ngCFHT Telescope and Enclosure Configuration
and Outer Pier Capacity Study

Dynamic Structures Ltd. &
University of British Columbia

January 2012
Revision History

<table>
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<th>Version</th>
<th>Date</th>
<th>Comments</th>
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<tr>
<td>1</td>
<td>Dec 22, 2011</td>
<td>Initial release</td>
</tr>
<tr>
<td>2</td>
<td>Jan 20, 2012</td>
<td>Information on ngCFHT telescope concept</td>
</tr>
<tr>
<td>3</td>
<td>Jan 30, 2012</td>
<td>Revisions based on review meeting</td>
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Introduction

This report summarizes the results of a study on the Next Generation Canada-France-Hawaii Telescope (ngCFHT). The work was conducted from October to December 2011. This is the third phase of development work carried out by DSL and UBC on the ngCFHT project. The first was the telescope pier capacity study done by UBC. The second was DSL’s work on defining the telescope mass properties and modeling of the azimuth track interface for the first study.

There are two primary objectives of the present work phase:

- The first objective is to develop an ngCFHT system with the telescope and calotte enclosure configuration meeting the dimensional constraints of the current CFHT inner and outer piers and site considerations. The goal is to develop a new telescope and enclosure system with the calotte enclosure matching the three dimensional volume of the current CFHT enclosure while accommodating a new and fully functional 10 m class segmented mirror telescope within the interior space. The telescope and enclosure configuration should include realistic space allocations for the telescope azimuth journal and its structural interface with the inner pier, enclosure wall thickness with realistic structural member dimensions, and enclosure bogies and their structural interfaces with the outer pier. Additional outcomes are the mass, centre of gravity (CG), mass moment of inertia estimates of the telescope and enclosure. The new ngCFHT system should have sufficient details to allow for a ROM cost estimate.

- The second objective of this study is to determine the load capacity, statically and dynamically, of the current enclosure foundation and pier based on the latest structural design codes and including all environmental loads. The outcome is to determine if the current CFHT outer pier has sufficient load capacity for the calotte enclosure configuration.

Statement of Work: Based on the site and as-built information, and design and functional parameters:

1. Develop a conceptual ngCFHT telescope and enclosure system compatible to the current CFHT geometry
2. Determine the load carrying capacity of the current enclosure foundation and pier
3. Provide recommendations to retrofit the enclosure pier should its capacity deemed to insufficient
4. Provide assessments of the suitability of the current telescope foundation/pier and enclosure foundation/pier for future upgrade and potential dynamic interactions
1. Concept Design
The goal of the concept design is to develop a new enclosure and telescope system. This should match the space envelope of the current observatory as closely as possible, including realistic space allocations for structural and mechanical subsystems.

1.1 Requirements
The following summarizes the requirements on the concept design of the enclosure and telescope system.

- **General**
  - The goal of ngCFHT is to preserve existing aspects of the CFHT observatory as much as possible. Preserving the overall observatory space envelope will facilitate the permitting process. Reusing the inner and outer piers is beneficial for cost and schedule considerations.

- **Optical configuration:**
  - The primary mirror is an f/2 segmented 10 m diameter mirror
  - The distance from primary mirror to image plane is 17.9m (see Figure 1). The space envelope for the fibre position system and telescope guide camera is an additional 1.5m for a total distance of 19.4m from the primary mirror vertex to the top of the telescope
  - The enclosure will provide a clear zenith angle observing range from 0 degrees to 65 degrees. The telescope will be able to point to 90 degrees to facilitate maintenance of the top end components.

- **Handling requirements:** A dome-mounted crane (or cranes) will handle the M1 segments and the prime focus components. The top end components can be modular such that they stack together. A piece size of 1 m dia. X 0.5 m thick and 500 kg is assumed.
1.2 Enclosure Concept

A calotte configuration for the enclosure was selected due to its compact design and structural efficiency. These features give it the best possibility of matching the existing enclosure size and mass, which would allow the existing enclosure pier to be utilized. Figure 2 below shows a CAD model of the existing CFHT enclosure and telescope. Figure 3 shows the proposed ngCFHT calotte enclosure.
Figure 2: Existing CFHT Enclosure and Telescope

Figure 3: Proposed ngCFHT calotte enclosure
1.2.1 Geometry

The dimensions of the proposed ngCFHT enclosure are shown in Figure 4 below. A telescope sweep radius of 14.0m is assumed. The inside radius of the dome is set at 16.0m (inside of the dome insulation) and the outside radius is set at 17.0m. A goal of the design is to reuse the existing rotating ring girder and azimuth bogie system if possible. Since the sphere of the dome must therefore intersect the existing ring girder, this determines the approximate height of the elevation axis, which is presently set to 24.0m above grade. There is some opportunity to reduce this if necessary by using a variable radius on the dome so that the elevation axis can be lowered, and the dome sphere will still intersect azimuth ring girder. Figure 5 shows a comparison of the proposed ngCFHT enclosure dimensions to the existing CFHT enclosure dimensions.
Figure 5: Comparison of ngCFHT enclosure dimensions (left) to existing CFHT enclosure dimensions (right)
1.2.2 Shutter

The shutter concept for the ngCFHT is shown in Figure 6 below. The geometry of the enclosure permits a fixed shutter concept to be used, where the shutter (shown in blue in Figure 6) is a cantilevered extension of the base structure. The aperture is closed by rotating the aperture over top of the shutter. This approach eliminates the need for additional major mechanisms to open and close the shutter.

In the closed position, a locking mechanism is likely required to secure the aperture to the shutter at points around its perimeter. A sealing mechanism would also be required to seal the interface. An inflatable seal is a possible solution here and has been proposed and prototyped for TMT.

Figure 6: Shutter concept (cap is removed to show shutter, aperture indicated by dotted lines)
Since the shutter is located on the inside of the dome when open, the shutter would need to be clear of large accumulations of snow and ice prior to opening. The center of the shutter is sloped at 45° relative to horizontal, so much of the snow should shed naturally. Ice and snow accumulation could be mitigated by pointing the shutter downwind during storms. Ice and snow shedding could be enhanced by adding a heating system if necessary. Once the enclosure is opened, any run-off from residual snow or ice on the shutter could be controlled by a gutter system around the shutter perimeter.

Access to the perimeter of the shutter is likely required to inspect the locking mechanisms and seals. If possible, it is desirable that the access would all be located inside the dome such that the exterior of the dome is kept free of walkways, ladders, and other protrusions that would accumulate snow and ice.

1.2.3 Structure

The basic structural elements include ring girders at the circular mechanical interfaces, and a rib structure for the spherical portions of the structure. Cladding is provided by a welded steel plate skin. The basic structural approach is similar to that used on the existing CFHT dome, and other domes such as Keck and Gemini.

1.2.4 Mechanisms

The major mechanisms are the azimuth bogie and drive system, and the cap bogie and drive system. It is intended that the existing azimuth bogie and drive system can be reused, but this will need to be verified in further design studies. Designs for the cap bogie and drive system have been well developed for TMT. The cap bogies are distributed around the perimeter of the inclined cap plane. The cap drives consist of a rack and roller pinion drive system located at the lowest point of the inclined cap plane. The drive system is similar to that used on the existing CFHT shutter.

