TEMPLE STEEPLE SEISMIC DESIGN

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BRIGHAM YOUNG UNIVERSITY

TO: The Church of Jesus Christ of Latter-day Saints, Brent Maxfield

FROM: CHART Engineering

SUBJECT: Temple Steeple Steel Design

DATE: April 9, 2012

CHART Engineering presents this report to The Church of Jesus Christ of Latter-day Saints. Findings for the design of a temple steeple, located in South America, are presented only as a recommendation. Research and tests were performed at Brigham Young University under the supervision of Dr. Richards, PE. The information contained within is intended only for those involved with this project and is to not be reproduced or copied.

The following is outlined in the report:

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EXECUTIVE SUMMARY

The Church of Jesus Christ of Latter-day Saints is in need of a steeple structure design for a temple in a high seismic activity zone. This steeple must meet performance and architectural requirements while optimizing cost.

Members of the Church of Jesus Christ of Latter-day Saints worship in temples regularly, and for some members, this may require traveling significant distances and require great personal sacrifices. For this reason, temples are in high demand and are being built around the world at an increasing rate for members in various countries and circumstances. Each temple is designed accordingly for the specific location, as each location has unique geographical characteristics. Differing methods of construction and design must be utilized to meet these characteristics. These areas often include the possibility of natural disasters such as hurricanes, floods or earthquakes.

This challenge has presented itself with the design of a new temple in an unannounced location. This location, as mentioned above, is a high seismic area. A structural framing system for the steeple must be designed to withstand significant seismic loading and meet the architectural and performance criteria that are standard for any temple. The structure must remain safe for the Maximum Considered Earthquake (MCE) and only suffer minimal damage under the design basic earthquake (2/3 MCE).

This task includes evaluating two different American Society of Civil Engineers (ASCE) approaches to seismic design and evaluating different steel framing systems. The steeple structure is one of the most important parts to the exterior of temples. Steeples must be structurally sound and aesthetically pleasing, as they are often the first recognized part on a temple when seen from a distance. For this reason the highest quality of materials, most advanced seismic design analysis, and highest standards must be considered and used.

SPONSOR MEETING SUMMARY

As the projected is located in South America, CHART was unable to do a site visit. However, CHART had the opportunity to meet with the project sponsor, Brent Maxfield, at the Church Office Building in Salt Lake City, Utah. Maxfield explained the history of the specific project, including the challenges in steeple design, the scope of the project and a brief introduction to seismic design.

Maxfield first discussed the history of temple steeple design in the Church. Over 100 temples have been constructed, each with unique steeple designs, and yet none of them have been completely perfect. Catastrophic failures have never occurred, but problems have existed due to excessive seismic loading. As an example, Maxfield described a recent event with the steeple on the Washington DC Temple, where an earthquake that was imperceptible to citizens shook some of the cladding loose. The cladding fell and damaged part of the temple wall as it came down. Although this damage was due to a small problem with the design, the failure resulted in expensive damage control and could have potentially been dangerous to people. The reason this project was proposed was to create a design that could eliminate similar problems in the future.

After discussing the history of steeple designs, the scope of the specified project was discussed. The specific temple being design for this project is located in South America. Although the specific temple name nor location was provided, architectural drawings detailing all needed dimensions and requirements were given. Maxfield explained that both ASCE 7-10 requirements should be studied, one for non-structural components and one for a two-stage approach, and the team was to decide which one fit the best with this project. Three different systems under the ASCE 7-10 approaches would also be studied in order to derive the best design possible.

In the decision making process for both the system and design approach parameters were given in order to guide the design based on architectural and structural constraints. As stated before, the temple is in a high seismic area, so certain design considerations must be taken into account. The cladding is granite, with a specified average thickness of 4 centimeters. The statue of Angel Moroni adorning the top of the steeple weighs approximately 700 pounds and must be supported by a central tube of a specified size. Local materials for the area should be utilized if possible, and include square tube shapes no larger than HSS4x4 inches, small channels no larger than 6 inches, and small angles no larger than L3x3x3/8 inches. The team was also instructed to take into account all natural loads with the exception of wind loads, which will be neglected in this case.

There are also significant architectural constraints. The architectural design of the temple is already set in place, requiring that any steel design fit within the allotted space. There are also long, rectangular windows that adorn each of the flat faces of the design. These windows are to be lit at night with a single pillar of light that begins at the base of the steeple and shines, uninterrupted, to the top. This requires that no steel cross the beam of light in the interior of the steeple, or over the windows at the edges. This puts significant restriction on possible designs.

The last part of the meeting was devoted to instruction on basic seismic design. The main focus was describing what seismic activity meant in terms of building design. Low seismic activity does not necessarily mean that earthquakes are of a low magnitude, but rather that earthquakes occur less frequently. In most seismic design cases, buildings are design for about one-eighth of the maximum seismic loads expected.

Many different parameters are needed when calculating seismic loading for any building. Many of these parameters are site specific as well as lower-building specific. Maxfield provided all of these variables, and they are presented in Table 1 and Table 2.

Table 1. Given seismic factors.

Importance Factor I_p	Site Class	Response Accelerations S_s	S_1	Spectral Response Coefficients S_{DS}	S_{D1}	Seismic Design Category
1.25	Sd	1.65	0.75	1.1	0.75	E

Table 2. Given seismic factors for the base structure.

