Structural System
Analysis and Redesign

Structural Requirements

Pre-Cast Concrete Design

Engineered Wood Design

Structural System Analysis and Redesign:

Structural Requirements
Gravity System Requirements

As Boyds Bear Country is home to multiple use spaces, it is subject to multiple loading conditions. In the application of the alternate framing systems, the only gravity loading condition to vary is that of floor dead loads. Gravity loading requirements can be seen below.

Original Design Floor Dead Loads:	**[psf]**
Composite Floor Slab 5½" with 3" Deck	50
Beams / Columns	8
Flooring Finish	1
Ceiling Finish	1
Mechanical / Electrical	5
Total	**65**

Pre-Cast Concrete Design Floor Dead Loads:	**[psf]**
Double-Tee	82
Girders / Columns	25
Flooring Finish	1
Ceiling Finish	1
Mechanical / Electrical	5
Total	**114**
Compared to Original Steel System	**175%**

Wooden Design Floor Dead Loads:	**[psf]**
Joists / Sheathing	15
Girders / Columns	3
Flooring Finish	1
Ceiling Finish	1
Mechanical / Electrical	5
Total	**25**
Compared to Original Steel System	**38%**

Design Floor Live Loads:	[psf]
Retail Areas	100
Office Areas	100
Stairs, including landings, platforms, and exits	100
Light Storage Area	125

Design Roof Dead Loads:	[psf]
Wooden trusses	15
Sheathing and finish	5
Total	**20**

Design Roof Live Loads:		[psf]
Minimum Roof Live Load		20
Ground Snow Load	Pg	15
Flat Roof Snow Load	Pf	15.0
Snow Exposure Factor	Ce	1.0
Snow Load Importance Factor	I	1.0
Thermal Factor	Ct	1.0

Additional Design Dead Loads:		Unit
Escalators (each)	30	kips
Stairs	100	psf
Elevator	100	psf
Decorative Timbers (each)	10	kips
Fireplace (ground to 2nd floor)	150	pcf
(3rd and 4th floors)	75	pcf
Exterior Light Gauge Walls	10	psf

Special considerations are made for additional systems in the original structural design, and are likewise applied to each redesign. Additional loads are supplied by decorative wooden timbers, at 10 kips each, the masonry fireplace, which is considered to carry its own weight, and two large escalators, weighing approximately 30 kips each. Stairwells and elevators are designed as would be in any typical building at 100 psf.

Lateral System Requirements

The original lateral design of Boyds Bear Country references both ASCE 7-95 and ASCE 7-98 within its calculations in addition to the requirements of the Standard Building Code. All of these calculations were completed using Allowable Stress Design methods, and steel members were originally chosen using AISC Manual of Steel Construction, 9th Edition (1989).

For the purposes of this redesign, current codes are applied, including the 2003 International Building Code and ASCE 7-05. New members are selected using Load Factor and Resistance Design. Load combinations applied to the design are as follows:

1.4 Dead
1.2 Dead + 1.6 Live + 0.5 Roof Live
1.2 Dead + 1.6 Roof Live + (0.5 Live or 0.8 Wind)
1.2 Dead + 1.6 Wind + 0.5 Live + 0.5 Roof Live
1.2 Dead + 1.0 Earthquake + 0.5 Live + 0.2 Roof Live
0.9 Dead + (1.0 Earthquake or 1.6 Wind)

The controlling load combination for the lateral system of both the original structural system and the pre-cast concrete system is 1.2 Dead + 1.0 Earthquake + 0.5 Live + 0.2 Roof Live. The controlling load combination for the lateral design of the wooden structure is 1.2 Dead + 1.6 Wind + 0.5 Live + 0.5 Roof Live. Derivations of these loads may be found in the appendix, while the resulting lateral load values of each can be seen below.

Original System Vertical Seismic Distribution:	**$w_x h_x$**	**C_{vx}**	**V [kips]**	**V / Wall**
1st floor	51541	0.113	101	25
2nd floor	89984	0.197	176	44
3rd floor	126276	0.277	246	62
4th floor	187889	0.412	366	92
Total (Base)	**455691**	**1.000**	**889**	**222**

Pre-Cast Concrete System Vertical Seismic Distribution:	**$w_x h_x$**	**C_{vx}**	**V [kips]**	**V / Wall**
1st floor	69642	0.108	105	26
2nd floor	121499	0.188	183	46
3rd floor	173549	0.269	261	65
4th floor	279921	0.434	421	105
Total (Base)	**644611**	**1.000**	**968**	**242**

Wooden System Vertical Seismic Distribution:	$w_x h_x$	C_{vx}	V [kips]	V / Wall
1st floor	37061	0.109	56	14
2nd floor	64772	0.191	97	24
3rd floor	87352	0.258	131	33
4th floor	149618	0.442	225	56
Total (Base)	**338803**	**1.000**	**510**	**127**

Wooden System Vertical Wind Distribution:	Wall A [sf]	Roof A [sf]	V [kips]	V / Wall
Wind Loading E-W				
Grade	2088	0	41.8	10.4
1st floor	4152	0	85.4	21.4
2nd floor	4152	0	92.3	23.1
3rd floor	4152	0	96.6	24.2
4th floor	2064	7368	288.4	72.1
Total (Base Shear)	**16608**	**7368**	**604.5**	**151.1**
Wind Loading N-S				
Grade	1044	0	16.8	4.2
1st floor	2076	0	34.6	8.6
2nd floor	2076	0	38.0	9.5
3rd floor	2076	0	40.2	10.0
4th floor	3795	921	97.2	24.3
Total (Base Shear)	**11067**	**921**	**226.7**	**56.7**

Foundation System Requirements

Existing foundations of Boyds Bear Country consist of shallow footings. All wall footings are simple thickened slabs measuring 2’-0” wide and 1’-0” thick. Column footings extend to a maximum of 3’-0” thick.

