REPORT ON
PHASE I AND II SEISMIC EVALUATION
CANADIAN HIGH COMMISSION
BRIDGETOWN, BARBADOS

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REPORT ON
ASCE 31-03 TIER 1 AND TIER 2 SEISMIC EVALUATION
CANADIAN HIGH COMMISSION
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EXECUTIVE SUMMARY

A seismic evaluation was performed for the Canadian High Commission in Bridgetown, Barbados.

A site visit was performed March 8th and 9th, 2010, to visually verify the "as built" construction of the High Commission and assess the overall condition. At the request of DFAIT no exploratory openings or intrusive testing was performed during the site visit. The Seismic Force Resisting System (SFRS) was analyzed and evaluated based on the evaluation procedure detailed in ASCE 31-03 and modified by DFAIT's Terms of Reference, incorporating calculations as specified in the 2005 NBCC. The structural drawings were available and provided most of the details required to perform the seismic evaluation; however, where necessary conservative assumptions were made. A modal response spectrum dynamic analysis was performed.

The Bridgetown High Commission consists of steel braced frames with HSS columns, structural steel beams, structural steel diagonal braces and deck diaphragms that are metal deck or composite metal deck with reinforced concrete topping. The seismic force resisting system consists of the HSS columns, structural steel beams, and the structural steel diagonal braces acting as braced frames. The geometry of the structural steel elements was confirmed, where possible, during the site visit.

The results of the analysis indicate that the braced frames do not have sufficient capacity to resist the design seismic event. The results of the Tier 2 Immediate Occupancy analysis indicate the structure would likely suffer more damage than acceptable. Specifically, the strength of the steel deck diaphragms was found to be inadequate to transfer loads to the SFRS, the diagonal bracing members were found to have inadequate capacity, and the foundations were found to have inadequate uplift capacity.

Based on the results of the analysis, several key load path upgrades are considered to be requirements for this structure. These upgrades include strengthening the steel deck diaphragms at the second storey and roof levels, the addition of horizontal braces to the isolated

portion of the steel roof deck, and the upgrade of foundations with rock anchors. These upgrades are required regardless of how the brace capacity is addressed.

Two seismic retrofit options were investigated for the High Commission, including the replacement of existing diagonal braces and the addition of extra braced frames. Each retrofit option was examined from the standpoint of the retrofitted Lateral Force Resisting System being able to resist 60%, 100%, and 150% of the applied seismic forces. The additional benefit of employing friction dampers in each of the retrofit solutions was considered.

Overall, it is recommended that the addition of new braced frames (either with or without friction dampers) and the replacement of the existing diagonal braces be pursued for the seismic retrofit of the High Commission. If this strategy is employed with friction dampers, the effects of the retrofit in terms of cost, schedule and intrusiveness to the building occupants will be minimized.

Finally, a number of both structural and non-structural items identified in the completed ASCE 31-03 Checklists were found to be deficient. These items should be addressed and corrective action taken where necessary.

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1.0 INTRODUCTION

The Canadian High Commission is located at Bishop's Court Hill, St. Michael, Bridgetown, Barbados. This report presents the results of a detailed seismic evaluation of the structure, required by DFAIT, as part of the Group B Seismic Evaluations of the Americas. The Bridgetown High Commission is listed as a Group 1 infrastructure and is evaluated based on the Immediate Occupancy (IO) performance criteria.

This report was prepared in accordance with the Standing Offer Agreement SO-ARP-SEISMIC-003AMS and Project Number SRDSS-100/TBSS-100, signed March 5, 2009 and J.L. Richards & Associates Limited's proposal dated February 9, 2010.

The objective of this report is to determine the seismic performance of the structure, based on the evaluation procedure detailed in ASCE 31-03 and modified by DFAIT's Terms of Reference, incorporating calculations as specified in the 2005 NBCC. Seismic retrofit options are also proposed if mitigation strategies are found to be necessary based on the performance of the building.

The scope of work for this project, as defined by DFAIT, is as follows:

- Seismic Evaluation in accordance with ASCE/SEI 31-03 as modified in the Statement of Work. Group 1 infrastructure is to be evaluated based on an Immediate Occupancy (IO) criteria. The details of the seismic evaluation procedure are provided in Appendix A.

- Perform a Tier 1 and Tier 2 evaluation.

- Review all existing documentation made available by DFAIT.

- For Group 1 missions, plan and initiate exploratory demolition openings to verify existing structural details, structural condition, material properties, etc

- Develop a 3-D structural model and perform a linear dynamic analysis.

- Verify Site Class/Seismic Hazard, assume a Site Class D where no information is provided.

- Calculate lateral load capacity and compare to the equivalent static base shear calculated using the National Building Code of Canada 2005 (NBCC 2005).

- For a Group 1 infrastructure, review seismic performance of non-structural elements.

- Develop seismic retrofit options. Each seismic retrofit option is to provide a lateral force resisting system capable of supporting 100% of the forces, as required by the 2005 NBCC.

- Investigate the implications of increasing or decreasing the seismic resistance to 150% or 60%, respectively, of the 2005 NBCC requirements.

- Propose upgrade options for non-structural building components.

- Prepare concept drawings and sketches of each of the proposed seismic retrofit options.

- Prepare a written report and attend a meeting in Ottawa to review the draft report.

The structural evaluation included a review of the existing structural drawings, a site visit to confirm that the structure generally conforms to the drawings provided, and to inspect the structure for any visible signs of distress, the development of a 3-D model and dynamic analysis, evaluation of the structural analysis and preparation of this report.

2.0 DESCRIPTION OF THE BUILDING AND SITE

2.1 Building Description

It is understood that the Bridgetown High Commission is owned by DFAIT and the level of performance is specified as Immediate Occupancy. The structure was constructed in 1984 and is two storeys in height above grade with one partial basement level. The approximate plan dimensions of the structure are 26 metres by 25 metres.

The height from the ground floor to the second floor is approximately 3.6 metres and the height from the second floor to the roof is approximately 3.4 metres. The total structure height is approximately 7 metres above grade.

Based on J.L. Richards & Associates Limited's site visit and the structural drawings, the structure consists of steel columns, steel joists, concrete floor slabs on metal decks and reinforced concrete footings. The structure is finished with mortar or plaster panels and glazing. The interior finishes are varied and consist of glazing, concrete masonry and gypsum wall board partitions.

2.2 Site Description

The High Commission is located in St. Michael's Parish of Barbados. Figure 2, a modified Google Earth image, shows the estimated extents of the property and the arrangement of the three primary buildings on the site. The dimensions of the property are approximately 115 m x 150 m in plan. It can also be seen in Figure 1 that a significant amount of the property is devoted to green space.

Figure 1: Site Layout

2.3 Seismic History

Barbados is considered to be a high seismic activity zone. The U.S. Geological Survey (USGS) estimates that Bridgetown can expect one earthquake with peak ground acceleration between 1.6 and 2.4 metres^2/second every 475 years. Since the High Commission was constructed in 1984, there have been several significant earthquakes in the Caribbean, including the Cayman Islands M6.8 2004, Martinique M7.4 2007 and notably Haiti M7.0 2010 (USGS). Given the seismicity of the region, the High Commission has likely been subjected to the effects of a number of seismic events over the course of its lifetime.

It should be understood that extrapolating the future performance of a structure based on past seismic events can be very misleading. A key component to be understood is the nature of the ground motions in the past events and the concentrations of the ground motion frequencies. A structure that has been subjected to previous, large magnitude, seismic events and appears to have performed well may either be 1) inherently structurally sound; or 2) may not have been subjected to ground motions that are rich in frequencies matching the natural frequencies of the building structure. The past performance of a structure under seismic load is relevant and should be considered carefully, but is no guarantee of satisfactory future behaviour.

2.4 Site Observations

Ms. Jennifer Stephenson, P.Eng., visited the Bridgetown High Commission on March 8th and 9th, 2010. The purpose of the site visit was to visually verify the “as built” construction of the High Commission and assess the overall condition. At the request of DFAIT no exploratory openings or intrusive testing was performed during the site visit.

The “as built” condition was ascertained by collecting a broad range of measurements of key dimensions, as well as observations during the site visit. A description of the observations from the site visit is provided below.

Structural Observations Foundations and Basement Level

The foundations were not visible. The basement slab and foundation walls were visible in a crawlspace accessed at the basement level. The basement walls consist of concrete masonry, as shown on the drawings. The HSS columns rest on base plates secured with two anchor bolts to the foundations. The concrete slab in the crawlspace appeared to be in good condition. The concrete masonry walls had areas of efflorescent staining, no other defects were noted. Photograph Nos. 1 and 2 show the basement foundation walls and efflorescence.

Photograph No. 1 – Basement Foundation Wall Efflorescence

Photograph No. 2 - Basement Foundation Wall Efflorescence

At the basement level, corrosion of a steel beam and open web steel joist (OWSJ) were noted in the supply room. The corrosion is thought to be caused by moisture from an air conditioning unit. Maintenance staff is aware of the problem and have removed the ceiling panels in this area as shown in Photograph Nos. 3 and 4. No other damage or deterioration was noted at the basement level.

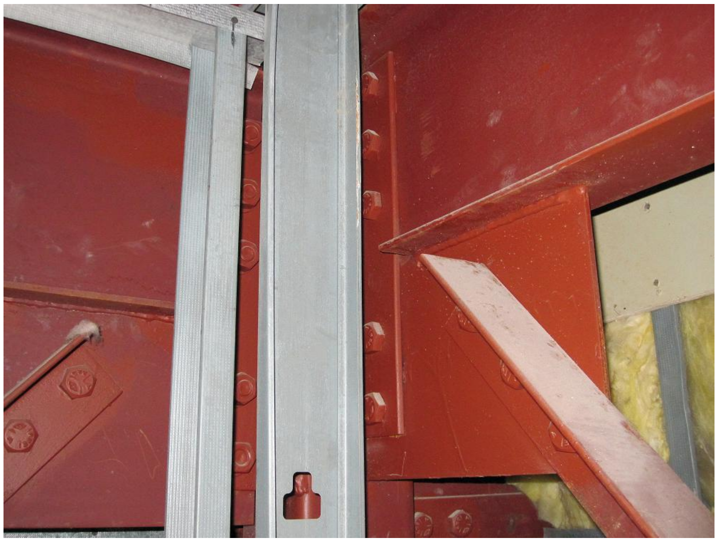
Photograph No. 3 – Basement Level Member Corrosion

Photograph No. 4 – Basement Level Member Corrosion

<u>Structural Observations Ground Floor, Second Floor and Roof</u>

The structural steel columns, beams, bracing and metal deck floor were made visible by selectively removing the suspended acoustic ceiling panels on the ground and second floors. The columns, beams and bracing were measured in several locations on both the ground and first floor. Measurements were taken to verify the steel framing size and found to correspond to the sizes indicated on the structural drawings. No corrosion or damage was noted in the steel structure with the exception of the corrosion in the maintenance supply room noted in the basement level. Examples of the structural steel columns and connections are presented in Photograph No. 5. Examples of the structural steel bracing and connections are presented in Photograph No. 6.

Photograph 5 - : Structural Steel Column and Connection



Photograph No. 6 – Structural Steel Column and Connection

Concrete masonry partition walls that are unsupported at the top of the wall were noted, as shown in Photograph No. 7.

Photograph 7: Unsupported Concrete Masonry Partition Wall

The concrete slab on metal deck floor system and OWSJ, were observed to be as indicated on the structural drawings.

<u>Exterior Façade</u>

The exterior façade consists of glazing and concrete or masonry panels approximately 8 mm thick. No damage or deterioration was visible on the exterior façade. Photograph Nos. 8 and 9 present photographs of the exterior façade.

Photograph No. 8 - Exterior Façade

Photograph 9: Exterior Façade

In summary, the site visit did not produce any observations that would suggest that the condition of the structure should be considered in the assessment of the seismic capacity. More details regarding the site visit are presented in Section 4.0 Checklist Results.

3.0 DESIGN DATA / EXISTING DOCUMENTATION

The following documents were available for our review:

- Structural Drawings provided by DFAIT, prepared by Consortium CRS dated May 11, 1984.
- Rapid Seismic Screening provided by DFAIT, prepared by SNC Lavalin dated June 3, 2006.
- Geotechnical Report provided by DFAIT, prepared by Dessau dated April 2008.

The spectral acceleration values listed below were obtained from DFAIT. Based on the geotechnical information available, a Site Class C was assumed. The final results of the geotechnical investigation indicated that the site class is a Site Class A which would reduce the seismic forces presented by 20 percent. Details of the analysis method are described in Appendix A.

Spectral Period	Spectral Acceleration	Site Class	Fa	Fv	SDS	SD1
0.2	1.263	C	1	1.3	0.76	0.31
0.5	1.005					
1	0.517					
2	0.287					
PGA	0.545					

Table 1: Seismic Hazard Data

The seismic hazard curve for the spectral acceleration coordinates provided by DFAIT Site Class C, is presented in Figure 2.

Figure 2: Seismic Hazard Curve Bridgetown, Site Class C

4.0 CHECKLIST RESULTS

The following checklists from ASCE Standard 31-03 were performed:

- 3.7.4 Basic Structural Checklist for Building Type S2: Steel Braced Frames with Stiff Diaphragms.
- 3.7.4S Supplemental Structural Checklist for Building Type S2: Steel Braced Frames with Stiff Diaphragms.
- 3.8 Geologic Site Hazards and Foundations Checklist.
- 3.9.1 Basic Nonstructural Component Checklist.
- 3.9.2 Intermediate Nonstructural Component Checklist.
- 3.9.3 Supplemental Nonstructural Component Checklist.

ASCE-31-03 requires that in areas of high seismicity the Basic Structural, Supplemental Structural, Geologic Hazards and Foundations, Basic Nonstructural, Intermediate Nonstructural and Supplemental Non-Structural Checklists be performed for the immediate occupancy performance objective.

Any deficient items or items that could not be determined are discussed in order below. It should be noted that items that could not be determined at the time of the inspection are noted as non-compliant. The completed checklist forms are attached in Appendix C.

3.7.4 Basic Structural Checklist

Torsion: The roof has two separate diaphragms. The estimated distance between the smaller diaphragm's centre of mass and its centre of rigidity is greater than 20% of the smaller plan dimension of the overall building (ASCE 31-03 4.3.2.6).

Deterioration of Steel*:* There is visible rusting of a steel beam and joist in the maintenance supply room.

Redundancy: The number of braced bays in each line is less than 3, as required for immediate occupancy (ASCE 31-03, Section 4.4.3.2.2).

Axial Stress Check: The axial stress in the diagonal bracing members was found to be greater than the maximum allowable stress according to ASCE 31-03, Section 3.5.3.4.

Column Splices: Information about column splice details is not known.

Transfer to Steel Frames: Details of the connection from the diaphragms to the steel frames are not known.

Steel Columns: The steel column anchorage was found to not be able to develop the uplift capacity of the foundation as required for immediate occupancy (ASCE 31-03, Section 4.6.3.1).

3.7.4S Supplemental Structural Checklist

Slenderness of Diagonals: Diagonal elements KL/r ratios were all greater than the maximum value of 120 (ASCE 31-03, Section 4.4.3.1.4). Diagonal elements were analyzed based on the “tension only” assumption.

Connection Strength: Information about connection details is not known.

3.8 Geologic Site Hazards and Foundations Checklist

Liquefaction / Surface Fault Rupture: The liquefaction and surface fault potential are unknown.

Overturning: The ratio of the horizontal dimension of the lateral-force-resisting-system at the foundation level to the building height was found to be less than the minimum value of 0.6 S_a as specified in ASCE-31-03, Section 4.7.3.2, therefore the braced bays are vulnerable to overturning.

3.9.1 Basic Nonstructural Component Checklist

Unreinforced Masonry: Unreinforced concrete masonry units are not braced.

Cladding and Glazing: The details of the cladding anchors, cladding isolation, connections and inserts are not known.

Attached Equipment: Mechanical equipment weighing over 20 lbs. was not braced.

Fire Suppression Piping: Fire suppression piping was not braced.

Flexible Couplings: There were no flexible couplings noted on the fire suppression system.

3.9.2 Intermediate Nonstructural Component Checklist

Glazing: The exterior window glazing has no safety film.

3.9.3 Supplemental Nonstructural Component Checklist

Tops*:* The tops of partitions that extend to the ceiling line are not laterally braced to the building structure.

Edges*:* The edges of integrated suspended ceilings were not separated from the enclosed walls by a minimum of half an inch.

Glazing: The exterior glazing is not laminated or heat-strengthened safety glass.

Cabinet Doors and Drawers: Cabinet doors and drawers do not always have latches to keep them closed during an earthquake.

Electrical Equipment: Electrical equipment and associated wiring is not laterally braced.

Fluid and Gas Piping: Fluid and gas piping was not braced.

Shut off Valves: Details of any required shut-off valves is not known.

5.0 ANALYSIS PRODEDURE

The Bridgetown High Commission was analyzed and evaluated based on the evaluation procedure detailed in ASCE 31-03 and modified by DFAIT's Terms of Reference, incorporating calculations as specified in the 2005 NBCC. Structural calculations required by the ASCE 31-03 Tier 1 and 2 analyses were performed when required. The Seismic Force Resisting System (SFRS) was evaluated using the Dynamic Analysis Procedure, as described in the 2005 NBCC. A 3D model of the High Commission was developed to complete the required analysis. Renderings of the High Commission structure, generated using the model, have been provided in Appendix D. The results of the dynamic analysis were compared to the results obtained using the Equivalent Static Force Procedure (ESFP) as described in the 2005 NBCC. The main parameters used to complete the ESFP are outlined below:

- The material overstrength and ductility factors, R_d and R_o, were taken as 1.5 and 1.3 respectively for conventional construction of braced frames.

- As per Clause 4.1.8.11 (3), the period of the structure was calculated as follows:
  - for the steel braced frame structure: T_a=0.025(h_n), T_a=0.18 seconds;
  - the periods calculated from the dynamic analysis were T_y=0.53s in the Y-direction and T_x=0.51 in the X-Direction;
  - from NBCC 4.1.8.11(3d) T_a must be no greater than 2(0.18 sec) = 0.36 seconds.
  - $S(T_a) = 1.263$.

- As per Clause 4.1.8.11 (5), the higher mode factor, M_v, was taken as 1.0.

- The importance factor, I_E, as per DFAIT's Terms of Reference, was taken as 1.0.

- The overall building weight was estimated to be 3,980 kN.

- The equivalent static base shear was calculated as follows:
  $V_o = S(T_a)M_vI_eW/(R_dR_o)$, V_o = 2,578 kN

- The maximum equivalent static base shear is as follows:
  $V_{max} = (2/3)\ S(0.2)\ I_e\ W\ /\ (R_dR_o)$, V_{max} = 1,719 kN

- The minimum equivalent static base shear is as follows:
  V_{min}=S(2.0) I_e W/($R_d R_o$); V_{min}=585 KN

The maximum equivalent static base shear governs and is distributed as storey shears as shown in Table 2 below.

Structure	Steel Braced Frames
Storey	Shear
Roof	1000 KN
2nd	719 KN

Table 2: Storey Shears

The results of the ESFP, as outlined above, were used to calibrate the results of the dynamic analysis results for the structure. As per the scope of services provided by DFAIT, a Linear Dynamic Analysis was completed for the structure. The factored base shears resulting from the dynamic analysis were factored to be 80% of the ESFP base shear as specified for regular structures in clause 4.1.8.12 (6).