1.2.5 Environmental Control

1.2.5.1 Insulation

It is assumed the dome will be insulated. Sandwich panels with aluminum skins and expanded polystyrene core are commercially available with thicknesses from 4” to 10”. These provide a cost effective solution, and can provide good insulation to a spherical dome if proper connection details are specified.

1.2.5.2 Vents

No ventilation specifications have been developed at this point. Options exist to ventilate the interstitial space of the enclosure (as on the existing CFHT dome) and also to passively ventilate the entire dome interior via large ventilation openings (as on Gemini). For the TMT calotte design, commercially available roll-up doors with high wind load ratings were proposed for the ventilation doors. A secondary set of insulated doors provides the thermal break when the doors are closed. This approach could be used on the ngCFHT calotte also. For practicality reasons, it is preferred to only locate the ventilation openings on the base portion of the enclosure (below the inclined bearing).

1.2.5.3 Flaps

Aperture flaps are wind-deflecting features around the perimeter of the aperture, which are used to reduce wind speeds around the telescope top end during observing. Aperture flaps were deemed to be necessary for the TMT calotte to reduce wind speeds. On a site such as Mauna Kea that is subject to snow and ice storms, the flaps must be retractable such that they do not accumulate an excessive amount of snow and ice during storms. This requires additional mechanical and access complexity. For the ngCFHT dome it is assumed that aperture flaps will not be required.
1.2.6 Handling

A dome-mounted handling crane may be used to handle the primary mirror segments and also components at the telescope top-end. A crane similar to the Keck segment handling crane is proposed. The handling of the primary mirror segments generally requires large crane reach and low capacity, whereas the handling of the top end components may require higher capacity but shorter reach. Thus it may be beneficial to provide two separate cranes to serve these two basic functions.
1.3 Enclosure Loads

Assumed environmental survival loads are tabulated below.

Table 1: Environmental loads for survivability requirements

<table>
<thead>
<tr>
<th>Description</th>
<th>Value</th>
<th>Notes/assumptions</th>
</tr>
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<tbody>
<tr>
<td>Wind – shutter closed</td>
<td>78 m/s</td>
<td>3sec gust @ 10m, 50 year return</td>
</tr>
<tr>
<td>Wind – shutter open</td>
<td>35 m/s</td>
<td></td>
</tr>
<tr>
<td>Snow</td>
<td>150 kg/m²</td>
<td></td>
</tr>
<tr>
<td>Ice</td>
<td>68 kg/m²</td>
<td></td>
</tr>
<tr>
<td>Seismic – lateral spectral acceleration</td>
<td>0.5g</td>
<td>See section 2.2.2.6 below for details</td>
</tr>
<tr>
<td>Temperature</td>
<td>-16°C / +30 ºC</td>
<td></td>
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</table>

1.4 Telescope Concept

The telescope structure and mechanical concept can largely be derived from the Keck telescopes, which are also 10m diameter segmented alt-az telescopes. The primary baseline for the optical design for the telescope has been outlined above and as these are nearly identical to the twin Kecks the structure and mechanical systems will be of similar capacity.

There are a few major differences between the Keck telescopes and the ngCFHT proposal due to the nature of the instrumentation and data focal plane proposed. These differences significantly reduce the side and rear space requirements for the support structure on the telescope azimuth however as they are not driving factors in the telescope envelope no savings is realized in the telescope rotational sweep. The telescope sweep envelope is governed by the elevation sweep of the telescope optical assembly itself. This sweep is identified in Figure 4 above. The space savings between the Keck design and the ngCFHT arise from the removal of the two Nasmyth Platforms and the rear Cassegrain Platform which are not required for the proposed optical design. The removal of these three structures will allow a significant reduction in the rotating mass of the Azimuth structure and related reduction in the size of the drives required.

Mechanically the telescope structure has two rotational axis; the Elevation axis which is located at what would be the Nasmyth level and the Azimuth axis which is at the bottom of the telescope support structure. The rotational elements of the Elevation axis are a pair of conventional bearings that, although custom ordered, are available from several suppliers. The Azimuth Bearing is a custom designed oil bearing. The sizing of this bearing could be slightly smaller than the current Keck design due to the savings in mass noted above. The two drive systems are designed to be friction systems and these are stable designs with identified deficiencies. A future option could be to look at alternate drive methods such as integral direct drive or Linear Induction Motors. Advances in encoder technology since the production of the Keck telescopes have allowed for higher resolution for positioning and control.

The last area of consideration is the Secondary cage and its related mechanical systems. These are largely a fall-out of the operational requirements of the instrumentation and must be designed in concert with those constraints. At the very minimum, methods for focusing and instrumentation need to be included.
2. Existing Enclosure Pier Evaluation

The goal of the existing enclosure pier evaluation is to provide an evaluation of the possible reusing of the current enclosure pier and foundation for future upgrade and potential dynamic interactions. This section presents in details the procedure used to perform the assessment of the outer pier as well as the analysis results and conclusions.

The structure is first described in detail. Following that, the methodology and modeling assumptions are presented. Finally, the analysis results and design checks that were performed to assess the structure are reviewed.

2.1 Structure description

This section of the report presents a detailed review of the structure and foundation components. It characterizes the geometry and structural configuration of the outer steel frame pier, footings and dome idealization. The material properties and soil information of the site are also described.

2.1.1.1 Geometry and structural configuration

The outer pier covers the inner concrete pier building supporting the telescope as shown in Figure 7. The outer pier building is a 5 storey steel structure of an overall height of 14.9 m and of an outer diameter of 28.8 m. The 1st level is referred as the ground level and the 5th level as the observation level. The inner diameter is of 16.8 m and there is a spacing of 76 mm between the inner concrete pier and the outer steel pier.

Figure 7: Outer pier framing during its construction

The structure is divided in 12 bays along its perimeter. Each column is numbered from 1 to 12. For each level and each bay, the typical bay framing is drawn. Figure 8 to Figure 12 show the plan view of the different levels. The main opening of the outer pier is located at the ground level between columns 1 and 12.
The x-axis is perpendicular to the opening and the y-axis parallel to the opening. The z-axis is in the vertical direction.

The third level has horizontal bracings joining the tangential beams together. The observation floor has a framing at its center. The beams in the x-direction on the center of the observation level are composite beams, this means that they act in a composite way with the concrete slab on top of it.

Concrete slabs cover the floor framings, the ground, second, third and fourth slabs are 64 mm thick and the observation level slab is 250 mm thick. The slab is continuous around the perimeter of the ground floor and no slab is present between columns 1 and 12 for the higher levels.

Figure 8: Ground level plan
Figure 9: Second level plan
Figure 10: Third level plan

Figure 11: Fourth level plan
Figure 12: Fifth level plan with wide welded flange section

The height from the footing top to the ground level is 0.77 m and the interstorey height for the second and third levels is 3.91 m and 4.01 m, respectively. For both the fourth and observation levels, the interstorey height is 3.09 m. The fifth level is the observation level and a balcony is located on its slab. The balcony is not shown on the drawings. The dome is placed on top of the observation level on the external ring girder.

