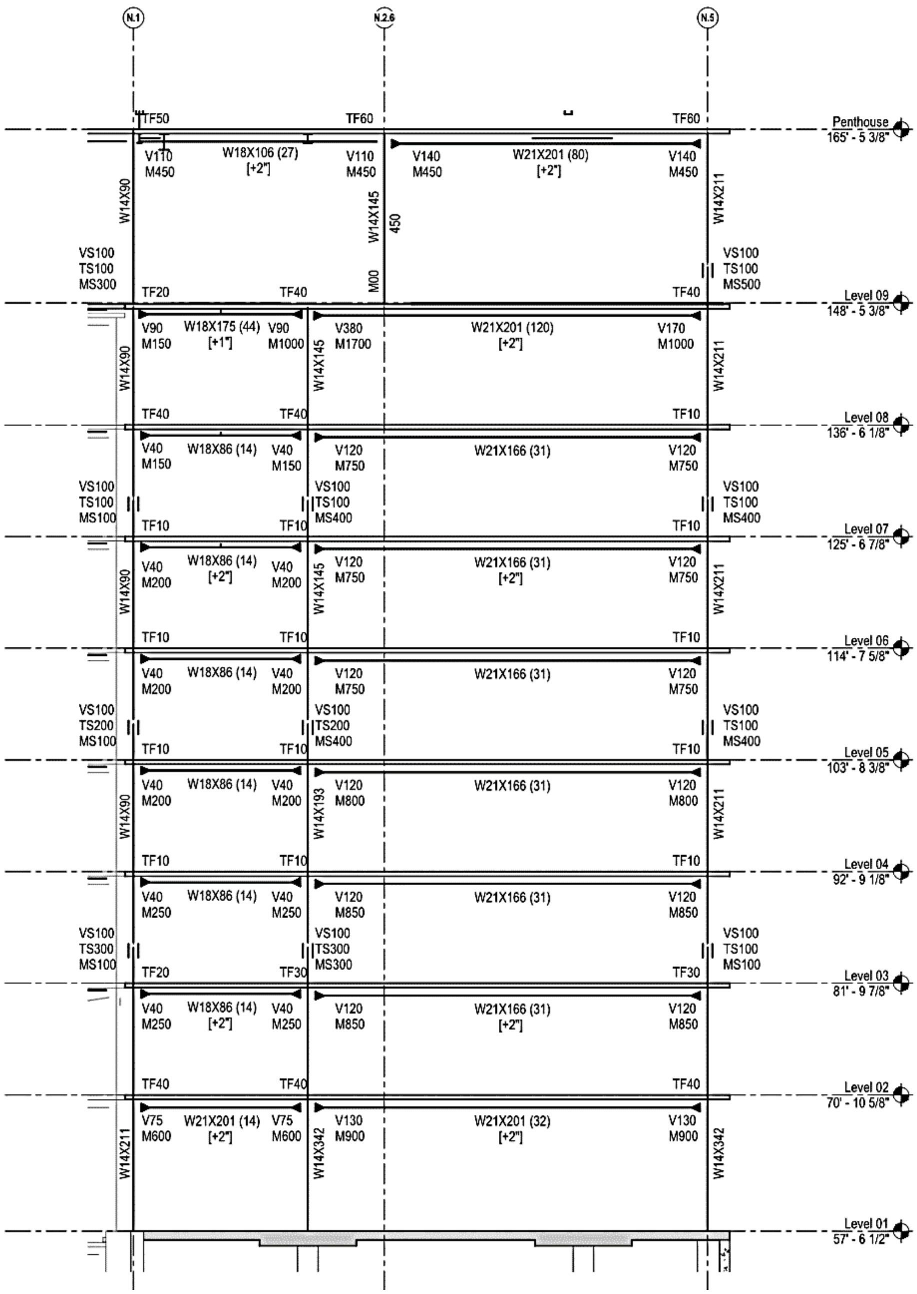


Figure 1-4: Original Moment Frame 3

1.3.4: Existing Lateral System in the North-South Direction – Vertical Truss

In the original design, two steel vertical trusses at the Eastern and Western edges of the building provided lateral resistance. The original design is shown in Figure 1-5.

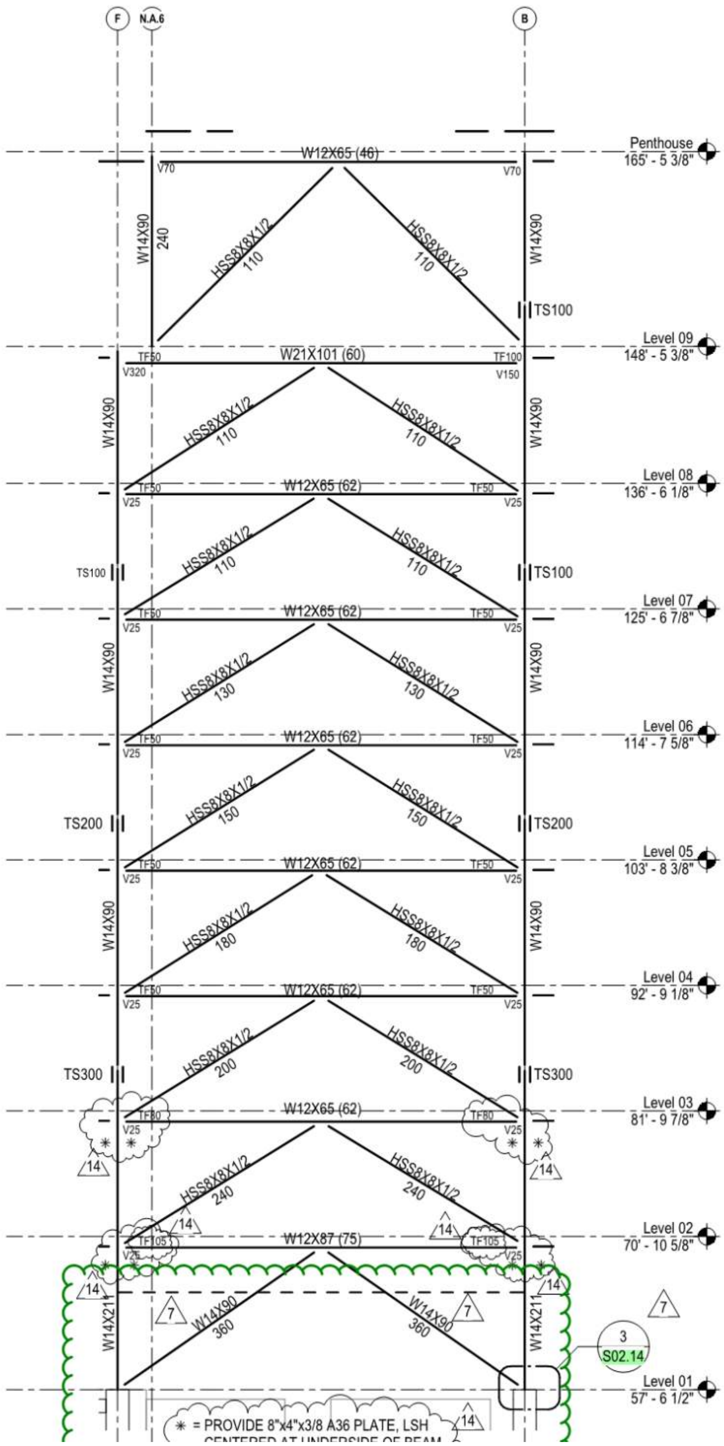


Figure 1-5: Original Vertical Truss

1.4 Proposed Alternative Structural System

1.4.1 Proposed Structural Timber Construction System Overview

The redesign of the LiUNA Expansion Building uses engineered wood products (EWP) and is a case study investigating the issues and solutions of structural timber construction (STC). The floor system is cross-laminated timber floor deck supported by glulam girders. The girders are attached to columns via custom steel knife plate bolted connections. The building's lateral system in the East-West direction are five moment frames that use a custom moment-resisting bolted connection with steel plates. Two vertical trusses exist in the North-South direction.

1.4.2 Proposed Alternative Structural Grid

Before a STC system was selected for this project, several feasibility studies were conducted. The main point of these studies was to see if the dimensional requirements of a wood system would be incompatible with the office occupancy and proposed floor-to-floor height. The CLT floor systems were found to be controlled by vibrations and their span is practically limited to 20'-25' depending on the loads present. Since the original structural grid featured spans in the range of 30' to 35' in various locations, CLT would not work within the existing grid.

Additional feasibility studies for other systems were also conducted. A 2-way concrete floor system was investigated and was initially favored because it would provide the best correlation between a structural alternative and the main type of construction in the urban D.C. area (concrete). The two-way system was found to be practically limited to spans ranging from 20' to 30.' While post-tensioned slabs were a possibility, they also posed issues due to the need for dead ends at the slab limits and pour strips. These results indicated that revisions to the existing structural grid would be required almost regardless of the system selected. Since the structural grid did not bear directly on the office interior finishing, it was possible to change the layout of the structural grid so long as the continuous views through both faces of the curtainwall were preserved.

The resultant revised alternative structural grid is shown in Figure 1-6.

The proposition of a new structural grid affects the below-grade parking lot. The original design featured 17 parking spaces per level. The new design allows for 16 spaces, but there is enough space for the storage of mopeds or bikes – alternative modes of transportation that were mentioned in the LEED Certification documents. While the LEED rating calculation assumed alternative transportation via bike or moped, the parking lot documents do not call out a set area allotting space for mopeds. The revised design accomplishes this goal. Note that the below-grade parking deck extends out beyond the edge of the structural gird by approximately 15' to the edge of the property.

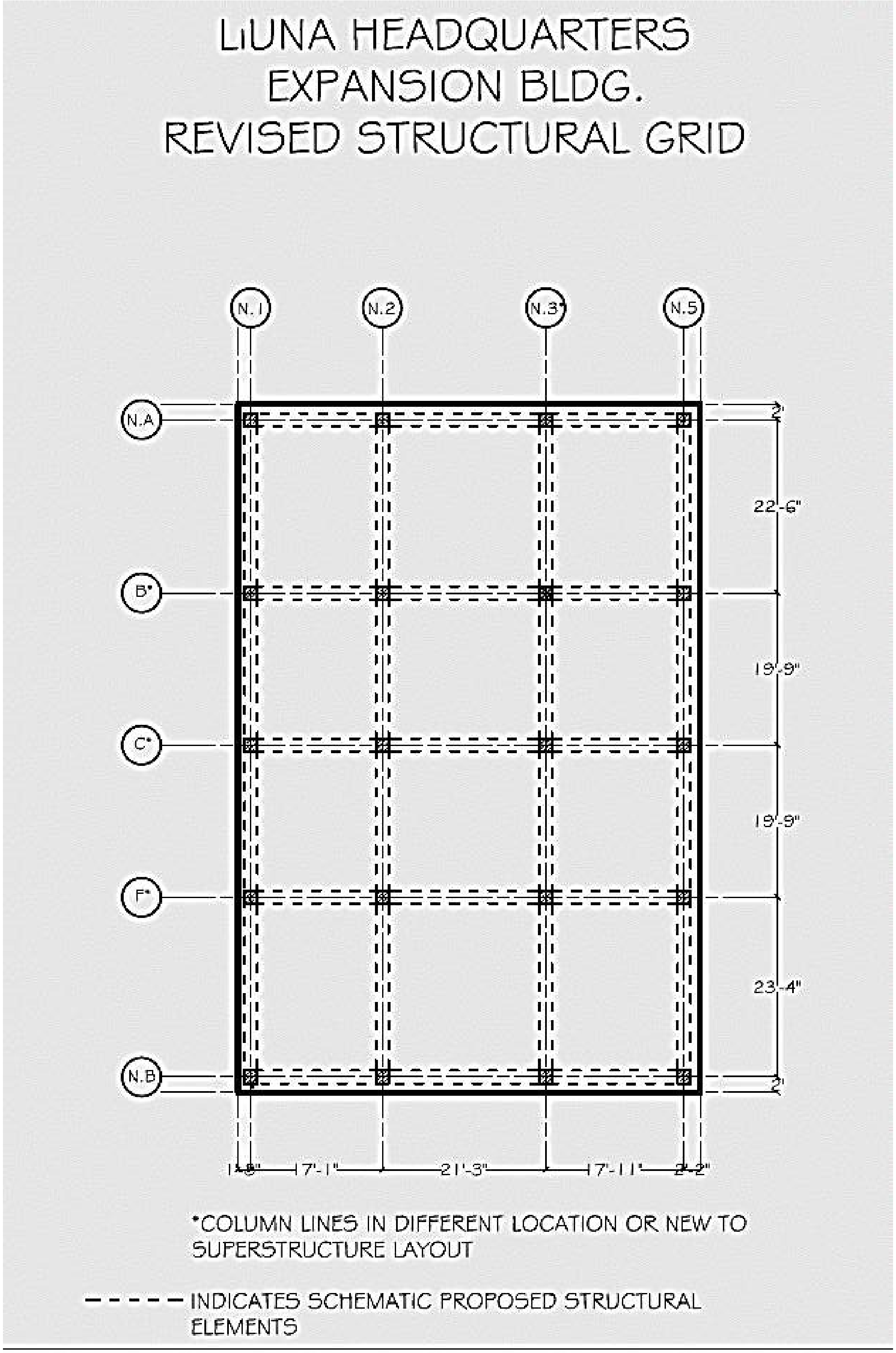


Figure 1-6: Proposed Alternative Structural Grid

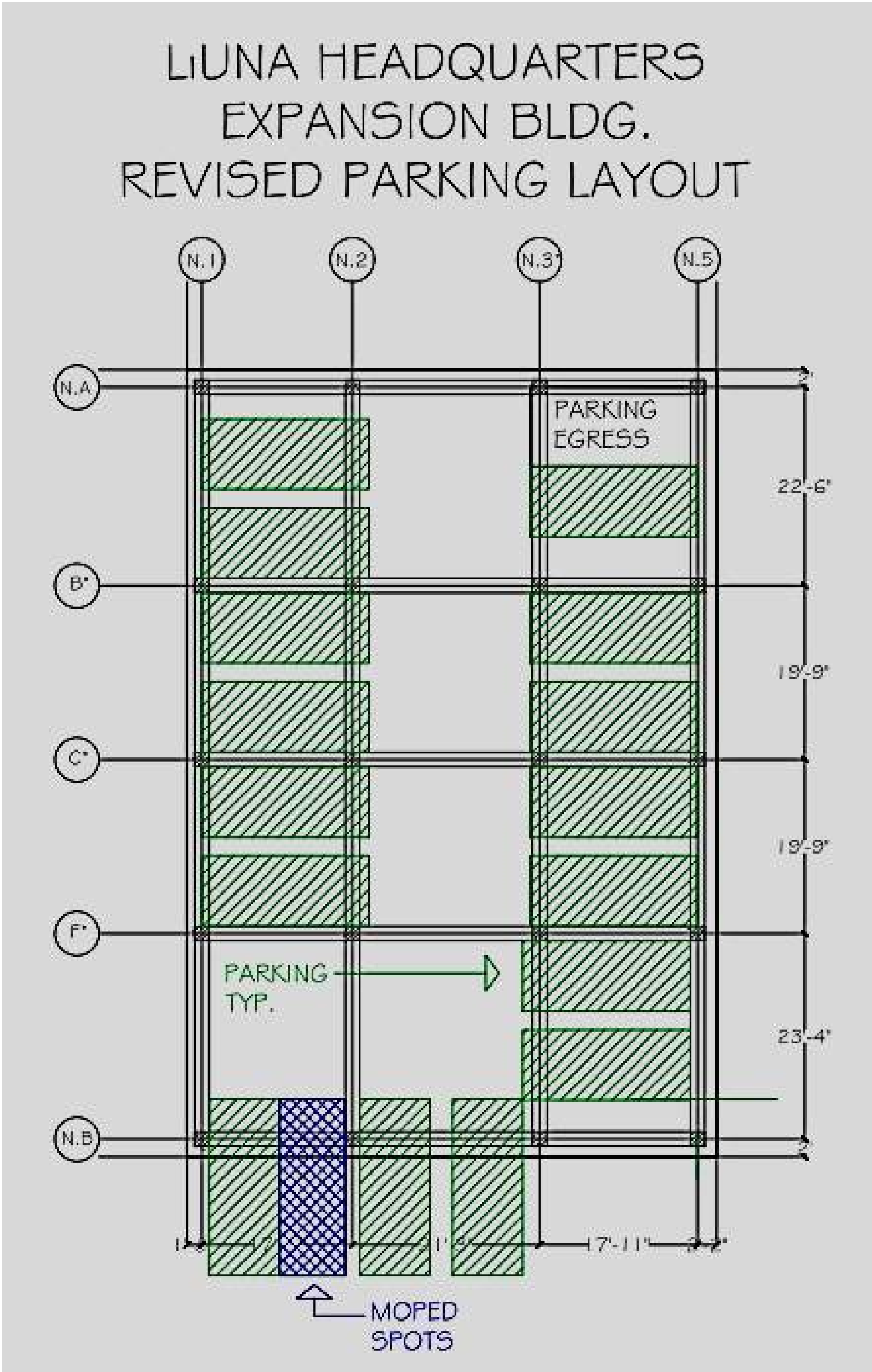


Figure 1-7: Revised Bellow-Grade Parking

Chapter 2: Wood Building Systems

2.1 History of Wood Structural Systems

2.1.1 History of Wood Design

Wood construction stands alongside stone masonry and adobe construction as one of the oldest construction types known to man. While many ancient wood structures have been lost to time, there are some that have survived and help display the evolution of wood craftsmanship and the durability of the material. In Western history, one of the oldest and best known uses of timber comes courtesy of Julius Caesar and the 140m long by 5-6m wide temporary timber bridge he used to cross the Rhine River in his 55 BC invasion of Gaul (modern-day Germany). Built using hand tools, trees cut on-site, and his large forces of legionnaires, Caesar was able to cross the Rhine after only 10 days of work. That moment in history would serve as an indication to the versatility and constructability of wood structures.

Some of the oldest wood buildings in the world are religious structures. The Yakushi-ji East Pagoda in Nara, Japan, shown in Figure 2-1, was built in the 8th century and still stands in a larger Buddhist Temple Complex that has been rebuilt over the years. While the original outlier buildings on the temple campus have been replaced in later generations, the East Pagoda has withstood centuries of earthquakes, wars, and weather to amaze visitors (Larsen, 2).

Figure 2-1: The Yakushi-ji East Pagoda, Image Courtesy of Wikipedia

In Horu-Ji, Japan, the Golden Hall still stands from 677 as a Buddhist temple. Originally commissioned by Empress Suiko in 607, the temple complex was intended to accommodate the needs of the new Buddhism population. Buddhism came to Japan in 552. While its original predecessor was built in 607 and burned down in 670, the Golden Hall and surrounding five-story pagoda, inner gate, and surrounding outdoor passageways represent almost the entire history of Buddhism in Japan in carved wood and craftsmanship.

Across the Yellow Sea, the wood that would go into the Forbidden City nearly 800 years later would be known as *shan mu* or "sacred timber" for its importance in creating the highly symmetrical design intended to represent the perfection of heaven and the tremendous effort required to ship the wood to the capital city. 200 years later, when rebuilding efforts were undertaken to replace losses due to fire, Lei Fada would be promoted to the role of head of construction on the board of works. Lei Fada's family would go on to establish a set of procedures surrounding the production, approval, and usage of construction documents in a day and age where the great master builders of Europe were still drawing plans in the dirt.

The fiords of Norway and the surrounding area also gave rise to an interesting intersection of Pagan tradition, expanding Christian influence, and wood construction. Dating from the 12th century, Nordic Churches combined an interesting combination of Romanesque architecture, shipbuilding forms, beautifully carved softwood relief sculptures, and a functional perspective of adaptive design. The many structures still-standing bear witness to additions built over time. Urnes, Hopperstad, Loemn, and Borgund are four of the twenty-eight structures that survive. In Figure 2-2, Borgud shines an example of unaltered medieval wood craftsmanship.

Figure 2-2: Borgud Stave Church, Norway. Image Courtesy of Wikipedia

Wood quickly provided a means of survival for colonists looking to survive in the New World. Indebted to companies financing expeditions to a new land and new life, colonists found themselves producing tradable commercial goods. The vast timber reserves of North America proved to be a viable financial venture, albeit a painstaking one, for many a colonist. Plagued by decades of deforestation and natural resource mismanagement, Old-World European powers found themselves in desperate need of lumber not only for buildings but for the ships which would flatten the globe and allow ever-expanding powers to become entangled in never-ending naval warfare up to the days of Napoleon.

Austere Puritan Cape Cods and Saltboxes would give way to meetinghouses, quintessential white church steeples dotting the New England landscape, and water-powered mills and factories. Founded out of necessity, these structures would prove to be simple in form, practical, and omnipresent in the ever-expanding American industrial fabric up to the 20th century (Pryce 32-33, 62-65, 76-83, 172-197).

2.1.2 Modern Wood Construction

One of the last major wood-frame buildings to be built in America that is still standing is Butler Square, located in the warehouse district of Minneapolis. Built in 1906, the building represents main construction methods used at the turn of the 1900s. Load-bearing masonry walls surrounded the 500,000 square foot warehouse's nine-story walls. Douglas fir beams and columns fit together on a grid measuring about 14' x 16' and are held together with cast iron rings at the connections. Originally used as an industrial warehouse for a mail-order retail company, the floors were designed for long-term storage and the associated heavy loads. The Butler Center would get new life in the 1970s thanks to a renovation that opened up the interim atrium and converted it to prime downtown office space. Thanks to the unique interior, open-air layout, and warm feel provided by the wood finishes, the Butler Center continues to be an

Figure 2-3: Butler Square Interior, Image courtesy of Butler Square Homepage

example of the design possibilities provided by wood design as shown in Figure 2-3 (Butler Square, Gateway to the Warehouse District).

The importance of the Butler Square building is that is marks the last point in the history of the American construction industry where wood was routinely used as a major structural material. Within 20 years, the load-bearing masonry walls would be replaced by punched concrete walls with veneer masonry. Wide-flange steel shapes would become more available nation-wide. Advances in concrete made fire-resistant structures a possibility in an era before modern automatic sprinkler systems. Lateral resistance would increasingly be provided by steel frames or concrete shear walls, not by the fortress-like load-bearing masonry walls previously seen. The cast iron connections used in Butler Square would quickly be replaced by rivets, bolts, and welded steel. With the rise of concrete and steel, the traditional methods of using wood as a primary structural material would fall to the wayside.

Out of the past 100 years of dormancy, the topic of wood engineering has begun to experience a renaissance. While beginning slowly overseas, wood buildings have once again become a popular topic in American design discussion circles. Projects from around the world and here in the United States are serving as case studies for further research and development. These are their stories.

2.1.3 Bullitt Center

In 2013, the 6-story Bullitt Center in Seattle, Washington opened as part of the design team's mission to build the "greenest commercial building in the world" and "to drive change in the marketplace faster and further by showing what's possible today." While there are many innovative features, the focus feature from a structural perspective is the multifaceted structural system that includes an all-wood gravity system. Figure 2-4 highlights the structural system. 6" x 14" beams and 10" x 10" glulam members from Vancouver, Canada reduced waste on-site and allowed the building to sequester 545 metric tons of CO_2 (Bullitt Center, 2016).

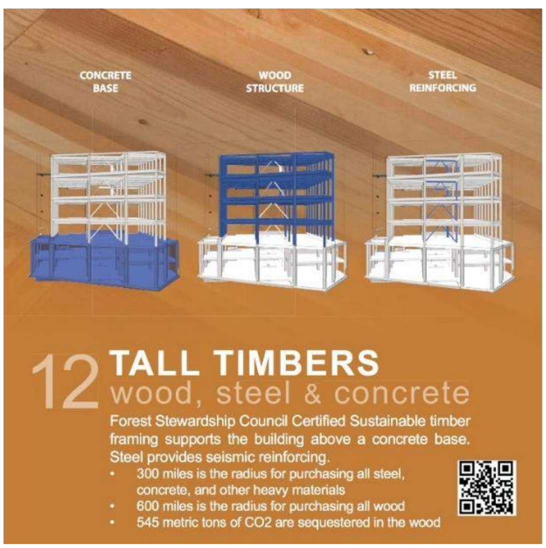


Figure 2-4: The Bullitt Center Complete Structural System, Image courtesy of the Bullitt Center

Figure 2-5: Bullitt Center Interior as Shown in the 2014 Wood Works Design Awards, Image Courtesy of Wood Works

2.1.4 T3

At the time of this report's publication, the T3 building in Minneapolis, MN is the first "tall wood" project in the United States in recent history. The project is seven stories with the top six featuring glulam beams and columns for the gravity framing. A traditional concrete shear wall core is the lateral system. The first tall timber building in America, T3 is at the vanguard of a new approach in wood design. By considering modern fire sprinkler systems, the charring fire-protection properties of wood, and the spirit of the fire protection code, the design team was able to work with the authority having jurisdiction to get support for the project. Rather than use CLT, the project uses nailed laminated timber, NLT, floor panels due to the reduction in cost, production time, and lack of structural CLT available in the United States.

The Figure 2-5, 2-7, and 2-8 come from the Structure Craft blog about the T3 project.

Figure 2-6: Interior Rendering of T3, Rendering Courtesy of Structure Craft

Figure 2-7: Construction of T3, Image Courtesy of Structure Craft

Figure 2-8: Exterior Rendering of T3, Image Courtesy of Structure Craft

2.1.5 Framework Tower

In 2015, the United States Department of Agriculture announced the winners of the "U.S. Tall Wood Building Prize Competition," an initiative intended to incentive the design of tall wood buildings. Framework Tower in Portland Oregon and 475 West 18th in New York City were announced as the winners.

Designed by Lever Architecture, Framework Tower features 12 above-grade stories, all of which are completed with wood products. The lateral shear wall core is built using CLT panels while glulam framing composes the structural system. At 130 feet, this will be the tallest all-wood building in the world when complete (Grozdanic). Figure 2-9 is a model of the structural system and Figure 2-10 is an exterior rendering of the final project.

The United States government has taken a keen interest in wood as a building material. Projects like the Tall Wood Building Prize Competition represent an awareness of the potential impact wood manufacturing and production can have on the environment and the American economy. While quick-growing softwood trees represent a quick way to sequester carbon dioxide in buildings and continually harvest the only renewable building material, they also have the potential to impact the field of American job creation. According to the tall wood competition panel and USDA, "35 jobs are created for each million board feet of wood processed." If engineered wood products can grow by 5 to 15% in the non-residential construction market, that growth would represent an increase of 0.8 to 2.4 billion feet of wood consumed. Using the estimate provided by the USDA, that type of market-share increase could result in 28,000 to 84,000 new jobs that are directly related to the production of American-made building materials (U.S. Tall Wood Building Prize Competition).

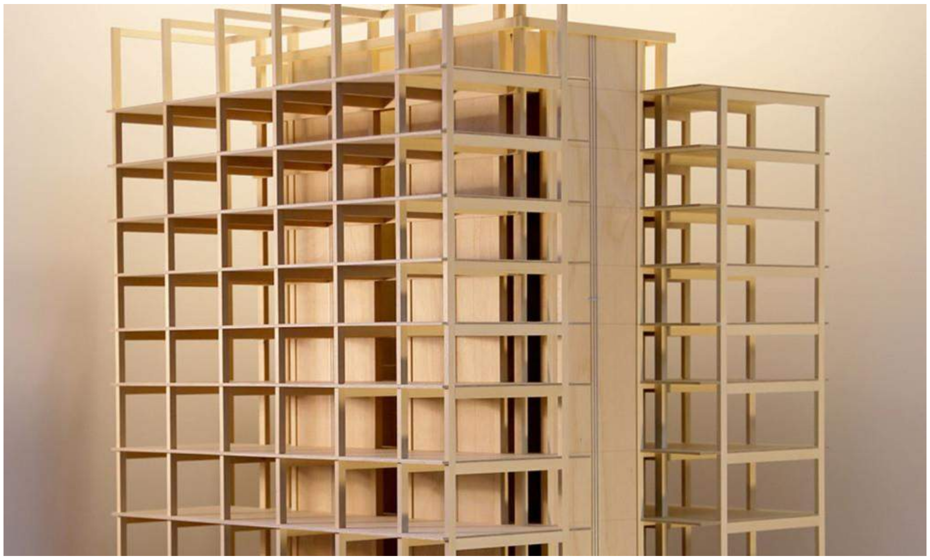


Figure 2-9: Framework Tower Design, Image Courtesy of Inhabitat

Figure 2-10: Framework Tower Exterior Rendering, Image Courtesy of Inhabitat

2.1.6 University of Massachusetts at Amherst Integrated Design Building

At the time of this report's publication, one of the most technologically advanced wood buildings in North American is being built. This building is The Integrated Design Building on the campus of the University of Massachusetts at Amherst. Set to be the home of the Landscape Architecture and Regional Planning, Department of Architecture, and the Building Construction Technology Program, the IDB will serve as a place to teach modern design in a building that practices what is taught. Designed to be a zero net energy building, the IDB steps beyond the focus of energy to consider the sustainable contributions of the building materials. Originally intended to be a steel composite building, wood was selected due to its ability to sequester carbon and other renewable qualities. (Turner).

Within the building, a plethora of technology from around the world is used. Beyond the net zero design, the wood products are state of the art. CLT panels are manufactured and shipped from Nordic Structures in Quebec. A real time feedback loop between the timber erection team composed of Union Carpenters, the timber framing specialists of Bensonwood, and the CLT manufacturing group at Nordic allowed the design and construction teams to provide instantaneous feedback based upon conditions observed in the field. This feedback loop could inform decisions regarding connection detail alterations. The quick response would enhance constructability of elements still on the manufacturing line. The communication allowed the design team to adapt to the variety of connections used on the project. Seen in Figure 2-11 is a composite deck connection between the CLT panel and concrete slab to be poured at a later date. This type of composite system, developed in Germany, is new to the United States and applies the theory of composite floors to wood deck in an attempt to gain strength and alleviate the vibration concerns apparent in many CLT designs. Figures 2-12, 2-13, and 2-14 show the building under construction.

Figure 2-11: CLT Composite Floor System with Steel Plate Composite Connection, Image Courtesy of Alex Schreyer

Figure 2-12: The Integrated Design Building, Image Courtesy of Alex Schreyer

Figure 2-13: Weather Protection Measures at the IDB Site, Image Courtesy of Alex Schreyer

Figure 2-14: A Connection within the IDB Using a Flitch Beam, Glulam Beam and Column, and Steel HSS Base Plate Connection, Image Courtesy of Alex Schreyer

The IDB represents the future of structural timber construction in the United States. Through its emphasis on sustainable design, the application of technology new to the United States, and high profile status as a design center at a land-grant University, the IDB brings the possibilities of wood to center stage and can serve as a powerful case study for projects to come.

2.2 Wood within the Context of LiUNA

2.2.1 East-Coast Wood Construction and The Importance of a Case Study

As shown in the case studies above, structural wood construction is a growing component of the construction industry but is not yet prominent enough to be considered a mainstream design possibility. Buildings like those previously discussed help show that structural timber construction is possible, but those case studies can be seen as special cases. Through either the initiative of the owner or incentivized design competitions, wood has been used in select instances. In order for wood to be used more often, design professionals need to explore the limits of its applications. That is why more case studies are needed.

The goal of this paper is to see how a wood building could be built on the East Coast. This paper does not seek to identify wood as the easiest material with which to work, the cheapest, or the most applicable to the project requirements at hand. Rather, this project looks to take a look at some of the more challenging aspects of wood design. The LiUNA project requires the use of moment frames, something that is not easily done in wood. In this manner, the redesign of the LiUNA Expansion Building represents the worst case design scenario: a building that has no lateral shear wall core and required the use of moment frames so that every column is now a beam-column.

Within the context of this project, it would be impossible to identify and discuss all aspects of this project. Components such as construction costs, the logistics of just-in-time shipping, the visual appearance of mechanical systems, final interior finish selection, lighting design, and building enclosure attachment are all valid questions surrounding a structural timber building but are not addressed in this piece. Instead, emphasis is placed on components that are essential to the success of the project and for which there exists little published information. Construction productivity rates, construction schedule, passive fire protection, and mechanical fire protection are the four issues identified as being critical to understanding the projects potential of being successful. If solutions to those topics could be presented, then the stage could be set for the next wave of design questions.

Chapter 3: Structural Depth – Gravity System

3.1 CLT Floor System

3.1.1 CLT Specifications and Design Process

Cross-Laminated Timber, or CLT, is an engineered wood product made from dimensional lumber laminated together with layers arranged in an alternating orthogonal fashion. The dimensional lumber used can be of any species, but in North America, softwood is primarily used with Douglas fir and Southern Pine being the two main species.

The strands perpendicular to the supporting girder are known as the parallel strands and are represented as the "longitudinal planks" below. The parallel strands are composed of higher grade material than the perpendicular strands. The increased quality of material properties in the parallel strands make them more suitable for resisting stresses due to bending. The perpendicular strands are composed of lower grade wood with Number 3 (No. 3) Grade lumber being the most common. The lamination of the perpendicular strands to the layers above and below helps provide continuity between the stronger layers. Figure 3-1 below shows the alternating layers within a CLT panel.

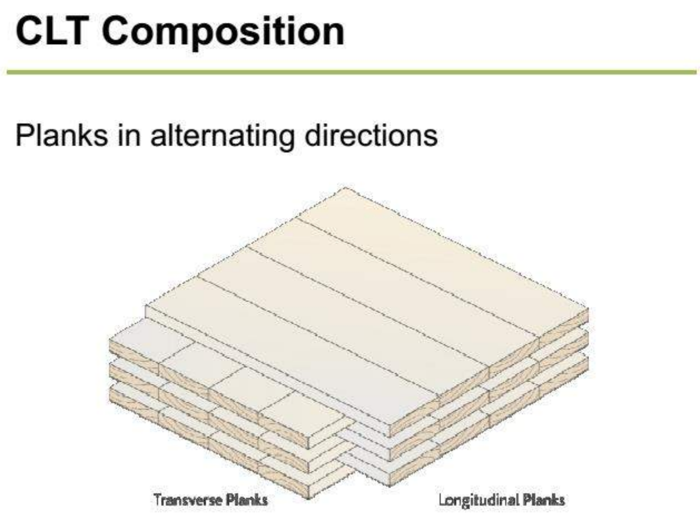


Figure 3-1: CLT Panel Composition from Breneman, Scott

Originally developed in Europe and increasingly used more in Canada, Australia, and New Zealand, CLT is a new to the United States. In response to increased interest in the material, the American Plywood Association published a guide to the structural properties of CLT configurations available in the North American market. *ANSI/APA PRG 320-2012, Standard for Performance-Rated Cross-Laminated Timber* more commonly known by the abbreviation "PRG 320," lists the requirements for grading and manufacturing CLT. A total of seven CLT layup configurations are listed. For the design of the LiUNA Expansion Building, the layup named "E4" was chosen since it provided the best allowable bending capacity of all the softwoods typically available in the Eastern United States. E4 uses "1950f-1.7 E Southern pine MSR lumber in all parallel layers and No. 3 Southern pine lumber in all perpendicular layers"

(American Plywood Association, 2012). A panel thickness of 9 5/8” was selected. The allowable bending capacities provided in Figure 3-2, Table A.2 of the PRG 320, used for this project are listed below:

TABLE A2.
THE ALLOWABLE BENDING CAPACITIES[a,b,c] FOR CLT LISTED IN TABLE A1 (FOR USE IN THE U.S.)

		Lamination Thickness (in.) in CLT Layup							Major Strength Direction			Minor Strength Direction		
CLT Grade	CLT t (in.)	■	⊥	■	⊥	■	⊥	■	$F_bS_{eff,0}$ (lbf-ft/ft)	$EI_{eff,0}$ (10^6 lbf-in.^{2}/ft)	$GA_{eff,0}$ (10^6 lbf/ft)	$F_bS_{eff,90}$ (lbf-ft/ft)	$EI_{eff,90}$ (10^6 lbf-in.^{2}/ft)	$GA_{eff,90}$ (10^6 lbf/ft)
E1	4 1/8	1 3/8	1 3/8	1 3/8					4,525	115	0.46	160	3.1	0.61
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			10,400	440	0.92	1,370	81	1.2
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	18,375	1,089	1.4	3,125	309	1.8
E2	4 1/8	1 3/8	1 3/8	1 3/8					3,825	102	0.53	165	3.6	0.56
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			8,825	389	1.1	1,430	95	1.1
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	15,600	963	1.6	3,275	360	1.7
E3	4 1/8	1 3/8	1 3/8	1 3/8					2,800	81	0.35	110	2.3	0.44
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			6,400	311	0.69	955	61	0.87
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	11,325	769	1.0	2,180	232	1.3
E4	4 1/8	1 3/8	1 3/8	1 3/8					4,525	115	0.53	180	3.6	0.63
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			10,425	441	1.1	1,570	95	1.3
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	18,400	1,090	1.6	3,575	360	1.9
V1	4 1/8	1 3/8	1 3/8	1 3/8					2,090	108	0.53	165	3.6	0.59
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			4,800	415	1.1	1,430	95	1.2
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	8,500	1,027	1.6	3,275	360	1.8
V2	4 1/8	1 3/8	1 3/8	1 3/8					2,030	95	0.46	160	3.1	0.52
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			4,675	363	0.91	1,370	81	1.0
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	8,275	898	1.4	3,125	309	1.6
V3	4 1/8	1 3/8	1 3/8	1 3/8					2,270	108	0.53	180	3.6	0.59
	6 7/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8			5,200	415	1.1	1,570	95	1.2
	9 5/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	9,200	1,027	1.6	3,575	360	1.8

For SI: 1 in. = 25.4 mm; 1 ft = 304.8 mm; 1 lbf = 4.448 N

(a) See Section 4 for symbols.

(b) This table represents one of many possibilities that the CLT could be manufactured by varying lamination grades, thicknesses, orientations, and layer arrangements in the layup.

(c) Custom CLT grades that are not listed in this table shall be permitted in accordance with Section 7.2.1.

Figure 3-2 Allowable Bending Capacities of E4 CLT Panels

Note that while a different layup could have been selected, the only other layup configurations that seemed compatible with the availability of engineered wood products on the East coast were E3 and V3, both of which displayed considerable decreases in capacities per inch of depth as compared to E4.

3.1.2 Strength Design

While the NDS recently has added information regarding the use and selection of CLT, significant additional design guidance was found in the lecture "CLT Floor Design: Strength, Deflection, and Vibrations" presented by Scott Breneman of the Wood Products Council. Breneman's work is a summary of the process prescribed in the CLT Handbook. Breneman's document was used in the design of the CLT floor decks for the LiUNA Expansion. That document serves as a condensed version of the provisions presented in the *CLT Handbook*. The *CLT Handbook* was published in 2013 by FPInnovations and is the collaborative work of the American Wood Council, Wood Works, the American Plywood Association, the Wood Products Council, the Forest Products Laboratory, and FP Innovations. This document serves as the parent design guide for all aspects of CLT design.

At every floor, the dead load was assumed to be 35 PSF and the live load was 65 PSF. The live load accounted for the 50 PSF live load typical for offices and an additional 15 PSF for partitions. The flexural design of CLT floor panels is governed by:

$$M_b \geq (F_b S_{eff})'$$

Where:

$$\left(F_b S_{eff}\right)' = C_D C_T C_m \left(F_b S_{eff}\right)$$

The adjustment factors are for load duration, temperature, and moisture content. Due to the live load condition, the duration factor was 1.0. Temperature and moisture adjustment values typically have a value of 1.0

$$M_b = \frac{wl^2}{8}$$

Where "l" is the span between the supporting gravity girders. For frames N.1-N.2, N.2-N.3*, N.3*-N.5, l was taken to be 17'-1", 21'-3", and 17'-11", respectively. The input load uses the unit strip method, or PSF per foot of width of the panel. Therefore, the section analyzed is a one foot wide strip representative of the entire floor. Using the loads assumed for the redesign:

$$w = w_D + w_L = 35\ PSF + 65\ PSF\ per\ Linear\ Foot\ of\ Width$$

Using the $F_b S_{eff}$ of 18,400 lbf-ft/lb provided by 9 5/8" E4 CLT layup, all flexural demands were met. Due to the low self-weight and flexible properties, flexural design rarely controls the design of CLT floors. Rather, long-term creep deflections or vibrations typically control.

3.1.3 Deflection Design

There are three sets of deflection scenarios that are applicable to CLT floors:

1. Short-term live load deflections
2. Short-term dead and live load deflections
3. Long-term deflections due to dead and live load, also known as creep

The controlling scenario is creep deflection. Creep deflection can be found as:

$$\Delta_T = K_{CR} * \Delta_{LT} + \Delta_{ST}$$

Where:

Δ_{ST} = Short-term deflections due to the effects of the live load

Δ_{LT} = Long-term deflections due to the effects of the dead load

K_{CR} = 2.0 and is a modifier to account for the continued presence of live loads over time and assumes that the CLT is used in dry service conditions

The deflection can be found using:

$$\Delta = \frac{5}{384}\frac{wl^4}{EI_{EFF}} + \frac{1}{8}\frac{wl^4}{\frac{5}{6}GA_{EFF}}$$

E4 9 5/8" CLT Layup has the following properties:

EI_{EFF} = 1090 in2/ft
GA_{EFF}= $1.6*10^6$ lbf-in^2/ft

The deflection criteria of L/240 and L/360 were checked for all spans and were satisfied. The comparison of L/240 and L/360 was completed in anticipation of vibration criteria controlling the design, so while L/240 might be typical of floor systems, the higher standard of L/360 was a good indication of conformance with the vibration criteria.

3.1.4 Vibrations Design

The performance of CLT floors with respect to vibration is controlled by the apparent flexural stiffness of each panel. A combination of the material properties and the longitudinal span, the apparent flexural stiffness is represented by:

$$EI_{APP} = \frac{EI_{APP}}{1 + \frac{K_S\, EI_{EFF}}{GA_{EFF}\, L^2}}$$

Where K_S = 11.5 and is determined by the load pattern and support conditions. In the design of the floors, pinned connections at the supports and a uniformly distributed load were assumed.

While the natural frequency of a CLT floor panel of one foot width can be found, the actual controlling limitation for CLT floors is the maximum recommended span. That approach, recommended as the simplified method by the *CLT Handbook*, makes use of the apparent flexural stiffness and eliminates comparison of natural frequency criteria. This simplified approach is promoted to reduce the learning curve and effort required to evaluate CLT. Natural frequency is still checked against a lower bound of 9.0 Hz.

The natural frequency of a CLT panel can be found to be:

$$f = \frac{2.188}{2L^2}\sqrt{\frac{EI_{APP}}{\rho A}} \geq 9.0\ Hz.$$

Where ρ is the specific gravity of the wood species used. For Southern Pine, the specific gravity was assumed to be 0.55 per page 84 of the 2015 NDS.

A is the cross-sectional area of the CLT panel. Using the unit-strip method, the area was 9.625" x 12" = 115.5 in^2

The maximum span recommended is:

$$L \geq \frac{1}{12.05}\,\frac{{EI_{APP}}^{0.293}}{\rho A^{0.122}} \geq The\ longitudinal\ span\ of\ the\ CLT\ panel$$

Throughout the design process, conservative values of centerline-to-centerline span distances were used rather than the actual span of the panels. Considering that the supporting gravity girders are 10.5" in width, this means that the design accounted for the CLT panels being 21" longer than they actually were. That simplification allows for reserve capacity since the long-term performance of North America CLT layups is not as well-known as other comparable systems.