Foundations of the building are designed with a bearing pressure of 3,000 psf based on geotechnical investigations of the site. Typically, exterior footings extend to 3’ below finished grade, to account for frost depths.

Footings consist of 3,000 psi cast-in-place concrete with reinforcing billet steel of ASTM A615, grade 60, with class B splices.

Portions of the Structure Unchanged in Redesign Options

Several portions of the existing building are designed in a manner which clearly is the best option available. As such, these portions of the structure will remain consistent in all variations of structural designs considered.

The roof framing is accomplished through the use of wooden trusses, manufactured to specifically meet the requirements of the building. The trusses used in the original structure were designed and manufactured by Witt Building Materials of Knoxville, Tennessee. Trusses are spaced at 2 feet on center across the roof of the building. Variations in this layout are introduced to incorporate gables and copulas incorporated into the roof structure. These roof trusses are the most efficient and effective means of roof support, and thus will remain generally unchanged in all three structural designs.

All forms of vertical transportation are to remain generally the same as well. The two large escalators flanking the interior atrium of the building are specially designed for the space. Two elevators service the building. In the structural redesigns, one of these elevators is relocated to mirror the location of the other; however the elevator itself is left unchanged. This relocation allows for a symmetrical placement of shear walls and a continuous diaphragm. The walls surrounding the elevators are changed from masonry to pre-cast concrete panels to allow for a decreased requirement of field labor and to match the shear walls. Two stairwells are located in the building as well. These remain in the same location in all designs, and similarly to the elevators, the walls of the stairwells are changed from masonry to pre-cast concrete panels.

The existing wall framing system is composed of cold-rolled steel studs. This system remains in place as is in the pre-cast concrete redesign. Wall framing in the wooden redesign is adjusted to traditional wood stud framing. Standard dimension studs may be used in all cases.

In addition to the main structure, Boyds Bear Country also features several small buildings on the property, a silo, and pavilion or porch spaces. These areas are not part of the main structure and not included in either of the structural redesigns.

Pre-cast Concrete System Design

Floor System Design

The conversion from structural steel to pre-cast concrete allows for larger spans and thus larger bays within the building as determined by specific member designs. Bay size in this application is increased from 30’x30’ to 30’x45’, allowing for a decrease number of columns, connections, and individual members. The typical framing layout may be seen in Figure 11.

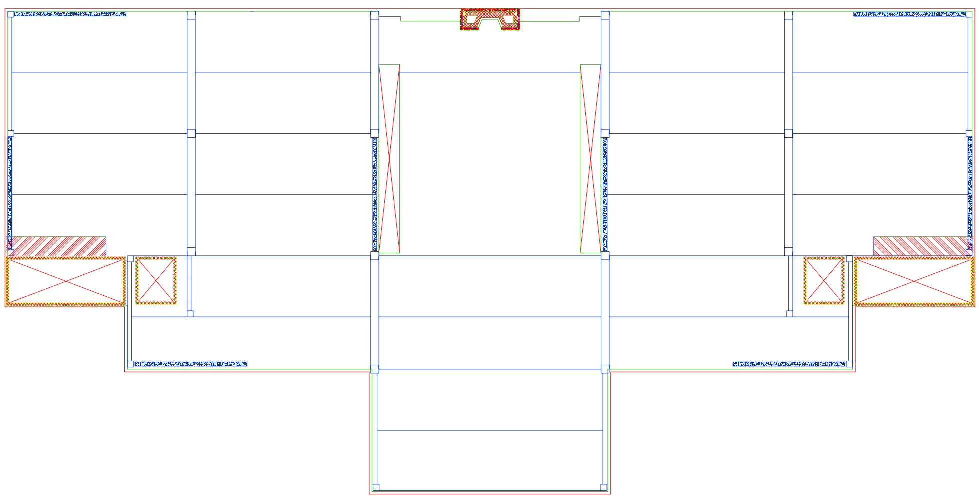

Figure 11: Pre-Cast Concrete Floor Plan

The major framing component of the pre-cast concrete gravity system is the standard 15’ wide double-tee. These members were designed by determining live load requirements and desired spans. Manufacturer supplied load tables were then used which conform to standard PCI section design, and are included in the appendix. Double-tees located in the majority of the building, subject to retail and office loading, are designed to withstand 100 psf of live load. Double-tees located below mechanical areas, along the Eastern and Western faces of the structure, are designed to withstand 125 psf of live load. Members subject to the typical 100 psf live load are selected as 15DT34 128-S members, a 34” deep section with 12 half-inch pre-stressing strands in each stem. Members subjected to 125 psf live load are selected as 15DT34 168-S members, a 34” deep section with 16 half-inch pre-stressing strands in each stem.