The vertical load on the observatory level is distributed to the levels below using both inner and outer columns. The load is then carried by the diagonal columns and the outer columns to the foundation. The tension hangers support the ground and second levels and are attached to the beams of the third level. Figure 13 presents the outer perimeter of the pier structure, and Figure 14 displays the inner perimeter of the pier structure.

The bracings are chevron types and are the elements resisting the lateral loads applied to the structure. The bays are braced at each bay for the top level and every two bays for the lower ones.
Figure 13: Outer perimeter of the outer pier structure

Figure 14: Inner perimeter of the outer pier structure

Diagonal columns are run continuously from the pinned support at the foundation to the third level beams as shown in the next figure.

Figure 15: Typical radial view

The following figure presents the three-dimensional view of the structure.
2.1.1.2 Footing

The foundation is a ring footing of 4.267 m width and 1.828 m height. Longitudinal reinforcement bars are constituted of equally distributed 20 #8 bars on top and bottom faces. Radial reinforcement on the bottom is formed of #8 bars spaced at 203 mm. Two sets of #4 vertical shear reinforcement stirrups are provided. The stirrups are spaced at 203 mm at column locations and at 406 mm between the columns. Figure 17 below shows a cross section of the ring footing.
2.1.1.3 Dome

To evaluate the outer pier capacity, it is necessary to consider the forces produced by the dome on top of the structure. The dome is a complex structure and requires a considerable amount of time to be modeled in detail. Therefore, it is simplified as a pyramidal frame with the mass of the dome concentrated at its top. The frame is connected to the pier top on the external ring girder through spring elements. The spring elements have translational rigidity only; $K_z$ is the vertical stiffness, $K_r$ is the radial stiffness and $K_t$ is the tangential stiffness. The pyramidal frame members are considered as rigid elements. The behavior and stiffness of the frame is controlled by the spring elements. To determine the stiffness of the spring, it is assumed that deflections due to the telescope dead load should be between 10 and 20 mm. Also the $K_z$ and $K_r$ are assumed to have the same stiffness and the tangential stiffness $K_t$ to be 10% of the two other stiffnesses. The pyramidal frame nodes are interconnected with the rigid elements in the pattern presented in Figure 18. The pyramidal frame is connected at each exterior column top. This idealization of the dome is judged to be sufficient to represent the global behavior of the dome and to transmit the forces to the outer pier properly. It should be noted that the dome forces on the real structure will be transferred to the ring girder as point loads at the locations of the azimuth bogies.
Figure 18: Dome configuration

The table below presents the spring stiffness in different directions that were assigned to the finite element model.

**Table 2: Dome modeling parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<tr>
<td>Vertical stiffness - $K_z$ (kN/mm)</td>
<td>27.777</td>
</tr>
<tr>
<td>Radial stiffness - $K_r$ (kN/mm)</td>
<td>27.777</td>
</tr>
<tr>
<td>Tangential stiffness - $K_t$ (kN/mm)</td>
<td>2.777</td>
</tr>
<tr>
<td>Center of gravity height over observation level (m)</td>
<td>11.929</td>
</tr>
<tr>
<td>Number of spring supports</td>
<td>12</td>
</tr>
</tbody>
</table>

**2.1.2 Structural Weights**

Table 3 below summarizes the weight of the structure with the existing and future dome systems. Replacing the existing dome with the new dome will increase the total weight of the structure by approximately 10%.

**Table 3: Weight of the structure with the existing and the new telescope**

<table>
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<tr>
<th>Weight Type</th>
<th>Existing structure</th>
<th>New dome</th>
</tr>
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<tr>
<td>Dome weight (kN)</td>
<td>3783</td>
<td>5000</td>
</tr>
<tr>
<td>Outer pier weight without dome (kN)</td>
<td>8757</td>
<td>8825</td>
</tr>
<tr>
<td>Total weight (kN)</td>
<td>12539</td>
<td>13825</td>
</tr>
</tbody>
</table>

**2.1.3 Materials**

Steel sections material properties are defined according to CSA G40.12 - 1971 with a yield stress of 304 MPa, an ultimate stress of 448 MPa and a modulus of elasticity of 200 000 MPa. The reinforcing bars have a yield strength of 413 MPa, and a modulus of elasticity is 200,000 Mpa. Concrete compressive strength is 20.7 MPa and the elasticity modulus is evaluated to be 21,525 MPa.
2.1.4 Soil
Soil data is taken from the Foundation Investigation Report prepared by Dames & Moore, (1973). The density of such soil varies from 700 kg/m$^3$ to 2300 kg/m$^3$, (Dames & Moore, 1973). In the calculation, an average of 1500 kg/m$^3$ is assumed for simplicity.

The Structural Design Brief for the Peripheral Building (SNC, 1974) for the outer pier uses a bearing capacity of 161 kPa, this value is lower and is used for the analysis.

2.2 Methodology and assumptions
A methodology was defined to evaluate the capacity of the structure. The following steps were pursued to successfully complete the assessment.

1. Building codes and design requirements to be used.
2. Load cases and load combinations definition.
3. Creation of a finite element model (FEM) of the structure.
4. Static and dynamic analysis of the structure.
5. Capacity checks
   a. Beams
   b. Columns
   c. Bracings
   d. Footing
   e. Foundation
   f. Deflections


2.2.1 Building codes and design philosophy
The building was designed in 1974 conforming to the Uniform Building Code (1973). In the current study, the loads are defined by the American Society of Civil Engineers 7 - Minimum Design Loads of Buildings and Other Structures (ASCE 7) 2010. The design checks for the steel structure are based on the American Institute of Steel Construction (AISC) 2005 and the concrete capacity based on the Canadian concrete code CAN/CSA-A23.3-04.

Two design philosophies are used for design. The structural and footing designs are done using limit state design (LSD) and the soil foundation is evaluated using allowable stress design (ASD). For more details please refer to ASCE-7 (2010).

2.2.2 Load cases
This section of the report presents the load cases and combinations applied to the outer pier structure. The definition of load cases and combinations are provided in details in the ASCE-7 2010. The loads acting on the outer pier include dead, live, ice, snow, wind and earthquake loads. It should be noted that the center of gravity of the dome is located at a 1 m horizontal distance from the center of dome sphere due to the opening in the dome configurations. As a result, all the dome loads are applied as combination of point load and point moments on the center of sphere, which is located at a height of 11.929 m from the observation level.

2.2.2.1 Dead load
Dead load is the structural mass of the outer pier, which includes the total weight of the construction materials. Steel beam mass is intrinsic to each beam element. These loads are obtained from the Structural Design Brief for the Peripheral Building (SNC, 1974) as a pressure load on each floor. They are applied on...
the beam in the model as vertical uniformly distributed forces. Table 4 presents the dead load applied on each floor.