The typical maximum span allowed was 21.5' at the typical floor compared to 21.25' as the actual centerline to centerline span used. Those results validated the assumption that vibrations would control the floor design.

3.1.5 Fire Protection Design of the CLT Deck

The design of passive fire protection for wood members is intrinsically tied to the responsibilities of the structural engineer. In a fire, the outer edges of the member(s) exposed to fire are assumed to burn and char. Much like a large log being placed on a fire, the outer edges of the wood burn, and as the charred layer develops, the interior wood layers are protected. Eventually, the fire is unable to continue beyond the char layer if the member is big enough, and the fire self-extinguishes.

The structural engineer must account for the loss of the char layer in the design of the CLT floor if the design team wishes to expose the CLT unit in the final finished space. While Type X drywall can be used to protect structural elements from fire, one of the benefits of using CLT is the sense of atmosphere its finished surface can bring to a space. Therefore, it benefits the architect, engineer, and client to analyze the intrinsic fire-protection contributions of CLT.

Chapter 6 of the NDS and *Technical Report No. 10: Calculating the Fire Resistance of Exposed Wood Members* provide guidance for the design of fire resistance using the char depth method. Assuming the need for two hours of fire resistance, the effective char rate is 1.58" per hour or 3.2" of effective char depth for two hours of burning time. The remaining laminations are then checked to see what their cumulative reserve capacity is. When analyzing the remaining laminations, there are two approaches. Either all remaining laminations can be evaluated or only the laminations not exposed to the char depth can be checked. The later scenario is conservative and is shown below:

Table 3-1: Modified Section Properties of a 9 5/8"CLT Panel Exposed to 2 HR. of Charring

Layer Orientation	Lamination Depth	E of layer	Z (distance from center of layer to Neutral Axis)	$E*bh^3/12$	$E*A*z^2$	$(E*bh^3/12) + (E*A*z^2)$
	in	$Psi * 10^6$	In			
Parallel	1.375	1.7	2.0625	4.42	119.3	123.72
Perpendicular	1.375	0.04	0.6875	0.1	0.31	0.41
Parallel	1.375	1.7	0.6875	0.1	13.3	17.72
Perpendicular	1.375	0.04	2.0625	4.42	2.8	2.9
					EI_{EFF} $(Lb*in^2)/LF$	144.75

$$S_{EFF} = \frac{2EI_{EFF}}{E_{\parallel Layers}h} = 30.96 \text{ in}^3 \text{ for the 9 5/8" E4 CLT Deck}$$

$$F_b S_{EFF} = \frac{0.85 * F_b * S_{EFF}}{12'' \ per \ LF}$$ =4277 PLF-Ft for the 9 5/8” E4 CLT Deck

Finally the flexural strength for the modified section shall be amended such that it can resist the original flexural stress found in strength design so that:

$$M_b \leq (F_b S_{EFF})' = 2.85 \ C_F \ C_V C_{FU} \ C_L \ (F_b S_{EFF})$$

The selected CLT deck thickness met the charring design fire-resistance requirements.

3.2 Glulam Gravity Girders

3.2.1 Glulam Specifications and Design Process

The design of glulam girders supporting the CLT floor assumed that all girders were simply supported and the ASD load case of D + L controlled. In keeping with the assumptions made in the design of the CLT floor, only Southern Pine sizes and varieties of glulam were considered. The selected stress class was 24 F-1.8 E, specifically 24F-V3 for the Southern Pine species, with a Modulus of Elasticity of 1.8 x 10^6 psi for deflection calculations.

3.2.2 Strength Design

As used in the CLT floors, the dead and live loads were 35 PSF and 65 PSF, respectively. The tributary width of each beam was found by dividing the intersecting CLT panel length by 2. To decrease the structural depth, each beam only supports the CLT panels from one direction, not about both sides of the structural grid line. The two adjacent beams are connected together via the CLT-supporting steel saddle as explained later.

The flexural design of each beam is governed by:

$$f_b \leq \ F_b{}'$$

$$f_b = \frac{M_b}{S} \ and \ M_b = \frac{wl^2}{8} \ and \ S \ is \ the \ Section \ Modulus \ of \ the \ beam$$

$$F_b' = \ F_b C_D C_m C_T C_L C_v C_{fu} C_c C_i$$

Where: F_b= 2400 psi

C_D= 1.0 as the duration factor for the live load

C_L= 1.0 as the lateral stability factor assuming the beam is fully braced by the CLT panels

$C_v = \left(\frac{21}{L}\right)^{\frac{1}{x}} * \left(\frac{12}{d}\right)^{\frac{1}{x}} * \ \left(\frac{5.125}{b}\right)^{\frac{1}{x}}$ as the volume factor for every beam and x=20 for Southern Pine glulam members

3.2.3 Deflection Design

Designing for the control of deflections in glulam beams is very similar to that of CLT floor planks. The same three sets of deflection scenarios are applicable:

1. Short-term live load deflections
2. Short-term dead and live load deflections
3. Long-term deflections due to dead and live load, also known as creep

The controlling scenario is creep deflection. Creep deflection can be found as:

$$\Delta_T = K_{CR} * \Delta_{LT} + \Delta_{ST}$$

Where:

Δ_{ST} = Short-term deflections due to the effects of the live load

Δ_{LT} = Long-term deflections due to the effects of the dead load

K_{CR} = 2.0 and is a modifier to account for the continued presence of live loads over time and assumes that the CLT is used in dry service conditions

The deflection can be found using:

$$\Delta = \frac{5}{384}\frac{wl^4}{EI}$$

Note that the deflection of glulam beams is not dependent on the shear resistance characteristics of the member. Initially, only the L/240 criteria was checked using the centerline to centerline span dimension of each girder. All beams passed easily. While L/240 is typically the value used for floor systems, L/360 was checked to be consistent with the criteria of the CLT panels. Even though the girders are stiffer than the CLT panels, both criteria were checked. An actual girder length of the centerline to centerline span minus the column width of 10.5” was used. Almost all girders passed the criteria, and those that did not were within 10%.

The deflections showed in the glulam girder calculations in the appendix show the L/360 deflection check.

3.2.4 Fire Protection Design of Gravity Girders

The minimum distance between the bottom of the beam and the bottom of the CLT floor panel will be 5.5." Therefore, about 1/3 of the beam will be exposed to fire from three sides rather than from just the bottom edge as was the case in the CLT deck. Since the intended level of passive fire protection will be 2 hours, 3.2" of char depth is required at all edges exposed to fire. Removing 3.2" from each side of the beam in addition to 3.2" of depth at the tensile face will reduce the flexural capacity to the point of failure. Furthermore, as represented below by the dashed green profile, a steel saddle will be required to support the CLT decks and provide a connection to the beam.

In order to provide adequate fire protection to the beam, a sacrificial layer of wood 6" wide will be provided to cover both the steel saddle and sides of the beam. Figure 3-3 shows this layer in blue highlight. The height of the member shall be such that the entire side of the beam from the bottom face to bottom of the deck is protected.

With the member only being exposed to fire from the bottom face now, the same process can be used as was used in the CLT floor fire protection. The section modulus of the beam is adjusted for the reduction of 3.2" of depth and the allowable flexural stress capacity is adjusted by 2.85. All beams met the fire protection requirements.

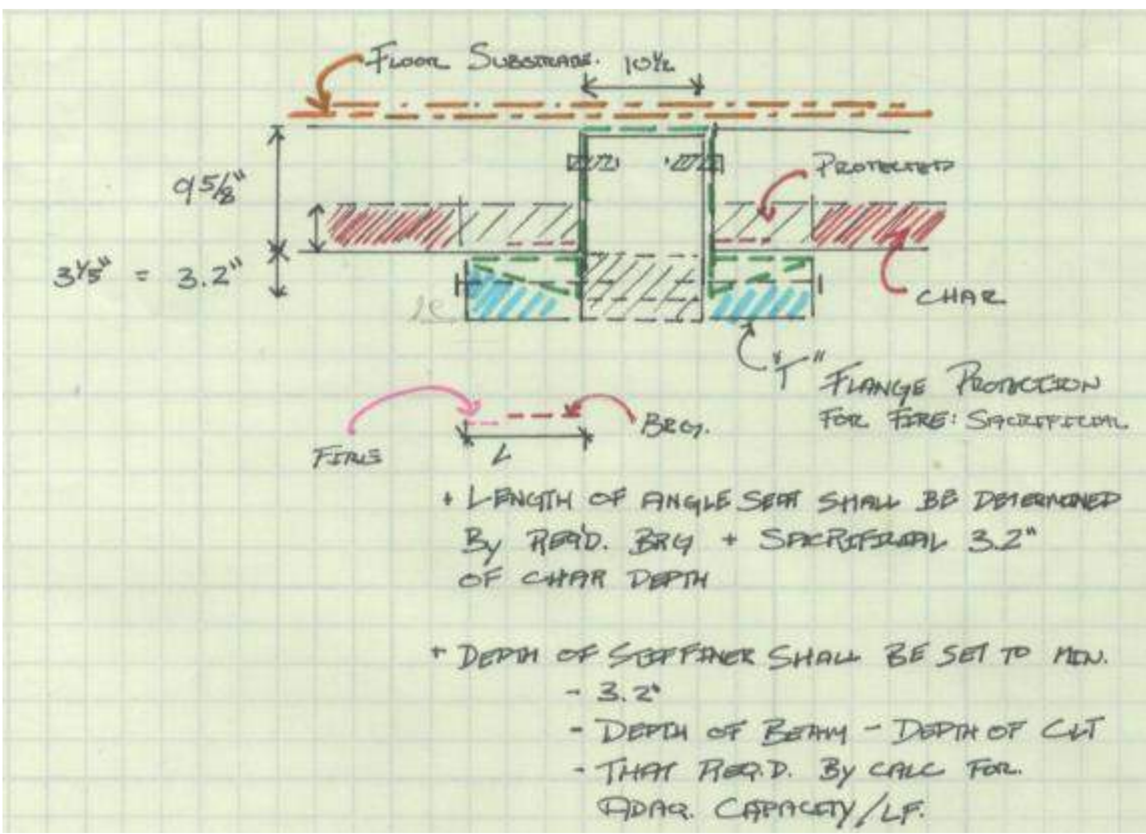


Figure 3-3: Passive Fire Protection Measures at the Glulam Girders and CLT Saddle Connection

3.3 Gravity System Connections

3.3.1 Connection Design Overview

The design of connections between elements, whether they are wood-to-wood, wood-to-steel, or wood-to-concrete, is governed by chapters 11 and 12 of the NDS. The preferred method of analysis and design is to look at the respective plane of failure and determine the controlling failure mode between the wood and dowel connection. The connections for this project utilized dowel connections of two types:

1. ¼" dia. Lag screws
2. 1" dia. A325 Steel bolts

There are three types of connections between gravity system elements. They are:

1. Between the CLT floor panels transverse to the orientation of the supporting gravity girders
2. Between the CLT panel, supporting steel saddle, and glulam gravity girder
3. The steel plate embedded into the girder providing a shear connection

3.3.2 CLT Floor Spline Connections

The interface between CLT panels provides continuity in the floor in the lateral load transfer mechanisms in the floor diaphragm. Like most floors in wood construction, a plywood flooring substrate material can be placed over the exposed face of the CLT panels in preparation for the final flooring material, i.e. carpet. The substrate helps provide some continuity, but the topic of lateral force transfer must still be addressed.

Brenerman presents several different connection styles in CLT systems. The typical details for the connection between panel interfaces recycles the concept of a wood "spline" from those typically used in Structural Insulated Panel (SIP) construction. The spline is composed of a wood material typically similar in species and properties to that of the CLT. In order to control differential movement due to shrinkage and temperature, it is recommended that the spline be made of an engineered wood product, preferably of the same species and final finish as the CLT. With regard to the LiUNA project, the spline should be of a southern pine dimensional lumber product.

The spline interface is what helps transfer the lateral load through the floor diaphragm to the respective lateral elements. While this issue is more-so related to the topic of lateral loads, it is addressed in this gravity section for the purpose of keeping all CLT information together. Brenerman's examples can be seen on the next page in Figure 3-4.

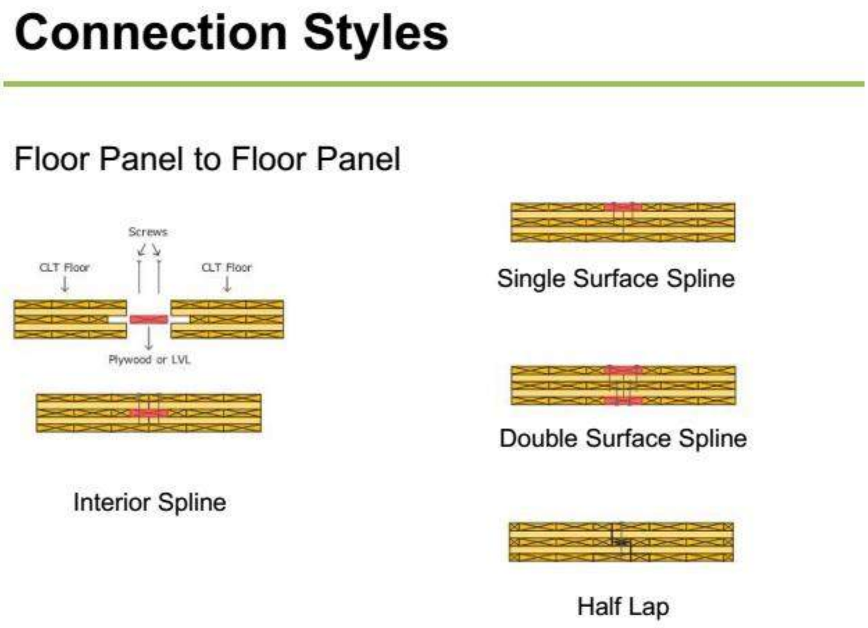


Figure 3-4: CLT Connection Styles Utilizing a Spline as Presented by Breneman

The story force in the East-West direction at the 2nd Floor is 90.5 K. The CLT floor panels run 56.25' parallel to the direction of the lateral force application. The lateral force is distributed to the CLT panels in the same manner as the lateral force is distributed to the five moment frames based upon tributary widths using the flexible diaphragm assumption. At moment frame 4 (MF-4), 21.8K are applied over 56.25' resulting in a load of 388 PLF. A 0.164" dia. wood screw has a capacity of 101 pounds per screw for load perpendicular to grain where the side member (in this case the spline) has a width of 1.5." These screws would be spaced a 4" o.c. so that there is a total capacity of 404 PLF along the spline detail shown in Figure 3-5. The wood screw capacity can be found in Table 12L, pg. 107 of the NDS for Southern Pine G=0.55. In order for the capacity of the screw to not be affected by geometry, the typical spacing rules prescribed in chapter 11 are followed resulting in the typical detail seen bellow. This detail is typical throughout the project at all floor locations. While conservative in many locations, this detail is typical for the CLT industry and is designed to reduce the opportunity for error with a construction crew that will be unfamiliar with CLT construction.

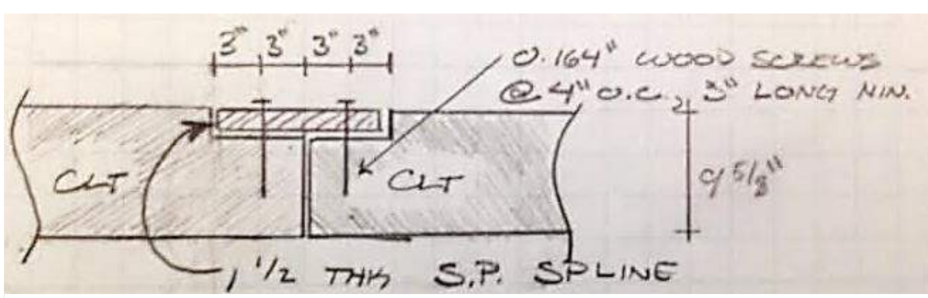


Figure 3-5 CLT Spline Connection

3.3.3 Floor Deck to Gravity Girder Steel Saddle

In order to reduce the structural depth, a steel saddle was designed so that the top of the CLT deck could sit flush with the top of the girder. Due to the sizes of the wood members involved, the saddle will have to be fabricated using welded A36 steel plates. A36 steel is recommended by the NDS because steel limit states rarely control the failure of wood-to-steel connections.

When designing the saddle connections, two approaches need to be used depending upon the component being analyzed. For all wood-to-wood or wood-to-steel connections, ASD design must be used to be consistent with the allowable load per connection tables provided in Chapter 12 of the NDS. For the steel limit states, LRFD methodology must be used.

Using the loads of 35 PSF dead and 65 PSF live over a 1 foot-width and span of 21.25,' the maximum load seen by the saddle is 1551 PLF. The steel limit states of angle shear yield, seat angle flexure, and weld rupture were analyzed; a thickness of ¼" was found to provide beyond necessary reserve capacity. ¼" is also one of the default thickness for which dowel connection capacities are provided in the NDS. Weld thickness between the plates is controlled by the thickness of the plate. The typical connection detail is provided in Figure 3-6 is for the instance where the beam beneath the saddle is (1) 10.5" wide beam. At instances where there are (2) 10.5" wide beams adjacent to one another, as is typical throughout many locations, the width of the top plate is adjusted.

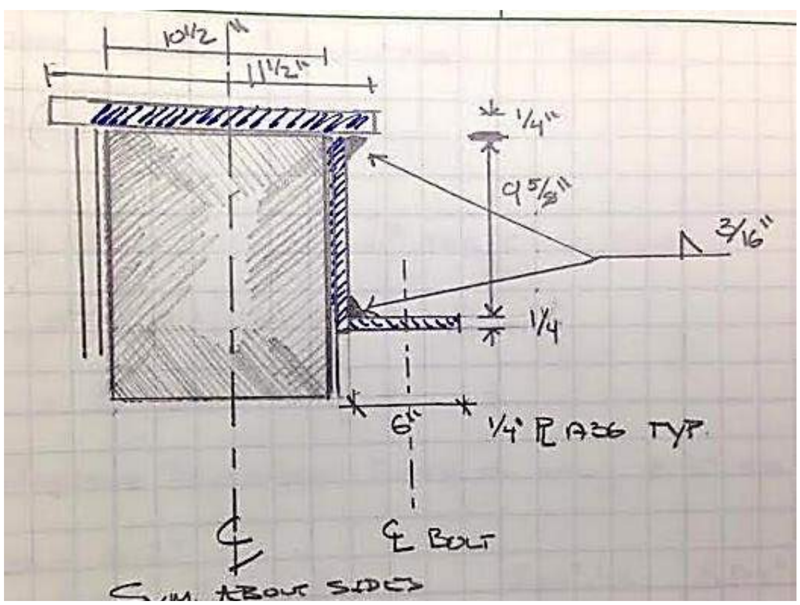


Figure 3-6: Typical Saddle Connection at CLT to Gravity Girder Interface

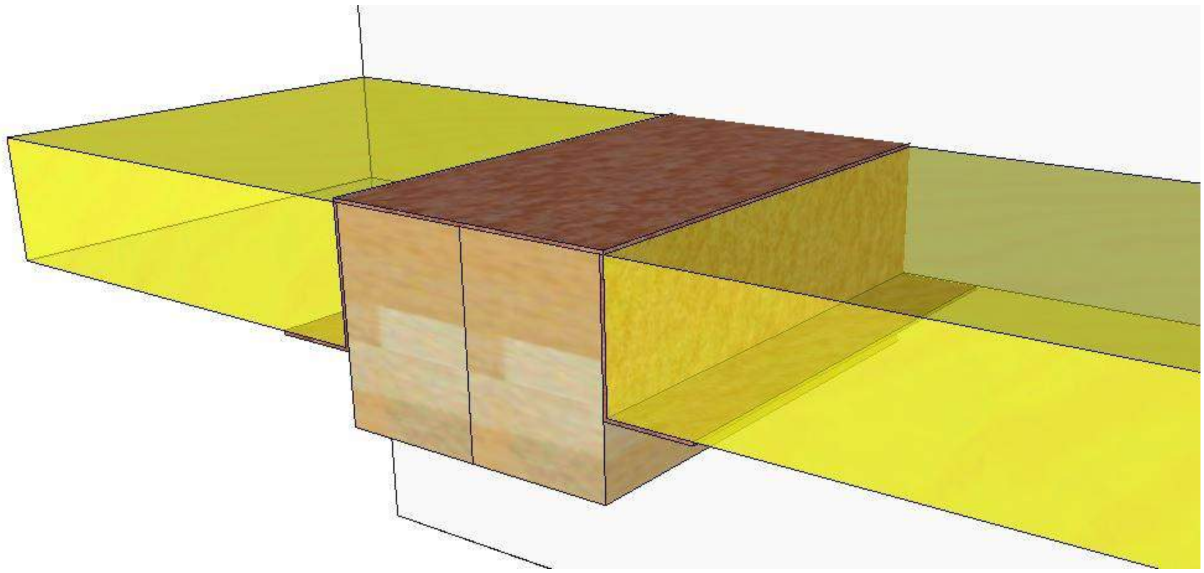


Figure 3-7: Steel Saddle Connection at Interface of Two Adjacent Beams and CLT Deck (Shown in Translucent Yellow)

Just as the CLT-to-CLT spline transfered lateral loads throughout the floor diaphragm, so too, the steel saddle connection is required to transfer North-South lateral loads. Lateral loads are transferred via the path of:

1. CLT to fastener
2. Fastener to shear against the steel plate
3. Steel plate to fastener in the glulam
4. Fastener to the glulam itself.

The story force of 65.7 K was distributed through the second floor along column lines 1, 2, 3*, and 5 according to the tributary width of each frame along the transverse span of 85.33.' The controlling load was 143 PLF. ¼" dia. lag screws have a perpendicular to grain shear strength of 160 lbs in a wood-to-steel interface; (1) ¼" dia. screw is required every 12" o.c.

The decision to pick lag screws as opposed to wood screws was multifaceted. The dowel-type fastener tables in chapter 12 list capacities for wood screws in shear with ASTM 653 Grade 33 steel side plate stock, not the A36 steel plate stock specified throughout the NDS. Lag screws do have shear capacities listed for A36. Therefore, lag screws were more compatible with the steel stock specified. Wood screws are typical wood-to-wood connections and eliminate the need for pre-drilling. Lag screws are more typical for steel-to-wood connections and do require pre-drilling. The difference in fastener types does provide a convenient visual cue for inspection to see if the correct screw was applied in the correct location. The requirement for pre-drilling should help reduce opportunities for using the wrong screw in the wrong location.

On the following page, the typical spacing detail is provided.

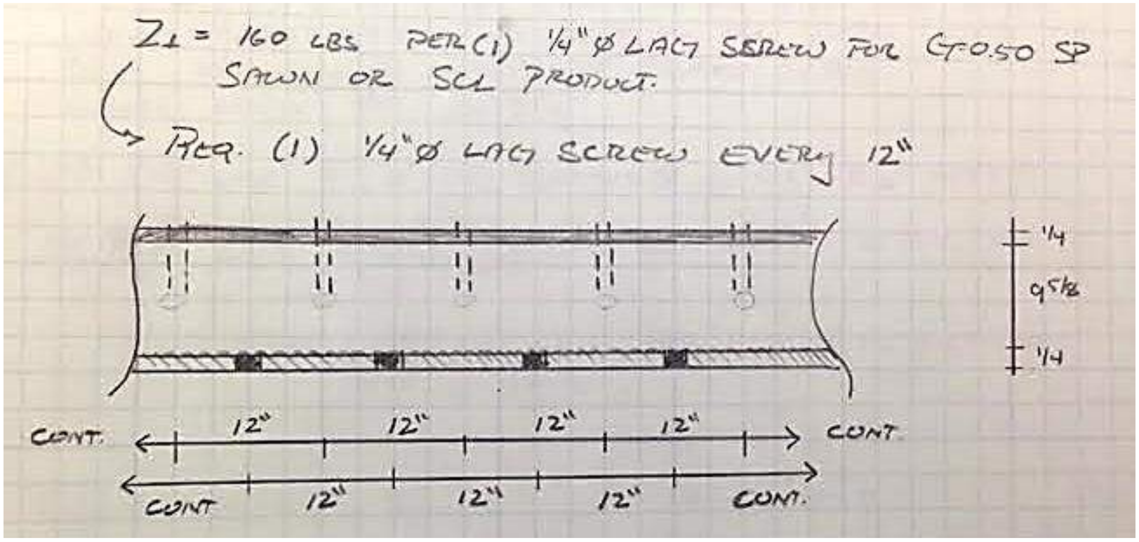


Figure 3-8: Spacing of 1/4" dia. Lag Screws at Steel Saddle Interface, Typical Detail

3.3.4 Gravity Beam to Column Saddle Knife Plate Connection

In heavy timber construction, there are several different ways to create a shear connection between the interface of a beam or girder and the column. In the LiUNA project, a connection type called a “knife plate” was selected for its intrinsic fire-protection properties. In a knife plate, the steel is embedded into the heart of the member to hide the steel, expose the wood, and utilize the wood as protection for the steel in the case of fire. A typical knife plate connection from the Sketch Up model is shown in Figure 3-9.

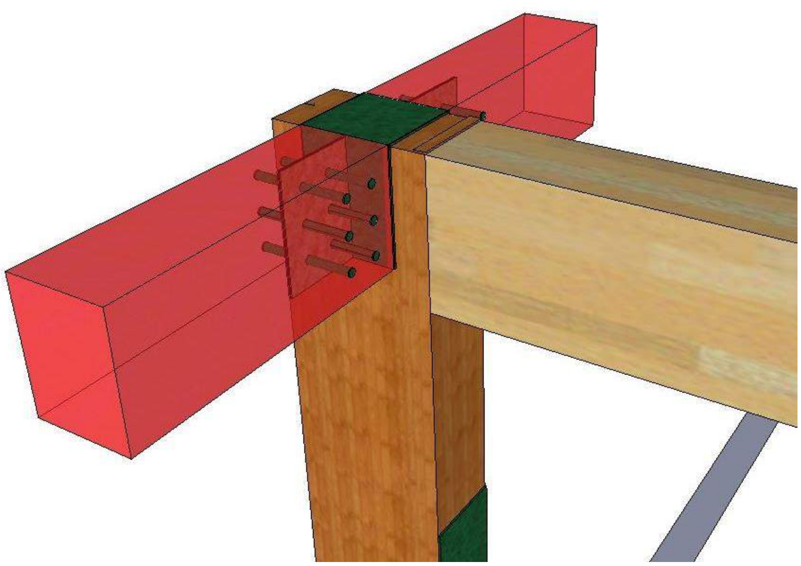


Figure 3-9: Steel Knife Plate with Gravity Girders Shown in Red, Additional Moment Connection Details Eliminated For Clarity

Just as was the case with the steel saddle, the wood components had to be designed via ASD loads while the connection was checked for steel limit states with an LRFD approach. The NDS design standards controlled the design.

While the NDS gives dowel-type fastener connection capacities for typical connections, the scenario presented by steel knife plates is not covered. That is because the steel plate is the "main member" while the wood on either side of the steel composes the "side members." In order to determine the connection strength, yield modes I_M, I_S, III_S, and IV_S need to be checked. For a ¾" dia. A325 steel bolt, a capacity of 2405 pounds per fastener was found for load perpendicular to grain. Applying all load reduction factors, $Z'_{perpindcualr}$ = 2381 pounds per bolt.

The selection of ¾" bolt was made for dimensional purposes. A 1" bolt would have required a larger edge distance (3" for the ¾" bolt and 4" for the 1" bolt). To reduce steel, the smaller bolt was selected. If the steel fabricator, wood erection team, or any other member of the design team would have wanted to use a consistent 1" dia. bolt at all connections gravity and lateral, this change could easily be made. For the ease of the fabricator, both possible sets of knife plate configurations are shown below.

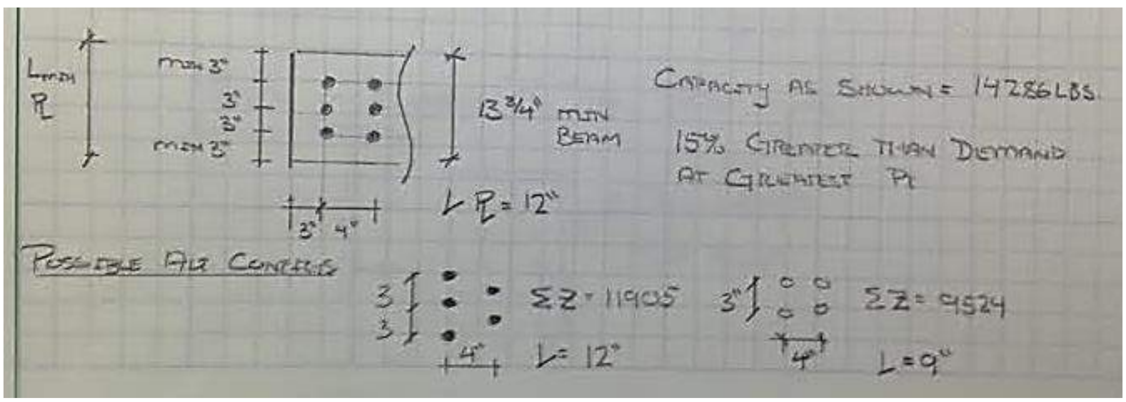


Figure 3-5: Knife Plate Detail with 3/4" dia., 6 Bolt Configuration Typical

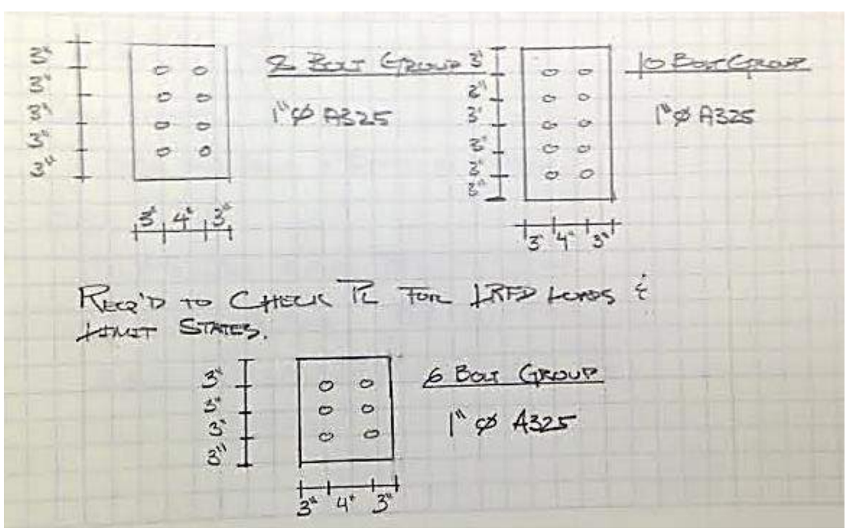


Figure 3-11: Knife Plate Detail with 1" dia. Bolt Configurations

Chapter 4: Structural Depth: Lateral System

4.1 Moment Frame Strength Design

4.1.1 Strength Design Overview

In the original design, four steel moment frames provided lateral resistance in the East-West direction. In the redesign, a moment frame was the only possible solution for lateral resistance if the original architectural intent was to be honored. The floor plan needed to remain open and continuous curtainwall was to remain as the façade system. In order to preserve the original architectural intent and the desire to showcase the potential of wood design, wood moment frames were required.

There were three major design challenges with the wood moment frames:

1. The columns had to be designed as beam-columns due to the presence of both axial and flexural stresses. The combined loading meant that the modest column sizes of 10”x10” and 12”x12” found in other tall-wood projects would not be possible.

2. The design of a moment frame hinged upon the successful design of a moment connection in wood. The majority of the initial research endeavors were devoted to this topic. Of the ideas that were found, only a few provided guidance on how to perform the design and were promising. The eventual design combined an analysis approach used in portal frame analysis and a design methodology consistent with that used for bolted-bolted moment connections in steel.

3. Due to the low modulus of elasticity, wood is unable to provide the stiffness required. While all members provided capacity greater than the design stresses, additional stiffness was required. In order to solve this issue, steel plate stock was added to the exterior flanges of the column members. The analysis of this design for conformance to deflection criteria was performed using a hand-calculation approach taught in AE 530.

The final design would be controlled by moment frame four, MF-4, at the interior condition and moment frame five, MF-5, at the exterior condition. MF-4 is at column line F*, and MF-5 is at column line N.B. These two locations controlled because they had the greatest tributary width for the interior and exterior frames, respectively. In order to determine design loads, the following ASD load combinations were run on moment frames four and five in SAP 2000.

- D + 0.6W
- D + 0.75 L + 0.45 W + 0.75 S
- 0.6D + 0.6 W

After finding design loads, excel spreadsheets were used to size all members in lieu of having access to wood design software.

4.1.2 Column Design

Of the three lateral load cases, D + 0.75L + 0.45 controlled the design of the column elements. Using the design axial and flexural loads, an initial column size could be selected knowing the ratios between the allowable bending and compressive stresses and those calculated on the selected section. 49 N1M visually graded Southern Pine was selected with an F_c = 2100 psi, an F_b = 1800 psi, and an E_{min} = 900,000 psi. The design of compression members with flexural load about the strong axis is governed by the NDS equation:

$$\left(\frac{f_c}{F_c{}'}\right)^2 + \frac{f_{b1}}{F'_{b1} * [1 - \left(\frac{f_c}{F_{cE1}}\right)]} \leq 1.0$$

Where:

$$f_b = \frac{M}{S}$$

$$f_c = \frac{P}{A}$$

The effective compressive capacity can be found using the relationships of stiffness and buckling such that:

$$F'_C = F_c{}^* C_P = F_c C_D C_M C_t C_P;\ C_D = 1.6\ for\ wind\ loads$$

The column stability factor, C_P above, can be found as:

$$C_P = \frac{1 + \left(\frac{F_{CE}}{F_c{}^*}\right)}{2c} - \sqrt{\left[\frac{1 + \left(\frac{F_{CE}}{F_c{}^*}\right)}{2c}\right]^2 - \frac{\left(\frac{F_{CE}}{F_c{}^*}\right)}{c}}$$

Where the critical buckling design values are a relationship of:

$$k = 0.5\ for\ fixed - fixed\ moment\ connections$$

$$l_e = k_e l\ where\ "l"\ is\ the\ unbraced\ length\ in\ the\ axis\ considered$$

The column slenderness ratios:

$$\frac{l_{e2}}{d_2} \leq 50\ where\ d_2\ is\ the\ dimensions\ about\ weak\ axis\ bending$$

$$\frac{l_{e1}}{d_1} \leq 50\ where\ d_1\ is\ the\ dimension\ about\ strong\ axis\ bending$$

Such that:

$$F_{CE2} = \frac{0.822\ E'_{ymin}}{\left(\frac{l_{e2}}{d_2}\right)^2}$$

$$F_{CE1} = \frac{0.822\ E'_{xmin}}{\left(\frac{l_{e1}}{d_1}\right)^2}$$

The smaller critical buckling value controls the design and should be used to calculate the column stability.

The effective bending capacity can be found such that:

$$F'_{bx} = F^*_{bx}\ C_v + f_c \ \leq\ F^*_{bx}\ C_L$$

$$F'_{bx} = F_{bx}\ C_D C_M C_t C_v + f_c \ \leq\ F_{bx}\ C_D C_M C_t C_L$$

The beam stability factor, C_L above, can be found as:

$$C_L = \frac{1 + \left(\frac{F_{bE}}{F_b{}^*}\right)}{2c} - \sqrt{\left[\frac{1 + \left(\frac{F_{bE}}{F_b{}^*}\right)}{2c}\right]^2 - \frac{\left(\frac{F_{bE}}{F_b{}^*}\right)}{c}}$$

Where the beam stability factor is a relationship of:

$$l_e = 1.84l\ because\ {}^{l_u}\!/_{d} > 14.3$$

The beam slenderness ratio:

$$R_B = \sqrt{\frac{l_{e2}d}{b^2}} \leq 50$$

Such that:

$$F_{bE} = \frac{1.20\ E'_{ymin}}{R_B{}^2}$$

The volume factor, C_V above, can be found as:

$$C_v = \left(\frac{21}{L}\right)^{\frac{1}{x}} * \left(\frac{12}{d}\right)^{\frac{1}{x}} * \left(\frac{5.125}{b}\right)^{\frac{1}{x}}$$ x=20 for Southern Pine glulam

Please see the final design drawings at the end of this chapter for the final determination of all member sizes for both moment frames.

4.1.3 Beam Design

After using SAP to analyze the moments found in all of the members, the flexural demands were compared to the capacity provided by a variety of different Southern Pine glulam sizes assuming the same species used in the gravity girders (24 F-1.8 E). Beams ranging in sizes from 6.75” x 6.875” to 10.5” x 11” were considered.

The deciding factor for the size of beam selected was compatibility at the moment connection between the width of the column, the width of the steel plates, and the width of the beam. Since the columns had to be 10.5” wide to reduce the depth of each column (and consequential impact on the floor plan), the steel plate had to also be 10.5” wide. The maximization of the steel plate’s width helped provide additional lateral stiffness, a topic to be discussed later. Since the plate stock at the beam and column had to be the same to reduce the possibility of stress concentrations at a transition point between plates, the beam had to have a width greater than or equal to the width of the plate. Therefore, all of the beams used in the moment frames had to be at least 10.5” x 11,” the minimum depth offered with a 10.5” width.

4.1.4 Bolted Moment Frame Connection Design

Using SAP, the shears were found at every connection for the 4 members in every joint as shown in Figure 4-1.

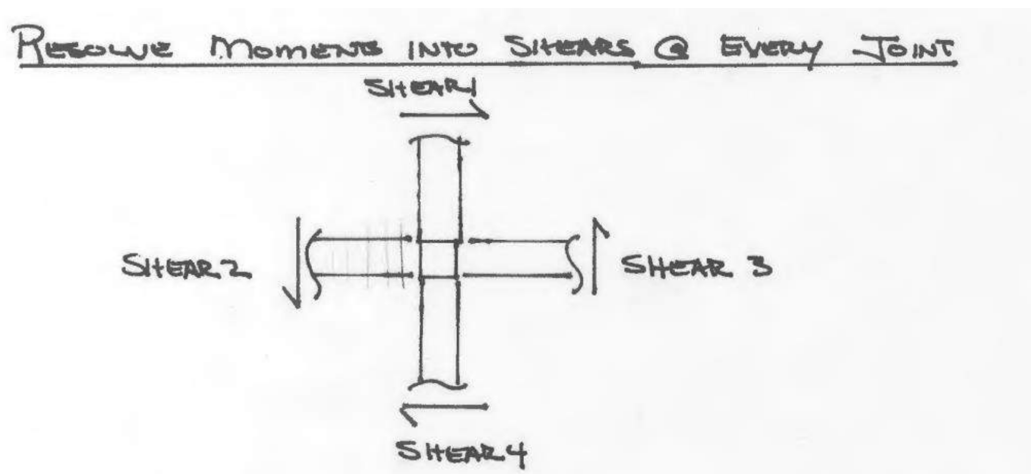


Figure 4-1: Resolution of Moments into Shears at Every Member at a Rigid Joint

This approach is consistent with two proven methodologies used in industry. First, in the analysis of lateral frames, the portal method says that the moment at a joint is equal to the shear at that location multiplied by the distance from the joint to the plastic hinge. This concept is shown in Dr. Boothby's *AE 308, Fundamentals of Structural Analysis* and is replicated in Figures 4-2 and 4-3:

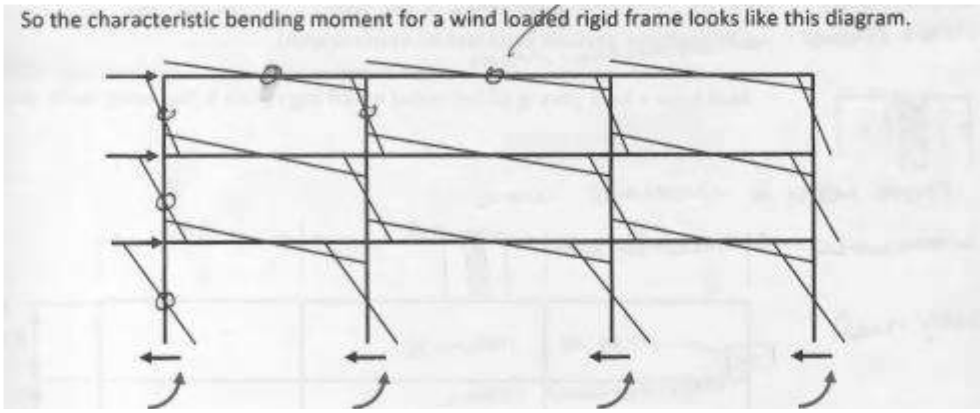


Figure 4-2: A Rigid Frame Laterally Loaded

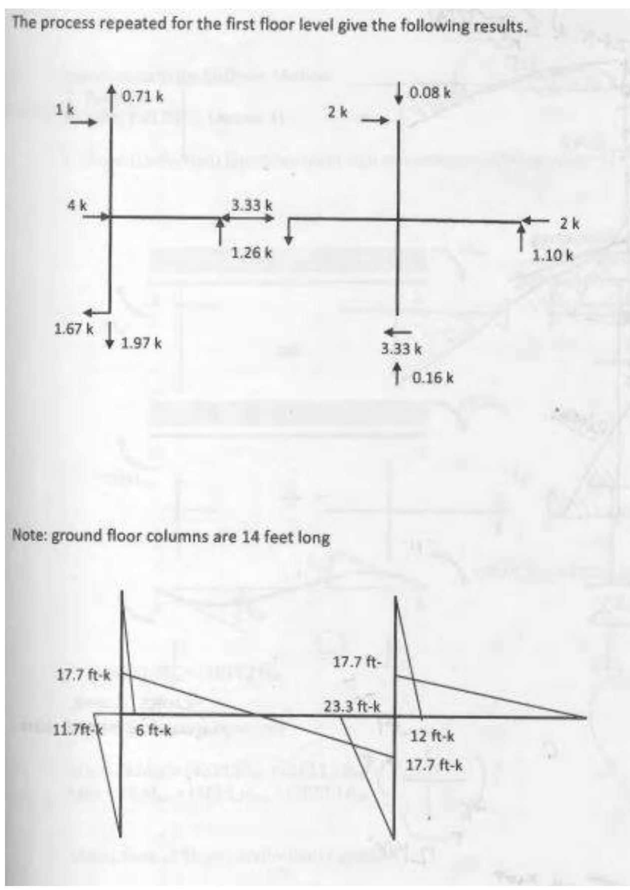


Figure 4-3: Resolution of Joint Shears into Moments in a Rigid Frame Laterally Loaded

Secondly, when designing bolted-bolted moment connections, moment is resisted at a joint via:

1. Resisting the shear between the girder web and column flange
2. Resisting the tension and compression in the flange plates; these forces are equal to the traverse shear seen by the columns at the connection.

In this area, Dr. Hanagan's *Design and Analysis of Steel Connections Lecture Notes: AE 534* proved to be very helpful.