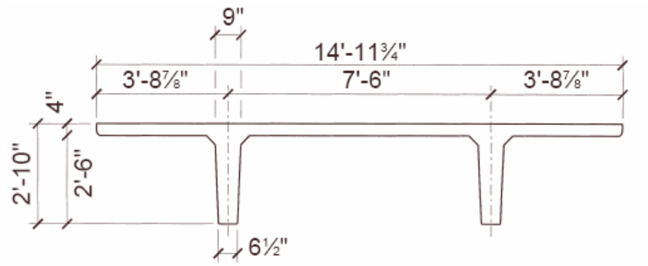

Figure 12: 15DT34 Cross-Section

The double-tees are supported with pre-cast L and inverted T girders, which are designed using similar load tables. The L girders are used where the structure frames into only one side of the beam, while the inverted T girders are implemented where the structure will be framing into both sides of the beam. Loading is determined by the live load applied to the supported double tees, and the girders span the width of two double-tees. All L girders in the structure are 12LB36 118-S sections, being a 36" deep section with 11 half-inch pre-stressing strands. Inverted T beams are designed as 24IT36 228-S and 24IT36 268-S sections, being 36" deep with 22 or 26 half-inch strands respectively.

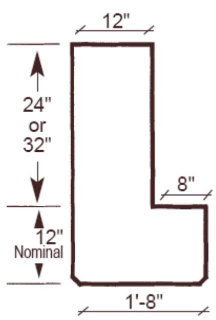

Figure 13: 12LB36 Cross-Section

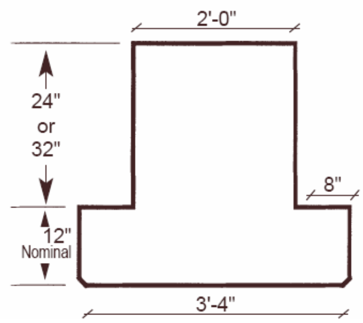

Figure14: 24IT36 Cross-Section

The pre-cast double-tees which act as the bulk of the floor system allow for larger spans then the original steel framing. As a result, bay sizes may be increased and the amount of columns required in the structure may be reduced. As applied in Boyds Bear Country, bay sizes increase from 30'x30' to 30'x45' and 30'x60'. Typical bay sizing can be seen in floor plans included in the appendix. The implementation of larger bay sizes allows for a reduced number of columns. Within a typical floor, the number of columns decreases from 36 to 28.

The columns used in this system are pre-cast concrete as well. All columns are considered to support only gravity loads, the loading of which was determined of both dead and live loads; internal loading was determined from these values using RISA-3D. These internal loading results were then applied to PCAColumn, which determined required sizes for both concrete and steel. Interior columns measure 24"x24" and all exterior columns measure 18"x18". Concrete strengths range from 6,000 psi to 8,000 psi as required by loading conditions. All specific calculations and final designs of pre-cast columns may be found in the appendix.

Lateral System Design

In the implementation of a fully pre-cast floor system, pre-cast concrete panels may be used as shear walls to resist lateral loads in the structure. These panels can be manufactured at, and shipped from, the same plant as all pre-cast floor members previously selected. The panels and the fully connected unit of the shear walls were designed using finite element analysis in RISA-3D. Specific information from these models can be found within this section, in the appendix, and remaining information is available upon request.

In the pre-cast concrete structural system seismic loads control the design. Lateral forces are applied at each floor level as were determined previously, and gravity loads are applied at each floor on the East-West force resisting walls.

All shear walls in the building are sized equally, each measuring 26.5 feet wide, 14 inches thick, and composed of 7,000 psi concrete. With all panels and the shear walls being the same size and composition, their relative stiffnesses are all equal, thus simplifying the analysis of the structure and minimizing influences such as building torsion.

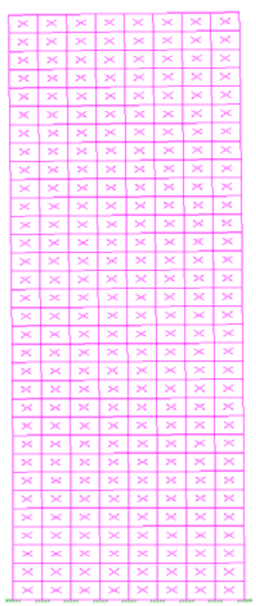

Figure 15: Deflected Shape of
East-West Force Resisting Wall
Magnified 40 Times in Pre-Cast System

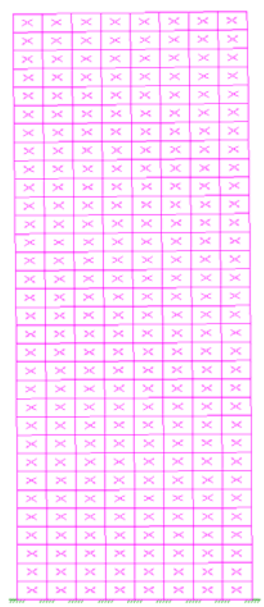

Figure16: Deflected Shape of
North-South Force Resisting Wall
Magnified 40 Times in Pre-Cast System

Total seismic base shear for the concrete redesign is 1237 kips. As all the shear walls in the structure possess the same rigidity, the building base shear may be evenly distributed amongst the walls, resulting in the loading conditions which may be found in

previous sections and the appendix. The controlling applied loads at the base of the member are as follows:

Shear walls resisting East-West forces: 91.3 kips/ linear foot
Shear walls resisting North-South forces: 111.8 kips/ linear foot

Both of these values fall below the required stress limit for pre-cast concrete members of $\frac{5\sqrt{f'c}\cdot b\cdot d}{12\cdot 1000}$ = 155.2 kips / linear foot.

