DESIGN FOR REUSE OF EXISTING BUILDINGS APPLIED TO THE CANADA-FRANCE-HAWAII-TELESCOPE STRUCTURE

by

MATHIEU ANGERS

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ABSTRACT

Reusing structures is a complex process. However, it can provide significant economic, social and environmental benefits. The reuse of structures requires their prior structural evaluation. This document presents a methodology to evaluate the possibility of reusing an existing structure through a case study of the Canada-France-Hawaii Telescope (CFHT), located in on Mauna Kea Volcano, Hawaii. The operators of the CFHT mandated a research group with members from the University of British Columbia and Empire Dynamic Structures to evaluate the possibility of having the current pier building support the mass and configuration of a proposed new telescope. The original pier was designed in 1974, and thus it was necessary to verify if the structure would meet current codes, particularly those of seismic requirements. The current telescope has a diameter of 3.6 m and the new design would be a 10 or 12 m instrument. The methodology was used to perform a structural evaluation of the CFHT pier building supporting the new CFHT telescope. From the analysis, the following points were concluded:

- The bending moment and shear capacities were found to be high enough to resist resulting forces of the proposed new structure. Required steel reinforcement in the walls and slabs of the pier building are comparable to those found in the current structure. They were judged to be sufficient for supporting the new telescope.
- The footing structural resistance was found to be satisfactory. Also, differential settlements were found to be under an acceptable level.
- The soil bearing capacity was evaluated by Dames & Moore (1973) to be 191 kPa. Under gravity loads, the pressure induced by the footings was considered to be satisfactory. However, in an earthquake condition, the design bearing capacity of the soil is commonly assumed to be 33% greater than in a static condition due to the dynamic nature of the loading. With this assumption, the capacity of the soils is found to be satisfactory. However, it is recommended that there be further geotechnical soil and foundation evaluation.

It is concluded the CFHT pier can be reused for the installation of the proposed new telescope.

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1 INTRODUCTION

The reuse of existing structures is of growing interest as it may be a good alternative to building new ones. It can offer several advantages such as economic and, environmental benefits, time gains, and address social or historical concerns. In North America, most of the present infrastructures were designed in the middle of the 20th century and are now exceeding or approaching their design life. In the case of building structures, their reuse poses a number of questions, such as: does the structural capacity meet the modern building codes, was the onsite control sufficient enough to insure an acceptable level of confidence in the construction, is their adequate soil testing and assessment, and has structural degradation caused a reduction of its performance? For these reasons, it may be necessary to proceed with an evaluation of the structural capacity and induced loads according to the modern building codes in order to allow reuse. When a structure does not meet modern building code requirements, rehabilitation or retrofit can be considered. Rehabilitation consists of giving back the original capacity to a structure, and retrofit includes the reinforcing of a structure to meet new requirements or loads that the original design did not consider.

1.1 Reusing of Structures

In this document, reusing of structures refers to the whole process involved in modifying an existing structure or its purpose. These modifications can be summarized in three different groups: change of occupancy of the structure, an addition to it, or its alteration.

1.1.1 Change of Occupancy

A change in occupancy would be a modification of what the structure was first intended for. This usually implies more or less significant changes in the design requirements or loads so that it meets the current building codes. Changes in occupancy can take several forms. A first example would be a commercial building that is intended to be transformed into a school. Another one would be the transformation of an industrial building into commercial and residential units (Cantell, 2005). A third case could be the preservation of older historical structures. Many of these cases may require rehabilitation and strengthening of the structural systems (ASCE-11, 1999).

1.1.2 Additions

An addition consists in a major structural modification of a building (ASCE-7, 2005). It will often result in adding storeys, raising the height of the building or increasing its floor area. These changes can modify the loads and the structural system.

1.1.3 Alterations

Alterations are referred to as changes that are not covered by additions (ASCE-7, 2005). Upgrading equipment or modifying the structural system to reuse a structure counts as an "alteration". Industry is also concerned with the evaluation of their infrastructures. For example, new and heavier equipment will induce higher forces in the structural systems (ASCE-11, 1999). Also, new precision equipment (for instance, a telescope) can have specific requirements for vibrations and deflections that need to be addressed. The high installation cost justifies the equipment not experiencing damage or excessive vibration during catastrophic events like an earthquake.

1.1.4 Structural Evaluation

Change of occupancy, additions and alterations of an existing structure usually imply changes in the loads and in the structural system. It is necessary that a structure with these types of modification meet the requirements of the building codes in effect at the time of the changes. This verification is done by performing a structural evaluation of the existing structure. A structural evaluation consists of verifying if the capacity of the different structure members is sufficient to resist the forces induced by the loads. When it is assessed that the capacity of the structure is not sufficient, a retrofit can be performed. If damage is found to have reduced the capacity of the structure to a point where it is not sufficient anymore, rehabilitation can be performed to restore it. Seismic requirements and damage due to environment are two cases that should be carefully considered when planning the reusing of a structure.

Modern building codes have much more severe requirements regarding seismic design than older ones (Bracci et al., 1997). The demand induced by seismic loads is much higher, and in regions of high seismicity, the capacity of a structure may not be sufficient enough. In this case, a retrofit can be performed.

With time, structures are subjected to environmental conditions that can degrade their structural integrity. For that reason, the evaluation of the structure has to take into account these damages. If it is found that the structure needs to have its capacity restored, rehabilitation can be undertaken. For instance, due to deicing products and salt present in the sea water, bridges in coastal and cold climates experience corrosion which can result in a reduction of the capacity of the structure. Exceptional damages such as bombing can also require an evaluation of the capacity and a rehabilitation of the structure (Vion and Deschamps, 2010).

Reusing (or recycling of) structures is often considered instead of a new building. However, its process can be complex and involves many steps. For that reason, a careful prior structural evaluation is required to support decision making.

1.2 Context and Purpose of the Study

This document presents a methodology developed to evaluate a structure when it is intended to be reused for a major modification. The work was motivated by a request from the Canada-France-Hawaii-Telescope (CFHT) observatory. The CFHT mandated the University of British Columbia (UBC) and Empire Dynamic Structures (EDS) to study the possibility of replacing the current instrument which is located on the Mauna Kea volcano, Hawaii, USA for a next generation telescope. Figure 1 presents a picture of the current installations. The structure supporting the telescope is a cylindrical concrete pier that has been designed in 1974.

The objective of the work was to evaluate if the concrete pier building, the footings and foundations can be reused for the next generation telescope. The pier building supporting the new telescope needs to meet the requirements of modern building codes. The study is based on the information, plans and reports from the original design. It is the first of a series that will define the baseline configuration of the telescope structure and enclosure system for the next generation CFHT instrument.

1.3 Outline

Chapter 2 presents a methodology describing each step of the evaluation that will lead to recommendations on the reuse of a structure. In Chapter 3 the evaluation of an existing structure through a case study is presented. The different steps of the evaluation are reviewed and described in detail as well as the assumptions and the recommendations. Chapter 4 presents the conclusions including future work.

Figure 1: Current CFHT facilities on Mauna Kea volcano, Hawaii (CFHT, 2009, with permission)

2 METHODOLOGY

The process leading to the decision of reusing an existing structure has to be well planned in order to make an optimal and efficient assessment. A methodology is proposed in this chapter to achieve this goal. The methodology is inspired from the recommendations of the ASCE-11 (1999) structural assessment code. The methodology was developed keeping in mind the specificity of the reusing of the CFHT pier building and should not be used for a professional evaluation.

2.1 Existing Structure Review

Reusing a structure involves modifications to the loads or structural system and requires an evaluation according to the up-to-date codes. The first step is to perform a review of the existing structural system. The structural system and soils parameters can be obtained from a study of the available documentation and from site inspection and site testing.

2.1.1 Documentation Review

First, all the documentation on the structure needs to be gathered. The structural plans are essential to determine the geometry, the sections of the members and the connection details. The geometry is fundamental to model the structure properly and to determine the loads. It is generally required to obtain the height of the structure, the storey heights, the bay lengths, and the diameter in the case of a cylindrical structure. In the case of concrete structures, the openings in the walls have to be noted and in the case of steel frames, the position of all members carefully obtained.

The material properties have to be assessed. Usually, for concrete structures, the strength and stiffness of the concrete as well as the yield strength, ultimate strength, stiffness of the reinforcement bars are necessary. For steel members, the yield and ultimate strengths as well as the stiffness should be documented.

The next step is to get the section of all members of the structure. For a concrete structure, the steel reinforcement ratio and positioning as well as the section geometry are needed for the walls, columns, beams and slabs. For steel structures, the section types have to be obtained. The connection details for both concrete and steel members have to be acquired. The footings sizes and reinforcement details are also needed.

The soil properties also have to be obtained to assess the footing structural capacity, the foundation and the settlements. Different parameters have to be acquired like the bearing capacity and stiffness of the soil. This information is usually found in soil reports.

Design notes also usually provide valuable information on the materials and design assumptions that can be useful for the modeling and evaluation.

2.1.2 Site Inspection

The site inspection is performed to ensure that the plans correspond to the actual situation. It is frequent that the construction does not match exactly the original plans. The site inspection is also required to identify damages to the structural system. Cracking of concrete as well as corrosion of steel reinforcement have to be accounted for in the evaluation because it can reduce the capacity. Corrosion of steel frames as well as cracking have to be accounted for. In cases where a site inspection cannot be realized, ASCE-11 (1999) gives recommendations on this specific aspect but also on how the site inspections should be conducted.

2.2 Proposed Structure Review

The changes resulting from the reuse of a structure have to be studied in details. The magnitude and type of loads ensuing from the modifications have to be assessed and the structural modifications also need to be identified. Additions and modifications to the structural system should be carefully examined so that the forces in the members of the existing structure are not significantly increased. In the case of addition of new equipments, the connections between these and the structure have to represent the structural behavior in a realistic way. Generally, spring elements are used to model these connections. If the occupancy of the building is changed, the resulting new loads should also be determined.

2.3 Statutory Regulations

Building codes differ depending on the location of the structure. It is important that the codes regulating structural design conform to the country/province/state requirements when performing an evaluation. Usually, the loads and load combinations are defined in a building code and the capacity of the members is obtained from the steel or concrete codes.

2.4 Demand Analysis

The demand analysis involves two steps. The loads to which the structure has to resist first have to be defined. Once the loads are defined, the forces (demands) induced in the members have to be assessed. This can be achieved using different approaches. Finite element and rational analysis are two common ways to achieve this.

2.4.1 Loading

The different load types and their magnitude are defined following the recommendations of the appropriate building code. These loads are then added together to form different load combinations according to the building code to evaluate the worst case scenarios a structure can be subjected to.

The gravity loads are divided in dead and live loads. The dead load is the structural mass of the structure and the live loads are the loads produced by the use of the structure. Other loads to consider are wind, snow, ice, and seismic loads. The type of seismic analysis chosen is critical to achieve reliable results. It is advisable to follow the seismic code requirements as in place.

2.4.2 Finite Element Model

After the loads and load combinations have been obtained, the forces induced by these loads have to be calculated. The first method is to use finite element software to perform the analysis. This is done by creating a model representing the structure and load cases are applied to it. The outputs are the forces, displacements, stresses and reactions that are compared to the capacities of the different members. Finite element analysis is complex and only knowledgeable personnel should perform it.

2.4.3 Rational Analysis

The forces in the members can also be evaluated performing a rational analysis which consists of using material mechanic principles. This type of analysis is often used to verify how accurate the results of a finite element analysis are.

2.5 Structural Division

The evaluation of the structure is facilitated by dividing it into groups of members. An efficient way of doing this is to group together members of the same type: slabs, primary beams, secondary beams, columns, walls, bracings, etc. This division allows to evaluate the capacity of each member type efficiently and to quickly identify those that are problematic or do not have sufficient capacity.

2.6 Member Evaluation

The member evaluation consists of assessing the capacity of each member and comparing it to the forces obtained in the demand analysis. Members can be subjected to axial, bending, shear or torsional forces that they need to resist to. Their capacity to resist these forces is usually dependant on the materials, section types and details, and connections. For each member of the structure, the maximum forces from the different combinations are computed with a finite element analysis or rational analysis and compared to their capacity. When the capacity exceeds the demand, it is concluded that the member has sufficient capacity.

2.7 Structure Evaluation

Once all the members are individually evaluated, the overall structure has to be assessed. If no member failures are reported, the structure can be considered as safe. In the case of one or more failures, it is necessary to assess if the local member failure can lead to a global failure of the structure. In such a case, retrofitting of the members can then be performed to increase the capacity.

3 CASE STUDY – CFHT PIER BUILDING

3.1 Canada-France-Hawaii Telescope

The CFHT observatory has been in operation since 1979. It is located in Hawaii on a dormant volcano, Mauna Kea, at an altitude of 4200 meters. The telescope hosted by the CFHT observatory is a world-class 3.6 m optical/infrared telescope. The CFHT organisation wants to replace the current instrument as stated by a review committee (Grundmann, 1997): “It is the opinion of this committee that recent advances in optical and infrared astronomy have been primarily the consequence of improved capabilities in angular resolution, light-gathering power and the ability to conduct observations in non-traditional wavelength regimes. While the CFHT has served its users well by setting the technical standard in these areas, we believe that the "long term" (i.e. starting 2005) facility needs of the three communities would be best served by replacing the existing 3.6 m telescope with a segmented mirror instrument of 12-16 m aperture on the same site (possibly using the same pier)”. In order to replace the current telescope, one first needs to evaluate if the existing pier can be safely reused.