**Table 4: Dead loads per floor**

<table>
<thead>
<tr>
<th>Floor</th>
<th>Pressure (kPa)</th>
<th>Weight (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground</td>
<td>2.873</td>
<td>1182.96</td>
</tr>
<tr>
<td>2</td>
<td>2.873</td>
<td>1182.96</td>
</tr>
<tr>
<td>3</td>
<td>2.873</td>
<td>1182.96</td>
</tr>
<tr>
<td>4</td>
<td>2.873</td>
<td>1182.96</td>
</tr>
<tr>
<td>Observatory- Peripheral</td>
<td>6.281</td>
<td>2586.35</td>
</tr>
<tr>
<td>Observatory- Center</td>
<td>6.281</td>
<td>1506.42</td>
</tr>
<tr>
<td>Outer pier total</td>
<td></td>
<td>8824.64</td>
</tr>
</tbody>
</table>

The current dome situated on the outer pier is wished to be replaced to accommodate the new telescope. The current dome weighs 3783 kN and it is approximated that the new dome will have a weight of approximately 5000 kN due to its overall bigger size. The dome dead load is applied as a point load and a point moment on the dome center.

**Table 5: Dead load for the dome**

<table>
<thead>
<tr>
<th>Vertical reaction (kN)</th>
<th>Horizontal eccentricity (m)</th>
<th>Moment w.r.t. dome center (kN.m)</th>
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<tbody>
<tr>
<td>-5000</td>
<td>1.0</td>
<td>5000</td>
</tr>
</tbody>
</table>

The total dead weight of the structure including the dome is 13,825 kN or 1409 tonne.

### 2.2.2 Live load

Live load is the load due to the use and occupancy of the building. Similar to the previous section, the live loads are obtained from the original Structural Design Brief for the Peripheral Building (SNC, 1974). It is assumed that live loads have not changed significantly over the years, and they are applied as vertical uniformly distributed loads on each beam. Table 6 summarizes the live loads per floor. Note that based on building code requirements a portion of the live load may or may not need to be included in the seismic weight (to model inertial effects of the live load), depending on the nature of the occupancy. The live load requirements should be defined for final design.

**Table 6: Live load per floor**

<table>
<thead>
<tr>
<th>Floor</th>
<th>Pressure (kPa)</th>
<th>Weight (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground</td>
<td>7.182</td>
<td>2957.42</td>
</tr>
<tr>
<td>2</td>
<td>4.789</td>
<td>1971.614</td>
</tr>
<tr>
<td>3</td>
<td>4.789</td>
<td>1971.614</td>
</tr>
<tr>
<td>4</td>
<td>4.789</td>
<td>1971.614</td>
</tr>
<tr>
<td>Observatory – Peripheral</td>
<td>9.576</td>
<td>3943.23</td>
</tr>
<tr>
<td>Observatory - Center</td>
<td>4.788</td>
<td>1148.36</td>
</tr>
</tbody>
</table>
2.2.2.3 Ice load

Since the structure is located on Mauna Kea where freezing ice storms occur, it is necessary to take into account ice loads. It is approximated that the pressure applied by ice load is 68 kg/m². For the peripheral building, the vertical force per bay is 5.031 kN/m of height. The ice load is applied over half the circumference of the outer pier (approximately 45 tonnes). The dome also has to resist ice loads on half of its surface. The dome loads are summarized in the following table.

Table 7: Ice load on the dome

<table>
<thead>
<tr>
<th>Vertical Reaction (kN)</th>
<th>Horizontal Eccentricity (m)</th>
<th>Moment w.r.t. dome center (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-851</td>
<td>9.1</td>
<td>7740</td>
</tr>
</tbody>
</table>

2.2.2.4 Snow load

The pressure applied by snow is approximated to be 150 kg/m² on the dome. It is assumed that snow loads can be neglected for the vertical walls of the outer pier. The snow load is applied as a combination of point load and point moment on the center of gravity of the dome.

Table 8: Snow load on the dome

<table>
<thead>
<tr>
<th>Vertical Reaction (kN)</th>
<th>Horizontal Eccentricity (m)</th>
<th>Moment w.r.t. dome center (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>-334</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>

2.2.2.5 Wind load

Effect of winds on the structure has also been considered in this study. The maximum wind speed that the structure resists is evaluated to be 78 m/s. The ASCE-7 2010 procedure for dome roofs is used to compute the forces induced on the structure by the wind. This procedure is used for the dome and the outer pier walls. It evaluates a force for each node of the dome. The lateral and vertical wind forces for the dome are summed and applied at a height of 6.865 m from the observation level (the center of pressure).

The wind loads applied to the exterior walls are concentrated at each intersection of column and a beam. This method gives a realistic assumption of the wind effects since it considers the positive and negative pressure effects over the whole perimeter of the outer pier. The forces are applied perpendicularly to the surface of the outer pier. The following table summarizes the wind loads. Two cases have to be considered according to the ASCE-7 2010, cases A and B.

Table 9: Lateral wind loads on outer pier

<table>
<thead>
<tr>
<th>Bay</th>
<th>Lateral perpendicular force per vertical meter- Case A (kN/m)</th>
<th>Lateral perpendicular force per vertical meter- Case B (kN/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7.40</td>
<td>13.72</td>
</tr>
<tr>
<td>2</td>
<td>-5.23</td>
<td>2.05</td>
</tr>
<tr>
<td>3</td>
<td>-17.87</td>
<td>-15.43</td>
</tr>
<tr>
<td>4</td>
<td>-21.15</td>
<td>-21.15</td>
</tr>
<tr>
<td>5</td>
<td>-15.09</td>
<td>-15.09</td>
</tr>
<tr>
<td>6</td>
<td>-9.03</td>
<td>-9.03</td>
</tr>
<tr>
<td>7</td>
<td>-9.03</td>
<td>-9.03</td>
</tr>
</tbody>
</table>
It should be noted that positive forces are compressive forces applied perpendicular to the surface of the outer pier and negative forces are tension forces. Table 10 presents the total lateral and vertical wind loads applied to the dome top.

**Table 10: Wind loads on dome**

<table>
<thead>
<tr>
<th>Lateral reaction (kN)</th>
<th>Vertical uplift reaction (kN)</th>
<th>Center of lateral reaction (m)</th>
<th>Center of vertical reaction (m)</th>
<th>Moment w.r.t. dome center (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1876</td>
<td>+1855</td>
<td>0.92 (height above dome center)</td>
<td>0</td>
<td>1718</td>
</tr>
</tbody>
</table>

### 2.2.2.6 Earthquake

The ASCE-7 2010 requires the structure to resist earthquake load for 2500 years return period. The structure is assumed to have a low ductility and energy dissipation capability. As a result, in order to have a conservative evaluation, ductility factor (R) is chosen to be equal to 2. The soil in place is evaluated to be a soil class C.