The dynamic analysis of the structure was completed using the Modal Response Spectrum method. The dynamic mode shapes and frequencies of the structure were calculated and the first six periods are presented in Table 3 below.

	Steel Braced Frames
Mode	Period (sec)
1	0.53
2	0.51
3	0.34
4	0.21
5	0.20
6	0.13

Table 3: Dynamic Modes Shapes and Frequencies

The first mode has a mass participation in the Y-direction of 93% and the second mode has a mass participation in the X-direction of 92%. These values meet or exceed the 90%

minimum amount of mass participation recommended in Commentary J of the NBC 2005 Structural Commentaries.

To account for torsion, the lateral loads in the equivalent static and dynamic analysis were applied at 10% eccentricity from the centre of mass.

The material properties of the braced frames were taken from the structural drawings (Consortium CRS, 1984), the values used in the analysis and evaluation are presented in Table 4 below.

Material	Property	Value
Hollow Structural Steel	fy	350 MPa
	Es	200,000 MPa
Structural Steel	fy	300 MPa
	Es	200,000 MPa
Bolts	Fu	830 MPa

Table 4: Material Properties

6.0 RESULTS OF EVALUATION

The structural calculations were completed in general conformance to the NBCC 2005 and the material codes referenced therein. Structural calculations required for ASCE31-03 Tier 1 and 2 analyses were performed and are described below.

The following load combinations and factors were considered, as specified in the 2005 NBCC:

- 1.0D+0.5L+1.0E

The effective masses of the two storeys were checked and found to not change more than 50% as specified in ASCE 31-03, Section 4.3.2.5.

The estimated distance between the storey centre of mass and the storey centre of rigidity was checked and found to be greater than the maximum 20% of the minimum building width as specified in ASCE 31-03, Section 4.3.2.6. This is due to the smaller roof diaphragm which has a center of rigidity located on the outer edge of the building.

The Axial stress in the columns subjected to overturning forces was checked using the procedure specified in ASCE 31-03, Section 3.5.3.6, as follows:

$$p_{ot} = \frac{1}{m}\left(\frac{2}{3}\right)\left(\frac{Vh_n}{Ln_f}\right)\left(\frac{1}{A_{col}}\right) \leq 0.3F_y$$

Where V is the pseudo lateral force, h_n is the height to the roof level, m is 1.3 for the immediate occupancy performance level and n_f is the number of frames in the direction of loading. The axial stress was found to be less than the allowable limit.

The redundancy of the building was checked in accordance with ASCE 31-03, Section 4.4.3.1.1. It was found that the number of braced bays in each line of braced frames was inadequate for immediate occupancy.

The axial stresses in the diagonal bracing members were checked using the procedure specified in ASCE 31-03, Section 3.5.3.4, as follows:

$$f_j^{avg} = \frac{1}{m}\left(\frac{V_j}{sN_{br}}\right)\left(\frac{L_{br}}{A_{br}}\right) \leq 0.5F_y$$

Where V_j is the maximum storey shear at each level, L_{br} is the average length of the braces, m is 1.5 for the immediate occupancy performance level, s is the average span length of the braced spans, N_{br} is the number of diagonal braces in tension, and A_{br} is the average area of the diagonal brace. The axial stress was found to be greater than the allowable limit.

The anchorage of the steel columns was evaluated and compared to the tensile capacity of the columns and uplift capacity of the foundation. The connection of the steel columns in the lateral force resisting brace frames was found to be unable to develop either capacity as specified in ASCE 31-03, Section 4.6.3.1. A Tier 2 analysis was performed and the uplift/tension forces on the columns were compared to the capacity of the column anchorage. The uplift/tension forces in all of the columns were found to exceed the capacity of the anchorage.

The frame elements were checked for conformance to section requirements of ASCE 31-03, Section 4.4.1.3.7. The width/thickness ratios of all frame elements were found to be within the allowable range.

The slenderness of each of the diagonal bracing members was checked according to ASCE 31-03, Section 4.4.3.1.4. None of the bracing members were within the allowable range for carrying compression forces and the bracing was assumed to be tension only.

The base – height ratio of the lateral force resisting system was checked in comparison to 0.6 x $S_a(T_n)$ as specified in ASCE 31-03, Section 4.7.3.2. The ratio was found to be within the allowable limit.

The foundation elements are restrained by a strip footing and are therefore adequate for ASCE 31-03, Section 4.7.3.3.

The seismic force resisting system was evaluated as described in Section 6.1 below.

6.1 Building Element Analysis

The seismic force resisting system (SFRS) in the Bridgetown High Commission consists of braced frames made up of steel hollow structural sections (HSS) columns and W-section beams with diagonal L-shaped angle braces.

Lateral forces produced by a seismic event are applied to the structure at the centre of mass and are functions of the displacement of the structure (acceleration) and the inertial weight of the structure.

These lateral forces follow a load path from the floor or roof diaphragms of the structure through the SFRS to the foundations.

The roof and second floor diaphragms were evaluated for shear resistance capacity as follows:

Diaphragms

The structural drawings provided to JLR make no reference to seismic consideration in the design and the lateral braces are referred to specifically as wind braces. Lateral forces due to wind are applied to a building in a manner that is significantly different to that of seismic lateral forces. In a seismic design the storey diaphragms and diaphragm connections become considerably more significant to the performance of the building.

The upper clear storey roof with plan dimensions of 7 m x 8.5 m has no lateral support in the north-south direction. This roof section requires additional lateral bracing members placed along its east and west edges to transfer any lateral loading into the SFRS which is located at the exterior column lines. The analysis of the lateral diaphragms was based on the assumption that these additional braces will be installed.

The estimated strengths of the steel deck diaphragms were taken from the tabulated values included in the Design of Steel Deck Diaphragms (3^{rd} Edition) by Canadian Sheet Steel Building Institute (CSSBI).

The roof diaphragm consists of gauge 22 steel deck with 33 mm high flutes. The steel deck is button punched every 450 mm and welded to the joists at 450 mm centre to centre spacing. This connection pattern is not an acceptable pattern for the transfer of lateral loads. Canadian steel deck diaphragm producers do not consider patterns with spacing between welds less than 300 mm when creating tabulated steel deck capacities. Increasing this spacing results in a less stiff element with a lower shear capacity.

For the purposes of this report, the resultant shears of the applied loads were compared to a steel deck diaphragm with 300 mm weld spacing because there is tabulated data for this case. When the seismic loads within the steel deck diaphragm were compared to the higher capacity deck, the demand/capacity ratios of the diaphragm in shear were found to be greater than 1.0, indicating an overstressed condition. These calculations show that the in-situ steel deck, which has a lesser capacity than the published values, would not perform satisfactorily during the design seismic event.

The second floor diaphragm consists of a gauge 22 steel deck with 33 mm high flutes with a 50 mm concrete topping over the steel deck. The steel deck is button punched every 600 mm and welded at 600 mm centre to centre spacing. This connection pattern is also not acceptable by Canadian standards for the transfer of lateral loads.

The applied loads were compared to tabulated data for a steel deck diaphragm with 300 mm weld spacing and a 65 mm concrete topping. When the seismic loads within the steel deck diaphragm were compared to the higher capacity deck, the demand/capacity ratios of the diaphragm in shear were found to be greater than 1.0. These calculations show that the in-situ steel deck, which has a lesser capacity than the published values, would not perform satisfactorily during the design seismic event.

The existing perimeter chord member consists of an 89 mm x 50 mm x 3 mm angle. This member does not have adequate capacity to transfer the compression and tension forces from the diaphragm to the SFRS.

The existing deck diaphragms do not have adequate capacity to transfer the shear forces in the diaphragm to the SFRS, specifically around diaphragm openings, where the diaphragm width is reduced or in the case of the roof where the diaphragm is connected with light steel beams. A retrofit to improve the shear resistance of the steel deck diaphragms is required.

HSS Steel Columns

To evaluate the seismic demands on the HSS column members in the SFRS, it was assumed that the seismic forces could be transferred to the SFRS from the steel diaphragms. The applied axial force and moment for each load combination were compared to the moment capacity of the section at that axial load level. Interaction diagrams were produced to illustrate the allowable range of loading for the columns. An example of an interaction diagram for a typical column and load combinations is presented in Figure 3 below.

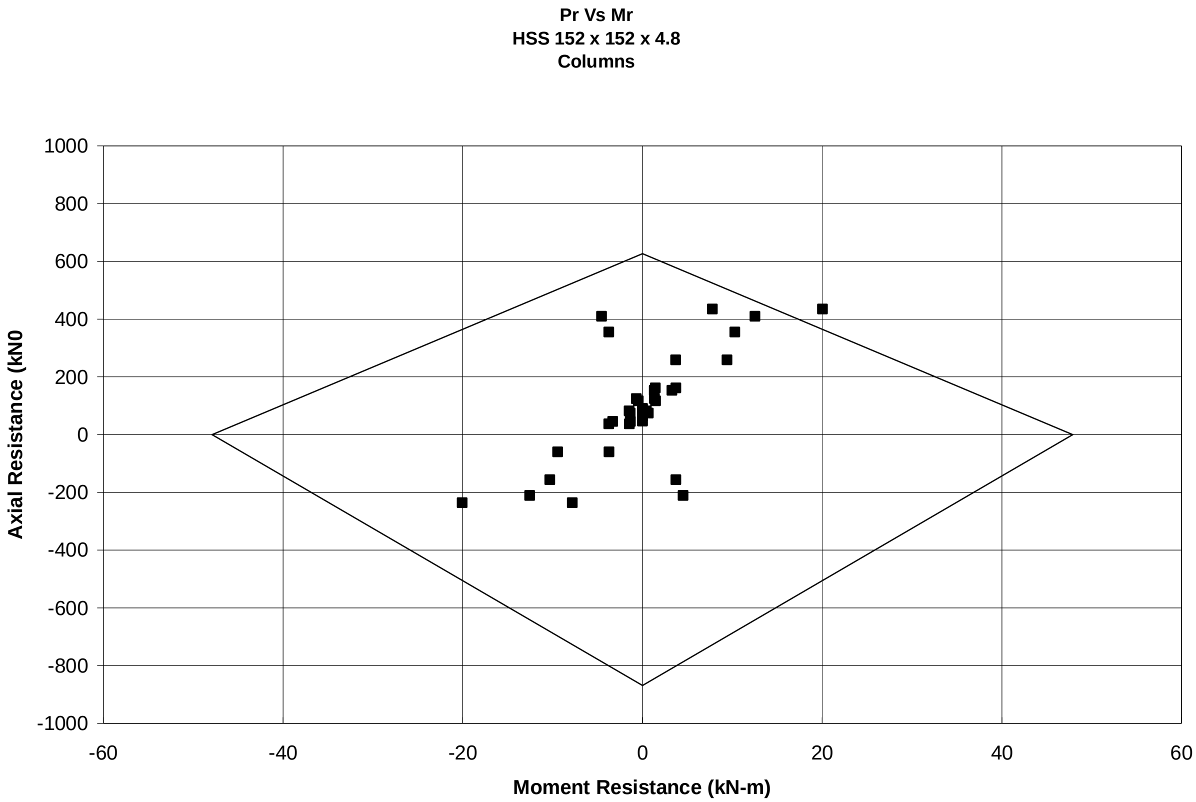


Figure 3: Interaction Diagram for HSS Columns

The interaction diagram in Figure 3 illustrates the allowable loading range for an HSS 152 x 152 x 4.8 section according to CAN/CSA S16-01. The lines represent the boundary of the allowable range of loading. Each point shown in the figure represents the loading of a column element under the governing load conditions. As can be seen, the column loading observed was generally within of the allowable limits, except for one load combination.

The location of these columns on the floor plan is shown in Figure 4. Structural steel capacities were evaluated in accordance with CAN/CSA S16-01 Limit States Design of Steel Structures.

The moment capacity of the steel section was calculated according to CAN/CSA S16-01 Limit States Design of Steel Structures clause 13.5, as follows:

$$M_r = \phi Z F_y$$

The axial capacity of the steel section was calculated for compression (C_r) and tension (T_r) respectively according to CAN/CSA S16-01 Limit States Design of Steel Structures, Clause 13.3.1, as follows:

$$C_r = \phi A F_y (1+\lambda^{2n})^{-1/n}$$

$$\lambda = \frac{Kl}{r}\sqrt{\frac{F_y}{\pi^2 E}}$$

$$T_r = \phi A_g F_y$$

$$\frac{M_f}{M_r} + \frac{P_f}{P_r} \leq 1.0$$

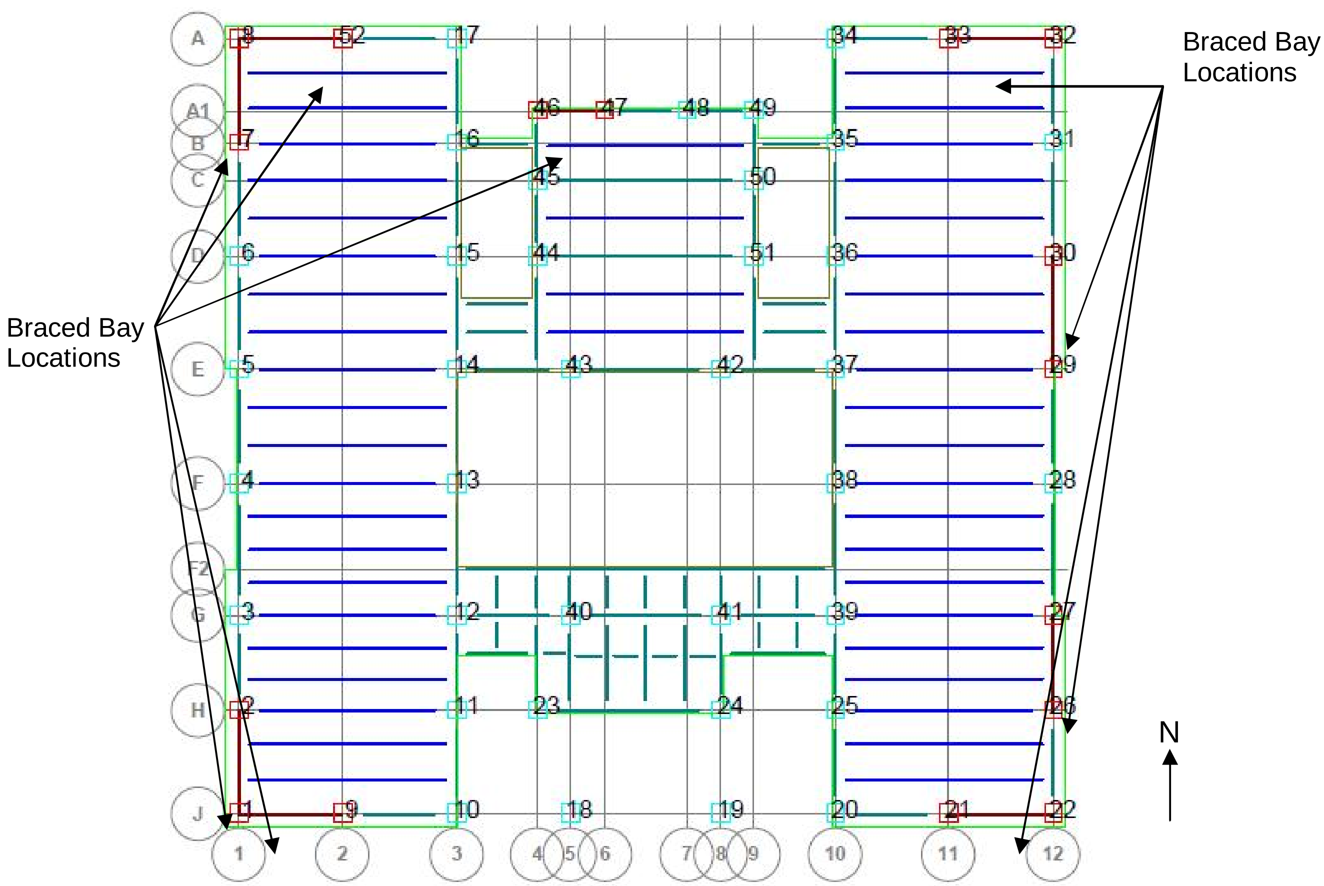


Figure 4: Column Locations (Second Floor and Roof)

Table 5 shows the demand/capacity ratios for the columns subjected to both axial and bending forces. Demand/Capacity ratios greater than 1 indicate columns with insufficient capacity for the design seismic loading.

Column Number‡	Floor Level	Demand/Capacity Ratios† (Cf/Cr or Tf/Tr) + Mf/Mr		
		60%	100% Life Safety	150% Immediate Occupancy
1	2^{nd} - Ground	0.66	0.96	1.51
2	2^{nd} - Ground	0.88	0.99	1.49
7	2^{nd} - Ground	0.71	0.93	1.38
8	2^{nd} - Ground	0.75	1.01	1.60
9	2^{nd} - Ground	1.01	0.98	1.45
52	2^{nd} - Ground	0.89	1.06	1.71
46	2^{nd} - Ground	0.57	0.86	1.38
47	2^{nd} - Ground	0.63	0.79	1.31
21	2^{nd} - Ground	0.66	0.91	1.45
33	2^{nd} - Ground	0.71	1.06	1.70
22	2^{nd} - Ground	1.06	1.03	1.50
26	2^{nd} - Ground	0.76	1.02	1.54
27	2^{nd} - Ground	0.93	1.04	1.55
29	2^{nd} - Ground	0.75	0.97	1.44
30	2^{nd} - Ground	0.92	0.97	1.43
32	2^{nd} - Ground	0.94	1.11	1.75
1	Roof - 2^{nd}	0.22	0.31	0.48
2	Roof - 2^{nd}	0.37	0.33	0.49
7	Roof - 2^{nd}	0.24	0.30	0.43
8	Roof - 2^{nd}	0.33	0.34	0.53
9	Roof - 2^{nd}	0.42	0.40	0.52
52	Roof - 2^{nd}	0.37	0.38	0.59
46	Roof - 2^{nd}	0.19	0.29	0.46
47	Roof - 2^{nd}	0.24	0.26	0.44
21	Roof - 2^{nd}	0.24	0.33	0.50
33	Roof - 2^{nd}	0.26	0.38	0.59
22	Roof - 2^{nd}	0.42	0.40	0.52
26	Roof - 2^{nd}	0.25	0.33	0.48
27	Roof - 2^{nd}	0.40	0.36	0.53
29	Roof - 2^{nd}	0.26	0.33	0.47
30	Roof - 2^{nd}	0.39	0.36	0.52
32	Roof - 2^{nd}	0.37	0.38	0.58

† *Demand/Capacity Ratio greater than one represents an overstressed condition.*

‡ *Refer to Figure 4 for the location of the columns.*

Table 5: Demand/Capacity Ratios for HSS Columns

As shown in Table 5 for the 100% Loading (Life Safety) Condition 5, the demand/capacity ratios for the columns range from 0.26 to 1.11 for the columns analyzed. The columns from the ground floor to the second floor have higher demand/capacity ratios than the columns from the second floor to the roof. The demand/capacity ratios for seven of the sixteen columns are over 1.0, the overstresses range from one percent to eleven percent. Considering the low overstress levels and the number of columns affected, these overstresses would probably not significantly affect the performance of the structure.

