Next Generation CFHT

DSL Phase II Report:

Programmatic Study for Upgrade of Telescope Structure and Enclosure

Dynamic Structures Ltd. &
University of British Columbia

October 2012
Revision History

<table>
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<tr>
<th>Version</th>
<th>Date</th>
<th>Comments</th>
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<tr>
<td>DRAFT1</td>
<td>Sep 20, 2012</td>
<td>Interim report including deconstruction plan and cost estimate</td>
</tr>
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<td>REV 1</td>
<td>Oct 31, 2012</td>
<td>Added manufacturing &amp; construction plan and cost estimate; updated deconstruction plan based on Sept 27, 2012 telecon with K.Szeto &amp; CFHT staff</td>
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1. Introduction

This report is a summary of the results of a study on the Next Generation Canada-France-Hawaii Telescope (ngCFHT). The work was carried out by Dynamic Structures Ltd. (DSL) in conjunction with the University of British Columbia (UBC) in September and October 2012. The objectives of the study are to develop and outline the programmatic requirements including the sequence, cost, schedule and construction equipment required to upgrade the current CFHT to the ngCFHT facility.

The work builds on previous studies carried out by DSL and UBC in 2011, which included an assessment of existing CFHT telescope and enclosure pier capacity, and the development of the ngCFHT telescope and enclosure configuration.

Deliverables for this phase of work include the following:

1. Provide cost and schedule estimates for deconstruction of the current CFHT facility including removal of the existing enclosure and telescope structure and the associated services
2. Provide cost and schedule estimates to upgrade the load capacities of the existing enclosure and telescope piers to meet current structural code requirements based on findings from the Phase I studies
3. Provide cost and schedule estimates for construction of the new ngCFHT facility including the new enclosure and telescope structure systems based on the baseline enclosure and telescope configuration developed in Phase I
4. Provide CAD model support for the ngCFHT CFD aero-thermal study which will consist of reviewing the model provided by the client for geometrical correctness and proposing ventilation schemes that are consistent with the structural/mechanical design concept
5. Provide photo-realistic rendering of telescope and enclosure using summit topology provided by the client

The information used for this study is based on:

1. Historical data of the current CFHT dome and telescope available at DSL and CFHT
2. Findings from the previous pier load capacity studies and ngCFHT enclosure and telescope configuration study
3. Where information is insufficient, best engineering estimates are used and the assumptions are documented in the study report.
4. It is noted that the designs of the ngCFHT enclosure and telescope structures are presently at an early conceptual level, and the cost estimating methodology and associated contingencies used during this phase reflect this. The cost estimating uses a combination of top-down cost scaling from other observatories as well as bottom-up cost estimating.
2. Deconstruction Plan

2.1 Introduction/Methodology
DSL worked with Jim Waldbauer, who was one of the foremen who worked on the construction of the dome and telescope for the original CFHT, to develop the deconstruction plan and cost estimate outlined below. The original construction drawings for the telescope structure were used to determine component sizes and weights.

2.2 Deconstruction Assumptions
The following outlines the assumptions made by DSL in the development of the deconstruction plan and cost estimate:

- General assumptions on pre-deconstruction conditions:
  - All instruments removed before deconstruction
  - All control room and computer equipment removed before deconstruction
  - All lab and instrument equipment removed before deconstruction
  - All furniture and office equipment removed before deconstruction
  - All telescope optics removed before deconstruction

- Scope of enclosure deconstruction:
  - All steel work to be removed, including azimuth ring girder, bogies, and drive system
  - Components do not need to be preserved for future use

- Scope of telescope deconstruction:
  - All steel work to be removed
  - Removal of observation floor around telescope structure and glycol lines
    - It is assumed that the existing observation floor (5th floor) will remain as the observation floor for the new observatory. It will be required that the center portion of the observation floor around the existing telescope will be removed in order to allow deconstruction of the existing telescope and to allow clearance for construction and operation of the new telescope.
    - Components do not need to be preserved for future use

- The following items have not been included in the scope of deconstruction estimated by DSL
  - Removal of coating chamber/aluminizing tank
  - Remove the glycol chiller and distribution facility
  - Remove hydraulic lines, pump and distribution equipment
  - Removal of electrical distribution equipment
  - Removal of air handling units

2.3 Site Set-up for Deconstruction
A mobile crane will be set-up on the south side of the building where there is good access for the crane. The telescope will be disassembled prior to the dome using a combination of the existing dome crane (12 ton capacity) and a temporary 40 ton capacity crane system mounted to the dome arch girders. These cranes will be used in conjunction with the dome rotation to disassemble the telescope and place the disassembled
components on the south side of the observatory floor. From this point the mobile crane can reach through the dome slit and lift the components from the observatory floor and remove them from the dome. After the telescope deconstruction is complete the dome will be deconstructed using the mobile crane. The figures below show the site set-up for deconstruction, and further details of the deconstruction sequence are provided below.

Figure 1: CFHT observatory from the south
Figure 2: Plan view showing mobile crane set-up for deconstruction
Figure 3: West elevation of site set-up for deconstruction
A 250 tonne hydraulic crane is assumed for the preliminary mobile crane selection. The crane has sufficient capacity to reach into the dome between the arch girders and remove the telescope components from the south side of the observatory floor. Furthermore, it is hoped that this crane has sufficient capacity that it can also be used to erect the new telescope and enclosure in order to save costs on freight and mobilization.

Figure 4: Terex 250 tonne hydraulic crane

2.4 Telescope Deconstruction Sequence

The major components of the telescope structure are labelled in the figure below and are referenced in the deconstruction sequence.
Figure 5: Existing telescope main components
The telescope deconstruction sequence is as follows:

1. Remove top end & upper tubes with top-end crane
2. Attach rigging and temporary crane to dome arch girders
3. Erect center section of false work
4. Erect temporary support for east and west tube
5. Erect temporary supports for horseshoe
6. Prep horseshoe, east and west tubes
7. Remove horseshoe horns (28.6 tons each)
8. Remove horseshoe center section (28.6 tons)
9. Remove east and west tubes (9.9 tons each)
10. Remove south yoke (26.7 tons including bearing)
11. Remove center ("caisson central") section (27.6 tons)
12. Prep telescope floor and glycol lines
13. Remove telescope floor and glycol lines
14. Remove south pedestal
15. Remove north pedestal
16. Remove triangular base frame
17. Clean-up observatory floor
18. Provide weather protection over floor and floor opening
19. Load trucks
20. Remove rigging and temporary crane from dome

2.5 **Dome Deconstruction Sequence**

The major components of the dome are labelled in the figure below and are referenced in the deconstruction sequence.

![Figure 7: Existing dome main components](image-url)

The dome deconstruction sequence is as follows:

1. Remove dome insulation
2. Remove mezzanine, catwalks and air handling units on observation floor
3. Remove catwalks and bus-bar
4. Remove shutters
5. Remove wind screen & drive system
6. Erect exterior work platform on arch girders
7. Remove vent platforms
8. Prep intermediate infill shell plates
9. Remove intermediate infill shell plates
10. Install temporary pipe struts on arch
11. Remove rack at arch splices
12. Prep shell modules, including vents (if required)
13. Remove shell modules, including vents (if required)
14. Remove outer skirts
15. Adjust bogie loads as required to distribute even loads as dome elements are removed
16. Prep arch falsework
17. Erect arch falsework
18. Connect arch girders and tower
19. Gouge arch girders splices
20. Gouge tie beam
21. Prep back shell
22. Remove back shell and tie beam
23. Remove top-end crane
24. Remove two arch center sections
25. Remove four arch internal sections
26. Remove four arch horn sections
27. Dismantle erection falsework tower
28. Gouge ring girder sections
29. Erect temporary bogie supports
30. Remove ring girder sections, including bus bar
31. Remove bogies
32. Load & transport dome parts

2.6 Deconstruction Cost Estimate & Schedule
A preliminary cost estimate was developed for the deconstruction activities and scope assumptions described above. The following outlines the estimate methodology and exclusions.