Equivalent static procedure is used to represent the effect of seismic forces on the building. It assumes the response of the structure under seismic excitation is mostly concentrated in the first lateral mode of vibration. The structure is pushed with a lateral force distribution representing the lateral deformation of the structure in its first vibration mode shape. To obtain these forces it is first necessary to obtain the first natural period of the structure. The chosen fundamental period is 0.5 seconds. The period will be influenced by the final design of the enclosure including the compliance of the mechanical interfaces, so only an estimate is possible at this point. This period is used to get the spectral acceleration at the base of the structure. The spectral acceleration is obtained using the design spectrum which is function of the emplacement and the soil parameters. For a fundamental period of 0.5 seconds and the Mauna Kea site, the design spectral acceleration is 0.5g. The spectral acceleration is then multiplied by the seismic weight which is equal to the dead weight to acquire the base shear. The base shear is distributed over the height of the structure as a triangular distribution. The base shear is equal to 50% of the seismic weight (i.e. 0.5g times the seismic weight). The forces at each floor are presented in Table 11.

**Table 11: Lateral seismic force at each level for the outer pier**

<table>
<thead>
<tr>
<th>Floor</th>
<th>Lateral force (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground</td>
<td>28.05</td>
</tr>
<tr>
<td>2</td>
<td>169.71</td>
</tr>
<tr>
<td>3</td>
<td>315.02</td>
</tr>
<tr>
<td>4</td>
<td>427.24</td>
</tr>
<tr>
<td>Observatory</td>
<td>1866.40</td>
</tr>
<tr>
<td>Dome</td>
<td>4105.88</td>
</tr>
<tr>
<td>Total base shear</td>
<td>6912.32</td>
</tr>
</tbody>
</table>
2.2.3 Load combinations

Following the ASCE-7 requirements, the load cases are combined to represent the worst loading conditions to apply on the structure. It is important to note that no load factor is applied to the dome dead load since the mass will be known accurately, and the estimated 500 tonnes is felt to be reasonably. The capacity of the structure is compared to each of the load combination to ensure that the structure meets the requirements. The structural capacity is evaluated using limit state design and the foundations capacity is evaluated using allowable stress design. Each method uses different load combinations. The structural capacity of the beams, columns, bracings and footings are evaluated using limit state design. The following load combinations are used for the LSD, in which, D is the dead load of the outer pier, L the live load on the outer pier, S the snow load on the dome, I the ice load on the outer pier and dome, W the wind load on the outer pier and dome and E the overall earthquake load. Also, \(D_{\text{Dome}}\) is the dead load of the dome and \(W_A\) and \(W_B\) are wind load from case A and B, alternatively.

1. \[1.4D + 1.0E + 0.2S + 1.2D_{\text{Dome}}\]
2. \[1.4D + D_{\text{Dome}}\]
3. \[1.2D + 1.6L + 0.5S + 0.2I + D_{\text{Dome}}\]
4. \[1.2D + 1.6S + 1.0L + D_{\text{Dome}}\]
5. \[1.2D + 1.6S + 0.5W_A + D_{\text{Dome}}\]
6. \[1.2D + 1.0W_A + 1.0L + 0.5S + 1.0I + D_{\text{Dome}}\]
7. \[0.9D + 1.0W_A + 1.0I + D_{\text{Dome}}\]
8. \[0.7D + E + 0.8D_{\text{Dome}}\]
9. \[1.2D + 1.6S + 0.5W_B + D_{\text{Dome}}\]
10. \[1.2D + 1.0W_B + L + 0.5S + 1.0I + D_{\text{Dome}}\]
11. \[0.9D + 1.0W_B + D_{\text{Dome}}\]

The soils are evaluated using allowable stress design. The combinations used are listed below.

1. \[1.0D + D_{\text{Dome}}\]
2. \[1.0D + 1.0L + 0.7I + D_{\text{Dome}}\]
3. \[1.0D + 1.0S + 0.7W_A + 0.7I + D_{\text{Dome}}\]
4. \[1.0D + 1.0S + 0.7W_B + 0.7I + D_{\text{Dome}}\]
5. \[1.0D + 0.75L + 0.75S + D_{\text{Dome}} + 0.525I + 0.525W_A\]
6. \[1.0D + 0.6W_A\]
7. \[1.0D + 0.6W_B\]
8. \[1.14D + 0.7E + 1.14D_{\text{Dome}}\]
9. \[1.0D + 0.45W_A + 0.75S + D_{\text{Dome}}\]
10. \[1.0D + 0.45W_B + 0.75S + D_{\text{Dome}}\]
11. \[1.1D + 0.75L + 0.525E + 0.75S\]
12. \[0.6D + 0.42W_A + 0.42I + 0.6D_{\text{Dome}}\]
13. \[0.6D + 0.42W_B + 0.42I + 0.6D_{\text{Dome}}\]
14. \[0.46D + 0.7E + 0.74D_{\text{Dome}}\]
15. \[1.0D + 0.75L + 0.75S + D_{\text{Dome}} + 0.525I + 0.525W_B\]

The design checks are done for each of the combinations described to assess the structure and foundation performance.
2.3 Finite element modeling of the structure

Finite element assumptions are fundamental to get accurate results and need to be reflected carefully. Inaccurate assumptions can drastically change the distribution of forces, deflections and dynamic properties. An extensive analysis was performed to make sure the behavior of structure was well represented. The structure is modeled in the structural analysis software SAP2000. SAP2000 permits to execute static and dynamic analysis. AISC sections are also available and the graphical interface permits to easily model and analyze its behavior. The steel sections are modeled using frame elements; wide flange sections are imported from AISC sections and built up sections defined in SAP2000.

Most of the beams are attached to other beams or columns through their webs only. This means that moments are not fully transmitted to the members it is attached to. For this reason the connections are designed as pinned connections, i.e. the moments are released at their extremities. Although, some of the connections do transfer moments, the design plans were carefully studied to assure the connections are properly modeled to obtain the most representative behavior of the real structure.

For simplicity, most floor framings are modeled using typical bay framing shown in the drawings. The structural model does not include the balcony on the observatory level. This was decided because the balcony does not induce significant forces and that it was over complicating the model.

External columns are drawn continuously and are pinned at the base. The connections between the ring girder members are clamped. The diagonal columns are also pinned at their base and clamped at their top at the connection with the 3rd floor radial beams. Tension hangers on the inner side support the second and third floor beams and are not supported on the ground. The bracings are also assumed to be pinned.

There are different ways of accounting for slabs in FEM. In simple structures, they can be ignored or rigid diaphragms can be applied. The CFHT outer pier is a complex 3D structure that requires realistic assumptions and accurate modeling to obtain reliable results. It was decided to model the slabs in the FEM. Different area elements exist to model them. Since there are no shear connectors joining the slabs and steel frame, membrane elements are chosen. Membrane elements are used to avoid having the slabs resisting the gravity loads because they only support in-plane forces and normal moments. Concrete slabs are modeled using membrane area element with a thickness of 63.5 mm for all levels expect for the observation level which has a thickness of 250 mm.

The slab on the center of the observation floor is connected with shear connectors to the beams in x direction. This means the slab participates in combination with the beams to resist loads. For simplification and conservatism, the composite action of the slab is ignored. Figure 19 presents the SAP2000 finite element model of the pier and dome structures including the membrane elements.