Response Coefficient C_s	Response Modification Factor R	Period T
0.28	5	0.18

DESIGN PROCESS

CHOOSING A SYSTEM

The initial task that CHART Engineering was faced with was filling the gap produced by the lack of knowledge and experiences the group members had pertaining to seismic design. The meeting with the sponsor, Brent Maxfield, was the first step to closing that gap, but more work was required of the group members. Team members read and studied ASCE 7-10, specifically the two methods the project focused on. It was decided that the first step to be taken in the design of the temple steeple would be to choose systems to focus on designing.

Based on the standards outlined in ASCE 7-10, eight systems were determined to be a possibility based on the design constraints. The seismic area factor (E) and the tall height of the steeple (roughly 20 meters) were the two biggest constraints considered. The eight systems included eccentrically and concentrically braced building frame systems, two moment resisting frames, and dual system special moment frames. CHART Engineering researched these systems and their differences in applications were discovered. The type of connections used between the steel members differentiated the systems.

This research, as well as conferring with Dr. Paul Richards, the team faculty mentor, led CHART to the decision to focus on a concentrically braced frame. The concentric steel frame design employs diagonally braced steel connected with gusset plates. This design is particularly useful for resisting lateral earthquake loads of the type that are expected to be acting on the steeple. Concentrically braced steel frames are also equipped to handle infrequent but high seismic loads, the type of earthquake behavior that was outlined for the project location.

The team decided to choose a special concentrically braced frame over an ordinary concentrically braced frame because of the seismic properties afforded to the special frame. Choosing a special concentrically braced frame does require that extra provisions and requirements be met before proceeding, particularly in construction of the steel connections.

DESIGNING THE SYSTEM

The architectural design for the steeple posed some challenges for designing the steel framing. Most of the temple steeple structures that were studied were flat-faced, meaning they were square or rectangular in shape. These shapes allowed simple rectangular trusses to be utilized in design. The architectural plans for this project include octagonal shapes rather than the normal rectangular shape, producing angled sidewalls instead of simple flat-faced walls. The plans also restricted steel beams from extending across the front windows. The combination of these two restrictions made it difficult to design a steel structure that was able to support the surrounding cladding.

The first design proposed was a right-triangle truss design located at each of the four corners. The hypotenuse of the triangle would span across the sloped surface of the wall. This design seemed promising until it was drafted with the correct dimensions in AutoCAD. This original design is shown in Figure 1. The problem with this design is clear upon seeing the computer drawing; there is too much space between the diagonal steel beam and the corresponding cladding. There was no reasonable addition to the design that would fill the space, meaning the cladding would have to be cantilevered with little support.

The second adaptation of the design used steel columns that would emulate the exact shape of the walls surrounding the structure. This involved five columns in each corner, creating a trapezoidal shape, with trusses spanning the length of each shape. This design is shown in Figure 2 on the following page. The problem with this design is also evident; the amount of steel involved would be costly both in added material and labor. This was undesirable as a constraint of the steeple design was to minimize cost where possible.

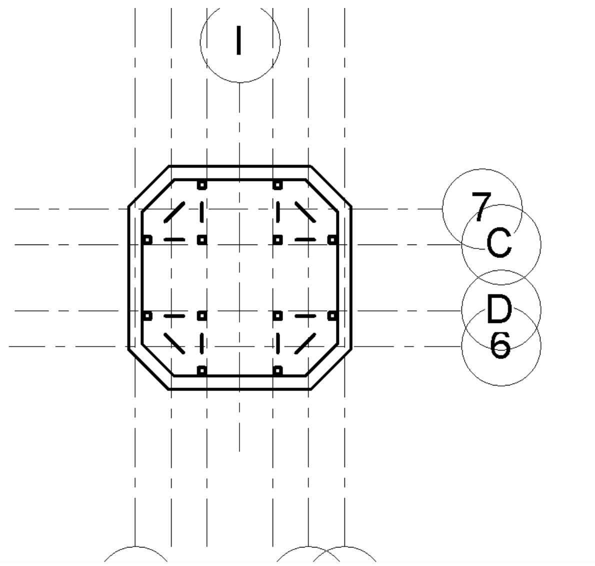


Figure 1. First proposed steeple design.

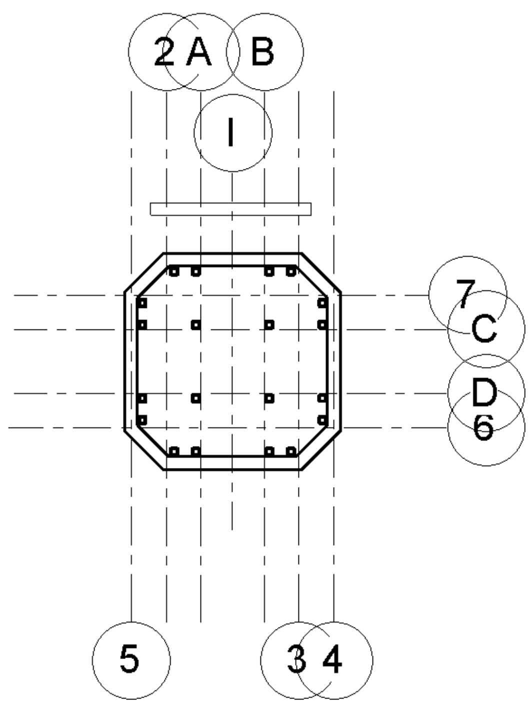


Figure 2. Second proposed steeple design.