A preliminary draft design for the possible telescope frame concept and mass properties was realized by Dr. Michael Gedig from EDS. This draft included the telescope, its supporting frame, bearings and ring girder on the pier structure. Gedig’s work was based on Grundmann’s (1997) report that “identifies the largest telescope which could reasonably be installed making use of the existing pier”. A 12 to 15 m segmented mirror telescope was studied and Grundmann (1997) came to the conclusion that a 12 m telescope would be the maximum size that the current structure could hold because of the current pier and dome track dimension and design. In addition, he stated that a 10 m segmented mirror telescope would be the best fit for the current installations.

This case study evaluates the CFHT pier building capacity to support the next generation telescope using the approach described in Chapter 2.

3.2 Existing CFHT Pier Building

The first step of the analysis was to review the existing CFHT pier building using plans, design notes and reports. No site inspection could be done, so damage was assumed to be minimal and therefore not considered.

3.2.1 Geometry

The pier building is a three storey reinforced concrete cylindrical pier structure. Figure 2 shows the finished pier building with the enclosure steel frame walls during construction. 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).

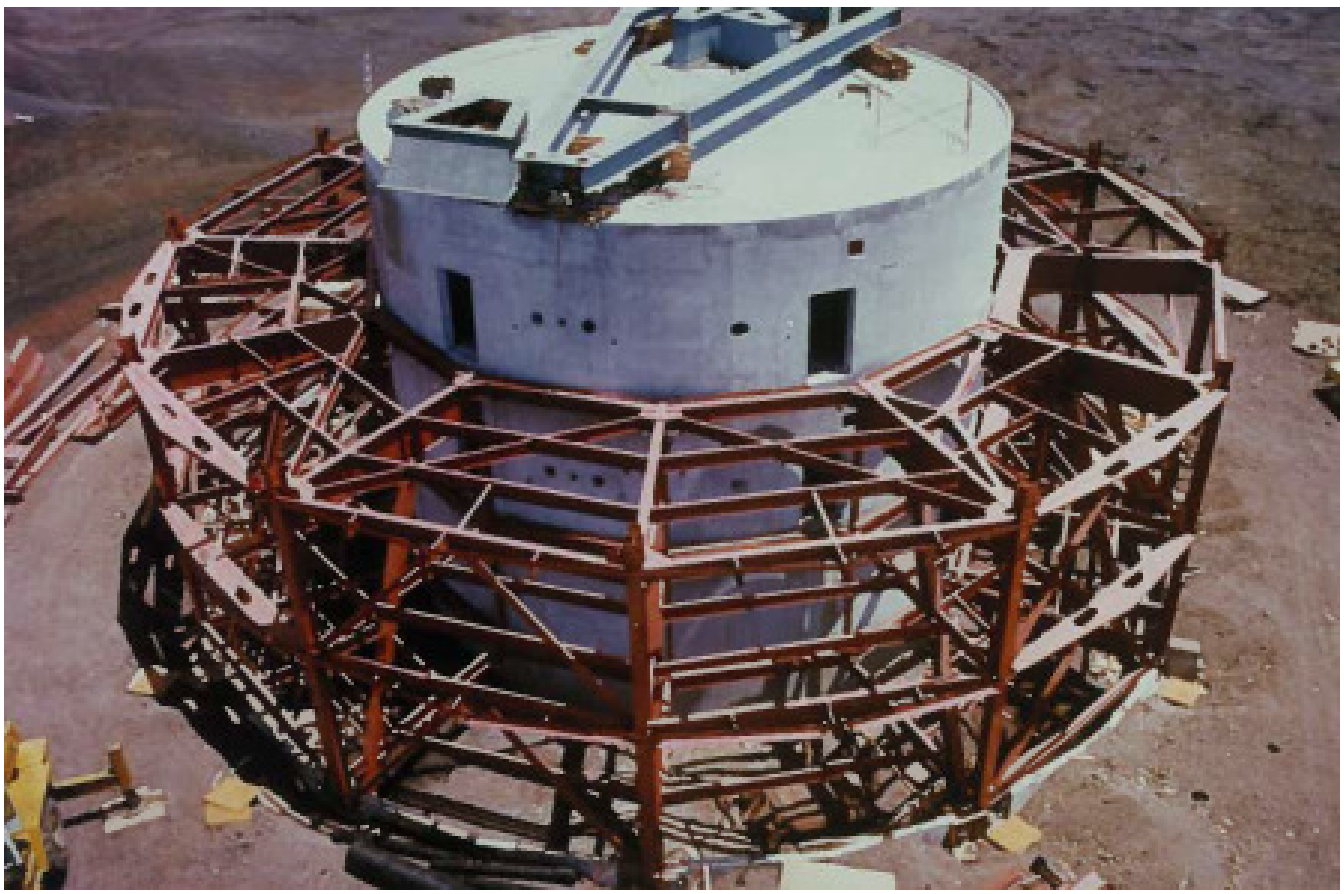

Figure 2: CFHT pier building during the dome support construction (CFHT, 1974, with permission)

3.2.2 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.

$$E_c = 57000\sqrt{f'_c} \tag{1}$$

The reinforcing bars have a yielding strength of 413 MPa and an elasticity modulus of 200 000 MPa.

3.2.3 Soils

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.

3.3 Proposed Modifications

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 3. 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. Figure 4 presents a possible design for the azimuth track and a box steel ring girder.

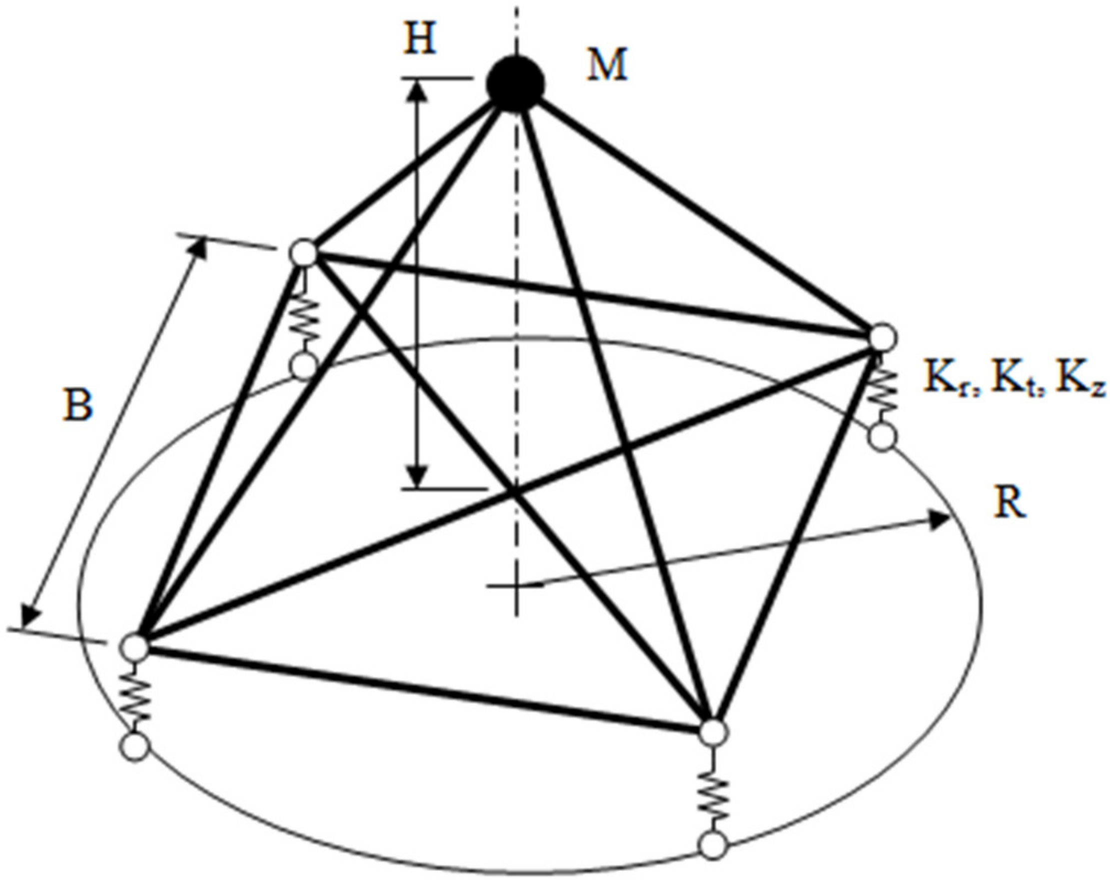

Figure 3: Idealization of the telescope frame, bearings and ring girder

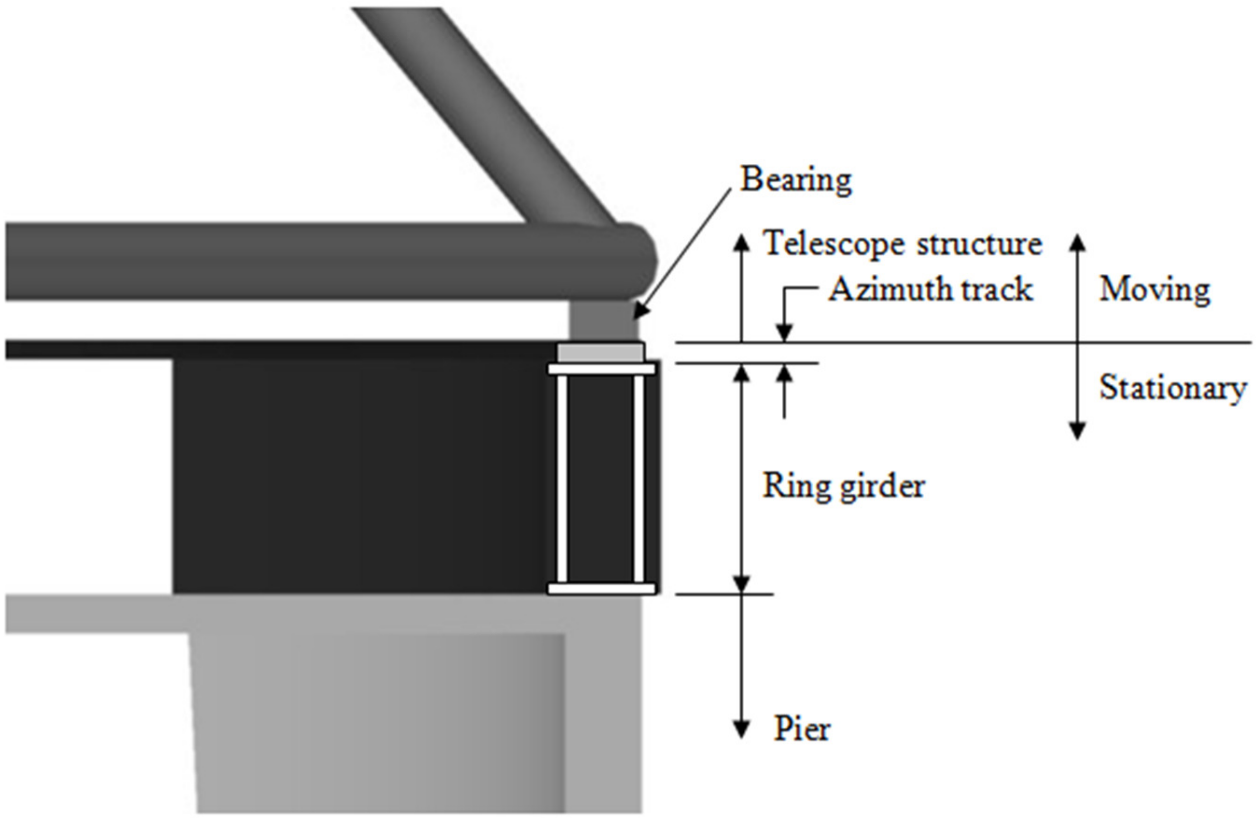

Figure 4: Proposed CFHT ring girder and azimuth track

3.4 Hawaii Statutory Regulations

The first step was to determine the forces, reactions and displacements induced in the various members of the structure due to the different loadings. 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.

3.5 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.

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.

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.

$$1.4D \tag{2}$$

$$1.2D + 1.6L \tag{3}$$

$$(0.9 - 0.2)D + 1.0E \tag{4}$$

$$(1.2 + 0.2D) + 1.0E \tag{5}$$

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):

$$1.0D \tag{6}$$

$$1.0D + 1.0L \tag{7}$$

$$1.0D + 0.525E + 0.75L \tag{8}$$

$$1.0D + 1.0E \tag{9}$$

$$0.6D + 0.7E \tag{10}$$

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

3.5.3 Seismic Loading

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.

3.5.3.1 Seismic Analysis

The ASCE-7 (2005) defines different procedures to perform the seismic analysis of a structure. They depend on the level of precision required and on the structure specificities. If the structure is a structurally regular and low height building, the equivalent lateral force procedure can be used. It is a simple procedure that can easily represent the behavior induced by this type of loading. The static equivalent method represents an earthquake loading by a pattern of forces pushing laterally on the vertical axis of the structure. If more precision in the analysis is required, a response spectrum analysis can be performed. It allows the evaluation of the contribution of the higher modes more precisely and efficiently than the static method. The contribution of each vibration mode is included and evaluated with the response spectrum of the site. Finally, if a more precise and realistic response of the structure under an earthquake is required, a linear time history seismic analysis can be performed. It consists of applying an earthquake acceleration record over time to the structure. This method provides the time response of the structure. However, this method is time consuming and can generate an important amount of data to analyze. Also, the selected earthquakes may not be representative of a future seismic event and the number of earthquake records for certain zones may be limited.