After the structural finite element model is completed the loads are applied to the different members and nodes. After the combinations are defined, the analysis can be run and the capacity checked.
Analysis

For each of the load combinations described in the precedent section, the forces induced by these loads in the different structure’s members have to be compared to the capacity of the members. Therefore, the beams, columns, hangers, bracings, footings and foundation have to be assessed. This chapter presents the design checks and results for each of the member types as well as the dynamic properties and the deflections under earthquake loads.

The capacity of the slabs of the structure is not verified because it is assumed the gravity loads on the floors of the outer pier are equal to the ones used for the initial design and that they do not participate in strengthening of the structure.

The following methodology is used to evaluate the strength performance of the structure’s members. The axial forces, shear forces and moments at the extremities and center of each members for each load combination is exported from the SAP2000 model to EXCEL spreadsheets. These spreadsheets are set up according to the AISC requirements depending on the section type and the type of loads being resisted.

This section summarizes the design assumptions and results of the analyses that are performed to assess the performance and capacity of the outer pier. The first part presents the modal analysis of the outer pier and...
dome idealization. The second part shows the design checks results that are done to assess the performance of the structure. The deflections of the structure are also assessed. The earthquake and wind loads are applied parallel to the y-axis because of the smaller number of braced bays participating in resisting the loads in that direction.

2.4.1 Modal analysis

To study the dynamic behavior of the structure, modal analysis is performed using SAP2000. Modal analysis of the outer pier allows evaluating if the modeling of the structure has been done properly. Also, obtaining the fundamental period is necessary to determine the base shear and the earthquake loads described earlier in this document. In the load cases description, it was motioned that a period of 0.5 seconds is used. Modal analysis is highly dependent on modeling inaccuracies and assumptions, the absence of non structural elements, etc. This leads to great variability in the natural periods of the structure. To make sure the structure is evaluated with conservatism a lower value for the first natural period should be chosen. This approach is in place building codes and they usually states that the period taken from a modal analysis should be limited to a certain value based on engineering judgement. It is determined to limit the first natural period to a value of 0.5 seconds to assure conservatism in the results. The results of the modal analysis and equivalent static force analysis obtained from the model are shown below.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Natural period from modal analysis (sec)</th>
<th>Mode shape description</th>
<th>Natural period for equivalent static force analysis (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.64</td>
<td>Translational mode in X direction</td>
<td>0.5</td>
</tr>
<tr>
<td>2</td>
<td>0.64</td>
<td>Translational mode in Y direction</td>
<td>0.5</td>
</tr>
</tbody>
</table>

The first two modes have a natural period of 0.64 seconds and their mode shape is a lateral translation of the whole structure. The mode shapes are plotted in the next figures.

![Figure 20: First translational mode of a radial segment](image)
2.4.2 Beams

The beams are referred as the elements of the structure with negligible axial loads. The beams in the outer pier are the horizontal steel members forming the framing of each floor. The gravity loads are applied perpendicularly to their longitudinal axis, so the axial load is negligible. The sections forming the floors are W sections, but for the external ring girder on the observatory floor which is a built up hollow structural section (HSS). According to the AISC, the following checks have to be done and the procedure is explained in details in the code.

Two limit states have to be verified for members being checked for flexure design. First, flexural lateral torsional buckling is evaluated, it verifies if the combination of flexural yield strength and the lateral torsional buckling strength is high enough. Finally, the shear strength has to be checked, it is defined as the capacity of the section to resist shear forces.

It was observed that all the beams have sufficient capacity to resist the loads applied on them. None of the beams present a demand higher than their capacity. These results were expected since the gravity loads on the floors of the outer pier have no changed compared to the original design. As a result, all the beams have a sufficient capacity.

2.4.3 Columns

The columns are members subjected to compression forces and bending moments. The columns in the proposed model are divided in three categories, the external columns, the internal columns present on the fourth and observation levels and the diagonal columns running from the footing to third floor. Each of these members is checked for the forces at their extremities and at mid span.

For each load combination and for each element, both the elastic buckling and the flexural lateral torsional buckling and the combination of those are checked.
The capacity of the columns is sufficient under every load combination. The results for each level for the worst internal, external and diagonal columns are given in the next tables.

2.4.3.1 Exterior columns
The exterior columns are W12x65 sections for each bay and level. The load combination inducing the greatest demand for each level is $1.4D + 1.0E + 0.2S + 1.2D_{\text{Dome}}$. Only the column presenting the highest ratio of demand over capacity is given.

**Table 13: Exterior columns performance under the worst load combination**

<table>
<thead>
<tr>
<th>Level</th>
<th>Ratio of demand to capacity for $1.4D + 1.0E + 0.2S + 1.2D_{\text{Dome}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground</td>
<td>0.87</td>
</tr>
<tr>
<td>2</td>
<td>1.01</td>
</tr>
<tr>
<td>3</td>
<td>0.64</td>
</tr>
<tr>
<td>4</td>
<td>0.78</td>
</tr>
<tr>
<td>Observation</td>
<td>0.58</td>
</tr>
</tbody>
</table>

The column of the second level has a ratio of demand to capacity of 1.01. This result being really close to 1.0, it is judged that it has a sufficient capacity.

2.4.3.2 Interior columns
The steel sections for the interior columns are W12x65. Again, the interior columns are present on the inner perimeter of the outer pier on the third and fourth levels. The load combination governing the design check is $1.2D + 1.6L + 0.5S + 0.2I + 1.0D_{\text{Dome}}$.

**Table 14: Interior columns performance under the worst load combination**

<table>
<thead>
<tr>
<th>Level</th>
<th>Ratio of demand to capacity for $1.2D + 1.6L + 0.5S + 0.2I + 1.0D_{\text{Dome}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>0.47</td>
</tr>
<tr>
<td>Observation</td>
<td>0.36</td>
</tr>
</tbody>
</table>

The interior columns have sufficient capacity to resist the loads applied to them.

2.4.3.3 Diagonal columns
The sections are W12x96 for the ground level and W12x65 for the second and third level. The load combination inducing the highest demand is $1.2D + 1.6L + 0.5S + 0.2I + 1.0D_{\text{Dome}}$.

**Table 15: Diagonal columns performance under the worst load combination**

<table>
<thead>
<tr>
<th>Level</th>
<th>Demand vs. capacity ratio for $1.2D+1.6L+0.5S+0.2I+1.0D_{\text{Dome}}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground</td>
<td>0.54</td>
</tr>
<tr>
<td>2</td>
<td>0.84</td>
</tr>
<tr>
<td>3</td>
<td>0.96</td>
</tr>
</tbody>
</table>

The diagonal columns have adequate capacity to sustain the forces induced by the load combination $1.2D + 1.6L + 0.5S + 0.2I + 1.0D_{\text{Dome}}$. 
2.4.4 Hangers
Hangers are the vertical steel members at the interior of outer pier supporting the ground and the second floors. They are only subjected to axial tension forces.

Three ultimate limit states are checked for the hangers. The first limit state to be checked is yielding of the cross section in tension. Following that, the rupture of the section at the connection is assessed. Finally, the rupture of the bolts in shear is checked.