These derivations led to the preliminary design that was used for the remainder of the project. This was a modification on the first design in that we kept a triangular shape. The triangle was rotated and revised so that the hypotenuse directly followed the sloped outside wall. The sidewalls would be supported, or "hung", off of steel that connected the truss shapes above the windows. This design allowed steel bracing, in some manner, to support every wall yet still minimized steel material. This initial design is shown in Figure 3 and in three-dimensional conceptual in Figure 4.

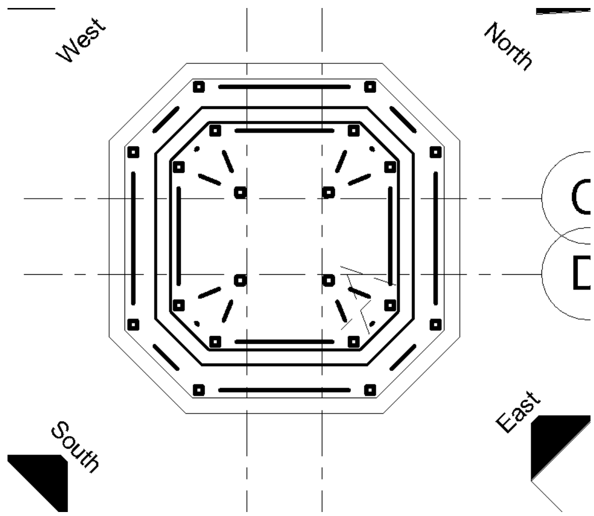


Figure 3. Steel design iteration chosen for project.

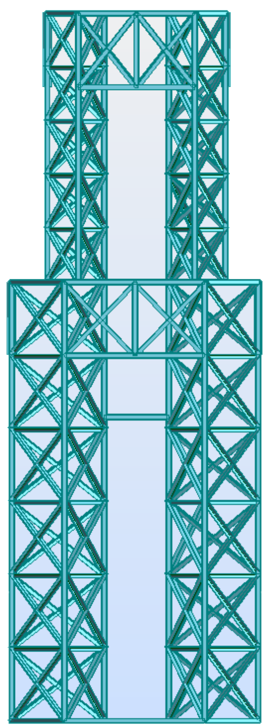


Figure 4. 3-D steel concept drawing.

The shape of the bracing was chosen to be x-shaped in order to come into compliance with demands based on the specially braced concentric frame. In order to keep some part of the frame in tension at all times, the bracing could not be the typical triangular shape.

BASIC CONNECTION DESIGN

The connection designs for special concentrically braced frames are complicated and require certain conditions to be met. These conditions include specifics for analysis, expected brace strength, lateral force distribution, diagonal braces, critical welding, beam-to-column connections, and required strength for tensile, compressive and buckling. The design of such a connection for our steeple is out of the scope of our current project. As such, CHART decided to design a simple connection in order to evaluate the ability of the steeple design to resist seismic loads.

The connection design will follow the basic design required by special concentrically braced frames in that it will be a welded gusset plate connection. The design dimensions and specifications are given below in Figure 5.

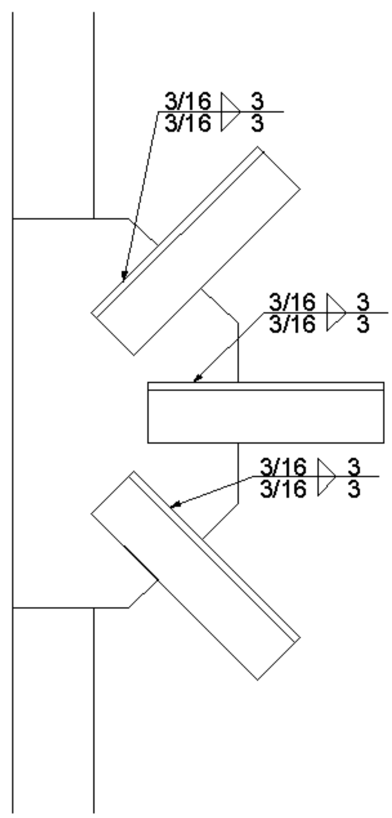


Figure 5. Weld design for the simple connection.

EVALUATING THE SYSTEM

ASCE 7-10 APPROACH

Two design approaches using ASCE 7-10 were evaluated: seismic design based on a two-stage or a component approach. A two-stage approach was the initial approach chosen by CHART to pursue because that approach would result in a smaller over all seismic load. The two-stage approach had considerable requirements to be met before being used. These requirements included:

- Stiffness of lower building must be at least 10 times stiffer than upper portion
- The period of the entire structure shall not be greater than 1.1 times the period of the upper portion
- Separate R and ρ values for the steeple and the base of the building
- The ratio of R/ρ of upper tower over R/ρ of the lower building portion should not be greater than one
- Should be analyzed with equivalent lateral force or modal response spectrum procedure

The period of the base structure was given as 0.18. Using equations to calculate the period found in ASCE 7-10, the tower was calculated to have a period of 0.13 (specific calculations can be found in the appendix). This period is small enough to meet the two-stage approach condition.