Deflection at the top of the shear wall is minimal, calculated at a maximum of 0.17”. This value is below the suggested live load deflection maximum of L/360 which results as 2.3”.

Uplift forces are created within the base of the shear walls as the result of an overturning moment generated through the height of the wall about its base. In walls resisting East-West lateral forces, an uplift force of 313 kips is created; in walls resisting North-South lateral forces, an uplift of 277 kips is created. Both forces are counteracted with the placement of 4 number 11 bars in each corner. The summations of all forces on both walls results in a positive reaction value, causing no overturning in the wall as a whole.

The lateral forces of the structure are transferred to the shear walls through the diaphragm. This is accomplished with additional reinforcement placed in the pour strips on the surface of the double-tees. The full calculation of the required amount of reinforcement can be found in the appendix. In the North-South Direction, 2 #6 bars should be run continuously along the edges of all 8 bays, and in the East-West direction 4 #6 bars should be run continuously along the edges of all 4 bays.

The layout of shear walls within the building was determined so that building torsion is kept to a minimum and lateral forces are evenly distributed throughout the building. The placement of these walls may be seen in Figure 17, and a floor plan of the original steel design may be seen in Figure 18. In order to successfully connect the diaphragm to the shear walls, it was necessary to relocate the Southern mechanical chase. More information regarding this adjustment may be found in the architectural floor plan analysis.