The hangers are tension members made of two back to back C6x10.5 sections. The capacity is controlled by the shear resistance of the bolts of the connections. The load combination that develops the most demand in the hangers is 1.2D + 1.6L + 0.5S + 0.2I + 1.0D_Dome.

Table 16: Hangers performance under the worst load combination

<table>
<thead>
<tr>
<th>Level</th>
<th>Ratio of demand to capacity for 1.2D + 1.6L + 0.5S + 0.2I + 1.0D_Dome</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground</td>
<td>0.52</td>
</tr>
<tr>
<td>2</td>
<td>0.77</td>
</tr>
</tbody>
</table>

The hangers present sufficient capacity and could even sustain higher loads.

2.4.5 Bracings
The bracings are the elements resisting the lateral loads induced by earthquakes and wind hazards. They are the diagonal elements on the external perimeter of the outer pier. For a load applied in one direction, in each bay, one bracing resists the load in tension and the other one resist it in compression. It is desirable to have both of these bracings staying in their elastic range. Only axial loads are developed in these members since they have pinned connections.

The tension bracings are checked for yielding of the cross section and rupture of the connection. The compression members are checked for axial compression capacity which is the elastic buckling strength and the rupture of the connection. The shear resistance of the bolts has the lowest resistance so it is assumed that the compression and tension capacity of the bracings is the one of the bolts in shear.

Between the observation and fourth levels bracings are incorporated at each bay and the bracings are composed of a built up section of 4 steel angles L5x3x1/2 forming an I-shaped section. Between the lower levels bracings are formed of double back to back angles L6x4x1/2 and only alternate bays are braced, see Figure 13. The braces are attached at the base of the node points of the lower floor and at midspan of the beams of the higher level.

The load combination that induces the most demand in the bracing is 1.4D + 1.0E + 0.2S + 1.2D_Dome. The results are summarized in the next table.

Table 17: Bracing performance under the worst load combination

<table>
<thead>
<tr>
<th>Level</th>
<th>Ratio of demand to capacity for 1.4D + 1.0E + 0.2S + 1.2D_Dome</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground and 2</td>
<td>4.19</td>
</tr>
<tr>
<td>3</td>
<td>3.73</td>
</tr>
<tr>
<td>4</td>
<td>2.95</td>
</tr>
<tr>
<td>Observation</td>
<td>1.57</td>
</tr>
</tbody>
</table>

It is observed that the braces at each of the floors do not have sufficient capacity to resist the forces the earthquake loads induces in them. It is mostly noticeable at the lower levels. The higher forces in the bracings...
are expected. The outer pier was originally designed for a ratio of base shear over seismic weight \( (V_{\text{base}}/W) \) of 12%. The ratio of \( V_{\text{base}}/W \) the structure is assessed for in this study is of 50%. This original base shear is 4.16 times lower than the one used for the evaluation. This explains the demand to capacity ratio of 4.19.

To ensure the accuracy of the results obtained from the SAP2000 model, first principles approach is used to evaluate the forces in the bracings at the ground level. The following section presents the hand calculations.

### 2.4.5.1 First principles approach bracing forces calculations

It is wished to evaluate the force in the bracings at the ground level to compare to the values obtained in the finite element analysis. To do so, the number of bays participating in resisting the lateral forces is first calculated:

- Bays participating if a lateral earthquake load is applied in X direction: \( 2 + 4 \cos(60^\circ) = 4.0 \)
- Bays participating if a lateral earthquake load is applied in Y direction: \( 4 \cos(30^\circ) = 3.46 \)

**Figure 22: Braced bay representation**

The shear per bay can then be obtained by dividing the total base shear by the smallest number of participating bays:

- \( V_{\text{base}} = 6912 \text{ kN} \) (calculated from ASCE-7 2010)
- Lateral load applied to each bay: \( V_{\text{per bay}} = V_{\text{base}} / 3.46 = 1995 \text{ kN} \)

The axial force in the bracings is calculated assuming both braces take the same amount of force and the angle of the brace:

- Angle between the horizontal beam and the bracing: \( \theta = \tan^{-1}(h_1 / (b / 2)) = 51.19^\circ \)
- Brace axial load: \( P_{\text{brace}} = V_{\text{per bay}} / 2 \cos(\theta) = 1595 \text{ kN} \)

The brace load obtained from SAP2000 is 1681 kN. Therefore, there is 5% difference between the values obtained by the hand calculation and the one from the model. This difference is in part due to the moment applied on top of the dome and gravity loads.

It is concluded that SAP2000 provides accurate results.
2.4.6 Footing

The structural capacity of the concrete footing is also verified. It is assumed that only axial loads are transmitted to the footing and that the diagonal columns do not induce moments. The Structural Design Brief for the Peripheral Building (SNC, 1974) also made that assumption.

The bending and shear capacities in the tangential and radial directions have to be assessed as well as the tension in the hoops. The bending capacity of the section is evaluated using Response2000 and the shear capacity using CAN/CSA-A23.3-04 requirements.

The bending moment around the tangential axis of the footing is resisted by the reinforcement bars parallel to the width of the footing. The bending moment around the radial axis is resisted by the reinforcement parallel to the length of the footing. The shear for a radial section is resisted by the concrete and steel shear stirrups. The shear for a tangential section is resisted by concrete in the tangential direction in the bottom part of the footing. Finally, the longitudinal tension in the circular footing induced by the radial horizontal forces of the columns is resisted by the longitudinal steel reinforcement bars.

The following table presents the demand to capacity ratio for each ultimate limit state.

<table>
<thead>
<tr>
<th>Ultimate state</th>
<th>Load combination</th>
<th>Demand to capacity ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bending around tangential axis</td>
<td>$1.4D + 1.0E + 0.2S + 1.2D_{Dome}$</td>
<td>0.24</td>
</tr>
<tr>
<td>Bending around radial axis</td>
<td>$1.4D + 1.0E + 0.2S + 1.2D_{Dome}$</td>
<td>0.49</td>
</tr>
<tr>
<td>Shear for tangential section</td>
<td>$1.4D + 1.0E + 0.2S + 1.2D_{Dome}$</td>
<td>0.44</td>
</tr>
<tr>
<td>Shear for radial section</td>
<td>$1.4D + 1.0E + 0.2S + 1.2D_{Dome}$</td>
<td>0.33</td>
</tr>
<tr>
<td>Tension in hoops</td>
<td>$1.2D + 1.6L + 0.5S + 0.21 + 1.0D_{Dome}$</td>
<td>0.41</td>
</tr>
</tbody>
</table>

It is concluded that the footing has sufficient capacity since the ratio of demand vs. capacity are below 1.0.

2.4.7 Foundation

The foundations have to be evaluated using allowable stress design load combinations. The bearing capacity and the sliding of the footing are checked in this section.

2.4.7.1 Bearing capacity

It is also necessary to evaluate if the soils supporting the footings can resist the loads transmitted. The pressure induced by each column to the footing is assumed to be equally distributed to the soils via the footing over its tributary area, the total area of footing divided by 12 columns. The pressure applied on the soil has to be smaller than the bearing capacity of the foundation.