No specific information regarding the stiffness, or other parameters, was given about the base structure. Because the period condition was met, CHART made the basic assumption that the stiffness of the lower portion of the building would be at least 10 times stiffer than the upper portion. In line with the limited information that was given about the base structure, this seems to be an appropriate assumption.

Based on the seismic design category and the type of system chosen, the R-value used for the tower will be 6. Seismic provisions for the redundancy factor, ρ, state that if the seismic design category is D through F then the redundancy factor should be 1.3. The redundancy factor for the main building was not provided, but based on these provisions, ρ=1.3 is a reasonable estimate for the bottom structure as well. The R-value of the base structure was given to be 5, thus the ratio of R/ρ of the upper tower over R/ρ of the lower tower is less than one and meets the requirement.

As the four requirements were met for a two-stage analysis, the seismic forces were calculated based on this approach. Summaries of the values that allow the two-stage approach to be used are given in Table 3.

Table 3. Summary of two-stage analysis requirements

Period Requirement			R and P Requirement		R and P Ratio Requirement		
T, Base Structure	T, Steeple	1.1*T, Steeple	R, Steeple	ρ, Steeple	R/ρ, Base Structure	R/ρ, Steeple	Ratio
0.18	0.13	0.143	6	1.3	3.85	4.62	0.83

SEISMIC ANALYSIS

Seismic forces were calculated based on provisions given in ASCE 7-10 for the two-stage analysis approach. The main equation used to compute the seismic force expected is given in Equation 1. The components of this seismic force equation are given in Equations 2 and 3. A full summary of these calculations, including calculated values, are given in the appendix.

$$E_m = E_{mh} + E_v \qquad \textbf{Equation 1.}$$

$$E_{mh} = \Omega_o * Q_E \qquad \textbf{Equation 2.}$$

$$E_v = 0.2 * S_{DS} * D \qquad \textbf{Equation 3.}$$

The ultimate seismic force was calculated to be 34.9 kips. The force was applied as a point load in the two most critical locations on the tower: the very top of the tower on each differing face. As there are only two main faces, the faces that are 90 degrees and faces angled at 45 degrees, these were the two cases tested. Seismic load would be considered to be the most powerful if it was fully applied in one direction on one face. As it is very unlikely an earthquake load would ever behave in this manner, it accounts for the highest loading case scenario.

The initial evaluation in Visual Analysis under the original steeple design yielded very few failed members. Figure 6, on the subsequent page, shows the program analysis, where red is a failed member. The blue side of the scale represents little to no stress while bright green is significant stress right before failure. The scale for severity of yielding is also given. As shown, only the members taking the brunt of the seismic loading at the top and one member at the connection of the tower fully failed. This, of course, is showing the loading from only one face of the building. Thus, if it were to be applied to the other faces, the symmetric beams and bracing would similarly fail.