The demand/capacity ratios for the columns for the 60% and 150% conditions are shown for comparison. Demand/capacity ratios range from 0.19 to 1.06 for the 60% condition, with two columns overstressed. While demand/capacity ratios range from 0.43 to 1.75 for the 150% condition, with the majority of ground floor columns overstressed.

<u>Steel Beams</u>

The horizontal elements of the moment frames in the Bridgetown High Commission consist of steel beams. The flexural and shear capacities of the beams were calculated in accordance with CAN/CSA S16-01 Limit States Design of Steel Structures as follows:

Moment, Clause 13.5: $M_r = \phi Z F_y$

Shear, Clause 13.4: $V_r = \phi A_w F_s$

The demand/capacity ratios for the steel beams are presented in Table 6.

The location of these beams on the floor plan is shown in Figure 5.

Beam‡	Level	Demand/Capacity Ratio Vf/Vr†
1	2nd - Ground	0.02
2	2nd - Ground	0.00
3	2nd - Ground	0.02
4	2nd - Ground	0.16
5	2nd - Ground	0.13
6	2nd - Ground	0.02
7	2nd - Ground	0.02
8	2nd - Ground	0.15
9	2nd - Ground	0.15
1	Roof - 2nd	0.09
2	Roof - 2nd	0.00
3	Roof - 2nd	0.09
4	Roof - 2nd	0.15
5	Roof - 2nd	0.13
6	Roof - 2nd	0.09
7	Roof - 2nd	0.09
8	Roof - 2nd	0.12
9	Roof - 2nd	0.12

† *demand/capacity Ratio greater than one represents an overstressed condition.*

‡ *Refer to Figure 5 for location of the beams.*

Table 6: Demand/Capacity Ratios for Steel Beams

Table 6 shows the demand/capacity ratios for the 100 % (Life Safety) load condition. The demand/capacity ratios for the beams in shear are all less than 1 and the beams have negligible bending. The beams likely have enough overstrength and ductility to allow for the inelastic deformations required by the design seismic event. The demand/capacity ratios indicate that the beams have sufficient capacity for all three design scenarios, i.e. 60%, 100% and 150%. No details are provided for the beam – column connections so the adequacy of the connection can not be commented on.

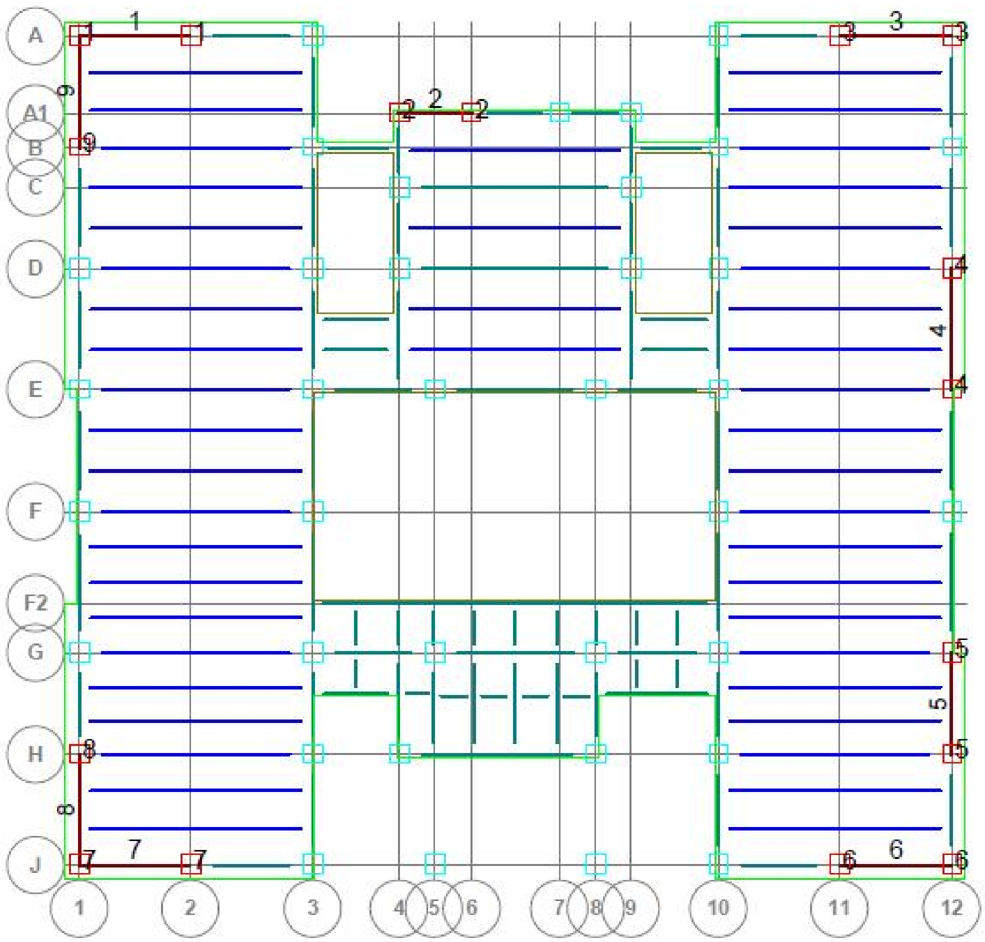


Figure 5: Braced Frame and Beam Locations (Second Floor and Roof)

<u>Diagonal Braces</u>

The diagonal braces in the SFRS are L76 x 76 x 6.4 members in Frames 1 and 3-9 and L51 x 51 x 6.4 members in Frame 2. The capacities of the braces were calculated in accordance with CAN/CSA S16-01 Limit States Design of Steel Structures as follows:

Clause 13.2 (a(i)): $T_r = \phi A_g F_y$

The tensile capacity is based solely on the member strength since no connection detail was provided for analysis. The strength of the connection should be designed to resist approximately 1.1 times the strength of the brace in order to ensure that failure occurs in the brace.

The slenderness of each of the diagonal bracing members was checked according to ASCE 31-03, Section 4.4.3.1.4 and none of the bracing members were within the allowable range for carrying compression forces. Because of this the braces are considered to be purely tension members.

The demand/capacity ratios for the diagonal steel braces in tension are shown in Table 7. The braces discussed in Table 7 are numbered such that the first number is the braced bay number and the second number denotes either the first or second diagonal for that braced bay at a particular storey level (i.e., Brace (2,1) Roof – 2nd is the first diagonal brace in the second braced bay from the roof to the second storey).

Table 7 shows that the demand/capacity ratios for the braces in tension range from 0.30 to 2.09. Seventeen out of 36, or 47% of the braces have demand capacity ratios greater than 1.0 indicating an overstressed condition. Demand/capacity ratios for the 60% and 150% load conditions are shown for comparison. The demand/capacity ratios range from 0.19 to 1.25 for the 60% load condition and 0.46 to 3.13 for the 150% load condition.

Brace Number‡	Level	Demand / Capacity Ratio†		
		60%	**100% Life Safety**	**150% Immediate Occupancy**
1,1	2nd - Ground	0.34	0.54	0.82
1,2	2nd - Ground	0.67	1.12	1.68
1,1	Roof - 2nd	0.56	0.90	1.37
1,2	Roof - 2nd	1.04	1.74	2.61
2,1	2nd - Ground	0.33	0.53	0.81
2,2	2nd - Ground	0.67	1.11	1.67
2,1	Roof - 2nd	0.55	0.89	1.35
2,2	Roof - 2nd	1.04	1.74	2.61
3,1	2nd - Ground	0.19	0.30	0.46
3,2	2nd - Ground	0.38	0.64	0.96
3,1	Roof - 2nd	0.31	0.50	0.76
3,2	Roof - 2nd	0.59	0.99	1.49
4,1	2nd - Ground	0.40	0.66	1.00

Brace Number‡	Level	Demand / Capacity Ratio†		
		60%	**100% Life Safety**	**150% Immediate Occupancy**
4,2	2^{nd} - Ground	0.80	1.33	1.99
4,1	Roof - 2^{nd}	0.64	1.04	1.58
4,2	Roof - 2^{nd}	1.21	2.02	3.02
5,1	2^{nd} - Ground	0.43	0.70	1.06
5,2	2^{nd} - Ground	0.85	1.42	2.12
5,1	Roof - 2^{nd}	0.66	1.07	1.63
5,2	Roof - 2^{nd}	1.25	2.09	3.13
6,1	2^{nd} - Ground	0.29	0.48	0.73
6,2	2^{nd} - Ground	0.78	1.07	1.60
6,1	Roof - 2^{nd}	0.47	0.76	1.15
6,2	Roof - 2^{nd}	1.19	1.62	2.43
7,1	2^{nd} - Ground	0.31	0.50	0.76
7,2	2^{nd} - Ground	0.80	1.07	1.60
7,1	Roof - 2^{nd}	0.48	0.78	1.19
7,2	Roof - 2^{nd}	1.19	1.62	2.43
8,1	2^{nd} - Ground	0.37	0.64	0.95
8,2	2^{nd} - Ground	0.78	1.07	1.60
8,1	Roof - 2^{nd}	0.62	1.06	1.58
8,2	Roof - 2^{nd}	1.23	1.65	2.48
9,1	2^{nd} - Ground	0.39	0.66	0.99
9,2	2^{nd} - Ground	0.34	0.54	0.82
9,1	Roof - 2^{nd}	0.67	1.12	1.68
9,2	Roof - 2^{nd}	0.56	0.90	1.37

† *Demand/Capacity Ratio greater than one represents an overstressed condition.*
‡ *Refer to Figure 6 for location of the braced frames.*

Table 7: Demand/Capacity Ratios for Diagonal Braces in Tension

Foundations

The uplift capacity of the foundations was found to be approximately 30 kN. This capacity was found by considering the dead load associated with each footing supporting the braced columns. The dead loads consider the weight of the concrete footing, a portion of the strip footing spanning between individual footings, a portion of the concrete block wall supported by the strip footings, and any soil engaged by the footings in uplift. The governing uplift force under the design seismic event was found to be approximately 510 kN. This corresponds to a demand/capacity ratio in uplift that is over 17. This shows clearly that the current foundations are inadequate to resist the uplift caused by the design seismic event.

The bearing capacity of the soil beneath the foundations of the High Commission is estimated to be between 750 and 1000 kPa based on the compressive strengths in rock core samples taken on site. The axial load was used to calculate a bearing load, the governing bearing pressure on the soil beneath the footings was found to be 725 kPa. Therefore the foundations have adequate capacity in bearing for the design seismic event.

7.0 SEISMIC RETROFIT OPTIONS

7.1 Initial Load Path Upgrades

Each of the seismic retrofit options outlined require five basic upgrades of key elements of the load path in order to transfer the seismic loads to the seismic force resisting system (SFRS) and foundations. These basic upgrades include improved capacity of the roof and second floor diaphragm, improved connectivity of the deck diaphragms to the SFRS, horizontal braces at the roof level and foundation upgrades.

The SFRS of the Bridgetown High Commission could be further improved by the installation of friction dampers within the braced bays. Friction dampers dissipate seismic energy by increasing the damping of a structure. Increased damping has the net effect of lowering the force function, as illustrated in the equation of motion below:

$$m\ddot{u} + c\dot{u} + ku = f(t)$$

$$m\ddot{u} + ku = f(t) - c\dot{u}$$

Where m is mass, c is damping, k is stiffness and f(t) is the force function. The incorporation of friction dampers in the seismic upgrade could reduce the seismic force by 50%, which would significantly reduce the impact of the upgrades in terms of schedule, economics and intrusiveness to the building occupants.

The details and magnitude of the initial load path upgrades would depend on the chosen design level. The estimated cost for these load path upgrades is included in each retrofit option's cost estimate. This upgrade would include the following five components.

7.1.1 *Roof Diaphragm Upgrades*

The roof diaphragm has inadequate capacity to transfer shear forces to the SFRS. Options to increase the shear capacity of the diaphragm include; horizontal braces, an additional steel diaphragm, or the replacement of the existing roof diaphragm with a higher gauge steel deck with a connection pattern suited to resist the applied loads.

7.1.2 *Second Floor Diaphragm Upgrades*

Depending on the design level and retrofit strategy, the second floor diaphragm may require upgrading to increase its shear capacity. Options to increase the second floor shear capacity are the same as for the roof. This would include welding the steel deck to the support members to create an acceptable connection pattern to develop the strength of the diaphragm.

7.1.3 *Diaphragm to SFRS Connection Upgrades*

The third component of the initial upgrade includes the replacement of the existing perimeter angle with a larger steel section to both transfer the loads to the SFRS and resist the tension and compression forces generated by flexure of the deck diaphragm. Currently there is no clearly defined load path from the diaphragm to the SFRS, which indicates the possibility of storey diaphragms shearing from their supports during a seismic event. The upper portion of the exterior wall will need to be removed to install the new perimeter chord member.

7.1.4 *Horizontal Roof Braces*

The roof level between grid lines B and E and grid lines 3 and 4, 9 and 10 requires horizontal bracing. These braces are required to provide a connection between the small portion of the roof diaphragm which is separate from the main diaphragm and the SFRS in the north-south direction. These horizontal braces are very important as the existing building has no defined means of laterally restraining this portion of the roof in the north-south direction.

7.1.5 *Foundation Upgrades*

The fifth portion of the initial upgrade which is required includes upgrading the braced bay foundations. This upgrade would include the installation of rock anchors to resist the significant uplift on the foundations from the SFRS. Rock anchors are considered necessary in all load cases including the 60% load case. Also required in this upgrade is a retrofit of the column-foundation connection to withstand the high tension forces present in the uplift condition. This connection would involve a larger steel baseplate and concrete pier to accommodate more anchor bolts to transfer forces to the rock anchors.

7.2 Option 1 – Replacing Bracing Members

The feasibility of replacing the diagonal bracing members within the braced bays of the High Commission with larger members, as a means of increasing the capacity of its seismic force resisting system, was investigated.

This option was investigated under the assumption that the diaphragms on the second storey and roof will be upgraded to increase their shear capacities and that the foundations will be upgraded to resist the necessary uplift forces. It is necessary in the design of braced frame buildings that the diagonal bracing members be the point in the SFRS where yielding occurs. As the strength of the diagonal members is increased, the required strength of the other components of the SFRS must be as well, including the member connection, the shear transfer between the braced frames and the diaphragms, the diaphragms, and the foundation capacity. In this option, the seismic forces are only distributed to five or four braced bays in the north and south direction respectively. Therefore the seismic forces are more concentrated, which results in higher loads in the deck diaphragms adjacent to the braced bays and foundations supporting the braced bays.

It is anticipated that the interior finishes in the area of each braced bay would be removed allowing access to the existing bracing members. The existing bracing members would be removed and the larger braces would be provided, including new connection details to suit the new member capacities. The end result would be a braced bay with a diagonal bracing members of higher tensile resistance capacity.

Initial designs for the 60%, 100% and 150% capacity load cases have been completed. The designs are based on seismic forces calculated in accordance with the 2005 NBCC, a Site Class of C for the High Commission as specified by Golder Associates, and material properties ascertained as part of the structural investigation. The overall seismic forces imparted on the building were distributed to each braced bay based on its relative stiffness. The member design was completed in general accordance with the 2005 NBCC and CAN/CSA S16-01 Limit States Design of Steel Structures.

7.2.1 *Option 1a – Replacing Bracing Members – 100% Capacity*

To achieve the 100% capacity threshold it was determined that L127 x 89 x 16 and L89 x 89 x 13 diagonal members would be required at the ground to second storey and second storey to roof levels respectively in the braced frames. These members are considerably larger than the existing members. The member size chosen is based on a required cross-sectional area.

It is necessary to reinforce the columns in all braced bays at the ground floor to second floor level except Braced Bay 2 (Columns 46 and 47). To reinforce the columns, two 12 x 100 plates should be welded to the two faces of each column to supplement the strong-axis bending capacity of the column (i.e., perpendicular to the wall direction) from the ground to second floor level.

It is estimated that $850,000 CDN would be required to meet the 100% capacity threshold. This estimate includes the initial load path upgrades, the replacement of existing diagonal members, strengthening of the member connection, and reinstatement of the architectural finishes.

It is estimated that between 8 and 16 months would be required to complete the work outlined above. The level of intrusiveness of this work on day-to-day operation of the High Commission is considered high. For efficiency of completing the work it is recommended that the High Commission be vacated for the majority of the time while the retrofit work is being completed.

7.2.2 *Option 1b – Replacing Bracing Members – 60% Capacity*

When examining the 60% capacity threshold it was determined that the existing L76 x 76 x 6.4 diagonal members are insufficient in all of the braced bays. In all bays new L89 x 76 x 13 and L76 x 76 x 9.5 angles were found to be sufficient to resist the seismic loads at the ground floor to second floor and second floor to roof levels respectively.

To reinforce the columns, steel plates should be welded to the two faces of each column perpendicular to the wall direction. These plates should be 6 x 100 steel plates for all columns, from the ground to second floor level.
It is estimated that $715,000 CDN would be required to meet the 60% capacity threshold. This cost estimate includes the initial required upgrades to the existing steel diaphragms and foundations noted above.

The estimated duration to complete the retrofit to the 60% capacity level remains unchanged at 8 to 16 months and the level of intrusiveness on day-to-day operation of the High Commission remains high.

7.2.3 *Option 1c – Replacing Bracing Members – 150% Capacity*

To achieve the 150% capacity threshold it was determined that L127 x 127 x 22 and L127 x 89 x 16 diagonal members would be required in each of the braced bays at the ground floor to second floor and second floor to roof levels respectively. These members are considerably larger than the existing members and it may be necessary to inspect the feasibility of fitting these members into the existing walls. The member size chosen is based on a cross-sectional area required to attain the tensile capacity.

It is also necessary to reinforce the columns in all braced bays at the ground floor to second floor level. To reinforce the columns, steel plates should be welded to the two faces of each column perpendicular to the wall direction, from the ground to second floor level. These plates should be 16 x 100 steel plates for all columns except Columns 2 and 7, which should be reinforced with two 20 x 100 plates.

It is estimated that approximately $1,300,000 CDN would be required to meet the 150% capacity threshold. The estimated cost to perform the initial required upgrades would likely increase as stronger diaphragms, diaphragm connection patterns, shear transfer members, and foundations would be required. These extra estimated costs are accounted for in the estimate.

The estimated duration to complete the retrofit to the 150% capacity level remains unchanged at 8 to 16 months and the level of intrusiveness on day-to-day operation of the High Commission remains high.

7.3 Option 2 – Additional Braced Bays

The feasibility of adding diagonal braces to currently unbraced steel frames in the High Commission as a means of increasing the capacity of its seismic force resisting system, was investigated.

The placement of the additional braced bays would be along the perimeter of the building like the existing braced bays. The new bays would be placed so as to not interfere with architectural details such as the large windows at the front entrance of the building. It would be necessary to place the braced bays in an arrangement such that the building's centre of rigidity is not shifted away from the building's centre of mass. This shift would cause the amplification of shear forces in the building due to increased torsion. The assumed layout of the additional braced bays is shown on Drawing S1 in Appendix F.

Increasing the number of braced bays in a given loading direction is an effective means of distributing the seismic forces applied to a building over a greater number of seismic force resisting elements which results in smaller loads in each element. Additional braced bays also contribute to the redundancy of the structure which is an item in the ASCE 31-03 - 3.7.4S Supplemental Structural Checklist for Building Type S2: Steel Braced Frames with Stiff Diaphragms.