- Deconstruction labour:
  - Supervision & crane operators: includes import DSL supervision (superintendent + site engineer), local foreman, mobile crane operator, and oiler
  - Telescope & enclosure labour assumes local (Hawaiian) labour is used for deconstruction tasks described above. Labour rates are based on local union rates and a 54hr working week (6 days x 9hrs/day) with overtime and travel time applied per union agreements
  - Live-out and travel: includes daily live-out allowance, air travel, and travel time for shift turn-around for both import and local workers
  - Workers compensation insurance is included in the estimate
  - Labor hours include an efficiency factor of 1.6 to account for working at high altitude.

- Deconstruction equipment:
  - Large equipment: includes rental of 250t mobile crane, 25t rough-terrain crane, 2 x aerial boom lifts, and tractor trailer unit. Freight, fuel, mobilization and de-mobilization costs are included for all equipment. Includes temporary 40 ton dome crane.
o Misc. equipment, tools & falsework: includes small equipment such as tools, compressors, welding equipment, consumables, temporary power distribution, and scaffolding. Includes 50 tonnes of falsework, including material, labour, and shipping costs.
o Ground transport & trucking: includes purchase of 1 x SUV and 1 x pickup truck for worker transport and fuel. Includes trucking of deconstructed components from summit to Hilo.

- Contingency: a suggested overall contingency of 20% is applied
- Exclusions:
  o Scrapping: no costs have been including for scrapping of the structure other than trucking of the components from the summit to Hilo
  o Hazardous material disposal: costs have not been included for proper disposable of any hazardous materials (such as glycol or hydraulic fluid)
  o Crane access ways (compacted, level roadways suitable for moving large cranes)
  o Water supply, toilets, and first aid facilities
  o See also scope assumptions and exclusions in section 2.2 Deconstruction Assumptions above.

The cost estimate summary is given in the table below. Costs are given in 2012 US dollars.

**Table 1: Deconstruction cost estimate**

<table>
<thead>
<tr>
<th>Deconstruction Labour</th>
<th></th>
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<tr>
<td>Supervision &amp; crane operators</td>
<td>$871,992</td>
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<td>Telescope labour</td>
<td>$380,289</td>
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<td>Enclosure labour</td>
<td>$766,561</td>
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<td>Live-out &amp; travel</td>
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<td>Workers comp insurance</td>
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<td><strong>Total labour</strong></td>
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</table>

<table>
<thead>
<tr>
<th>Deconstruction Equipment</th>
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</tr>
</thead>
<tbody>
<tr>
<td>Large equipment</td>
<td>$1,343,821</td>
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<tr>
<td>Misc. equipment, tools &amp; falsework</td>
<td>$396,951</td>
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<tr>
<td>Ground transport &amp; trucking</td>
<td>$198,638</td>
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<td><strong>Total equipment</strong></td>
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<td><strong>Subtotal</strong></td>
<td><strong>$4,784,558</strong></td>
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<td>Mark-Up (15%)</td>
<td>$717,684</td>
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<tr>
<td>Contingency (20%)</td>
<td>$1,100,448</td>
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<td><strong>TOTAL</strong></td>
<td><strong>$6,602,690</strong></td>
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A preliminary schedule for the deconstruction is shown below, assuming a nominal start date of Jan 1, 2014.
Figure 8: Preliminary deconstruction schedule
3. Outer Pier Structure Upgrade

The study done by DSL and UBC titled “ngCFHT Telescope and Enclosure Configuration and Outer Pier Capacity Study” in January 2012 revealed that diagonal braces along the outer steel pier need to be upgraded in order to meet the current design standards. The pier was designed and built in the mid-70s. The existing pier was analyzed under various combinations of loads, including wind, snow and seismic. The study determined that the diagonal braces on the outer pier do not meet the load requirements for seismic strength. Figure 9 below shows the pier under construction.

![Figure 9: Existing CFHT Pier Structure](image)

The previous report was reviewed and an estimate was developed for the cost and hours involving the upgrade. In addition, old drawings of the existing CFHT pier were reviewed to determine access and existing conditions.

### 3.1 Upgrade Assumptions

The following assumptions have been taken in order to estimate the hours:

1. The brace upgrade works will happen at the same time as enclosure deconstruction.
2. There are a total of 29 panels with braces, resulting in 58 individual braces in total.
3. The 25 ton RT crane used during the deconstruction phase will be utilized during brace upgrade works.
4. All access to braces inside the outer pier will be from outside. This work should be done under a tent that locally protects each opening from weather conditions.
5. The 4th floor cantilevered deck will be utilized for access to braces on the 4th floor. Access to remainder of floors will be via movable scaffolding.
6. New braces would be HSS 8x8x5/8 welded to new gussets plates on 1st, 2nd and 3rd floors.
7. New braces would be HSS 10x10x1/2 welded to new gusset plates on 4th floor.
8. The braces would be standard slotted HSS with reinforcing plates field welded to new gussets framed into existing beams and columns.

3.2 Brace Replacement Procedure

The following is a rough procedure outlining the main steps for removal and replacement of existing braces:
1. Setup scaffolding.
2. Cut exterior siding and remove along with insulation.
3. Setup and connect temporary strut underside of main ring beam on 4th floor only. This only applies to braces on 4th floor. A temporary strut is used to take the weight of outer ring beam, if required for safety. This step is omitted on other floors.
4. Gouge off gussets and braces.
5. Prep and clean up beam for fitting new gussets.
6. Fit and tack new gussets.
7. Hoist and install new braces.
8. Weld gussets and braces.
9. Apply fire retardant to new braces and gussets.
10. Re-install insulation and exterior siding and seal.
11. Move scaffolding to next panel.

3.2.1 Brace Upgrade Construction Schedule & Cost Estimate

A preliminary cost estimate was developed for the brace upgrade activities and scope assumptions described above. The following outlines the estimate methodology and exclusions.

- Construction labour:
  - Supervision & crane operators: includes import DSL supervision (superintendent shared from deconstruction phase), local foreman, mobile crane operator, and oiler also shared from the deconstruction phase.
  - Brace replacement labour assumes local (Hawaiian) labour is used for all tasks described above. Labour rates are based on local union rates and a 54hr working week (6 days x 9hrs/day) with overtime and travel time applied per union agreements.
  - Live-out and travel: includes daily live-out allowance, air travel, and travel time for shift turn-around for both import and local workers.
  - Worker’s compensation insurance is included in the estimate.
  - A labor efficiency factor of 1.6 has been applied to the estimated hours to account for effects of working at high altitude.