As shown in figure 4-1 above, the reactions at each joint were found. Assuming 1" dia. A325 bolts with ¼" A36 steel plates in double shear, each fastener in the connection had a shear capacity of 3180 pounds perpendicular to grain. The number of bolts required in the flange plate for each member could be found as:

$$Number\ of\ bolts\ required = \frac{Traverse\ Shear\ at\ the\ Member}{Z_{\perp} C_G C_D C_{\Delta}}$$

Where:

$$Z_{\perp} = 3180\ lbs\ per\ bolt\ per\ the\ double\ shear\ dowel - type\ fastener\ tables$$
$$C_G = 0.9\ as\ the\ Group\ Factor, Conservatively\ Used\ from\ Table\ 11.3.6c$$
$$C_D = 1.0\ as\ the\ Duration\ Factor, used\ as\ 1.0\ to\ be\ conservative$$
$$C_{\Delta} = 1.0\ as\ the\ Diaphragm\ Factor, 3\text{" edge distances \& 4"}\ btwn.\ bolts\ used$$

In the expression above, the duration factor did not take advantage of the 60% increase in capacity provided to components controlled by wind load. This decision was made in light of the lack of research of this connection type. Further research is required to see if the predicted behavior can be replicated in a laboratory environment and what adjustments, if any, are required to the predicted bolt capacity.

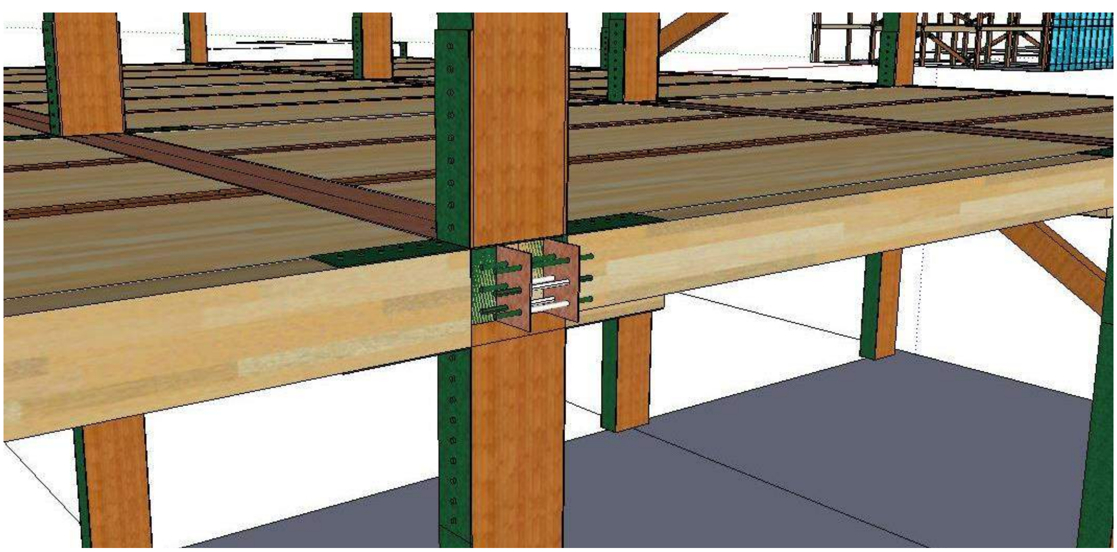


Figure 4-4: Moment Connection at MF-4, Second Floor, N.2-F*

4.1.5 Moment Frame Design

Please see Appenix A, Construction Documents, for the design of both moment frames.

Drawings related to the botled-bolted moment conneciton can be found after the drawings of the entire structure.

4.2 Moment Frame Serviceability Design

4.2.1 Serviceability Design Overview

As mentioned earlier, the last issue with the design of the moment frames was providing adequate stiffness to resist lateral deflections. When the strength-controlled design was run in SAP, the stiffness was in excess of the H/400 limit of 3.21." In order to provide additional stiffness, steel plates were attached at the outer flanges of the columns.

4.2.2 Analysis of Original Design

Before revising the original design, basic calculations were performed to confirm the unexpected behavior seen in the model. Using an approach from AE 530, Computer Modeling of Building Structures, the story stiffness was calculated to be:

$$\Sigma K_{Story} = (2)K_{End\ Columns} + (2)K_{End\ Columns}$$

Where:

$$K_{End\ Columns} = \frac{12EI_C}{h^3}\left[\frac{1}{1+\frac{I_C}{h\left(\frac{I_{b1}}{b}\right)}}\right]$$

$$K_{Middle\ Columns} = \frac{12EI_C}{h^3}\left[\frac{1}{1+\frac{I_{C2}}{h\left(\frac{I_{b1}}{b_1}+\frac{I_{b2}}{b_2}\right)}}\right]$$

The resulting maximum story drift is 12.12" at the roof. This result was unacceptable and resulted in the use of steel plate to increase the stiffness of the column elements. The total drift at a story was found via:

$$\Sigma\ Drift\ @\ Story = \frac{\Sigma Story\ Shear}{K_{Story}} + Drift\ Below$$

and

$$Individual\ Story\ Drift = \frac{Wind\ Story\ Force}{K_{Story}}$$

4.2.3 Analysis of Revised Design Including the Contribution of Steel Plates

When it was found that the stiffness of the frame was inadequate, two options were available.

A composite structure using either steel or concrete moment frames could have been used. Steel was what was used in the original design and investigated in the fall semester. For the purposes of this project, it would have been preferable to investigate another system. A concrete moment frame would have slowed down the construction schedule and eliminated the time savings provided by a speedy timber erection process.

The contributions of the existing flange plates in the moment connections could be considered. In order to provide continuous stiffness throughout the member, thereby making analysis easier, the steel plates were extended along the flanges of the column. These built-up column sections could be analyzed using the cumulative moment of inertia of both the wood and steel pieces.

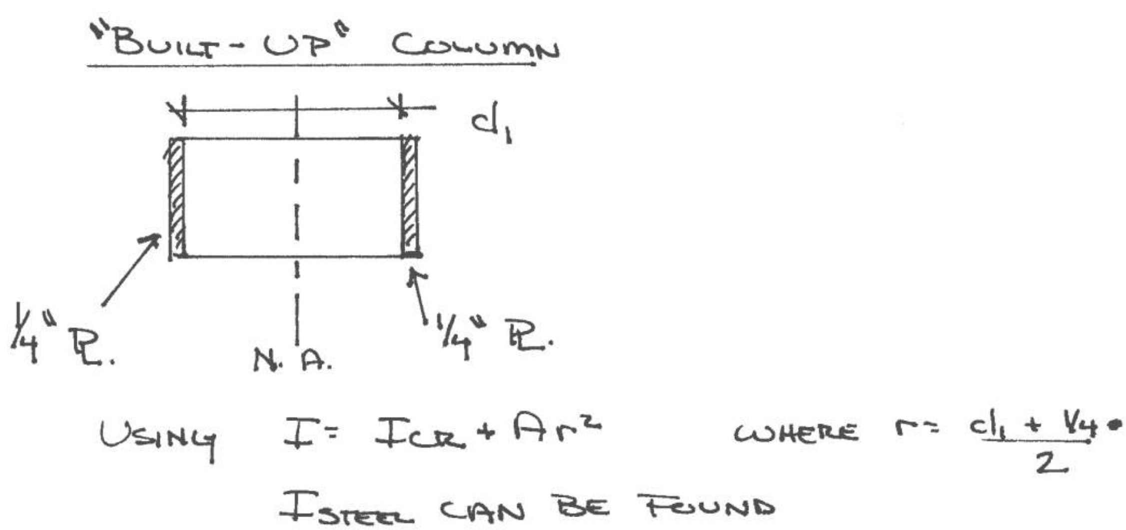


Figure 4-5: Built-Up Column Section and Corresponding Moment of Inertias

When calculating the story stiffness using the approach outlined earlier, the contributions of the steel and wood as they relate to the geometry of the frame and intersecting beams were assessed separately. Upon finding the stiffness of the wood column and steel flanges, their total stiffness was found for each built-up column and for the entire floor. The maximum drift at the roof was found to be 2.39," or 25.5% less than the 3.21 maximum. The analysis of the moment frames was completed using hand calculations due to limitations in SAP's ability to analyze a composite section of wood and steel. While efforts were made to build custom built-up columns in SAP, the software failed to recognize the contributions of the steel. Typically, in industry, RISA 3D software is used for structural engineering in wood. Since a RISA 3D software package was unavailable, the decision was made to keep all engineering analysis and design in Excel for consistency.

4.3 Vertical Truss Design

4.3.1 Overview of Vertical Truss Strength Design

Upon completion of the two controlling moment frames, the vertical truss was designed using the section properties of the beams and columns already designed. Using SAP 2000 to find the resultant controlling loads for all elements, the beam and column selections from the gravity girder and moment frame design processes, respectively, were checked. The beams previously designated as gravity girders were found to be controlled by the combination of lateral load and the flexural demand of the CLT deck. Therefore, these beams previously designed as flexural members were actually beam-column elements and required the same strong-axis bending combined loading approach used to size the moment frame beams. The beam designs were updated as can be seen in the final vertical truss design at the end of this section. The columns were checked for bending about the weak axis; due to the contributions of the bracing elements, the columns did not need to be resized.

Following the checks on the beams and columns, the bracing elements had to be designed. There were three main components to the bracing design:

1. The actual compression bracing had to be designed as column elements subject to buckling due to their long unbraced length. Bracing elements ranged in width from 5 1/8" to 8 ½" in an attempt to minimize the amount of timber used.

2. The bracing connections had to be designed using the shear parallel to grain capacity of a 1" dia. bolt in a knife plate configuration. The knife plate design was also used in the bracing connection so that the steel would be protected in fire and for aesthetic reasons.

3. The complete truss design had to be checked for serviceability with drift criteria.

The following three sections discuss the details of these components of the design.

4.3.2 Compression Bracing Design

In designing the vertical truss bracing, both the compression and tension failure modes were checked. As expected, the compressive failure was the controlling failure mode. Since the bracing elements were not subject to additional flexural loads, they could be designed according to the equations for slender columns where:

$$F_C' = F_C{}^* C_P = F_c C_D C_M C_t C_P; \; C_D = 1.6 \, for \; wind \; loads$$

The critical buckling value is again determined as a relationship of the minimum slenderness ratio where:

$$F_{CE1} = \frac{0.822 \, E'_{xmin}}{\left(\frac{l_{e1}}{d_1}\right)^2}$$

Such that the column stability factor can then be found to be:

$$C_P = \frac{1 + \left(\frac{F_{CE}}{F_c{}^*}\right)}{2c} - \sqrt{\left[\frac{1 + \left(\frac{F_{CE}}{F_c{}^*}\right)}{2c}\right]^2 - \frac{\left(\frac{F_{CE}}{F_c{}^*}\right)}{c}}$$

The complete calculations for the design of the bracing elements can be found in Appendix K.

4.3.3 Bracing Connection Design

In the same manner as the knife plate design of the gravity girders, the bolt capacity was found as a relationship to the bolt diameter and grain orientation. Since the size issues were not as tight as they were in the gravity girder knife plates, 1” diameter bolts were used. The increase in bolt size meant that an increase of 3” to 4” on-center between bolts in the bolt group was required to have a group geometry factor of 1.0.

The calculation of group geometry factors proved to be more tedious. Since the group factor is determined by a relationship between the width of the wood member, the number of bolts in a row, the diameter of the bolt, and the material properties of the steel and wood, the group factor had to be calculated for every variance in member width and number of bolts in a row. The group factors ranged from 0.91 for 6 bolts in a row in a 6 ¾” width beam to 0.97 for 4 bolts in a row in a 6 ¾” width beam or 5 and 6 bolts in a row in an 8 ½” width beam.

Since the compressive force works through the longitudinal axis of the member, the desired capacity was shear parallel to grain or $Z_{parallel}$. Found to be 3084 pounds per bolt, the number of bolts required could then be found as:

$$Number\ of\ bolts\ required = \frac{Compressive\ Force\ at\ the\ Member}{Z_{\parallel} C_G C_D C_{\Delta}}$$

Where:

$Z_{\perp} = 3084\ bs\ per\ bolt$
$C_G = 0.91 - 0.97\ as\ the\ Group\ Factor$
$C_D = 1.6\ as\ the\ Duration\ Factor, 1.6\ used\ for\ wind$
$C_{\Delta} = 1.0\ as\ the\ Diaphragm\ Factor,$ 3" edge distances & 4" $btwn.bolts\ used$

The resultant number of bolts required for every brace can be seen in the Appendix. Figure 4-6 is a sample view of the bracing connection at the second floor.

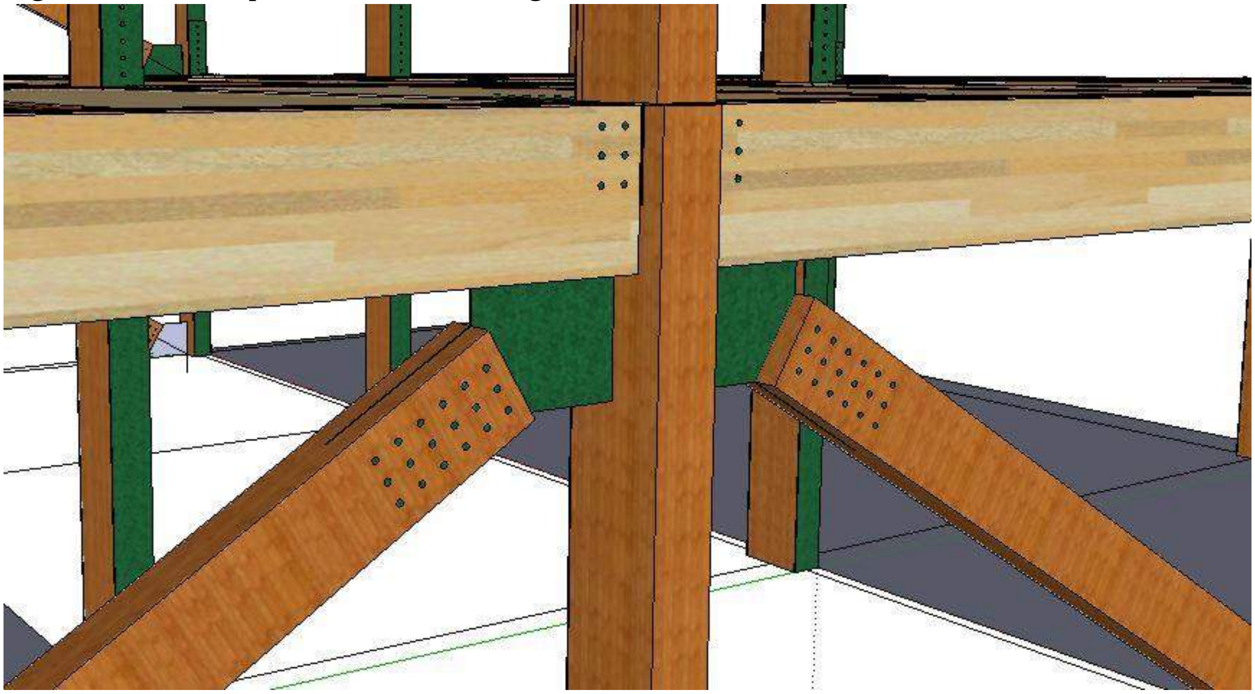


Figure 4-6: Vertical Truss Bracing Connection at the Second Floor Framing

4.3.4 Serviceability Design

The drift limitation of H/400, or 3.21," was applied to the vertical truss design. After running the three applicable wind load cases, load case D+0.6W was found to control with an overall building drift of 1.69." This result was only 39% of the maximum allowed drift, thereby indicating that there is a significant amount of additional ability to resist drift. Furthermore, the design of the vertical truss elements was determined by member yielding, not by lack of stability as was the case with the moment frame.

Chapter 5: Mechanical Breath

5.1 Fire Protection Requirements and Sprinkler Design

5.1.1 Code-Based Fire Protection Provisions

As currently designed, the LiUNA Expansion Building is not compliant with the International Building Code, 2015 Ed. The focus of this project however, was to use wood as the primary structural material and then explore strategies that would allow the building to meet the intent of the building code. In order to meet the spirit of the fire code requirements, a combination of passive and mechanical fire protection measures were designed. Through the application of this joint approach, the design could then be in evaluated by the opinion of the authority having jurisdiction for compliance with the performance requirements of the IBC.

Currently, the LiUNA Expansion Building would fall under the occupancy classification of "business." According to Table 504.3 "Allowable Building Height in Feet above Grade Plane," the LiUNA Expandion Building at 107' tall would require Type I A or B construction with or without sprinklers. The same result is obtained upon analyzing Table 504.4 "Allowable Number of Stories above Grade Plane." Type I B construction requires that the primary structural frame has a fire-resistance rating of 2 hours.

The proposed structural redesign of the LiUNA Expansion Building does not classify as Type I construction. All structural timber construction is classified as Type IV construction or "Heavy Timber." Type IV construction has rapidly expanded recently to include CLT. While the heavy timber classification recognizes the ability of thicker timber elements to char under fire and provide intrinsic fire protection, there are limitations on the potential for how big a Heavy Timber building can be. Table 504.3 specifies that for a business occupancy, Type IV construction can only be 65' above grade if the building is not sprinkled or 85' above grade if the building is sprinkled. Table 504.4 states that for a business occupancy, Type IV construction is limited to five stories if not sprinkled or 6 stories if sprinkled.

The LiUNA Expansion Building is 107' tall and has eight occupied stories above grade with a penthouse area on the roof. Therefore, according to the letter of the code, Type IV construction is not a possible design solution. Through the use of sprinklers and passive fire protection measures, a combined fire-rating behavior is developed. This combination attempts to represents an explanation of the measures used by industry professionals today.

5.1.2 NFPA Sprinkler System Design Requirements

In NFPA 13-16, the National Fire Protection Agency gives a series of rules for the placement of sprinklers within the fire protection system. Those rules are as follows:

- Table 8.6.2.2.1(a): For combustible construction unobstructed, each sprinkler may cover a maximum area of 130 SF
- Table 8.6.2.2.1(b): The maximum spacing per sprinkler is 15'-0."

- Section 8.6.3.2.4.1: The maximum distance between the edge of the wall and the next closest sprinkler is 9'-0."
- Section 8.6.3.4.1: Each sprinkler will be spaced not less than 6'-0" on center.
- Section 8.6.4.1.1.1: The pendant of each sprinkler must hang 1" to 12" bellow the underside of the ceiling.

With regard to obstructions, if the sprinkler is within the range of 4' to 4'-6" from the obstruction in question, 7" are allowed from the bottom of the sprinkler to the bottom of the obstruction.

With the implementation of these measures, the sprinkler component of the IBC's fire-protection measures can be met.

On the following page in Figure 5-1, the proposed sprinkler system is shown in plan with the location of each sprinkler shown and the area it is responsible for protecting.

5.1.3 Sprinkler System Design

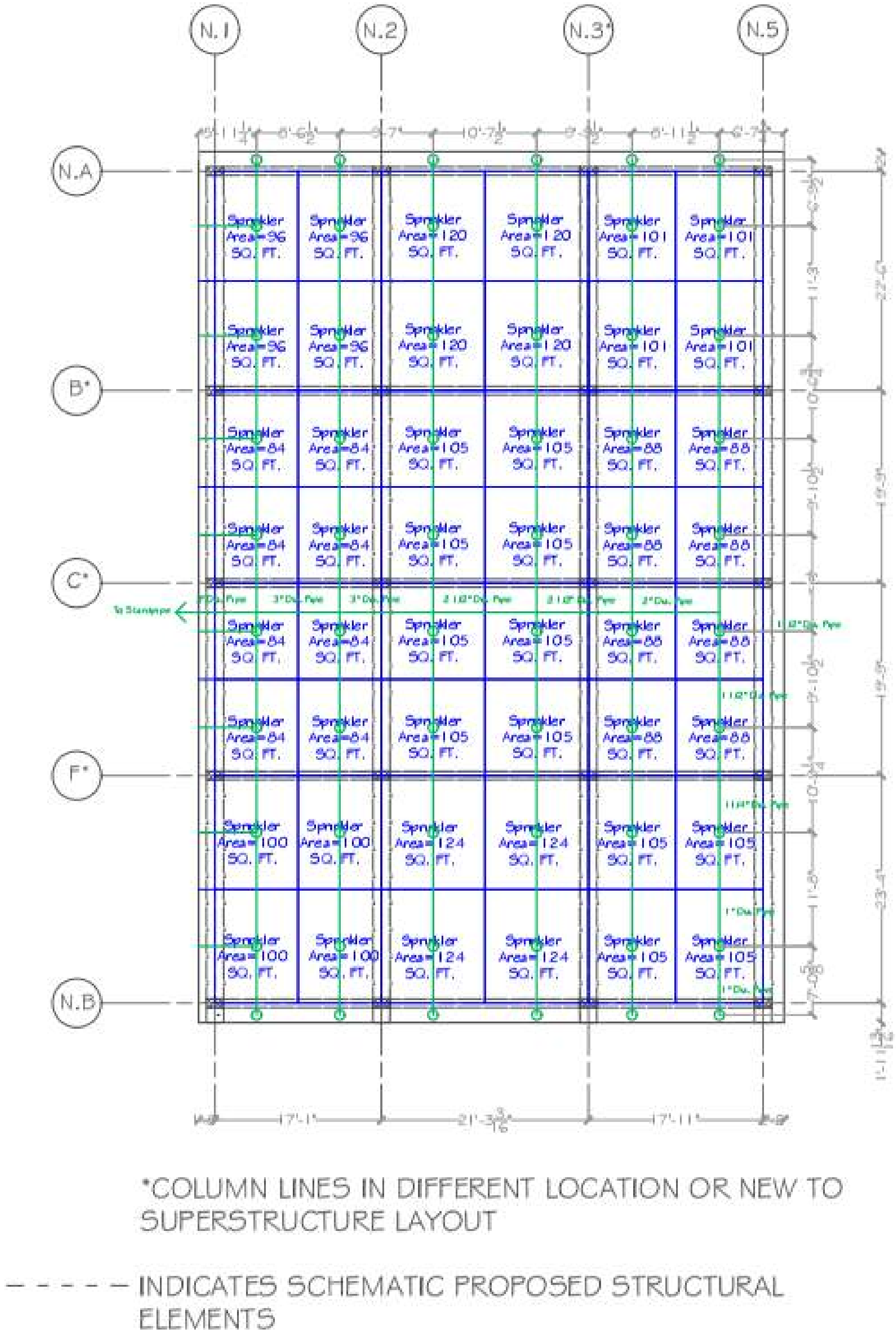


Figure 5-1: Design of the Sprinkler System for a Typical Floor According to NFPA 13.

5.2 Passive Fire Protection Practices

5.2.1 Charring and Encapsulation Methods Overview

Wood has inherent fire protection properties. While wood will initially burn, if the member is thick enough, only the outer layers will burn. The interior core of the member will remain untouched. This behavior is called charring and can be used as part of the fire protection plan for wood members. Figure 5-2 is taken from Technical Report 10 of the NDS from the American Wood Council and shows the char layer of a member exposed to fire.

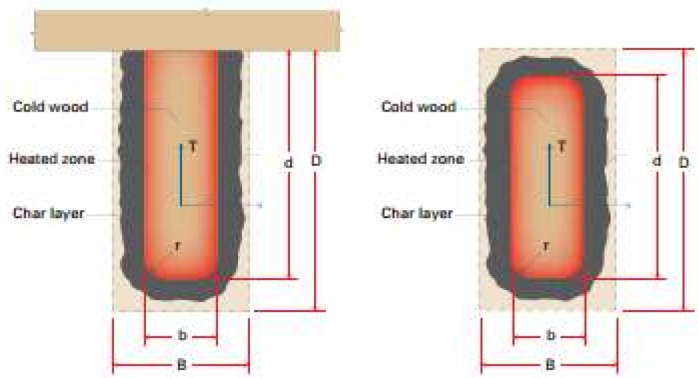


Figure 5-2: The Charred Layers of Wood Elements Exposed to Fire on Three of Four Faces and on Four of Four Faces. Image Courtesy of the American Wood Council, Technical Report 10.

Chapter 16 of the NDS provides guidance on how to estimate char depths. For a given required fire rating, the wood is predicted to char at a set effective char rate known as β_{eff}. For a certain β_{eff}, the effective char depth a_{eff} can be found. For a two-hour fire rating, the effective char rate is 1.58 in/hr and results in an effective char depth of 3.2 in. When evaluating the effectiveness of the member against collapse, this effective char depth must be removed from the cross sectional area on all sides exposed to fire. For a floor element, fire only attacks the surface from one side. For beams, as seen above, fire typically attacks from three sides. Columns are attacked on all four sides, thereby making them most susceptible to loss of sectional strength due to the significant loss in area.

Once the resultant cross sectional area is found, the member must be evaluated for total capacity versus the demand load. Table 16.2.2 provides adjustment factors for fire design. The purpose of the factors is to predict the actual capacity of the members following a fire so as to prevent building collapse. Once the adjusted allowable strength is found in psi, it can be multiplied by the new cross section and evaluated against the demand load. For bending members, this process is as follows:

$$Adjusted\ Allowable\ Bending\ Capacity = F_b\ X\ 2.85\ X\ C_F C_V C_{fu} C_L$$

$$Adjusted\ Capacity\ of\ the\ Member = F'_{b\ FIRE}\ X\ S'_{EFF\ FIRE} \geq Demand\ Load$$

In addition to the protection provided by the charring method, the contributions of additional surrounding material may be considered. Gypsum board and plywood have known fire resistance properties and can be counted on to provide additional fire-rated protection. Type X 5/8" gypsum board has a listed rating of 40 minutes. When used in combination, though, fire ratings are not additive. They increase as shown above.

Table 5-1: Fire Rating of Gypsum Wall Board Assemblies

Material	Fire Rating (min)	Increase from 1 to 2 Layers
3/8 in. GWB	10	2.5
Double 3/8 in. GWB	25	
1/2 in. GWB	15	2.67
Double 1/2 in. GWB	40	
5/8 in Type X GWB	40	2.59 (average of two increases above)
Double 5/8 in Type X GWB	103*	
*Value extrapolated from data above.		

When used in combination with ½" Douglas Fir plywood with a fire rating of 10 minutes, it is reasonable to extrapolate that a combination of double 5/8" Type X GWB and ½" Douglas Fir plywood would have a cumulative fire rating of 120 minutes. The American Wood Council has not published data related to this assembly and would need to be tested for acceptance by the authority having jurisdiction.

5.2.2 Protection of Elements

As described in sections 3.1.5 "Fire Protection Design of the CLT Deck" and 3.2.4 "Fire Protection Design of Gravity Girders," the structural and fire protection designs of wood members are tied to one another. If the selected members can be shown to meet the required strength with the reduction due to charring depth, then additional measures can be avoided. Using gypsum wall board is time-consuming, imposes additional cost, and can detract from the architectural intent of using exposed wood. Exposed wood is what makes structural timber construction different from other types of construction.

While the CLT deck and girders met the charring requirements, the columns did not. Since these elements are exposed on all four sides, they lose 6.4" of depth on each face. Since all columns are 10.5" inches wide, the loss of 6.4" makes these columns incredibly slender and subject to buckling failure. Two methods are available for protecting the columns. The encapsulation method of using two layers of 5/8" Type X GWB and ½" Douglas Fir plywood could provide two hours of protection. This method would, however, be hiding the original wood member with gypsum only to further cover it with a faux wood finish. While possibly necessary to meet code, it could be perceived as being a disingenuous design selection as it hides the real wood structure. The second possibility is a suggestion that was developed through consultation with the American Wood Council. The use of a water curtain surrounding the column could provide direct protection of the column.

Traditionally, fire protection was practiced with a two-pronged approach. While sprinklers could be used, the passive fire protection approach was promoted. Passive fire protection provided a backup plan if the mechanical protection failed. According to the NFPA, when sprinklers operate, they are effective 96% of the time. When sprinklers fail to operate, it is commonly because the water source was cut off from the sprinkler, manual intervention with the system during the fighting of the fire, or lack of maintenance. Those are preventable issues that can be avoided if a sound maintenance plan is followed (Hall).

The discussion of the protection of steel connections in wood construction is something that is open to interpretation. The NDS states in section 16.3, "Connectors and fasteners shall be protected from fire exposure by wood, fire-rated gypsum board, or any other coating approved for the required endurance time" (American Wood Council). The lack of specification has resulted in a discrepancy in what is required, and the predominant design decision made in industry uses an option not directly covered by that statement. Typically, steel connections are left exposed and covered in intumescent paint.

As seen below, in the T3 project, the steel connections are exposed. The wood elements will be exposed in the final design. Therefore, the steel plate seen will also be exposed. In order to provide adequate fire protection, the steel must be covered with an intumescent coating.

Figure 5-3: An Exposed Steel Connection in T3, Image Courtesy of Structure Craft

This precedent extends to other structural timber construction projects in the United States. The Bullit Center used a similar approach. As seen in Figure 5-4, the steel connections were covered with intumescent paint and a sprinkler system was exposed. This design approach allowed the wood elements to remain exposed, eliminated the use of gypsum wall board, and enforced a clean layout of the mechanical systems so as to be aesthetically pleasing. The necessity of a clean layout will pay dividends to the building owner in the future as mechanical servicing and replacement is required.

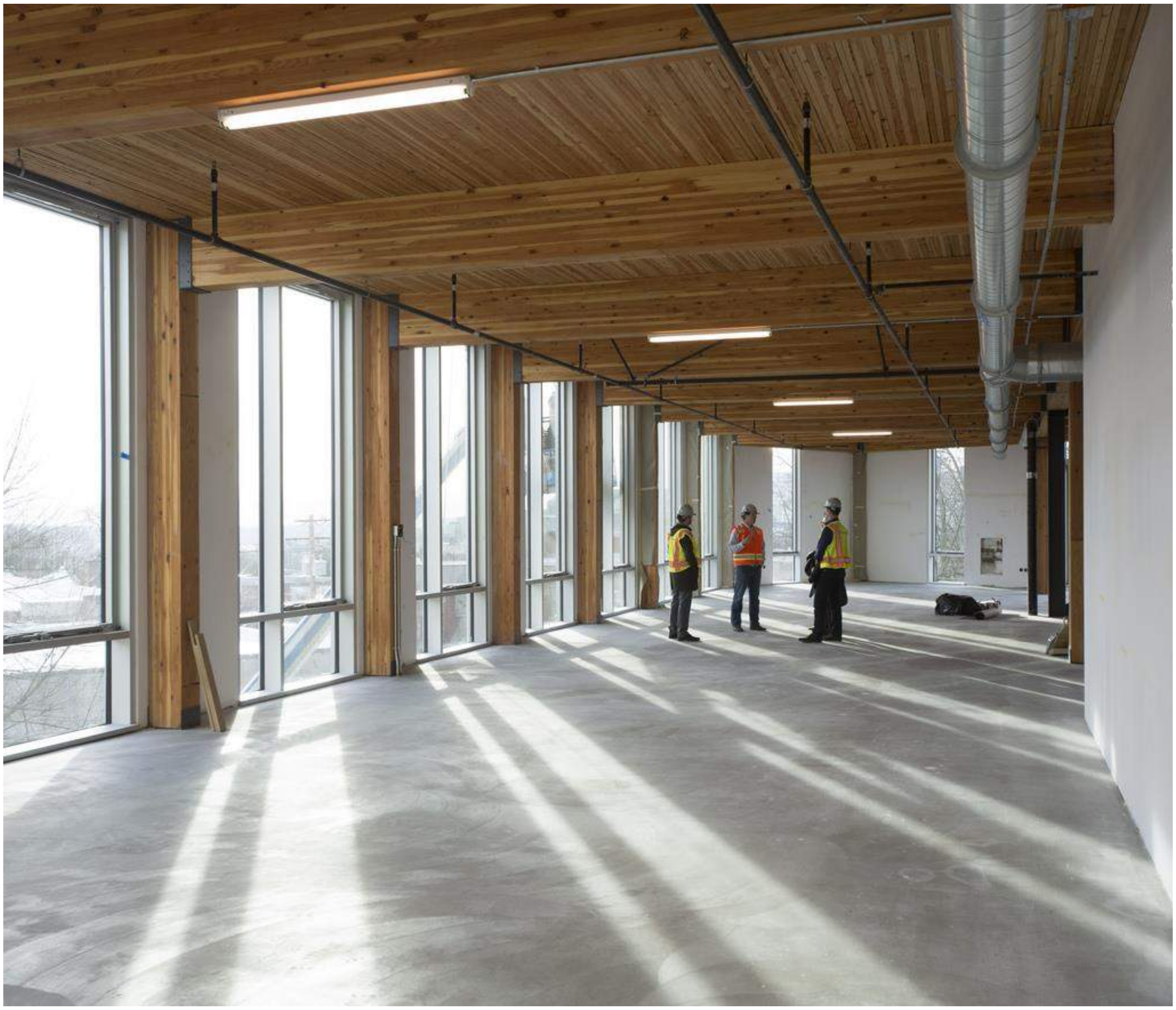


Figure 5-4: The Interior the Bullitt Center. Exposed steel connections can be seen at the intersection of the columns and beams along the exterior façade. The sprinkler and HVAC systems have also been left exposed giving the space an industrial appearance. Image courtesy of the Bullit Center.

The use of intumescent paint was the final decision for the protection of the steel elements in the LiUNA project due to the clean appearance, industrial feel, and elimination of gypsum that would otherwise hide the connection.

Chapter 6: Construction Management Breath: The Constructability of Tall Wood Buildings on the East Coast

6.1 Structural Timber Construction

6.1.1 The Solid Timber Construction Market

While the use of timber is one of the oldest forms of construction across the world and intertwined with early American industry, today, it represents a small, if ever-growing, segment of the overall construction industry. As mentioned earlier, the T3 building in Minneapolis represents the first tall wood building built East of the Mississippi River since the era of the first World War. Today, a number of smaller timber-framing companies exist across the country, and the majority of them are specialized in the construction of custom homes, barns, and small communal buildings such as libraries and houses of worship. As of right now in the United States, there are only a small number of firms and individuals who have the skills and knowledge required to build the proposed LiUNA Expansion Building.

In planning the construction process of the LiUNA expansion building, the first question is, "Who is capable of completing the construction process?" Heavy timber construction shares similar means and methods to several larger and established trades. The three most notable crossover industries are that of precast concrete, steel, and traditional carpentry and framing. A combination of the skills and means of construction found in each of these three industries is representative of those within the heavy timber construction process.

6.1.2 Transferability of Skills from Existing Markets to Solid Timber

While CLT is a developing industry, the panelized system is comparable in size to precast concrete elements. Even though they typically weigh less than precast concrete components, CLT floor and wall panels require similar lifting and handling methods. As explained in Chapter 12 of the CLT Handbook, some of the means and methods used to rig CLT panels come from the precast industry. Since holes can easily be drilled in CLT panels, a wide variety of slings and rigging apparatus are available to contractors looking to use CLT on a project. Manufacturers of CLT panels will provide guidance as to how their panels should be lifted and installed as part of their overall scope of work in manufacturing the panels. It is the responsibility of the timber erection team to follow those instructions.

6.1.3 The Steel Industry and Structural Timber Construction

The connections used in the design of the LiUNA Expansion Building are not typically found in the wood construction industry. However, if the possibility of tall wood buildings in urban environments is to be realized, unique connections will be required. In the LiUNA project, unique steel connections were required for both the gravity and lateral systems. Those custom connections are vastly different from those in typical wood construction and will require the skills of a structural steel fabricator.

Today, some of the most common connections in wood are wood-to-wood dowel connections. That nomenclature applies to any connection where two or more structural wood elements are in contact with one another and are held together with dowels acting to resist shear perpendicular or parallel to grain. The term dowel can apply to any cylindrical element upwards of 1" in diameter made of metal or wood; nails, screws, bolts, rivets, and pegs can all be classified as dowels. Wood-to-wood dowel connections were used in the LiUNA project for the connection between CLT panels via the EWP panel spline.

The first place where custom steel connections were used was at the CLT panel saddle connection. Developed as a way to keep the top face of the deck flush with the top face of the beam, the saddle allowed panels to "hang" from the sides of the gravity girders. As shown in Figure 3-5 the saddle is composed of five pieces of A36 steel plate stock welded together and has punched holes at the locations of the bolts. The second place where custom steel connections are used is at the column caps. The column cap connection, shown below in Figure 6-1 in combination with the moment connection, is a combination of the knife plate shear tab for the gravity girders, moment frame welded angle brace, and bearing connection for the moment frame beams.

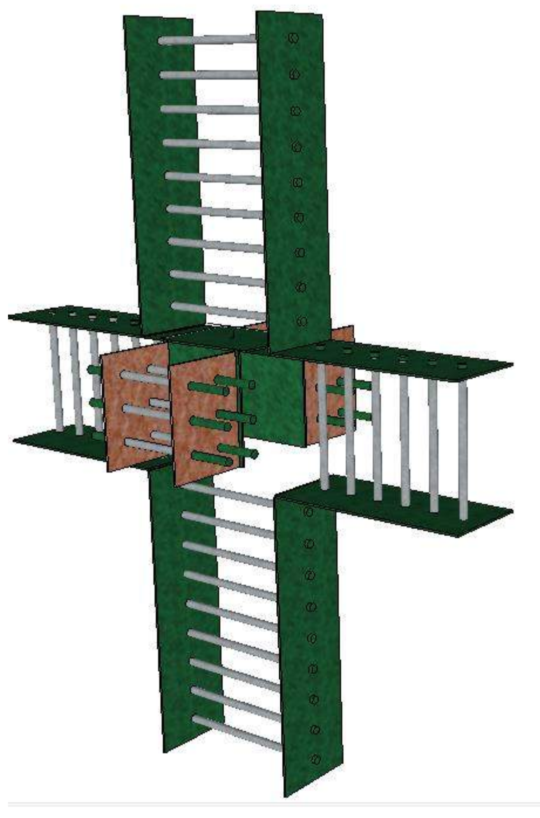


Figure 6-1: Typical Column Cap Connection

At locations within the braced frame, the column cap connections also include the knife plate that is a part of the vertical truss bracing connection as shown below in Figure 6-2. Bolts in the moment connection have been omitted for clarity.

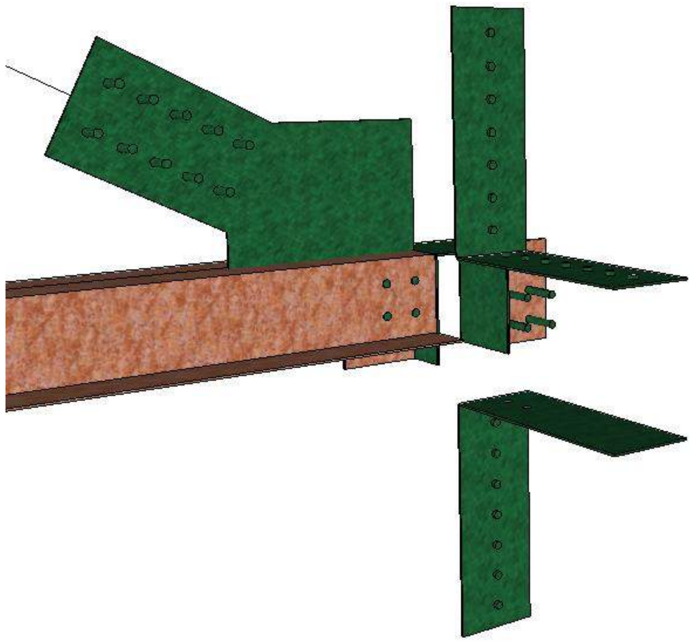


Figure 6-2: Column Cap Connection with Welded Vertical Truss Bracing Plate

All of the custom connections will need to be fabricated by a steel fabrication shop and documented via shop drawings. While the fabrication of plate connections via stock plate is nothing exceptional, typical carpentry projects do not encounter such a large quantity of steel. Furthermore, the connections will most likely be attached to the timber elements in the heavy timber mill shop before they arrive to site. In the fabrication of steel structural elements, it is not uncommon for shear tabs and other various connection elements to be attached to one of the intersecting structural members in the shop. By completing work in the shop, the fabricator is able to have more control on the quality of the product and reduce construction time on site. For the same reasons, the steel connections should be attached to the timber elements before they reach the site.

The use of steel connections will require an investment in the education of several key members of the construction team. The timber fabrication team will have to coordinate with the steel fabrication shop regarding fastening the steel to the timber. The timber erection team, which may or may not be associated with the timber fabrication team, will need to become educated in reading steel shop drawings, handling timber members with steel already attached, and working with atypical connection details. At the site, the construction management team will need to how to schedule and prepare for the timber erection process. Through education and communication among team members, unique steel connections can be utilized to create solutions to some of the challenges presented by heavy timber construction.

6.1.4 The Carpentry Industry and Structural Timber Construction

Heavy timber and wood framing are two very different types of construction. However, since both fields address the use of wood as a structural material, they will utilize the same workforce in a union environment – carpenters. Therefore, understanding the background and experiences of the typical union carpenter are a good starting point for addressing potential carryover topics between the two fields and topics which will require additional education.

As mentioned above, the two biggest differences between heavy timber and wood framing are the size of the members being used and the types of connections. While carpenters might not have first-hand prior experience with structural timber construction, they are skilled in the basic tools used and typical handling practices of wood on-site. In particular, they should have prior knowledge of typical best practices regarding the protection of wood materials from water and weather elements during the construction process. Continued protection of wood elements from water intrusion can be a critical piece of the construction process, and while the timber supplier and fabricator can provide instruction on proper water protection techniques and requirements, it will be up to to the timber erection team on site to implement the prescribed protocols.

The biggest difference between the typical experiences of a union carpenter and timber framer is the scale of the members they are used to handling. Within the Washington D.C. environment, tasks such as scaffolding, formwork, interior molding compose the majority of the work completed by union carpenters. In order to help bridge the gap between typical experience and that which is still needed, repeated workshops and mock-up building events hosted by a combination of the carpenters union and a timber specialist can quickly help the timber erection team of union carpenters to expand their skills and be successful on the project.

6.2 Constructability Analysis

6.2.1 Productivity Investigation

One of the benefits of identifying crossover industries was that it provided a starting point for identifying tasks that could be used to predict the productivity rate of a project team building a tall wood building. Typically, databases and guides such as RS Means can supply information regarding the workers, funds, and time required for different construction process. Since heavy timber is not a large part of the construction industry and since the topic of tall wood buildings is in its nascent stages, there is a dearth of direct information on the topic accessible to those not already in the industry. Therefore, information available for the three crossover industries was scrutinized for similarities to the timber erection process.

On the following page, the RS Means productivity rates for different elements in the precast, steel, and wood industries is presented.