3.5.3.2 Seismic Analysis Parameters

The first step to perform a seismic analysis is to evaluate the soil class and the risk category. The risk category was determined using the ASCE-7 (2005), and the CFHT pier building was considered as a category III structure. Category III implies that the failure of the building or structure could pose a significant risk to human life (ASCE-7, 2005). The soil class was computed with the shear wave velocities as a function of the depth. This information was obtained from Dames and Moore’s (1973) soil report. With the following equation provided by the ASCE-7 (2005), the average shear wave velocity for the soil was calculated.

$$v_s = \frac{\sum d_i}{\sum \frac{d_i}{v_{si}}} \tag{11}$$

Where v_{si} is the shear wave velocity for layer i and d_i, the thickness of layer i. The average shear wave velocity varied between 366 and 762 m/s, defining a soil class C which corresponds to a very dense soil or soft rock (ASCE-7, 2005)

After the soil class and risk category were determined, it was possible to evaluate the mapped spectral acceleration for short periods (S_s) and 1 second (S_1). This was done using the maps found in the ASCE-7 (2005) and the location of the CFHT in Hawaii. For the Mauna Kea, S_S equals to 1.5g and S_1 to 0.6g. These mapped spectral acceleration needed to be transformed into maximum considered earthquake spectral response acceleration for short period (S_{MS}) and at 1 second (S_{M1}) to account for the soil conditions with site coefficients F_a and F_v.

$$S_{MS} = F_a S_S \tag{12}$$

$$S_{M1} = F_v S_1 \tag{13}$$

Where F_a and F_v are 1.0 and 1.3, respectively from tables of the ASCE-7 (2005). S_{MS} and S_{M1} allowed evaluating the design spectral response acceleration parameters (S_{DS}) and (S_{D1}) used to build the response spectrum.

$$S_{DS} = \frac{2}{3} S_{MS} \tag{14}$$

$$S_{D1} = \frac{2}{3} S_{M1} \tag{15}$$

These two spectral response acceleration parameters were the values used to draw the response spectrum. Details on the construction of the design response spectrum can be found in the ASCE-7 (2005).

For the Mauna Kea and a soil class C, S_{DS} was found to be equal to 1 and S_{D1} equal to 0.52. S_s was equal to 1.5 second and S_1 to 0.6 second. The design spectrum for Mauna Kea accounting for a soil class C and 5% damping is shown in Figure 5.

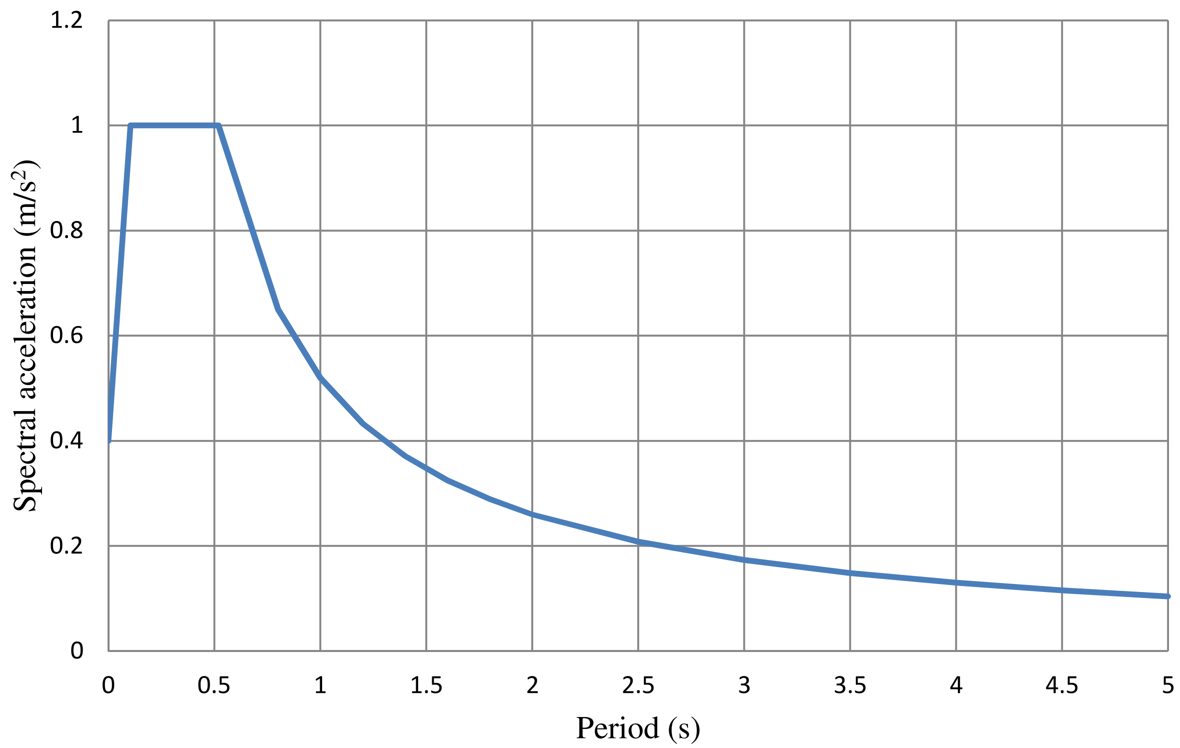

Figure 5: Design response spectrum for Mauna Kea, Hawaii, soil class C and damping ratio of 5%

In this analysis, it is also necessary to evaluate the seismic design category. It is determined using S_{DS} and S_{D1} and the tables of the ASCE-7 (2005). For the present case, a category D was obtained, which corresponds to the most severe category.

3.5.3.3 Natural Period Determination

The natural periods of a building can be determined by performing a modal analysis. But, it is required by the ASCE-7 (2005) that the value of the first natural period must not exceed the product of the coefficient for upper limit on calculated period (C_u) and the approximate fundamental period T_a. The coefficient C_u depends on the design spectral response acceleration parameter at 1.0 second.

Different empirical equations are available to evaluate the first fundamental period; they depend on the type of lateral force resisting system. The concrete pier building was assumed to have a lateral forces resisting system equivalent to concrete shear walls. Equation 16 was used to evaluate the period:

$$T_a = C_t h_n^x \tag{16}$$

Where h_n is the structural height and C_t and x parameters determined with the ASCE-7 (2005).

Using this equation, the period T_a of the pier building was equal to 0.361 seconds and the maximum period that can be used for the static equivalent method was 0.505 seconds. The modal analysis performed with the finite element software gave a first natural period with the telescope in place of 0.279 seconds. It is smaller than 0.505 seconds and is the value that was used to perform the equivalent lateral force procedure on the pier building. A maximum natural period was applied because of non-structural elements that are not accounted for in the modeling of the structure such as imprecision in structural modeling or differences between the design and what was actually done in the field during construction.

3.5.3.4 Choice of the Seismic Analysis Method

The ASCE-7 (2005) states that an equivalent static force method can be used when a building meets certain requirements. The pier building is a structure without irregularities that has a height lower than 49 meters. These characteristics allow the use of an equivalent lateral force analysis. This method is simpler and quicker to perform, but for a more precise response it may be necessary to use a response spectrum analysis or a linear time history analysis. The equivalent lateral force procedure is described in the next section.

3.5.3.5 Equivalent Lateral Force Procedure

The first step when performing an equivalent lateral force procedure is to determine the seismic base shear, which is given by Equation 17 (ASCE-7, 2005):

$$V = C_s W \tag{17}$$

Where C_s is the seismic response coefficient given in Equation 18 and W the seismic weight. The seismic weight was considered to be the dead weight including the telescope.

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)} \tag{18}$$

S_{DS} is the design spectral response acceleration parameter in the short period range, R the response modification factor, and I_e the importance factor. I_e is equal to 1.0. The response modification is a function of the type of seismic resisting system. The cylindrical walls of the pier building were considered to be ordinary reinforced shear walls and therefore R equals to 4.0. C_s should be smaller than:

$$C_s \leq \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} \; for \; T \leq T_L \tag{19}$$

$$C_s \leq \frac{S_{D1}T_L}{T^2\left(\frac{R}{I_e}\right)} \; for \; T \leq T_L \tag{20}$$

$$Cs = 0.044 S_{DS} I_e \geq 0.01 \tag{21}$$

The long period transition period (T_L) is 12 seconds and is used later in this section. And if S_1 is equal or greater to 0.6g:

$$C_s \geq \frac{0.5S_1}{\left(\frac{R}{I_e}\right)} \tag{22}$$

Where S_{d1} is the design spectral response acceleration parameter at a period of 1.0 second, T is the fundamental period of the structure which can be determined following the instructions of the previous section of this document (3.5.3.3).

After the base shear is obtained, it is necessary to distribute this force on each level of the building as a function of the mass distribution. This is done automatically using a finite element software.

In the analysis, torsion also needs to be accounted for. This is done by calculating the eccentricity between the center of mass and the center of rigidity. If a structural analysis software is used, this is also done automatically. Accidental torsion is another aspect that has to be accounted for. It was assumed to be caused by a displacement of the center of mass each way from its location by a 5% the length of the dimension of the structure perpendicular to the earthquake force (ASCE-7, 2005).

3.6 CFHT Pier Building Analysis

As stated earlier, loads acting on a structure induce forces that have to be resisted to by the structure members. The first step is to evaluate the forces in the different elements of a structure caused by the different loads. These forces can be evaluated by hand, but it is much more efficient to use a structural analysis software.

3.6.1 Finite Element Analysis

3.6.1.1 Software Overview

The Structural Analysis Program 2000 (CSI Berkley, 2006), commonly known as SAP2000, was used to compute the forces, the displacements, and the base reactions. SAP2000 provides the user with an interactive interface that allows to quickly and simply model a structure, without requiring programming skills and knowledge of the language. Also, design codes like the National Building Code of Canada (NBCC), the International Building Code (IBC), and the American Concrete Institute (ACI) code are implemented in the software. Frame sections, load cases and design tools are available and facilitate the modeling and analysis. However, the creation of the model is not automated and does not allow quick parametric studies. Extra work is required each time a change is made to the model geometry or meshing. For more detailed analysis and advanced design, the use of this kind of software quickly becomes fastidious and inefficient. More advanced structural analysis software like ANSYS Mechanical can be used in such cases.

3.6.1.2 Pier Building Finite Element Model

The walls of the pier structure as well as the slabs were modeled in SAP2000 using shell elements. The shell elements that were used have in-plane and out-of-plane stiffness to represent as accurately as possible the behavior of the pier. Details on shell elements theory can be found in ETABS User's Manual (CSI Berkley, 1999). The thicknesses were the one of the walls or slabs. Because the second and third storey floors are hollow slabs as shown on Figure 7, their thickness was multiplied by

0.9636. This is the ratio of the inertia of a 1219 mm wide I beam over a rectangular beam of the same width. This provided a reasonable approximation and made it a lot simpler to model than a hollow slab made as a grillage with frame elements. Figure 6 presents the SAP2000 finite element model of the pier and telescope structures.

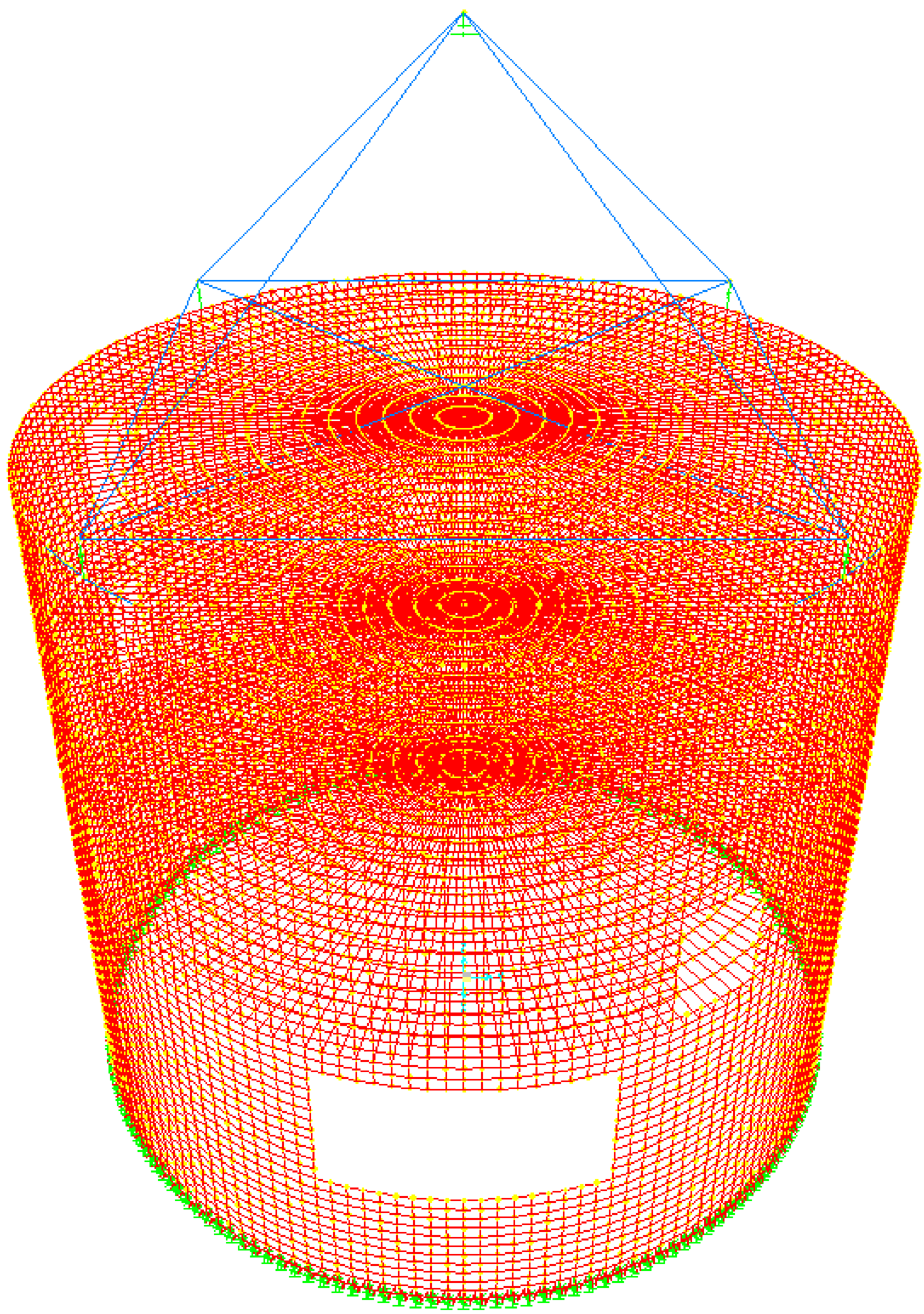

Figure 6: SAP2000 pier and telescope frame finite element model

The first step to create the pier model was to generate a grid that would provide a base to draw the structure elements. Because of the cylindrical nature of the pier building, a cylindrical mesh was created keeping in mind the different constraints. Rectangular shell elements were chosen to be able to easily integrate the forces, to link them to the capacities and to organize the results. The sizes of the elements have to be chosen carefully to have the right height of storeys, and to have openings at their exact location and with their proper dimensions. The rectangular meshes were kept as square as possible to get accurate displacement and forces at each node. Once the mesh was created, nodes were generated at each intersection and shell elements generated between the nodes.