- Construction equipment:
  - Large equipment: includes rental of the 25t rough-terrain crane (shared during the deconstruction phase), and 2 x aerial boom lifts. Freight, fuel, mobilization and de-mobilization costs were included in the deconstruction phase. Therefore, these were not included in the costs for brace upgrade.
  - Misc. equipment, tools such as: compressors, welding equipment, consumables, temporary power distribution, and movable scaffolding included.
Ground transport & trucking: The cost of 2 pickups has been included along with fuel and maintenance for the additional crew responsible for brace replacement works.

- Contingency: a suggested overall contingency of 30% is applied. A higher rate is suggested due to some of the risk involved in such remediation project, where there is always some element of risk until the walls are actually opened to see actual conditions.

- Exclusions
  - Crane access ways (compacted, level roadways suitable for moving large cranes)
  - Water supply, toilets, and first aid facilities
  - See also scope assumptions and exclusions in sections 2.6 and above.
  - Permits

The cost estimate summary is given in the table below. Costs are given in 2012 US dollars.

### Table 2: Outer Pier Upgrade Cost Estimate

<table>
<thead>
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<th>Labour &amp; Material</th>
<th>Cost</th>
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<td>Supervision &amp; crane operators</td>
<td>$131,131</td>
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<tr>
<td>Live-out &amp; travel</td>
<td>$104,945</td>
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<td>Brace Replacement Labor</td>
<td>$667,450</td>
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<td>Materials</td>
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<td>Shipping</td>
<td>$58,870</td>
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<td>Insurance</td>
<td>$239,574</td>
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<td><strong>Total labour</strong></td>
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<th>Construction Equipment</th>
<th>Cost</th>
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<td>Large equipment</td>
<td>$22,605</td>
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<tr>
<td>Misc. equipment, tools &amp; falsework</td>
<td>$180,528</td>
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<td>Ground transport &amp; trucking</td>
<td>$90,212</td>
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<td><strong>Total equipment</strong></td>
<td><strong>$293,345</strong></td>
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**Subtotal** $1,798,365

| Mark-Up (15%)                             | $269,755 |
| Contingency (30%)                         | $620,436 |
| **TOTAL**                                 | **$2,688,556** |

The duration of this work is expected to take approximately 7 months with two crews of 2 welders, one helper and one foreman. The two crews would work in parallel as shown next page. A preliminary schedule for the brace upgrade works is shown below, assuming a nominal start date of Jan 1, 2014 (same start date as deconstruction phase). Note, the “lines” in tasks in schedule below refer to bay lines as depicted on the original CFHT architectural drawings.
Figure 10: Brace Upgrade Schedule
4. Foundations Assessment
The reappraisal of the footing and foundation of the ngCFHT Phase II work was performed by the UBC team. The objective of this work was to determine the capacity of the footing and foundation of the ngCFHT pier and enclosure by using recent design codes. This analysis was performed based on the ngCFHT Phase I work, the information, plans and reports from the original design. In conclusion it was found that no changes to the pier and enclosure footing and foundations are necessary. Please refer to Appendix A - ngCFHT Foundation Capacity Study for further details.

5. Manufacturing & Construction Plan
The proposed ngCFHT telescope and enclosure concept was presented by DSL and UBC in a previous study report titled “ngCFHT Telescope and Enclosure Configuration and Outer Pier Capacity Study” in January 2012. The proposed enclosure will have a calotte dome configuration, similar to TMT. Figure 11 shows views of the proposed enclosure on top of the existing enclosure pier along with names of the main enclosure components. The red line in Figure 11 represents the approximate division between vent and base structures. Figure 12 shows a cross-section view of the proposed enclosure and telescope inside.

Figure 11: Proposed ngCFHT Enclosure Components
Figure 12: Cross-section of Proposed Telescope & Enclosure

The proposed telescope design is similar to Keck II. Figure 13 shows side and isometric views of the proposed telescope in a rough 3D model. Various telescope components referred to in the installation sequence are labeled in the following figure.
5.1 Manufacturing Cost Estimate

Since the enclosure and telescope are at a very early conceptual level, the cost estimate is based largely on scaling from existing information, primarily the DSL estimate for TMT which provides the most recent estimating data for manufacturing costs for large optical telescopes. The TMT estimate itself utilizes data from previous DSL projects including the Gemini enclosures and the Keck enclosure and mount. Based on the cost estimating methodology the estimates are rough-order-of-magnitude at this point. The cost estimate below is based on Canadian Ironworkers Union manufacturing rates. For the telescope, trial assembly is assumed to include a factory trial assembly of the structural components, and does not include the dummy masses.
Table 3: ngCFHT enclosure manufacturing estimate

<table>
<thead>
<tr>
<th>Enclosure Manufacturing</th>
<th>Amount</th>
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<td>PM, Engineering, DO, Travel</td>
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<td>Superstructure</td>
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<td>Cladding</td>
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<td>Insulation</td>
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<td>Azimuth mechanical</td>
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<td>Cap/base interface mechanical</td>
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<td>Shutter structural/mechanical</td>
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<td>Ventilation doors</td>
<td>$507,564</td>
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<td>Walkways, cranes</td>
<td>$765,674</td>
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<td>Electrical &amp; control</td>
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<td>Shipping</td>
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<td><strong>Subtotal</strong></td>
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<td>Mark-Up (15%)</td>
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<td>Contingency (20%)</td>
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<td><strong>TOTAL</strong></td>
<td><strong>$11,979,880</strong></td>
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Table 4: ngCFHT telescope manufacturing estimate

<table>
<thead>
<tr>
<th>Telescope Manufacturing</th>
<th>Amount</th>
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<td>PM, Engineering, DO, Travel</td>
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<td>Azimuth Track</td>
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<td>Azimuth Structure</td>
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<td>Elevation Structure</td>
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<td>Trial Assembly</td>
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<td>Shipping</td>
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<td>Contingency (20%)</td>
<td>$2,453,283</td>
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<td><strong>TOTAL</strong></td>
<td><strong>$14,719,699</strong></td>
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5.2 Construction Plan

Construction of the ngCFHT is broken down into two phases: enclosure and telescope. Phase I: enclosure construction is assumed to start immediately after deconstruction of the existing enclosure and telescope. Phase II: telescope construction would ideally start after the enclosure is complete. Telescope installation may have a slight overlap with enclosure installation, since once the enclosure is secure, the telescope work could begin. Further discussion on construction schedules will be presented in subsection 5.2.4. The 250 ton
A hydraulic crane used for deconstruction of the existing telescope will be used for erecting the ngCFHT. The installation sequence for the new calotte enclosure will be very similar to the sequence used on TMT. The following subsections summarize the main steps involving the construction of ngCFHT enclosure and telescope respectively.

5.2.1 Construction Assumptions

The following assumptions have been taken for enclosure construction:

1. The existing CFHT azimuth girder, bogies and drives are not re-used for the proposed ngCFHT enclosure and a new system is installed (this assumption can be revisited during future design and development work as it may provide cost savings).
2. Proposed enclosure has vent modules.
3. The crane remains in one location with the enclosure rotating during construction. Note this is the same 250ton crane using during deconstruction phase.
4. Falsework towers are designed to rotate with new enclosure structure during construction.

The following assumptions have been taken for telescope construction:

1. Telescope parts are fed through the aperture by the 250 ton hydraulic crane.
2. Telescope is constructed after enclosure is complete or substantially complete.
3. The telescope would be installed using special falsework and a combination of a mobile gantry crane and a dome mounted crane.
4. The telescope is constructed while pointing to zenith.