The bearing capacity from the Dames & Moore Soil Report evaluates the bearing capacity to 191 kPa. Although the Structural Design Brief for the Peripheral Building (SNC, 1974) uses a bearing capacity of 161 kPa. To obtain conservative results, the value of 161 kPa is used for the checks. The pressure includes the weight of the footing and of the soil in place over the footing. The bearing capacity results for the worst column for the earthquake load combination and gravity load combination are given in the next table.

<table>
<thead>
<tr>
<th>Bearing capacity (kPa)</th>
<th>Demand to capacity ratio for $1.1D + 0.75L + 0.525E + 0.75S$</th>
<th>Demand to capacity ratio for $1.0D + 1.0L + 0.7I + 1.0D_{Dome}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>161</td>
<td>0.97</td>
<td>0.66</td>
</tr>
</tbody>
</table>
The bearing capacity for earthquake and gravity loads is not exceeded. The capacity is concluded to be sufficient.

### 2.4.7.2 Sliding of the footing

The sliding of the foundation is also verified. It consists of checking if the total lateral load is smaller than the sliding capacity. The sliding capacity is the summation of the friction between the footing and the foundation and of the passive pressure soil resistance placed around the footing.

The sliding of the footing calculations procedure was taken from Coduto (2001). It assumes the sliding resistance to be dependent on the allowable coefficient of friction of the soil, the area of the footing, the axial load applied on the foundation. The loads are also resisted by passive pressure which is dependent on the soil type and weight. The results are summarized in the next table.

**Table 20: Sliding capacity performance for earthquake loads**

<table>
<thead>
<tr>
<th>Demand to capacity ratio for 0.46D + 0.7E + 0.74D&lt;sub&gt;Dome&lt;/sub&gt;</th>
<th>Demand to capacity ratio for 1.14D + 0.7E + 1.14D&lt;sub&gt;Dome&lt;/sub&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.74</td>
<td>0.60</td>
</tr>
</tbody>
</table>

The load combination 0.46D + 0.7E + 0.74D<sub>Dome</sub> results in a lower ratio because of the lower axial loads compared to the load combination 1.14D + 0.7E + 1.14D<sub>Dome</sub>. The sliding capacity of the structure is sufficient to resist the loads applied.

### 2.4.8 Deflections

Deflections of the structure under earthquake loadings are needed to be evaluated to verify if the outer pier is interacting with the concrete inner pier and if the ASCE-7 2010 interstorey drifts limitations are respected.

The distance at the fourth level between the outer and inner pier is 76.2 mm and the maximum interstorey drift is 2.5% according to the ASCE-7 2010. The displacements induced by the earthquake lateral forces applied are elastic deformation. To obtain the maximum inelastic displacements, the deformations need to be multiplied by the deflection amplification factor which is equal to 2 (since R=2). The deflections are checked for each level in X and Y directions in tables below.

**Table 21: Outer pier deflections in Y direction summary**

<table>
<thead>
<tr>
<th>Level</th>
<th>Elastic displacements (mm)</th>
<th>Inelastic displacements (mm)</th>
<th>Demand to capacity ratio</th>
<th>Interstorey drifts</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground</td>
<td>3.0</td>
<td>6.0</td>
<td>0.08</td>
<td>0.77%</td>
</tr>
<tr>
<td>2</td>
<td>18.0</td>
<td>35.9</td>
<td>0.47</td>
<td>0.77%</td>
</tr>
<tr>
<td>3</td>
<td>34.5</td>
<td>68.9</td>
<td>0.90</td>
<td>0.82%</td>
</tr>
<tr>
<td>4</td>
<td>47.0</td>
<td>94.0</td>
<td>1.23</td>
<td>0.81%</td>
</tr>
<tr>
<td>Observation</td>
<td>50.6</td>
<td>101.1</td>
<td>-</td>
<td>0.23%</td>
</tr>
</tbody>
</table>

**Table 22: Outer pier deflections in X direction summary**

<table>
<thead>
<tr>
<th>Level</th>
<th>Elastic displacements (mm)</th>
<th>Inelastic displacements (mm)</th>
<th>Demand to capacity ratio</th>
<th>Interstorey drifts</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground</td>
<td>2.4</td>
<td>4.9</td>
<td>0.06</td>
<td>0.63%</td>
</tr>
<tr>
<td>2</td>
<td>14.5</td>
<td>29.1</td>
<td>0.38</td>
<td>0.62%</td>
</tr>
<tr>
<td>3</td>
<td>27.9</td>
<td>55.7</td>
<td>0.73</td>
<td>0.66%</td>
</tr>
</tbody>
</table>
It is observed that the deflections in Y direction are higher than in X direction. The first principle approach of the bracings design taught us that more bracings are participating in X direction than in Y directions which permits us to expect higher deflections in Y direction. This confirms again that it was a good decision to perform the analysis in Y direction.

The interstorey drifts are all under 2.5% meaning the structure is stiff enough for the ASCE-7 requirements. The deflections in Y direction are higher than the space between the concrete internal pier and the outer steel pier which makes us conclude that interaction between both structures can be expected during major earthquake events. It should also be noted that the inner pier will also sustain deflections that are not necessary in the same direction as the outer pier because they have different vibration periods. The deflections in X direction are smaller than the allowable space between the two structures.

### 2.5 Conclusions from Existing Enclosure Pier Evaluation

The upgrade of the current Canada-France-Hawaii telescope and dome necessitated evaluating if the outer pier supporting the dome has enough capacity to accommodate the new design. The larger and heavier dome inducing higher forces to the outer pier and the new code requirements justify reassessing the capacity of the outer pier. The seismic requirements were analyzed with great care since the modern codes prescribes much higher demands than the ones from the original design. A methodology is developed to perform the assessment of the structure using finite element modeling and first principle approach. The load cases and combinations were defined using the ASCE-7 2010 requirements. The capacity of the beams, columns, bracings, footing and foundation were evaluated and the performance of these members assessed. The AISC requirements were used for the steel design. The following points were concluded:

- The beams forming the different level floors have sufficient capacity to resist the loads applied on them.
- The different columns transferring the loads to the footings have their capacities exceeding the demands induced by the different load combinations. The ratio of demand vs. capacity of the external columns is of 1.01 for the second level. There is no capacity left for the external columns.
- The bracings resisting the lateral loads have their demand exceeding their capacity by a ratio of 4.19. This outcome results from the fact that the earthquake forces are 416% higher than for the original design requirements. Different solutions exist to strengthen the bracings.
- The maximum interstorey drift requirement of 2.5% is met for all load combinations and the maximum deflection at the fourth floor is of 101 mm and exceeds the 76 mm space between the outer and inner pier. Interaction under high seismic hazard can be expected.
- The footings have sufficient bending, shear and tension structural capacity to resist the demands of every load combination. The ratios of demand to capacity are under 0.50.
- The maximum bearing pressure induced by the footing to the foundation is 97% of the 161 kPa bearing capacity. The sliding capacity is also sufficient since the demand to capacity ratio is of 0.74.
References