If this option is employed, the magnitude of the forces in the braced bays would be distributed over more bays, therefore the associated forces in the diaphragms would be reduced. It is anticipated that there would be a reduction in the level of work needed to reinforce the diaphragms and foundations.

The interior finishes in the area of each braced bay to be modified would be removed allowing access to the existing steel members. Each additional braced bay would require the installation of plates for connection details similar to those observed on site. The strength of the new connections details must be such that the connection is approximately 1.1 times as strong as the diagonal member. It is also necessary in this option to provide uplift capacity at the foundations of the new braced bays.

A preliminary analysis was completed to determine the feasibility of adding additional braced bays to resist 60%, 100%, and 150% of the seismic forces calculated in accordance with the 2005 NBCC. The results of the preliminary analysis for each load case are outlined in the subsequent sub-sections. In each case, the preliminary analysis was completed in general accordance with the 2005 NBCC and CAN/CSA S16-01 Limit States Design of Steel Structures.

7.3.1 *Option 2a – Additional Braced Bays – 100% Capacity*

It was determined that the addition of braced bays as shown in Drawing S1 in Appendix E reduces the loading in the diagonal braces a considerable amount; however, all of the existing braced bays still require larger diagonal members. It was determined that a member with a cross-sectional area of 2100 mm^2 would be sufficient to resist the applied loads. This cross-sectional area could be achieved by using L89 x 89 x 13 angles as the new diagonal members in the additional bays to be braced as well as the existing braced bays.

For the building to achieve 100% capacity in the arrangement of Option 2, the columns require reinforcement between the ground floor and second storey to gain adequate capacity. The columns require two 12 x 100 plates to be welded to the faces of the columns aligned perpendicularly to the direction of the wall.

It is estimated that $500,000 CDN would be required to institute the four additional braced bays. This cost estimate includes the estimated cost to add the new braces, upgrade the existing diagonal braces, strengthen the underlying footings, and reinstate the architectural finishes.

The estimated duration to complete the retrofit to the 100% capacity level for this option is between 8 and 16 months. The level of intrusiveness on day-to-day operation of the High Commission is considered to be high. As such, it would be expected that the High Commission would be required to be vacated for the majority of the time while the retrofit work was being completed.

7.3.2 *Option 2b – Additional Braced Bays – 60% Capacity*

It was determined that under the 60% capacity load case, adding braced bays results in demand/capacity ratios of less than 1.0 for the existing diagonal members. It is assumed bracing members in the new braced bays match the existing bracing (76 x 76 x 6.4).

All columns were found to have sufficient capacity to resist the 60% capacity threshold.

It is estimated that $420,000 CDN would be required to meet the 60% capacity threshold. This cost estimate includes the initial required upgrades to the existing steel diaphragms noted above and the necessary upgrades to the existing footings.
The estimated duration to complete the retrofit to the 60% capacity level remains unchanged at 8 to 16 months and the level of intrusiveness on day-to-day operation of the High Commission remains high.

7.3.3 *Option 2c – Additional Braced Bays – 150% Capacity*

It was determined that under the 150% case all of the existing braced bays would require larger diagonal members despite the additional braced bays. The initial required upgrades would also be necessary and the upgrade of the footings. The diagonal members required for this load case were found to be L102 x 102 x 13 members.
It is also necessary to reinforce the columns in all braced bays at the ground floor to second floor level. To reinforce the columns, steel plates should be welded to the two faces of each column perpendicular to the wall direction. These plates should be 12 x 100 steel plates for all columns except Columns 2, 7, 3 and 4, which should be reinforced with two 16 x 100 plates.

It is estimated that $730,000 CDN would be required to retrofit the building to 150% of the required capacity using additional braced bays and member replacement. The estimated cost to perform the initial required upgrades would likely increase as stronger diaphragms, diaphragm connection patterns, shear transfer members, and foundations would be required. This increase is accounted for in the estimate.

The estimated duration to complete the retrofit to the 150% capacity level for this option is between 12 and 16 months. The level of intrusiveness is considered to be high and would likely require the High Commission to be vacated for the majority of the time whilst the retrofit work was being completed.

7.4 Summary of Retrofit Options

The key points of each retrofit option are summarized in Table 8 for comparison purposes.

Retrofit Option	Load Case	Cost Estimate (CDN)	Estimated Duration	Level of Intrusiveness
Replace existing bracing members	60%	$0.71M	8 – 16 months	High
	100%	$0.85M	8 – 16 months	High
	150%	$1.30M	8 – 16 months	High
Additional Braced Bays	60%	$0.42M	8 – 16 months	High
	100%	$0.50M	8 – 16 months	High
	150%	$0.73M	12 – 16 months	High

Table 8 – Summary of Retrofit Options

7.5 Explanation of Cost Estimates

The cost estimates developed for each seismic retrofit option are based on average North American labour and material rates taken from published construction estimating manuals with factors applied to account for difficult and/or intrusive work. These rates are applicable to Canada only. Application of these rates to Bridgetown's construction market should be verified. A 15% contingency was also included in each cost estimate.

8.0 NON-STRUCTURAL COMPONENT UPGRADES

A number of non-structural components within the High Commission were identified as having the potential, with some seismic retrofit, to increase the life safety of building occupants. These items include the masonry walls, windows, gas and fire suppression supply piping, and mechanical and electrical equipment bracing. The proposed seismic retrofit solution for each component is discussed below.

During a seismic event there is the possibility that portions of the unreinforced masonry walls and partition walls of the High Commission will topple into the adjacent area within the building. This poses a serious hazard to the life safety of the building occupants. The installation of a steel angle member to brace the tops of these walls is an effective means of mitigating this potential hazard. The steel angle would be required at the tops of the walls. The estimated cost to complete this work is $10,000 CDN.

There is no indication that the windows of the High Commission contain safety glass or have been treated with a safety film. The purpose of safety glass or an applied safety film is to prevent the pane of glass from dislodging from the frame and/or shattering during a seismic or other similar event and subsequently causing harm to building occupants. The estimated cost to either apply a safety film to each window or replace the windows with laminated safety glass, is $25,000 and $75,000 CDN, respectively.

Damage caused to natural gas supply lines during a seismic event has a high potential of causing a building fire. The installation of braces to the gas lines and fire suppression system lines would help to alleviate this risk. The estimated cost to supply and install these braces would be about $200 CDN per brace.

Flexible couplings should be installed in the fire suppression piping to allow for some lateral movement of the system without compromising the pipes. The estimated cost to supply and install the couplings would be about $150 CDN per coupling.

Large pieces of furnishings, such as standing cabinets and bookshelves, can pose a safety hazard during a seismic event due to the potential for them to topple over onto building occupants or by blocking means of egress from the building. Anchoring furnishings to adjacent walls is an effective and relatively inexpensive means of reducing this hazard. The bracing of any suspended light fixtures or equipment should also be considered.

9.0 RECOMMENDATIONS

Assumptions

The analysis results presented in Tables 5, 6 and 7 are based on the following assumptions:

- Member sizes and reinforcement details are as shown on the structural drawings.

- Material properties are as shown on the structural drawings and Table 4 of this report.

- A Site Class of C was assumed based on preliminary indications from the site investigation being undertaken by Golder Associates. The final geotechnical report indicated that a Site Class A may be used, which would reduce the seismic forces by

20 percent. No consideration was given to slope failure or liquefaction of the underlying subgrade.

- The spectral acceleration values provided by DFAIT and not considering any local or near fault affects.

The analysis results indicate that the Bridgetown High Commission Seismic Force Resisting System (SFRS) has inadequate capacity to perform to the Immediate Occupancy performance objective during the design seismic event.

Recommendations

Five basic load path upgrades are recommended in order for lateral seismic forces to be properly transmitted to the SFRS. These upgrades are as follows:

1. The addition of horizontal braces or a higher capacity steel diaphragm to the roof or the replacement of the existing roof diaphragm with a higher gauge steel deck with a connection pattern to suit the applied loads.

2. Upgrading the capacity of the second floor diaphragm using the same methods as discussed for the roof diaphragm above. This would also include welding the steel deck to the support members to create an acceptable connection pattern to develop the strength of the diaphragm.

3. Replacement of the existing perimeter beam along the perimeter of the diaphragms to adequately transfer the loads to the SFRS.

4. The placement of horizontal braces at the roof level between grid lines B and E and grid lines 3 and 4, 9 and 10. These braces are required to provide a connection between the small portion of the roof diaphragm which is separate from the main diaphragm and the SFRS in the north-south direction.

5. The provision of rock anchors and other foundation upgrades to provide sufficient uplift capacity to the braced frames to resist the design seismic event.

It is recommended that the strength of the structural steel comprising the SFRS be confirmed through intrusive testing, the capacity of the SFRS is based directly on the material strength and therefore it contributes substantially to the capacity of the SFRS.

Considering estimated costs, duration and level of intrusiveness, it is recommended that the addition of new braced bays, as well as the replacement of existing bracing members, be pursued for the seismic retrofit of the Bridgetown High Commission. This option requires the retrofit of a greater number of braced bays; however, the effects on the deck diaphragm and foundation will be reduced. This option reduces the seismic forces on the deck diaphragm and foundation, which are the most costly, time consuming and intrusive aspects of the retrofit project. It is recommended that for a moderate increase in estimated cost the 150% capacity threshold be met, which represents seismic performance to the Immediate Occupancy performance level as required. The incorporation of friction dampers into the seismic retrofit strategy would reduce the impact of the retrofit in terms of construction schedule and cost and the impact to the building occupants. Friction dampers are specifically designed for use in braced bay seismic force resisting systems. The use of friction dampers will significantly reduce the seismic forces in the diaphragms and foundations to a level where the existing diaphragms and foundations may have adequate capacity to resist the design seismic event, which would significantly reduce the cost of the retrofit.

A number of the nonstructural components were found to be non-compliant, as outlined in Section 4.0. Some items such as glazing, unbraced mechanical and electrical equipment and unbraced fluid and gas piping could prove a hazard to the life safety of the building occupants and affect the operation of the building following a seismic event. It is understood that the glazing does not have anti-shatter film and is neither tempered nor strengthened glass. If the glazing becomes loose during a seismic event it could fall, posing a threat to the building occupants. Unbraced mechanical and/or electrical equipment or fluid and gas piping, should also be braced to the structure to prevent movement or toppling in a seismic event and to ensure continued operation. Partition walls and unreinforced masonry walls were noted to not be braced at the top. This could cause the walls to topple during a seismic event which poses a threat to building occupants.

10.0 SUMMARY AND CONCLUSION

The Bridgetown High Commission is a steel braced frame structure consisting of steel HSS columns, steel beams and diagonal steel braces. The Bridgetown High Commission is owned by DFAIT and is a Group 1 Infrastructure. The High Commission is expected to perform to the Immediate Occupancy performance objective. The structural drawings were used to determine the layout of the structure, member sizes and reinforcement details and material properties of structural components. An equivalent static force procedure and a dynamic modal response analysis were performed to evaluate the seismic performance of the structure.

The results of the analysis indicate that the seismic force resisting system does not have adequate capacity to perform to the Immediate Occupancy performance objective during the design seismic event in accordance with the 2005 NBCC and ASCE 31-03, as noted in Appendix A. The diagonal steel braces have demand/capacity ratios greater than 1.0, the foundation uplift capacity is inadequate for the design loads, and the steel deck diaphragms of the second storey and the roof are inadequate to transfer seismic loads to the seismic force resisting system. The combination of these three issues indicate that the lateral force resisting system in the Bridgetown High Commission would not perform to the desired performance objective and could be significantly damaged during the design seismic event. This would mean more significant repairs, a period of possible in-operation after the design seismic event, and potential loss of life.

A more detailed site study of these components is recommended, including removal of finishes and intrusive testing, which would provide more information on the ability of these components to meet the Immediate Occupancy performance objective. Included in the scope of the study would be an examination of the as-built welding pattern used to attach the steel diaphragms to the support members and an examination of the connection details at the diagonal members.

A number of nonstructural components were found to be noncompliant with the ASCE 31-03 Checklists, these items include the glazing, unbraced partition and unreinforced masonry walls, unbraced mechanical and electrical systems and unbraced fluid and gas piping. These items could pose a threat to the life safety of building occupants and may affect the operation of the structure after the design seismic event. The Bridgetown High Commission is intended to perform to the Immediate Occupancy performance objective.

Addressing these deficiencies would improve the performance of the structure after the design seismic event.

Two retrofit options to increase the seismic capacity of the High Commission were developed and their costs estimated. Considering estimated cost, duration and level of intrusiveness, it is recommended that the addition of new braced bays and the replacement of existing bracing members with friction dampers be pursued for the seismic retrofit of the Bridgetown High Commission. It is recommended that for a moderate increase in estimated cost the 150% capacity threshold could be met. This retrofit option would minimize required upgrades to the steel diaphragms at the roof and second storey levels as well as to the foundations including the installation of rock anchors to properly resist the uplift forces associated with a design seismic event.

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Reviewed by:

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Nov 1/2011
PROVINCE OF ONTARIO

APPENDIX A
SEISMIC EVALUATION METHOD

The scope of work for this project, as defined by DFAIT, is as follows:

- Perform a seismic evaluation in accordance with ASCE/SEI 31-03 as modified in the Statement of Work.
- Group 1 infrastructure are to be evaluated based on immediate occupancy (IO) criteria and group 2 infrastructure are to be evaluated based on life safety (LS) criteria.
- A Tier 1 and Tier 2 evaluation are required for all infrastructures.
- Review all existing documentation made available by DFAIT.
- For Group 1 missions, plan and initiate exploratory demolition openings to verify existing structural details, structural condition, material properties, etc.
- A 3-D structural model is required for all Group 1 infrastructures and as required for Group 2 infrastructures (deficiencies).
- Verify Site Class/Seismic Hazard, assume a Site Class D where no information is provided.
- Calculate lateral load capacity and compare to the equivalent static base shear calculated using the National Building Code of Canada 2005 (NBCC 2005).
- For group 1 infrastructure review seismic performance of non-structural elements.
- Prepare a written report and attend a meeting in Ottawa to review draft report.

Seismic Evaluation Method per ASCE 31-03 as modified to comply (IMHO) with NBCC 2005 section 4.1.8.

Tier 1

The following material properties are to be used, unless material properties are specified in the existing documentation provided by DFAIT:
Concrete–f'c+13 MPa;
Reinforcing Steel–fy=210 MPa;
Structural Steel–Fy=210 MPa; and
Masonry–f'm=6 MPa.

3.2 No exemption will be made for benchmark buildings.

3.5 The pseudo lateral force will be calculated as per 2005 NBCC section 4.1.8.11 including the spectral acceleration values, Mv factor, Ie factor, the seismic weight W and Rd and Ro factors. Except that the spectral acceleration values provided by DFAIT will be modified by S(t)*0.6. The fa and fv factors will be based on NBCC 2005 table 4.1.8.4B and 4.1.8.4C.

The storey shears will be distributed as described in NBCC 2005 section 4.1.8.11-6.

Higher mode effects will be calculated as described in NBCC 2005 section 4.1.8.11-5.

The base overturning moment will be reduced as described in NBCC 2005 section 4.1.8.11-7.

The spectral acceleration values will be calculated as per NBCC 2005 section 4.1.8.4 except that a modification factor of 0.6 will be applied to the spectral acceleration values.

Building period will be calculated as per NBCC 2005 section 4.1.8.11-3

Quick Checks for Story drift of Moment Frames, Shear Stress in Concrete frame columns, shear stress in shear walls, diagonal bracing, precast connections, axial stress due to overturning, flexible diaphragm connection forces and prestressed elements will be performed where applicable.

3.7 The appropriate structural checklists will be performed. For Life safety structural, supplemental structural, geologic site hazards/foundation, basic non-structural and intermediate nonstructural will be performed. For the immediate occupancy performance objective structural, supplemental structural, geologic site hazards/foundation, basic non-structural, intermediate nonstructural and supplemental nonstructural checklists will be performed.

Tier 2

A dynamic analysis will be performed as per NBCC 2005 4.1.8.12.

APPENDIX B
SITE PHOTOGRAPHS


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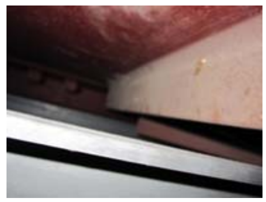
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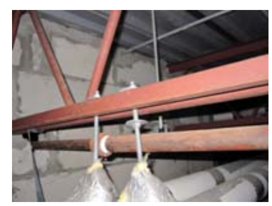
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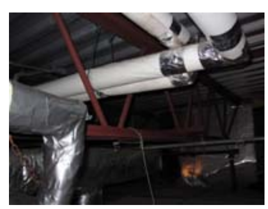
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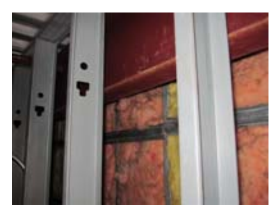
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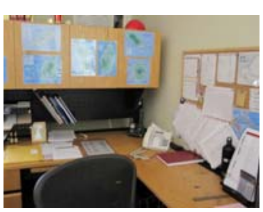
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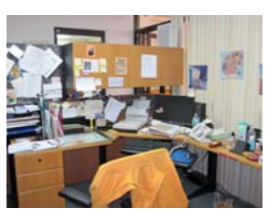
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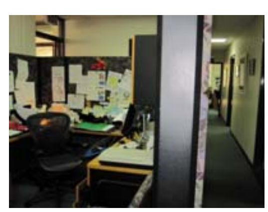
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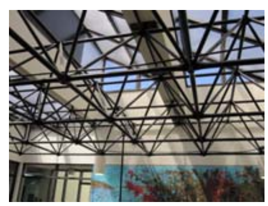
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APPENDIX C
ASCE/SEI 31-03 CHECKLISTS

3.7.4 Basic Structural Checklist for Building Type S2: Steel Braced Frames with Stiff Diaphragms

Building System

C NC N/A LOAD PATH: The structural shall contain a minimum of one complete load path for Life Safety and Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation. (Tier 2: Sec. 4.3.1.1)

C NC N/A ADJACENT BUILDINGS: The clear distance between the building being evaluate and any adjacent building shall be greater than 4 percent of the height of the shorter building for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.1.2)

C NC N/A MEZZANINES: Interior mezzanine levels shall be braced independently from the main structure or shall be anchored to the lateral-force-resisting elements of the main structure. (Tier 2: Sec. 4.3.1.3)

C NC N/A WEAK STOREY: The strength of the lateral-force-resisting system in any storey shall not be less than 80 percent of the strength in an adjacent storey, above or below, for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.1)

C NC N/A SOFT STOREY: The stiffness of the lateral-force-resisting system in any storey shall not be less than 70 percent of the lateral-force-resisting system stiffness in an adjacent storey above or below, or less than 80 percent of the average lateral-force-resisting system stiffness of the three stories above or below for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.2)

C NC N/A GEOMETRY: There shall be no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a storey relative to adjacent stories for Life Safety and Immediate Occupancy, excluding one-storey penthouses and mezzanines. (Tier 2: Sec. 4.3.2.3)

C NC N/A VERTICAL DISCONTINUITIES: All vertical elements in the lateral-force-resisting system shall be continuous to the foundation. (Tier 2: Sec. 4.3.2.4)

C NC N/A MASS: There shall be no change in effective mass more than 50 percent from one storey to the next for Life Safety and Immediate Occupancy. Light roofs, penthouses and mezzanines need not be considered. (Tier 2: Sec. 4.3.2.5)