Table 6-1: Precast, Steel, and Timber Element Assembly Information per RS Means

LiUNA Construction Assemblies Work Crew Estimates Using RS Means

Construction Type	Task Number	Task Name	Crew	Labor Hours per crew	Daily Output	Labor Hours per item	Unit Per Item
Precast Concrete Elements	03 41 33.10-1200	Precast Beams, 20' Span, 12"x20"	C-11	72	32	2.25	Each
	03 41 33.10-1400	Precast Beams, 30' Span 12"x36"	C-11	72	24	3	Each
	03 41 33.15-0020	Precast Columns Rectangular to 12' high 16"x16"	C-11	72	120	0.6	LF
	03 41 05.15-0700	Precast Columns 24' High, 1 haunch, 12" x 12"	C-11	72	32	2.25	Each
	03 41 05.15-0800	Precast Columns 24' High, 1 haunch, 20" x 20"	C-11	72	28	2.571	Each
	03 41 13.50-0150	Precast slab planks, 10" thick	C-11	72	3600	0.02	SF
Steel	05 05 23.10-3000	A307 Bolts and Hex Nuts, 1" dia. 12" Long	1 Sswk	Skilled Worker Average	75	0.107	Each
	05 05 23.25-1550	A325 High Strength Bolts, 1" dia. 8" Long	1 Sswk	Skilled Worker Average	85	0.094	Each
	05 12 23.17-7000	W Shape A992 Steel, 2 tier W10 X 45	E-2	56	1032	0.054	LF
	05 12 23.17-7050	W Shape A992 Steel, 2 tier W10 X 68	E-2	56	984	0.057	LF
	05 12 23.17-7100	W Shape A992 Steel, 2 tier W10 X 112	E-2	56	960	0.058	LF
	05 12 23.75-0600	Beam or Girder W 10 X 12	E-2	56	600	0.093	LF
	05 12 23.75-0620	Beam or Girder W 10 x 15	E-2	56	600	0.093	LF
	05 12 23.75-0700	Beam or Girder W 10 X 22	E-2	56	600	0.093	LF
Wood and Timber Members	06 18 13.30-8144	Straight Glulam Beam 20' Span, 6-3/4" x 19-1/2"	F-3	40	28	1.429	Each
	06 18 13.30-8146	Straight Glulam Beam 20' Span, 6-3/4" x 21"	F-3	40	28	1.429	Each
	06 18 13.30-8148	Straight Glulam Beam 20' Span, 6-3/4" x 22-1/2"	F-3	40	27	1.481	Each
	06 18 13.30-8150	Straight Glulam Beam 20' Span, 6-3/4" x 24"	F-3	40	27	1.481	Each
	06 18 13.30-8152	Straight Glulam Beam 20' Span, 6-3/4" x 25-1/2"	F-3	40	27	1.481	Each
	06 18 13.30-8154	Straight Glulam Beam 20' Span, 6-3/4" x 27"	F-3	40	26	1.538	Each
	06 18 13.30-8156	Straight Glulam Beam 20' Span, 6-3/4" x 28-1/2"	F-3	40	26	1.538	Each
	06 18 13.30-8158	Straight Glulam Beam 20' Span, 6-3/4" x 30"	F-3	40	26	1.538	Each
	06 18 13.20-8290	Straight Glulam Beam 30' Span, 6-3/4" x 19-1/2"	F-3	40	28	1.429	Each
	06 18 13.20-8292	Straight Glulam Beam 30' Span, 6-3/4" x 21"	F-3	40	28	1.429	Each
	06 18 13.20-8294	Straight Glulam Beam 30' Span, 6-3/4" x 22-1/2"	F-3	40	27	1.481	Each
	06 18 13.20-8296	Straight Glulam Beam 30' Span, 6-3/4" x 24"	F-3	40	27	1.481	Each
	06 18 13.20-8298	Straight Glulam Beam 30' Span, 6-3/4" x 25-1/2"	F-3	40	27	1.481	Each
	06 18 13.20-8300	Straight Glulam Beam 30' Span, 6-3/4" x 27"	F-3	40	26	1.538	Each
	06 18 13.20-8320	Straight Glulam Beam 30' Span, 6-3/4" x 28-1/2"	F-3	40	26	1.538	Each
	06 18 13.20-8340	Straight Glulam Beam 30' Span, 6-3/4" x 30"	F-3	40	26	1.538	Each

The CLT Handbook notes that the erection of CLT panels is similar to that of precast planks. A combination of the information for precast and glulam members was selected as shown below in Table 6-2 in order to create a new data set from which inferences about the productivity rate of structural timber construction could be made. Even where there was information provided for glulam beams, a combination between glulam beams and precast beams was used due to the increase in weight in the beams due to the attached saddle and moment frame connections.

Table 6-2: Adjusted Productivity Rates for Structural Timber Elements

Average Floor Estimate					
Task	***Item Rate Referenced***	***Qty. @ Flr.***	***Units***	***Crew***	***Modified Daily Output***
Setting Columns	Precast Columns 12' High, 1 haunch, 20" x 20"	20	Ea.	F-3	31
Setting MF Beams	Avg. of: Precast Beams 30' Span 12" x 36" & Straight Glulam Beam 30' Span 6-3/4" x 30"	15	Ea.	F-3	22
Setting Gravity Beams	Avg. of: Precast Beams 30' Span 12" x 36" & Straight Glulam Beam 30' Span 6-3/4" x 30"	16	Ea.	F-3	22
Setting Vertical Truss Bracing	Straight Glulam Beam 30' Span, 6 3/4" x 30"	4	Ea.	F-3	26
Setting CLT Floor System	Precast slab planks, 10" thk.	5250	SF	F-3	2000

As noted in Table 6-1, each of the tasks has an associated crew size, labor hours per crew, labor hours per item, and a daily output. For instance, task "Precast Beams, 20' Span, 12" x 20""" is associated with crew "C-11," the 72 labor hours for that crew, 2.25 labor hours per item, and a daily output of 32 beams. In order to create adjusted productivity rates, the modified daily output needed to be found for each item with a crew typical of timber construction. Crew F-3 provided by RS Means features 4 carpenters and 1 crane operator for a total of 40 labor hours. To find modified daily outputs, the labor hours per crew was divided by labor hours per item. With this information, there was now an estimated time it would take a typical carpentry construction crew to install structural timber elements.

6.2.2 Adjusted Productivity Rate Analysis and Predicted Schedule

With the information provided by Table 6-2 in hand, it is possible to create an estimated superstructure construction schedule. By dividing the total number of elements at each floor by the modified daily output, the total number of hours and days required per item at each floor can be found. Those results are shown in Table 6-3 on the next page.

Table 6-3: Average Floor Construction Time Estimate with Modified Daily Outputs

Average Floor Construction Time Estimate						
Task	***Item Rate Referenced***	***Qty. @ Flr.***	***Units***	***Modified Daily Output***	***Days Req'd***	***Hrs. Req'd***
Setting Columns	Precast Columns 12' High, 1 haunch, 20" x 20"	20	Ea.	31	0.6	5.1
Setting MF Beams	Avg. of: Precast Beams 30' Span 12" x 36" & Straight Glulam Beam 30' Span 6-3/4" x 30"	15	Ea.	22	0.7	5.5
Setting Gravity Beams	Avg. of: Precast Beams 30' Span 12" x 36" & Straight Glulam Beam 30' Span 6-3/4" x 30"	16	Ea.	22	0.7	5.8
Setting Vertical Truss Bracing	Straight Glulam Beam 30' Span, 6 3/4" x 30"	4	Ea.	26	0.2	1.2
Setting CLT Floor System	Precast slab planks, 10" thk.	5250	SF	2000	2.6	21.0
		Sum per Floor			4.8	38.7
		Weeks Req.'d Per Floor			1.0	

Using the modified daily outputs, it would take one F-3 timber erection crew 38.7 hours to erect one floor’s worth of structural elements. Included in this estimate is the time require to initially set the members and make the necessary connections. It is assumed that a second crew of similar composition minus the crane operator is on site to assist in the final tightening of connections, setting the CLT spline connections, and other additional work related to the securing of structural elements.

Assuming a 40 hour work week, each floor could be built in a work week, resulting in a total construction time of nine weeks. This estimate does not include workers’ initial unfamiliarity with the systems, weather delays, and other unforeseen setbacks. If a conservative approach were to be used, it is possible to predict that the superstructure could be built in 12 weeks, allowing a 33% factor of safety against the actual prediction of 9 weeks. By comparison, the Forte Tower in Melbourne, Australia took 10 weeks to erect with 5 skilled laborers, 1 supervisor, and 1 trainer. It is not clear if the crane operator was included in that total. In the Forte Tower, simple connections of steel angles and lag screws were used making a very repetitive and easy-to-build structure (Griffin, 26).

The Forte Tower serves as the best case study to date in terms of number of stories, building footprint, and urban construction restraints for a comparison with the estimated superstructure schedule of the proposed LiUNA redesign. The similarity in superstructure construction times between the actual project and proposed estimate gives credence to the construction productivity estimates developed. With this information, it is possible to have a baseline for future tall wood construction projects. Moving forward, a continued analysis of case study buildings is required by the wood industry to continually educate the overall construction and design community.

6.3 Industry Feedback and Professional Input

6.3.1. Need for Professional Input

While there are resources on the topic of structural engineering with wood as a material, there are few resources about the construction of such structures. In comparison with the information available for other construction types, the introductory knowledge of typical means and methods of timber construction remains mostly with those who practice the craft. In order to supplement the assumptions and findings in this report, several industry professionals were consulted to provide additional insight as to how one might build a tall wood building.

6.3.2. Mark Taylor, Nitterhouse Concrete Products

RS Means was used to gather information regarding precast concrete element erection, but there were initial concerns about the accuracy of the information available. In order to verify the assumed data, Mark Taylor of Nitterhouse Concrete Products was contacted via email. Mark Taylor is the President and Chief Operating Officer of Nitterhouse, a company specializing in precast concrete elements in the Pennsylvania-Maryland region.

A typical crew size for a precast concrete erection sequence is 7 people and composed of:

- 1 Foreman: Responsible for overseeing the operation
- 1 Laborer: Responsible for rigging all elements coming off the truck
- 2 Laborers: Responsible for erecting elements in-place
- 2 Laborers: Responsible for setting element connections
- 1 Crane Operator

This crew size of 7 is consistent with the size used to build the Forte Tower.

For an average precast beam or column element, about 30 minutes is required for the erection sequence. This rate results in a production rate of 2 elements per hour. The modified daily output of a glulam gravity girder with a CLT saddle pre-attached was estimated to be 13 girders per day, or 1.625 per hour when using information only related to precast elements. The final modified daily output of 22 beams per day, or 2.75 beams per hour, results from averaging the modified rates of glulam beams and precast beams. Since the erection sequence for precast elements typically includes tack welding, it is not surprising that prefabricated glulam beams have a slightly higher erection rate. The elimination of welding reduces construction time.

6.3.3. Alex Schreyer, University of Massachusetts, Amherst

Alex Schreyer is a professor of Building Construction and Technology at the University of Massachusetts, Amherst. With a background in structural engineering, wood science, and BIM with a specific specialty in Sketch Up, Mr. Schreyer has served alongside Dr. Peggi Clouston as experts in the field of wood construction as the University has undertaken the construction of their new Integrated Design Building using CLT and other EWP solutions.

One of the areas in which Mr. Schreyer was able to shed light was that of the project team's composition. Like a typical building, a construction management company was hired and held the contract for the project. The timber erection team was composed of union carpenters; while experienced in the field of wood construction, these carpenters did not have significant experience with large-scale timber framing. In order to educate the timber erection team, the construction manager, and the entire team, a timber framing professional was hired. That company was Bensonwood of Walpole, NH.

6.3.4. Tedd Benson, Bensonwood and Unity Homes

Tedd Benson is the founder of Bensonwood and its specialty offspring company, Unity Homes. Bensonwood has traditionally focused on barns, libraries, homes, and other typical applications of timber framing. Unity Homes was recently developed to explore the possibilities of lean manufacturing, montage building, and building production modeling for homes. Mr. Benson was able to answer some questions in person about the Integrated Design Building and his work at the Pennsylvania Housing Research Center (PHRC) Conference hosted by the Departments of Architectural and Civil Engineering at Penn State.

As the education arm of the IDB project team, Bensonwood has had the opportunity to shape the process by which a large-scale wood building would be built on the East Coast. In order to provide continued support for the timber team on site, two of Bensonwood's employees are on site and host twice-a-day meetings with the team. These meetings go over the process and logistics of the work ahead of the crew. Many of the processes required by the building's design have been foreign to those in the field. Particularly, the timber connections are atypical from those within the repertoire of the carpenters on-site. A composite CLT-to-concrete floor system using a technology from Germany, bolted steel connections, and a wide variety of details presented a steep learning curve. The twice-a-day meetings allow the team to review drawing details and complete mock-up workshop sections where workers get to build sample connections before completing the work in the field. Every member of the timber erection team has been trained in how to correctly build each connection type.

Two logistical challenges of the Integrated Design Building have been the size of the CLT deliveries made to site and the use of just-in-time delivery. In order to keep the construction site clear and protect the wood from the elements, the CLT panels are delivered as they are needed in the erection sequence. The quick turnaround between manufacturing and installing has allowed the team to provide live feedback to the manufacturing team at Nordic Structures in Montreal, Quebec. As the timber erection team found inefficiencies in the connection details, they were able to communicate with Nordic to make revisions to the panels currently on the assembly line. This direct communication helped make the overall project more efficient.

One of the last points Mr. Benson made was the great pride the timber erection team took in their work and the enthusiasm surrounding the education process. Through open dialogue between the framing experts and the timber erection team, both groups were able to collaborate continually. The future success of tall wood hinges on experiences such as these. While structural timber construction is only a sliver of the entire market, it has the potential to grow thanks to developments in educational and communication used on tall wood projects.

Chapter 7: Software Investigation

7.1 Software in the Timber Industry

While timber framing is an ancient craft, modern wood working has made use of technology in order to produce a reliable and efficient product. From the initial design of the structure through the cutting and placement of each timber, software is omnipresent in the timber construction industry.

Within the last decade, the topics of Building Information Modeling (BIM), clash detection, virtual reality, and cloud-based design have risen to the main stage of discussion within the design and construction community. Often touted as tools capable of bringing efficiency, teamwork, and integration to projects, these concepts have become an accepted component of modern practice. In this regard, timber framing is at the vanguard of the building industry in its usage of these tools. Without advanced software, the possibility of tall wood buildings would not exist.

After the first sketches have been drawn, a modern timber frame will first be built digitally in one of several products. While Sketch Up is used throughout the U.S. and is readily available, the top-of-the line software package available is CadWorks. Developed by a company in Germany, CadWork has a wide variety of uses. The detail of every joint, whether it be mortise-and-tendon or modern knife plate, can automatically be modeled using parametric elements that can be readily replicated. The individual details of every joint can then be used to produce 2D and 3D drawings of any joint that requires special attention. The ability to make accurate 3D drawings of complicated connections can be of great benefit to the timber erection team in the field.

Upon completion of the model, a complete schedule of all beam species, sizes, and locations can be produced and sent to a CNC machine. Computer Numerical Control machines allow a timber framer to rapidly cut members from timber stock based on information made in a computer model. In that way, the element shown in the digital model can be precisely manufactured.

From a structural engineering perspective, the RISA 3D software package has become a favorite due to its ability to model the cross-grain properties of wood elements. While traditional materials have isotropic material properties, wood's anisotropic behavior and related array of design conditions presents significant challenges for other software packages.

7.2 Software Usage in the LiUNA Expansion Building Project

Neither CadWork nor RISA 3D were available at Penn State for use in this project. In order to overcome this challenge, several solutions were developed.

During the initial stages of design, SAP 2000 was used to create models of the three types of frames that controlled the design of the building: an interior moment frame, an exterior moment frame, and a vertical truss. All three of these models were loaded with their respective gravity and lateral loads found by hand. Using a flexible diaphragm assumption, lateral loads were distributed by hand to all three frames. The models were run for all applicable load cases; resultant design loads and stresses were found.

Traditionally, wood design has focused on the use of hand calculations based on formulas found in the NDS. Using these equations, Excel spreadsheets were programed for all element types found in the LiUNA project. In these spreadsheets, the designer has the option to input the necessary loads found via analysis and then manually input section properties (height and width) and wood species material strength information. The spreadsheets then ran all checks for strength, deflection, vibration, and whatever additional criteria were necessary. The final results of these spreadsheets can be found in the appendices of this report.

In lieu of CadWork, Sketch Up was used to create a full model of the proposed design. While making a full 3D model is not typically necessary for the AE senior thesis project, the creation of such a model served two purposes. First, the model was able to accurately represent the variety of details found throughout the project. While drawing out all of the details by hand was necessary during the design of individual components, the completed model provided a visual representation of how otherwise discontinuous elements would intersect in space. At locations like the moment frame – to - column cap – to – knife plate connection, three elements that were designed at separate times came together in one continuous steel element. The result of which can be seen in Figure 6-1. Second, the model presented a tangible piece of evidence that could be presented to make the concept of tall wood buildings come alive. In discussions with industry professionals, the model served as a valuable starting point for discussion and understanding. In many locations, topics such as the CLT Saddle connection, moment frame connection, and vertical truss bracing elements were hard to describe accurately. In these moments, a picture could speak a thousand words. That one picture could create excitement and interest in the possibility that tall wood can be built. These concepts of moment frames and bracing might seem impossible when put in the context of wood, but when rendered with the level of detail made possible by technology like Sketch Up, they can take on familiar forms found in the everyday building vernacular.

7.3 Lessons Learned through the Use of Software

While new materials and design mentalities can be regarded as having associated high costs for both design and graphical representation software, these perceptions do not have to apply to wood. Using skills and tools available to many engineers in practice today, gaps were bridged between the results needed and the resources available.

Not only did Excel provide a solution to the problem of not having design software, it seemed to be well-suited for the wood design field. Already available to almost all engineers, the use of Excel removes the possibility of increased overhead costs for design professionals. As the American timber community looks to grow enthusiasm for wood structural solutions, one of the biggest selling points can be that an engineer looking to enter into this field doesn't have to buy one new piece of software. Furthermore, the manual input of all relationships and equations necessary to program the design calculations gives the engineer an increased understanding of how the equations work. How many engineers have this same level of comprehensive understanding as to how their other software is programed?

Sketch Up presents another unique story. While not as powerful as other options, it does present its selling point along the same lines as Excel: Everyone can use it. For design professionals looking to enter the field on a smaller scale, Sketch Up presents an easy to obtain and use solution to the question of how to quickly represent custom wood components and connections. Through additional tools such as translucent hatching and hidden line representation, exploded views of connections can quickly be produced to help explain concepts. Through modeling, professionals can quickly begin to speak the same language about a project.

By approaching the design of the LiUNA Expansion Building as a real project rather than a senior thesis project, two realistic solutions were developed that show promise of being adaptable for industry acceptance. The use of Excel eliminated the use of repetitious hand calculations and allowed for many aspects of the project to quickly be designed. In a project where emphasis was placed on using efficient section sizes so as to reduce shipping concerns, the capability to quickly reiterate section designs was of great importance. Sketch Up presented a way to represent the wide variety of custom connections found on the project even though the preferred software package was unavailable. Since both of these software packages are readily available, they can represent a portion of the solution in allowing the tall wood design movement to gain acceptance and further develop.

Chapter 8: Conclusion

8.1 Project Overview and Investigative Conclusions

8.1.1 Design Overview

Originally designed as a steel composite building, the redesign of the LiUNA Expansion Building in Washington D.C. served as a study into the issues and solutions surrounding the field of structural timber construction. A recent increase in the attention paid by the design community to wood buildings has raised new questions for structural engineers looking to use wood. While wood has served as a trusted material for residential construction throughout the last century of American homebuilding, wood has only recently become a proposed structural solution for larger structures. Most of those proposed and completed projects have used wood in geographic locations and occupancy types where there was some precedent. Recently, however, tall wood buildings have opened up a new door to additional applications of wood construction.

The LiUNA redesign did not serve to state that an STC system would be the most efficient, cost-effective, or practical. Rather, the goal was to investigate issues surrounding a proposed wood building in an urban setting along the Atlantic seaboard. By investigating these issues, the conversation surrounding wood buildings can be oriented in the direction of productive inquiry, discussion, and investigation.

8.1.2 Structural Depth Conclusions

The main structural goal was to propose a solution to the worst-case scenario for lateral resistance in an all-wood structure: the required use of moment frames as the main lateral system. In addition to answering this question, the investigation also sought to develop connection details that would reduce structural depth and help make STC more practical for restrictive floor-to-floor height situations. The proposed moment frame is an unproven system but makes use of concepts used in the design of steel bolted moment connections. While the evaluation of the dowel-capacities and overall system behavior is based on research found in the writing of this report, there appears to be no previous investigation into the use of perpendicular steel plates bolted to the glulam elements as is proposed. If this concept is to be used in practice, further research is required.

8.1.3 Mechanical Breath Conclusions

While STC is classified as Type IV construction according to the IBC and is not defined as having a fire resistance rating of two hours, this study proposes an alternative perspective on the fire-resistive properties of wood members. By using a cumulative analysis of the intrinsic charring properties of wood in addition to the additional contributions of gypsum wall board and other coatings, member cross-sections can be shown to have a cumulative fire resistance equaling or exceeding that required for a two hour fire rating. In that way, the addition of material to a structural member’s surface is analogous to the application of spray-on fire protection on steel. Furthermore, the use of sprinkler systems according to industry guidelines provides additional protection.

8.1.4 Construction Management Breath Conclusions

There is a lack of available information regarding the construction of timber systems. Information that is traditionally available for the scheduling, pricing, and construction of typical structural systems is not available for STC. Therefore, inferences must be made based upon assumptions, similarities, and what information has been published. Using case study research and limited guidance from industry guides, precast concrete can be used to approximate the schedule of an STC project. The usage of this approach yielded an approximate schedule similar to that for case studies found via the University of Utah. The experiences of those involved with wood buildings, such as Alex Schreyer and Tedd Benson, are critical to the industry's education and successful expansion into this field.

8.1.5 Software Investigation

While today's practice of structural engineering is heavily based on the use of sophisticated software, the engineering of wood is not at the same standard. While sophisticated software exists, it may not be easily accessible or financially feasible for engineers looking for a foray into the field. Therefore, the redesign of the LiUNA Expansion Building was an opportunity to evaluate the relative ease of reapplying existing software for application to a structural timber project. For the design of the members, excel spreadsheets were easily programed to allow the engineer to reduce calculations to an informed initial guess about the species and section required. Sketch Up was an easy-to-use medium with which the various connections could be modeled.

8.2 <u>Conclusion</u>

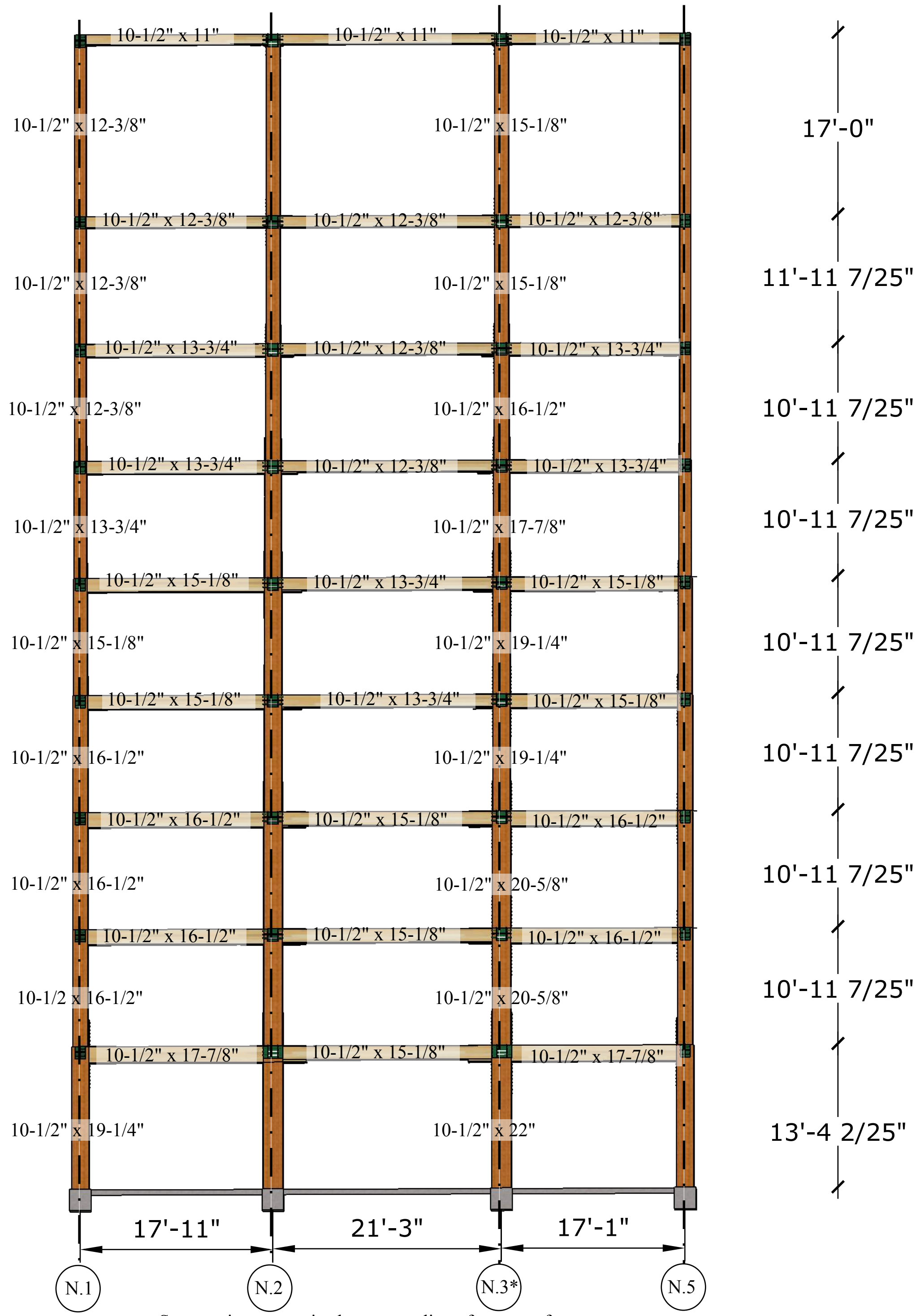
The LiUNA Expansion Building in Washington D.C. served as useful case study on the topic of the feasibility and applicability of structural timber construction. While STC was not the most cost effective or practical solution to this project, it shows promise as a realistic solution for future projects. If the interest in and development of the structural timber construction continues, the industry stands a chance to grow. Such growth would reduce the challenges identified in the mechanical, construction management, and software breaths.

Structural timber construction appears to be served as a solution to low to mid-rise projects that do not feature large floor loads and utilize shear wall cores as their lateral system. Projects that have occupancy types of educational, light commercial, low-rise assembly, low to mid-rise business, and multi-family residential can make use of wood in a practical manner. Multifamily residential projects stand to greatly benefit from the use of STC due to the decrease in construction time resulting in quicker occupancy.

Timber is the only main construction material that is completely renewable. As the architecture, engineering, and design community searches for solutions to the 21st century questions of renewable resources, environmental protection, and financially-feasible options, wood can prove to be part of the solution.

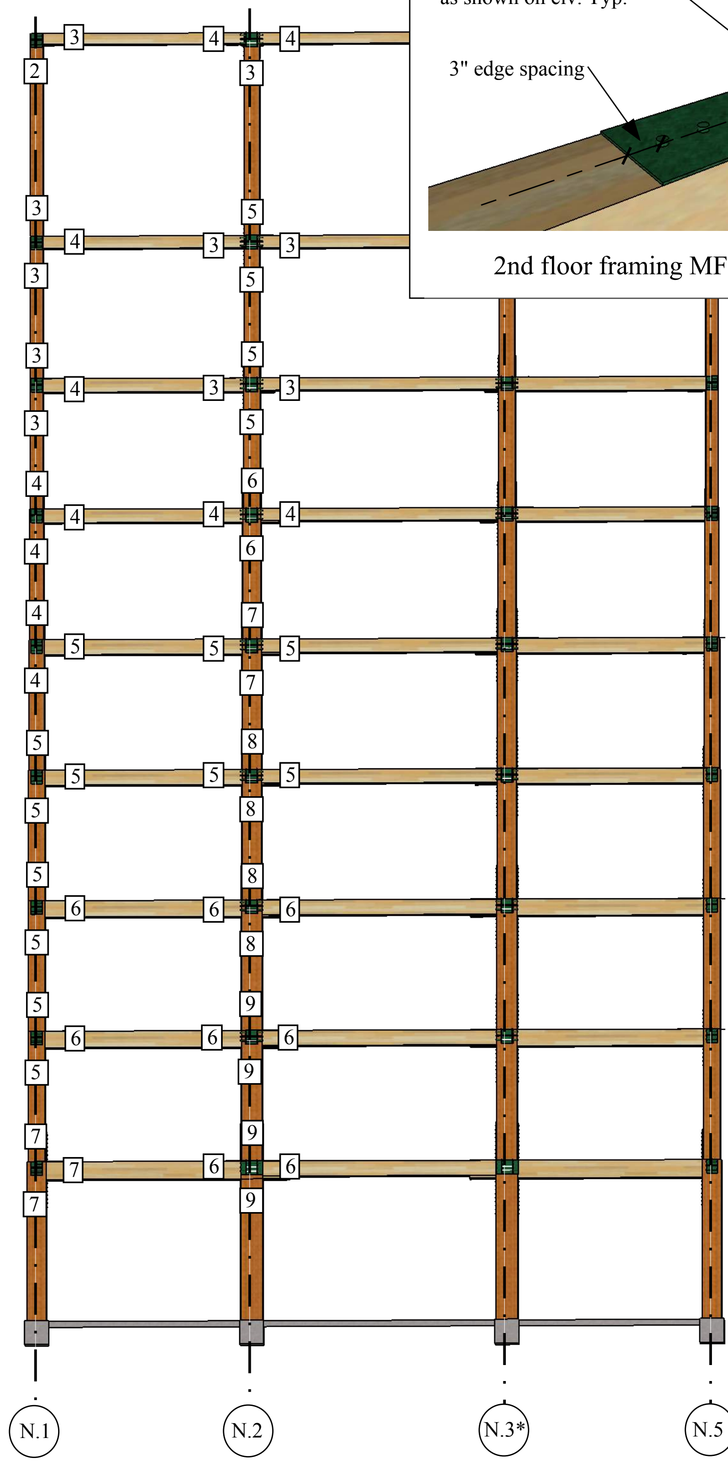
Appendix A: Structural Drawings

LiUNA Expansion Building	Structural Drawings
Josh Jaskowiak	
Penn State University, Department of Architectural Enigneering, Senior Thesis	

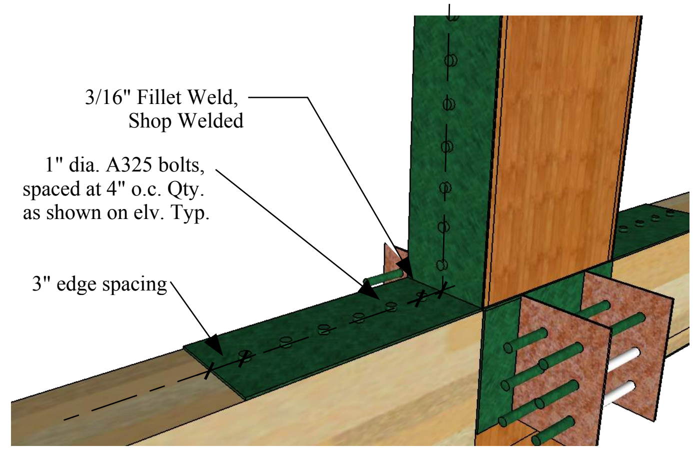


Stucture is symmetric about centerline of moment frame structure.
Column sizes shown on CL N.1 are representative of those on N.1 and N.5. Column sizes shown on CL N.3* are representative of those on N.3* and N.2.
Floor-to-floor heights shown are representative of those for those in momemt frame 5 and the vertical truss.

LiUNA Expansion Building	Moment Frame 4: Interior MF
Josh Jaskowiak	
Penn State University, Department of Architectural Enigneering, Senior Thesis	



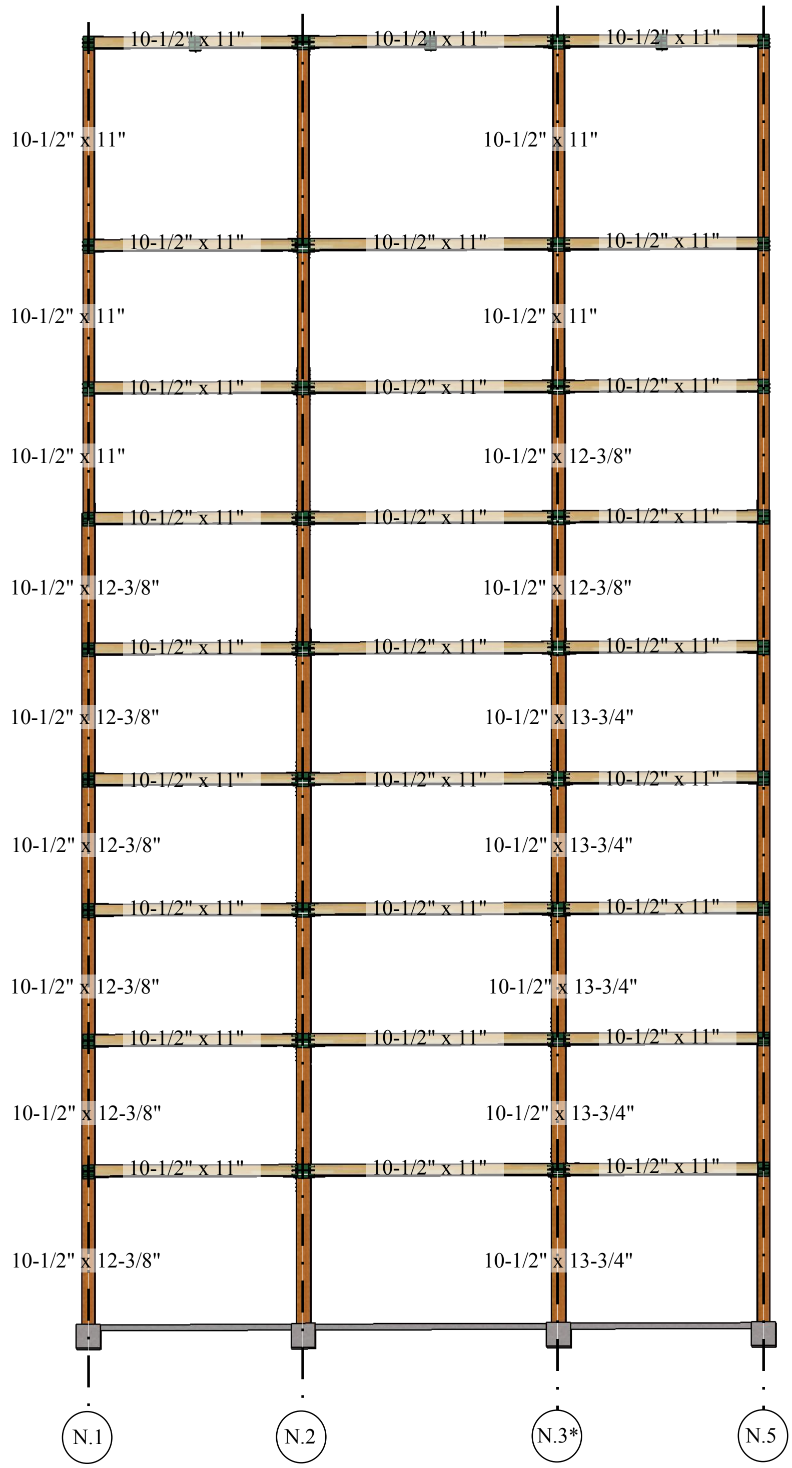
Moment Frame Connection Detail, Typ.



2nd floor framing MF-4 at column line N.B shown

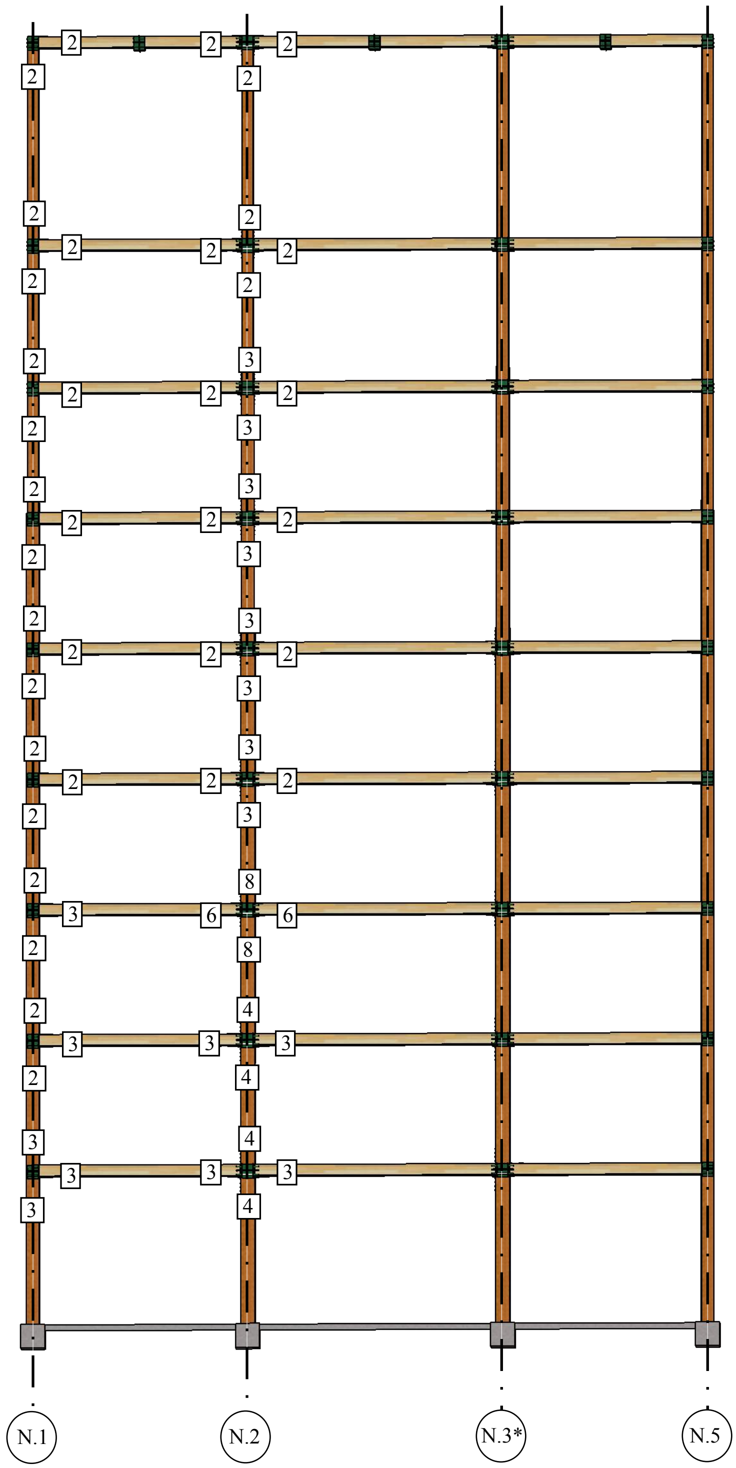
Stucture is symmetric about centerline of moment frame structure.
Number of 1" dia. bolts required along the longitudinal direction of each member is shown against that respective member.

LiUNA Expansion Building	MF-4: Bolted Connections
Josh Jaskowiak	
Penn State University, Department of Architectural Enigneering, Senior Thesis	



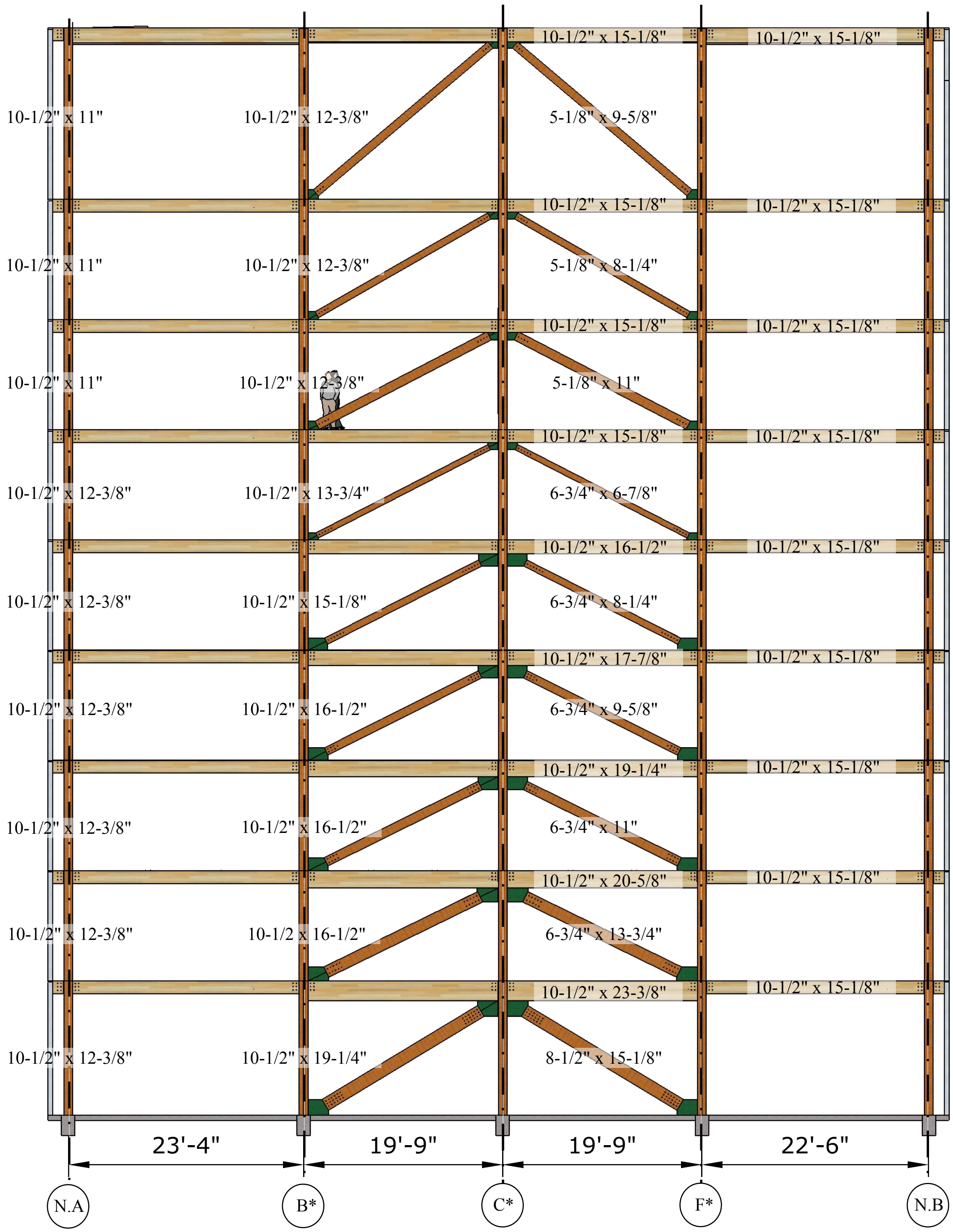
Stucture is symmetric about centerline of moment frame structure.
Column sizes shown on CL N.1 are representative of those on N.1 and N.5. Column sizes shown on CL N.3* are representative of those on N.3* and N.2.