5.2.2 Enclosure Construction

The following summarizes the main steps for enclosure construction:

1. Prep summit site.
2. Prep pre-assembly site.
3. Start pre-assembly of vent, base and cap shell modules at the pre-assembly site at lower elevation. Shell modules are pre-assembled with skin plates and insulation. This is ongoing through most of the enclosure install phase, with the pre-assembly site trucking shell modules up to the summit.
4. Erect falsework towers.
5. Prep existing azimuth rail and bogies, ensure azimuth ring is set horizontal at correct elevation and get ready for vent module installation.
6. Install azimuth drives.
7. Erect ventilation modules and brace back into falsework.
8. Survey and align ventilation modules.
10. Erect base shell modules and brace back into falsework.
11. Align base ring girder and bogies.
12. Erect cap ring girder and align. Drive components installation is included in this step.
13. Erect shutter support structure and align.
14. Erect cap shell modules and brace back into falsework.
15. Erect shutter.
16. Perform overall survey of enclosure geometry and adjust as required.
17. Weld infill skin plates between pre-assembled modules.
18. Install infill insulation between pre-assembled modules.
19. Release enclosure from falsework towers.
20. Remove falsework towers.
21. Lower enclosure onto azimuth ring bogies.
22. Install seals.
23. Paint touch-up welded areas.
24. Install control system and electrical.

It should be noted that some of the steps above may be done in parallel as will be illustrated in the construction schedule in subsequent section.

5.2.3 Telescope Construction

The following summarizes the main steps for telescope construction:

1. Erect azimuth track sections and level.
2. Erect azimuth bogies/bearings.
3. Erect azimuth structure falsework.
4. Erect azimuth structures (both sides).
5. Erect mirror cell falsework.
6. Erect mirror cell structure.
7. Erect lower tube structure.
8. Survey and align mirror cell structure.
9. Survey and align lower tube structure.
10. Erect elevation journals.
11. Survey and align elevation journals.
12. Erect hex ring girders.
13. Survey and align hex ring structure.
15. Survey and align upper tube members.
16. Erect spider ties on top of upper tube structure.
17. Install the prime focus unit support.
18. Check overall telescope geometry specially the mirror cell structure. Make adjustments as required.
19. Install hydraulic services (piping, power unit, etc)
20. Install drives, encoders and brakes.
22. Remove azimuth structure falsework.
23. Remove mirror cell falsework and transfer telescope weight to azimuth structures and azimuth bearing.
24. Install dummy mirror segments using dome mounted crane.
25. Install controls and electrical equipment.
26. Commission all drive systems.
27. Perform telescope site acceptance testing.
28. Remove dummy mirror segments and install actual mirror segments using dome mounted crane.
5.2.4 Construction Cost Estimate & Schedule

A preliminary cost estimate was developed for the construction activities and scope assumptions described above. The following outlines the estimate methodology and exclusions.

- Construction labour:
  - Due to lack of more specific information about the new enclosure and telescope, most of the construction activities and hours were adopted from detailed estimates done for TMT. For most cases, hours were scaled down by factor of 0.25 to convert from TMT hours to ngCFHT estimate. The new CFHT dome radius is approximately half of TMT’s. Since typically costs are related to dome surface area which is a function of radius squared, the scale factor of 0.25 was derived to arrive at most of the labor hours. It should be noted that TMT estimated hours already included a labor efficiency factor of 1.6 to account for effects of high altitude.
  - Supervision & crane operators: includes import DSL supervision (superintendent + site engineer), local foreman, mobile crane operator, and oiler
  - Telescope & enclosure labour assumes local (Hawaiian) labour is used for all tasks described above. Labour rates are based on local union rates and a 54hr working week (6 days x 9hrs/day) with overtime and travel time applied per union agreements
  - Live-out and travel: includes daily live-out allowance, air travel, and travel time for shift turn-around for both import and local workers
  - Worker’s compensation insurance is included in the estimate

- Construction equipment:
  - Large equipment: includes rental of 250t mobile crane, 25t rough-terrain crane, 2 x aerial boom lifts, and tractor trailer unit. Freight, mobilization and de-mobilization costs were included in the deconstruction phase. Therefore, these were not included. Fuel has been included for this equipment.
  - Misc. equipment, tools & falsework: includes small equipment such as tools, compressors, welding equipment, consumables, temporary power distribution, and scaffolding. This also includes 190 tonnes of falsework, including material, labour, and shipping costs.
  - Ground transport & trucking: these items were included in the deconstruction estimate. Only additional fuel and maintenance costs were accounted for the construction phase.

- Contingency: a suggested overall contingency of 20% is applied to the cost.

- Exclusions
  - Crane access ways (compacted, level roadways suitable for moving large cranes)
  - Water supply, toilets, and first aid facilities
  - See also scope assumptions and exclusions in sections 3.2.1 and above.
  - Permits

The cost estimate summary is given in the table below. Costs are given in 2012 US dollars.
Table 5: ngCFHT enclosure & telescope construction estimate

<table>
<thead>
<tr>
<th></th>
<th>Enclosure Labour</th>
<th>Telescope Labour</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Supervision &amp; crane operators $1,416,987</td>
<td>Supervision &amp; crane operators $1,307,988</td>
</tr>
<tr>
<td></td>
<td>Live-out &amp; travel $685,994</td>
<td>Live-out &amp; travel $534,064</td>
</tr>
<tr>
<td></td>
<td>Enclosure labour $4,855,082</td>
<td>Enclosure labour $2,392,314</td>
</tr>
<tr>
<td></td>
<td>Shipping $979,697</td>
<td>Shipping $2,174,545</td>
</tr>
<tr>
<td></td>
<td>Insurance $1,449,846</td>
<td>Insurance $1,097,082</td>
</tr>
<tr>
<td><strong>Total Enclosure</strong></td>
<td><strong>$9,387,605</strong></td>
<td><strong>Total Telescope</strong> $7,505,993</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Construction Equipment</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Large equipment $4,093,665</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Misc. equipment, tools &amp; falsework $2,193,673</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Ground transport &amp; trucking $152,800</td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>Total equipment</strong> $6,440,138</td>
<td></td>
</tr>
<tr>
<td><strong>Subtotal</strong></td>
<td><strong>$23,333,736</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Mark-Up (15%) $3,500,060</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Contingency (20%) $5,366,759</td>
<td></td>
</tr>
<tr>
<td><strong>TOTAL</strong></td>
<td><strong>$32,200,556</strong></td>
<td></td>
</tr>
</tbody>
</table>