C NC N/A TORSION: The estimated distance between the storey centre of mass and the storey centre of rigidity shall be less than 20 percent of the building width in either plan dimension for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.6)

C NC N/A DETERIORATION OF STEEL: There shall be no visible rusting, corrosion, cracking or other deterioration in any of the steel elements or connections in the vertical- or lateral-force-resisting systems. (Tier 2: Sec. 4.3.3.3)

C NC N/A DETERIORATION OF CONCRETE: There shall be no visible deterioration of concrete or reinforcing steel in any of the vertical- or lateral-force-resisting elements. (Tier 2: Sec. 4.3.3.4)

Lateral-Force-Resisting System

C NC N/A AXIAL STRESS CHECK: The axial stress due to gravity loads in columns subjected to overturning forces shall be less than $0.10F_y$ for Life Safety and Immediate Occupancy. Alternatively, the axial stress due to overturning forces alone, calculated using the Quick Check procedure of Section 3.5.3.6 shall be less than $0.30F_y$ for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.4.1.3.2)

C NC N/A REDUNDANCY: The number of lines of braced frames in each principal direction shall be greater than or equal to 2 for Life Safety and Immediate Occupancy. The number of braced bays in each line shall be greater than 2 for Life Safety and 3 for Immediate Occupancy. (Tier 2: Sec. 4.4.3.1.1)

C NC N/A AXIAL STRESS CHECK: The axial stress in the diagonals, calculated using the Quick Check procedure of Section 3.5.3.4 shall be less than $0.50F_y$ for Life Safety and for Immediate Occupancy. (Tier 2: Sec. 4.4.3.1.2)

C NC N/A COLUMN SPLICES: All column splice details located in braced frames shall develop the tensile strength of the column. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.3.1.3)

Connections

C NC N/A TRANSFER TO STEEL FRAMES: Diaphragms shall be connected for transfer of loads to the steel frames for Life Safety and the connections shall be able to develop the lesser of the strength of the frames or the diaphragms for Immediate Occupancy. (Tier 2: Sec. 4.6.2.2)

C NC N/A STEEL COLUMNS: The columns in lateral-force-resisting frames shall be anchored to the building foundation for Life Safety and the anchorage shall be able to develop the lesser of the tensile capacity of the column, the tensile capacity of the lowest level column splice (if any), or the uplift capacity of the foundation, for Immediate Occupancy. (Tier 2: Sec. 4.6.3.1)

3.7.4S Supplemental Structural Checklist for Building Type S2: Steel Braced Frames with Stiff Diaphragms

Lateral-Force-Resisting System

C NC N/A COMPACT MEMBERS: All frame elements shall meet section requirements set forth by *Seismic Provisions for Structural Steel Buildings* Table I-9-1 (AISC, 1997). (Tier 2: Sec. 4.4.1.3.7)

C NC N/A SLENDERNESS OF DIAGONALS: All diagonal elements required to carry compression shall have Kl/r ratios less than 120. (Tier 2: Sec. 4.4.3.1.4)

C NC N/A CONNECTION STRENGTH: All of the brace connections shall develop the yield capacity of the diagonals. (Tier 2: Sec. 4.4.3.1.5)

C NC N/A OUT-OF-PLANE BRACING: Braced frame connections attached to beam bottom flanges located away from beam-column joints shall be braced out-of-plane at the bottom flange of the beams. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.3.1.6)

C NC N/A K-BRACING: The bracing system shall not include K-braced bays. (Tier 2: Sec. 4.4.3.2.1)

C NC N/A TENSION-ONLY BRACES: Tension-only braces shall not compromise more than 70 percent of the total lateral-force-resisting capacity in structures over two storeys in height. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.3.2.2)

C NC N/A CHEVRON BRACING: The bracing system shall not include chevron or V-braced bays. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.3.2.3)

C NC N/A CONCENTRICALLY BRACED FRAME JOINTS: All of the diagonal braces shall frame into the beam-column joints concentrically. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.3.2.4)

Diaphragms

C NC N/A OPENINGS AT BRACED FRAMES: Diaphragm openings immediately adjacent to the braced frames shall extend less than 25 percent of the frame length for Life Safety and 15 percent of the frame length for Immediate Occupancy. (Tier 2: Sec. 4.5.1.5)

C NC N/A PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.7)

C NC N/A DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.8)

Connections

C NC N/A UPLIFT AT PILE CAPS: Pile caps shall have top reinforcement and piles shall be anchored to the pile caps for Life Safety, and the pile cap reinforcement and pile anchorage shall be able to develop the tensile capacity of the piles for Immediate Occupancy. (Tier 2: Sec. 4.6.3.10)

3.8 Geologic Site Hazards and Foundations Checklist

Partitions

C NC N/A LIQUEFACATION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.7.1.1)

C NC N/A SLOPE FAILURE: The building site shall be sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or shall be capable of accommodating any predicted movements without failure. (Tier 2: Sec. 4.7.1.2)

C NC N/A SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site is not anticipated. (Tier 2: Sec. 4.7.1.3)

Condition of Foundations

C NC N/A FOUNDATION PERFORMANCE: There shall be no evidence of excessive foundation movement such as settlement or heave that would affect the integrity or strength of the structure. (Tier 2: Sec. 4.7.2.1)

C NC N/A DETERIORATION: There shall not be evidence that foundation elements have deteriorated due to corrosion, sulphate attack, material breakdown or other reasons in a manner that would affect the integrity or strength of the structure. (Tier 2: Sec. 4.7.2.2)

Capacity of Foundations

C NC N/A POLE FOUNDATIONS: Pole foundations shall have a minimum embedment depth of 4 feet for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.7.3.1)

C NC N/A OVERTURNING: The ratio of the horizontal dimension of the lateral-force-resisting system at the foundation level to the building height (base/height) shall be greater than $0.6S_a$. (Tier 2: Sec. 4.7.3.2)

C NC N/A TIES BETWEEN FOUNDATION ELEMENTS: The foundation shall have ties adequate to resist seismic forces where footings, piles and piers are not restrained by beams, slabs or soils classified as Class A, B or C. (Section 3.5.2.3.1, Tier 2: Sec. 4.7.3.3)

C NC N/A DEEP FOUNDATIONS: Piles and piers shall be capable of transferring the lateral forces between the structure and the soil. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.7.3.4)

C NC N/A SLOPING SITES: The difference in foundation embedment depth from one side of the building to another shall not exceed one storey in height. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.7.3.5)

3.9.1 Basic Nonstructural Component Checklist

Partitions

C NC N/A UNREINFORCED MASONRY: Unreinforced masonry or hollow clay tile partitions shall be braced at a spacing equal to or less than 10 feet in levels of low or moderate seismicity and 6 feet in levels of high seismicity. (Tier 2: Sec. 4.8.1.1)

Ceiling Systems

C NC N/A SUPPORT: The integrated suspended ceiling system shall not be used to laterally support the tops of gypsum board, masonry or hollow clay tile partitions. Gypsum board partitions need not be evaluated where only the Basic Nonstructural Component Checklist is required by Table 3-2. (Tier 2: Sec. 4.8.2.1)

Light Fixtures

C NC N/A EMERGENCY LIGHTING: Emergency lighting shall be anchored or braced to prevent falling during an earthquake. (Tier 2: Sec. 4.8.3.1)

Cladding and Glazing

C NC N/A CLADDING ANCHORS: Cladding components weighing more than 10 psf shall be mechanically anchored to the exterior wall framing at a spacing equal to or less than 4 feet. A spacing of up to 6 feet is permitted where only the Basic Nonstructural component checklist is required by Table 3-2. (Tier 2: Sec. 4.8.4.1)

C NC N/A DETERIORATION: There shall be no evidence of deterioration, damage or corrosion in any of the connection elements. (Tier 2: Sec. 4.8.4.2)

C NC N/A CLADDING ISOLATION: For moment frame buildings of steel or concrete, panel connections shall be detailed to accommodate a storey drift ration of 0.02. Panel connection detailing for a storey drift ration of 0.01 is permitted where only the Basic Nonstructural Component Checklist is required by Table 3-2. (Tier 2: Sec. 4.8.4.3)

C NC N/A MULTI-STOREY PANELS: For multi-storey panels attached to each floor level, panel connections shall be detailed to accommodate a storey draft ration of -.02. Panel connection detailing for a storey drift ratio of 0.01 is permitted where only the Basic Nonstructural Component Checklist is required by Table 3-2. (Tier 2: Sec. 4.8.4.4)

C NC N/A BEARING CONNECTIONS: Where bearing connections are required, there shall be a minimum of two bearing connections for each wall panel. (Tier 2: Sec. 4.8.4.5)

C NC N/A INSERTS: Where inserts are used in concrete connections, the inserts shall be anchored to reinforcing steel or other positive anchorage. (Tier 2: Sec. 4.8.4.6)

C NC N/A PANEL CONNECTIONS: Exterior cladding panels shall be anchored out-of-plane with a minimum of 4 connections for each wall panel. Two connections per wall panel are permitted where only the Basic Nonstructural Component Checklist is required by Table 3-2. (Tier 2: Sec. 4.8.4.7)

Masonry Veneer

C NC N/A SHELF ANGLES: Masonry veneer shall be supported by shelf angles or other elements at each floor 30 feet or more above ground for Life Safety and at each floor above the first floor for Immediate Occupancy. (Tier 2: Sec. 4.8.5.1)

C NC N/A TIES: Masonry veneer shall be connected to the back-up with corrosion-resistant ties. The ties shall have a spacing equal to or less than 24 inches with a minimum of one tie for every 2-2/3 square feet. A spacing of up to 36 inches is permitted where only the Basic Nonstructural Component Checklist is required by Table 3-2. (Tier 2: Sec. 4.8.5.2)

C NC N/A WEAKENED PLANES: Masonry veneer shall be anchored to the back-up adjacent to the weakened planes, such as at the locations of flashing. (Tier 2: Sec. 4.8.5.3)

C NC N/A DETERIORATION: There shall be no evidence of deterioration, damage or corrosion in any of the connection elements. (Tier 2: Sec. 4.8.5.4)

Parapets, Cornices, Ornamentation and Appendages

C NC N/A URM PARAPETS: There shall be no laterally unsupported unreinforced masonry parapets or cornices with height-to-thickness ratios greater than 1.5. A height-to-thickness ration of up to 2.5 is permitted where only the Basic Nonstructural Component Checklist is required by Table 3-2. (Tier 2: Sec. 4.8.8.1)

C NC N/A CANOPIES: Canopies located at building exits shall be anchored to the structural framing at a spacing of 6 feet or less. An anchorage spacing of up to 10 feet is permitted where only the Basic Nonstructural Component Checklist is required by Table 3-2. (Tier 2: Sec. 4.8.8.2)

Masonry Chimneys

C NC N/A URM CHIMNEYS: No unreinforced masonry chimney shall extend above the roof surface more than twice the least dimension of the chimney. A height above the roof surface of up to three times the least dimension of the chimney is permitted where only the Basic Nonstructural Component Checklist is required by Table3-2. (Tier 2: Sec. 4.8.9.1)

Stairs

C NC N/A URM WALLS: Walls around stair enclosures shall not consist of unbraced hollow clay tile or unreinforced masonry with a height-to-thickness ration greater than 12 to 1. A height-to-thickness ratio of up to 15 to 1 is permitted where only the Basic Nonstructural Component Checklist is required by Table 3-2. (Tier 2: Sec. 4.8.10.1)

C NC N/A STAIR DETAILS: In moment frame structures, the connection between the stairs and the structure shall not rely on shallow anchors in concrete. Alternatively, the stair details shall be capable of accommodating the drift calculated using the Quick Check procedure of Section 3.5.3.1 without including tension in the anchors. (Tier 2: Sec. 4.8.10.2)

Building Contents and Furnishing

C NC N/A TALL NARROW CONTENTS: Contents over 4 feet in height with a height-to-depth or height-to-width ration greater than 3 to 1 shall be anchored to the floor slab or adjacent structural walls. A height-to-depth or height-to-width ratio of up to 4 to 1 is permitted where only the Basic Nonstructural Component Checklist is required by Table 3-2. (Tier 2: Sec. 4.8.11.1)

Mechanical and Electrical Equipment

C NC N/A EMERGENCY POWER: Equipment used as part of an emergency power system shall be mounted to maintain continued operation after an earthquake. (Tier 2: Sec. 4.8.12.1)

C NC N/A HAZARDOUS MATERIAL EQUIPMENT: HVAC or other equipment containing hazardous material shall not have damaged supply lines or unbraced isolation supports. (Tier 2: Sec. 4.8.12.2)

C NC N/A DETERIORATION: There shall be no evidence of deterioration, damage or corrosion in any of the anchorage or supports of mechanical or electrical equipment. (Tier 2: Sec. 4.8.12.3)

C NC N/A ATTACHED EQUIPMENT: Equipment weighing over 20 pounds that is attached to ceilings, walls or other supports, 4 feet above the floor level, shall be braced. (Tier 2: Sec. 4.8.12.4)

Piping

C NC N/A FIRE SUPPRESSION PIPING: Fire suppression piping shall be anchored and braced in accordance with NFPA-13 (NFPA, 1996). (Tier 2: Sec. 4.8.13.1)

C NC N/A FLEXIBLE COUPLINGS: Fluid, gas and fire suppression piping shall have flexible couplings. (Tier 2: Sec. 4.8.13.2)

3.9.2 Intermediate Nonstructural Component Checklist

Ceiling Systems

C NC N/A LAY-IN TILES: Lay-in tiles used in ceiling panels located at exits and corridors shall be secured with clips. (Tier 2: Sec. 4.8.2.2)

C NC N/A INTEGRATED CEILINGS: Integrated suspended ceilings at exits and corridors or weighing more than 2 pounds per square foot shall be laterally restrained with a minimum of four diagonal wires or rigid members attached to the structure above at a spacing equal to or less than 12 feet. (Tier 2: Sec. 4.8.2.3)

C NC N/A SUSPENDED LATH AND PLASTER: Ceilings consisting of suspended lath and plaster or gypsum board shall be attached to resist seismic forces for every 12 square feet of area. (Tier 2: Sec. 4.8.2.4)

Light Fixtures

C NC N/A INDEPENDENT SUPPORT: Light fixtures in suspended grid ceilings shall be supported independently of the ceiling suspension system by a minimum of two wires at diagonally opposite corners of the fixtures. (Tier 2: Sec. 4.8.3.2)

Cladding and Glazing

C NC N/A GLAZING: Glazing in curtain walls and individual panes over 16 square feet in area, located up to a height of 10 feet above an exterior walking surface, shall have safety glazing. Such glazing located over 10 feet above an exterior walking surface shall be laminated annealed or laminated heat-strengthened safety glass or other glazing system that will remain in the frame when glass is cracked. (Tier 2: Sec. 4.8.4.8)

Parapets, Cornices, Ornamentation and Appendages

C NC N/A CONCRETE PARAPETS: Concrete parapets with height-to-thickness ratios greater than 2.5 shall have vertical reinforcement. (Tier 2: Sec. 4.8.8.3)

C NC N/A APPENDAGES: Cornices, parapets, signs and other appendages that extend above the highest point of anchorage to the structure or cantilevered from exterior wall faces and other exterior wall ornamentation shall be reinforced and anchored to the structural system at a spacing equal to or less than 10 feet for Life Safety and 6 feet for Immediate Occupancy. This requirement need not apply to parapets or cornices compliant with Section 4.8.8.1 or 4.8.8.3. (Tier 2: Sec. 4.8.8.4)

Masonry Chimneys

C NC N/A ANCHORAGE: Masonry chimneys shall be anchored at each floor level and the roof. (Tier 2: Sec. 4.8.9.2)

Mechanical and Electrical Equipment

C NC N/A VIBRATION ISOLATORS: Equipment mounted on vibration isolators shall be equipped with restraints or snubbers. (Tier 2: Sec. 4.8.12.5)

Ducts

C NC N/A STAIR AND SMOKE DUCTS: Stair pressurization and smoke control ducts shall be braced and shall have flexible connections at seismic joints. (Tier 2: Sec. 4.8.14.1)

Hazardous Materials Storage and Distribution

C NC N/A TOXIC SUBSTANCES: Toxic and hazardous substances stored in breakable containers shall be restrained from falling by latched doors, shelf lips, wires or other methods. (Tier 2: Sec. 4.8.15.1)

3.9.3 Supplemental Nonstructural Component Checklist

Partitions

C NC N/A DRIFT: Rigid cementitious partitions shall be detailed to accommodate a drift ratio of 0.02 in steel moment frame, concrete moment frame and wood frame buildings. Rigid cementitious partitions shall be detailed to accommodate a drift ratio of 0.005 in other buildings. (Tier 2: Sec. 4.8.1.2)

C NC N/A STRUCTURAL SEPARATIONS: Partitions at structural separations shall have seismic or control joints. (Tier 2: Sec. 4.8.1.3)

C NC N/A TOPS: The tops of framed or panelized partitions that only extend to the ceiling line shall have lateral bracing to the building structure at a spacing equal to or less than 6 feet. (Tier 2: Sec. 4.8.1.4)

Ceiling Systems

C NC N/A EDGES: The edges of integrated suspended ceilings shall be separated from enclosing walls by a minimum of ½ inch. (Tier 2: Sec. 4.8.2.5)

C NC N/A SEISMIC JOINT: The ceiling system shall not extend continuously across any seismic joint. (Tier 2: Sec. 4.8.2.6)

Cladding and Glazing

C NC N/A PENDANT SUPPORTS: Light fixtures on pendant supports shall be attached at a spacing equal to or less than 6 feet and, if rigidly supported, shall be free to move with the structure to which they are attached without damaging adjoining materials. (Tier 2: Sec. 4.8.3.3)

C NC N/A LENS COVERS: Lens covers on light fixtures shall be attached or supplied with safety devices. (Tier 2: Sec. 4.8.3.4)

Cladding and Glazing

C NC N/A GLAZING: All exterior glazing shall be laminated, annealed or laminated heat-strengthened safety glass or other glazing system that will remain in the frame when glass is cracked. (Tier 2: Sec. 4.8.4.9)

Masonry Veneer

C NC N/A MORTAR: The mortar in masonry veneer shall not be easily scraped away from the joints by hand with a metal tool and there shall not be significant areas of eroded mortar. (Tier 2: Sec. 4.8.5.5)

C NC N/A WEEP HOLES: In veneer braced by stud walls, functioning weep holes and base flashing shall be present. (Tier 2: Sec. 4.8.5.6)

C NC N/A STONE CRACKS: There shall be no visible cracks or signs of visible distortion in the stone. (Tier 2: Sec. 4.8.5.7)

Metal Stud Back-Up System

C NC N/A STUD TRACKS: Stud tracks shall be fastened to structural framing at a spacing equal to or less than 24 inches on centre. (Tier 2: Sec. 4.8.6.1)

C NC N/A OPENINGS: Steel studs shall frame window and door openings. (Tier 2: Sec. 4.8.6.2)

Concrete Block and Masonry Back-Up Systems

C NC N/A ANCHORAGE: Back-up shall have a positive anchorage to the structural framing at a spacing equal to or less than 4 feet along the floors and roof. (Tier 2: Sec. 4.8.7.1)

C NC N/A URM BACK-UP: There shall be no unreinforced masonry back-up. (Tier 2: Sec. 4.8.7.2)