LiUNA Expansion Building	Moment Frame 5: Exterior MF
Josh Jaskowiak	
Penn State University, Department of Architectural Enigneering, Senior Thesis	



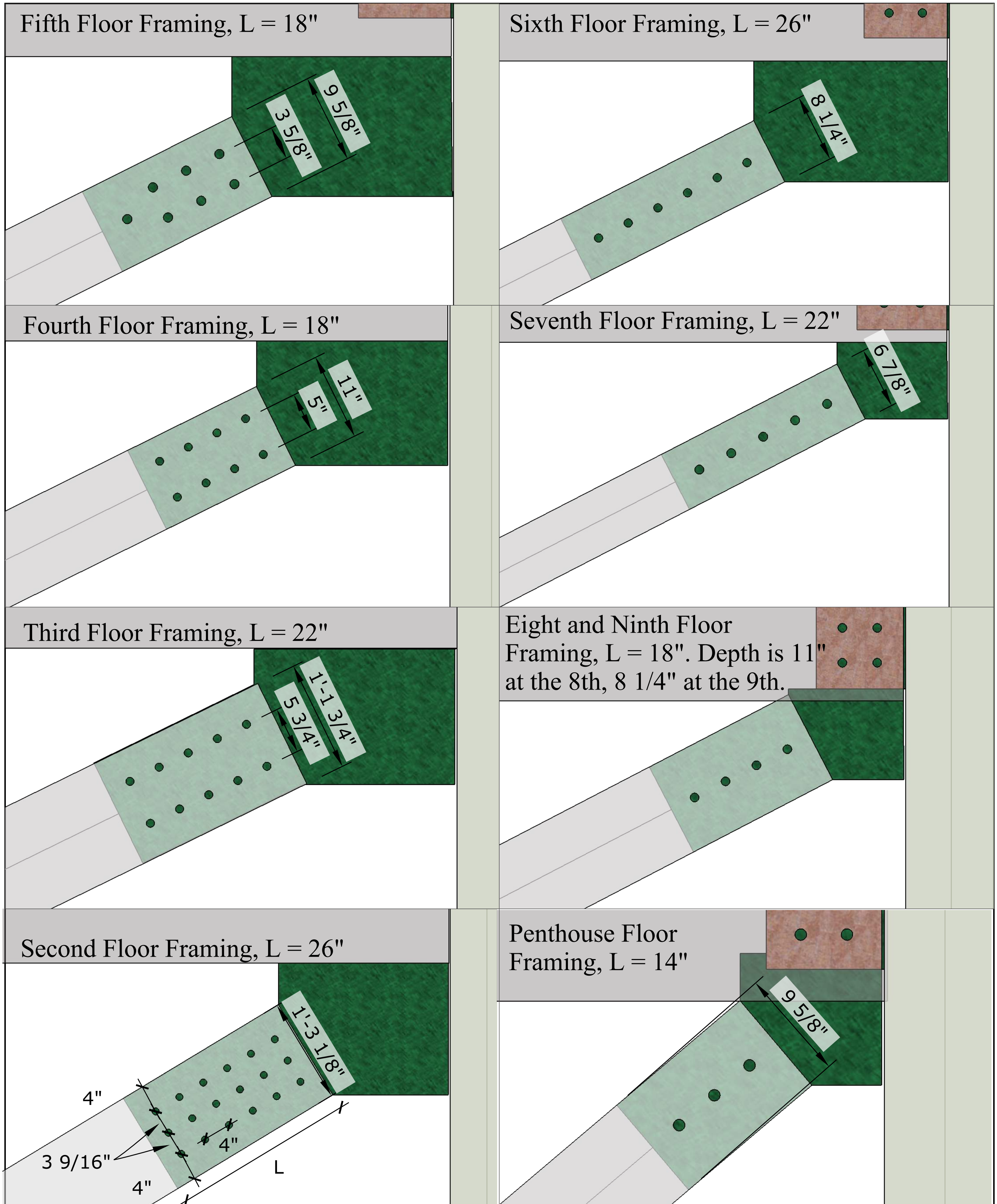
Stucture is symmetric about centerline of moment frame structure.
Number of 1" dia. bolts required along the longitudinal direction of each member is shown against that respective member.

LiUNA Expansion Building	MF-5: Bolted Connections
Josh Jaskowiak	
Penn State University, Department of Architectural Enigneering, Senior Thesis	



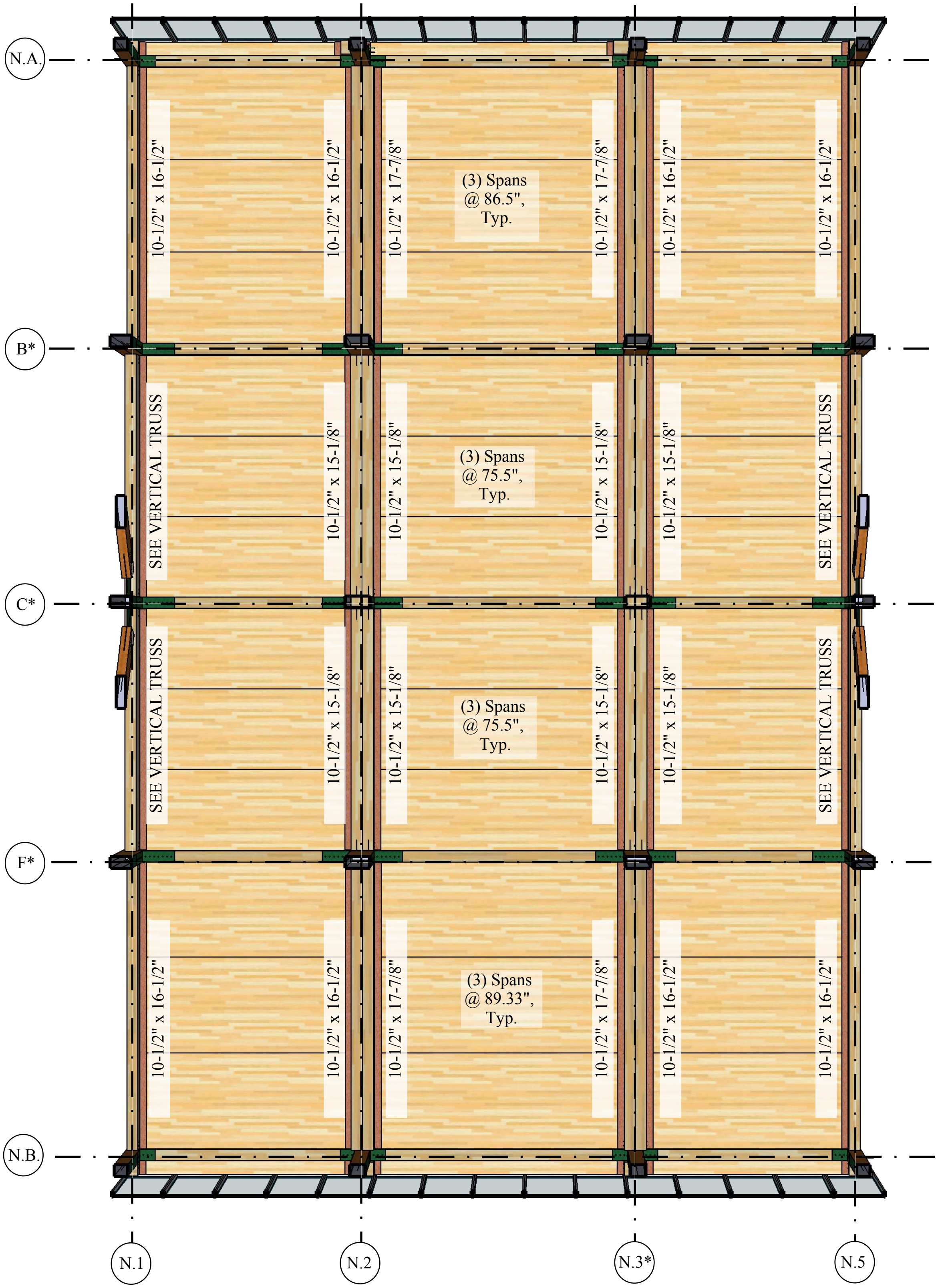
Stucture is symmetric about centerline of vertical truss structure.
Column sizes shown on CL B* are representative of those on B*, C*, and F*. Column sizes shown on CL N.A are representative of those on N.A and N.B.

LiUNA Expansion Building	Vertical Truss
Josh Jaskowiak	
Penn State University, Department of Architectural Enigneering, Senior Thesis	



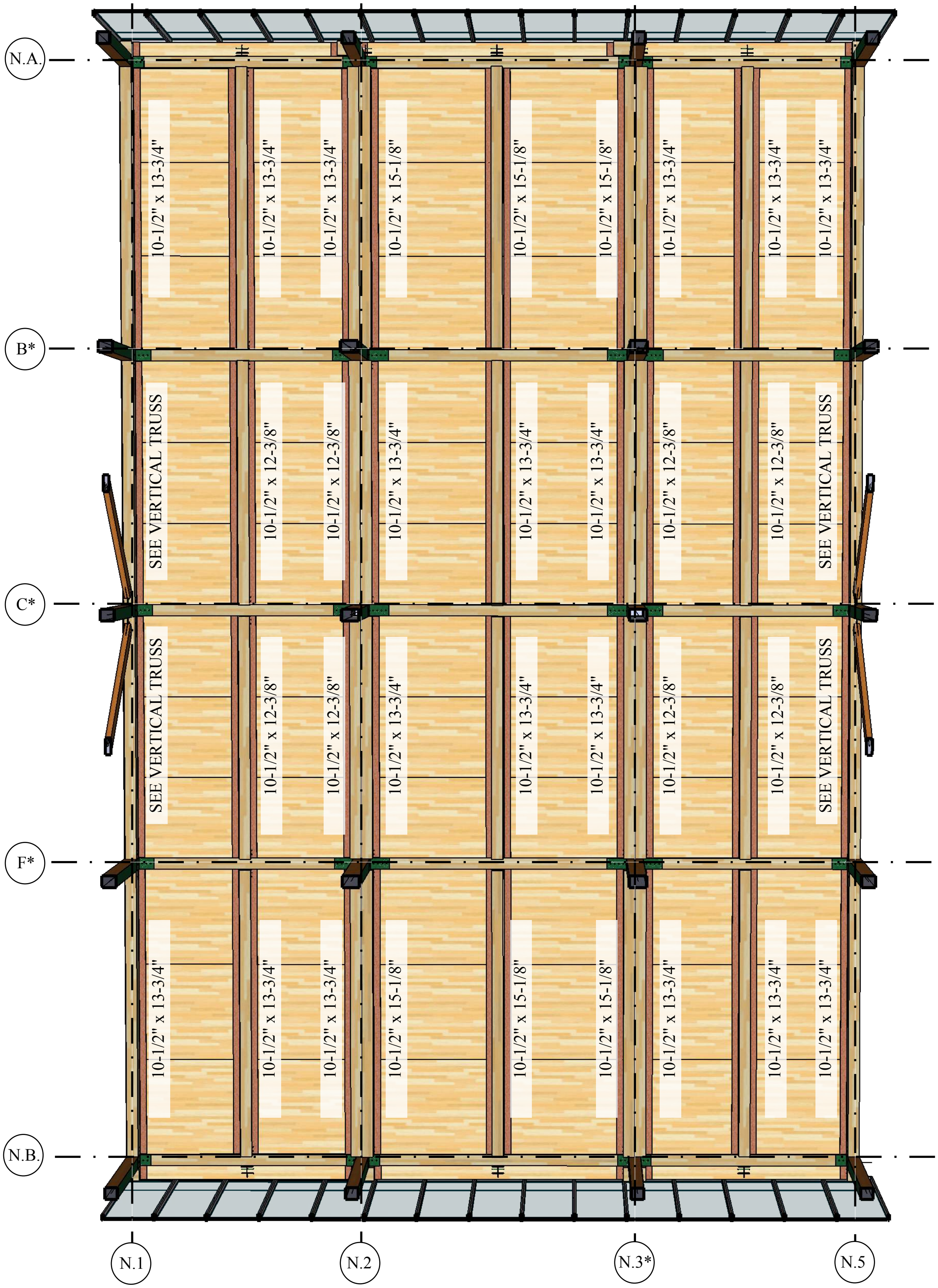
All bolts are 1" diameter with 3" edge spacing and 4" spacing between bolts along the longitdinal direction of the member.

LiUNA Expansion Building	Vertical Truss Connections
Josh Jaskowiak	
Penn State University, Department of Architectural Enigneering, Senior Thesis	



All beams shown on column lines N.B., F*, C*, B*, and N.A. are within a moment frame. See moment frame 4 for information regarding interior frames and momnet frame 5 for information regarding exterior frames.

LiUNA Expansion Building	Typical Floor
Josh Jaskowiak	
Penn State University, Department of Architectural Enigneering, Senior Thesis	



All beams shown on column lines N.B., F*, C*, B*, and N.A. are within a moment frame. See moment frame 4 for information regarding interior frames and momnet frame 5 for information regarding exterior frames.

LiUNA Expansion Building	Penthouse Floor
Josh Jaskowiak	
Penn State University, Department of Architectural Enigneering, Senior Thesis	

Appendix B: CLT Floor System Calculations

LiUNA Headquarters Expansion Building Re-Design

Document Name: **CLT Floor System Calculations**

CLT Deck Design Calculation

		Loading					Flexural Des		
		Span	Dead	Snow	Live	w	M=wl^2/8	Duration Factor	$F_bS_{eff\ req'd} > M_b/C_d$
Floor	CLT Deck Grid	Ft	PSF	PSF	PSF	PLF	Ft-Lb/LF	C_D	lbf-ft/lb
Penthouse Roof	A1	8.54	180	30	0	210	1914.45	1	1914.45
	A2	10.625	180	30	0	210	2963.38	1	2963.38
	A3	8.96	180	30	0	210	2107.39	1	2107.39
	B1	8.54	180	30	0	210	1914.45	1	1914.45
	B2	10.625	180	30	0	210	2963.38	1	2963.38
	B3	8.96	180	30	0	210	2107.39	1	2107.39
	C1	8.54	180	30	0	210	1914.45	1	1914.45
	C2	10.625	180	30	0	210	2963.38	1	2963.38
	C3	8.96	180	30	0	210	2107.39	1	2107.39
	D1	8.54	180	30	0	210	1914.45	1	1914.45
	D2	10.625	180	30	0	210	2963.38	1	2963.38
	D3	8.96	180	30	0	210	2107.39	1	2107.39
Typical Floor 2-9	A1	17.08	35		65	100	3646.58	1	3646.58
	A2	21.25	35		65	100	5644.53	1	5644.53
	A3	17.92	35		65	100	4014.08	1	4014.08
	B1	17.08	35		65	100	3646.58	1	3646.58
	B2	21.25	35		65	100	5644.53	1	5644.53
	B3	17.92	35		65	100	4014.08	1	4014.08
	C1	17.08	35		65	100	3646.58	1	3646.58
	C2	21.25	35		65	100	5644.53	1	5644.53
	C3	17.92	35		65	100	4014.08	1	4014.08
	D1	17.08	35		65	100	3646.58	1	3646.58
	D2	21.25	35		65	100	5644.53	1	5644.53
	D3	17.92	35		65	100	4014.08	1	4014.08

ign		CLT Selected				Sho
F_bS_{eff} Selected	Acceptable?	Section Properties		Material Properties		LL Deflection
lbf-ft/lb	F_bS_{eff} Req'd < F_bS_{eff} Select	Type	Thk. (in)	EI_{eff}	GA_{Eff}	in
18400	yes	E4	9.625	1090000000	1600000	0.00
18400	yes	E4	9.625	1090000000	1600000	0.00
18400	yes	E4	9.625	1090000000	1600000	0.00
18400	yes	E4	9.625	1090000000	1600000	0.00
18400	yes	E4	9.625	1090000000	1600000	0.00
18400	yes	E4	9.625	1090000000	1600000	0.00
18400	yes	E4	9.625	1090000000	1600000	0.00
18400	yes	E4	9.625	1090000000	1600000	0.00
18400	yes	E4	9.625	1090000000	1600000	0.00
18400	yes	E4	9.625	1090000000	1600000	0.00
18400	yes	E4	9.625	1090000000	1600000	0.00
18400	yes	E4	9.625	1090000000	1600000	0.00
18,400	yes	E4	9.625	1090000000	1600000	0.14
18,400	yes	E4	9.625	1090000000	1600000	0.31
18,400	yes	E4	9.625	1090000000	1600000	0.16
18,400	yes	E4	9.625	1090000000	1600000	0.14
18,400	yes	E4	9.625	1090000000	1600000	0.31
18,400	yes	E4	9.625	1090000000	1600000	0.16
18,400	yes	E4	9.625	1090000000	1600000	0.14
18,400	yes	E4	9.625	1090000000	1600000	0.31
18,400	yes	E4	9.625	1090000000	1600000	0.16
18,400	yes	E4	9.625	1090000000	1600000	0.14
18,400	yes	E4	9.625	1090000000	1600000	0.31
18,400	yes	E4	9.625	1090000000	1600000	0.16

rt-Term Live Load Deflection		Short-Term Dead and Live Load Deflection		
Deflection Criteria	Acceptable	Total Max	Deflection Criteria	Acceptable
L/360	LL Def < Criteria ?	in	L/240	Total Defl < Criteria?
0.28	yes	0.04	0.28	yes
0.35	yes	0.08	0.35	yes
0.30	yes	0.05	0.30	yes
0.28	yes	0.04	0.28	yes
0.35	yes	0.08	0.35	yes
0.30	yes	0.05	0.30	yes
0.28	yes	0.04	0.28	yes
0.35	yes	0.08	0.35	yes
0.30	yes	0.05	0.30	yes
0.28	yes	0.04	0.28	yes
0.35	yes	0.08	0.35	yes
0.30	yes	0.05	0.30	yes
0.57	yes	0.21	0.57	yes
0.71	yes	0.47	0.71	yes
0.60	yes	0.25	0.60	yes
0.57	yes	0.21	0.57	yes
0.71	yes	0.47	0.71	yes
0.60	yes	0.25	0.60	yes
0.57	yes	0.21	0.57	yes
0.71	yes	0.47	0.71	yes
0.60	yes	0.25	0.60	yes
0.57	yes	0.21	0.57	yes
0.71	yes	0.47	0.71	yes
0.60	yes	0.25	0.60	yes

Creep Deflection = 2.0*Dead Load Deflec. + Live Load Deflec.				
2.0 Dead Deflection	Live Load Deflection	Total Creep Deflection	Deflection Criteria	
in	in	in	L/240	L/360
0.07	0.00	0.07	0.43	0.28
0.14	0.00	0.14	0.53	0.35
0.08	0.00	0.08	0.45	0.30
0.07	0.00	0.07	0.43	0.28
0.14	0.00	0.14	0.53	0.35
0.08	0.00	0.08	0.45	0.30
0.07	0.00	0.07	0.43	0.28
0.14	0.00	0.14	0.53	0.35
0.08	0.00	0.08	0.45	0.30
0.07	0.00	0.07	0.43	0.28
0.14	0.00	0.14	0.53	0.35
0.08	0.00	0.08	0.45	0.30
0.15	0.14	0.28	0.85	0.57
0.33	0.31	0.64	1.06	0.71
0.17	0.16	0.34	0.90	0.60
0.15	0.14	0.28	0.85	0.57
0.33	0.31	0.64	1.06	0.71
0.17	0.16	0.34	0.90	0.60
0.15	0.14	0.28	0.85	0.57
0.33	0.31	0.64	1.06	0.71
0.17	0.16	0.34	0.90	0.60
0.15	0.14	0.28	0.85	0.57
0.33	0.31	0.64	1.06	0.71
0.17	0.16	0.34	0.90	0.60

Acceptable		EI_{APP} Calculation		
Less than L/240?	Less than L/360?	K_S	EI_{EFF}	GA_{EFF}
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000
yes	yes	11.5	1090000000	1600000

Floor Vibration Design				
		Specific Gravity		Natural Frequency
L^2	EI_{APP}	ρ	A	f (Hz)
10502.15	624291881.2	0.55	115.5	47.02
16256.25	735527305.7	0.55	115.5	32.98
11560.55	649706027.5	0.55	115.5	43.58
10502.15	624291881.2	0.55	115.5	47.02
16256.25	735527305.7	0.55	115.5	32.98
11560.55	649706027.5	0.55	115.5	43.58
10502.15	624291881.2	0.55	115.5	47.02
16256.25	735527305.7	0.55	115.5	32.98
11560.55	649706027.5	0.55	115.5	43.58
10502.15	624291881.2	0.55	115.5	47.02
16256.25	735527305.7	0.55	115.5	32.98
11560.55	649706027.5	0.55	115.5	43.58
42008.60	918672576.7	0.55	115.5	14.26
65025.00	972795196.2	0.55	115.5	9.48
46242.20	932085625.8	0.55	115.5	13.05
42008.60	918672576.7	0.55	115.5	14.26
65025.00	972795196.2	0.55	115.5	9.48
46242.20	932085625.8	0.55	115.5	13.05
42008.60	918672576.7	0.55	115.5	14.26
65025.00	972795196.2	0.55	115.5	9.48
46242.20	932085625.8	0.55	115.5	13.05
42008.60	918672576.7	0.55	115.5	14.26
65025.00	972795196.2	0.55	115.5	9.48
46242.20	932085625.8	0.55	115.5	13.05

Minimum Frequency	Acceptable	Maximum Span	Acceptable?	Org. CLT Thk.	Char Depth
Hz	Natural < Maximum?	Ft	Actual Span < Max Span	in.	in
9	yes	18.88436105	Yes	9.625	3.2
9	yes	19.81377155	Yes	9.625	3.2
9	yes	19.10643906	Yes	9.625	3.2
9	yes	18.88436105	Yes	9.625	3.2
9	yes	19.81377155	Yes	9.625	3.2
9	yes	19.10643906	Yes	9.625	3.2
9	yes	18.88436105	Yes	9.625	3.2
9	yes	19.81377155	Yes	9.625	3.2
9	yes	19.10643906	Yes	9.625	3.2
9	yes	18.88436105	Yes	9.625	3.2
9	yes	19.81377155	Yes	9.625	3.2
9	yes	19.10643906	Yes	9.625	3.2
9	yes	21.14753748	Yes	9.625	3.2
9	yes	21.50522438	Yes	9.625	3.2
9	yes	21.23754223	Yes	9.625	3.2
9	yes	21.14753748	Yes	9.625	3.2
9	yes	21.50522438	Yes	9.625	3.2
9	yes	21.23754223	Yes	9.625	3.2
9	yes	21.14753748	Yes	9.625	3.2
9	yes	21.50522438	Yes	9.625	3.2
9	yes	21.23754223	Yes	9.625	3.2
9	yes	21.14753748	Yes	9.625	3.2
9	yes	21.50522438	Yes	9.625	3.2
9	yes	21.23754223	Yes	9.625	3.2

Fire Protection Design				
$S_{EFF}F_b$ At Char Depth	$S_{EFF}F_b$ Beyond Char Depth	$S_{EFF}F_{bf}'$ at Char Depth	$S_{EFF}F_{bf}'$ Beyond Char Dept	$F_bS_{eff\ req'd} > M_b/C_d$
PLF-FT	PLF-FT	PLF-Ft	PLF-Ft	lbf-ft/lb
8616	4277	24555.6	4277	1914.45
8616	4277	24555.6	4277	2963.38
8616	4277	24555.6	4277	2107.39
8616	4277	24555.6	4277	1914.45
8616	4277	24555.6	4277	2963.38
8616	4277	24555.6	4277	2107.39
8616	4277	24555.6	4277	1914.45
8616	4277	24555.6	4277	2963.38
8616	4277	24555.6	4277	2107.39
8616	4277	24555.6	4277	1914.45
8616	4277	24555.6	4277	2963.38
8616	4277	24555.6	4277	2107.39
8616	4277	24555.6	4277	3646.58
8616	4277	24555.6	4277	5644.53
8616	4277	24555.6	4277	4014.08
8616	4277	24555.6	4277	3646.58
8616	4277	24555.6	4277	5644.53
8616	4277	24555.6	4277	4014.08
8616	4277	24555.6	4277	3646.58
8616	4277	24555.6	4277	5644.53
8616	4277	24555.6	4277	4014.08
8616	4277	24555.6	4277	3646.58
8616	4277	24555.6	4277	5644.53
8616	4277	24555.6	4277	4014.08

		Steel Angle Saddle Design						
		Dead Load	Live Load	Span	ASD Load	LRFD Load	ASD V	LRFD V
Dem< Cap at Char?	Dem < Cap Beyond Char?	PSF	PSF	Ft.	PLF	PLF	wl/2	wl/2
yes	yes	180	0	8.54	180	216	768.6	922.32
yes	yes	180	0	10.625	180	216	956.25	1147.5
yes	yes	180	0	8.96	180	216	806.4	967.68
yes	yes	180	0	8.54	180	216	768.6	922.32
yes	yes	180	0	10.625	180	216	956.25	1147.5
yes	yes	180	0	8.96	180	216	806.4	967.68
yes	yes	180	0	8.54	180	216	768.6	922.32
yes	yes	180	0	10.625	180	216	956.25	1147.5
yes	yes	180	0	8.96	180	216	806.4	967.68
yes	yes	180	0	8.54	180	216	768.6	922.32
yes	yes	180	0	10.625	180	216	956.25	1147.5
yes	yes	180	0	8.96	180	216	806.4	967.68
yes	yes	35	65	17.08	100	146	854	1246.84
yes	no	35	65	21.25	100	146	1062.5	1551.25
yes	yes	35	65	17.92	100	146	896	1308.16
yes	yes	35	65	17.08	100	146	854	1246.84
yes	no	35	65	21.25	100	146	1062.5	1551.25
yes	yes	35	65	17.92	100	146	896	1308.16
yes	yes	35	65	17.08	100	146	854	1246.84
yes	no	35	65	21.25	100	146	1062.5	1551.25
yes	yes	35	65	17.92	100	146	896	1308.16
yes	yes	35	65	17.08	100	146	854	1246.84
yes	no	35	65	21.25	100	146	1062.5	1551.25
yes	yes	35	65	17.92	100	146	896	1308.16

Appendix C: Gravity Girder Design Calculations

LiUNA Headquarters Expansion Building Re-Design	
Document Name:	Gravity Girder Design

Girders Supporting CLT Decks

		Input Information				Loads		Flexural Demands					Beam Design								
						Live load	Dead Load	w	Shear	Moment	f_b of Girder	Section Modulus Required	S chosen	Depth	Width	Cv Factors				E of Beam	I of Beam
Floor	Girder ID	Span	Actual Span	Trib Width	Tribuatry Area	PSF	PSF	PLF	wl/2	wl^2/8	psi	in^3	in^3	in	in	Factor 1	Factor 2	Factor 3	Cv	psi	in^4
Penthouse Roof	A1	22.50	21.63	4.27	96.08	30	180.00	896.70	10087.88	56744.30	2400.00	283.72	559.2	17.875	10.50	0.996556	0.980272	0.964773	0.942484	1800000	4997
	A12	22.50	21.63	4.27	96.08	30	180.00	896.70	10087.88	56744.30	2400.00	283.72	559.2	17.875	10.50	0.996556	0.980272	0.964773	0.942484	1800000	4997
	A2L	22.50	21.63	4.27	96.08	30	180.00	896.70	10087.88	56744.30	2400.00	283.72	559.2	17.875	10.50	0.996556	0.980272	0.964773	0.942484	1800000	4997
	A2R	22.50	21.63	5.31	119.53	30	180.00	1115.63	12550.78	70598.14	2400.00	352.99	648.5	19.25	10.50	0.996556	0.976647	0.964773	0.938998	1800000	6242
	A23	22.50	21.63	5.31	119.53	30	180.00	1115.63	12550.78	70598.14	2400.00	352.99	648.5	19.25	10.50	0.996556	0.976647	0.964773	0.938998	1800000	6242
	A3L	22.50	21.63	5.31	119.53	30	180.00	1115.63	12550.78	70598.14	2400.00	352.99	648.5	19.25	10.50	0.996556	0.976647	0.964773	0.938998	1800000	6242
	A3R	22.50	21.63	4.48	100.80	30	180.00	940.80	10584.00	59535.00	2400.00	297.68	559.2	17.875	10.50	0.996556	0.980272	0.964773	0.942484	1800000	4997
	A34	22.50	21.63	4.48	100.80	30	180.00	940.80	10584.00	59535.00	2400.00	297.68	559.2	17.875	10.50	0.996556	0.980272	0.964773	0.942484	1800000	4997
	A4	22.50	21.63	4.48	100.80	30	180.00	940.80	10584.00	59535.00	2400.00	297.68	559.2	17.875	10.50	0.996556	0.980272	0.964773	0.942484	1800000	4997
	B1	19.75	18.88	4.27	84.33	30	180.00	896.70	8854.91	43721.13	2400.00	218.61	476.4	16.5	10.50	1.003073	0.984203	0.964773	0.952451	1800000	4997
	B12	19.75	18.88	4.27	84.33	30	180.00	896.70	8854.91	43721.13	2400.00	218.61	476.4	16.5	10.50	1.003073	0.984203	0.964773	0.952451	1800000	4997
	B2L	19.75	18.88	4.27	84.33	30	180.00	896.70	8854.91	43721.13	2400.00	218.61	476.4	16.5	10.50	1.003073	0.984203	0.964773	0.952451	1800000	3931
	B2R	19.75	18.88	5.31	104.92	30	180.00	1115.63	11016.80	54395.43	2400.00	271.98	559.2	17.875	10.50	1.003073	0.980272	0.964773	0.948647	1800000	4997
	B23	19.75	18.88	5.31	104.92	30	180.00	1115.63	11016.80	54395.43	2400.00	271.98	559.2	17.875	10.50	1.003073	0.980272	0.964773	0.948647	1800000	4997
	B3L	19.75	18.88	5.31	104.92	30	180.00	1115.63	11016.80	54395.43	2400.00	271.98	559.2	17.875	10.50	1.003073	0.980272	0.964773	0.948647	1800000	4997
	B3R	19.75	18.88	4.48	88.48	30	180.00	940.80	9290.40	45871.35	2400.00	229.36	476.4	16.5	10.50	1.003073	0.984203	0.964773	0.952451	1800000	3931
	B34	19.75	18.88	4.48	88.48	30	180.00	940.80	9290.40	45871.35	2400.00	229.36	476.4	16.5	10.50	1.003073	0.984203	0.964773	0.952451	1800000	3931
	B4	19.75	18.88	4.48	88.48	30	180.00	940.80	9290.40	45871.35	2400.00	229.36	476.4	16.5	10.50	1.003073	0.984203	0.964773	0.952451	1800000	3931
	C1	19.75	18.88	4.27	84.33	30	180.00	896.70	8854.91	43721.13	2400.00	218.61	476.4	16.5	10.50	1.003073	0.984203	0.964773	0.952451	1800000	3931
	C12	19.75	18.88	4.27	84.33	30	180.00	896.70	8854.91	43721.13	2400.00	218.61	476.4	16.5	10.50	1.003073	0.984203	0.964773	0.952451	1800000	3931
	C2L	19.75	18.88	4.27	84.33	30	180.00	896.70	8854.91	43721.13	2400.00	218.61	476.4	16.5	10.50	1.003073	0.984203	0.964773	0.952451	1800000	3931
	C2R	19.75	18.88	5.31	104.92	30	180.00	1115.63	11016.80	54395.43	2400.00	271.98	559.2	17.875	10.50	1.003073	0.980272	0.964773	0.948647	1800000	4997
	C23	19.75	18.88	5.31	104.92	30	180.00	1115.63	11016.80	54395.43	2400.00	271.98	559.2	17.875	10.50	1.003073	0.980272	0.964773	0.948647	1800000	4997
	C3L	19.75	18.88	5.31	104.92	30	180.00	1115.63	11016.80	54395.43	2400.00	271.98	559.2	17.875	10.50	1.003073	0.980272	0.964773	0.948647	1800000	4997
	C3R	19.75	18.88	4.48	88.48	30	180.00	940.80	9290.40	45871.35	2400.00	229.36	476.4	16.5	10.50	1.003073	0.984203	0.964773	0.952451	1800000	3931
	C34	19.75	18.88	4.48	88.48	30	180.00	940.80	9290.40	45871.35	2400.00	229.36	476.4	16.5	10.50	1.003073	0.984203	0.964773	0.952451	1800000	3931
	C4	19.75	18.88	4.48	88.48	30	180.00	940.80	9290.40	45871.35	2400.00	229.36	476.4	16.5	10.50	1.003073	0.984203	0.964773	0.952451	1800000	3931
	D1	23.33	22.46	4.27	99.62	30	180.00	896.70	10460.01	61007.98	2400.00	305.04	648.5	19.25	10.50	0.994753	0.976647	0.964773	0.937299	1800000	6242
	D12	23.33	22.46	4.27	99.62	30	180.00	896.70	10460.01	61007.98	2400.00	305.04	648.5	19.25	10.50	0.994753	0.976647	0.964773	0.937299	1800000	6242
	D2L	23.33	22.46	4.27	99.62	30	180.00	896.70	10460.01	61007.98	2400.00	305.04	648.5	19.25	10.50	0.994753	0.976647	0.964773	0.937299	1800000	6242
	D2R	23.33	22.46	5.31	123.94	30	180.00	1115.63	13013.77	75902.79	2400.00	379.51	744.4	20.625	10.50	0.994753	0.973284	0.964773	0.934071	1800000	7677
	D23	23.33	22.46	5.31	123.94	30	180.00	1115.63	13013.77	75902.79	2400.00	379.51	744.4	20.625	10.50	0.994753	0.973284	0.964773	0.934071	1800000	7677
	D3L	23.33	22.46	5.31	123.94	30	180.00	1115.63	13013.77	75902.79	2400.00	379.51	744.4	20.625	10.50	0.994753	0.973284	0.964773	0.934071	1800000	7677
	D3R	23.33	22.46	4.48	104.52	30	180.00	940.80	10974.43	64008.37	2400.00	320.04	648.5	19.25	10.50	0.994753	0.976647	0.964773	0.937299	1800000	6242
	D34	23.33	22.46	4.48	104.52	30	180.00	940.80	10974.43	64008.37	2400.00	320.04	648.5	19.25	10.50	0.994753	0.976647	0.964773	0.937299	1800000	6242
	D4	23.33	22.46	4.48	104.52	30	180.00	940.80	10974.43	64008.37	2400.00	320.04	648.5	19.25	10.50	0.994753	0.976647	0.964773	0.937299	1800000	6242
Typical Floor 2-9	A1	22.50	21.63	8.54	192.15	65	35.00	854.00	9607.50	54042.19	2400.00	270.21	476.4	16.5	10.5	0.996556	0.984203	0.964773	0.946263	1800000	3931
	A2L	22.50	21.63	8.54	192.15	65	35.00	854.00	9607.50	54042.19	2400.00	270.21	476.4	16.5	10.5	0.996556	0.984203	0.964773	0.946263	1800000	3931
	A2R	22.50	21.63	10.63	239.06	65	35.00	1062.50	11953.13	67236.33	2400.00	336.18	559.2	17.785	10.5	0.996556	0.98052	0.964773	0.942722	1800000	4997
	A3L	22.50	21.63	10.63	239.06	65	35.00	1062.50	11953.13	67236.33	2400.00	336.18	559.2	17.785	10.5	0.996556	0.98052	0.964773	0.942722	1800000	4997
	A3R	22.50	21.63	8.96	201.60	65	35.00	896.00	10080.00	56700.00	2400.00	283.50	476.4	16.5	10.5	0.996556	0.984203	0.964773	0.946263	1800000	3931
	A4	22.50	21.63	8.96	201.60	65	35.00	896.00	10080.00	56700.00	2400.00	283.50	476.4	16.5	10.5	0.996556	0.984203	0.964773	0.946263	1800000	3931
	B1	19.75	18.88	8.54	168.67	65	35.00	854.00	8433.25	41639.17	2400.00	208.20	400.3	15.125	10.5	1.003073	0.988495	0.964773	0.956604	1800000	3028
	B2L	19.75	18.88	8.54	168.67	65	35.00	854.00	8433.25	41639.17	2400.00	208.20	400.3	15.125	10.5	1.003073	0.988495	0.964773	0.956604	1800000	3028
	B2R	19.75	18.88	10.63	209.84	65	35.00	1062.50	10492.19	51805.18	2400.00	259.03	400.3	15.125	10.5	1.003073	0.988495	0.964773	0.956604	1800000	3028
	B3L	19.75	18.88	10.63	209.84	65	35.00	1062.50	10492.19	51805.18	2400.00	259.03	400.3	15.125	10.5	1.003073	0.988495	0.964773	0.956604	1800000	3028
	B3R	19.75	18.88	8.96	176.96	65	35.00	896.00	8848.00	43687.00	2400.00	218.44	400.3	15.125	10.5	1.003073	0.988495	0.964773	0.956604	1800000	3028
	B4	19.75	18.88	8.96	176.96	65	35.00	896.00	8848.00	43687.00	2400.00	218.44	400.3	15.125	10.5	1.003073	0.988495	0.964773	0.956604	1800000	3028
	C1	19.75	18.88	8.54	168.67	65	35.00	854.00	8433.25	41639.17	2400.00	208.20	400.3	15.125	10.5	1.003073	0.988495	0.964773	0.956604	1800000	3028
	C2L	19.75	18.88	8.54	168.67	65	35.00	854.00	8433.25	41639.17	2400.00	208.20	400.3	15.125	10.5	1.003073	0.988495	0.964773	0.956604	1800000	3028
	C2R	19.75	18.88	10.63	209.84	65	35.00	1062.50	10492.19	51805.18	2400.00	259.03	400.3	15.125	10.5	1.003073	0.988495	0.964773	0.956604	1800000	3028
	C3L	19.75	18.88	10.63	209.84	65	35.00	1062.50	10492.19	51805.18	2400.00	259.03	400.3	15.125	10.5	1.003073	0.988495	0.964773	0.956604	1800000	3028
	C3R	19.75	18.88	8.96	176.96	65	35.00	896.00	8848.00	43687.00	2400.00	218.44	400.3	15.125	10.5	1.003073	0.988495	0.964773	0.956604	1800000	3028
	C4	19.75	18.88	8.96	176.96	65	35.00	896.00	8848.00	43687.00	2400.00	218.44	400.3	15.125	10.5	1.003073	0.988495	0.964773	0.956604	1800000	3028
	D1	23.33	22.46	8.54	199.24	65	35.00	854.00	9961.91	58102.84	2400.00	290.51	476.4	16.5	10.5	0.994753	0.984203	0.964773	0.944551	1800000	3931
	D2L	23.33	22.46	8.54	199.24	65	35.00	854.00	9961.91	58102.84	2400.00	290.51	476.4	16.5	10.5	0.994753	0.984203	0.964773	0.944551	1800000	3931
	D2R	23.33	22.46	10.63	247.88	65	35.00	1062.50	12394.06	72288.37	2400.00	361.44	559.2	17.875	10.5	0.994753	0.980272	0.964773	0.940778	1800000	4997
	D3L	23.33	22.46	10.63	247.88	65	35.00	1062.50	12394.06	72288.37	2400.00	361.44	559.2	17.875	10.5	0.994753	0.980272	0.964773	0.940778	1800000	4997
	D3R	23.33	22.46	8.96	209.04	65	35.00	896.00	10451.84	60960.36	2400.00	304.80	476.4	16.5	10.5	0.994753	0.984203	0.964773	0.944551	1800000	3931
	D4	23.33	22.46	8.96	209.04	65	35.00	896.00	10451.84	60960.36	2400.00	304.80	476.4	16.5	10.5	0.994753	0.984203	0.964773	0.944551	1800000	3931
	E1	2.00	1.13	8.54	17.08	65	35.00	854.00	854.00	427.00	2400.00	2.14	330.9	13.75	10.5	1.124759	0.993217	0.964773	1.077776	1800000	3028
	E2L	2.00	1.13	8.54	17.08	65	35.00	854.00	854.00	427.00	2400.00	2.14	330.9	13.75	10.5	1.124759	0.993217	0.964773	1.077776	1800000	3028
	E2R	2.00	1.13	10.63	21.25	65	35.00	1062.50	1062.50	531.25	2400.00	2.66	330.9	13.75	10.5	1.124759	0.993217	0.964773	1.077776	1800000	3028
	E3L	2.00	1.13	10.63	21.25	65	35.00	1062.50	1062.50	531.25	2400.00	2.66	330.9	13.75	10.5	1.124759	0.993217	0.964773	1.077776	1800000	3028
	E3R	2.00	1.13	8.96	17.92	65	35.00	896.00	896.00	448.00	2400.00	2.24	330.9	13.75	10.5	1.124759	0.993217	0.964773	1.077776	1800000	3028
	E4	2.00	1.13	8.96	17.92	65	35.00	896.00	896.00	448.00	2400.00	2.24	330.9	13.75	10.5	1.124759	0.993217	0.964773	1.077776	1800000	3028