Enclosure construction was estimated to take approximately 12 months with a crew of 14 and 3 supervisors. Telescope construction was estimated to take approximately 12 months from start to finish with a crew of 10 and 3 supervisors. A preliminary schedule for the construction of ngCFHT enclosure is shown below, assuming a nominal start date of September 1, 2014. No explicit schedule contingency has been applied.
A preliminary schedule for the construction of ngCFHT telescope is shown below, assuming a nominal start date of September 1, 2015.
<table>
<thead>
<tr>
<th>ID</th>
<th>Task Name</th>
<th>Duration</th>
</tr>
</thead>
<tbody>
<tr>
<td>102</td>
<td>Install hydraulic test for elevation bearings</td>
<td>1 day</td>
</tr>
<tr>
<td>103</td>
<td>Instability test</td>
<td>3 days</td>
</tr>
<tr>
<td>104</td>
<td>Adjust cable wraps with partial cable load</td>
<td>3 days</td>
</tr>
<tr>
<td>105</td>
<td>Dummy Masses</td>
<td>24 days</td>
</tr>
<tr>
<td>106</td>
<td>Install primary mirror segment dummy masses</td>
<td>21 days</td>
</tr>
<tr>
<td>107</td>
<td>Installability dummy mass</td>
<td>1 day</td>
</tr>
<tr>
<td>108</td>
<td>Install mirror dummy mass</td>
<td>2 days</td>
</tr>
<tr>
<td>109</td>
<td>Drive System Commissioning</td>
<td>42 days</td>
</tr>
<tr>
<td>110</td>
<td>Safety Check</td>
<td>8 days</td>
</tr>
<tr>
<td>111</td>
<td>Test of supply system</td>
<td>2 days</td>
</tr>
<tr>
<td>112</td>
<td>Anchor elevation topend structure with winch</td>
<td>2 days</td>
</tr>
<tr>
<td>113</td>
<td>Static test elevation and armouth bearings</td>
<td>5 days</td>
</tr>
<tr>
<td>114</td>
<td>Static test elevation and armouth break systems</td>
<td>2 days</td>
</tr>
<tr>
<td>115</td>
<td>Test azimuth drive system</td>
<td>10 days</td>
</tr>
<tr>
<td>116</td>
<td>Check clearances and adjust alignment</td>
<td>6 days</td>
</tr>
<tr>
<td>117</td>
<td>Set up measuring equipment</td>
<td>3 days</td>
</tr>
<tr>
<td>118</td>
<td>Apply pressure to azimuth bearings</td>
<td>1 day</td>
</tr>
<tr>
<td>119</td>
<td>Rotate azimuth structure with winch</td>
<td>2 days</td>
</tr>
<tr>
<td>120</td>
<td>Check clearances and adjust alignments</td>
<td>5 days</td>
</tr>
<tr>
<td>121</td>
<td>Test azimuth drive system</td>
<td>6 days</td>
</tr>
<tr>
<td>122</td>
<td>Cornea balance elevation structure</td>
<td>15 days</td>
</tr>
<tr>
<td>123</td>
<td>Set up measuring equipment</td>
<td>3 days</td>
</tr>
<tr>
<td>124</td>
<td>Apply pressure to elevation bearings</td>
<td>3 days</td>
</tr>
<tr>
<td>125</td>
<td>Apply forces to top end structure on winch</td>
<td>2 days</td>
</tr>
<tr>
<td>126</td>
<td>Install elevation structure to horizontal winch</td>
<td>2 days</td>
</tr>
<tr>
<td>127</td>
<td>Check clearances and adjust alignments</td>
<td>3 days</td>
</tr>
<tr>
<td>128</td>
<td>Telescope acceptance testing</td>
<td>10 days</td>
</tr>
<tr>
<td>129</td>
<td>Perform dimensional alignment</td>
<td>6 days</td>
</tr>
<tr>
<td>130</td>
<td>Critical path alignments</td>
<td>2 days</td>
</tr>
<tr>
<td>131</td>
<td>Masking</td>
<td>2 days</td>
</tr>
<tr>
<td>132</td>
<td>Mount control system functional and performance tests</td>
<td>5 days</td>
</tr>
<tr>
<td>133</td>
<td>Range of motion tests</td>
<td>2 days</td>
</tr>
<tr>
<td>134</td>
<td>Limit weight and check functional tests</td>
<td>2 days</td>
</tr>
<tr>
<td>135</td>
<td>Breaks performance test</td>
<td>1 day</td>
</tr>
<tr>
<td>136</td>
<td>Drive and encoder performance tests</td>
<td>2 days</td>
</tr>
<tr>
<td>137</td>
<td>Test segments handling cranes</td>
<td>2 days</td>
</tr>
<tr>
<td>138</td>
<td>Test utility services</td>
<td>2 days</td>
</tr>
<tr>
<td>139</td>
<td>Telescope installation complete</td>
<td>1 day</td>
</tr>
</tbody>
</table>
6. Conclusions

This report outlined a programmatic study for the upgrade of the telescope structure and enclosure for the CFHT observatory. This includes the deconstruction of the existing CFHT telescope and enclosure, reinforcement of the existing outer pier structure that supports the enclosure, and the manufacture, shipping, and construction of the Next Generation CFHT enclosure and telescope structure. Further investigation of the telescope pier indicates that with the present assumptions the pier foundation will not need any remedial work to support the new telescope structure to current design codes. The next phases of design and development should focus on developing conceptual and preliminary designs for the enclosure and telescope structures.

A cost summary for the overall deconstruction, upgrade, and manufacture and construction of the enclosure and telescope structures is given in the table below. The costs shown include contingencies as outlined in the more detailed costing tables in this report.

Table 6: ngCFHT cost and schedule summary for enclosure and telescope structure

<table>
<thead>
<tr>
<th>PHASE</th>
<th>Cost</th>
<th>Year 1</th>
<th>Year 2</th>
<th>Year 3</th>
<th>Year 4</th>
<th>Year 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design - Enclosure</td>
<td>$11,979,880</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Manufacture - Enclosure</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Design - Telescope</td>
<td>$14,719,699</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Manufacture - Telescope</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deconstruction - Telescope</td>
<td>$6,602,690</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deconstruction - Enclosure</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Outer Pier Upgrade</td>
<td>$2,688,556</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Construction - Enclosure</td>
<td>$32,200,556</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Construction - Telescope</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>$68,191,381</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
APPENDIX A: ngCFHT Foundation Capacity Study (Phase II)

By Drs. Wu Di, Stiemer, Liu, and Mr. Angers

University of British Columbia
October 24rd 2012
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1. **Executive Summary**

This report presents a reappraisal study on the Next Generation Canada-France-Hawaii Telescope (ngCFHT). In Phase I of the ngCFHT work, the telescope concrete pier and the enclosure capacity were studied by the UBC team while the development of the telescope properties and azimuth track were studied by DSL.

In this report, the reappraisal of the footing and foundation of the ngCFHT Phase II work was performed by the UBC team. The objective of this report is to determine the capacity of the footing and foundation of the ngCFHT pier and enclosure by using recent design codes. This analysis was performed based on the ngCFHT Phase I work, the information, plans and reports from the original design.

1.1 **Findings in Summary**

1.1.1 **Pier Footing and Foundation Evaluation**
- The differential settlements were evaluated to be less than 10.0 mm and satisfactory.
- The soil allowable bearing capacity under gravity loads is sufficient as the estimated pressure induced by the footing was equal to the capacity.
- The bearing capacity of the soil under earthquake loads is exceeded by 33%. Since the structure having been designed in 1974, the soil capacity of existed building can increase over time. Also because of the dynamic nature of these loadings, it is common practice to increase the capacity by one-third. This allows considering the capacity as sufficient.

1.1.2 **Enclosure Footing and Foundation Evaluation**

The larger dome inducing higher forces to the enclosure and the new code requirements justify assessing 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. The load cases and combinations were defined using the ASCE-7 2010 requirements. The capacity of the footing and foundation were evaluated. The following points were concluded:
- 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.

1.2 **Conclusion**

No changes to the pier and enclosure footing and foundations are necessary. No cost and schedule estimates needed.
Figure shows the finished pier building with the enclosure steel frame walls during construction.