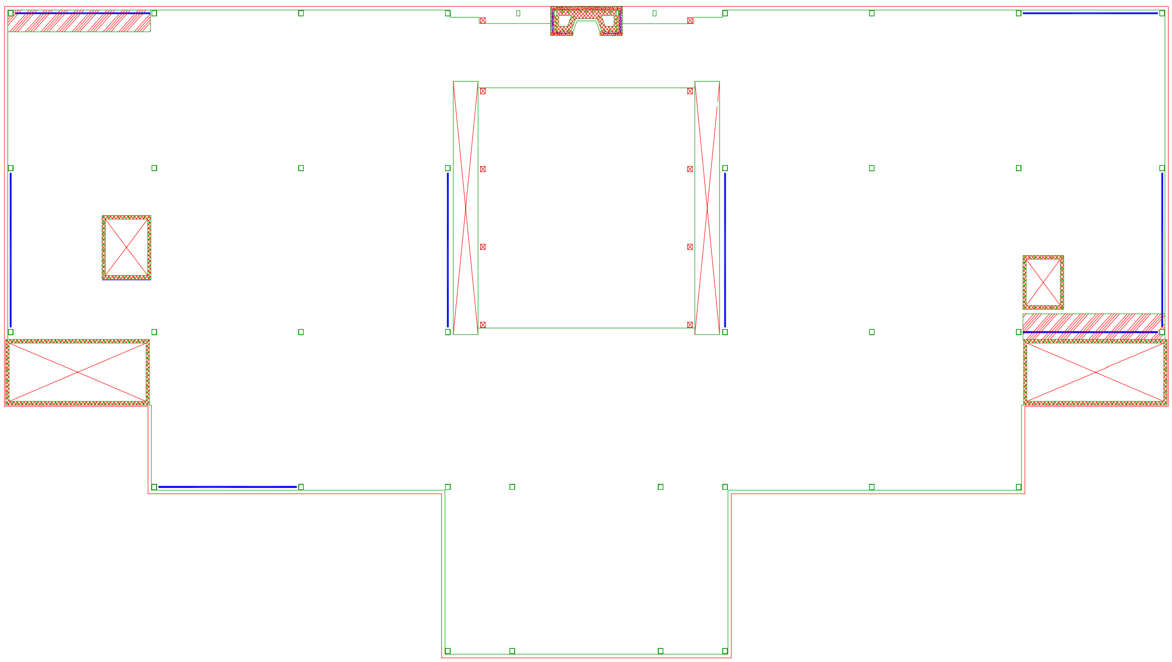

Figure 17: Original Steel Lateral System Plan

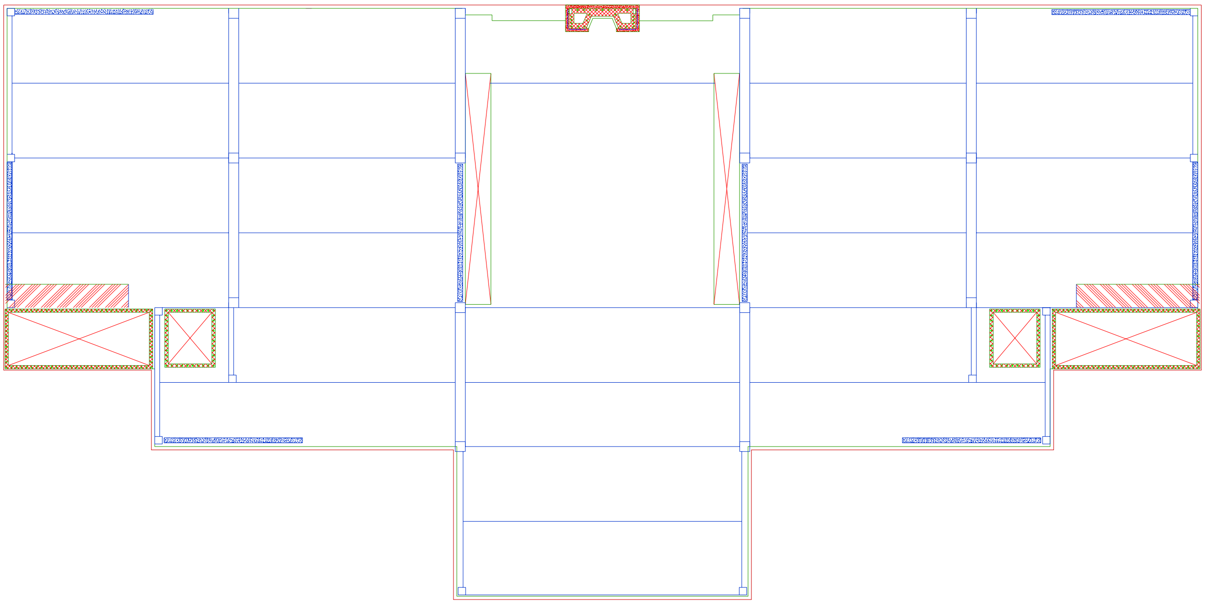

Figure 18: Pre-Cast Concrete Lateral System Plan

Effects on Foundation Design

Foundations supporting the structure have altered requirements under the loading conditions created by the change from a steel structure to a pre-cast concrete structure. The column footings decrease in number and increase in size under the adjusted loading conditions.

The specific design of the footings is completed using hand calculations in MathCAD. In the case of typical column footings, no lateral load is to be carried, and thus overturning and uplift are not considered on the foundation itself. The typical column footings are designed to satisfy ACI requirements of punching shear and one-way shear. Specific calculations may be seen in the appendix.

In the design of the footing supporting typical interior column, the width of the footing remains the same at 12.5 feet, however the depth of the footing is increased from 30 inches to 36 inches. This represents a 20% increase in individual volumetric footing size. When the adjusted number of footings throughout the building is considered, there is approximately a 15% increase in total footing volume.

Wooden System Design

Floor System Design

The conversion from structural steel to engineered wood products creates the need for smaller spans and thus smaller bays within the building, controlled by specific member design. Bay size in this application is decreased from 30'x30' to 22.5'x30'; this causes an increase in the number of columns, connections, and individual members, however more traditional installation methods may be used in construction. The typical framing layout may be seen in Figure 19.

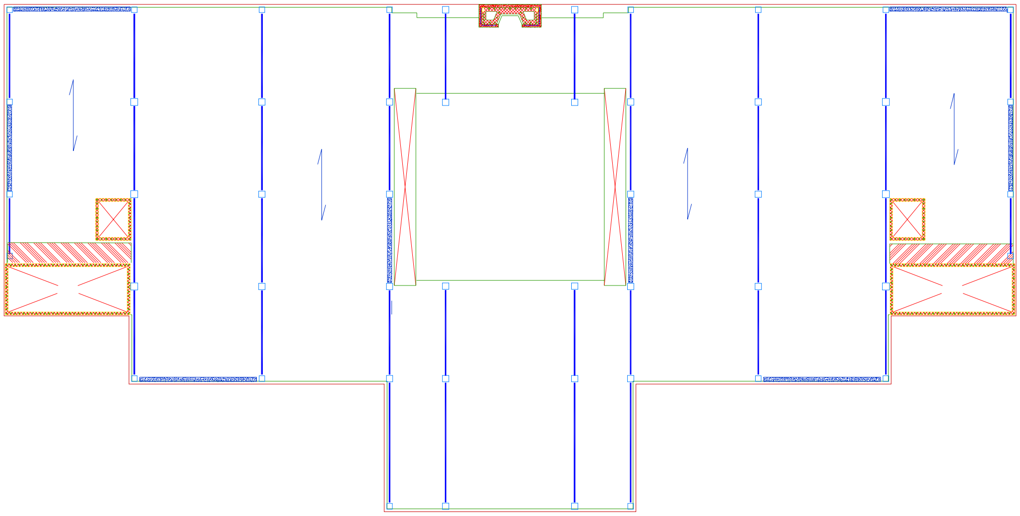

Figure 19: Engineered Wood Floor Plan

The main floor structure is composed of open web floor trusses, as specified by iLevel and Trus Joist commercial. These members were designed by determining the supported live loads and total loads of trusses spaced at varying distances. In order to balance strength and service requirements without exceeding a reasonable member depth, all trusses are designed at 30 inches deep, spanning 30 feet, and spaced at 2 feet on center. The selection of these members also allows for the use of standard pieces, minimizing additional design and manufacturing casts.

TJM trusses with parallel chords are selected to support typical areas subject to 100 psf live load. TJH trusses with parallel chords are selected to support areas subject to mechanical loads of 125 psf. Specific information regarding the sizing and construction of TJM and TJH trusses may be found in the appendix.

Supporting the trusses are Parallam PSL girders, also specified by iLevel and TrusJoist. These members were designed using hand calculations in MathCAD, and follow all NDS requirements. These calculations are included in the appendix. Most adjustment factors

apply as a factor of 1.0 in the design of the girders as the members are used in a climate controlled building and not subject to unusual loading conditions. Standard adjustments are made to account for member size and volume as required. Supported loads applied to the girders are idealized as a uniform load.

Girders span approximately 22 ½ feet each and all selected members are of standard dimensions to minimize required lead time and construction costs. Typical girders measure 10 ½ inches thick and 28 inches deep. Girders supporting higher loads extend to 34 inches deep.