Deflection Calculations								Fire Design						
Live Load	Short Term Deflection	Dead Load	Long Term Deflection	K_{CR}	Total Creep Deflecton	Criteria	Acceptable?	Char Depth	Orignal Girder Depth	Depth at Char	Original Girder Width	Char Section Modulus	F_b at Char	Flexural Capacity
PLF	in	PLF	in			L/360	Creep<Allowable?	in	in	in	in	in^3	psi	Ft-lbs
128.1	0.070076799	768.6	0.420460796	1.5	0.700767993	0.75	Yes	3.2	17.875	14.675	10.50	154.0875	6840.00	87829.875
128.1	0.070076799	768.6	0.420460796	1.5	0.700767993	0.75	Yes	3.2	17.875	14.675	10.50	154.0875	6840.00	87829.875
128.1	0.070076799	768.6	0.420460796	1.5	0.700767993	0.75	Yes	3.2	17.875	14.675	10.50	154.0875	6840.00	87829.875
159.375	0.069796061	956.25	0.418776364	1.5	0.697960606	0.75	Yes	3.2	19.25	16.05	10.50	168.525	6840.00	96059.25
159.375	0.069796061	956.25	0.418776364	1.5	0.697960606	0.75	Yes	3.2	19.25	16.05	10.50	168.525	6840.00	96059.25
159.375	0.069796061	956.25	0.418776364	1.5	0.697960606	0.75	Yes	3.2	19.25	16.05	10.50	168.525	6840.00	96059.25
134.4	0.073523199	806.4	0.441139196	1.5	0.735231993	0.75	Yes	3.2	17.875	14.675	10.50	154.0875	6840.00	87829.875
134.4	0.073523199	806.4	0.441139196	1.5	0.735231993	0.75	Yes	3.2	17.875	14.675	10.50	154.0875	6840.00	87829.875
134.4	0.073523199	806.4	0.441139196	1.5	0.735231993	0.75	Yes	3.2	17.875	14.675	10.50	154.0875	6840.00	87829.875
128.1	0.040672197	768.6	0.244033183	1.5	0.406721972	0.658333333	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
128.1	0.040672197	768.6	0.244033183	1.5	0.406721972	0.658333333	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
128.1	0.051701595	768.6	0.310209569	1.5	0.517015949	0.658333333	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
159.375	0.050602119	956.25	0.303612713	1.5	0.506021189	0.658333333	Yes	3.2	17.875	14.675	10.50	154.0875	6840.00	87829.875
159.375	0.050602119	956.25	0.303612713	1.5	0.506021189	0.658333333	Yes	3.2	17.875	14.675	10.50	154.0875	6840.00	87829.875
159.375	0.050602119	956.25	0.303612713	1.5	0.506021189	0.658333333	Yes	3.2	17.875	14.675	10.50	154.0875	6840.00	87829.875
134.4	0.054244296	806.4	0.325465777	1.5	0.542442962	0.658333333	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
134.4	0.054244296	806.4	0.325465777	1.5	0.542442962	0.658333333	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
134.4	0.054244296	806.4	0.325465777	1.5	0.542442962	0.658333333	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
128.1	0.051701595	768.6	0.310209569	1.5	0.517015949	0.658333333	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
128.1	0.051701595	768.6	0.310209569	1.5	0.517015949	0.658333333	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
128.1	0.051701595	768.6	0.310209569	1.5	0.517015949	0.658333333	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
159.375	0.050602119	956.25	0.303612713	1.5	0.506021189	0.658333333	Yes	3.2	17.875	14.675	10.50	154.0875	6840.00	87829.875
159.375	0.050602119	956.25	0.303612713	1.5	0.506021189	0.658333333	Yes	3.2	17.875	14.675	10.50	154.0875	6840.00	87829.875
159.375	0.050602119	956.25	0.303612713	1.5	0.506021189	0.658333333	Yes	3.2	17.875	14.675	10.50	154.0875	6840.00	87829.875
134.4	0.054244296	806.4	0.325465777	1.5	0.542442962	0.658333333	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
134.4	0.054244296	806.4	0.325465777	1.5	0.542442962	0.658333333	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
134.4	0.054244296	806.4	0.325465777	1.5	0.542442962	0.658333333	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
128.1	0.065221024	768.6	0.391326147	1.5	0.652210244	0.777666667	Yes	3.2	19.25	16.05	10.50	168.525	6840.00	96059.25
128.1	0.065221024	768.6	0.391326147	1.5	0.652210244	0.777666667	Yes	3.2	19.25	16.05	10.50	168.525	6840.00	96059.25
128.1	0.065221024	768.6	0.391326147	1.5	0.652210244	0.777666667	Yes	3.2	19.25	16.05	10.50	168.525	6840.00	96059.25
159.375	0.065976748	956.25	0.39586049	1.5	0.659767484	0.777666667	Yes	3.2	20.625	17.425	10.50	182.9625	6840.00	104288.625
159.375	0.065976748	956.25	0.39586049	1.5	0.659767484	0.777666667	Yes	3.2	20.625	17.425	10.50	182.9625	6840.00	104288.625
159.375	0.065976748	956.25	0.39586049	1.5	0.659767484	0.777666667	Yes	3.2	20.625	17.425	10.50	182.9625	6840.00	104288.625
134.4	0.068428616	806.4	0.410571695	1.5	0.684286158	0.777666667	Yes	3.2	19.25	16.05	10.50	168.525	6840.00	96059.25
134.4	0.068428616	806.4	0.410571695	1.5	0.684286158	0.777666667	Yes	3.2	19.25	16.05	10.50	168.525	6840.00	96059.25
134.4	0.068428616	806.4	0.410571695	1.5	0.684286158	0.777666667	Yes	3.2	19.25	16.05	10.50	168.525	6840.00	96059.25
555.1	0.386013649	298.9	0.207853503	1.5	0.697793904	0.75	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
555.1	0.386013649	298.9	0.207853503	1.5	0.697793904	0.75	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
690.625	0.377804758	371.875	0.203433331	1.5	0.682954756	0.75	Yes	3.2	17.785	14.585	10.50	153.1425	6840.00	87291.225
690.625	0.377804758	371.875	0.203433331	1.5	0.682954756	0.75	Yes	3.2	17.785	14.585	10.50	153.1425	6840.00	87291.225
582.4	0.404997927	313.6	0.218075807	1.5	0.732111637	0.75	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
582.4	0.404997927	313.6	0.218075807	1.5	0.732111637	0.75	Yes	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
555.1	0.290852774	298.9	0.156613032	1.5	0.525772323	0.658333333	Yes	3.2	15.125	11.925	10.50	125.2125	6840.00	71371.125
555.1	0.290852774	298.9	0.156613032	1.5	0.525772323	0.658333333	Yes	3.2	15.125	11.925	10.50	125.2125	6840.00	71371.125
690.625	0.361863083	371.875	0.194849352	1.5	0.654137111	0.658333333	Yes	3.2	15.125	11.925	10.50	125.2125	6840.00	71371.125
690.625	0.361863083	371.875	0.194849352	1.5	0.654137111	0.658333333	Yes	3.2	15.125	11.925	10.50	125.2125	6840.00	71371.125
582.4	0.305157009	313.6	0.164315313	1.5	0.551629978	0.658333333	Yes	3.2	15.125	11.925	10.50	125.2125	6840.00	71371.125
582.4	0.305157009	313.6	0.164315313	1.5	0.551629978	0.658333333	Yes	3.2	15.125	11.925	10.50	125.2125	6840.00	71371.125
555.1	0.290852774	298.9	0.156613032	1.5	0.525772323	0.658333333	Yes	3.2	15.125	11.925	10.50	125.2125	6840.00	71371.125
555.1	0.290852774	298.9	0.156613032	1.5	0.525772323	0.658333333	Yes	3.2	15.125	11.925	10.50	125.2125	6840.00	71371.125
690.625	0.361863083	371.875	0.194849352	1.5	0.654137111	0.658333333	Yes	3.2	15.125	11.925	10.50	125.2125	6840.00	71371.125
690.625	0.361863083	371.875	0.194849352	1.5	0.654137111	0.658333333	Yes	3.2	15.125	11.925	10.50	125.2125	6840.00	71371.125
582.4	0.305157009	313.6	0.164315313	1.5	0.551629978	0.658333333	Yes	3.2	15.125	11.925	10.50	125.2125	6840.00	71371.125
582.4	0.305157009	313.6	0.164315313	1.5	0.551629978	0.658333333	Yes	3.2	15.125	11.925	10.50	125.2125	6840.00	71371.125
555.1	0.448776838	298.9	0.241649067	1.5	0.811250438	0.777666667	No	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
555.1	0.448776838	298.9	0.241649067	1.5	0.811250438	0.777666667	No	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
690.625	0.439233238	371.875	0.236510205	1.5	0.793998545	0.777666667	No	3.2	17.875	14.675	10.50	154.0875	6840.00	87829.875
690.625	0.439233238	371.875	0.236510205	1.5	0.793998545	0.777666667	No	3.2	17.875	14.675	10.50	154.0875	6840.00	87829.875
582.4	0.47084783	313.6	0.253533447	1.5	0.851148	0.777666667	No	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
582.4	0.47084783	313.6	0.253533447	1.5	0.851148	0.777666667	No	3.2	16.5	13.3	10.50	139.65	6840.00	79600.5
555.1	3.67059E-06	298.9	1.97647E-06	1.5	6.63529E-06	0.066666667	Yes	3.2	13.75	10.55	10.50	110.775	6840.00	63141.75
555.1	3.67059E-06	298.9	1.97647E-06	1.5	6.63529E-06	0.066666667	Yes	3.2	13.75	10.55	10.50	110.775	6840.00	63141.75
690.625	4.56674E-06	371.875	2.45902E-06	1.5	8.25527E-06	0.066666667	Yes	3.2	13.75	10.55	10.50	110.775	6840.00	63141.75
690.625	4.56674E-06	371.875	2.45902E-06	1.5	8.25527E-06	0.066666667	Yes	3.2	13.75	10.55	10.50	110.775	6840.00	63141.75
582.4	3.85111E-06	313.6	2.07367E-06	1.5	6.96162E-06	0.066666667	Yes	3.2	13.75	10.55	10.50	110.775	6840.00	63141.75
582.4	3.85111E-06	313.6	2.07367E-06	1.5	6.96162E-06	0.066666667	Yes	3.2	13.75	10.55	10.50	110.775	6840.00	63141.75

		Connection Design with LRFD						
Flexural Demand	Acceptable?				Live load	Dead Load	w= 1.2D+1.	Shear
Ft-lbs	Demand<Capacity?	Span	Trib Width	Tribuatry A	PSF	PSF	PLF	wl/2
56744.30	yes	22.50	8.54	192.15	30	165.00	2100.84	23634.45
56744.30	yes							
56744.30	yes	22.50	8.54	192.15	30	165.00	2100.84	23634.45
70598.14	yes	22.50	10.63	239.06	30	165.00	2613.75	29404.69
70598.14	yes							
70598.14	yes	22.50	10.63	239.06	30	165.00	2613.75	29404.69
59535.00	yes	22.50	8.96	201.60	30	165.00	2204.16	24796.80
59535.00	yes							
59535.00	yes	22.50	8.96	201.60	30	165.00	2204.16	24796.80
43721.13	yes	19.75	8.54	168.67	30	165.00	2100.84	20745.80
43721.13	yes							
43721.13	yes	19.75	8.54	168.67	30	165.00	2100.84	20745.80
54395.43	yes	19.75	10.63	209.84	30	165.00	2613.75	25810.78
54395.43	yes							
54395.43	yes	19.75	10.63	209.84	30	165.00	2613.75	25810.78
45871.35	yes	19.75	8.96	176.96	30	165.00	2204.16	21766.08
45871.35	yes							
45871.35	yes	19.75	8.96	176.96	30	165.00	2204.16	21766.08
43721.13	yes	19.75	8.54	168.67	30	165.00	2100.84	20745.80
43721.13	yes							
43721.13	yes	19.75	8.54	168.67	30	165.00	2100.84	20745.80
54395.43	yes	19.75	10.63	209.84	30	165.00	2613.75	25810.78
54395.43	yes							
54395.43	yes	19.75	10.63	209.84	30	165.00	2613.75	25810.78
45871.35	yes	19.75	8.96	176.96	30	165.00	2204.16	21766.08
45871.35	yes							
45871.35	yes	19.75	8.96	176.96	30	165.00	2204.16	21766.08
61007.98	yes	23.33	8.54	199.24	30	165.00	2100.84	24506.30
61007.98	yes							
61007.98	yes	23.33	8.54	199.24	30	165.00	2100.84	24506.30
75902.79	yes	23.33	10.63	247.88	30	165.00	2613.75	30489.39
75902.79	yes							
75902.79	yes	23.33	10.63	247.88	30	165.00	2613.75	30489.39
64008.37	yes	23.33	8.96	209.04	30	165.00	2204.16	25711.53
64008.37	yes							
64008.37	yes	23.33	8.96	209.04	30	165.00	2204.16	25711.53
54042.19	yes	22.50	8.54	192.15	65	35.00	1246.84	14026.95
54042.19	yes	22.50	8.54	192.15	65	35.00	1246.84	14026.95
67236.33	yes	22.50	10.63	239.06	65	35.00	1551.25	17451.56
67236.33	yes	22.50	10.63	239.06	65	35.00	1551.25	17451.56
56700.00	yes	22.50	8.96	201.60	65	35.00	1308.16	14716.80
56700.00	yes	22.50	8.96	201.60	65	35.00	1308.16	14716.80
41639.17	yes	19.75	8.54	168.67	65	35.00	1246.84	12312.55
41639.17	yes	19.75	8.54	168.67	65	35.00	1246.84	12312.55
51805.18	yes	19.75	10.63	209.84	65	35.00	1551.25	15318.59
51805.18	yes	19.75	10.63	209.84	65	35.00	1551.25	15318.59
43687.00	yes	19.75	8.96	176.96	65	35.00	1308.16	12918.08
43687.00	yes	19.75	8.96	176.96	65	35.00	1308.16	12918.08
41639.17	yes	19.75	8.54	168.67	65	35.00	1246.84	12312.55
41639.17	yes	19.75	8.54	168.67	65	35.00	1246.84	12312.55
51805.18	yes	19.75	10.63	209.84	65	35.00	1551.25	15318.59
51805.18	yes	19.75	10.63	209.84	65	35.00	1551.25	15318.59
43687.00	yes	19.75	8.96	176.96	65	35.00	1308.16	12918.08
43687.00	yes	19.75	8.96	176.96	65	35.00	1308.16	12918.08
58102.84	yes	23.33	8.54	199.24	65	35.00	1246.84	14544.39
58102.84	yes	23.33	8.54	199.24	65	35.00	1246.84	14544.39
72288.37	yes	23.33	10.63	247.88	65	35.00	1551.25	18095.33
72288.37	yes	23.33	10.63	247.88	65	35.00	1551.25	18095.33
60960.36	yes	23.33	8.96	209.04	65	35.00	1308.16	15259.69
60960.36	yes	23.33	8.96	209.04	65	35.00	1308.16	15259.69
427.00	yes	2.00	8.54	17.08	65	35.00	1246.84	1246.84
427.00	yes	2.00	8.54	17.08	65	35.00	1246.84	1246.84
531.25	yes	2.00	10.63	21.25	65	35.00	1551.25	1551.25
531.25	yes	2.00	10.63	21.25	65	35.00	1551.25	1551.25
448.00	yes	2.00	8.96	17.92	65	35.00	1308.16	1308.16
448.00	yes	2.00	8.96	17.92	65	35.00	1308.16	1308.16

Appendix D: Wind Load Story Force Calculation and Distribution

LiUNA Headquarters Expansion Building Re-Design	
Document Name:	Wind Load Calcs, Story Forces, & Distribution

Wind Load Distribution

East-West Wind Story Force Calculator (For Moment Frames)									
Trib H	Trib W	WW	LW	Total	Story Force LRFD			Story Force ASD	
(ft).	(ft)	PSF	PSF	PSF	Lbs	K		K	K
16	89.33	45.33	30.22	75.55	107982.1	107.9821		64.78926	
8.5	89.33	34.88	21.8	56.68	43037.41	43.03741	151.0195	25.82244	90.61171
14.47	89.33	33.16	21.8	54.96	71041.58	71.04158	71.04158	42.62495	42.62495
11.44	89.33	32.12	21.8	53.92	55102.75	55.10275	55.10275	33.06165	33.06165
10.94	89.33	30.4	21.8	52.2	51013.5	51.0135	51.0135	30.6081	30.6081
10.94	89.33	29.02	21.8	50.82	49664.87	49.66487	49.66487	29.79892	29.79892
10.94	89.33	27.92	21.8	49.72	48589.87	48.58987	48.58987	29.15392	29.15392
10.94	89.33	25.21	21.8	47.01	45941.47	45.94147	45.94147	27.56488	27.56488
10.94	89.33	22.45	21.8	44.25	43244.21	43.24421	43.24421	25.94652	25.94652
12.12	89.33	19.68	21.8	41.48	44909.55	44.90955	44.90955	26.94573	26.94573

		Moment Frame ASD Loads				
		MF-1	MF-2	MF-3	MF-4	MF-5
Floor	Floor Load	0.148	0.236	0.221	0.241	0.153
P	90.61171	13.4	21.4	20.0	21.8	13.9
9	42.62495	6.3	10.1	9.4	10.3	6.5
8	33.06165	4.9	7.8	7.3	8.0	5.1
7	30.6081	4.5	7.2	6.8	7.4	4.7
6	29.79892	4.4	7.0	6.6	7.2	4.6
5	29.15392	4.3	6.9	6.4	7.0	4.5
4	27.56488	4.1	6.5	6.1	6.6	4.2
3	25.94652	3.8	6.1	5.7	6.3	4.0
2	26.94573	4.0	6.4	6.0	6.5	4.1

		Moment Frame LRFD Loads				
		MF-1	MF-2	MF-3	MF-4	MF-5
Floor	Floor Load	0.148	0.236	0.221	0.241	0.153
P	151.0195	22.4	35.6	33.4	36.4	23.1
9	71.04158	10.5	16.8	15.7	17.1	10.9
8	55.10275	8.2	13.0	12.2	13.3	8.4
7	51.0135	7.5	12.0	11.3	12.3	7.8
6	49.66487	7.4	11.7	11.0	12.0	7.6
5	48.58987	7.2	11.5	10.7	11.7	7.4
4	45.94147	6.8	10.8	10.2	11.1	7.0
3	43.24421	6.4	10.2	9.6	10.4	6.6
2	44.90955	6.6	10.6	9.9	10.8	6.9

North-South Wind Story Force Calculator (For Braced Frames)									
Trib H	Trib W	WW	LW	Total	Story Force LRFD			Story Force ASD	
(ft).	(ft)	PSF	PSF	PSF	Lbs	K		K	K
16	60.08	45.33	30.22	75.55	72624.7	72.6247		43.57482	
8.5	60.08	44.42	27.76	72.18	36860.88	36.86088		22.11653	65.69135
14.47	60.08	42.22	27.76	69.98	60837.64	60.83764		36.50259	36.50259
11.44	60.08	40.9	27.76	68.66	47191.06	47.19106		28.31464	28.31464
10.94	60.08	38.7	27.76	66.46	43682.51	43.68251		26.20951	26.20951
10.94	60.08	36.95	27.76	64.71	42532.28	42.53228		25.51937	25.51937
10.94	60.08	34.75	27.76	62.51	41086.27	41.08627		24.65176	24.65176
10.94	60.08	32.1	27.76	59.86	39344.49	39.34449		23.6067	23.6067
10.94	60.08	28.59	27.76	56.35	37037.46	37.03746		22.22247	22.22247
12.12	60.08	25.06	27.76	52.82	38461.92	38.46192		23.07715	23.07715

		Braced Frame	
		BF-1	BF-2
Floor	Floor Load	0.5	0.5
P	65.69135	32.8	32.8
9	36.50259	18.3	18.3
8	28.31464	14.2	14.2
7	26.20951	13.1	13.1
6	25.51937	12.8	12.8
5	24.65176	12.3	12.3
4	23.6067	11.8	11.8
3	22.22247	11.1	11.1
2	23.07715	11.5	11.5

Appendix E: Moment Frame Beam Design Calculations

LiUNA Headquarters Expansion Building Re-Design	
Document Name:	Moment Frame Beam Design

Glulam Moment Frame Section Modulus Capacity

All beams at 10 1/2" Width

fb	cd	S	M		Depth
PSI		in^3	Lb-in	K-ft	in
2400	1.6	211.8	813312	68	11
2400	1.6	268	1029120	86	12 3/8
2400	1.6	330.9	1270656	106	13 3/4
2400	1.6	400.3	1537152	128	15 1/8
2400	1.6	476.4	1829376	152	16 1/2
2400	1.6	559.2	2147328	179	17 7/8
2400	1.6	648.5	2490240	208	19 1/4

All beams at 8 1/2" Width

fb	cd	S	M		Depth
PSI		in^3	Lb-in	K-ft	in
2400	1.6	131.2	503808	42	9 5/8
2400	1.6	171.4	658176	55	11
2400	1.6	216.9	832896	69	12 3/8
2400	1.6	267.8	1028352	86	13 3/4
2400	1.6	324.1	1244544	104	15 1/8
2400	1.6	385.7	1481088	123	16 1/2
2400	1.6	452.6	1737984	145	17 7/8
2400	1.6	525	2016000	168	19 1/4

All beams at 6 3/4" Width

fb	cd	S	M		Depth
PSI		in^3	Lb-in	K-ft	in
2400	1.6	53.17	204172.8	17	6 7/8
2400	1.6	76.57	294028.8	25	8 1/4
2400	1.6	104.2	400128	33	9 5/8
2400	1.6	136.1	522624	44	11
2400	1.6	172.3	661632	55	12 3/8
2400	1.6	212.7	816768	68	13 3/4
2400	1.6	257.4	988416	82	15 1/8
2400	1.6	306.3	1176192	98	16 1/2
2400	1.6	359.3	1379712	115	17 7/8
2400	1.6	416.9	1600896	133	19 1/4

Moment Frame 4 (Controlling MF of 2, 3, and 4)

Location of Beam

	Controlling Side Span Case			Center Span			Controlling Load Case
	Design Moment	Moment Capacity	Sized Beam	Design Moment	Moment Capacity	Sized Beam	ASD Cases Used
Floor	K-ft	K-ft		K-ft	K-ft		SAP LFRS Model 3.0
P	59	68	10 1/2 x 11	66	68	10 1/2 x 11	D+0.6W
9	80	86	10 1/2 x 12 3/8	68	86	10 1/2 x 12 3/8	D+0.6W
8	88	106	10 1/2 x 13 3/4	76	86	10 1/2 x 12 3/8	D+0.6W
7	99	106	10 1/2 x 13 3/4	84	86	10 1/2 x 12 3/8	D+0.6W
6	112	128	10 1/2 x 15 1/8	93	106	10 1/2 x 13 3/4	D+0.6W
5	126	128	10 1/2 x 15 1/8	102	106	10 1/2 x 13 3/4	D+0.6W
4	138	152	10 1/2 x 16 1/2	110	129	10 1/2 x 15 1/8	D+0.6W
3	145	152	10 1/2 x 16 1/2	114	129	10 1/2 x 15 1/8	D+0.6W
2	159	179	10 1/2 x 17 7/8	115	129	10 1/2 x 15 1/8	D+0.6W

Moment Frame 5 (Controlling MF of 1 and 5)

Location of Beam

	Controlling Side Span Case			Center Span			Controlling Load Case
	Design Moment	Moment Capacity	Sized Beam	Design Moment	Moment Capacity	Sized Beam	ASD Cases Used
Floor	K-ft	K-ft		K-ft	K-ft		SAP LFRS Model 3.0
P	20	68	10 1/2 x 11	15	68	10 1/2 x 11	D+0.6W
9	32	68	10 1/2 x 11	23	68	10 1/2 x 11	D+0.6W
8	36	68	10 1/2 x 11	27	68	10 1/2 x 11	D+0.6W
7	41	68	10 1/2 x 11	31	68	10 1/2 x 11	D+0.6W
6	46	68	10 1/2 x 11	35	68	10 1/2 x 11	D+0.6W
5	51	68	10 1/2 x 11	40	68	10 1/2 x 11	D+0.6W
4	56	68	10 1/2 x 11	43	68	10 1/2 x 11	D+0.6W
3	59	68	10 1/2 x 11	44	68	10 1/2 x 11	D+0.6W
2	66	68	10 1/2 x 11	46	68	10 1/2 x 11	D+0.6W

Appendix F: Moment Frame Column Design Calculations

LiUNA Headquarters Expansion Building Re-Design	
Document Name:	Moment Frame Column Design

Column Loading from SAP Model

		Penthouse					9th Floor					8th Floor					7th Floor					D+0
		D+0.6W		D+0.75L+0.45W			D+0.6W		D+0.75L+0.45W			D+0.6W		D+0.75L+0.45W			D+0.6W		D+0.75L+0.45W			
		P Dem.	M Dem.	P Dem.	M Dem.	Height	P Dem.	M Dem.	P Dem.	M Dem.	Height	P Dem.	M Dem.	P Dem.	M Dem.	Height	P Dem.	M Dem.	P Dem.	M Dem.	Height	P Dem.
		K	K'	K	K'	ft	K	K'	K	K'	ft	K	K'	K	K'	ft	K	K'	K	K'	ft	K
MF-4	Ext. Col	46	54	49	44	17	62	43	74	38	13.34	79	49	99	42	10.94	97	59	124	47	10.94	116
(along F)	Int. Col	76	77	85	63	17	87	64	108	62	13.34	99	84	132	68	10.94	112	95	157	75	10.94	126
MF-5 (@	Ext. Col	28	19	54	20	17	36	18	47	18	13.34	45	21	62	20	10.94	54	24	78	21	10.94	65
N.B)	Int. Col	49	27	33	25	17	56	28	72	28	13.34	64	32	89	30	10.94	73	37	107	33	10.94	81

LiUNA Headquarters Expansion Building Re-Design	
Document Name:	Moment Frame Column Design

Column Design

Compressive Stress Capacity   Bending Stress Capacity

Use 49 N1M 16 Visually Graded Southern Pine Glulam Spec.   Emin (psi) 900000 Cd 1.6 Fc (psi) 2100 FcxCd=Fc* 3360 Fb (psi) 1800 FbxCd=Fb* 2880   K 0.5

		Penthouse																				
		D+0.6W		D+0.75L+0.45W			Initial Trial Section Selection								Design Stresses		Column Slenderness Ratios					Critic
		P Dem.	M Dem.	P Dem.	M Dem.	Height	Design P	Design M		S req	S used	A used			fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1
		K	K'	K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2	d1	d2	psi		in		Yes/No		Yes/No	psi
MF-4	Ext. Col	46	54	49	44	17	49	44	528000	183.3333	268	129.9	12.375	10.5	1970.149	377.2132	102	8.242424	yes	9.714286	yes	10889.41
(along F)	Int. Col	76	77	85	63	17	85	63	756000	262.5	400.3	158.8	15.125	10.5	1888.584	535.2645	102	6.743802	yes	9.714286	yes	16266.9
MF-5 (@	Ext. Col	28	19	54	20	17	54	20	240000	83.33333	211.8	115.5	11	10.5	1133.144	467.5325	102	9.272727	yes	9.714286	yes	8603.979
N.B)	Int. Col	49	27	33	25	17	33	25	300000	104.1667	211.8	115.5	11	10.5	1416.431	285.7143	102	9.272727	yes	9.714286	yes	8603.979

		9th Floor																				
		D+0.6W		D+0.75L+0.45W			Initial Trial Section Selection								Design Stresses		Column Slenderness Ratios					Critic
		P Dem.	M Dem.	P Dem.	M Dem.	Height	Design P	Design M		S req	S used	A used			fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1
		K	K'	K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2	d1	d2	psi		in		Yes/No		Yes/No	psi
MF-4	Ext. Col	62	43	74	38	13.34	62	43	516000	179.1667	268	129.9	12.375	10.5	1925.373	477.2902	80.04	6.467879	yes	7.622857	yes	17684.41
(along F)	Int. Col	87	64	108	62	13.34	87	64	768000	266.6667	400.3	158.8	15.125	10.5	1918.561	547.8589	80.04	5.291901	yes	7.622857	yes	26417.45
MF-5 (@	Ext. Col	36	18	47	18	13.34	36	18	216000	75	211.8	115.5	11	10.5	1019.83	311.6883	80.04	7.276364	yes	7.622857	yes	13972.87
N.B)	Int. Col	56	28	72	28	13.34	56	28	336000	116.6667	211.8	115.5	11	10.5	1586.402	484.8485	80.04	7.276364	yes	7.622857	yes	13972.87

		8th Floor																				
		D+0.6W		D+0.75L+0.45W			Initial Trial Section Selection								Design Stresses		Column Slenderness Ratios					Critic
		P Dem.	M Dem.	P Dem.	M Dem.	Height	Design P	Design M		S req	S used	A used			fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1
		K	K'	K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2	d1	d2	psi		in		Yes/No		Yes/No	psi
MF-4	Ext. Col	79	49	99	42	10.94	79	49	588000	204.1667	268	129.9	12.375	10.5	2194.03	608.1601	65.64	5.304242	yes	6.251429	yes	26294.66
(along F)	Int. Col	99	84	132	68	10.94	99	84	1008000	350	476.4	173.3	16.5	10.5	2115.869	571.2637	65.64	3.978182	yes	6.251429	yes	46746.07
MF-5 (@	Ext. Col	45	21	62	20	10.94	45	21	252000	87.5	211.8	115.5	11	10.5	1189.802	389.6104	65.64	5.967273	yes	6.251429	yes	20776.03
N.B)	Int. Col	64	32	89	30	10.94	64	32	384000	133.3333	211.8	115.5	11	10.5	1813.031	554.1126	65.64	5.967273	yes	6.251429	yes	20776.03

		7th Floor																				
		D+0.6W		D+0.75L+0.45W			Initial Trial Section Selection								Design Stresses		Column Slenderness Ratios					Critic
		P Dem.	M Dem.	P Dem.	M Dem.	Height	Design P	Design M		S req	S used	A used			fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1
		K	K'	K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2	d1	d2	psi		in		Yes/No		Yes/No	psi
MF-4	Ext. Col	97	59	124	47	10.94	97	59	708000	245.8333	330.9	144.4	13.75	10.5	2139.619	671.7452	65.64	4.773818	yes	6.251429	yes	32462.55
(along F)	Int. Col	112	95	157	75	10.94	112	95	1140000	395.8333	559.2	187.7	17.825	10.5	2038.627	596.6969	65.64	3.682468	yes	6.251429	yes	54555.21
MF-5 (@	Ext. Col	54	24	78	21	10.94	54	24	288000	100	211.8	115.5	11	10.5	1359.773	467.5325	65.64	5.967273	yes	6.251429	yes	20776.03
N.B)	Int. Col	73	37	107	33	10.94	73	37	444000	154.1667	211.8	115.5	11	10.5	2096.317	632.0346	65.64	5.967273	yes	6.251429	yes	20776.03

		6th Floor																				
		D+0.6W		D+0.75L+0.45W			Initial Trial Section Selection								Design Stresses		Column Slenderness Ratios					Critic
		P Dem.	M Dem.	P Dem.	M Dem.	Height	Design P	Design M		S req	S used	A used			fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1
		K	K'	K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2	d1	d2	psi		in		Yes/No		Yes/No	psi
MF-4	Ext. Col	116	64	151	52	10.94	116	64	768000	266.6667	400.3	158.8	15.125	10.5	1918.561	730.4786	65.64	4.339835	yes	6.251429	yes	39279.68
(along F)	Int. Col	126	105	183	83	10.94	126	105	1260000	437.5	648.5	202.1	19.25	10.5	1942.945	623.4537	65.64	3.40987	yes	6.251429	yes	63626.59
MF-5 (@	Ext. Col	65	26	94	22	10.94	65	26	312000	108.3333	211.8	115.5	11	10.5	1473.088	562.7706	65.64	5.967273	yes	6.251429	yes	20776.03
N.B)	Int. Col	81	42	125	34	10.94	81	42	504000	175	268	129.9	12.375	10.5	1880.597	623.5566	65.64	5.304242	yes	6.251429	yes	26294.66

		5th Floor																				
		D+0.6W		D+0.75L+0.45W			Initial Trial Section Selection								Design Stresses		Column Slenderness Ratios					Critic
		P Dem.	M Dem.	P Dem.	M Dem.	Height	Design P	Design M		S req	S used	A used			fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1
		K	K'	K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2	d1	d2	psi		in		Yes/No		Yes/No	psi
MF-4	Ext. Col	138	69	178	56	10.94	138	69	828000	287.5	476.4	173.3	16.5	10.5	1738.035	796.307	65.64	3.978182	yes	6.251429	yes	46746.07
(along F)	Int. Col	140	116	210	92	10.94	140	116	1392000	483.3333	648.5	202.1	19.25	10.5	2146.492	692.7264	65.64	3.40987	yes	6.251429	yes	63626.59
MF-5 (@	Ext. Col	75	29	110	23	10.94	75	29	348000	120.8333	211.8	115.5	11	10.5	1643.059	649.3506	65.64	5.967273	yes	6.251429	yes	20776.03
N.B)	Int. Col	90	46	144	37	10.94	90	46	552000	191.6667	268	129.9	12.375	10.5	2059.701	692.8406	65.64	5.304242	yes	6.251429	yes	26294.66

		4th Floor																				
		D+0.6W		D+0.75L+0.45W			Initial Trial Section Selection								Design Stresses		Column Slenderness Ratios					Critic
		P Dem.	M Dem.	P Dem.	M Dem.	Height	Design P	Design M		S req	S used	A used			fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1
		K	K'	K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2	d1	d2	psi		in		Yes/No		Yes/No	psi
MF-4	Ext. Col	160	76	206	60	10.94	160	76	912000	316.6667	476.4	173.3	16.5	10.5	1914.358	923.2545	65.64	3.978182	yes	6.251429	yes	46746.07
(along F)	Int. Col	157	124	237	99	10.94	157	124	1488000	516.6667	744.4	216.6	20.625	10.5	1998.925	724.8384	65.64	3.182545	yes	6.251429	yes	73040.73
MF-5 (@	Ext. Col	86	31	127	25	10.94	86	31	372000	129.1667	211.8	115.5	11	10.5	1756.374	744.5887	65.64	5.967273	yes	6.251429	yes	20776.03
N.B)	Int. Col	100	49	163	39	10.94	100	49	588000	204.1667	330.9	144.4	13.75	10.5	1776.972	692.5208	65.64	4.773818	yes	6.251429	yes	32462.55

		3rd Floor																				
		D+0.6W		D+0.75L+0.45W			Initial Trial Section Selection								Design Stresses		Column Slenderness Ratios					Critic
		P Dem.	M Dem.	P Dem.	M Dem.	Height	Design P	Design M		S req	S used	A used			fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1
		K	K'	K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2	d1	d2	psi		in		Yes/No		Yes/No	psi
MF-4	Ext. Col	184	72	235	56	10.94	184	72	864000	300	476.4	173.3	16.5	10.5	1813.602	1061.743	65.64	3.978182	yes	6.251429	yes	46746.07
(along F)	Int. Col	173	126	265	96	10.94	173	126	1512000	525	744.4	216.6	20.625	10.5	2031.166	798.7073	65.64	3.182545	yes	6.251429	yes	73040.73
MF-5 (@	Ext. Col	98	30	144	24	10.94	98	30	360000	125	211.8	115.5	11	10.5	1699.717	848.4848	65.64	5.967273	yes	6.251429	yes	20776.03
N.B)	Int. Col	110	50	182	39	10.94	110	50	600000	208.3333	330.9	144.4	13.75	10.5	1813.237	761.7729	65.64	4.773818	yes	6.251429	yes	32462.55

		2nd Floor																				
		D+0.6W		D+0.75L+0.45W			Initial Trial Section Selection								Design Stresses		Column Slenderness Ratios					Critic
		P Dem.	M Dem.	P Dem.	M Dem.	Height	Design P	Design M		S req	S used	A used			fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1
		K	K'	K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2	d1	d2	psi		in		Yes/No		Yes/No	psi
MF-4	Ext. Col	209	96	265	73	13.34	209	96	1152000	400	648.5	202.1	19.25	10.5	1776.407	1034.142	80.04	4.157922	yes	7.622857	yes	42791.91
(along F)	Int. Col	192	132	295	100	13.34	192	132	1584000	550	847	231	22	10.5	1870.13	831.1688	80.04	3.638182	yes	7.622857	yes	55891.47
MF-5 (@	Ext. Col	110	39	162	30	13.34	110	39	468000	162.5	268	129.9	12.375	10.5	1746.269	846.8052	80.04	6.467879	yes	7.622857	yes	17684.41
N.B)	Int. Col	121	53	204	40	13.34	121	53	636000	220.8333	330.9	144.4	13.75	10.5	1922.031	837.9501	80.04	5.821091	yes	7.622857	yes	21832.61

6th Floor				5th Floor					4th Floor					3rd Floor					2nd Floor			
.6W	D+0.75L+0.45W			D+0.6W		D+0.75L+0.45W			D+0.6W		D+0.75L+0.45W			D+0.6W		D+0.75L+0.45W			D+0.6W		D+0.75L+0.45W	
M Dem.	P Dem.	M Dem.	Height	P Dem.	M Dem.	P Dem.	M Dem.	Height	P Dem.	M Dem.	P Dem.	M Dem.	Height	P Dem.	M Dem.	P Dem.	M Dem.	Height	P Dem.	M Dem.	P Dem.	M Dem.
K'	K	K'	ft	K	K'	K	K'	ft	K	K'	K	K'	ft	K	K'	K	K'	ft	K	K'	K	K'
64	151	52	10.94	138	69	178	56	10.94	160	76	206	60	10.94	184	72	235	56	10.94	209	96	265	73
105	183	83	10.94	140	116	210	92	10.94	157	124	237	99	10.94	173	126	265	96	10.94	192	132	295	100
26	94	22	10.94	75	29	110	23	10.94	86	31	127	25	10.94	98	30	144	24	10.94	110	39	162	30
42	125	34	10.94	90	46	144	37	10.94	100	49	163	39	10.94	110	50	182	39	10.94	121	53	204	40

al Buckling for Compression			Column Stability Factor					Critical Buckling for "Beam"						Beam Stability Factor				Volume Factor where x=20				Adjusted
FCE 2		FCE/(Fc*C	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?	FBE	FBE/Fb	CL Calculation				(5.125/b)	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi	Lesser FCE	d)	Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No	psi		Part 1	Part 2	Part 3	CL	^(1/x)	x)	x)	CV	fc
7839.576	7839.576	2.333207	1.851782	3.429096	2.592452	0.9371	3148.655	204	375.36	6.490936	yes	25633.57	7.62904	4.5416	20.62613	8.030568	0.992577	0.964773	0.998463	1.010621	0.973522	3180.955
7839.576	7839.576	2.333207	1.851782	3.429096	2.592452	0.9371	3148.655	204	375.36	7.175999	yes	20972.92	6.241942	3.811548	14.5279	6.570465	0.990656	0.964773	0.988495	1.010621	0.963802	3311.016
7839.576	7839.576	2.333207	1.851782	3.429096	2.592452	0.9371	3148.655	204	375.36	6.119713	yes	28837.77	8.58267	5.04351	25.437	9.034389	0.993497	0.964773	1.00436	1.010621	0.979272	3287.835
7839.576	7839.576	2.333207	1.851782	3.429096	2.592452	0.9371	3148.655	204	375.36	6.119713	yes	28837.77	8.58267	5.04351	25.437	9.034389	0.993497	0.964773	1.00436	1.010621	0.979272	3106.017

al Buckling for Compression			Column Stability Factor					Critical Buckling for "Beam"						Beam Stability Factor				Volume Factor where x=20				Adjusted
FCE 2		FCE/(Fc*C	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?	FBE	FBE/Fb	CL Calculation				(5.125/b)	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi	Lesser FCE	d)	Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No	psi		Part 1	Part 2	Part 3	CL	^(1/x)	x)	x)	CV	fc
12731.48	12731.48	3.78913	2.660628	7.07894	4.210144	0.966876	3248.703	160.08	294.5472	5.749906	yes	32666.47	9.722165	5.643245	31.84621	10.23386	0.994336	0.964773	0.998463	1.022947	0.985395	3315.227
12731.48	12731.48	3.78913	2.660628	7.07894	4.210144	0.966876	3248.703	160.08	294.5472	6.35676	yes	26727.11	7.954498	4.712894	22.21137	8.373156	0.992919	0.964773	0.988495	1.022947	0.975557	3357.463
12731.48	12731.48	3.78913	2.660628	7.07894	4.210144	0.966876	3248.703	160.08	294.5472	5.421063	yes	36749.78	10.93744	6.282861	39.47434	11.51309	0.995021	0.964773	1.00436	1.022947	0.991215	3166.387
12731.48	12731.48	3.78913	2.660628	7.07894	4.210144	0.966876	3248.703	160.08	294.5472	5.421063	yes	36749.78	10.93744	6.282861	39.47434	11.51309	0.995021	0.964773	1.00436	1.022947	0.991215	3339.547

al Buckling for Compression			Column Stability Factor					Critical Buckling for "Beam"						Beam Stability Factor				Volume Factor where x=20				Adjusted
FCE 2		FCE/(Fc*C	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?	FBE	FBE/Fb	CL Calculation				(5.125/b)	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi	Lesser FCE	d)	Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No	psi		Part 1	Part 2	Part 3	CL	^(1/x)	x)	x)	CV	fc
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	5.207047	yes	39832.79	11.855	6.765788	45.77589	12.47895	0.995438	0.964773	0.998463	1.033142	0.995215	3474.381
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	6.01258	yes	29874.59	8.891248	5.20592	27.1016	9.359209	0.993748	0.964773	0.984203	1.033142	0.981003	3396.551
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	4.909251	yes	44811.89	13.33687	7.545722	56.93793	14.03881	0.995981	0.964773	1.00436	1.033142	1.001094	3272.76
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	4.909251	yes	44811.89	13.33687	7.545722	56.93793	14.03881	0.995981	0.964773	1.00436	1.033142	1.001094	3437.263

al Buckling for Compression			Column Stability Factor					Critical Buckling for "Beam"						Beam Stability Factor				Volume Factor where x=20				Adjusted
FCE 2		FCE/(Fc*C	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?	FBE	FBE/Fb	CL Calculation				(5.125/b)	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi	Lesser FCE	d)	Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No	psi		Part 1	Part 2	Part 3	CL	^(1/x)	x)	x)	CV	fc
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	5.488709	yes	35849.51	10.6695	6.141841	37.72221	11.23105	0.994885	0.964773	0.993217	1.033142	0.989986	3522.906
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	6.249333	yes	27653.9	8.230328	4.858067	23.60082	8.663503	0.993185	0.964773	0.98041	1.033142	0.977221	3411.094
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	4.909251	yes	44811.89	13.33687	7.545722	56.93793	14.03881	0.995981	0.964773	1.00436	1.033142	1.001094	3350.683
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	4.909251	yes	44811.89	13.33687	7.545722	56.93793	14.03881	0.995981	0.964773	1.00436	1.033142	1.001094	3515.185

al Buckling for Compression			Column Stability Factor					Critical Buckling for "Beam"						Beam Stability Factor				Volume Factor where x=20				Adjusted
FCE 2		FCE/(Fc*C	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?	FBE	FBE/Fb	CL Calculation				(5.125/b)	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi	Lesser FCE	d)	Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No	psi		Part 1	Part 2	Part 3	CL	^(1/x)	x)	x)	CV	fc
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	5.756607	yes	32590.47	9.699544	5.631339	31.71198	10.21005	0.994321	0.964773	0.988495	1.033142	0.98528	3568.085
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	6.494329	yes	25606.8	7.62107	4.537405	20.58805	8.022179	0.992569	0.964773	0.976647	1.033142	0.973471	3427.049
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	4.909251	yes	44811.89	13.33687	7.545722	56.93793	14.03881	0.995981	0.964773	1.00436	1.033142	1.001094	3445.921
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	5.207047	yes	39832.79	11.855	6.765788	45.77589	12.47895	0.995438	0.964773	0.998463	1.033142	0.995215	3489.777

al Buckling for Compression			Column Stability Factor					Critical Buckling for "Beam"						Beam Stability Factor				Volume Factor where x=20				Adjusted
FCE 2		FCE/(Fc*C	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?	FBE	FBE/Fb	CL Calculation				(5.125/b)	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi	Lesser FCE	d)	Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No	psi		Part 1	Part 2	Part 3	CL	^(1/x)	x)	x)	CV	fc
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	6.01258	yes	29874.59	8.891248	5.20592	27.1016	9.359209	0.993748	0.964773	0.984203	1.033142	0.981003	3621.595
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	6.494329	yes	25606.8	7.62107	4.537405	20.58805	8.022179	0.992569	0.964773	0.976647	1.033142	0.973471	3496.322
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	4.909251	yes	44811.89	13.33687	7.545722	56.93793	14.03881	0.995981	0.964773	1.00436	1.033142	1.001094	3532.501
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	5.207047	yes	39832.79	11.855	6.765788	45.77589	12.47895	0.995438	0.964773	0.998463	1.033142	0.995215	3559.061

al Buckling for Compression			Column Stability Factor					Critical Buckling for "Beam"						Beam Stability Factor				Volume Factor where x=20				Adjusted
FCE 2		FCE/(Fc*C	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?	FBE	FBE/Fb	CL Calculation				(5.125/b)	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi	Lesser FCE	d)	Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No	psi		Part 1	Part 2	Part 3	CL	^(1/x)	x)	x)	CV	fc
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	6.01258	yes	29874.59	8.891248	5.20592	27.1016	9.359209	0.993748	0.964773	0.984203	1.033142	0.981003	3748.542
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	6.722269	yes	23899.68	7.112999	4.269999	18.23289	7.487367	0.991962	0.964773	0.973284	1.033142	0.970118	3518.779
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	4.909251	yes	44811.89	13.33687	7.545722	56.93793	14.03881	0.995981	0.964773	1.00436	1.033142	1.001094	3627.739
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	5.488709	yes	35849.51	10.6695	6.141841	37.72221	11.23105	0.994885	0.964773	0.993217	1.033142	0.989986	3543.682

al Buckling for Compression			Column Stability Factor					Critical Buckling for "Beam"						Beam Stability Factor				Volume Factor where x=20				Adjusted
FCE 2		FCE/(Fc*C	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?	FBE	FBE/Fb	CL Calculation				(5.125/b)	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi	Lesser FCE	d)	Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No	psi		Part 1	Part 2	Part 3	CL	^(1/x)	x)	x)	CV	fc
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	6.01258	yes	29874.59	8.891248	5.20592	27.1016	9.359209	0.993748	0.964773	0.984203	1.033142	0.981003	3887.03
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	6.722269	yes	23899.68	7.112999	4.269999	18.23289	7.487367	0.991962	0.964773	0.973284	1.033142	0.970118	3592.648
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	4.909251	yes	44811.89	13.33687	7.545722	56.93793	14.03881	0.995981	0.964773	1.00436	1.033142	1.001094	3731.635
18930.23	18930.23	5.633996	3.685553	13.5833	6.259995	0.979392	3290.758	131.28	241.5552	5.488709	yes	35849.51	10.6695	6.141841	37.72221	11.23105	0.994885	0.964773	0.993217	1.033142	0.989986	3612.934

al Buckling for Compression			Column Stability Factor					Critical Buckling for "Beam"						Beam Stability Factor				Volume Factor where x=20				Adjusted
FCE 2		FCE/(Fc*C	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?	FBE	FBE/Fb	CL Calculation				(5.125/b)	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi	Lesser FCE	d)	Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No	psi		Part 1	Part 2	Part 3	CL	^(1/x)	x)	x)	CV	fc
12731.48	12731.48	3.78913	2.660628	7.07894	4.210144	0.966876	3248.703	160.08	294.5472	7.171393	yes	20999.88	6.249963	3.81577	14.5601	6.578908	0.99067	0.964773	0.976647	1.022947	0.963864	3810.071
12731.48	12731.48	3.78913	2.660628	7.07894	4.210144	0.966876	3248.703	160.08	294.5472	7.666541	yes	18374.89	5.468718	3.404588	11.59122	5.756545	0.989081	0.964773	0.970148	1.022947	0.95745	3588.626
12731.48	12731.48	3.78913	2.660628	7.07894	4.210144	0.966876	3248.703	160.08	294.5472	5.749906	yes	32666.47	9.722165	5.643245	31.84621	10.23386	0.994336	0.964773	0.998463	1.022947	0.985395	3684.742
12731.48	12731.48	3.78913	2.660628	7.07894	4.210144	0.966876	3248.703	160.08	294.5472	6.060933	yes	29399.83	8.749948	5.131552	26.33282	9.210472	0.993635	0.964773	0.993217	1.022947	0.980217	3660.976