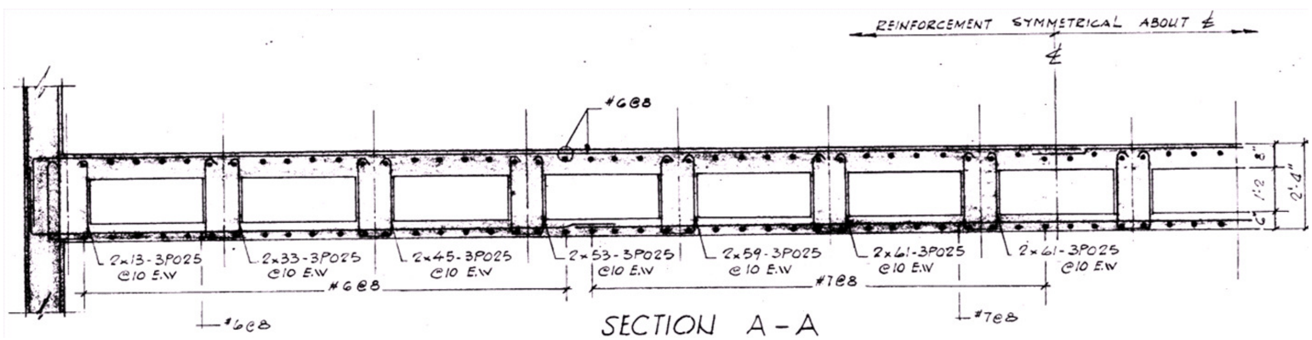

Figure 7: Hollow slabs of the second and third storeys (CFHT, 1974, with permission)

3.6.1.3 Pier Building Supports Modeling

Supports of structures in civil engineering are usually modeled for analysis as clamped or pinned bases. Assuming these kinds of support for the design or evaluation of a structure does not account for the stiffness of the foundation and of the soils. To get a more accurate representation of the behavior of a structure and more exact forces and displacements, soil-structure interaction should be considered.

A structure subjected to lateral loading such as an earthquake does not have its structural and soil displacement independent of each other. The process by which the response of the soil influences the motion of the structure and the motion of the structure influences the response of the soil is known as soil-structure interaction (Tuladhar, 2006). It has been a challenging problem in civil engineering for several years.

Because of the general complexity of soil behavior and of the lack of information on the soil conditions at the CFHT site, the soil-structure interface was idealized as a simpler model called subgrade model. It is based on Winkler idealization, Dutta and Roy (2002) state that it represents the soil as a system of identical but mutually independent, closely spaced, discrete and linearly elastic springs. The key point is to evaluate the stiffness of these springs which is called the coefficient of vertical subgrade reaction (k_s). It is the ratio between the pressure and the settlement produced by that pressure (Dutta and Roy, 2002). This method has its limitations as the springs are not coupled together and the shear stresses are not transmitted. It is important to carefully evaluate k_s to have an accurate model (Stavridis, 2000).

The evaluation of the vertical subgrade reaction is complex. For this analysis, it was suggested to the author to use Vesic's equation to evaluate this parameter. Also, a number of equations were evaluated by Sadrekarimi and Akbarzad (2009), and the Vesic's equation was found to be the most accurate according to their calculations. Vesic's equation (Equation 23) provides the stiffness (k_s) in N/mm^3. The vertical stiffness for a single point (k_v) has to be multiplied by the area of the foundation that has to be represented by that point.

$$k_s = \frac{0.65E_s}{B(1-\nu_s^2)} \sqrt[12]{\frac{E_s B^4}{EI}} \tag{23}$$

$$k_v = BDk_s \tag{24}$$

Vesic's equation is a function of the elasticity modulus of the soil (E_s), the Poisson ratio of the soil (υ_s), the width of the footing (B), the length of the footing to be considered (D), the elasticity modulus of the concrete of the footing (E), and the moment of inertia of the footing (I).

The elasticity modulus of concrete, the width of the foundation and its inertia are all known parameters. The elasticity modulus of the soil and the Poisson ratio are soil parameters that have to be evaluated and carefully chosen. EDS (2011) provided a value of 124 MPa for the elastic modulus (E_s) of the soil on Mauna Kea and a Poisson ratio of 0.3.

The pier building foundation was assumed as a spread footing. Priestley and Seible (1996) suggested that the soil provides stiffness in the vertical (k_v), horizontal (k_h) and rotational (k_r) directions such as presented in Figure 8. For a spread footing, horizontal direction can have its boundary conditions fixed (Priestley and Seible, 1996).

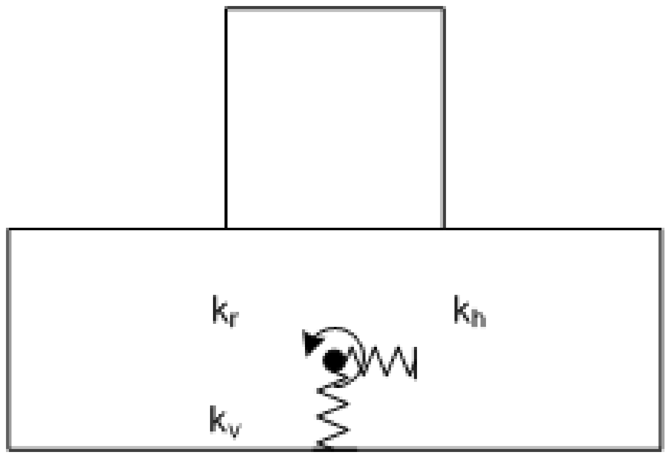

Figure 8: Spread footing support stiffnesses

The rotational stiffness was evaluated using equation 25 from Priestley and Seible (1996).

$$k_r = \frac{1}{12} B^3 D k_s \tag{25}$$

The stiffness values of the springs are summarized in Table 1. The values are for a foundation length of 304.8 mm. The other degrees of freedom at the supports were considered as fixed.

Table 1: Subgrade reaction coefficients and stiffnesses of support springs for a 304.8 mm foundation length

Parameter	Variable	Value
Concrete elasticity modulus (MPa)	E	21525
Soil elasticity modulus (MPa)	E_s	124
Soil Poisson's ratio	υ_s	0.3
Width of the footing (mm)	B	2240
Moment of inertia of the footing (mm^4)	I	$5285 \text{x} 10^6$
Length of the footing (mm)	D	304.8
Subgrade reaction coefficient (N/mm^3)	k_s	0.156
Vertical stiffness (N/mm)	k_v	58238
Rotational stiffness (N.mm)	k_r	$7213 \text{x} 10^6$

3.6.1.4 Telescope Modeling

The telescope steel frame described earlier in this document is composed of truss elements. Truss elements only take forces axially; they do not develop moments or torsion. The sections of the frame members were determined by keeping the dead load deflection of the pyramidal frame under 5 mm. The bearing stiffnesses were evaluated as a function of the periods of vibration of the frame. The first mode of the frame has to have a natural frequency equal to 4.0 Hz and the higher modes have to have natural frequencies over 4.0 Hz, but not so high as the pier stiffness increases too much. Each spring has a radial (K_r), tangential (K_t) and vertical (K_v) stiffness as shown in Figure 9. Once again the telescope mass was approximated at 270 000 kg compared to 255 000 kg for the old telescope, an increase of 5.9%.

Following these recommendations, natural frequencies, frame deflection, stiffnesses of the springs and frame sections were determined. The vertical deflection of the truss frame without the spring supports was found to be 3.03 mm. The spring stiffnesses K_r, K_t and K_v were determined to be of 137 kN/mm to meet the requirements. The modal results for the telescope frame idealization are presented in Table 2.

Table 2: Telescope natural frequencies

Mode	Frequency (Hz)	Mode shape description
1	3.99	x translation
2	4.20	y translation
3	5.60	z translation

The ring girder and azimuth track were modeled as rigid beam elements connected to the pier wall. The ring girder was modeled to not overly stiffen the structure and to distribute the forces more uniformly to the pier in order to avoid stress concentration. There are no rigid frame elements in SAP2000, so a really stiff beam was created with a high elasticity modulus. The telescope model was connected to the pier structure with its frame, springs and rigid ring girder elements (Figure 9). The ring girder elements shared nodes with the pier structure concrete walls.

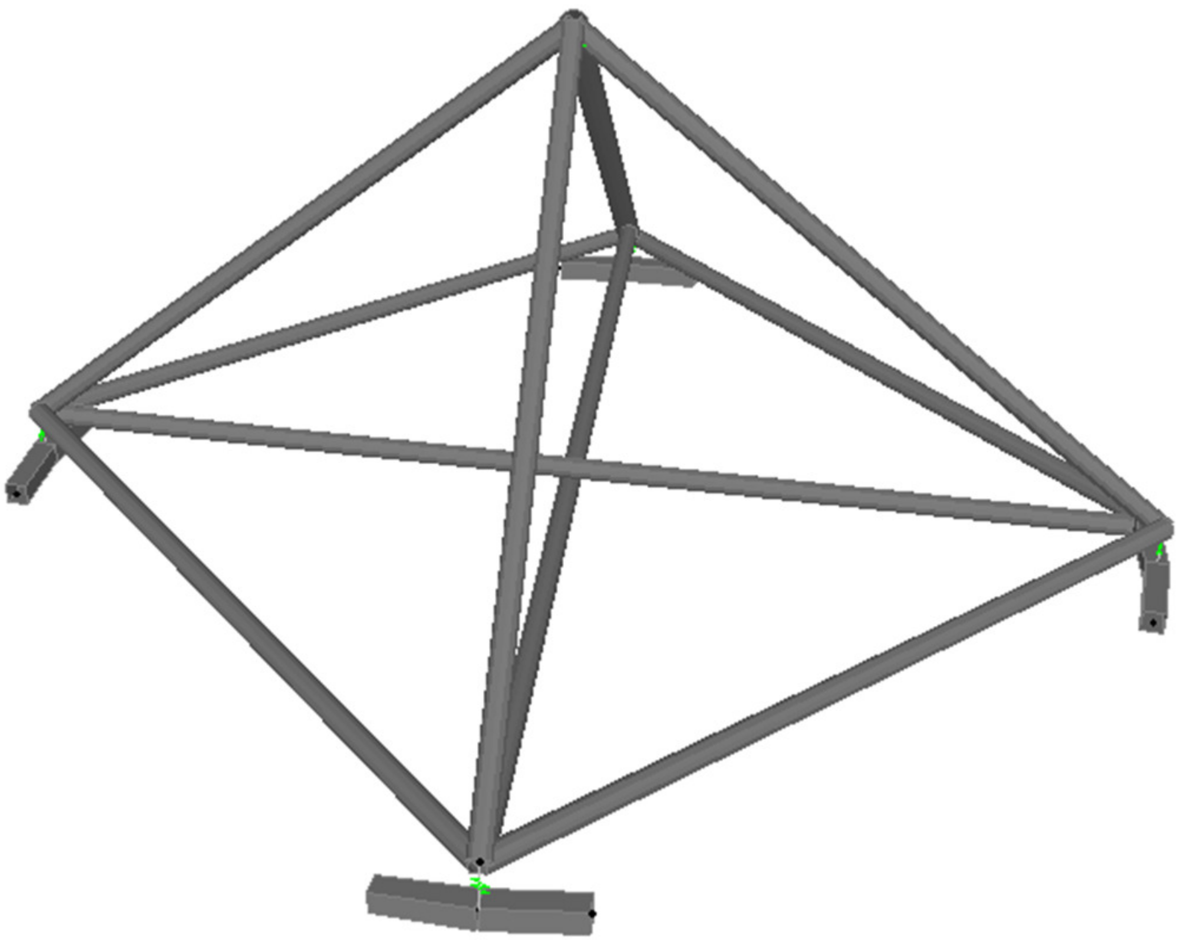

Figure 9: SAP2000 telescope frame, bearings and supports model

When the model construction was completed, the different load cases and combinations were implemented in SAP2000 and the analyses were run. The results can be obtained in graphical representation or listed in tables. These tables can be exported in Excel spreadsheets to perform the evaluation.