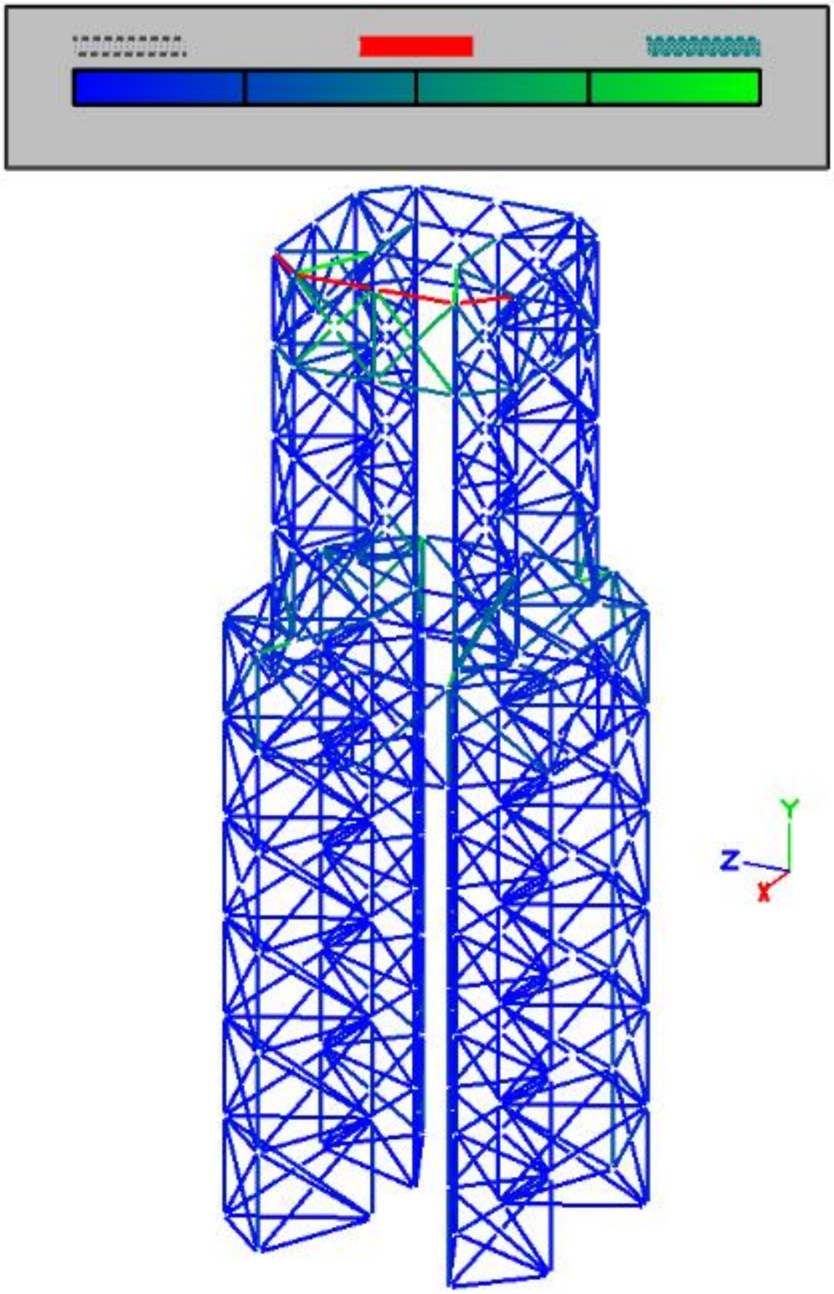


Figure 6. Initial seismic load application analysis.

Analysis on the 45-degree face yielded less stress on the members, making the 90-degree case the critical case to consider when resizing the steel members. The two critical locations for resizing included the very top of the structure and at the connection of the upper portion of the steeple to the lower, wider portion of the steeple.

CHART kept in mind the additional stipulation to optimize cost; so completely removing the local steel was not a favorable option. There were two simple options available: either change the steel sizing at those critical, failing locations, or add more steel at those locations to reinforce what was failing.

The second option was more favorable as it reduced both labor costs (less shapes to keep straight) as well as ordering costs, by allowing all steel used to still be local. Two simple cross bracing additions were made to the two critical locations (the top of the steeple and the connection between the lower and upper portions of the steeple). These additions are shown in a plan view in Figure 7. As seen in Figure 8, these simple additions completely removed the failed members and significantly increased the capacity of the others.

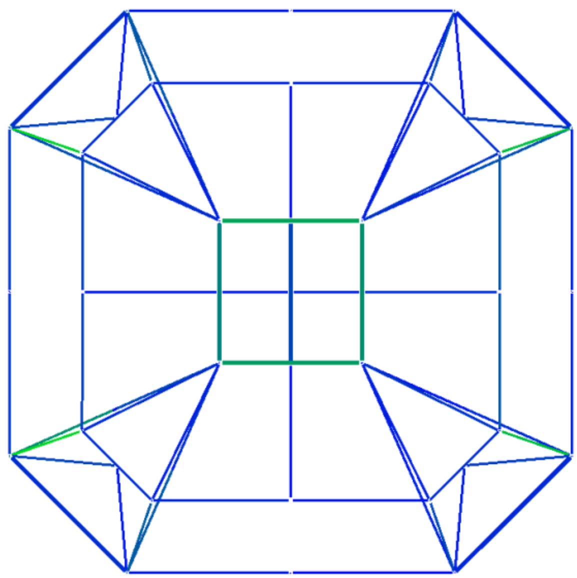


Figure 7. Top view showing additional bracing added.