Height
ft
13.34
13.34
13.34
13.34

Beding Design Value		Beam-Column Loading Interaction			
FBX*CL	Fbx'	(fc/Fc')^2	fb/(Fbx' * (1-(fc/FCE)))	Combined Loading Ratio	Combined < 1.0? Yes/No
2858.623	2858.623	0.119801	0.713925953	0.833727342	yes
2853.088	2853.088	0.169998	0.684466072	0.854463893	yes
2861.272	2861.272	0.148486	0.418784666	0.567271074	yes
2861.272	2861.272	0.090742	0.512038743	0.602780437	yes

Beding Design Value		Beam-Column Loading Interaction			
FBX*CL	Fbx'	(fc/Fc')^2	fb/(Fbx' * (1-(fc/FCE)))	Combined Loading Ratio	Combined < 1.0? Yes/No
2863.687	2863.687	0.146917	0.690989882	0.837907057	yes
2859.607	2859.607	0.168639	0.685126266	0.853765563	yes
2865.66	2865.66	0.095942	0.363999189	0.459941582	yes
2865.66	2865.66	0.149244	0.573490147	0.722733869	yes

Beding Design Value		Beam-Column Loading Interaction			
FBX*CL	Fbx'	(fc/Fc')^2	fb/(Fbx' * (1-(fc/FCE)))	Combined Loading Ratio	Combined < 1.0? Yes/No
2866.861	2866.861	0.184809	0.783427113	0.968235639	yes
2861.994	2861.994	0.173596	0.748445417	0.922041809	yes
2868.425	2868.425	0.118395	0.422719886	0.541115221	yes
2868.425	2868.425	0.168384	0.649384605	0.817769081	yes

Beding Design Value		Beam-Column Loading Interaction			
FBX*CL	Fbx'	(fc/Fc')^2	fb/(Fbx' * (1-(fc/FCE)))	Combined Loading Ratio	Combined < 1.0? Yes/No
2865.268	2865.268	0.204131	0.762522065	0.966652895	yes
2860.373	2860.373	0.181325	0.720595073	0.901920125	yes
2868.425	2868.425	0.142074	0.48496209	0.627036492	yes
2868.425	2868.425	0.192064	0.753755431	0.945818975	yes

Beding Design Value		Beam-Column Loading Interaction			
FBX*CL	Fbx'	(fc/Fc')^2	fb/(Fbx' * (1-(fc/FCE)))	Combined Loading Ratio	Combined < 1.0? Yes/No
2863.646	2863.646	0.221979	0.682667024	0.904645851	yes
2858.597	2858.597	0.189456	0.686410726	0.875866695	yes
2868.425	2868.425	0.171015	0.527850991	0.698866475	yes
2866.861	2866.861	0.189487	0.671911697	0.861398919	yes

Beding Design Value		Beam-Column Loading Interaction			
FBX*CL	Fbx'	(fc/Fc')^2	fb/(Fbx' * (1-(fc/FCE)))	Combined Loading Ratio	Combined < 1.0? Yes/No
2861.994	2861.994	0.241983	0.617805464	0.859788312	yes
2858.597	2858.597	0.210507	0.759155121	0.969661753	yes
2868.425	2868.425	0.197326	0.591289563	0.788615122	yes
2866.861	2866.861	0.210541	0.737894801	0.948436159	yes

Beding Design Value		Beam-Column Loading Interaction			
FBX*CL	Fbx'	(fc/Fc')^2	fb/(Fbx' * (1-(fc/FCE)))	Combined Loading Ratio	Combined < 1.0? Yes/No
2861.994	2861.994	0.28056	0.682366586	0.96292641	yes
2856.851	2856.851	0.220265	0.706708477	0.926973359	yes
2868.425	2868.425	0.226267	0.635073278	0.861339918	yes
2865.268	2865.268	0.210444	0.633695163	0.844139318	yes

Beding Design Value		Beam-Column Loading Interaction			
FBX*CL	Fbx'	(fc/Fc')^2	fb/(Fbx' * (1-(fc/FCE)))	Combined Loading Ratio	Combined < 1.0? Yes/No
2861.994	2861.994	0.322644	0.648412221	0.971056019	yes
2856.851	2856.851	0.242712	0.718841279	0.961553538	yes
2868.425	2868.425	0.257839	0.617791312	0.875630041	yes
2865.268	2865.268	0.231489	0.648040311	0.879528882	yes

Beding Design Value		Beam-Column Loading Interaction			
FBX*CL	Fbx'	(fc/Fc')^2	fb/(Fbx' * (1-(fc/FCE)))	Combined Loading Ratio	Combined < 1.0? Yes/No
2853.128	2853.128	0.318324	0.63803656	0.956361013	yes
2848.553	2848.553	0.255846	0.666429873	0.922276254	yes
2863.687	2863.687	0.26066	0.640465456	0.901124958	yes
2861.67	2861.67	0.257934	0.698453677	0.956387447	yes

Appendix G: Moment Frame Bolted Connection Design Calculations

LiUNA Headquarters Expansion Building Re-Design	
Document Name:	**Moment Frame Bolted Connection Design**

Connection Shear Information from SAP Model

		Connection Strength Per Table 12-I for 1" Dia. Bolts and 1/4" A36 Pl.								Group Fact
Glulam Strength Information		Z //	5960	lbs/bolt		Z Perp.	3180	lbs/bolt		Cg

		Penthouse					9th Floor					
MF-4 (along F)	Exterior Column	0.0		0.0	0.0		0.0		5.4	1.9		0.
		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Re
		0.0	Shear 2		Shear 3	8.2	0	Shear 2		Shear 3	8.8	
		1.9		Shear 4	2.9		2.2		Shear 4	3.1		2.
		Bolts Resisting 4		5.4	Bolts Resisting 3		Bolts Resisting 4		6.4	Bolts Resisting 3		Bolts Re
	Interior Column	2.9			0.0		2.9		8.3	2.9		3.
		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Re
		8.2	Shear 2		Shear 3	9.5	8.2	Shear 2		Shear 3	6.87	9
		2.9		Shear 4	3.3		4.2		Shear 4	2.4		5.
		Bolts Resisting 4		8.2	Bolts Resisting 3		Bolts Resisting 4		11.9	Bolts Resisting 3		Bolts Re
MF-5 (@ N.B.)	Exterior Column	0.0			0.0		0.0		1.8	0.6		0.
		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Re
		0.0	Shear 2		Shear 3	2.3	0	Shear 2		Shear 3	3.4	
		0.6		Shear 4	0.8		0.9		Shear 4	1.2		1.
		Bolts Resisting 4		1.7	Bolts Resisting 3		Bolts Resisting 4		2.5	Bolts Resisting 3		Bolts Re
	Interior Column	0.3			0.0		1.0		2.7	0.9		1.
		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Re
		1.0	Shear 2		Shear 3	2.1	2.8	Shear 2		Shear 3	2.5	3.3
		0.9		Shear 4	0.7		1.5		Shear 4	0.9		1.
		Bolts Resisting 4		2.7	Bolts Resisting 3		Bolts Resisting 4		4.3	Bolts Resisting 3		Bolts Re

or	Duration Factors (Conservative 1.0 Used)				
0.9	C_D	1	Wind	1	Live

8th Floor				7th Floor					6th Floor			
.0	6.4	2.2		0.0		8.1	2.8		0.0		9.5	3.
sisting 2	Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Re
Shear 2		Shear 3	9.6		Shear 2		Shear 3	10.9		Shear 2		Shear 3
.8	Shear 4	3.4		3.3		Shear 4	3.8		3.7		Shear 4	4.
sisting 4	8	Bolts Resisting 3		Bolts Resisting 4		9.4	Bolts Resisting 3		Bolts Resisting 4		10.7	Bolts Re
.1	11.9	4.2		3.6		14.4	5.0		4.2		16.5	5.
sisting 2	Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Re
Shear 2		Shear 3	7.5	10.4	Shear 2		Shear 3	8.3	11.9	Shear 2		Shear 3
.0	Shear 4	2.6		5.8		Shear 4	2.9		6.5		Shear 4	3.
sisting 4	14.4	Bolts Resisting 3		Bolts Resisting 4		16.5	Bolts Resisting 3		Bolts Resisting 4		18.6	Bolts Re
.0	2.5	0.9		0.0		3.2	1.1		0.0		3.8	1.
sisting 2	Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Re
Shear 2		Shear 3	3.9		Shear 2		Shear 3	4.4		Shear 2		Shear 3
.1	Shear 4	1.4		1.3		Shear 4	1.5		1.5		Shear 4	1.
sisting 4	3.2	Bolts Resisting 3		Bolts Resisting 4		3.8	Bolts Resisting 3		Bolts Resisting 4		4.3	Bolts Re
.2	4.3	1.5		1.4		5.4	1.9		1.6		6.3	2.
sisting 2	Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Re
Shear 2		Shear 3	2.8	3.9	Shear 2		Shear 3	3.2	4.5	Shear 2		Shear 3
.9	Shear 4	1.0		2.2		Shear 4	1.1		2.5		Shear 4	1.
sisting 4	5.4	Bolts Resisting 3		Bolts Resisting 4		6.3	Bolts Resisting 3		Bolts Resisting 4		7.2	Bolts Re

	5th Floor					4th Floor						
.3	0.0		10.7	3.7		0.0		12	4.2		0.0	
sisting 1	Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2	
12.5		Shear 2		Shear 3	14		Shear 2		Shear 3	15.5		Shear 2
.4	4.2		Shear 4	4.9		4.7		Shear 4	5.4		4.7	
sisting 3	Bolts Resisting 4		12	Bolts Resisting 3		Bolts Resisting 4		13.4	Bolts Resisting 3		Bolts Resisting 4	
.8	4.7		18.6	6.5		5.2		20.6	7.2		5.5	
sisting 1	Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2	
9.2	13.4	Shear 2		Shear 3	10.1	14.9	Shear 2		Shear 3	10.7	15.8	Shear 2
.2	7.2		Shear 4	3.5		7.8		Shear 4	3.7		8.8	
sisting 3	Bolts Resisting 4		20.6	Bolts Resisting 3		Bolts Resisting 4		22.2	Bolts Resisting 3		Bolts Resisting 4	
.3	0.0		4.3	1.5		0.0		4.8	1.7		0.0	
sisting 1	Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2	
5.1		Shear 2		Shear 3	5.8		Shear 2		Shear 3	6.4		Shear 2
.8	1.7		Shear 4	2.0		1.9		Shear 4	2.2		1.9	
sisting 3	Bolts Resisting 4		4.8	Bolts Resisting 3		Bolts Resisting 4		5.4	Bolts Resisting 3		Bolts Resisting 4	
.2	1.8		7.2	2.5		2.0		8	2.8		2.1	
sisting 1	Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1		Bolts Resisting 2	
3.6	5.2	Shear 2		Shear 3	4	5.8	Shear 2		Shear 3	4.3	6.1	Shear 2
.3	2.8		Shear 4	1.4		3.0		Shear 4	1.5		3.5	
sisting 3	Bolts Resisting 4		8	Bolts Resisting 3		Bolts Resisting 4		8.7	Bolts Resisting 3		Bolts Resisting 4	

3rd Floor			2nd Floor				
13.4	4.7		0.0		13.6	4.8	
Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1	
	Shear 3	16.4		Shear 2		Shear 3	17.8
Shear 4	5.7		6.5		Shear 4	6.2	
13.5	Bolts Resisting 3		Bolts Resisting 4		18.6	Bolts Resisting 3	
22.2	7.8		6.0		25.2	8.8	
Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1	
	Shear 3	11.1	17.2	Shear 2		Shear 3	11.1
Shear 4	3.9		7.8		Shear 4	3.9	
25.2	Bolts Resisting 3		Bolts Resisting 4		22.3	Bolts Resisting 3	
5.4	1.9		0.0		5.5	1.9	
Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1	
	Shear 3	6.7		Shear 2		Shear 3	7.3
Shear 4	2.3		2.6		Shear 4	2.6	
5.4	Bolts Resisting 3		Bolts Resisting 4		7.4	Bolts Resisting 3	
8.7	3.0		2.3		9.9	3.5	
Shear 1	Bolts Resisting 1		Bolts Resisting 2		Shear 1	Bolts Resisting 1	
	Shear 3	4.4	6.6	Shear 2		Shear 3	4.6
Shear 4	1.5		3.1		Shear 4	1.6	
9.9	Bolts Resisting 3		Bolts Resisting 4		8.8	Bolts Resisting 3	

Appendix H: Moment Frame 4 Drift Calculation, Original Design

LiUNA Headquarters Expansion Building Re-Design

Document Name:	**Moment Frame Drift Analysis Check**

Using Revised Moment Frame Design where 1/2" A36 Steel Plates are added to columns and all beams and columns are the same, estimate each story stiffness and deflection

Penthouse																						
floor to floor height		17	ft	204	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	1658	1700	3.984045	0.399969	1.593494		Wood	3028	1700	7.276048	0.402194	2.926384		Wood	3028	1700	7.276048	0.408541	2.972564		Wood	1658
Steel	0	29000	0	1	0	1.593494	Steel	0	29000	0	1	0	2.926384	Steel	0	29000	0	1	0	2.972564	Steel	0
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	1165	17.92	215.04				Beam 1	1165	17.92	215.04				Beam 1	1165	17.08	204.96				Beam 1	1165
Beam 2			0				Beam 2	1165	21.25	255				Beam 2	1165	21.25	255				Beam 2	

9th Floor																						
floor to floor height		11.94	ft	143.28	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	1658	1700	3.984045	0.399866	1.593084		Wood	3028	1700	21.00048	0.402091	8.44411		Wood	3028	1700	21.00048	0.408438	8.577386		Wood	1658
Steel	0	29000	0	1	0	1.593084	Steel	0	29000	0	1	0	8.44411	Steel	0	29000	0	1	0	8.577386	Steel	0
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	1658	17.92	215.04				Beam 1	1658	17.92	215.04				Beam 1	1658	17.08	204.96				Beam 1	1658
Beam 2			0				Beam 2	1658	21.25	255				Beam 2	1658	21.25	255				Beam 2	

8th floor																						
floor to floor height		10.94	ft	131.28	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	1658	1700	3.984045	0.455835	1.816065		Wood	3931	1700	35.44357	0.36324	12.87454		Wood	3931	1700	35.44357	0.370209	13.12152		Wood	1658
Steel	0	29000	0	1	0	1.816065	Steel	0	29000	0	1	0	12.87454	Steel	0	29000	0	1	0	13.12152	Steel	0
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	2275	17.92	215.04				Beam 1	2275	17.92	215.04				Beam 1	2275	17.08	204.96				Beam 1	2275
Beam 2			0				Beam 2	1658	21.25	255				Beam 2	1658	21.25	255				Beam 2	

7th Floor																						
floor to floor height		10.94	ft	131.28	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	2275	1700	5.466647	0.379071	2.07225		Wood	4997	1700	45.05508	0.309754	13.95598		Wood	4997	1700	45.05508	0.316205	14.24666		Wood	2275
Steel	0	29000	0	1	0	2.07225	Steel	0	29000	0	1	0	13.95598	Steel	0	29000	0	1	0	14.24666	Steel	0
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	2275	17.92	215.04				Beam 1	2275	17.92	215.04				Beam 1	2275	17.08	204.96				Beam 1	1175
Beam 2			0				Beam 2	1658	21.25	255				Beam 2	1658	21.25	255				Beam 2	

6th Floor																						
floor to floor height		10.94	ft	131.28	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	3028	1700	7.276048	0.379071	2.758141		Wood	6242	1700	56.28054	0.326048	18.35017		Wood	6242	1700	56.28054	0.332599	18.71887		Wood	3028
Steel	0	29000	0	1	0	2.758141	Steel	0	29000	0	1	0	18.35017	Steel	0	29000	0	1	0	18.71887	Steel	0
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	3028	17.92	215.04				Beam 1	3028	17.92	215.04				Beam 1	3028	17.08	204.96				Beam 1	3028
Beam 2			0				Beam 2	2275	21.25	255				Beam 2	2275	21.25	255				Beam 2	

5th Floor																						
floor to floor height		10.94	ft	131.28	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	3931	1700	9.445886	0.319845	3.021222		Wood	6242	1700	56.28054	0.326048	18.35017		Wood	6242	1700	56.28054	0.332599	18.71887		Wood	3931
Steel	0	29000	0	1	0	3.021222	Steel	0	29000	0	1	0	18.35017	Steel	0	29000	0	1	0	18.71887	Steel	0
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	3028	17.92	215.04				Beam 1	3028	17.92	215.04				Beam 1	3028	17.08	204.96				Beam 1	3028
Beam 2			0				Beam 2	2275	21.25	255				Beam 2	2275	21.25	255				Beam 2	

4th Floor																						
floor to floor height		10.94	ft	131.28	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	3931	1700	9.445886	0.379071	3.580665		Wood	7677	1700	69.21911	0.340222	23.54984		Wood	7677	1700	69.21911	0.346847	24.00843		Wood	3931
Steel	0	29000	0	1	0	3.580665	Steel	0	29000	0	1	0	23.54984	Steel	0	29000	0	1	0	24.00843	Steel	0
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	3931	17.92	215.04				Beam 1	3931	17.92	215.04				Beam 1	3931	17.08	204.96				Beam 1	3931
Beam 2			0				Beam 2	3028	21.25	255				Beam 2	3028	21.25	255				Beam 2	

3rd Floor																						
floor to floor height		10.94	ft	131.28	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	3931	1700	9.445886	0.379071	3.580665		Wood	7677	1700	69.21911	0.340222	23.54984		Wood	7677	1700	69.21911	0.346847	24.00843		Wood	3931
Steel	0	29000	0	1	0	3.580665	Steel	0	29000	0	1	0	23.54984	Steel	0	29000	0	1	0	24.00843	Steel	0
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	3931	17.92	215.04				Beam 1	3931	17.92	215.04				Beam 1	3931	17.08	204.96				Beam 1	3931
Beam 2			0				Beam 2	3028	21.25	255				Beam 2	3028	21.25	255				Beam 2	

2nd Floor																						
floor to floor height		13.34	ft	160.08	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	6242	1700	31.0415	0.37341	11.59122		Wood	9317	1700	46.33349	0.376278	17.43426		Wood	9317	1700	46.33349	0.383824	17.78391		Wood	6242
Steel	0	29000	0	1	0	11.59122	Steel	0	29000	0	1	0	17.43426	Steel	0	29000	0	1	0	17.78391	Steel	0
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	4997	17.92	215.04				Beam 1	4997	17.92	215.04				Beam 1	4997	17.08	204.96				Beam 1	4997
Beam 2			0				Beam 2	3028	21.25	255				Beam 2	3028	21.25	255				Beam 2	

Left Column									
E	Stiffness Calculation				Sum of K	Story Force		Def.	Drift
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	3.984045	0.411544	1.63961			36.4	21.84	2.391577	12.11243
29000	0	1	0	1.63961	9.132051				
Span									
ft	in								
17.08	204.96								
	0								

Left Column									
E	Stiffness Calculation				Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	11.49894	0.41144	4.73113			17.1	32.1	1.374985	9.720857
29000	0	1	0	4.73113	23.34571				
Span									
ft	in								
17.08	204.96								
	0								

Left Column									
E	Stiffness Calculation				Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	14.94924	0.467766	6.992747			13.3	40.08	1.151563	8.345872
29000	0	1	0	6.992747	34.80487				
Span									
ft	in								
17.08	204.96								
	0								

Left Column									
E	Stiffness Calculation				Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	20.51237	0.248581	5.098987			13.3	47.46	1.341668	7.194309
29000	0	1	0	5.098987	35.37387				
Span									
ft	in								
17.08	204.96								
	0								

Left Column									
E	Stiffness Calculation				Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	27.30174	0.390435	10.65957			13.3	54.66	1.08266	5.852641
29000	0	1	0	10.65957	50.48675				
Span									
ft	in								
17.08	204.96								
	0								

Left Column									
E	Stiffness Calculation				Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	35.44357	0.330378	11.70979			13.3	61.68	1.190732	4.76998
29000	0	1	0	11.70979	51.80006				
Span									
ft	in								
17.08	204.96								
	0								

Story Forces		Story Shear
36.4	21.84	21.84
17.1	10.26	32.1
13.3	7.98	40.08
12.3	7.38	47.46
12	7.2	54.66
11.7	7.02	61.68
11.1	6.66	68.34
10.4	6.24	74.58
10.8	6.48	81.06

Left Column									
E	Stiffness Calculation				Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	35.44357	0.390435	13.83843			13.3	68.34	1.051751	3.579248
29000	0	1	0	13.83843	64.97736				
Span									
ft	in								
17.08	204.96								
	0								

Left Column									
E	Stiffness Calculation				Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	35.44357	0.390435	13.83843			13.3	74.58	1.147784	2.527497
29000	0	1	0	13.83843	64.97736				
Span									
ft	in								
17.08	204.96								
	0								

Left Column									
E	Stiffness Calculation				Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	31.0415	0.38471	11.94197			13.3	81.06	1.379713	1.379713
29000	0	1	0	11.94197	58.75136				
Span									
ft	in								
17.08	204.96								
	0								

Appendix I: Moment Frame 4 Drift Calculation, Revised Design with 1/4" Steel Plate

LiUNA Headquarters Expansion Building Re-Design	
Document Name:	Moment Frame Drift Analysis Check

Using Revised Moment Frame Design where 1/2" A36 Steel Plates are added to columns and all beams and columns are the same, estimate each story stiffness and deflection

															Penthouse							
floor to floor height		17	ft	204	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	1658	1700	3.984045	0.399969	1.593494		Wood	3028	1700	7.276048	0.402194	2.926384		Wood	3028	1700	7.276048	0.408541	2.972564		Wood	1658
Steel	209.4	29000	8.58352	0.840711	7.216258	8.809751	Steel	311.1	29000	12.75231	0.867521	11.06289	13.98927	Steel	311.1	29000	12.75231	0.870518	11.10111	14.07367	Steel	209.4
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	1165	17.92	215.04				Beam 1	1165	17.92	215.04				Beam 1	1165	17.08	204.96				Beam 1	1165
Beam 2	347.2		0				Beam 2	1165	21.25	255				Beam 2	1165	21.25	255				Beam 2	
															9th Floor							
floor to floor height		11.94	ft	143.28	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	1658	1700	3.984045	0.399866	1.593084		Wood	3028	1700	21.00048	0.402091	8.44411		Wood	3028	1700	21.00048	0.408438	8.577386		Wood	1658
Steel	209.4	29000	24.77418	0.840653	20.82649	22.41958	Steel	311.1	29000	36.80633	0.867471	31.92844	40.37255	Steel	311.1	29000	36.80633	0.870469	32.03878	40.61617	Steel	209.4
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	1658	17.92	215.04				Beam 1	1658	17.92	215.04				Beam 1	1658	17.08	204.96				Beam 1	1658
Beam 2			0				Beam 2	1658	21.25	255				Beam 2	1658	21.25	255				Beam 2	
															8th floor							
floor to floor height		10.94	ft	131.28	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	1658	1700	3.984045	0.455835	1.816065		Wood	3931	1700	35.44357	0.36324	12.87454		Wood	3931	1700	35.44357	0.370209	13.12152		Wood	1658
Steel	209.4	29000	32.20774	0.868983	27.98798	29.80404	Steel	368.9	29000	56.74037	0.858732	48.72476	61.5993	Steel	368.9	29000	56.74037	0.862333	48.92908	62.0506	Steel	209.4
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	2275	17.92	215.04				Beam 1	2275	17.92	215.04				Beam 1	2275	17.08	204.96				Beam 1	2275
Beam 2			0				Beam 2	1658	21.25	255				Beam 2	1658	21.25	255				Beam 2	
															7th Floor							
floor to floor height		10.94	ft	131.28	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	2275	1700	5.466647	0.379071	2.07225		Wood	4997	1700	45.05508	0.309754	13.95598		Wood	4997	1700	45.05508	0.316205	14.24666		Wood	2275
Steel	257.7	29000	39.63674	0.843493	33.4333	35.50555	Steel	432	29000	66.44576	0.838471	55.71285	69.66883	Steel	432	29000	66.44576	0.842494	55.98014	70.2268	Steel	257.7
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	2275	17.92	215.04				Beam 1	2275	17.92	215.04				Beam 1	2275	17.08	204.96				Beam 1	1175
Beam 2			0				Beam 2	1658	21.25	255				Beam 2	1658	21.25	255				Beam 2	
															6th Floor							
floor to floor height		10.94	ft	131.28	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	3028	1700	7.276048	0.379071	2.758141		Wood	6242	1700	56.28054	0.326048	18.35017		Wood	6242	1700	56.28054	0.332599	18.71887		Wood	3028
Steel	311.1	29000	47.85018	0.85595	40.95736	43.7155	Steel	500	29000	76.90482	0.857946	65.98019	84.33036	Steel	500	29000	76.90482	0.861523	66.25526	84.97413	Steel	311.1
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	3028	17.92	215.04				Beam 1	3028	17.92	215.04				Beam 1	3028	17.08	204.96				Beam 1	3028
Beam 2			0				Beam 2	2275	21.25	255				Beam 2	2275	21.25	255				Beam 2	
															5th Floor							
floor to floor height		10.94	ft	131.28	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	3931	1700	9.445886	0.319845	3.021222		Wood	6242	1700	56.28054	0.326048	18.35017		Wood	6242	1700	56.28054	0.332599	18.71887		Wood	3931
Steel	368.9	29000	56.74037	0.833639	47.30099	50.32221	Steel	500	29000	76.90482	0.857946	65.98019	84.33036	Steel	500	29000	76.90482	0.861523	66.25526	84.97413	Steel	368.9
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	3028	17.92	215.04				Beam 1	3028	17.92	215.04				Beam 1	3028	17.08	204.96				Beam 1	3028
Beam 2			0				Beam 2	2275	21.25	255				Beam 2	2275	21.25	255				Beam 2	
															4th Floor							
floor to floor height		10.94	ft	131.28	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	3931	1700	9.445886	0.379071	3.580665		Wood	7677	1700	69.21911	0.340222	23.54984		Wood	7677	1700	69.21911	0.346847	24.00843		Wood	3931
Steel	368.9	29000	56.74037	0.866763	49.18043	52.76109	Steel	573	29000	88.13292	0.873558	76.98923	100.5391	Steel	573	29000	88.13292	0.876768	77.27209	101.2805	Steel	369.9
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	3931	17.92	215.04				Beam 1	3931	17.92	215.04				Beam 1	3931	17.08	204.96				Beam 1	3931
Beam 2			0				Beam 2	3028	21.25	255				Beam 2	3028	21.25	255				Beam 2	
															3rd Floor							
floor to floor height		10.94	ft	131.28	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	3931	1700	9.445886	0.379071	3.580665		Wood	7677	1700	69.21911	0.340222	23.54984		Wood	7677	1700	69.21911	0.346847	24.00843		Wood	3931
Steel	368.9	29000	56.74037	0.866763	49.18043	52.76109	Steel	573	29000	88.13292	0.873558	76.98923	100.5391	Steel	573	29000	88.13292	0.876768	77.27209	101.2805	Steel	368.9
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	3931	17.92	215.04				Beam 1	3931	17.92	215.04				Beam 1	3931	17.08	204.96				Beam 1	3931
Beam 2			0				Beam 2	3028	21.25	255				Beam 2	3028	21.25	255				Beam 2	
															2nd Floor							
floor to floor height		13.34	ft	160.08	inches																	
Left Column							Middle Column on Left							Middle Column on Right								
	I	E	Stiffness Calculation					I	E	Stiffness Calculation					I	E	Stiffness Calculation					I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	6242	1700	31.0415	0.37341	11.59122		Wood	9317	1700	46.33349	0.376278	17.43426		Wood	9317	1700	46.33349	0.383824	17.78391		Wood	6242
Steel	500	29000	42.41681	0.881513	37.39096	48.98218	Steel	651	29000	55.22669	0.896201	49.49421	66.92847	Steel	651	29000	55.22669	0.899143	49.65669	67.4406	Steel	500
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	4997	17.92	215.04				Beam 1	4997	17.92	215.04				Beam 1	4997	17.08	204.96				Beam 1	4997
Beam 2			0				Beam 2	3028	21.25	255				Beam 2	3028	21.25	255				Beam 2	

	Left Column								
E		Stiffness Calculation			Sum of K	Story Force		Def.	Drift
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	3.984045	0.411544	1.63961			36.4	21.84	0.477035	2.38905
29000	8.58352	0.847035	7.270544	8.910154	45.78285				
	Span								
ft	in								
17.08	204.96								
	0								

	Left Column								
E		Stiffness Calculation			Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	11.49894	0.41144	4.73113			17.1	32.1	0.248601	1.912016
29000	24.77418	0.84698	20.98323	25.71436	129.1227				
	Span								
ft	in								
17.08	204.96								
	0								

	Left Column								
E		Stiffness Calculation			Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	14.94924	0.467766	6.992747			13.3	40.08	0.212505	1.663415
29000	32.20774	0.874353	28.16093	35.15367	188.6076				
	Span								
ft	in								
17.08	204.96								
	0								

	Left Column								
E		Stiffness Calculation			Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	20.51237	0.248581	5.098987			13.3	47.46	0.225971	1.45091
29000	39.63674	0.744929	29.52654	34.62553	210.0267				
	Span								
ft	in								
17.08	204.96								
	0								

	Left Column								
E		Stiffness Calculation			Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	27.30174	0.390435	10.65957			13.3	54.66	0.20633	1.224939
29000	47.85018	0.861769	41.2358	51.89536	264.9154				
	Span								
ft	in								
17.08	204.96								
	0								

	Left Column								
E		Stiffness Calculation			Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	35.44357	0.330378	11.70979			13.3	61.68	0.221068	1.018609
29000	56.74037	0.840191	47.67275	59.38254	279.0092				
	Span								
ft	in								
17.08	204.96								
	0								

Story Forces		Story Shear
36.4	21.84	21.84
17.1	10.26	32.1
13.3	7.98	40.08
12.3	7.38	47.46
12	7.2	54.66
11.7	7.02	61.68
11.1	6.66	68.34
10.4	6.24	74.58
10.8	6.48	81.06

	Left Column								
E		Stiffness Calculation			Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	35.44357	0.390435	13.83843			13.3	68.34	0.214888	0.797541
29000	56.89418	0.871908	49.60649	63.44491	318.0256				
	Span								
ft	in								
17.08	204.96								
	0								

	Left Column								
E		Stiffness Calculation			Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	35.44357	0.390435	13.83843			13.3	74.58	0.234596	0.582653
29000	56.74037	0.87221	49.48952	63.32794	317.9086				
	Span								
ft	in								
17.08	204.96								
	0								

	Left Column								
E		Stiffness Calculation			Sum of K	Story Force		Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	31.0415	0.38471	11.94197			13.3	81.06	0.348057	0.348057
29000	42.41681	0.886436	37.59979	49.54177	232.893				
	Span								
ft	in								
17.08	204.96								
	0								

Appendix J: Moment Frame 5 Drift Calculation, Revised Design with 1/4" Steel Plate

LiUNA Headquarters Expansion Building Re-Design	
Document Name:	Moment Frame Drift Analysis Check

Using Revised Moment Frame Design where 1/2" A36 Steel Plates are added to columns and all beams and columns are the same, estimate each story stiffness and deflection

Penthouse

floor to floor height	17	ft	204	inches																		
			Left Column							Middle Column on Left							Middle Column on Right					
	I	E		Stiffness Calculation				I	E		Stiffness Calculation				I	E		Stiffness Calculation				I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	1165	1700	2.799404	0.486827	1.362826		Wood	1165	1700	2.799404	0.636186	1.780943		Wood	1165	1700	2.799404	0.642259	1.797942		Wood	1165
Steel	166.4	29000	6.820906	0.86914	5.928323	7.291149	Steel	166.4	29000	6.820906	0.924487	6.305838	8.086781	Steel	166.4	29000	6.820906	0.926305	6.318237	8.116178	Steel	166.4
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	1165	17.92	215.04				Beam 1	1165	17.92	215.04				Beam 1	1165	17.08	204.96				Beam 1	1165
Beam 2			0				Beam 2	1165	21.25	255				Beam 2	1165	21.25	255				Beam 2	

9th Floor

floor to floor height	11.94	ft	143.28	inches																		
			Left Column							Middle Column on Left							Middle Column on Right					
	I	E		Stiffness Calculation				I	E		Stiffness Calculation				I	E		Stiffness Calculation				I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	1165	1700	2.799404	0.399866	1.119387		Wood	1165	1700	8.079777	0.551203	4.453594		Wood	1165	1700	8.079777	0.557707	4.50615		Wood	1165
Steel	166.4	29000	19.68683	0.823473	16.21158	17.33097	Steel	166.4	29000	19.68683	0.895819	17.63585	22.08944	Steel	166.4	29000	19.68683	0.898251	17.68372	22.18987	Steel	166.4
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	1165	17.92	215.04				Beam 1	1165	17.92	215.04				Beam 1	1165	17.08	204.96				Beam 1	1165
Beam 2			0				Beam 2	1165	21.25	255				Beam 2	1165	21.25	255				Beam 2	

8th floor

floor to floor height	10.94	ft	131.28	inches																		
			Left Column							Middle Column on Left							Middle Column on Right					
	I	E		Stiffness Calculation				I	E		Stiffness Calculation				I	E		Stiffness Calculation				I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	1165	1700	2.799404	0.379071	1.061174		Wood	1658	1700	14.94924	0.441561	6.601003		Wood	1658	1700	14.94924	0.448064	6.698209		Wood	1165
Steel	166.4	29000	25.59392	0.810397	20.74123	21.80241	Steel	209.4	29000	32.20774	0.862272	27.77184	34.37284	Steel	209.4	29000	32.20774	0.86537	27.8716	34.56981	Steel	166.4
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	1165	17.92	215.04				Beam 1	1165	17.92	215.04				Beam 1	1165	17.08	204.96				Beam 1	1165
Beam 2			0				Beam 2	1165	21.25	255				Beam 2	1165	21.25	255				Beam 2	

7th Floor

floor to floor height	10.94	ft	131.28	inches																		
			Left Column							Middle Column on Left							Middle Column on Right					
	I	E		Stiffness Calculation				I	E		Stiffness Calculation				I	E		Stiffness Calculation				I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	1658	1700	3.984045	0.300192	1.195979		Wood	1658	1700	14.94924	0.441561	6.601003		Wood	1658	1700	14.94924	0.448064	6.698209		Wood	1658
Steel	209.4	29000	32.20774	0.772545	24.88193	26.07791	Steel	209.4	29000	32.20774	0.862272	27.77184	34.37284	Steel	209.4	29000	32.20774	0.86537	27.8716	34.56981	Steel	209.4
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	1165	17.92	215.04				Beam 1	1165	17.92	215.04				Beam 1	1165	17.08	204.96				Beam 1	1165
Beam 2			0				Beam 2	1165	21.25	255				Beam 2	1165	21.25	255				Beam 2	

6th Floor

floor to floor height	10.94	ft	131.28	inches																		
			Left Column							Middle Column on Left							Middle Column on Right					
	I	E		Stiffness Calculation				I	E		Stiffness Calculation				I	E		Stiffness Calculation				I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	1658	1700	3.984045	0.300192	1.195979		Wood	2275	1700	20.51237	0.365587	7.499054		Wood	2275	1700	20.51237	0.371715	7.624761		Wood	1658
Steel	209.4	29000	32.20774	0.772545	24.88193	26.07791	Steel	257.7	29000	39.63674	0.835723	33.12534	40.62439	Steel	257.7	29000	39.63674	0.839306	33.26736	40.89212	Steel	209.4
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	1165	17.92	215.04				Beam 1	1165	17.92	215.04				Beam 1	1165	17.08	204.96				Beam 1	1165
Beam 2			0				Beam 2	1165	21.25	255				Beam 2	1165	21.25	255				Beam 2	

5th Floor

floor to floor height	10.94	ft	131.28	inches																		
			Left Column							Middle Column on Left							Middle Column on Right					
	I	E		Stiffness Calculation				I	E		Stiffness Calculation				I	E		Stiffness Calculation				I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	1658	1700	3.984045	0.300192	1.195979		Wood	2275	1700	20.51237	0.365587	7.499054		Wood	2275	1700	20.51237	0.371715	7.624761		Wood	1658
Steel	209.4	29000	32.20774	0.772545	24.88193	26.07791	Steel	257.7	29000	39.63674	0.835723	33.12534	40.62439	Steel	257.7	29000	39.63674	0.839306	33.26736	40.89212	Steel	209.4
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	1165	17.92	215.04				Beam 1	1165	17.92	215.04				Beam 1	1165	17.08	204.96				Beam 1	1165
Beam 2			0				Beam 2	1165	21.25	255				Beam 2	1165	21.25	255				Beam 2	

4th Floor

floor to floor height	10.94	ft	131.28	inches																		
			Left Column							Middle Column on Left							Middle Column on Right					
	I	E		Stiffness Calculation				I	E		Stiffness Calculation				I	E		Stiffness Calculation				I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	1658	1700	3.984045	0.300192	1.195979		Wood	2275	1700	20.51237	0.365587	7.499054		Wood	2275	1700	20.51237	0.371715	7.624761		Wood	1658
Steel	209.4	29000	32.20774	0.772545	24.88193	26.07791	Steel	257.7	29000	39.63674	0.835723	33.12534	40.62439	Steel	257.7	29000	39.63674	0.839306	33.26736	40.89212	Steel	209.4
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	1165	17.92	215.04				Beam 1	1165	17.92	215.04				Beam 1	1165	17.08	204.96				Beam 1	1165
Beam 2			0				Beam 2	1165	21.25	255				Beam 2	1165	21.25	255				Beam 2	

3rd Floor

floor to floor height	10.94	ft	131.28	inches																		
			Left Column							Middle Column on Left							Middle Column on Right					
	I	E		Stiffness Calculation				I	E		Stiffness Calculation				I	E		Stiffness Calculation				I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	1658	1700	3.984045	0.300192	1.195979		Wood	2275	1700	20.51237	0.365587	7.499054		Wood	2275	1700	20.51237	0.371715	7.624761		Wood	1658
Steel	209.4	29000	32.20774	0.772545	24.88193	26.07791	Steel	257.7	29000	39.63674	0.835723	33.12534	40.62439	Steel	257.7	29000	39.63674	0.839306	33.26736	40.89212	Steel	209.4
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	1165	17.92	215.04				Beam 1	1165	17.92	215.04				Beam 1	1165	17.08	204.96				Beam 1	1165
Beam 2			0				Beam 2	1165	21.25	255				Beam 2	1165	21.25	255				Beam 2	

2nd Floor

floor to floor height	13.34	ft	160.08	inches																		
			Left Column							Middle Column on Left							Middle Column on Right					
	I	E		Stiffness Calculation				I	E		Stiffness Calculation				I	E		Stiffness Calculation				I
	in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4	ksi	Part 1	Part 2	K/in	Sum K		in^4
Wood	1658	1700	8.245243	0.343431	2.831673		Wood	2275	1700	11.31359	0.41269	4.669007		Wood	2275	1700	11.31359	0.419087	4.741373		Wood	1658
Steel	209.4	29000	17.76416	0.805508	14.30917	17.14084	Steel	257.7	29000	21.86162	0.861175	18.82669	23.49569	Steel	257.7	29000	21.86162	0.864293	18.89485	23.63623	Steel	209.4
Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I	Span					Adj. Beams	I
	in^4	ft	in					in^4	ft	in					in^4	ft	in					in^4
Beam 1	1165	17.92	215.04				Beam 1	1165	17.92	215.04				Beam 1	1165	17.08	204.96				Beam 1	1165
Beam 2			0				Beam 2	1165	21.25	255				Beam 2	1165	21.25	255				Beam 2	

	Left Column								
E		Stiffness Calculation			Sum of K		Story Force	Def.	Drift
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	2.799404	0.498826	1.396416			36.4	13.86359	0.449308	2.989166
29000	6.820906	0.874504	5.964913	7.361329	30.85544				
	Span								
ft	in								
17.08	204.96								
	0								