Columns are selected in a similar manner to that of the girders, and are made of SP 50 N1D14 Glu-lam, following requirements of NDS, and available from any engineered wood manufacturer. Columns are designed to reach the full height of each floor; however a bracing point is included on each column at the lower surface of the plenum. This is accomplished by including additional girder style members within the exiting ceiling system along all column lines. The existing structure has non-structural wooden members in similar application, as can be seen in Figure 20. In the central atrium where a ceiling plenum does not exist, diagonal bracing is applied, as in the original structure. The effect of this bracing creates an effective column length of 10 feet and can be seen in Figure 21.

Figure 20: Existing Faux Girders[2]

Figure 21: Existing Diagonal Column Bracing[2]

Wooden column sizes vary by floor due to the effects of supported cumulative loads, and only support gravity loads. Specific calculations may be found in the appendix, and a summary of typical sizes may be seen in the table below.

Typical Column Sizes			**[in]**
	Typical Loading	Central Span	Mech. Loading
Supporting Roof	7 x 7	7 x 7	7 x 7
4th Floor	12 x 12	14 x 14	12 x 12
3rd Floor	15 x 15	18 x 18	16 x 16
2nd Floor	18 x 18	20 x 20	20 x 20
1st Floor	20 x 20	22 x 22	22 x 22

In order to create the actual floor of the wooden floor system, standard wooden plank is used. These members were designed using hand calculations in MathCAD, and NDS requirements. Select Structural grade Southern Pine plank of nominal measurements of 2 inches by 6 inches span the joists spaced at 2 feet on center. The use of Southern Pine allows for the application of an economical product, as compared to engineered wood, and exhibits more highly researched strength and performance conditions, as compared to other species of woods.

Lateral System Design

In the preliminary design of the engineered wood system, masonry shear walls were to be applied as the lateral fore resistance system; through further investigation, pre-cast panels were selected to replace the initial masonry walls. This alteration in design adds an additional material to the structural system of the building, as masonry is used in the foundation and retaining walls. However, pre-cast panels prove to be more efficient and effective in this application and in this structure. Concrete panels are more highly suited to reach the heights required in the structure, reaching to 70 feet tall. The amount of site construction required for installation is greatly decreased, as the construction of the walls will only require the grouting of pre-installed connections. With the application of pre-cast concrete in the shear walls of the structure, pre-cast may also be used to surround the stairwells and elevators, thus lowing construction time and costs in additional areas of the building.

The individual panels and the fully connected unit of the shear walls were designed using finite element analysis in RISA-3D. Lateral loads are applied at each floor level as transferred from the wooden diaphragm. Gravity loads are applied to the North-South resisting walls at each floor level as well. Diagrams of the walls may be seen below.

Shear walls in the same direction are sized equally, with East-West load resisting walls measuring 26.5 feet wide and North-South resisting walls measuring 20 feet wide. All shear walls in this system are 12 inches thick, and composed of 7,000 psi concrete. With panels and the shear walls in the same direction being the same size and composition, their relative stiffnesses are equal, thus simplifying the analysis of the structure and minimizing influences such as building torsion.

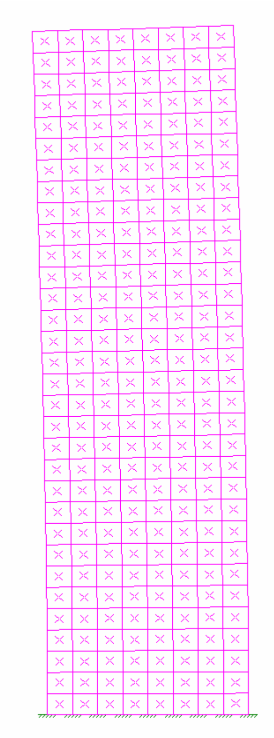

Figure 22: Deflected Shape of East-West Force Resisting Wall Magnified 40 Times in Wooden System

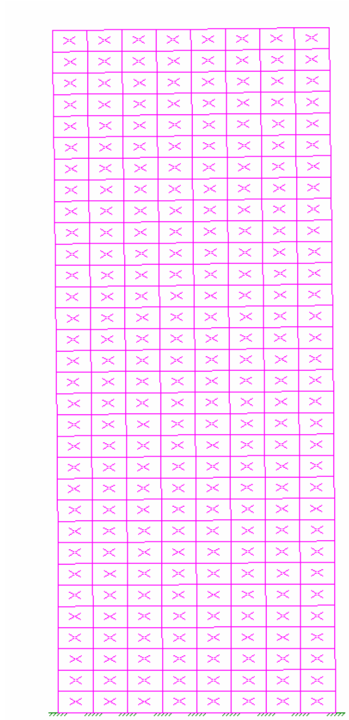

Figure 23: Deflected Shape of North-South Force Resisting Wall Magnified 40 Times in Wooden System

Total un-factored wind induced base shear for the wooden redesign is 605 kips. As all the shear walls in a direction possess the same rigidity, the building base shear may be evenly distributed amongst the walls, resulting in the loading conditions which may be found in the appendix. The controlling applied loads at the base of the member are as follows:

Shear walls resisting East-West forces: 102.8 kips/ linear foot
Shear walls resisting North-South forces: 53.3 kips/ linear foot

Both of these values fall below the required stress limit for pre-cast concrete members of $\frac{5\sqrt{f'c}\cdot b\cdot d}{12\cdot 1000}$ = 100.4 kips / linear foot and 133.0 kips / linear foot, respectively.

Deflection at the top of the shear wall is minimal, calculated at a maximum of 0.461" in the East-West force resisting walls and 0.193" in the North-South force resisting walls. Both of these values are below the suggested live load deflection maximum of L/360 which results as 2.3".