Building Contents and Furnishing

C NC N/A FILE CABINETS: File cabinets arranged in groups shall be attached to one another. (Tier 2: Sec. 4.8.11.2)

C NC N/A CABINET DOORS AND DRAWERS: Cabinet doors and drawers shall have latches to keep them closed during an earthquake. (Tier 2: Sec. 4.8.11.3)

C NC N/A ACCESS FLOORS: Access floors over 9 inches in height shall be braced. (Tier 2: Sec. 4.8.11.4)

C NC N/A EQUIPMENT ON ACCESS FLOORS: Equipment and computers supported on access floor systems shall be either attached to the structure or fastened to a laterally braced floor system. (Tier 2: Sec. 4.8.11.5)

Mechanical and Electrical Equipment

C NC N/A HEAVY EQUIPMENT: Equipment weighing over 100 pounds shall be anchored to the structure or foundation. (Tier 2: Sec. 4.8.12.6)

C NC N/A ELECTRICAL EQUIPMENT: Electrical equipment and associated wiring shall be laterally braced to the structural system. (Tier 2: Sec. 4.8.12.7)

C NC N/A DOORS: Mechanically operated doors shall be detailed to operate at a storey drift ration of 0.01. (Tier 2: Sec. 4.8.12.8)

Piping

C NC N/A FLUID AND GAS PIPING: Fluid and gas piping shall be anchored and braced to the structure to prevent breakage in piping. (Tier 2: Sec. 4.8.13.3)

C NC N/A SHUT-OFF VALVES: Shut-off devices shall be present at building utility interfaces to shut off the flow of gas and high-temperature energy in the event of earthquake-induced failure. (Tier 2: Sec. 4.8.13.4)

C NC N/A C-CLAMPS: One-sided C-clamps that support piping greater than 2.5 inches in diameter shall be restrained. (Tier 2: Sec. 4.8.13.5)

Ducts

C NC N/A DUCT BRACING: Rectangular ductwork exceeding 6 square feet in cross-sectional area and round ducts exceeding 28 inches in diameter shall be braced. Maximum spacing of transverse bracing shall not exceed 30 feet. Maximum spacing of longitudinal bracing shall not exceed 60 feet. Intermediate supports shall not be considered part of the lateral-force-resisting system. (Tier 2: Sec. 4.8.14.2)

C NC N/A DUCT SUPPORT: Ducts shall not be supported by piping or electrical conduit. (Tier 2: Sec. 4.8.14.3)

Hazardous Materials Storage and Distribution

C NC N/A GAS CYLINDERS: Compressed gas cylinders shall be restrained. (Tier 2: Sec. 4.8.15.2)

C NC N/A HAZARDOUS MATERIALS: Piping containing hazardous materials shall have shut-off valves or other devices to prevent major spills or leaks. (Tier 2: Sec. 4.8.15.3)

Elevators

C NC N/A SUPPORT SYSTEM: All elements of the elevator system shall be anchored. (Tier 2: Sec. 4.8.16.1)

C NC N/A SEISMIC SWITCH: All elevators shall be equipped with seismic switches that will terminate operations when the ground motion exceeds 0.10g. (Tier 2: Sec. 4.8.16.2)

C NC N/A SHAFT WALLS: All elevator shaft walls shall be anchored and reinforced to prevent toppling into the shaft during strong shaking. (Tier 2: Sec. 4.8.16.3)

C NC N/A RETAINER GUARDS: Cable retainer guards on sheaves and drums shall be present to inhibit the displacement of cables. (Tier 2: Sec. 4.8.16.4)

C NC N/A RETAINER PLATE: A retainer plate shall be present at the top and bottom of both car and counterweight. (Tier 2: Sec. 4.8.16.5)

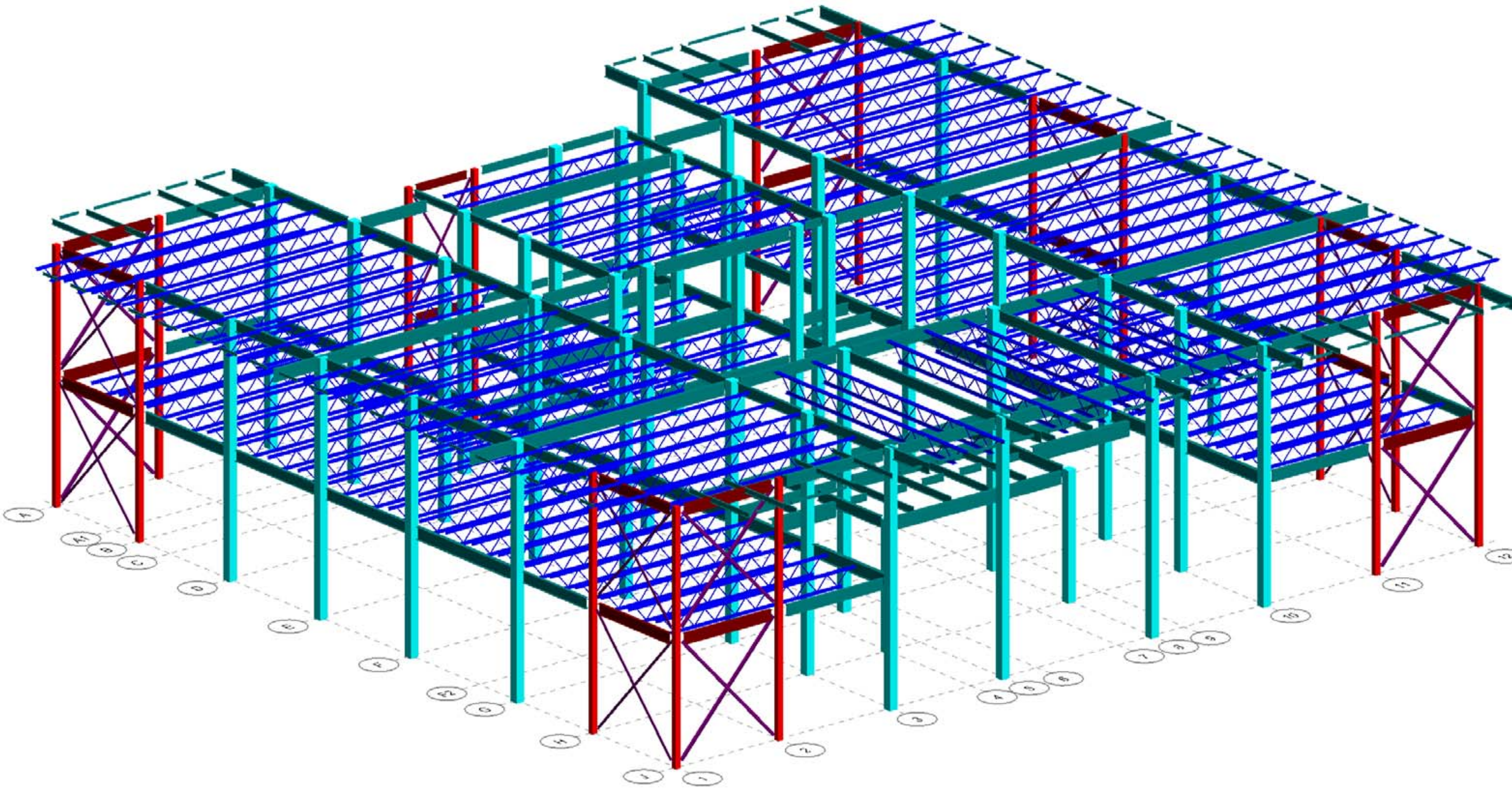
C NC N/A COUNTERWEIGHT RAILS: All counterweight rails and divider beams shall be sized in accordance with ASME A17.1. (Tier 2: Sec. 4.8.16.6)

C NC N/A BRACKETS: The brackets that tie the car rails and the counterweight rail to the building structure shall be sized in accordance with ASME A17.1 (Tier 2: Sec. 4.8.16.7)

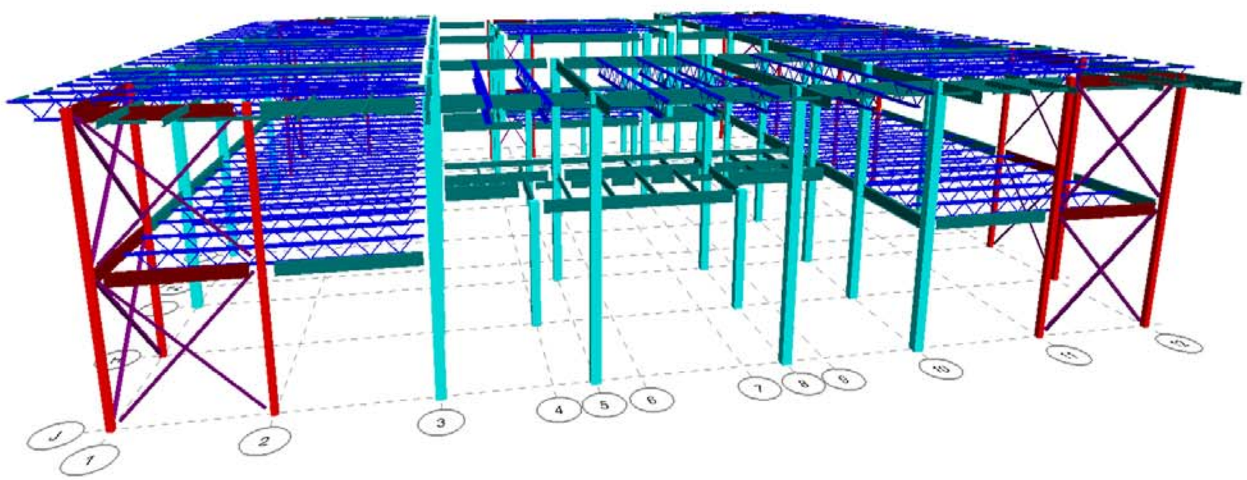
C NC N/A SPREADER BRACKET: Spreader brackets shall not be used to resist seismic forces. (Tier 2: Sec. 4.8.16.8)

C NC N/A GO-SLOW ELEVATORS: The building shall have a go-slow elevator system. (Tier 2: Sec. 4.8.16.9)

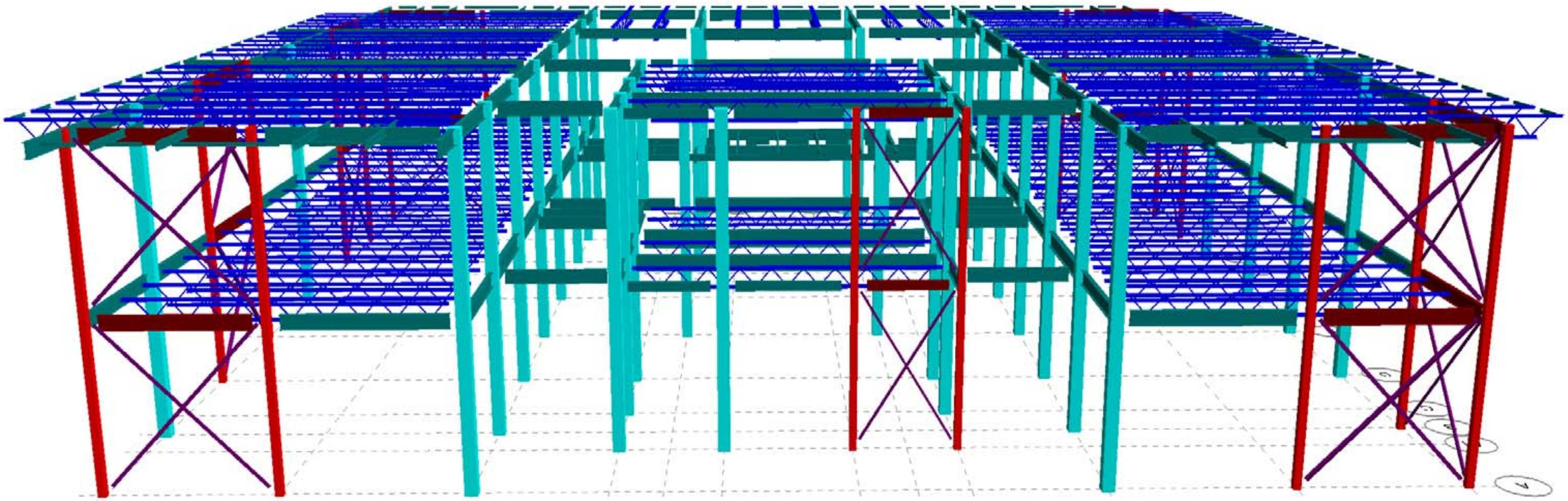
APPENDIX D
MODEL RENDERINGS



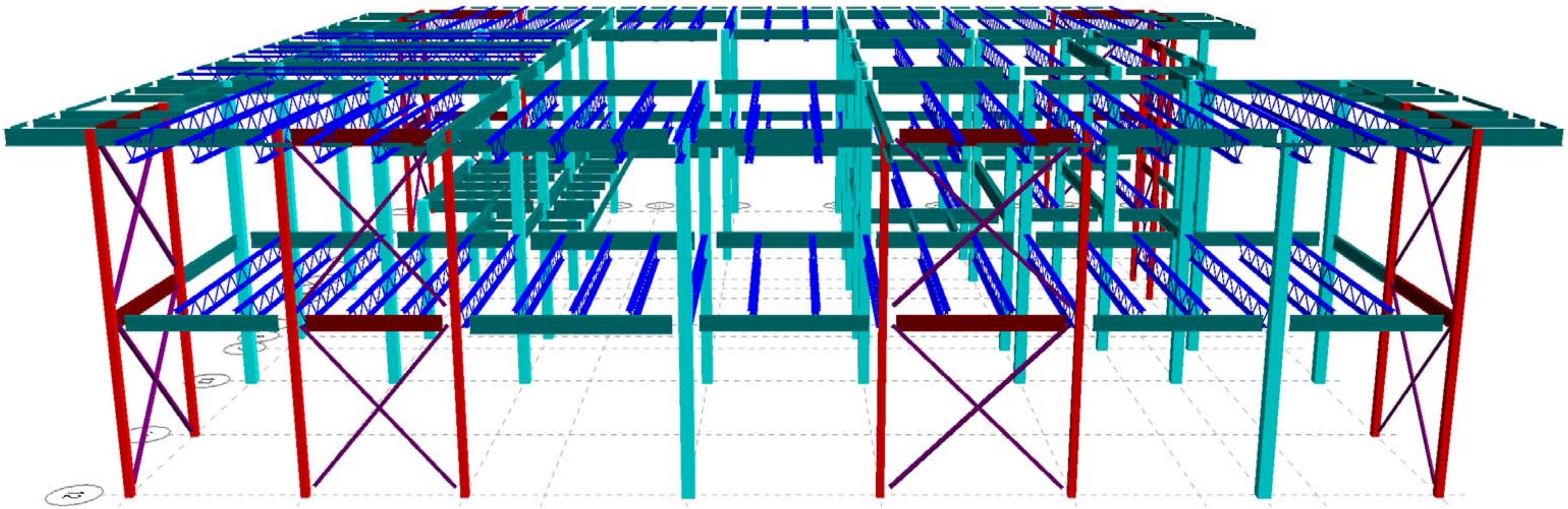
North-East Chancery Perspective



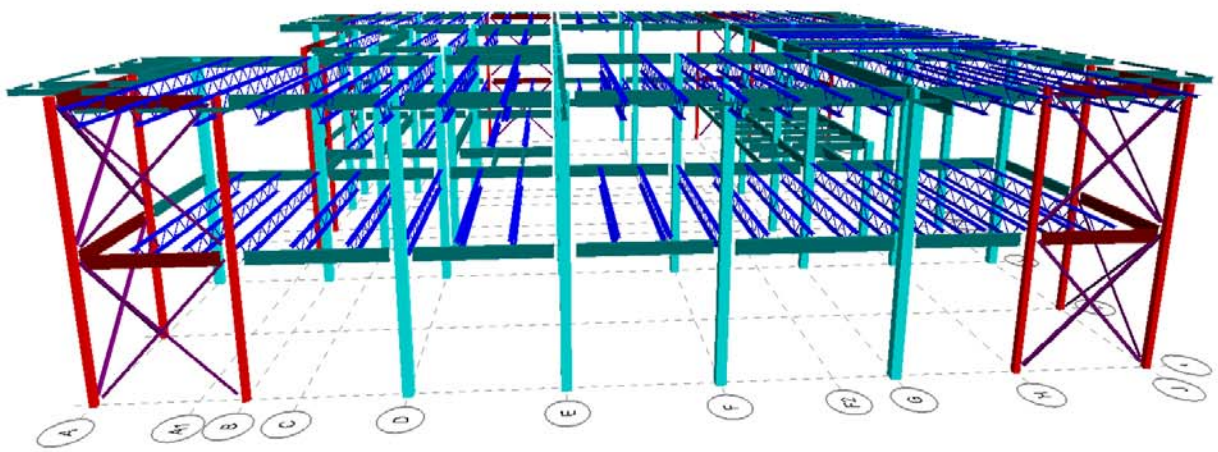
South Elevation of Chancery



North Elevation of Chancery

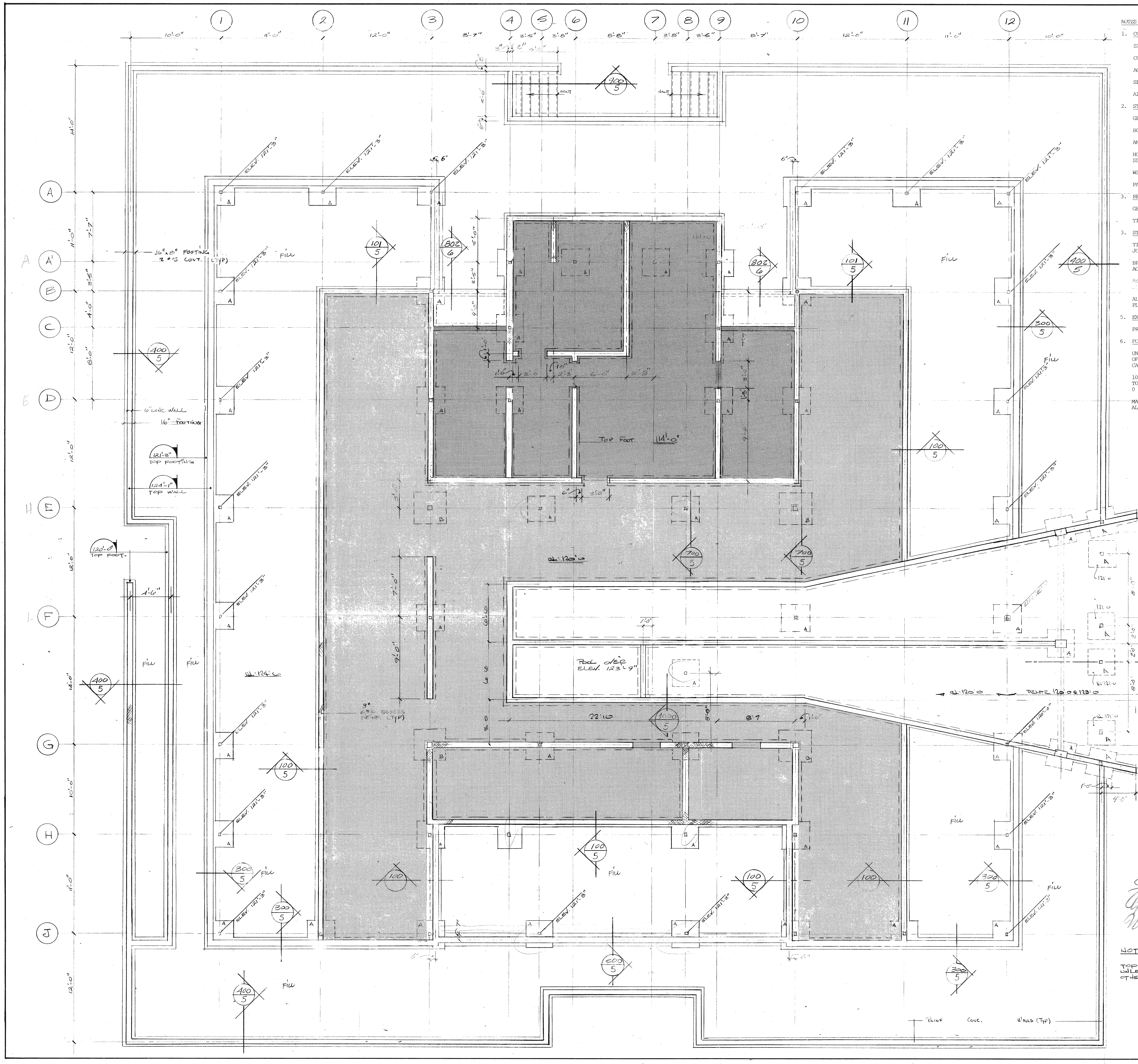


East Elevation of Chancery



West Elevation of Chancery

APPENDIX E
STRUCTURAL DRAWINGS



NOTES – GENERAL

1. CONCRETE

SETTING ACCORDING TO: CAN3-A23.1 M-77

COMPRESSIVE STRENGHT: 30 MPa TO 28/DAYS

AGGREGATE MAX. DIAMETER: 20 mm

SLUMP: 75 TO 125 mm

AIR ENTRAINED: GENERAL 3% TO 6%

2. STRUCTURAL STEEL

GENERAL: G40.21 GRADE 300 W

BOLTS: ASTM – A 325

ANCHORING BOLTS: G40.21 GRADE 300 W

HOLLOW STRUCTURAL SECTION : G40.21 GRADE 350 W CLASS H

WELDING: E 70xx W 59.1

PAINT: 1-GP – 40 M

3. REINFORCING STEEL

GENERAL: ACNOR G30.21M GRADE 400

TIES : ACNOR G30.12M GRADE 350

3. STEEL JOISTS (OPEN WEB)

TIE JOISTS: EXTEND BOTTOM CORD OF ALL JOISTS IN COLUMN AXIS, UP TO COLUMN.