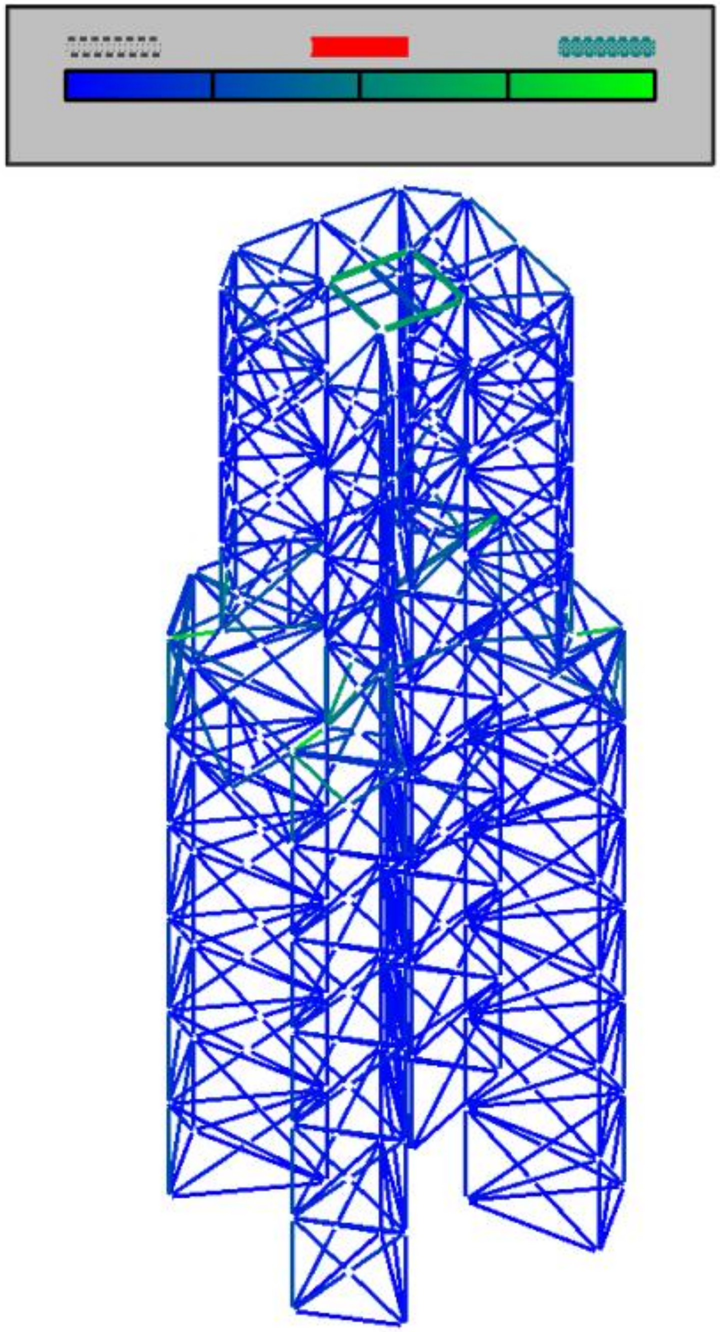


Figure 8. Iterative design under seismic loading.

FINAL DESIGN

The final design is presented below in Figure 9. The steeple was designed as a special concentrically braced frame with cross bracing. Two steel member sizes are used throughout the entire steeple structure. The main columns and bracing are designed as HSS4x4x5/16 and 2L3x3x3/16, respectively. The members hold the calculated seismic load adequately (as seen in Figure 8) and would not be expected to fail under these seismic conditions.

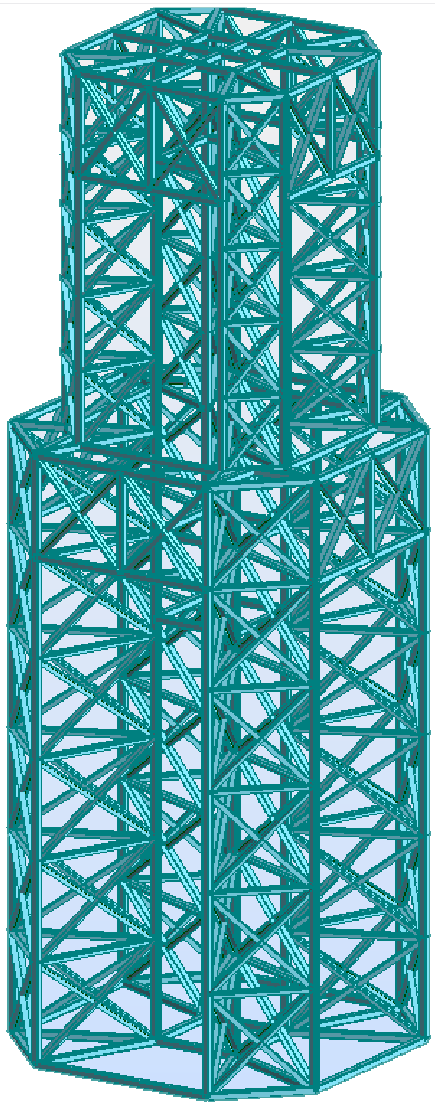


Figure 9. Final 3d steel steeple design.

COST ANALYSIS

A stipulation of the steeple project was to minimize cost. The most viable way for CHART to do this was through the following ways:

- Limit steel member usage
- Limit differing steel member sizes
- Use as much local material as possible
- Types of connection costs

As the main purpose of the steeple design was to keep the steeple workable under 2/3 of MCE, the first cost idea was not a priority to keep in mind. The amount of steel needed for the project to meet the requirements would be used, regardless of affect on cost.

The other options for limiting cost were all viable options that CHART kept in mind through the designing process. Initially, the steeple was designed with 100 percent local steel in order to keep cost down. As discussed in the analysis section of this report, some members failed under this steel sizing, however, more bracing was added, allowing the entire structure to remain as local steel.

Steel member sizes were also limited. The entire structure contains only two steel sizes, HSS4X4X5/16 and 2L3x3x3/16. This significantly cuts down ordering costs and labor costs, as there are no switches in member sizing throughout the entire steeple.

Finally, CHART looked into the types of connections for the steel members that would be most economical. Generally, in the United States, welding costs a significant amount more than bolting. Abroad, however, this is usually not the case. Pricing of welding and bolting abroad are usually comparable. Based on this information, the team decided to use welded connections for the entirety of the steeple.

Limiting the number of steel member sizes and using 95 percent local material significantly reduces labor costs and overall steel costs as a whole. In these ways CHART followed the cost constraint and produced a tower of reasonable price.

CONCLUSION

CHART Engineering presents this final steeple design to the Church of Jesus Christ of Latter-day Saints for review. Further iterations of the design may be required to meet stricter specifications or to reduce cost further.