Figure 1: CFHT pier building during the dome support construction (CFHT, 1974, with permission)
2. Methodology and assumptions
The following steps were pursued to successfully completed during 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. Footing
   b. Foundation (Deflections)

3. Pier Footing and Foundation Evaluation

3.1 Applicable Code
The loads and design requirements are defined by the International Building Code (IBC, 2006) and American Society of Civil Engineers 7 - Minimum Design Loads of Buildings and Other Structures (ASCE-7, 2005) which are the regulations in place in Hawaii. These forces were compared to the capacities of the different components that were evaluated according to the American Concrete Institute code (ACI, 2008). The evaluation of the structure and footing was done using limit state design (LSD), and the soil foundation was assessed using allowable stress design (ASD). The LSD approach insures that the different limit states are respected, for example the bending capacity of a beam, assuming a certain probability of rupture. The security associated with the limit state is dependent on the variability of the resistance and of the loads. Factors are applied to the loads and capacities to achieve that goal. The ASD philosophy is to make sure the service loads are under the elastic limit. This limit is usually reduced by a factor of safety, which is usually three (3) for foundation design.

3.2 Geometry
The pier building is a three storey reinforced concrete cylindrical pier structure. It has a 16.3 m diameter and is 14.4 m high. The walls are 304.8 mm thick over its whole height. The slabs of the first and second storey are hollow slab and 711 mm thick. The voids in the slabs are rectangular and have 914 x 914 x 356 mm dimensions. The top slab is a 304.8 mm thick slab. The first storey is 6.3 m high with an opening of 5.8 m wide and 3.2 m high. The second and third storeys are 4.0 m high and have three openings of 1020 x 2080 mm and one opening of 1800 x 2080 mm each. The foundation is a ring footing of 610 mm thickness and 2240 mm width. More details on the bar sizes and spacing are provided in the plans (CFHT, 1974).

3.3 Materials
Concrete compressive strength (\(f'_C\)) is 20.7 MPa and the elasticity modulus (\(E_c\)) was evaluated to be 21525 MPa according to the ACI (2008) with equation 1. Equation 1 has to be used with imperial units.

\[
E_c = \frac{57000 \sqrt{f'_C}}{}
\]
The reinforcing bars have a yielding strength of 413 MPa and an elasticity modulus of 200 000 MPa.

### 3.4 Soil

Soil data was taken from the Foundation Investigation Report prepared by Dames & Moore (1973). The evaluated maximum soil pressure capacity was of 191 kPa. The Design Criteria & Basis of Calculations for Concrete Telescope Support states that “Dames & Moore believes the maximum safe bearing pressure under the central pier slab on unfortified soil to be 4000 psf (191 kPa), from the standpoint of bearing capacity and differential settlement of less than 10.0 mm”.

Also, the report states that the water level is much below the surface. The footing top is located 2.5 m below the soil surface. The soil under the foundation consists of “sand and gravel size volcanic ash and cinders with occasional clinkers up to 152 mm. The ash is similar to furnace slag.” (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 1800 kg/m$^3$ was assumed for simplicity.

### 3.5 Loads

ASCE-7 (2005) defines the load cases and combinations to be considered for the design of a new structure; these were used as the loads that have to be resisted by the existing structure.

**Structural Weights**

In order to assess the pier building, details on the new telescope were required. At this stage, only a simple telescope model was developed as a steel frame idealization. The telescope steel frame idealization was modeled following the recommendation by Gedig (EDS, 2011). The proposed model is shown in Figure . The location of the telescope center of gravity (H) is 7.0 meter over the top slab of the pier structure. The radius is that of the pier building and is equal to 8.15 m. The mass of the telescope (M) is approximated to 270 000 kg compared to 255 000 kg for the old telescope. The mass is attached to the pier via truss steel frame elements forming a pyramidal structure. This pyramidal frame is supported at four locations on hydraulic bearings to allow rotation of the telescope around its vertical axis. The four bearings are spaced equally at a distance (B) of 11.53 m. The bearings are idealized as linear springs with radial stiffness ($K_r$), tangential stiffness ($K_t$) and vertical stiffness ($K_z$). The bearings are sliding on the azimuth track that is itself supported by the ring girder. The ring girder is placed on the pier wall perimeter. The actual design of these components is beyond the scope of this project and will be done in further studies.
3.5.1 Load Cases

It was considered that the loads to which the structure is submitted are the dead, live, and seismic loads. Wind loads were ignored because the enclosure covering the pier is isolated from the pier. The dead load includes the self weight of the structure and telescope mass. The live loads are the equipment and people loads. The live load values were taken from the document “Design Criteria & Basis of Calculations for Concrete Telescope Support” furnished by the CFHT.

Structures are subjected to different types of loadings. These loads vary more or less in time. If they are not varying excessively over time they can be considered as static, like dead loads. But, in certain cases, this assumption may not be possible or realistic. It is the case of wind, pedestrian or earthquake loadings that are dynamic loads. The response of the structure is then varying with time. The displacement, velocity and acceleration of the structure are the parameters that need to be evaluated. A structure can be analyzed either as a single degree of freedom (SDOF) or multi degree of freedom (MDOF) system.

Particular attention was paid to seismic analysis since the Mauna Kea is located in a high seismic zone and that older codes are not as severe regarding seismic requirements. For that reason, the seismic analysis is described in details in this section.
3.5.2 Load Combinations

For the evaluation of the pier walls, slabs, and footings, the limit state design procedure is used. The dead load (D), live load (L) and earthquake load (E) are factored and then combined according to the ASCE-7 (2005) requirements.

\[
\begin{align*}
1.4D & \\
1.2D + 1.6L & \\
(0.9 - 0.2)D + 1.0E & \\
(1.2 + 0.2D) + 1.0E & \\
\end{align*}
\]

(2) (3) (4) (5)

The combinations that include earthquake loads have a portion of their dead load added or removed to account for vertical vibration. The seismic loads were applied with different orientations to the structure. The forces, reactions and displacements were evaluated assuming the structure to remain in its elastic range. The forces induced by these factored loads were compared to the factored capacities.

The design of the foundation was realized using allowable stress design. Different load combinations were used and vertical vibration could be neglected as stated in the ASCE-7 (2005):

\[
\begin{align*}
1.0D & \\
1.0D + 1.0L & \\
1.0D + 0.525E + 0.75L & \\
1.0D + 1.0E & \\
0.6D + 0.7E & \\
\end{align*}
\]

(6) (7) (8) (9) (10)

The pressure induced by these loads under the footing was compared to the bearing capacity to which a safety factor was applied.

3.6 Evaluation

3.6.1 Footing

The structural capacity of the footings also needed to be assessed. The shear and flexural capacities have to exceed the forces at the supports due to the different load combinations. The evaluated foundation cross section is shown in the Figure. The footing of the pier structure was assumed to be a continuous footing. The footing was assumed to be linear between each node where the forces were evaluated. The capacity was evaluated at each of these nodes.
The vertical shear capacity of the footing was first evaluated. Only one-way shear needs to be checked for a continuous footing. Equation 41 has to be met in order to have a satisfactory design.

\[ V_u \leq \varphi V_n \]  \hspace{1cm} (11)

The factored shear force on critical shear surface \( V_u \) is described in Equation 42 (Coduto, 2001).

\[ V_u = P_u \left( \frac{B - c - 2d}{B} \right) \]  \hspace{1cm} (12)

Where \( P_u \) is the factored applied compressive load, \( c \) the width of the wall, \( B \) is the width of the footing and \( d \) is the effective depth.