	Left Column								
E		Stiffness Calculation			Sum of K		Story Force	Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	8.079777	0.41144	3.324347			17.1	20.38521	0.250798	2.539859
29000	19.68683	0.830344	16.34684	19.67119	81.28147				
	Span								
ft	in								
17.08	204.96								
	0								

	Left Column								
E		Stiffness Calculation			Sum of K		Story Force	Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	10.50414	0.390435	4.101187			13.3	25.44364	0.219771	2.289061
29000	25.59392	0.817664	20.92723	25.02841	115.7735				
	Span								
ft	in								
17.08	204.96								
	0								

	Left Column								
E		Stiffness Calculation			Sum of K		Story Force	Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	14.94924	0.310374	4.639848			13.3	30.12668	0.241379	2.06929
29000	32.20774	0.780871	25.15008	29.78993	124.8105				
	Span								
ft	in								
17.08	204.96								
	0								

	Left Column								
E		Stiffness Calculation			Sum of K		Story Force	Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	14.94924	0.310374	4.639848			13.3	34.68591	0.252474	1.827911
29000	32.20774	0.780871	25.15008	29.78993	137.3843				
	Span								
ft	in								
17.08	204.96								
	0								

	Left Column								
E		Stiffness Calculation			Sum of K		Story Force	Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	14.94924	0.310374	4.639848			13.3	39.14647	0.284941	1.575437
29000	32.20774	0.780871	25.15008	29.78993	137.3843				
	Span								
ft	in								
17.08	204.96								
	0								

Story Forces		Story Shear
23.10599	13.86359	13.86359
10.86936	6.521617	20.38521
8.43072	5.058432	25.44364
7.805066	4.68304	30.12668
7.598725	4.559235	34.68591
7.434251	4.46055	39.14647
7.029045	4.217427	43.36389
6.616364	3.969818	47.33371
6.871161	4.122697	51.45641

	Left Column								
E		Stiffness Calculation			Sum of K		Story Force	Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	14.94924	0.310374	4.639848			13.3	43.36389	0.315639	1.290496
29000	32.20774	0.780871	25.15008	29.78993	137.3843				
	Span								
ft	in								
17.08	204.96								
	0								

	Left Column								
E		Stiffness Calculation			Sum of K		Story Force	Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	14.94924	0.310374	4.639848			13.3	47.33371	0.344535	0.974857
29000	32.20774	0.780871	25.15008	29.78993	137.3843				
	Span								
ft	in								
17.08	204.96								
	0								

	Left Column								
E		Stiffness Calculation			Sum of K		Story Force	Def.	
ksi	Part 1	Part 2	K/in	Sum K	K/in	K	K	in	
1700	8.245243	0.354336	2.921589			13.3	51.45641	0.630322	0.630322
29000	17.76416	0.812919	14.44082	17.36241	81.63517				
	Span								
ft	in								
17.08	204.96								
	0								

Appendix K: Vertical Truss Bracing Design Calculations

LiUNA Headquarters Expansion Building Re-Design	
Document Name:	**Vertical Truss Diagonal Bracing Design**

LiUNA Headquarters Expansion Building Re-Design

Document Name:	**Vertical Truss Digaonal Bracing Design**

Design Values Fc 2100 psi Cd 1.6 Wind

			Member Information				
			Length	Column Size			
				d_1	d_2	A_g	A_{net}
Supporting PH Floor			in	in	in	in^2	in^2
Comp.	K	10	295	9.625	5.125	49.32813	39.07813
Tension	K	9.65	295	9.625	5.125	49.32813	39.07813
Supporting 9th Floor							
Comp.	K	11.6	259	8.25	5.125	42.28125	32.03125
Tension	K	15.3	259	8.25	5.125	42.28125	32.03125
Supporting 8th Floor							
Comp.	K	15.7	253	11	5.125	56.375	46.125
Tension	K	18	253	11	5.125	56.375	46.125
Supporting 7th Floor							
Comp.	K	20.6	253	6.875	6.75	46.40625	32.90625
Tension	K	19.8	253	6.875	6.75	46.40625	32.90625
Supporting 6th Floor							
Comp.	K	25.5	253	8.25	6.75	55.6875	42.1875
Tension	K	21.3	253	8.25	6.75	55.6875	42.1875
Supporting 5th Floor							
Comp.	K	31	253	9.625	6.75	64.96875	51.46875
Tension	K	22.4	253	9.625	6.75	64.96875	51.46875
Supporting 4th Floor							
Comp.	K	37.1	253	11	6.75	74.25	60.75
Tension	K	22.3	253	11	6.75	74.25	60.75
Supporting 3rd Floor							
Comp.	K	44.9	253	13.75	6.75	92.8125	79.3125
Tension	K	20.2	253	13.75	6.75	92.8125	79.3125
Supporting 2nd Floor							
Comp.	K	84.7	268	15.125	8.5	128.5625	111.5625
Tension	K	-9.8	268	15.125	8.5	128.5625	111.5625

Emin 900000 psi Ft 1350 psi

Slenderness Ratio			Critical Buckling				Column Sta	
l/d1	l/d2	Max. Ratio	with l/d1	with l/d2	Lesser FcE	FcE/Fc*	Part 1	Part 2
30.64935	57.56098	57.56098	787.5385	223.2842	223.2842	0.066454	0.592474	0.351026
31.39394	50.53659	50.53659	750.6244	289.6693	289.6693	0.086211	0.603451	0.364153
23	49.36585	49.36585	1398.488	303.5715	303.5715	0.090349	0.605749	0.366932
36.8	37.48148	37.48148	546.2843	526.6	526.6	0.156726	0.642626	0.412968
30.66667	37.48148	37.48148	786.6493	526.6	526.6	0.156726	0.642626	0.412968
26.28571	37.48148	37.48148	1070.717	526.6	526.6	0.156726	0.642626	0.412968
23	37.48148	37.48148	1398.488	526.6	526.6	0.156726	0.642626	0.412968
18.4	37.48148	37.48148	2185.137	526.6	526.6	0.156726	0.642626	0.412968
17.71901	31.52941	31.52941	2356.327	744.1879	744.1879	0.221484	0.678602	0.460501

Compression Member Sizing									
bility Factor		Gross Area Check							
		F'c	Ag	P capacity	P Demand	Pc>Pd		Pd/Pc	Fc*
Part 3	Cp	psi	in^2	Lbs	Lbs	?			psi
0.073837	0.065987	221.7178	49.32813	10936.92	10000	yes	0.914334		3360
0.09579	0.085413	286.9891	42.28125	12134.26	11600	yes	0.955971		3360
0.100387	0.08947	300.6176	56.375	16947.32	15700	yes	0.9264		3360
0.17414	0.153926	517.1907	46.40625	24000.88	20600	yes	0.858302		3360
0.17414	0.153926	517.1907	55.6875	28801.06	25500	yes	0.885384		3360
0.17414	0.153926	517.1907	64.96875	33601.24	31000	yes	0.922585		3360
0.17414	0.153926	517.1907	74.25	38401.41	37100	yes	0.96611		3360
0.17414	0.153926	517.1907	92.8125	48001.77	44900	yes	0.935382		3360
0.246094	0.215561	724.2848	128.5625	93115.86	84700	yes	0.909619		3360

Net Area Check					Tension Member Sizing		
A_{net}	P capacity	P Demand	$P_c>P_d$		Ft*Cd*An	T Demand	Td<Tc
in^2	Lbs	Lbs	?	P_d/P_c	lbs	lbs	?
39.07813	131302.5	10000	yes	0.07616			
					84408.75	9650	yes
32.03125	107625	11600	yes	0.107782			
					69187.5	15300	yes
46.125	154980	15700	yes	0.101303			
					99630	18000	yes
32.90625	110565	20600	yes	0.186316			
					71077.5	19800	yes
42.1875	141750	25500	yes	0.179894			
					91125	21300	yes
51.46875	172935	31000	yes	0.179258			
					111172.5	22400	yes
60.75	204120	37100	yes	0.181756			
					131220	22300	yes
79.3125	266490	44900	yes	0.168487			
					171315	20200	yes
111.5625	374850	84700	yes	0.225957			
					240975	-9800	yes

Appendix L: Vertical Truss Gravity Girder Design Calculations

LiUNA Headquarters Expansion Building Re-Design	
Document Name:	Vertical Truss Girder Design Check

VT Gravity Girder Design (Under the D+0.75L+).75(0.6W)+0.75S Load Case)

							Compressive S
Use 24-1.8E SP Variety		EMIN	950000	CD	1.6	Fc	1600

Penthouse									
D+0.75L+0.45W			Initial Trial Section Selection						
P Dem.	M Dem.	Length	Design P	Design M		S req	S used	A used
K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2
14.80	809.00	18.89	14.80	44.00	528000.00	137.50	268.00	129.90
9th Floor								
D+0.75L+0.45W			Initial Trial Section Selection					
P Dem.	M Dem.	Length	Design P	Design M		S req	S used	A used
K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2
15.60	566.00	18.89	62.00	43.00	516000.00	134.38	330.90	129.90
8th Floor								
D+0.75L+0.45W			Initial Trial Section Selection					
P Dem.	M Dem.	Length	Design P	Design M		S req	S used	A used
K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2
19.50	566.00	18.89	79.00	49.00	588000.00	153.13	330.90	129.90
7th Floor								
D+0.75L+0.45W			Initial Trial Section Selection					
P Dem.	M Dem.	Length	Design P	Design M		S req	S used	A used
K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2
21.60	566.00	18.89	97.00	59.00	708000.00	184.38	400.30	158.80
6th Floor								
D+0.75L+0.45W			Initial Trial Section Selection					
P Dem.	M Dem.	Length	Design P	Design M		S req	S used	A used
K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2
23.10	566.00	18.89	116.00	64.00	768000.00	200.00	476.40	173.30
5th Floor								
D+0.75L+0.45W			Initial Trial Section Selection					
P Dem.	M Dem.	Length	Design P	Design M		S req	S used	A used
K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2
24.30	566.00	18.89	138.00	69.00	828000.00	215.63	559.20	187.70
4th Floor								
D+0.75L+0.45W			Initial Trial Section Selection					
P Dem.	M Dem.	Length	Design P	Design M		S req	S used	A used
K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2
24.90	566.00	18.89	160.00	76.00	912000.00	237.50	648.50	202.10
3rd Floor								
D+0.75L+0.45W			Initial Trial Section Selection					
P Dem.	M Dem.	Length	Design P	Design M		S req	S used	A used
K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2
24.40	566.00	18.89	184.00	72.00	864000.00	225.00	744.40	216.60
2nd Floor								
D+0.75L+0.45W			Initial Trial Section Selection					
P Dem.	M Dem.	Length	Design P	Design M		S req	S used	A used
K	K'	ft	K	K'	Lbs-in	in^3	in^3	in^2
22.90	566.00	18.89	209.00	96.00	1152000.00	300.00	956.20	245.40

tress Capacity | | Bending Stress Capacity
$F_cC_D=F_c$* 2560 F_b 2400 $F_bC_D=F_b$* 3840 K 1

		Design Stresses		Column Slenderness Ratios					Critical Buckling	
		fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1	FCE 2
d1	d2	psi		in		Yes/No		Yes/No	psi	psi
12.38	10.50	1970.15	113.93	226.68	18.32	yes	21.59	yes	2327.34	1675.51

		Design Stresses		Column Slenderness Ratios					Critical Buckling	
		fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1	FCE 2
d1	d2	psi		in		Yes/No		Yes/No	psi	psi
13.75	10.50	1559.38	477.29	226.68	16.49	yes	21.59	yes	2873.26	1675.51

		Design Stresses		Column Slenderness Ratios					Critical Buckling	
		fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1	FCE 2
d1	d2	psi		in		Yes/No		Yes/No	psi	psi
13.75	10.50	1776.97	608.16	226.68	16.49	yes	21.59	yes	2873.26	1675.51

		Design Stresses		Column Slenderness Ratios					Critical Buckling	
		fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1	FCE 2
d1	d2	psi		in		Yes/No		Yes/No	psi	psi
15.13	10.50	1768.67	610.83	226.68	14.99	yes	21.59	yes	3476.64	1675.51

		Design Stresses		Column Slenderness Ratios					Critical Buckling	
		fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1	FCE 2
d1	d2	psi		in		Yes/No		Yes/No	psi	psi
16.50	10.50	1612.09	669.36	226.68	13.74	yes	21.59	yes	4137.49	1675.51

		Design Stresses		Column Slenderness Ratios					Critical Buckling	
		fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1	FCE 2
d1	d2	psi		in		Yes/No		Yes/No	psi	psi
17.88	10.50	1480.69	735.22	226.68	12.68	yes	21.59	yes	4855.80	1675.51

		Design Stresses		Column Slenderness Ratios					Critical Buckling	
		fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1	FCE 2
d1	d2	psi		in		Yes/No		Yes/No	psi	psi
19.25	10.50	1406.32	791.69	226.68	11.78	yes	21.59	yes	5631.58	1675.51

		Design Stresses		Column Slenderness Ratios					Critical Buckling	
		fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1	FCE 2
d1	d2	psi		in		Yes/No		Yes/No	psi	psi
20.63	10.50	1160.67	849.49	226.68	10.99	yes	21.59	yes	6464.83	1675.51

		Design Stresses		Column Slenderness Ratios					Critical Buckling	
		fb=M/S	fc=P/A	le	le/d1	le/d < 50?	le/d2		FCE 1	FCE 2
d1	d2	psi		in		Yes/No		Yes/No	psi	psi
23.38	10.50	1204.77	851.67	226.68	9.70	yes	21.59	yes	8303.71	1675.51

for Compression		Column Stability Factor					Critical Buckling for "Bear			
	FCE/(Fc*Cd)	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?
Lesser FCE		Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No
1675.51	0.65	0.92	0.84	0.73	0.58	1475.00	226.68	417.09	6.84	yes

for Compression		Column Stability Factor					Critical Buckling for "Bear			
	FCE/(Fc*Cd)	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?
Lesser FCE		Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No
1675.51	0.65	0.92	0.84	0.73	0.58	1475.00	226.68	417.09	7.21	yes

for Compression		Column Stability Factor					Critical Buckling for "Bear			
	FCE/(Fc*Cd)	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?
Lesser FCE		Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No
1675.51	0.65	0.92	0.84	0.73	0.58	1475.00	226.68	417.09	7.21	yes

for Compression		Column Stability Factor					Critical Buckling for "Bear			
	FCE/(Fc*Cd)	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?
Lesser FCE		Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No
1675.51	0.65	0.92	0.84	0.73	0.58	1475.00	226.68	417.09	7.56	yes

for Compression		Column Stability Factor					Critical Buckling for "Bear			
	FCE/(Fc*Cd)	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?
Lesser FCE		Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No
1675.51	0.65	0.92	0.84	0.73	0.58	1475.00	226.68	417.09	7.90	yes

for Compression		Column Stability Factor					Critical Buckling for "Bear			
	FCE/(Fc*Cd)	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?
Lesser FCE		Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No
1675.51	0.65	0.92	0.84	0.73	0.58	1475.00	226.68	417.09	8.22	yes

for Compression		Column Stability Factor					Critical Buckling for "Bear			
	FCE/(Fc*Cd)	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?
Lesser FCE		Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No
1675.51	0.65	0.92	0.84	0.73	0.58	1475.00	226.68	417.09	8.53	yes

for Compression		Column Stability Factor					Critical Buckling for "Bear			
	FCE/(Fc*Cd)	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?
Lesser FCE		Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No
1675.51	0.65	0.92	0.84	0.73	0.58	1475.00	226.68	417.09	8.83	yes

for Compression		Column Stability Factor					Critical Buckling for "Bear			
	FCE/(Fc*Cd)	Cp Calculation				Fc '	lu	le=1.84lu	RB	RB<50?
Lesser FCE		Part 1	Part 2	Part 3	Cp	psi	in	in		Yes/No
1675.51	0.65	0.92	0.84	0.73	0.58	1475.00	226.68	417.09	9.40	yes

n"		Beam Stability Factor				Volume Factor where x=20				Adjusted
FBE	FBE/Fb	CL Calculation				(5.125/b)^	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi		Part 1	Part 2	Part 3	CL	(1/x)	x)	x)	CV	fc
24350.46	9.51	5.53	30.61	10.01	0.99	0.96	1.00	1.01	0.97	3832.60

n"		Beam Stability Factor				Volume Factor where x=20				Adjusted
FBE	FBE/Fb	CL Calculation				(5.125/b)^	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi		Part 1	Part 2	Part 3	CL	(1/x)	x)	x)	CV	fc
21915.42	8.56	5.03	25.32	9.01	0.99	0.96	0.99	1.01	0.96	4176.42

n"		Beam Stability Factor				Volume Factor where x=20				Adjusted
FBE	FBE/Fb	CL Calculation				(5.125/b)^	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi		Part 1	Part 2	Part 3	CL	(1/x)	x)	x)	CV	fc
21915.42	8.56	5.03	25.32	9.01	0.99	0.96	0.99	1.01	0.96	4307.29

n"		Beam Stability Factor				Volume Factor where x=20				Adjusted
FBE	FBE/Fb	CL Calculation				(5.125/b)^	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi		Part 1	Part 2	Part 3	CL	(1/x)	x)	x)	CV	fc
19923.11	7.78	4.62	21.37	8.19	0.99	0.96	0.99	1.01	0.96	4292.38

n"		Beam Stability Factor				Volume Factor where x=20				Adjusted
FBE	FBE/Fb	CL Calculation				(5.125/b)^	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi		Part 1	Part 2	Part 3	CL	(1/x)	x)	x)	CV	fc
18262.85	7.13	4.28	18.33	7.51	0.99	0.96	0.98	1.01	0.95	4334.92

n"		Beam Stability Factor				Volume Factor where x=20				Adjusted
FBE	FBE/Fb	CL Calculation				(5.125/b)^	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi		Part 1	Part 2	Part 3	CL	(1/x)	x)	x)	CV	fc
16858.01	6.59	3.99	15.94	6.93	0.99	0.96	0.98	1.01	0.95	4386.14

n"		Beam Stability Factor				Volume Factor where x=20				Adjusted
FBE	FBE/Fb	CL Calculation				(5.125/b)^	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi		Part 1	Part 2	Part 3	CL	(1/x)	x)	x)	CV	fc
15653.87	6.11	3.74	14.02	6.44	0.99	0.96	0.98	1.01	0.95	4429.11

n"		Beam Stability Factor				Volume Factor where x=20				Adjusted
FBE	FBE/Fb	CL Calculation				(5.125/b)^	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi		Part 1	Part 2	Part 3	CL	(1/x)	x)	x)	CV	fc
14610.28	5.71	3.53	12.46	6.01	0.99	0.96	0.97	1.01	0.94	4474.39

n"		Beam Stability Factor				Volume Factor where x=20				Adjusted
FBE	FBE/Fb	CL Calculation				(5.125/b)^	(12/d)^(1/	(21/L)^(1/		FBX*CV +
psi		Part 1	Part 2	Part 3	CL	(1/x)	x)	x)	CV	fc
12891.42	5.04	3.18	10.09	5.30	0.99	0.96	0.97	1.01	0.94	4453.95

Beding Design Value		Beam-Column Loading Interaction			
			fb/(Fbx' * (1-	Combined	Combined < 1.0?
FBX*CL	Fbx'	(fc/Fc')^2	(fc/FCE)))	Loading Ratio	Yes/No
3817.72	3817.72	0.08	0.54	0.62	yes

Beding Design Value		Beam-Column Loading Interaction			
			fb/(Fbx' * (1-	Combined	Combined < 1.0?
FBX*CL	Fbx'	(fc/Fc')^2	(fc/FCE)))	Loading Ratio	Yes/No
3814.96	3814.96	0.32	0.49	0.81	yes

Beding Design Value		Beam-Column Loading Interaction			
			fb/(Fbx' * (1-	Combined	Combined < 1.0?
FBX*CL	Fbx'	(fc/Fc')^2	(fc/FCE)))	Loading Ratio	Yes/No
3814.96	3814.96	0.41	0.59	1.00	no

Beding Design Value		Beam-Column Loading Interaction			
			fb/(Fbx' * (1-	Combined	Combined < 1.0?
FBX*CL	Fbx'	(fc/Fc')^2	(fc/FCE)))	Loading Ratio	Yes/No
3812.13	3812.13	0.41	0.56	0.98	yes

Beding Design Value		Beam-Column Loading Interaction			
			fb/(Fbx' * (1-	Combined	Combined < 1.0?
FBX*CL	Fbx'	(fc/Fc')^2	(fc/FCE)))	Loading Ratio	Yes/No
3809.24	3809.24	0.45	0.50	0.96	yes

Beding Design Value		Beam-Column Loading Interaction			
			fb/(Fbx' * (1-	Combined	Combined < 1.0?
FBX*CL	Fbx'	(fc/Fc')^2	(fc/FCE)))	Loading Ratio	Yes/No
3806.28	3806.28	0.50	0.46	0.96	yes

Beding Design Value		Beam-Column Loading Interaction			
			fb/(Fbx' * (1-	Combined	Combined < 1.0?
FBX*CL	Fbx'	(fc/Fc')^2	(fc/FCE)))	Loading Ratio	Yes/No
3803.25	3803.25	0.54	0.43	0.97	yes

Beding Design Value		Beam-Column Loading Interaction			
			fb/(Fbx' * (1-	Combined	Combined < 1.0?
FBX*CL	Fbx'	(fc/Fc')^2	(fc/FCE)))	Loading Ratio	Yes/No
3800.14	3800.14	0.58	0.35	0.93	yes

Beding Design Value		Beam-Column Loading Interaction			
			fb/(Fbx' * (1-	Combined	Combined < 1.0?
FBX*CL	Fbx'	(fc/Fc')^2	(fc/FCE)))	Loading Ratio	Yes/No
3793.70	3793.70	0.58	0.35	0.93	yes

Appendix M: Vertical Truss Column Design Check Calculations

LiUNA Headquarters Expansion Building Re-Design	
Document Name:	**Vertical Truss Column Design Check**

Diagonal Design — Use 49 N1M 16 Visually Graded S.P.

	Compression Design Valu					
Design Values	Fc	2100	psi	CD	1.6	Wind

			Member Information				
			Length	Column Size			
				d1	d2	Ag	Anet
Supporting PH Floor			in	in	in	in^2	in^2
Ext. Col	K	58.5	204	12.375	10.5	129.9375	108.9375
Int. Col	K	64.5	204	12.375	10.5	129.9375	108.9375
Supporting 9th Floor							
Ext. Col	K	73.1	143.3	12.375	10.5	129.9375	108.9375
Int. Col	K	86.1	143.3	12.375	10.5	129.9375	108.9375
Supporting 8th Floor							
Ext. Col	K	86.1	131.3	12.375	10.5	129.9375	108.9375
Int. Col	K	108.1	131.3	12.375	10.5	129.9375	108.9375
Supporting 7th Floor							
Ext. Col	K	98.3	131.3	13.75	10.5	144.375	123.375
Int. Col	K	128	131.3	13.75	10.5	144.375	123.375
Supporting 6th Floor							
Ext. Col	K	109.6	131.3	15.125	10.5	158.8125	137.8125
Int. Col	K	146.2	131.3	15.125	10.5	158.8125	137.8125
Supporting 5th Floor							
Ext. Col	K	120.2	131.3	16.5	10.5	173.25	152.25
Int. Col	K	162.3	131.3	16.5	10.5	173.25	152.25
Supporting 4th Floor							
Ext. Col	K	130.2	131.3	16.5	10.5	173.25	152.25
Int. Col	K	175.3	131.3	16.5	10.5	173.25	152.25
Supporting 3rd Floor							
Ext. Col	K	140.4	131.3	16.5	10.5	173.25	152.25
Int. Col	K	183.7	131.3	16.5	10.5	173.25	152.25
Supporting 2nd Floor							
Ext. Col	K	151.5	160.1	19.25	10.5	202.125	181.125
Int. Col	K	151.2	160.1	19.25	10.5	202.125	181.125

es			
E_{MIN}	900000		psi

Slenderness Ratio			Critical Buckling				Column Sta	
l/d_1	l/d_2	Max. Ratio	with l/d_1	with l/d_2	Lesser F_{CE}	F_{CE}/F_C*	Part 1	Part 2
16.48485	19.42857	19.42857	2722.353	1959.894	1959.894	0.583302	0.879612	0.773717
16.48485	19.42857	19.42857	2722.353	1959.894	1959.894	0.583302	0.879612	0.773717
11.5798	13.64762	13.64762	5517.119	3971.92	3971.92	1.182119	1.212288	1.469643
11.5798	13.64762	13.64762	5517.119	3971.92	3971.92	1.182119	1.212288	1.469643
10.6101	12.50476	12.50476	6571.663	4731.115	4731.115	1.40807	1.337817	1.789753
10.6101	12.50476	12.50476	6571.663	4731.115	4731.115	1.40807	1.337817	1.789753
9.549091	12.50476	12.50476	8113.164	4731.115	4731.115	1.40807	1.337817	1.789753
9.549091	12.50476	12.50476	8113.164	4731.115	4731.115	1.40807	1.337817	1.789753
8.680992	12.50476	12.50476	9816.929	4731.115	4731.115	1.40807	1.337817	1.789753
8.680992	12.50476	12.50476	9816.929	4731.115	4731.115	1.40807	1.337817	1.789753
7.957576	12.50476	12.50476	11682.96	4731.115	4731.115	1.40807	1.337817	1.789753
7.957576	12.50476	12.50476	11682.96	4731.115	4731.115	1.40807	1.337817	1.789753
7.957576	12.50476	12.50476	11682.96	4731.115	4731.115	1.40807	1.337817	1.789753
7.957576	12.50476	12.50476	11682.96	4731.115	4731.115	1.40807	1.337817	1.789753
7.957576	12.50476	12.50476	11682.96	4731.115	4731.115	1.40807	1.337817	1.789753
7.957576	12.50476	12.50476	11682.96	4731.115	4731.115	1.40807	1.337817	1.789753
8.316883	15.24762	15.24762	10695.3	3182.074	3182.074	0.947046	1.081692	1.170058
8.316883	15.24762	15.24762	10695.3	3182.074	3182.074	0.947046	1.081692	1.170058

Compression Member Sizing								
		Gross Area Check						
bility Factor		F'_c	A_g	P capacity	P Demand	$P_c>P_d$		F_c*
Part 3	C_P	psi	in^2	Lbs	Lbs	?	P_d/P_c	psi
0.648113	0.525205	1764.689	129.9375	229299.3	58500	yes	0.255125	3360
0.648113	0.525205	1764.689	129.9375	229299.3	64500	yes	0.281292	3360
1.313466	0.817095	2745.441	129.9375	356735.7	73100	yes	0.204914	3360
1.313466	0.817095	2745.441	129.9375	356735.7	86100	yes	0.241355	3360
1.564522	0.863231	2900.457	129.9375	376878.2	86100	yes	0.228456	3360
1.564522	0.863231	2900.457	129.9375	376878.2	108100	yes	0.28683	3360
1.564522	0.863231	2900.457	144.375	418753.5	98300	yes	0.234744	3360
1.564522	0.863231	2900.457	144.375	418753.5	128000	yes	0.305669	3360
1.564522	0.863231	2900.457	158.8125	460628.9	109600	yes	0.237936	3360
1.564522	0.863231	2900.457	158.8125	460628.9	146200	yes	0.317392	3360
1.564522	0.863231	2900.457	173.25	502504.2	120200	yes	0.239202	3360
1.564522	0.863231	2900.457	173.25	502504.2	162300	yes	0.322982	3360
1.564522	0.863231	2900.457	173.25	502504.2	130200	yes	0.259102	3360
1.564522	0.863231	2900.457	173.25	502504.2	175300	yes	0.348853	3360
1.564522	0.863231	2900.457	173.25	502504.2	140400	yes	0.279401	3360
1.564522	0.863231	2900.457	173.25	502504.2	183700	yes	0.365569	3360
1.052273	0.738494	2481.341	202.125	501541	151500	yes	0.302069	3360
1.052273	0.738494	2481.341	202.125	501541	151200	yes	0.301471	3360

Net Area Check				
A_{net}	P capacity	P Demand	$P_c>P_d$	
in^2	Lbs	Lbs	?	P_d/P_c
108.9375	366030	58500	yes	0.159823
108.9375	366030	64500	yes	0.176215
108.9375	366030	73100	yes	0.19971
108.9375	366030	86100	yes	0.235227
108.9375	366030	86100	yes	0.235227
108.9375	366030	108100	yes	0.295331
123.375	414540	98300	yes	0.23713
123.375	414540	128000	yes	0.308776
137.8125	463050	109600	yes	0.236692
137.8125	463050	146200	yes	0.315733
152.25	511560	120200	yes	0.234968
152.25	511560	162300	yes	0.317265
152.25	511560	130200	yes	0.254516
152.25	511560	175300	yes	0.342677
152.25	511560	140400	yes	0.274455
152.25	511560	183700	yes	0.359098
181.125	608580	151500	yes	0.24894
181.125	608580	151200	yes	0.248447

Appendix N: Literature Review

LiUNA Headquarters Expansion Building Re-Design: Combining Old-World Heavy Timber Techniques with Modern Engineered Wood as a New Structural Solution

Literature Review
Josh Jaskowiak
AE 481 W Honors Thesis
For Dr. Thomas Boothby & Dr. Richard Mistrick: Structural and Honors Advisors, Respectively

As part of the requirements for completing the integrated BAE/MAE program while completing the requirements of the Schreyer Honors College, this literature review provides an overview of the topic of heavy timber construction. Topics discussed are the structural properties of timber design, strength and serviceability design of CLT systems, design of the charring method of fire protection, and moment-resisting connection details in heavy timber.

Industry-Published Design Aids and Manuals

The Timber Construction Manual by the American Institute of Timber Construction

The industry guide to heavy timber construction, The Timber Construction Manual moves beyond the scope of stick-frame applications and the NDS Design Guide to focus on heavy civil and commercial building applications. Recognizing that longer spans are often required, chapter 15 focuses on the design of moment splices at inflection points. Following an elastic analysis of the uncut section, a combination of tension straps, compression plates, and shear plates are used to complete the connection. Additional resources provide guidance on the design of glulam members for camber, spandrel beams, and glulam deck panels.

CLT Handbook by FP Innovations

Originally developed in Austria during the 1990’s, Cross-Laminated Timber or CLT has seen an increase in usage in the North American construction industry recently. In response to the increasing need for a unified design reference manual, FP Innovations with the Forest Product Laboratory, American Wood Council, APA, and Wood Products Council wrote the first edition of the CLT Handbook in 2013 based on current research in the fields of manufacturing methods, structural and performance design, material properties, and constructability of CLT systems.

Within the field of structural design, Ross, Gagnon, and Keith review the application of the “Shear Analogy” method developed by Kreuzinger for the analysis of shear and bending stresses in solid panels with perpendicular cross layers. Kreuzinger’s method summarizes the effects of the individual cross layers into two “beams” A and B along the top and bottom faces of the panel being analyzed. Through the summation of the individual contributions of each CLT layer’s stiffness in a particular direction, the net effect of all layers working to resist stresses can be quantified and used to size the member accordingly. The design equations used are integrally written with the NDS.

ANSI/APA PRG 320-2012 Standard for Performance-Rated Cross Laminated Timber by APA – The Engineered Wood Association

The standards for testing and material strength specifications for CLT products is published for use by professional engineers in design. The majority of the text focuses on the individual components, requirements, and qualifications of different CLT materials from a manufacturing perspective. Appendix A lists the mandatory design properties of products meeting the ANSI/APA PRG – 320 CLT standard.

***National Design Specification (NDS) for Wood Construction 2015 Edition* by the American Wood Council**

The 2015 NDS provides the industry standard for the design of solid-sawn, glulam, and CLT wood members for both strength and serviceability states. Within the field of connections, the NDS provides direction on the properties of bolts, nails, and screws and the yield limit equations applicable when these connectors are used in single and double shear conditions. However, the NDS is silent on the topic of moment-resistance connections. Therefore, additional research must be done to find a design criteria applicable to the moment resistance required by the project at-hand.

Additional Design References and Textbooks

"CLT Floor Design: Strength, Deflection, and Vibrations" by Scott Breneman

Three criteria govern the design of CLT floor design: strength, deflection, and vibration limitations. In his presentation, Breneman discusses the composition of CLT panels and the effect of varying properties between the minor and major axis on design. Through the assumption of one-way flexural action, panels can be designed according to the flexural capacity F'_bS_{eff} found within ANSI/APA PRG-320. Deflections can be found assuming one-way or two-way behavior. Vibration performance can be quantified by evaluating the natural frequency of the floor system to be greater than 9.0 Hz and the floor span being less that that prescribed.

***Design of Wood Structures ASD/LRFD* by Donald E. Breyer, Kelly E. Cobeen, Kenneth J. Fridley, and David G. Pollock**

Breyer's text is designed to be the accompanying narrative of the usage and capabilities of the NDS. Covering the application of the equations provided in the NDS, Breyer presents a clear guide for the design of wood structures for both strength and serviceability criteria using ASD and LRFD methodologies. Many of these topics are also covered at larger scale in the American Institute of Timber Construction Design Guide. Unfortunately, the Breyer text is also silent on the topic of moment resisting connections, therefore requiring further investigation in search of design aids.

***Timber Engineering* edited by Sven Thelandersson and Hans J. Larsen**

Utilizing an emphasis on finite element analysis and testing data, Thelandersson and Laresen provide a detailed discussion of wood connection elements in Part Three: Joint and Structural Assemblies. In order to achieve ductile failure, the European Yield Model supported by Johansen theory is presented for dowel fasteners.

***Multistory Timber Buildings Seismic Design Guide, 6th Edition* by M.P. Newcombe**

While Newcombe discusses many aspect of seismic analysis and wood design, his explanation of the design of moment-resisting steel gusset plate, expoxied rod connections, and post-tensioned connections for both strength and serviceability proves to be the most complete and recent design aid available.

Timber Bridges **by Christopher J. Mettem**

Published by the Timber Reseach and Development Association (TRADA), Mettem's overview of the design, construction, and testing of timber bridges across the world discusses the various types of structural materials and design methods that might be used to support modern bridge loading requirements. Beyond the typical scope of glulam and LVL materials, Mettem introduces various bridge deck framing schemes including LVL built-up decks, CLT decks, stressed-laminated glulam decks, and timber-concrete composite decks.

Moment-Resisting Connections and Lateral Design

"Basic Design Issues in Timber Frame Engineering II" by Tom Nehil and Amy Warren

In the investigation of raking forces in timber frames utilizing knee-brace construction, Nehil and Warren found that the traditional method of using braces at the intersection of the beam and columns was less effective than the use of braces at the intersection of the columns and base. The idealized infinite rigidty provided by the foundation provides more resistance to the axial forces transmitted by the knee braces, and therefore helps impose a condition more similar to a fixed connection at the upwind column base. Nehil and Warren do recognize, however, that the combination of shear wall or additional lateral force resisting systems are best suited for higher loads or multistory construction.

"Behavior of Traditional Timber Frame Structures Subjected to Lateral Load" by Robert G. Erikson and Richard J. Schmidt

In traditional timber-frames, knee braces with pegged mortise and tendon joinery provide lateral resistance. Typically used in residential, light commercial, or industrial applications, knee braces have been traditionally designed using rules of thumb without testing done to determine what stiffness is provided by the complete assembly. Erikson and Schmidt tested five (5) single story, single frame and four (4) two story, two frame knee brace configurations for response to a lateral load determined using assumptions based on the ASCE Wind Loading provisions.

The authors most commonly observed failure of the peg members located at the mortise and tendon joints rather than failure in the members themselves. Cyclic loading of varying duration was applied on the frame both with and without Structural Insulated Panels (SIPS). Erikson and Schmidt found that the stiffness provided by all knee brace configurations was not adequate to provide the required resistance to a lateral displacement of height/400 but was capable of providing adequate strength to resist lateral load. Therefore, the authors concluded that knee braces in conjunction with SIPs or additional lateral resistance is necessary in timber framing construction.

"Development of Moment Connections in Glued-Laminated Alberta Spruce and Pine Timber" by Peter Hattar and J.J. Rodger Cheng

The majority of the research surrounding wood moment connections is aimed at light commercial and residential applications. In order to make glulam framing systems more competitive, Hattar and Cheng looked to utilize rivet moment joints, shear plate joints, and circular bolted patterns for resisting rotation. Through a total of 15 different configurations, the properties of rivet connections versus bolted connections, butt joints versus lap joints, member thickness versus instantaneous center of rotation, and bottom and top bracing brackets versus none were studied.

The timber riveted connections performed better than the bolted connections. While the circular bolt patterns provided better results than the rectangular bolt patterns, a greater ultimate rotation angle was observed than that in the timber riveted connection. Splitting along the side grain at the rivets furthest from the I.C. was the primary mode of failure. The inclusion of a bottom and top bracket increased the stiffness at the joint, increased the ultimate moment capacity, and helped reduced wood crushing failure at the bottom inside corner of the beam and top inside corner of the column. A positive correlation between the increased spacing and reduced number of rivets along the side plates was observed.

Using the Hankinson Formula and an assumed behavior similar to that of eccentrically loaded bolt groups as presented by AISC, the authors conservatively predicted the ultimate moment capacity of the joints utilizing the circular bolted connections and timber riveted connections. These assumptions can be used to predict moment capacity, but further research is required to analyze the effects of additional components such as the bottom and top braces utilized in the later stages of testing.

"Seismic Design of Glulam Structures" by A.H. Buchanan and R.H. Fairweather

Commercial office spaces require open floor plans with limited possibilities for interruption with shear walls and large bracing elements. In order for an office building to be made out of wood, there must be a solution for lateral force resistance that does not impact space. In order to accomplish this goal, moment-resisting connections are preferred.

Several different types of moment connections in wood design are available. Although glued, nailed, and doweled connections are all available, Buchanan and Fairweather focused on the application of epoxied steel bars. Their requirements for ductile connections in the seismic region of New Zealand influenced this choice, but in a wind-governed design, stiffer connections would be acceptable.

Using reversed cyclic loading, Buchanan and Fairweather found that the most successful connections typically had a ductility factory between 1.5 and 3 with an ideal value of 2.0. This methodology prevents failure of the wood members while the connection members fail. While designs for single-story frames were investigated, four multi-story beam column connections were investigated. Three of the connections experienced brittle fracture in the column, the type of sudden failure within the structural member that is hopefully avoided in seismic loading. The fourth design featured steel brackets and nailon plates in addition to the threaded rods used before. The hysteresis loops exhibited a displacement ductility factor of approximately +/- 8.0, ensuring that ductile failure could occur.

Through their research, Buchanan and Fairweather show that ductile moment connections in heavy timber construction are possible when the lead design intent is to prevent brittle failure within the members and ensure ductile failure in the connecting elements. Through the use of a combination of steel plates and epoxied steel rods, this design intent can be achieved.

"Moment Resistance of Bolted Timber Connections with Perpendicular to Grain Reinforcements" by Frank Lam, Michael Schulte-Wrede, C.C. Yao, James J. Gu.

Hidden-plate connections are a popular choice among designers for their clean appearance and embedded fire protection of the steel plate within the wood. These connections, however, are not typically utilized for moment resistance. The authors used self-tapping wood screws to provide perpendicular-to-grain reinforcement based on the research from Blaß and Bejka. Using both monotonic and cyclic loading testing, the reinforced connections were found to have an increase in streght of 1.7 to 2.0 times that of their unreinforced counterparts. The both the unreinforced and reinforced connections utilized 11 3/8" x 5 1/8" beam and column members with ¾" bolts. The unreinforced connection had an ultimate moment of 26 Kip-feet while the reinforced connection had an ultimate moment of 46 Kip-feet. In order to increase strength and ductility, the inclusion of self-tapping screws perpendicular to grain is recommended.

Fire Protection and Charring Design

***Technical Report No. 10, Calculating the Fire Resistance of Exposed Wood Members* by the American Wood Council**

Since the 1960s, research has been conducted to determine the fire-resistive properties of wood members using the charring method. The charring method assumes that in a fire-damage event, the outside edges of the member will char and render this component of the section modulus inadequate to carry structural load, but this charring will prevent the fire from damaging the rest of the interior cross section.

Based on independent tests of flexural, compressive, tensile-loaded members, the American Wood Council has published design procedures for wood members. The strength provided by a member is directly affected by the loss of section modulus over time due to charring. Therefore, the effective char rate and char layer thickness for 1, 1.5, and 2 hour assemblies should be used to evaluate the remaining section modulus. Using the prescribed process, the member can be evaluated for expected gravity loads before and after the event.

In order to make fire-resistive design accessible, the American Wood Council provides guidance in two additional locations. First, within their "Design for Code Acceptance" Series, the AWC provides design guide *Design of Fire-Resistive Exposed Wood Members* formatted to accompany the NDS. Second, the NDS itself features an introductory design guide for typical prismatic sections and CLT members.

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ACADEMIC VITA

Education

The Pennsylvania State University, University Park, PA
The College of Engineering and Schreyer Honors College
Bachelor and Master of Architectural Engineering, Structural Option, ABET accredited
Expected graduation: May 2016 EIT status upon graduation

Thesis Information

Thesis Title: The Strucutral Redesign of the LiUNA Expansion Building Utilizing Cross-Laminated Timber and Engineered Wood Products and Associate Precedent Studies, Fire Protection Design, and Constructability Analysis

Thesis Supervisor: M. Kevin Parfitt, Professor of Architectural Engineering
Honors Adviser: Richard G. Mistrick

Work Experience & Professional Membership

The Pennsylvania State University, Architectural Engineering Department
Teaching Intern for AE 530: Computer Modeling of Structures **Spring 2016**
Teaching Intern for AE 430: Indeterminate Structures **Fall 2015**
Teaching Assistant and Woodshop Instructor AE 124S and AE 221:
Freshman Year Introduction Seminar and Building Materials **Fall 2012-Spring 2015**

Mulhern and Kulp, Ambler, PA **May 2015-August 2015**
Intern, Structural Engineering and Drafting

James G. Davis Construction Company, Rockville, MD **May 2014-August 2014**
Intern, Project Management

Skanska USA, Inc. Civil Southeast, Virginia Beach, VA **May 2013- August 2013**
Intern, Estimating Department

National Association of Home Builders, Penn State Chapter **August 2012-February 2014**
Project Manager and Course Coordinator of CE 411

Member of:
American Institute of Steel Construction
American Wood Council
National Association of Home Builders
Portland Concrete Association
Structural Engineers Association

Leadership

Penn State Division-Two Intercollegiate Baseball Club **September 2011- Present**
Collegiate Baseball Coach
-Helped lead the 2011-12, 2012-13, 2013-14. & 2014-2015 teams to the NCBA College World Series

Relevant Skills

Steel Connection Design Seismic and Wind Design Masonry Design Building Failures
Wood and Heavy Timber Design Steel Design Concrete Design Indeterminate Structures
Programs: Revit SketchUp AutoCAD Bluebeam
SAP 2000 ETABS STAAD Enercalc

Special Honors

Architectural Engineering Department Scholarship	2013, 2014, 2015
James M. Pohlen Memorial Scholarship	2015
Leonhard Engineering Honors Scholarship	2012-2015
College of Engineering, Dean's List	Fall 2011-Spring 2015
Recipient of Schreyer Honors College Scholarship	2011-2015