BRACINGS: HORIZONTAL, SPACING & DIMENSIONS ACCORDING TO CAN 3-S16.1 M78

SHOES. ALL JOISTS BEARING SHOES SHALL [illegible] mm HIGH

ALL JOISTS SHALL BE OF HEIGHT INDICATED ON PLANS

5. EXPAND GROUT

PROVIDE 25 mm OF EXPAND GROUT UNDER COLUMN

6. FOUNDATIONS

UNDER FOOTING AND SOLE PIECES, PROVIDE 100 mm OF COMPACTED CRUSHED STONE 0 – 20 WHERE NO ROCK CAN BE FOUND

100 mm CONCRETE SLAB WITH A CAPACITY OF 25 MPa TO 28/DAYS, LYING ON 200 mm CRUSHED STONE 0 – 20 COMPACTED

MAKE A SAW-CUT ACCORDING TO DETAILS F.1 ON ALL COLUMN AND FILL WITH HORNFLEX

POUR CONSTRUCTION
84-10-26

Shop Drawings #15
Approved as noted
Nov 15.84

NOTE:

TOP FOOTING ELE. 120'-0" UNLESS INDICATED OTHER WISE.

consortium
crs

STEVE PINEAU
INGÉNIEUR · ENGINEER
PROVINCE DE QUÉBEC

Code	Date	Description	Par

REVISIONS

Propriétaire:

EXTERNAL AFFAIRS CANADA

Site:

Bridgetown, Barbados

Titre du projet:

new canadian high commission

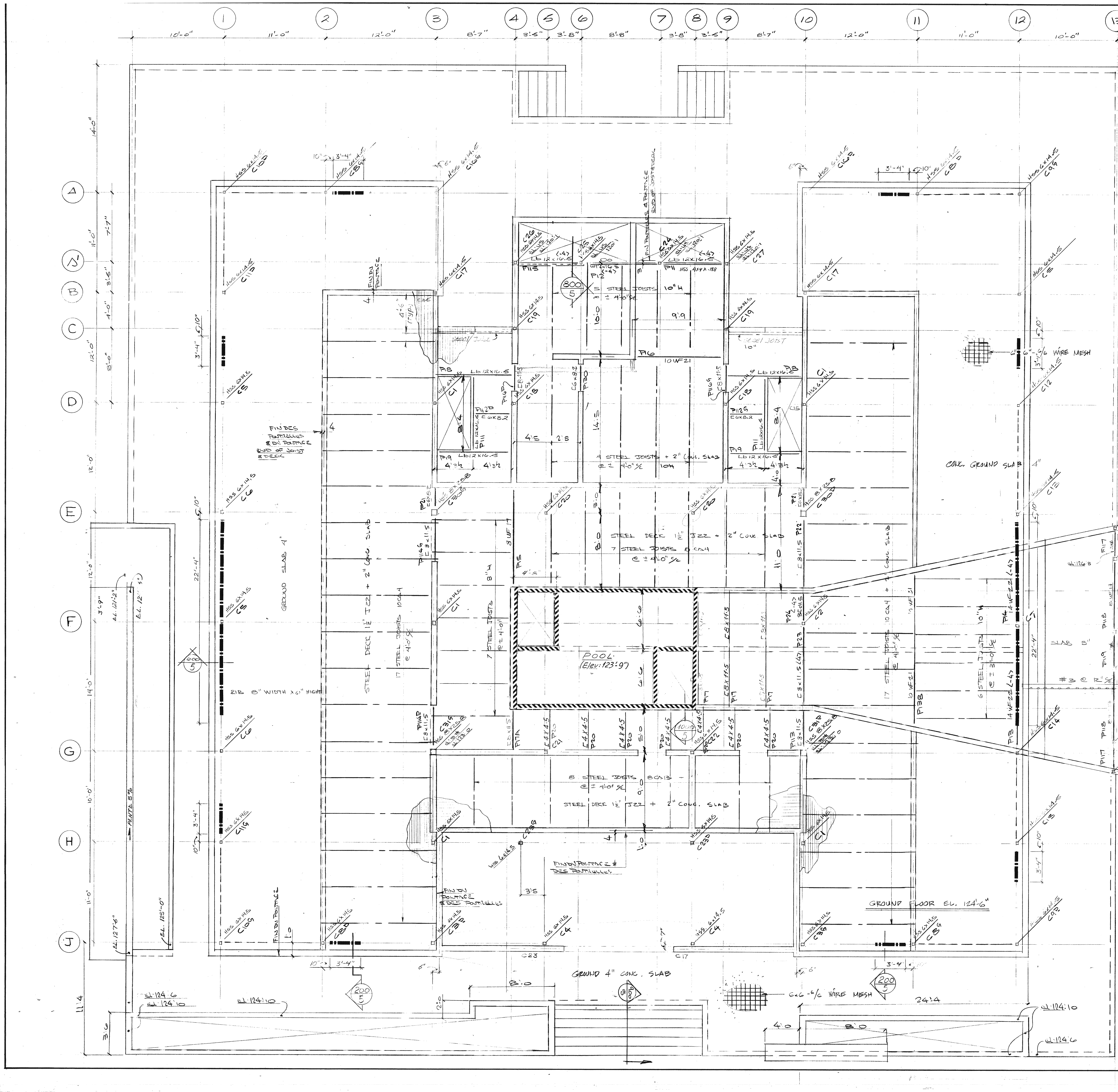
Titre du dessin:

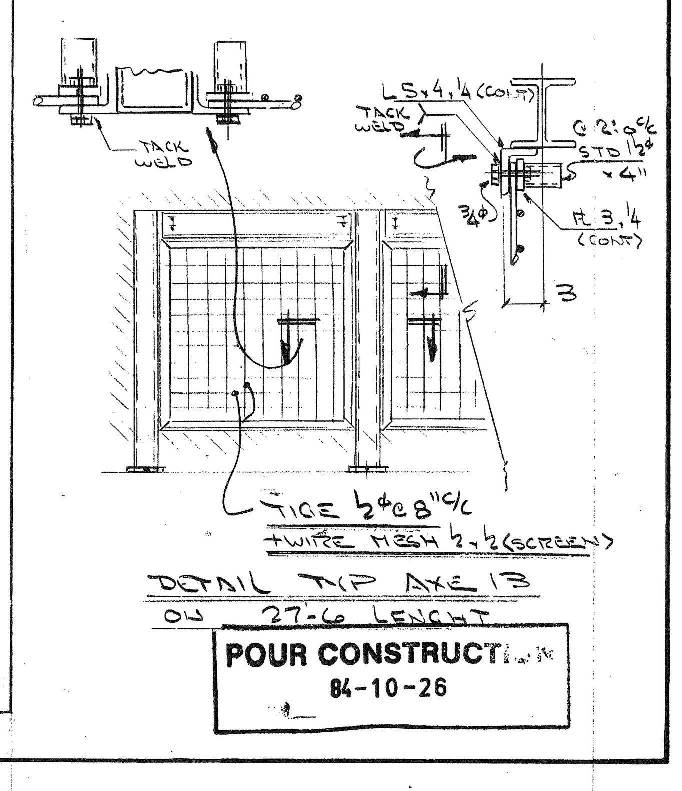
structure foundations plan

Projet n°:	Échelle: 1/4" = 1-0"
Dessiné par: s.p.	Date: May 11, 1984
Vérifié par: e.d.	Approuvé par: j-c.g.
Date pour soumission 84-05-15	Date pour construction

L'Entrepreneur devra vérifier toutes les dimensions avant de débuter les travaux.

Dessin n°: 1 de 8





NOTES – GENERAL

STEEL DECK

1. ALL STEEL DECK WILL BE GAUGE 22, SATIN FINISH AND 33 mm HIGH

ON FLOOR, IT WILL BE BOTTOM-PUNCHED EVERY 600 mm AND WELDERED TO THE JOISTS EVERY 600 mm CENTER TO CENTER

ON ROOF, IT WILL BE BOTTOM-PUNCHED EVERY 450 mm AND WELDERED (WITH WASHERS 3/4" Ø I.D.) TO THE JOISTS EVERY 450 mm CENTER TO CENTER

2. CONCRETE SLAB

90 mm THICK CONCRETE SLAB WILL COVER STEEL DECK. (50 mm OVER HIGH RIB)

WIRE MESH

ALL WIRE MESH WILL BE WELDED IN PANELS CATEGORIES 150 X 150 X 6/6

3. CONCRETE MASONRY

ALL MASONRY COMPONENTS WILL BE FIXED TO FRAME WORK (BEAMS, COLUMNS, JOISTS) AS INDICATED ON DETAILS, DRAWING #8.

HORIZONTAL REINFORCEMENT WILL BE THREE-POINTED EXTRA HEAVY DUTY, AS MANUFACTURED BY DUR-O-WALL OR EQUIVALENT, AT EVERY SECOND ROW.

VERTICAL REINFORCEMENT WILL CONSIST OF BARS 15 m TO 1200 C/C FOR ALL WALLS OR AS OTHERWISE INDICATED.

FOUNDATION WALLS PROTECTION – WALLS BELOW GRADE

- ALL EXTERIOR WALLS BELOW GRADE WILL BE WATERPROOFED ON THE EXTERIOR SURFACE EXTENDING FROM THE FOUNDATION PAD TO ABOVE-GRADE.
- WATERPROOFING WILL BE WITHER ASPHALT ACCORDING TO ASTM D-449, TYPE A, OR COAL-TAR PITCH ACCORDING TO ASTM D-450, TYPE B, OR AS OTHERWISE APPROVED.

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STEVE PINEAU

Code	Date	Description	Par
		REVISIONS	

Propriétaire:
EXTERNAL AFFAIRS CANADA

Site:
Bridgetown, Barbados

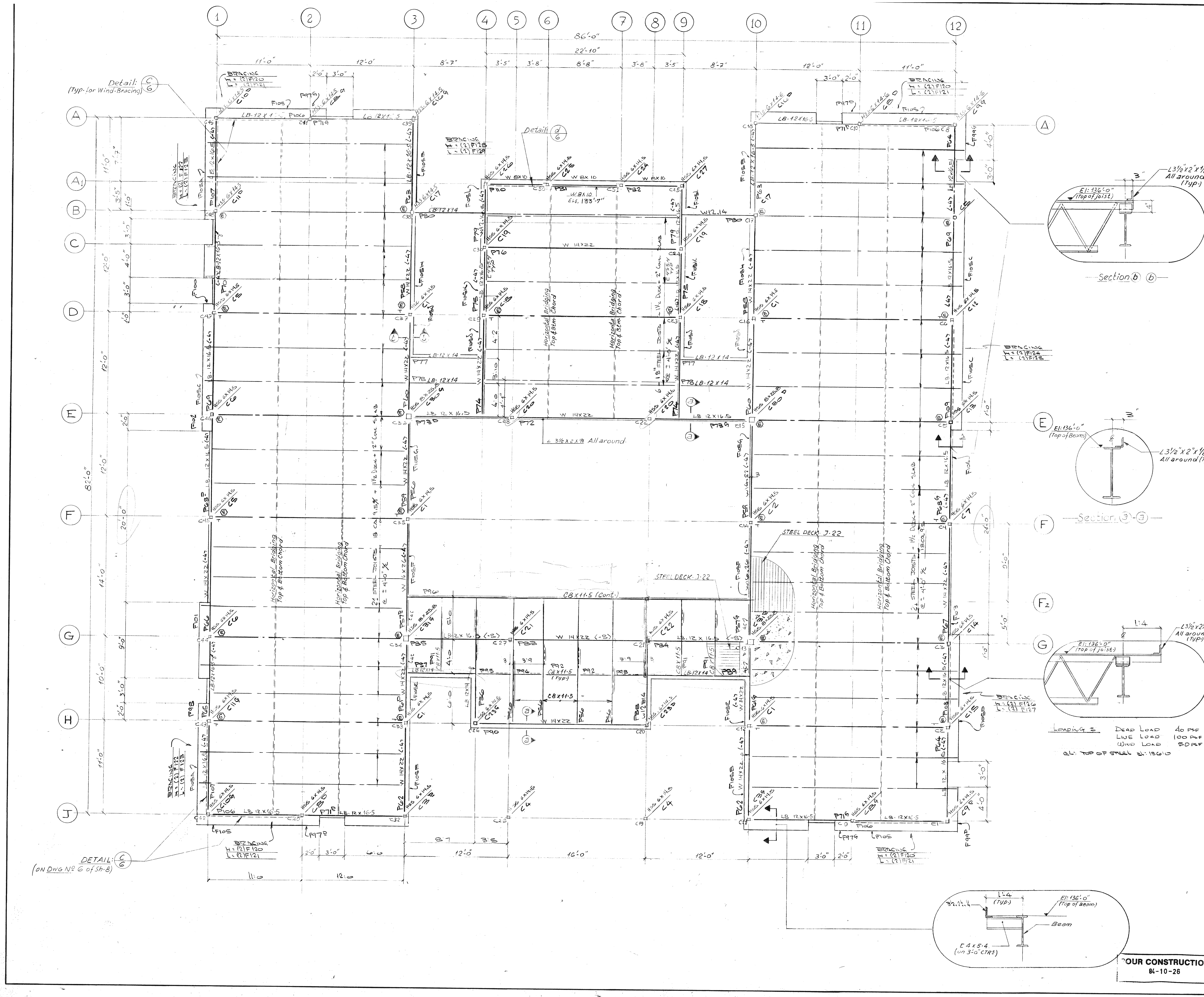
Titre du projet:
new canadian high commission

Titre du dessin:
structure ground floor

Projet n°:	Échelle: 1/4" = 1'-0"
Dessiné par: s.p.	Date: May 11, 1984
Vérifié par: e.d.	Approuvé par: j-c.g.
Date pour soumission 84-05-15	Date pour construction

L'Entrepreneur devra vérifier toutes les dimensions avant de débuter les travaux.

Dessin n°: 2 de: 8



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Code | Date | Description | Par

REVISIONS

Propriétaire:

EXTERNAL AFFAIRS CANADA

Site:

Bridgetown, Barbados

Titre du projet:

new canadian high commission

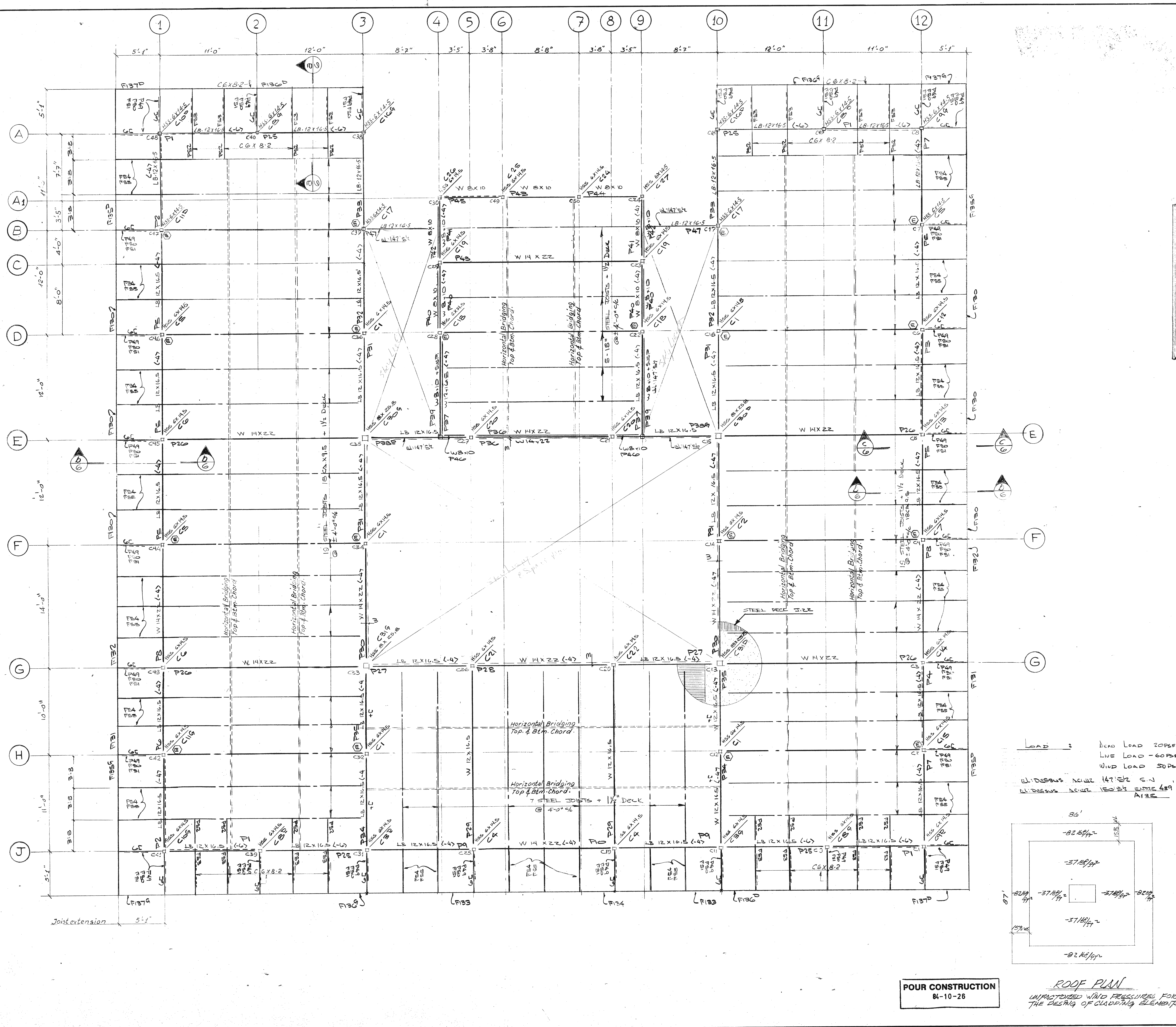
Titre du dessin:

structure second floor plan

Projet nº:	Échelle: 1/4"-1'0"
Dessiné par: s.p	Date: May 11, 1984
Vérifié par: e.d.	Approuvé par: j.c.g.
Date pour soumission 84-05-15	Date pour construction

L'Entrepreneur devra vérifier toutes les dimensions avant de débuter les travaux.