Uplift forces are created within the base of the shear walls as the result of an overturning moment generated through the height of the wall about its base. In walls resisting East-West lateral forces, an uplift force of 309 kips is created; in walls resisting North-South

lateral forces, an uplift of 230 kips is created. Both forces are counteracted with the placement of 4 number 11 bars in each corner. The summations of all forces on both walls results in a positive reaction value, causing no overturning in the wall as a whole.

The layout of shear walls within the building was determined so that building torsion is kept to a minimum and lateral forces are evenly distributed throughout the building. The placement of these walls may be seen in Figure 24, and a floor plan of the original steel design may be seen in Figure 25. In order to successfully connect the diaphragm to the shear walls, it was necessary to relocate the Southern mechanical chase. More information regarding this adjustment may be found in the architectural floor plan analysis.

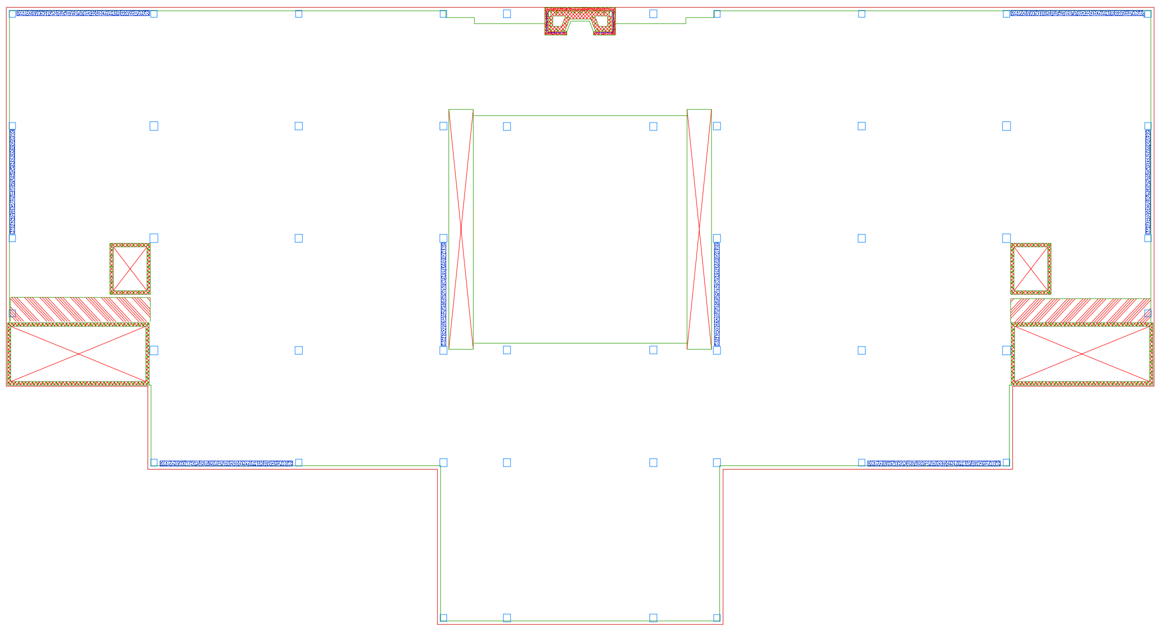

Figure 24: Engineered Wood Lateral System Plan

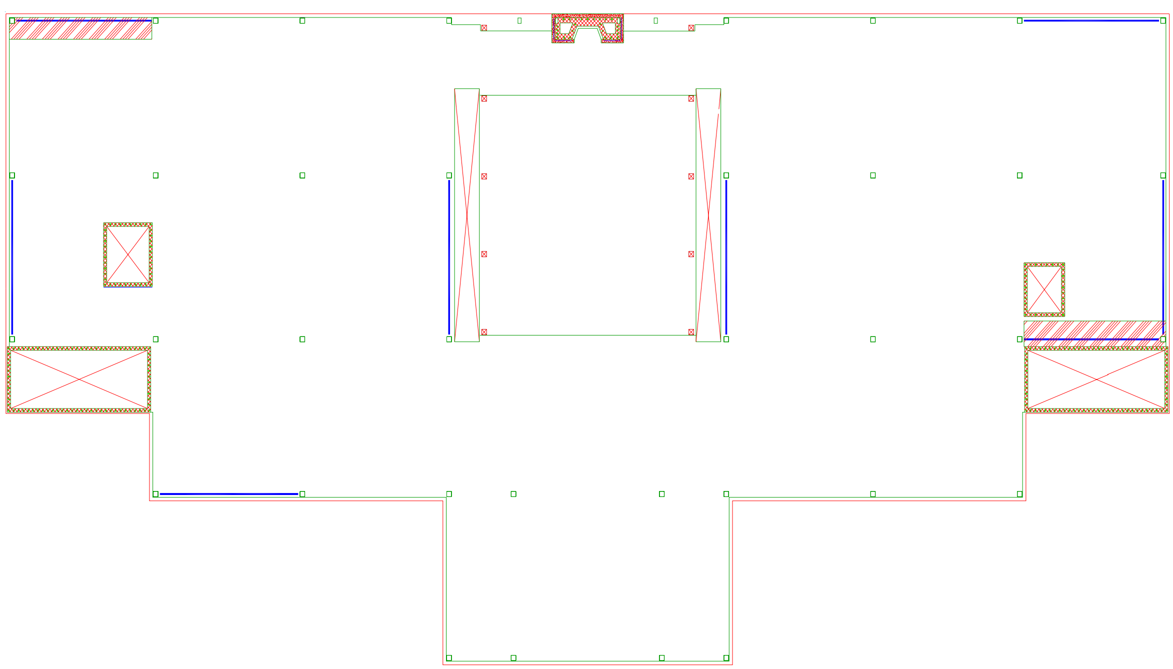

Figure 25: Original Steel Lateral System Plan

Effects on Foundation

Foundations supporting the structure have altered requirements under the loading conditions created by the change from a steel structure to an engineered wood structure. The column footings increase in number and decrease in size under the adjusted loading conditions.