APPENDIX

Dead Loads

Wall Areas

Bottom Front Face

$$b_{bf} := 2.54m$$

$$h_{bf} := 7.81m$$

$$A_{bf} := b_{bf} \cdot h_{bf} = 213.528 \cdot ft^2$$

Bottom Angled Face

$$b_{ba} := \left(.956^2 m \cdot .956^2 m\right)^{\left(\frac{1}{2}\right)} = 2.998 \cdot ft$$

$$h_{ba} := 7.81m$$

$$A_{ba} := b_{ba} \cdot h_{ba} = 76.831 \cdot ft^2$$

Top Front Face

$$b_{tf} := 2.088m$$

$$h_{tf} := 4.83m$$

$$A_{tf} := b_{tf} \cdot h_{tf} = 108.554 \cdot ft^2$$

Top Angle

$$b_{ta} := 1.352m$$

$$h_{ta} := 4.83m$$

$$A_{ta} := b_{ta} \cdot h_{ta} = 70.29 \cdot ft^2$$

Dome

$$A_d := \frac{\left[4\pi \cdot (1.44m)^2\right]}{2} = 140.241 \cdot ft^2$$

$$W_{granite} := 180\,\frac{lb}{ft^3}$$

Tributary Areas for Wall loads

$$w_c := 24\,\frac{lb}{ft^2}$$

$$w_s := 8\,\frac{lb}{ft^2}$$

$$w_{total} := w_c + w_s = 32 \cdot \frac{lb}{ft^2}$$

$$L_{bottomfront} := (w_{total}) \cdot (A_{bf}) = 6.833 \times 10^3 \cdot lb$$

$$Node_{bf} := \frac{L_{bottomfront}}{6} = 1.139 \times 10^3 \cdot lb$$

$$L_{bottomangle} := w_{total} \cdot A_{ba} = 2.459 \times 10^3 \cdot lb$$

$$Node_{ba} := \frac{L_{bottomangle}}{12} = 204.883 \cdot lb$$

$$L_{topfront} := (w_{total} \cdot A_{tf}) = 3.474 \times 10^3 \cdot lb$$

$$Node_{tf} := \frac{L_{topfront}}{6} = 578.957 \cdot lb$$

$$L_{topangle} := w_{total} \cdot A_{ta} = 2.249 \times 10^3 \cdot lb$$

$$Node_{ta} := \frac{L_{topangle}}{8} = 281.16 \cdot lb$$

$$L_{moroni} := \frac{700lb}{4} = 175 \cdot lb$$

$$L_{dome} := \frac{(A_d \cdot T_{granite} \cdot W_{granite})}{4} = 1.035 \times 10^3 \cdot lb$$

$$L_{top} := L_{moroni} + L_{dome} = 1.21 \times 10^3 \cdot lb$$

Seismic Loading

a_p

S.DS is the spectral response coefficent

W.p is the compenent operating weight (this includes the weight of the steel i assume, can we get that from either program?)

R.p

I.p is the seismic importance factor

z is the height in structure of point of attachement of component with respect to the base (This means the elevation of the roof, where the steeple attaches)

h is the average roof height of structure with respect to base
(I understand this to mean height of our steeple off the top
of the temple, temple roof = height of 0, not including
moroni or the dome)

F.p is the force applied independently in at lease two orthogonal horizontal directions.

C.s is the seismic response coefficient

R is the response modification factor of building

I.e

Dead Load

$W_{steel} := 44.67kip$

$W_{cladding} := 6.49kip$

$W_p := W_{steel} + W_{cladding} = 51.16 \cdot kip$

Seismic Variabls

$a_p := 2.5$

$S_{DS} := 1.1$

$W_p = 51.16 \cdot kip$

$R_p := 2.5$

$I_p := 1.25$

$I_e := 1.25$

$z := 112.3m$

$h := 12.77m + 112.3m = 410.335 \cdot ft$

$x := .75$

$C_t := .02$

$h_n := 12.70$

$\Omega_o := 2$

Period

$T_a := C_t \cdot h_n{}^x = 0.135$

Seismic Point Load

$$F_p := \frac{\left(.4a_p \cdot S_{DS} \cdot W_p\right)}{\frac{R_p}{I_p}} \cdot \left[1 + 2 \cdot \left(\frac{z}{h}\right)\right] = 78.668 \cdot kip$$

$R := 6$

$$C_s := \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} = 0.229$$

$V_s := C_s \cdot W_p = 11.724 \cdot kip$

$Q_E := V_s$

$E_{mh} := \Omega_o \cdot Q_E = 23.448 \cdot kip$

$E_v := .2 \cdot S_{DS} \cdot W = 0.22\,W$

Welded connections

plate is 1.2" thick $t := \frac{3}{8}in$

$D := 3$

Demands

$P_D := 1.139kip$

$P_L := 0$

$P_u := 1.2P_D + 1.6 \cdot P_L = 1.367 \cdot kip$

Pick a weld size

determine minimum weld size

from table J2.4 - 1/8"