The capacity \( (V_n) \) is the summation of the shear load capacity of concrete \( (V_c) \) and of steel \( (V_s) \). Since there are no ties contributing to the shear capacity, the total shear capacity is that of concrete. The shear capacity is the capacity of concrete, and according to Coduto (2001) the shear capacity in a footing is equal to:

\[ V_u = V_c = \frac{1}{6} 2b d a \sqrt{f'_c} \]  \hspace{1cm} (13)

Where \( b \) is the length of critical shear surface. The results indicate that the shear capacity is high enough and would resist the forces induced by all load combinations. The maximum ratio of factored shear force over factored capacity was found to be 0.48.

It was also required to evaluate if the flexure design of the footing meets the requirements. First, longitudinal steel, which are the bars parallel to the wall length, should be present in sufficient quantity in the footing to resist flexural stresses from non-uniform loading and soft spots in the soil. Also, longitudinal steel should be present in sufficient amount to resist temperature and shrinkage stresses (Coduto, 2001). The minimum ratio
of steel that should be present to resist these constraints is 0.002A_{g}. This criterion was found to be respected for the pier building footing since the amount of steel is 0.0026A_{g}.

Transverse steel should also be assessed. The required ratios of reinforcement (A_{g}) were evaluated for each load combination at each section. The next equations (Equations 44 and 45) from Coduto (2001) can be used.

\[
A_{g} = \left( \frac{f_{cb} b}{1.17 f_{y}} \right) \left( d - \sqrt{d^2 - 2.353 M_{uc} \varphi f_{cb}} \right)
\]

\[
M_{uc} = b \left( \frac{R_{b}}{2B} + \frac{2 M_{uc}}{B \phi} \right)
\]

Where \( M_{uc} \) is the factored moment at critical section, \( M_{a} \) is the factored applied moment load perpendicular to wall, \( \varphi \) is equal to 0.9 and \( l \) is the distance from edge of the wall to the edge of the footing. These ratios were all found to be smaller than the steel ratios present in the structure. The footing was therefore considered to have enough structural shear and flexural capacity.

### 3.6.2 Foundation

#### 3.6.2.1 Settlements

The “Final Investigation Report” by Dames & Moore (1974) states that the differential settlements should be kept to less than 10 mm. The settlements were evaluated using the plate bearing deflection curves. The plate load test consists of applying a loading at the height of the footing on a square steel plate to get the in-situ load-settlement data (Coduto, 2001). This method is not proven to be reliable because of the plate size that is much smaller than the foundation dimension. The depth under which the plate settles is smaller than for the real foundation and is only accounting for the soil close to the plate. Failures have been observed using this method (Terzaghi and Peck, 1967). But, because not much information on the soil properties was available to allow the use of more up-to-date methods, the plate load test results were used. The results therefore have to be considered with care.

Figure shows the curves of two tests realized at the site. For simplicity the deformations were assumed to be elastic. The pressures from the finite element model for the load case D + L were used. The maximum pressure around the ring footing was found to be 194 kPa and the minimum pressure, 153 kPa. To obtain conservative results, the maximum deflection was evaluated with the steepest curve (Test #2 on Figure ) and the minimum deflection was calculated with the least steep curve (Test #1 on Figure ). The maximum deflection was approximately 4.1 mm and the minimum, 0.5 mm. This gave a differential settlement of 3.6 mm. Using this method, a pressure of approximately 343 MPa would be required to achieve a 10 mm settlement.
It can be observed on Figure 4 that there is significant variability in the results of both tests. This suggests that more testing would be required.

As stated earlier, even if the plate load test can lead to unconservative results, the maximum differential settlement was found to be 36% of the allowable one, which can be considered acceptable.

3.6.2.2 Soil Capacity

The allowable stress design load combinations applied to the pier building induce forces at the base of the structure. These forces are resisted by the footings that distribute them to the soil. A bigger area of foundation results in a lower pressure applied to the soil. This bearing pressure induces compressive and shear stresses in the soil. When the shear stresses are high enough, they may exceed the shear strength of the soil, which is called a bearing failure (Coduto, 2001). Generally, three types of failure can occur: general...
shear failure, local shear failure and punching shear failure. For shallow foundation, it is generally only necessary to check for general shear failure. See Figure for a representation of a general shear failure.

**Figure 5: General shear failure of a shallow foundation**

The pressure distributed by the footing at each node of the modeled structure was compared to the capacity. Different methods are available to evaluate this requirement and are described in Coduto (1999). The bearing capacity ($q_{\text{cap}}$) taken from Dames & Moore “Investigation soil report” was of 191 kPa.

Both vertical force ($P$) and moment ($M$) present at the support of the structure induce pressure to the soil. The moment is transformed into a force being applied with an eccentricity ($e$), and an equivalent pressure ($q_{\text{equiv}}$) can be calculated and compared to the bearing capacity. A quick way to account for this is to use an effective footing width $B'$ (Coduto, 2001). The procedure to obtain the equivalent bearing pressure is described in Equations 46 to 48.

$$q_{\text{equiv}} = \frac{P + W_f}{B'D} - u_d < q_{\text{cap}}$$  \hspace{1cm} (16)

$$B' = B - 2e$$  \hspace{1cm} (17)

$$e = \frac{M}{P + W_f}$$  \hspace{1cm} (18)

Where $L$ is the length of the footing, $W_f$ the weight of the foundation, $u_d$ the pore water pressure ($u_d$ is 0 if at a greater distance than the height of the surface to the bottom of the footing. The eccentricity has to be smaller than $B/6$ to prevent lifting of the footing.
The calculations suggest that uplift of the foundation would be avoided for every load case. For the load combinations D and D + L, the maximum ratios of pressure over capacity were 0.91 and 1.02, respectively. For the load combinations including earthquake loads, the maximum pressure ratio was 1.33. Coduto (2001) states that geotechnical engineers usually increase the bearing capacity of soils by 33% for earthquake load combinations. This is allowed for four reasons (Coduto, 2001): 1) The shear strength of soils under dynamic loading is higher than during static loading resulting in a greater bearing capacity, 2) Lower factor of safety can be tolerated because earthquake are rare events, 3) Under dynamic loading, settlements are generally smaller and, 4) Larger settlements can be tolerated under rare events because population can accept more visible damage. Because of the variability of the soils, not every soil type will present that type of behavior. Increasing the bearing capacity by 33% is not recommended anymore in recent codes.

Our estimates suggest that the soil under dead and live load can bear the pressure distributed by the footings. Considering that there is an important safety factor for the bearing capacity, this is acceptable. For earthquake combinations, if a 33% increase in the bearing capacity is used, the design could be considered as safe. However, advice from geotechnical experts having studied the specific site conditions would be required.

If eventually the capacity is found to be insufficient, different solutions can be considered. An easy way to decrease the pressure induced by the footing would be to increase its width. In the case of the pier building, because of the presence of the dome structure around the pier, it would only be possible to increase the footing width inside the pier. Because of the restrained space and equipment in place, this solution may be difficult to implement.

An alternate solution would be to reinforce the soil bearing capacity with post-grouting piles through drilled holes. This consists in drilling holes through the footing and soil, and to insert steel pipes filled with grouting. The problem with this option is again the restrained space to drill and the fact that the bearing stratum may be far away from the surface. Other avenues could be studied to reinforce the soil.