3.6.2 Rational Analysis

3.6.2.1 Solid Mechanics Approach

The loads applied to the structure induce forces to be resisted to by the different members of the structure. If a body can be considered as being at rest, we are in the field of statics. Structural analysis is based on the equilibrium of a body. The requirements that a body be considered as static are that the sum of the forces acting on a body equals to zero (Equation 26), and to have the sum of the moments applied on a body at an arbitrary point 0 equals to zero (equation 27) (Craig, 2000).

$$\sum F = 0 \tag{26}$$

$$\left(\sum M\right)_0 = 0 \tag{27}$$

In order to have the body at equilibrium, the forces at the supports have to be determined. These are called the reactions and are points where the displacements are known. The supports enforce constraint (Craig, 2000). The reactions forces and moments are calculated by applying Equations 26 and 27 to the body.

The loads are transmitted to the reactions by internal forces. For a slender body, these internal forces are determined at different section cuts along a member. At each section cut, the equilibrium of the body has to be respected. To achieve equilibrium, six internal resultant forces are present. The force normal to the cross section and parallel to longitudinal axis of the member is termed the axial force, (P). The forces tangents to the cross section are the shear forces, (V). The moment about the longitudinal axis is the torque or torsion force, (T). The moments about the tangent axis of the cross section are the bending moments, (M).

3.6.3 Modal Analysis

A modal analysis is performed to evaluate the dynamic characteristics of the pier building. The modal analysis gives the natural frequencies or periods of the structure, the participating mass of each mode for horizontal and vertical directions. The base supports of the structure were idealized as spring elements described earlier in this document (section 3.6.1.3). Table 3 and Table 4 give the natural frequencies and cumulative participating mass ratios for the pier structure with the telescope and without it.

Table 3: Pier structure with telescope natural frequencies and cumulative participating mass ratios

Mode	Frequency (Hz)	Mass contribution ratio in x	Mass contribution ratio in z
1	3.58	0.21	0.00009
2	5.50	0.32	0.27
3	8.62	0.32	0.29
4	9.67	0.78	0.63
5	11.75	0.79	0.63
6	11.80	0.8	0.63
7	19.28	0.8	0.63
8	19.88	0.8	0.63
9	21.66	0.85	0.78
10	22.96	0.85	0.78
11	25.82	0.85	0.97
12	28.45	0.85	0.97
13	29.01	0.93	0.98

Table 4: Pier structure without telescope natural frequencies and cumulative participating mass ratios

Mod	Frequency	Mass contribution ratio in x	Mass contribution ratio in z
1	5.55	0.000049	0.090
2	8.20	0.73	0.095
3	9.99	0.74	0.54
4	11.65	0.76	0.54
5	19.18	0.76	0.54
6	20.66	0.76	0.72
7	22.25	0.81	0.73
10	23.28	0.81	0.73
11	26.90	0.81	0.97
12	28.24	0.81	0.97
13	32.58	0.92	0.97

It can be seen that the frequencies for the structure with the telescope are lower than without it. This is due to the fact that the telescope frame has a lower stiffness than the pier structure. The pier structure has high frequencies reflecting its high stiffness. The high frequencies of the pier are required to minimize wind induced vibration as well as the vibrations created by the movement of the telescope.

3.6.4 Finite Element Model Verification

It is also important to make sure that the finite element modeling was done properly. The finite element model has to behave as much as possible as the real structure so the displacements and forces are well approximated. To make that verification, a simpler model of the structure was used representing the structure as single frame elements to model the pier. The model was assumed to behave in its elastic range like the more refined model. Weights of the walls were incorporated in the self-weight of the frame elements and the slabs weights and telescope mass were lumped at each storey. The support was idealized as a clamped base because it was not possible to account precisely in a simple idealization for the stiffness of the soil and foundation.

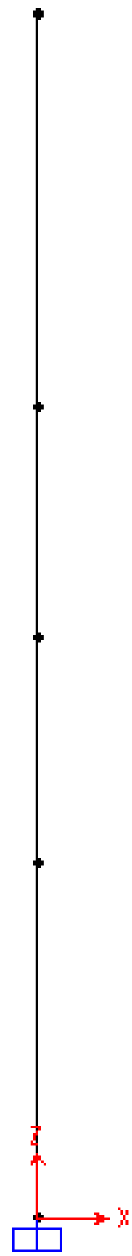

Figure 10: SAP2000 pier and telescope frame simplified model

In this analysis, the heart of the problem is to model the telescope frame as a one frame element. To achieve this, a horizontal force was applied on top of the telescope frame without the pier. The displacement was measured and the inertia of the frame representation was modified to match the same displacement as the frame. Once both displacements were equal, the lateral stiffness of both frames was considered equivalent. This is the case because stiffness is the displacement divided by the force.

The simplified pier model including the telescope parameters used for the modeling are given in Table 5.

Table 5: Four degree of freedoms pier building idealization parameters

Parameter	Variable	Value
Concrete modulus of Elasticity (MPa)	E_c	21525
Steel modulus of elasticity (MPa)	E_s	200000
Pier moment of inertia (mm^4)	I_p	5.205E14
Telescope frame moment of inertia (mm^4)	I_t	1.060E11
Pier wall axial area (mm^2)	A_p	15627337.0
Telescope frame axial area (mm^2)	A_t	436681.4

A modal analysis to obtain the natural frequencies of the simplified model was also performed and compared with the results of the detailed finite element model. The idealization was compared with two models, one with a spring base and one with a clamped base. The results are given in Table 6.

Table 6: Natural frequencies of lateral mode shapes comparison in X direction

Mode	Detailed 3D structure	Detailed 3D structure clamped	4 DOFs structure
	Hz	Hz	Hz
1	3.58	3.78	3.8
2	11.8	12.02	12.3
3	36.9	37.73	37.3

Also, the base reactions and displacements of each floor when subjected to the 1.2D + 1.0E + 0.6L load combination were evaluated and compared to the results of the refined models. Results are presented in Table 7.

Table 7: Base reaction and displacement comparison for 1.2D+1E+0.6L load combination

Parameter	3D structure	3D structure clamped	4 DOFs structure
Base moment (kN.m)	51930	522560	54110
Base shear (kN)	3791	3791	3856
Base vertical (kN)	22310	22310	22552
Telescope disp. (mm)	11.18	9.6	8.94
Top floor disp. (mm)	2.5	1.47	1.03
Second floor disp. (mm)	1.74	0.98	0.83
First floor disp. (mm)	0.97	0.53	0.43

The displacements at each floor, the mode shapes, the natural frequencies, as well as the base reactions gave really close results to the more complex model under the same load combination. These results provide confidence that the model will give reliable results.

3.7 CFHT Pier Building Structural Division

To simplify the evaluation of the pier building, a division of the structure was realized. Members of the same type or having similar characteristics were grouped together:

1. Walls – The group is formed of the reinforced concrete cylindrical walls that compose the pier building.

2. Slabs – Three slabs are present in the pier building. The first and second slabs are hollow slabs and the top one is full.

3. Openings – Doors and windows are present in the walls of the pier building. Reinforcement around in the walls around the openings have to be evaluated.

4. Footings – The pier building walls are supported by a reinforced concrete ring footing.

5. Foundations – The foundations is the soil on which the footing is sitting. The footing is distributing the forces induced by the pier building to the soil.

3.8 CFHT Pier Members Evaluation

3.8.1 Walls

3.8.1.1 Bending Capacity

Considering its location, the pier building has to resist to the maximum bending moment induced by earthquake loads. The maximum bending moment is located at the bottom of the walls. The bending capacity can be assessed using different parameters. The ultimate bending capacity (M_n) is the moment at which the bars start breaking; it is the maximum moment the section can resist. The bending capacity can also be assumed as the cracking moment (M_{cr}); it is the point at which the concrete cracks in tension. The point at which the bars in the section start yielding can also be used as an estimate of the capacity. For the present problem, it is required to have the pier behaving elastically, so the cracking point will be assumed as the capacity. The factored maximum bending moment (M_u) was determined from the four degree of freedom model and is equal to 54,110 kNm.

The bending cracking capacity can be evaluated with the following method. Figure 11 represents the distribution of the axial, bending and total stresses on the pier building. The capacity is a function of the bending tension cracking strength (f_{cr}), the inertia of the section (I), the radius of the pier (r), the area of the pier wall (A) and the axial load (P). The stress induced by the axial load was defined by Equation 28, and because it is a compression stress, its value is negative.

$$f_{axial} = \frac{P}{A} \tag{28}$$

The stress at the outer fiber due to the bending moment (M) at a distance r of the center of the section is defined by Equation 29. The fiber in tension has a positive sign.

$$f_{bending} = \frac{Mr}{I} \tag{29}$$

The stresses can be directly superimposed and equation 30 can be derived. Then M_{cr} can be isolated to obtain directly the bending cracking capacity in equation 31.

$$fcr = \frac{P}{A} + \frac{Mr}{I} \tag{30}$$

$$M_{cr} = \frac{f_{cr}I}{r} - \frac{PI}{A} \tag{31}$$

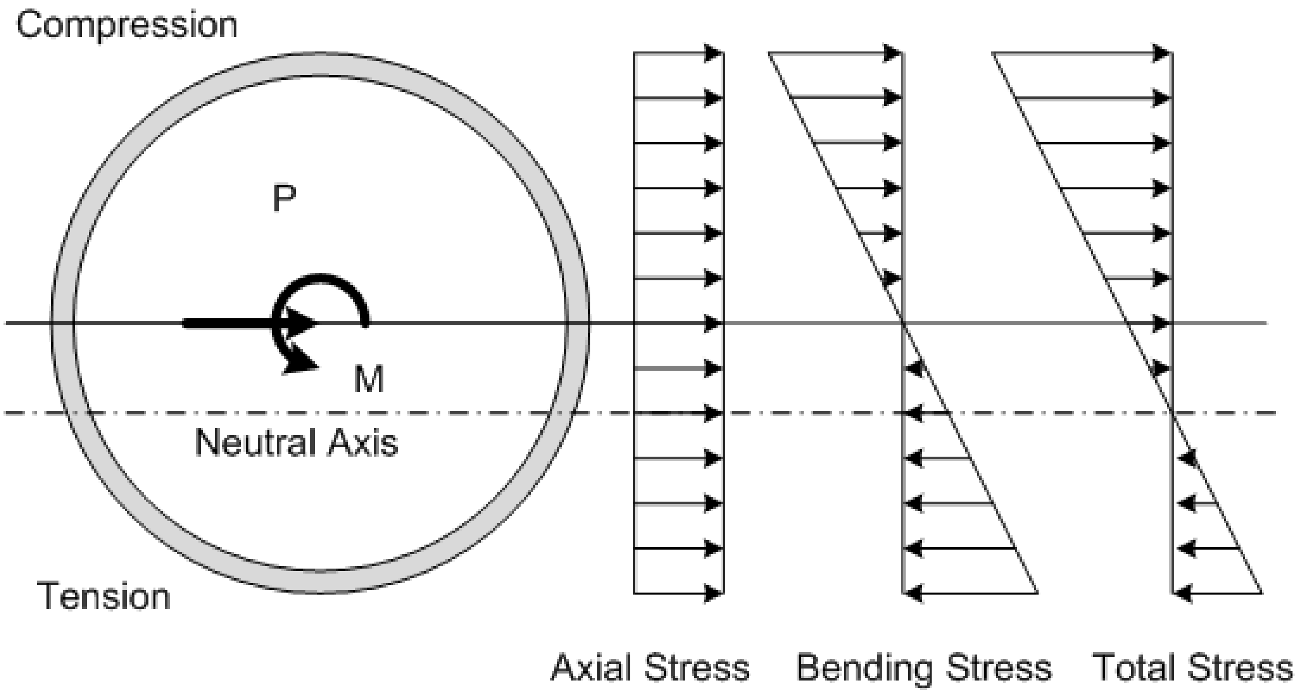

Figure 11: Distribution of stresses for the evaluation of the bending cracking capacity

The capacity was also evaluated with the Response 2000 software developed at the University of Toronto by Bentz and Collins (2000). With this software, the moment-curvature curve of a reinforced concrete section can be calculated. The bending moment capacity was computed for each increment of the curvature. The walls of the structure with the vertical reinforcement were modeled and the curve plotted in Figure 12.