determine maximum weld size

$t - \frac{1in}{16} = \frac{5}{16} \cdot in$

try a 3/16" weld size

Determine the required length

$\phi R_n > P_u$

Equation 8-2a

$1.392 \cdot D \cdot L > 1.3668$

$L > \frac{1.3668}{1.392 \cdot D}$

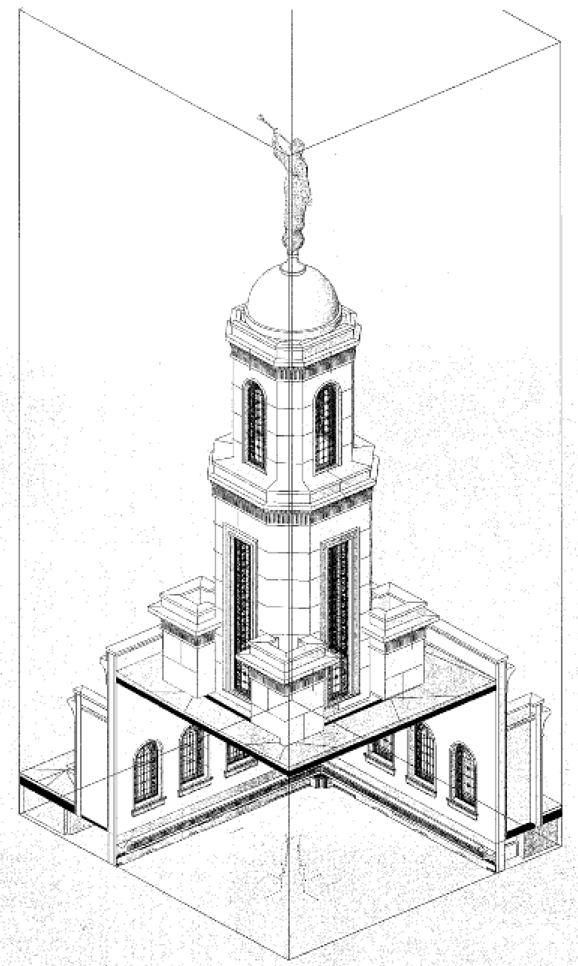
$\frac{1.3668}{1.392 \cdot D} = 0.327$ This is really low so we are going to try 3in

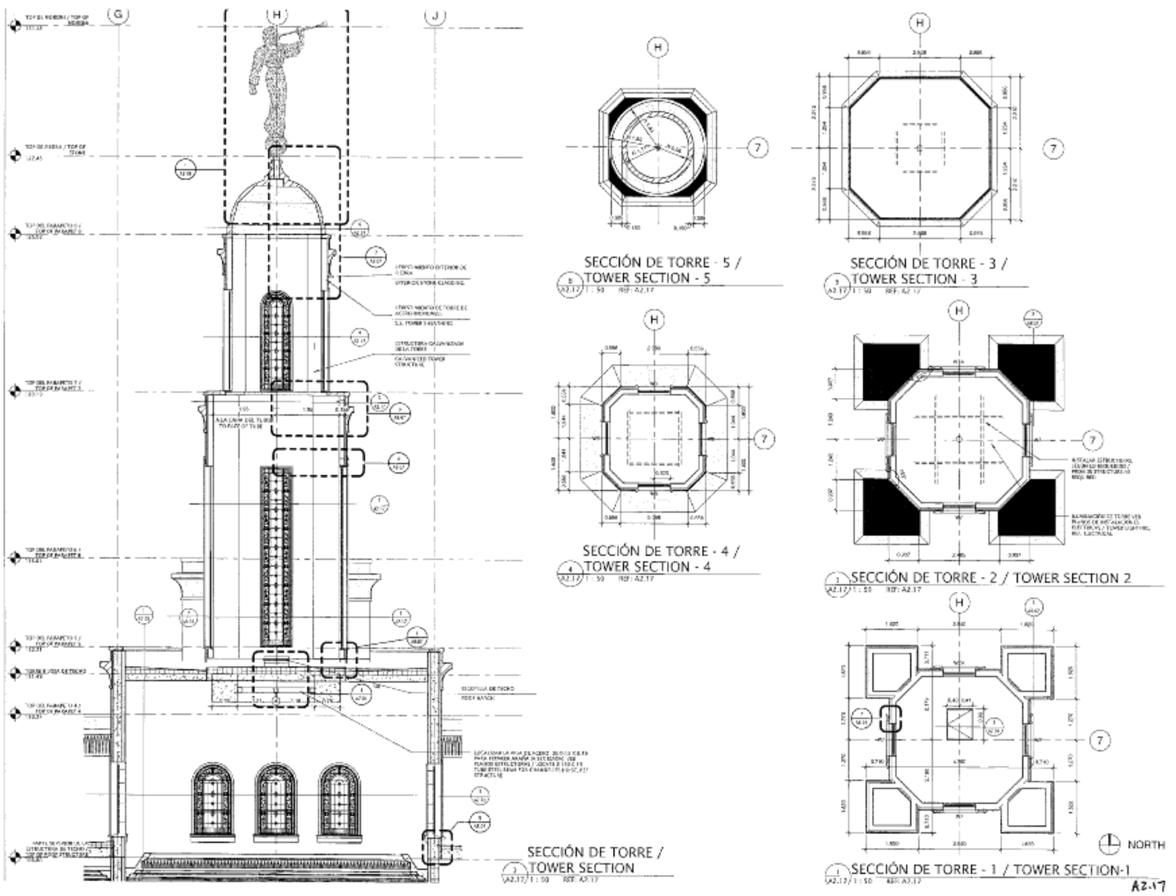
Check Capacity

$\phi R_n := .9 \cdot 36ksi \cdot \left(3 \cdot \frac{3}{8}\right) = 36.45 \cdot ksi$

min: 1/8" to 2/16"

max: 3/8" to 5/16"




SECCIÓN DE TORRE - 5 /
TOWER SECTION - 5
SECCIÓN DE TORRE - 3 /
TOWER SECTION - 3
SECCIÓN DE TORRE - 4 /
TOWER SECTION - 4
SECCIÓN DE TORRE - 2 / TOWER SECTION 2
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TOWER SECTION
SECCIÓN DE TORRE - 1 / TOWER SECTION-1
NORTH
A2.17