It should be noted that Dames & Moore (1973) recommended that the footing should be at least 3.0 m wide and that the soil should have been strengthen with cement grouting under the foundation in drilled holes. To our knowledge, the present footing is 2.24 m and the soil has not been reinforced. This may explain why the capacity is too low under earthquake load combinations.

### 3.7 Conclusions

A preliminary version of a telescope frame was designed and placed on top of the pier that represents the telescope static and dynamic behavior. The analysis was performed using the International Building Code and the American Concrete Institute code, and the seismic analysis was completed using an equivalent lateral force method. The foundation capacity and soil capacity were evaluated. They were then compared to the forces and steel in place. The most significant results are summarized in Table 1:

**Table 1:** CFHT pier building evaluation results summary
### Demand Capacity Ratio force vs capacity

<table>
<thead>
<tr>
<th></th>
<th>Demand</th>
<th>Capacity</th>
<th>Ratio force vs capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Footing bending structural capacity</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Required steel ratio</td>
<td>0.2%</td>
<td>0.27%</td>
<td>0.74</td>
</tr>
<tr>
<td><strong>Settlements – Plate load test evaluation</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravity load combination</td>
<td>3.6 mm</td>
<td>10.0 mm</td>
<td>0.36</td>
</tr>
<tr>
<td><strong>Foundation allowable pressure</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Earthquake load combination</td>
<td>254 kPa</td>
<td>191 kPa</td>
<td>1.33</td>
</tr>
<tr>
<td>Gravity load combination</td>
<td>195 kPa</td>
<td>191 kPa</td>
<td>1.02</td>
</tr>
</tbody>
</table>

The following points were concluded:

- The differential settlements were evaluated to be less than 10.0 mm and satisfactory.
- The soil allowable bearing capacity under gravity loads is sufficient as the estimated pressure induced by the footing was equal to the capacity.
- The bearing capacity of the soil under earthquake loads is exceeded by 33%. Since the structure having been designed in 1974, the soil capacity of existed building can increase over time. Also because of the dynamic nature of these loadings, it is common practice to increase the capacity by one-third. This allows considering the capacity as sufficient.

### 4. Enclosure Footing and Foundation Evaluation

#### 4.1 Applicable Code

The loads are defined by the American Society of Civil Engineers 7 - Minimum Design Loads of Buildings and Other Structures (ASCE 7) 2010. 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).

#### 4.2 Geometry

The outer pier covers the inner concrete pier building supporting the telescope. The enclosure 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.

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. The main opening of the enclosure is located at the ground level between columns 1 and 12.

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.

Table 2: Dome modeling parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<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>

4.3 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.

4.4 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.

4.5 Loads
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.

4.5.1 Load Cases
Dead load
Dead load is the structural mass of the outer pier, which includes the total weight of the construction materials. 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 3 presents the dead load applied on each floor.

Table 3: 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>
</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 4: 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>
</tr>
</thead>
<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 1 409 240 kg.

### 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 5 summarizes the live loads per floor.

### Table 5: 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>

### 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. The dome also has to resist ice loads on half of its surface. The dome loads are summarized in the following table.

### Table 6: 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>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>8824.644</td>
</tr>
</tbody>
</table>
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 7: 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>

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 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 considered according to the ASCE-7 2010, cases A and B.

Table 8: Lateral wind loads cases

<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>
<tr>
<td>8</td>
<td>-15.09</td>
<td>-15.09</td>
</tr>
<tr>
<td>9</td>
<td>-21.15</td>
<td>-21.15</td>
</tr>
<tr>
<td>10</td>
<td>-17.87</td>
<td>-15.43</td>
</tr>
<tr>
<td>11</td>
<td>-5.23</td>
<td>2.05</td>
</tr>
<tr>
<td>12</td>
<td>7.40</td>
<td>13.72</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 9 presents the total lateral and vertical wind loads applied to the dome top.
Table 9: 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</td>
<td>0</td>
<td>1718</td>
</tr>
</tbody>
</table>

**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 of 0.5 seconds. 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. The forces at each floor are presented in Table 10.

Table 10: Lateral 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>

**4.5.2 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 factor is applied to the dome dead load since its mass has more precision and that it is wish to evaluate the capacity with that precise mass. 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, L the live load, S the snow load, I the ice load, W the wind load and E the 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.

4.6 Evaluation

Therefore, the footings and foundation have to be assessed. This study presents the design checks and results for each of the member types as well as the dynamic properties and the deflections under earthquake loads.

4.6.1 Footing

The structural capacity of the concrete footing is verified. It is assumed that only axial loads are transmitted to the footing and that the diagonal columns do not induces 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 to ratio for each ultimate limit state.

**Table 11: Structural performance of the footing**

<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&lt;sub&gt;Dome&lt;/sub&gt;</td>
<td>0.24</td>
</tr>
<tr>
<td>Bending around radial axis</td>
<td>1.4D + 1.0E + 0.2S + 1.2D&lt;sub&gt;Dome&lt;/sub&gt;</td>
<td>0.49</td>
</tr>
<tr>
<td>Shear for tangential section</td>
<td>1.4D + 1.0E + 0.2S + 1.2D&lt;sub&gt;Dome&lt;/sub&gt;</td>
<td>0.44</td>
</tr>
<tr>
<td>Shear for radial section</td>
<td>1.4D + 1.0E + 0.2S + 1.2D&lt;sub&gt;Dome&lt;/sub&gt;</td>
<td>0.33</td>
</tr>
<tr>
<td>Tension in hoops</td>
<td>1.2D + 1.6L + 0.5S + 0.2I + 1.0D&lt;sub&gt;Dome&lt;/sub&gt;</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.

### 4.6.2 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.

#### 4.6.2.1 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 12: Bearing capacity performance for earthquake and gravity loads**

<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&lt;sub&gt;Dome&lt;/sub&gt;</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.

#### 4.6.2.2 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...
The 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 13: Sliding capacity performance for earthquake loads**

<table>
<thead>
<tr>
<th>Demand to capacity ratio for $0.46D + 0.7E + 0.74D_{Dome}$</th>
<th>Demand to capacity ratio for $1.14D + 0.7E + 1.14D_{Dome}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.74</td>
<td>0.60</td>
</tr>
</tbody>
</table>

The load combination $0.46D + 0.7E + 0.74D_{Dome}$ results in a lower ratio because of the lower axial loads compared to the load combination $1.14D + 0.7E + 1.14D_{Dome}$. The sliding capacity of the structure is sufficient to resist the loads applied.

### 4.6.2.3 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. The deflections are checked for each level in X and Y directions in tables below.

**Table 14: 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 15: 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>
<tr>
<td>4</td>
<td>38.1</td>
<td>76.2</td>
<td>1.00</td>
<td>0.66%</td>
</tr>
<tr>
<td>Observation</td>
<td>41.8</td>
<td>83.5</td>
<td>-</td>
<td>0.24%</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 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 that the allowable space between the two structures.

### 4.7 Conclusions

The larger dome inducing higher forces to the enclosure and the new code requirements justify assessing 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. The load cases and combinations were defined using the ASCE-7 2010 requirements. The capacity of the footing and foundation were evaluated. The following points were concluded:

- 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

1. ACI 318-08. (2008). “Building Code Requirements for Structural Concrete and Commentary.” American Concrete Institute, Farmington Hills, Michigan, USA.