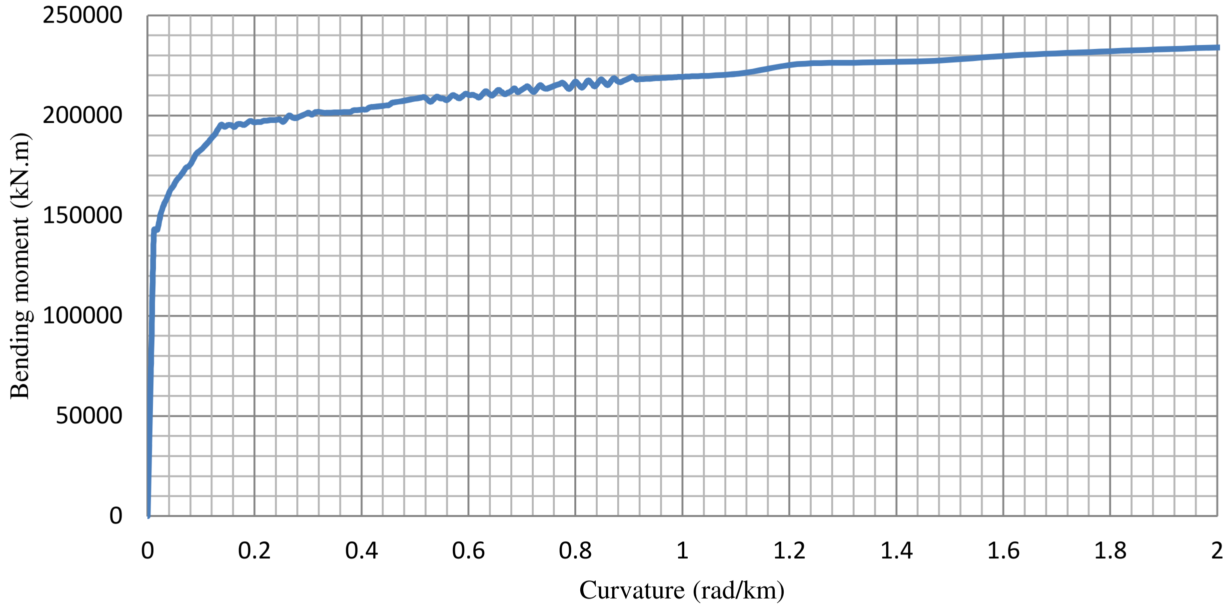

Figure 12: Moment-curvature response for the pier structure section

The first part of the moment curvature response is a straight line until the cracking of the concrete occurs. The concrete therefore behaves as a linearly elastic material until its maximum tensile stress is reached. The bending tensile strength of the concrete is chosen at a conservative value of 1.49 MPa (Bentz and Collins, 2000). After cracking, it is the steel that resists the tension forces in the section. The capacity keeps increasing until the steel starts to yield at which time the yielding capacity is achieved. Finally, the capacity increases until the ultimate capacity because of steel strain hardening. Because the bending capacity increases as the axial load increases, the axial load was assumed to be one time the dead load to obtain conservative results. The results are summarized in Table 8.

Table 8: Bending moment capacity of the pier building evaluated with Response2000

Limit State	Bending moment demand (kN.m)	Bending moment capacity (kN.m)	Ratio force vs capacity
Cracking	54280	142843	0.38
Yielding	54280	193259	0.28
Ultimate	54280	240114	0.23

The induced bending moment was 38% of the bending moment cracking point. This means that the earthquake loading would not cause cracking at the base of the structure. It is concluded that the structure remains elastic at its base when subjected to earthquake loads, which is the desired behavior.

3.8.1.2 Shear Capacity

Method 1

The horizontal forces induced by earthquake forces in the pier building walls also had to be verified. These horizontal forces are the shear forces. Shear forces (V) in reinforced concrete structure are resisted to by the shear capacity (V_n) that is composed of a steel portion (V_s) and a concrete portion (V_c). The steel contribution is given by the horizontal steel in the walls.

The first method used is derived from the ACI 371R (1998), "Guide for the Analysis, Design, and Construction of Concrete-Pedestal Water Towers". It assumes that the shear force (V) is resisted to on two straight parallel walls (Figure 13). The shear can be verified at different heights using this method. For heights without openings, the shear force at that height is equally distributed in each wall (V_w).

$$V_w = 0.5V \tag{32}$$

At heights where openings are present, the shear force is distributed as a function of the center of rigidity. Figure 13 shows the procedure and the next equation provides the shear force in each wall as a function of the opening length b.

$$V_w = 0.5V\left(1 + \frac{\psi}{2 - \psi}\right) \tag{33}$$

$$\psi = \frac{\mathrm{b}}{0.78\mathrm{d}} \tag{34}$$

The area of each wall (A_{cv}) contributing to the shear capacity is given by equation 35 (ACI 371R, 1998).

$$A_{cv} = 0.78(1 - \psi)\mathrm{h} \tag{35}$$

Where d is the diameter of the pier building and h the thickness of the wall.

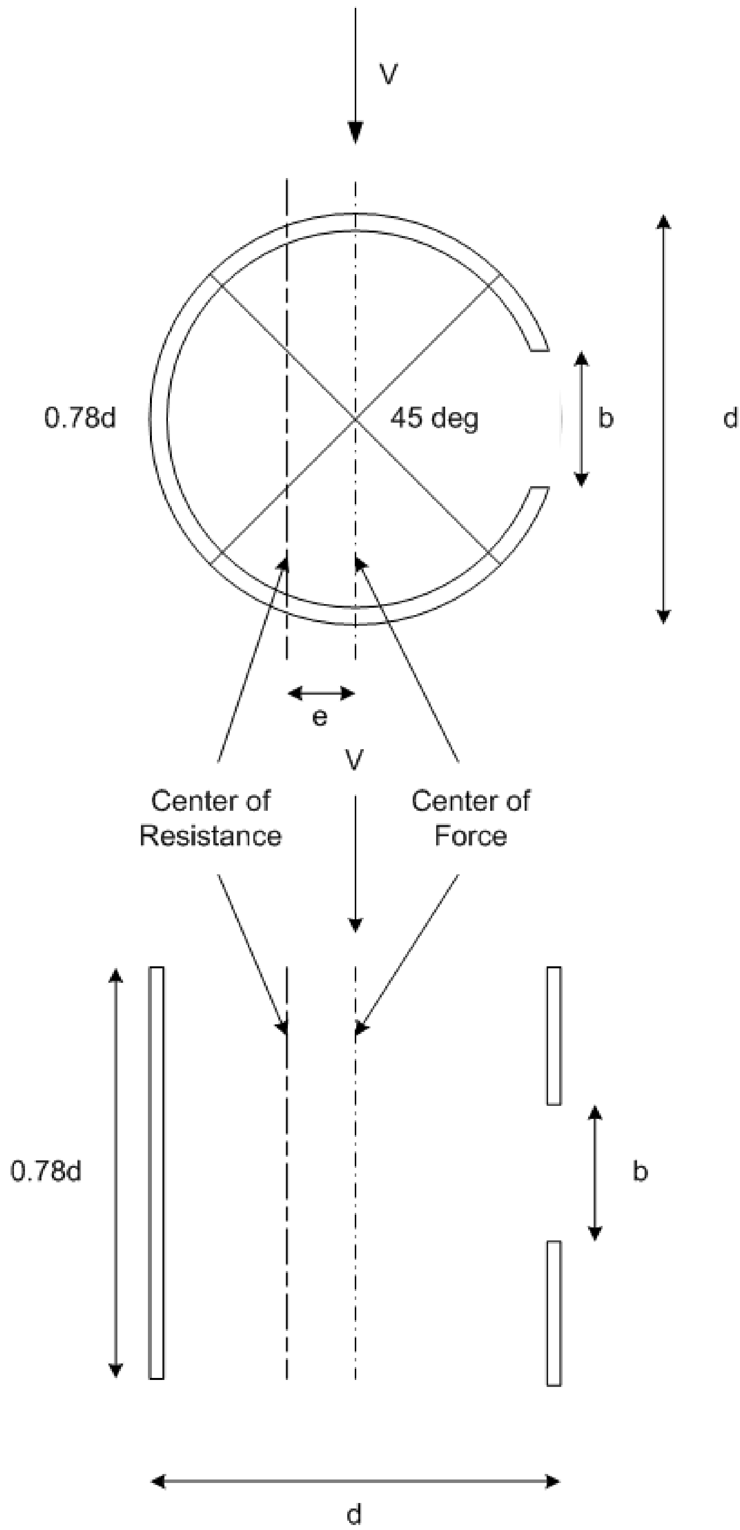

Figure 13: Shear forces distribution diagram in the walls of the pier structure

The factored capacity at the base of the pier and at the openings for each wall were evaluated and compared to the shear forces induced in each wall by the earthquake loads. This verification is described in Equation 36:

$$V_w \leq \varphi V_n \tag{36}$$

Where φ is the resistance factor with a value of 0.75 (ACI 318R, 2008). The capacity is the summation of the concrete and steel contributions to the shear capacity (Equation 37).

$$V_n = V_c + V_s \tag{37}$$

The concrete contribution of the capacity was determined using a conservative equation from the ACI 318R (2008) that does not include an increase of the capacity with the presence of an axial load (Equation 38). In order to obtain a conservative evaluation of the capacity, it was assumed that the axial force would not contribute to an increase of the concrete capacity.

$$V_c = 0.17\lambda\sqrt{\mathrm{f'_c}}\mathrm{A_{cv}} \tag{38}$$

Where λ is a factor to account for low density concrete; the CFHT pier building has regular concrete, λ is equal to 1.0 (ACI 318R, 2008). The contribution of the steel to the capacity was obtained using Equation 39 (ACI 318R, 2008).

$$V_s = \rho_h f_y A_{cv} \tag{39}$$

Where ρ_h is the horizontal steel ratio present in the wall. The horizontal steel ratio in the walls of the CFHT pier building is generally of 0.23 %.

The shear forces for each height were obtained from the 4 DOFs model. Capacities were verified at the base because this is where the shear force is the highest, and at the main opening because of the different distribution of forces. The results are summarized in Table 9.

Table 9: Pier walls shear capacities and forces using straight walls idealization

Location of the check	Shear force (kN)	Shear capacity (kN)	Factored shear capacity (kN)	Ratio
At foundation	3 856	10560	7920	0.48
At midheight of main opening				
Wall without opening	1 364	5 280	3 960	0.34
Wall with opening	2 398	3 004	2 253	1.06

The results suggest that the shear capacity at the base is sufficient and that at the openings height, the ratio of capacity over force is slightly over 1.0. Because of the conservative assumption of the concrete contribution and of the low 0.75 capacity reduction factor, it can be considered that the shear capacity is sufficient.

Method 2

The shear capacity was also estimated using a different approach to confirm the previous results. The SAP2000 finite element model was used and the tangential horizontal forces at each node of the structure walls were compared to the capacity of the section that each of the node covers. The shear forces envelope was plotted on the structure model in Figure 14.

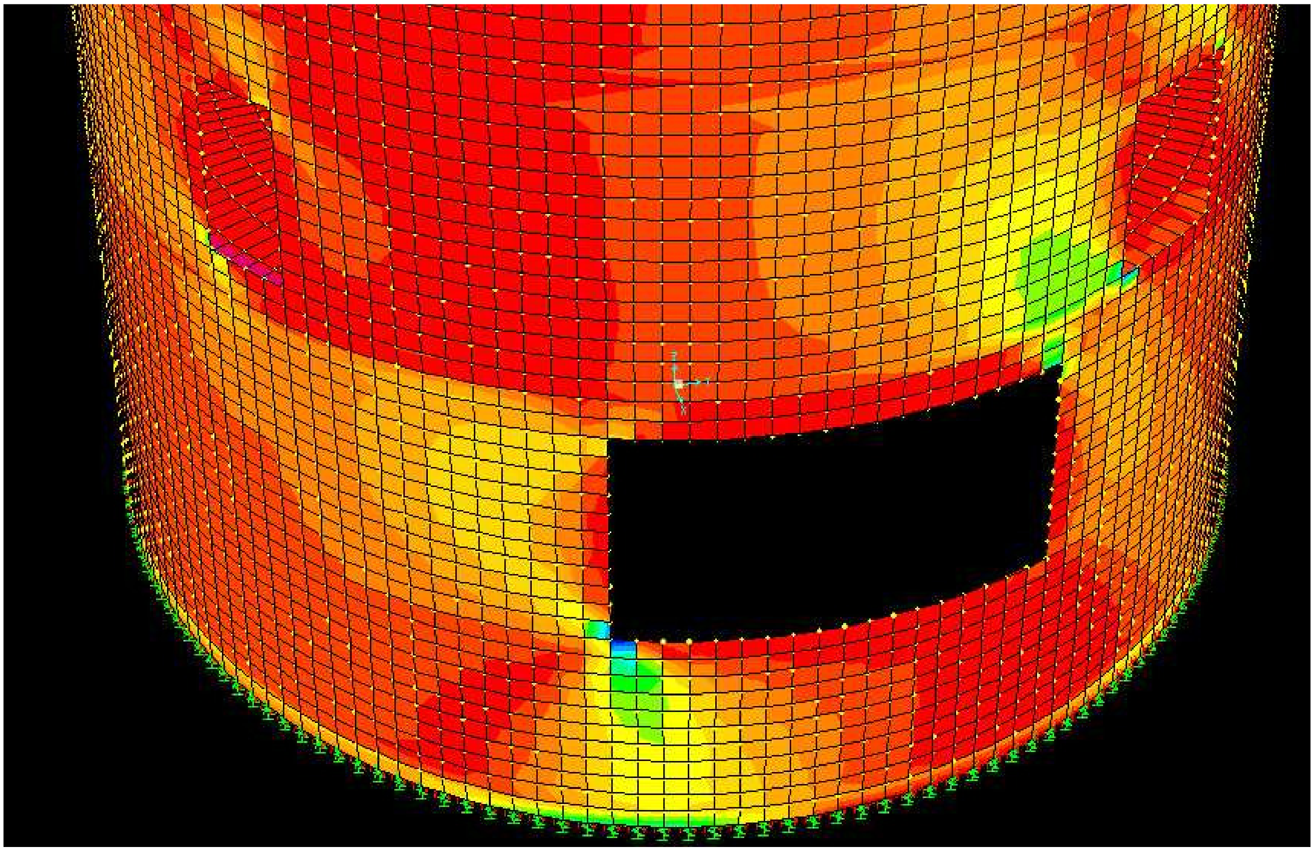

Figure 14: Shear forces envelope in the pier structure under earthquake loading

Nodes were placed every 304.8 mm. The capacity is the summation of the respective contributions of the concrete and steel over that length, as described in Equation 37. Because the finite element analysis provided the axial force at each node, an equation for the concrete shear capacity including the increase in the capacity due to compression load, was used (ACI 318, 2008). This equation is in imperial units.

$$V_c = 2(1 + \frac{P_u}{2000A_g})\lambda\sqrt{f'_c}b_w d \qquad (40)$$

Where P_u is the axial force at each node and A_g the area of the horizontal area of the section studied. As for method 1, the forces have to be compared to the capacities. The steel shear capacity was estimated using Equation 39. The capacity at each node was compared with the shear force for the different load combinations and earthquake orientations. The capacity was again reduced by a factor of 0.75 as recommended by the ACI 318 (2008). Each load combination at each section was verified. The forces were lower than the factored capacities in almost every case. Only a few nodes were slightly over the capacity with ratios varying between 1.0 and 1.3. This was observed mostly at the opening corners. Because of the stress redistribution after cracking, it was considered that the shear capacity is sufficient for the pier walls.