The specific design of the footings is completed using hand calculations in MathCAD. In the case of typical column footings, no lateral load is to be carried, and thus overturning and uplift are not considered on the foundation itself. The typical column footings are designed to satisfy ACI requirements of punching shear and one-way shear. Specific calculations may be seen in the appendix.

In the design of the footing supporting typical interior column, the size of the footing is decreased from 12.5 feet square by 30 inches deep to 10 feet square and 28 inches deep. This represents a 40% decrease in individual footing size. When the adjusted number of footings throughout the building is considered, there is approximately a 25% decrease in total footing volume.

Special Considerations – Deflection, Vibration, and Fireproofing

Vertical deflection is controlled as a summation of all individual member deflections. Wooden plank deflections are calculated to be minimal, as unsupported spans are limited to 2 feet. Floor truss deflections are controlled by manufacturer specifications and range from 0.446" to 0.47". Girder deflections measure 0.45" under live loads and 0.577" under total loads. As a result, total floor deflections measure approximately 1" or a deflection of L/360 under full loading. Commercial building deflection limits of L/600 consider only the first 50 psf of live load, thus permitting the deflections of the system as designed.

Vibrations in this type of engineered wood system have been studied in depth, however no specific design process or criteria is currently in place. The TJH and TJM joists used in the new design can reach spans up to 70 feet, and by limiting member size to 30 feet, less than half of this maximum value, the occurrence of vibrations is less likely. Girders in the system are limited to a span of 22.5 feet, also decreasing the potential for vibration issues as compared to the 30 foot span of the original steel system. In the case of vibrations greater than desired present in the structure, bracing may be added to the joists. This bracing is available from the same provider as the trusses and is shown below in Figure 26 as designed by Trus Joist and iLevel.

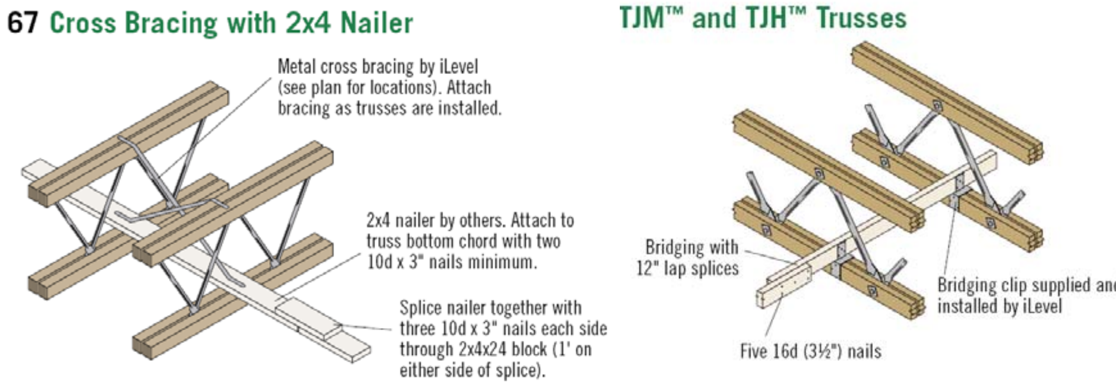

Figure 26: Examples of Approved Bracing for TJM and TJH Trusses

The requirements of the building range from one to two hour fire ratings. The original structure includes a complete sprinkler system on each floor. This is also applied to the redesigned wooden structure, giving the structure a higher hour fire rating. The engineered wood products alone have a fire rating satisfying 1 hour, but is raised to the minimum of 2 hours with the added sprinklers. APA, the Engineered Wood Association reports satisfactory wood dimensions for one hour fire ratings as shown in Figure 27.

TABLE 4

DIMENSIONS OF COMPONENTS FOR HEAVY TIMBER CONSTRUCTION (TYPICAL CODE PROVISIONS)

Heavy timber construction is generally defined in building codes and standards by the following minimum sizes for the various members or portions of a building:

	Inches, nominal
Columns	
Supporting floor loads	8x8
Supporting roof and ceiling loads only	6x8
Floor framing	
Beams and girders	6 wide x 10 deep
Arches and trusses	8 in any dimension
Roof framing – not supporting floor loads	
Arches springing from grade	6x8 lower half
	6x6 upper half
Arches, trusses, other framing springing from top of walls, etc.	4x6

	Inches, nominal
Floor covered with 1-inch nominal flooring, 15/32 or 1/2-inch plywood, or other approved surfacing	
Splined or tongue-and-groove plank	2
Planks set on edge	4
Roof decks	
Splined or tongue-and-groove plank	2
Plank set on edge	3
Tongue-and-groove plywood	1-1/8

Figure 27: Minimum Dimensions of Wooden Members for Fire Requirements