3.8.2 Steel Reinforcement Ratios

Ratios of steel reinforcement for the walls and the slabs also have to be verified. The method used was the one implemented in SAP2000 which is based on Brondum-Nielsen (1974) and Marti (1990). The structure was modeled with shell elements. When applying loads, these shell elements develop membrane forces, flexural moments and shear forces components. This design method assumes that the shells are composed of three layers: two steel reinforcement outer layers and one concrete core layer. This is often referred to as a “sandwich model” (CSI Berkeley, 2006). The outer layers take the moments and membrane forces, and the core takes the plane shear forces.

The six resulting forces cited earlier were transformed in pure membrane forces. These forces were then transformed into design forces in one and two directions for the top and bottom layers, respectively. These were obtained from Brondum-Nielsen (1974) equations. The design forces can then be transformed into steel area ratios, principal compressive forces and principal compressive stresses. More details on the procedure can be found in “Concrete Shell Reinforcement Design Technical Note and Design Information” (CSI Berkeley, 2006). The steel ratios were computed using a reduction factor of 0.9. Using this method, the steel ratios in the one and two directions and for the top and bottom layers were evaluated for the slabs and walls for the different load combinations.

These calculations suggest that the steel present in the top slab and in the first and second storey slabs is sufficient. The required steel ratio over the present steel ratio is under 1.0 for each slab section, for one and two directions and top and bottom.

The walls were more complex to analyze because of stress concentrations due to the slab connections and openings. The structure was first evaluated without any openings, and the required ratios were found to be lower or slightly over the steel ratios present in the structure. The structure was also analyzed with the openings. The ratios of required steel were evaluated to be slightly higher than the present steel, but not excessively. It was therefore considered that the steel ratios were acceptable because of stress redistribution when cracking occurs and because of strain hardening after yielding of the bars.

The minimum reinforcements were also verified. ACI 371R (1998) provides recommendations on the minimum amount of horizontal and vertical steel required in the pier walls. In high seismic zones, the ratio of steel should be higher than 0.25% in both directions. There should be two layers of steel in both directions, and the ratio of vertical steel should be higher than the horizontal one. The vertical steel ratio was 0.33% and the horizontal one 0.23%. For flexural members, the ratio of vertical steel should be equal or higher than 0.33%. The required horizontal steel ratio of steel was just slightly over the one found in the structure, and therefore the vertical steel requirement was considered to be met. The steel ratios were also considered to be sufficient especially considering that water towers have to support major loads.

3.8.3 Openings

The small openings were assumed to be sufficiently reinforced and to not significantly redistribute the stresses in the structure. However, the main first floor opening capacity had to be verified. The opening geometry and its steel reinforcement are shown in Figure 15.

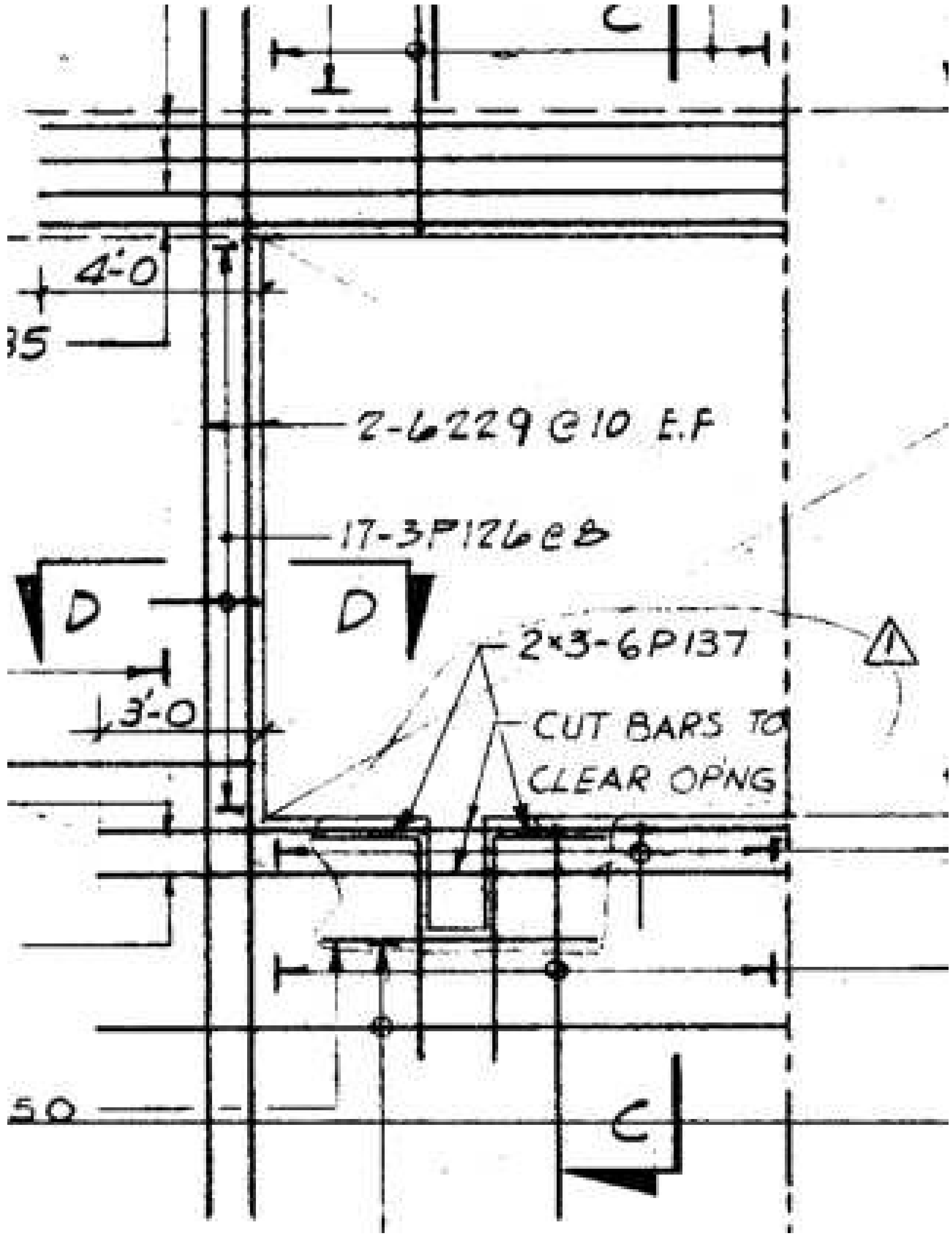

Figure 15: Reinforcement detail of the first storey opening (CFHT, 1974, with permission)

3.8.3.1 Top and Bottom of the Opening

The main opening top has to have enough horizontal steel to resist the maximum bending moment. The required amount of steel was evaluated using the ACI 371R (1998) requirements. For this method, the load was assumed to be equally distributed over the perimeter of the wall. The telescope load is concentrated on four points but, because of the ring girder redistributing the load uniformly over a large surface, it was assumed that the axial forces over the circumference at the opening height were of equal value. This is confirmed by checking the forces distribution of the finite element model. This distribution of forces allowed the use of the ACI 371R (1998) method. The area of steel should be 808 mm^2 over a height of 914 mm according to the calculations. The amount of steel present on top over a height of 711 mm is 2,272 mm^2. The required amount of steel for the bottom of the opening was the same as for the top and the area of steel present is 1,420 mm^2.

3.8.3.2 Sides of the Opening

The capacity of the sides of the main openings also had to be verified. They were assumed to perform as columns, and ACI 318 (2008) was therefore used. Recommendations of the ACI 371R (1998) were also followed for the applied loads and the width of the column. The design of the column was performed using SAP2000 which has a design function implemented. The capacity ratio was found to be 0.367. It was therefore concluded that there is capacity left in the sides of the main opening.

3.8.4 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 16. 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.

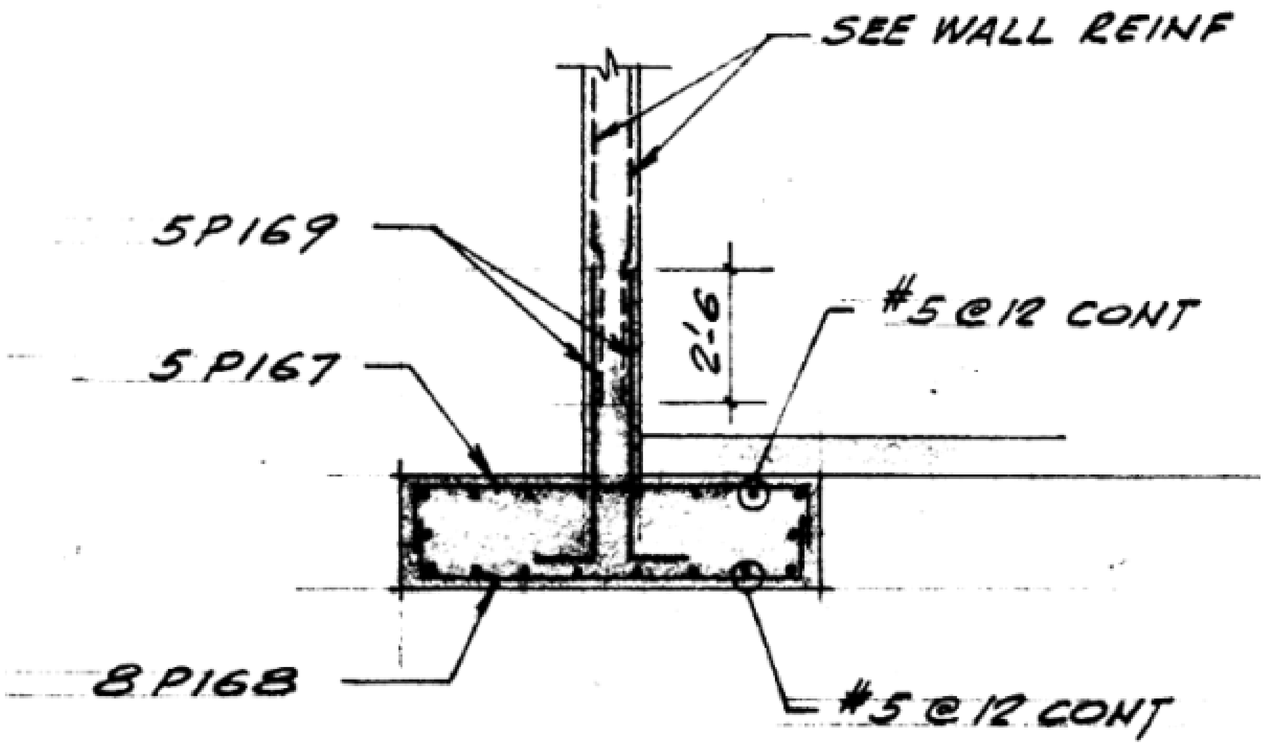

Figure 16: Foundation cross section and reinforcement detail (CFHT, 1974, with permission)

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.

$$V_u \leq \varphi V_n \tag{41}$$

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

$$V_u = P_u \left(\frac{B - c - 2d}{B} \right) \tag{42}$$

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:

$$V_n = V_c = \frac{1}{6} 2bd\sqrt{f'_c} \tag{43}$$

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_s) were evaluated for each load combination at each section. The next equations (Equations 44 and 45) from Coduto (2001) can be used.

$$A_s = \left(\frac{f'_c b}{1.176 f_y}\right)(d - \sqrt{d^2 - \frac{2.353 M_{uc}}{\varphi f'cb}} \tag{44}$$

$$M_{uc} = b\left(\frac{\left(\frac{P_u}{b}\right) l^2}{2B} + \frac{2\left(\frac{M_u}{b}\right) l}{B}\right) \tag{45}$$

Where M_{uc} is the factored moment at critical section, M_u is the factored applied moment load perpendicular to wall, φ 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.

3.8.5 Foundation

3.8.5.1 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 17 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 17) and the minimum deflection was calculated with the least steep curve (Test #1 on Figure 17). 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.

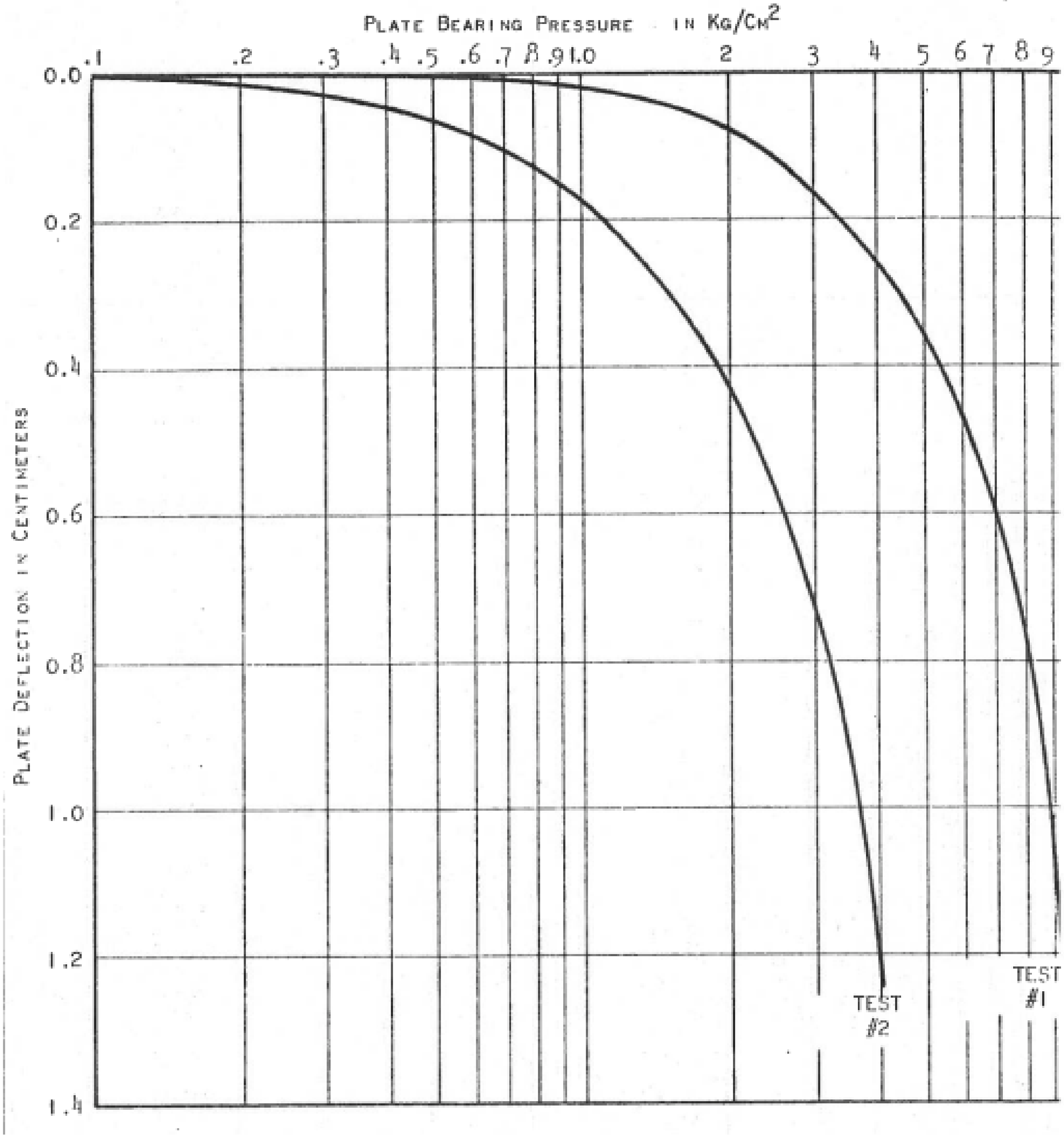

Figure 17: Plate bearing deflection curves (Dames & Moore, 1974, with permission)

It can be observed on Figure 17 that there is significant variability in the results of both tests. This suggests that more testing would be required.

As stated earlier, even if he 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. For further design, more refined methods should be used to evaluate the differential settlements, but additional data of the soil in place would be required.

3.8.5.2 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 to 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 18 for a representation of a general shear failure.

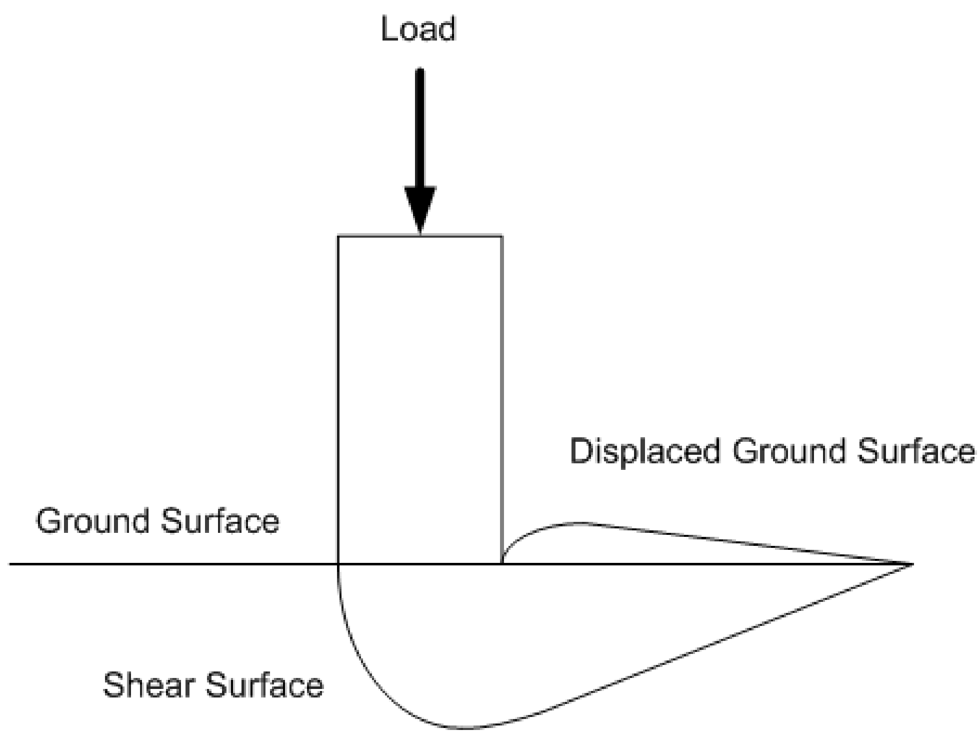

Figure 18: General shear failure of a shallow foundation

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_{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_{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.

$$q_{equiv} = \frac{P + W_f}{B'D} - u_d < q_{cap} \tag{46}$$

$$B' = B - 2e \tag{47}$$

$$e = \frac{M}{P + W_f} \tag{48}$$

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.

3.9 CFHT Pier Building Evaluation

The pier concrete structure, its footings and the foundations were evaluated for their possible reuse. The project is to replace the current telescope with a next generation telescope that has different geometry and mass.

The evaluation of the capacity of each member of the structure allowed for a global evaluation of the CFHT pier building. Failures were only observed in the soils where the pressure induced by the footing exceeded the bearing capacity. This type of failure can lead to a global failure of the entire building and has to be taken into account. The other members of the structure, the walls, slabs, openings, footings were all considered to have sufficient capacity to support the next generation CFHT telescope.

Since the bearing capacity of the soils was found to be exceeded under earthquake loads, it is recommended that a geotechnical investigation be conducted to reevaluate the soil parameters and verify if the pressure produced by the footing during earthquakes is low enough to be resisted by the soils.

4 CONCLUSIONS

Reusing and recycling structures can results in important economic, social and environmental benefits. The first step of the process is to evaluate the structural and geotechnical capacities. Through a case study, this document presents a methodology illustrating the evaluation of a structure that is considered to be reused. The Canada-France-Hawaii telescope organization wishes to upgrade the current 3.6 m telescope to a possibly 10-12 m telescope. The first step was to insure that the current concrete pier structure can support the geometry and mass of the new telescope. The pier was originally designed in 1974. Building codes have evolved since, especially regarding the seismic design rules. The pier building of the CFHT was analyzed for the next generation telescope according to the modern requirements for structural design.

First a methodology was developed to evaluate the capacity of the different members and loads applied to them. Then, a description of the structure, foundation and soil was realized. 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 shear capacity, bending capacity, steel reinforcement, 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 10.

Overall, it is concluded that:

- The walls, slabs and footings were considered to have sufficient structural capacity.
- 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%. 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.
- If the next generation CFHT is planned to be built, a geotechnical investigation should be performed to reevaluate the soil parameters and foundation capacity.

Table 10: CFHT pier building evaluation results summary

	Demand	Capacity	Ratio force vs capacity
Bending capacity of the pier			
At foundation	54280 kN.m	142843 kN.m	0.38
Shear capacity Method 1 – Straight walls idealization			
At foundation	3856 kN	10560 kN	0.48
Wall without opening	1364 kN	5280 kN	0.34
Wall with opening	2398 kN	2004 kN	1.06
Footing bending structural capacity			
Required steel ratio	0.2%	0.27%	0.74
Settlements – Plate load test evaluation			
Gravity load combination	3.6 mm	10.0 mm	0.36
Foundation allowable pressure			
Earthquake load combination	254 kPa	191 kPa	1.33
Gravity load combination	195 kPa	191 kPa	1.02

This study is only the first part of the project leading to the upgrade of the CFHT observatory. The new telescope will need a new support frame and will require a ring girder supporting the bearings. The next generation instrument being larger, it will also require a new enclosure. In the context of the project, it is desirable to conserve the steel frame walls of the enclosure on which the dome is sitting. The following aspects ought to be studied in the future:

- Design of a new enclosure that will fit the new telescope and have its base supported by the current enclosure walls. The type of enclosure, its dimensions and geometry will have to be analyzed with care to get the most cost and performance efficient design.
- The steel frame enclosure walls will have to be assessed to determine if they can support the loads created by the new enclosure as well as by the modern code requirements.
- The foundations supporting the enclosure and enclosure walls will also need to be evaluated due to the modern code requirements and different loads.
- The next generation CFHT telescope and ring girder will also necessitate to be designed in such a way that it will fit the concrete pier and the next generation telescope.

REFERENCES

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

ACI 371R-98. (1998). “Guide for the Analysis, Design, and Construction of Concrete-Pedestal Water Towers.” American Concrete Institute, Farmington Hills, Michigan, USA.

American Society of Civil Engineers. (1999). “Guideline for Structural Condition Assessment of Existing Buildings - ASCE 11.” Reston, Virginia, USA.

American Society of Civil Engineers. (2005). “Minimum Design Loads of Buildings and Other Structures – ASCE 7.” Reston, Virginia, USA.

Bentz E. and Collins M.P. (2000), “Response-2000, Version 1.0.5, Reinforced Concrete Sectional Analysis using the Modified Compression Field Theory.” University of Toronto, Ontario, Canada.

Bracci, J. M., Kunnath, S. K., Reinhorn, A. M. (1997) “Seismic Performance and Retrofit Evaluation of Reinforced Concrete Structures.“ Journal of Structural Engineering, Vol. 123, No. 1.

CFHT. (1974). “Pier Building Drawings.” Surveyer Nenniger and Chênevert Inc.

Cantell, S. F. (2005). “The Adaptive Reuse of Historic Industrial Buildings: Regulation Barriers, Best Pratices and Case Studies.“ Virginia Polytechnic Institute and State University, blacksburg, Virginia, USA.

Chopra, A. (2007). “Dynamics of Structures – Theory and Applications to earthquake Engineering.” Prentice-Hall, New Jersey, USA.

Coduto, D.P. (2001). “Foundation Design – Principles and Practices.” Prentice-Hall, New Jersey, USA.

Computers and Structures, Inc., Berkley. (2009). “Structural Analysis Program – SAP2000 Version 14.” Berkley, California, USA.

Computers and Structures, Inc., Berkley. (2006). “Concrete Shell Reinforcement Design Technical Note and Design Information.” Berkley, California, USA.

Computers and Structures, Inc., Berkley. (1999). “ETABS User’s Manual.” Berkley, California, USA.

Computers and Structures, Inc., Berkley. (1998). “SAP2000 - Basic Analysis Reference.” Berkley, California, USA.

Dames and Moore. (1973). “Foundation Investigation Report.” Hawaii, USA.

Dames and Moore. (1974), “Final Investigation Report.” Hawaii, USA.

IBC. (2006). “International Building Code.” International Code Council, Country Club Hills, Illinois, USA.

Day, R. W. (1999). “Geotechnical and Foundation Engineering: Design and Construction.” McGraw-Hill, New York, USA.

Grundmann, W. (1997). “A Canada-France-Hawaii 12-16m Telescope Study.”

Langston, C., Wong, F. K. W., Hui, E. C. M., Shen, L-Y. (2008) “Strategic Assessment of Building Adaptive Reuse Opportunities in Hong Kong.“ Building and Environment, Vol 43, pp. 1709-1718.

Pillai, S. U., Marie-Anne Erki, and D. W. Kirk. (2008). “Reinforced Concrete Design.” Tata McGraw-Hill, Toronto, Ontario, Canada.

Priestley, M. J. N., Seible, F. and Calvi, G. M. (1996). “Seismic Design and Retrofit of Bridges.” John Wiley and Sons, Inc. New York, NY, USA.

Stavridis, L. T. (2000) “Simplified Analysis of Layered Soil-Structure Interaction.” Journal of Structure Engineering, ASCE, February: 224-230.

Terzaghi, K. and Peck, R. (1967). “Soil Mechanics in Engineering Practice 2nd ed..” John Wiley, New York, New York, USA.

Vion, P. and Deschamps, D. (2010). “L’expertise et la réparation du pont en arc de Maamelten (Liban).” 17e Colloque sur la progression de la recherche québécois sur les ouvrages d’art, Québec, Canada, 10-11 May 2010.

Winkler, E. (1867). “Die Lehre von der Elasticitaet und Festigkeit.” 1867, Prag, Dominicus