Dessin nº: 3 de: 8



POUR CONSTRUCTION
84-10-26

consortium
crs

Code	Date	Description	Par
		REVISIONS	

Propriétaire:
EXTERNAL AFFAIRS CANADA

Site:
Bridgetown, Barbados

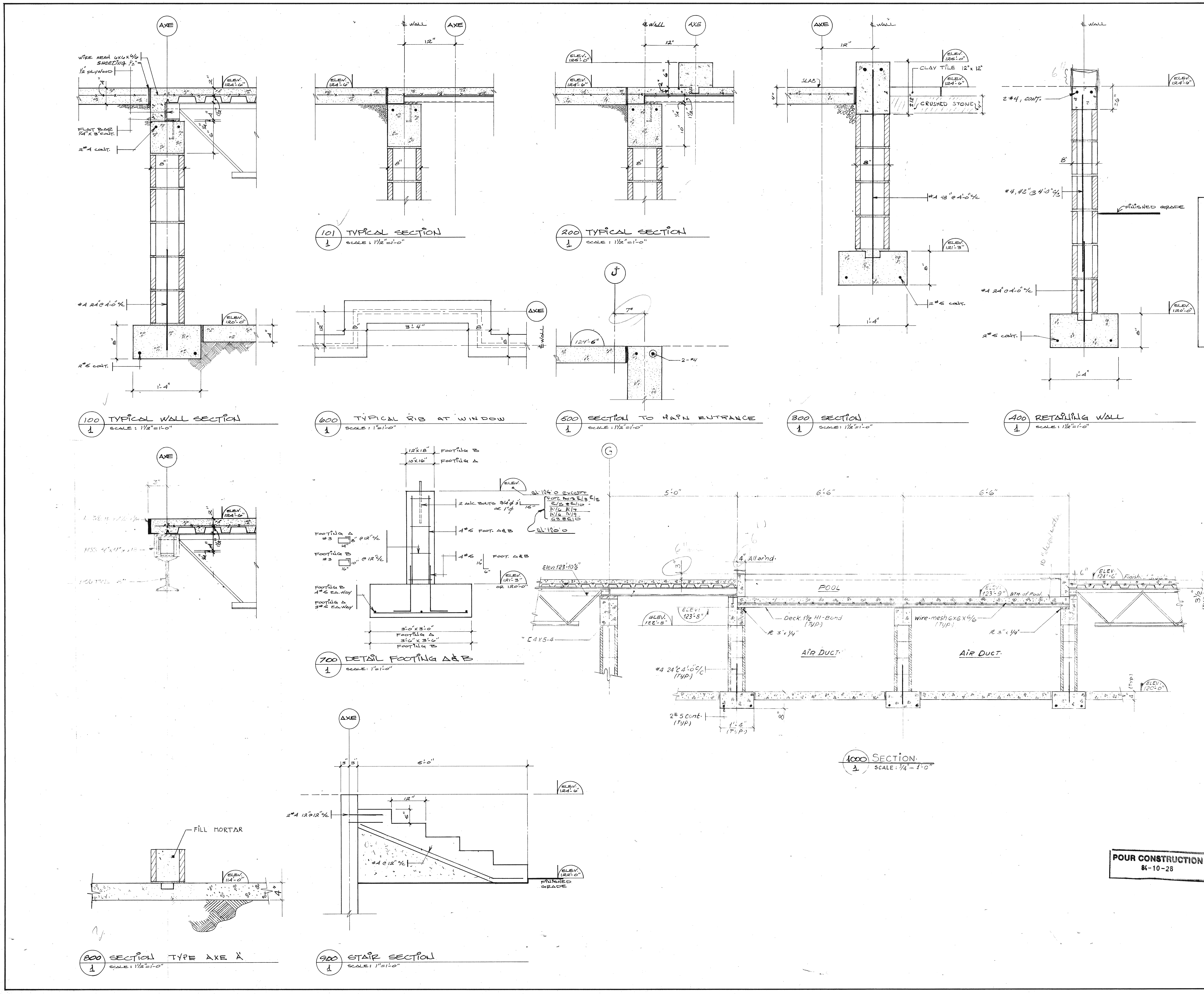
Titre du projet:
new canadian high commission

Titre du dessin:
structure roof plan

Projet n°:	Echelle: 1/4 = 1-0
Dessiné par: s.p.	Date: May 11,1984
Vérifié par: e.d.	Approuvé par: j-c.g.
Date pour soumission: 84-05-15	Date pour construction:

L'Entrepreneur devra vérifier toutes les dimensions avant de débuter les travaux.

Dessin n°: 4 de 8



consortium crs

Code | Date | Description | Par

REVISIONS

Propriétaire:

EXTERNAL AFFAIRS CANADA

Site:

Bridgetown, Barbados

Titre du projet:

new canadian high commission

Titre du dessin:

STRUCTURE DETAILS

Projet n°:

Échelle: INDICATED

Dessiné par: A.ST-L.

Date: May 11, 1984

Vérifié par: e.d.

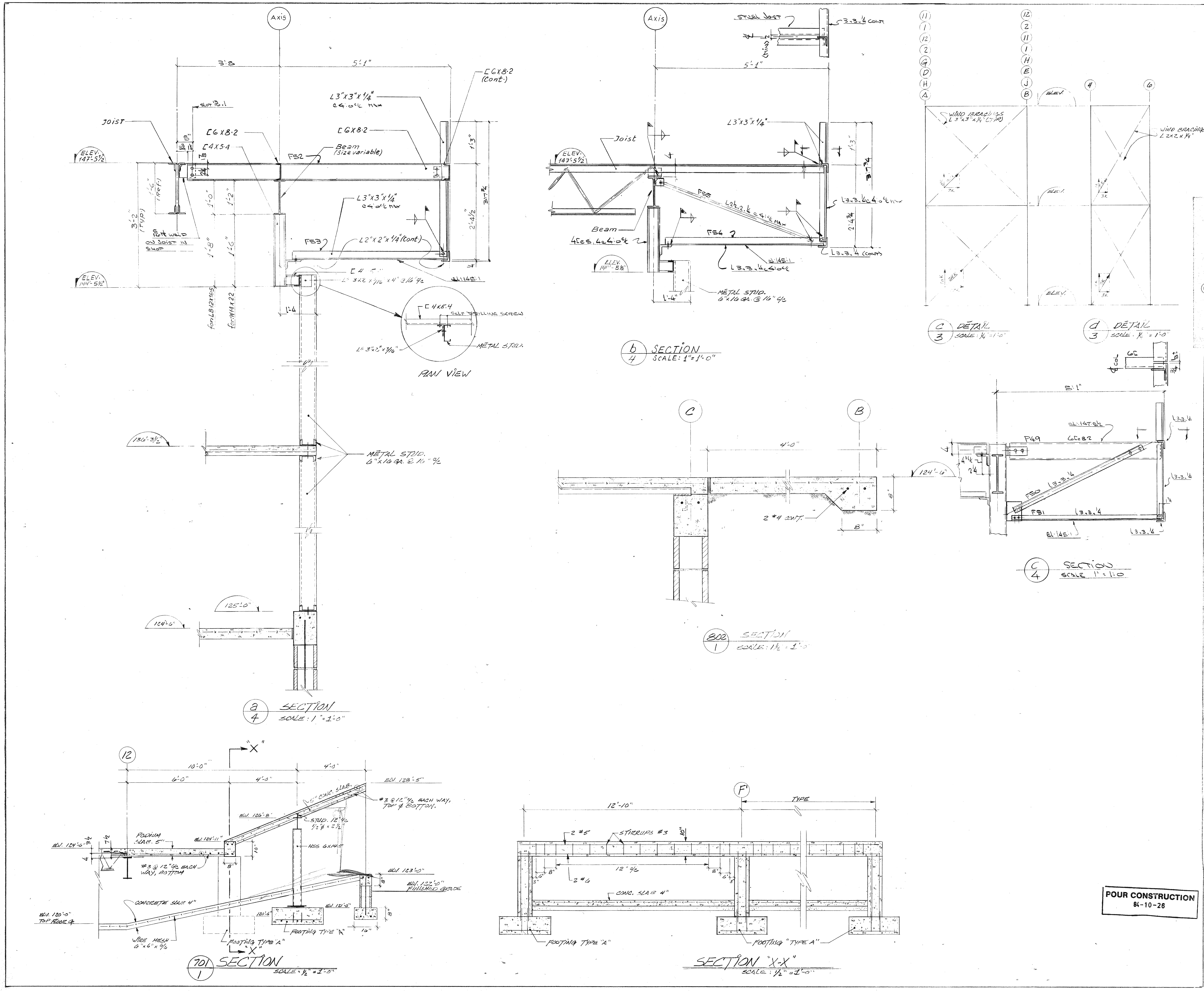
Approuvé par: j.-c.g.

Date pour soumission: 84-05-16

Date pour construction

L'Entrepreneur devra vérifier toutes les dimensions avant de débuter les travaux.

Dessin n°: 5 de: 8



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Code	Date	Description	Par

REVISIONS

Propriétaire:

EXTERNAL AFFAIRS CANADA

Site:

Bridgetown, Barbados

Titre du projet:

new canadian high commission

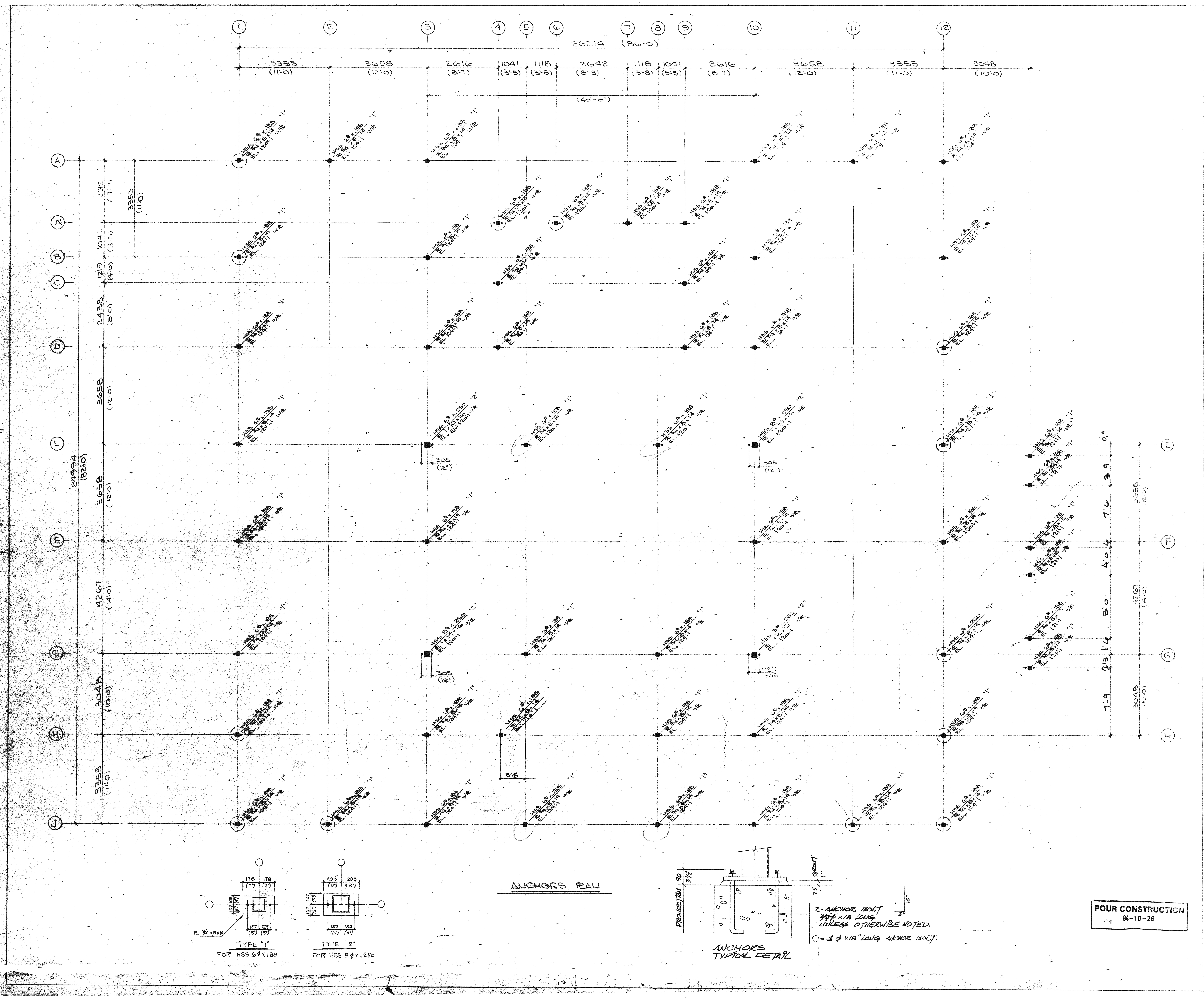
Titre du dessin:

structure details & sections

Projet n°:	Echelle: INDICATED
Dessiné par: G·D·H·	Date: May 11, 1984
Vérifié par: e.d.	Approuvé par: j-c.g.
Date pour soumission 84-05-15	Date pour construction

L'Entrepreneur devra vérifier toutes les dimensions avant de débuter les travaux.

Dessin n°: 6 de 8



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crs

Code Date Description Par
REVISIONS

Propriétaire:
EXTERNAL
AFFAIRS
CANADA

Site:
Bridgetown, Barbados

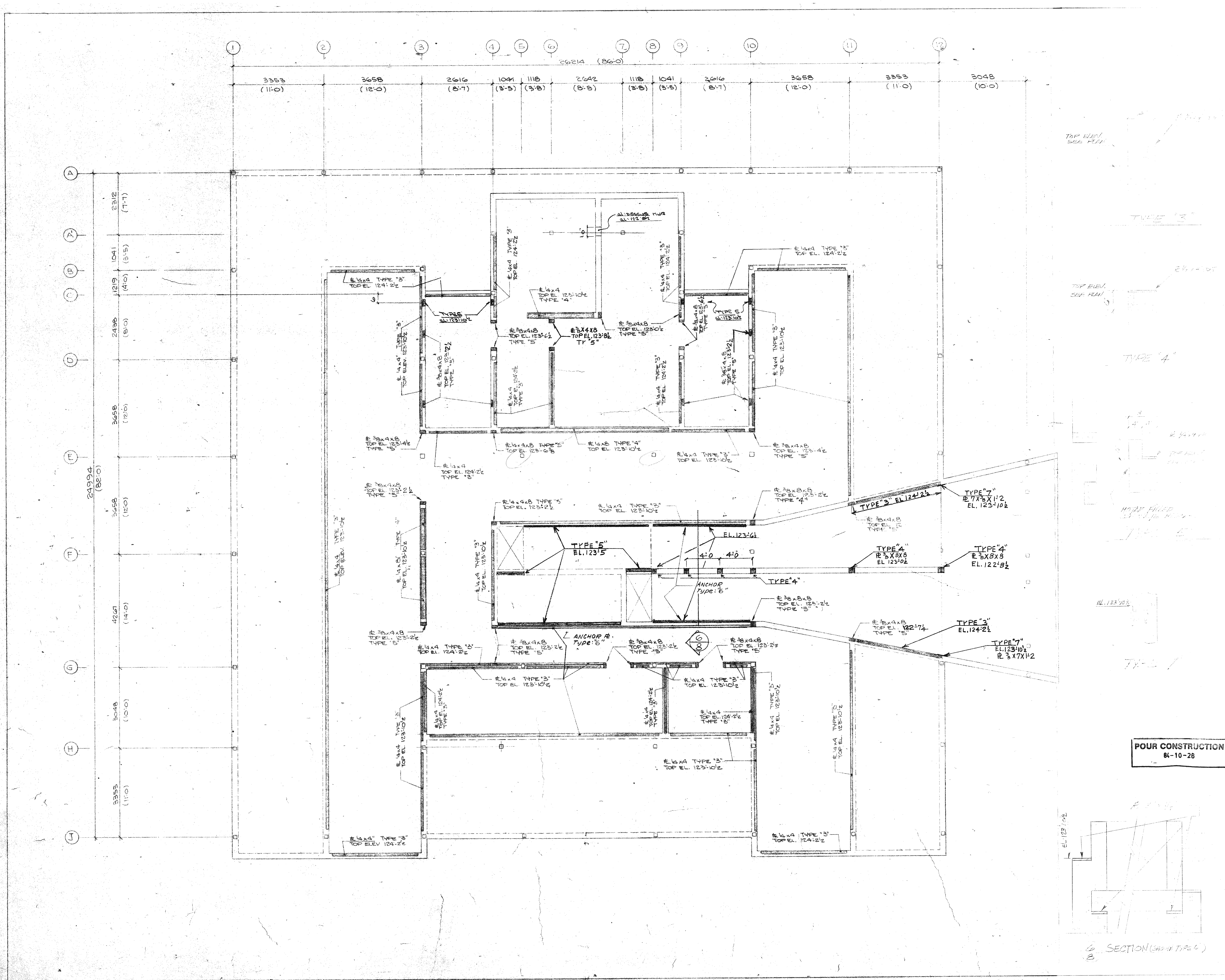
Titre du projet:
new canadian high commission

Titre du dessin:
STRUCTURE DETAILS

Projet n°:	Echelle:
Dessiné par:	Date: May 11, 1984
Vérifié par: e.d.	Approuvé par: j-c.g.
Date pour soumission	Date pour construction

L'Entrepreneur devra vérifier toutes les dimensions avant de débuter les travaux.

Dessin n°: 7 de 8



consortium
crs

Code	Date	Description	Par
		REVISIONS	

Propriétaire:

EXTERNAL AFFAIRS CANADA

Site:

Bridgetown, Barbados

Titre du projet:

new canadian high commission

Titre du dessin:

STRUCTURE DETAILS

Projet n°: Echelle:

Dessiné par: Date: May 11, 1984

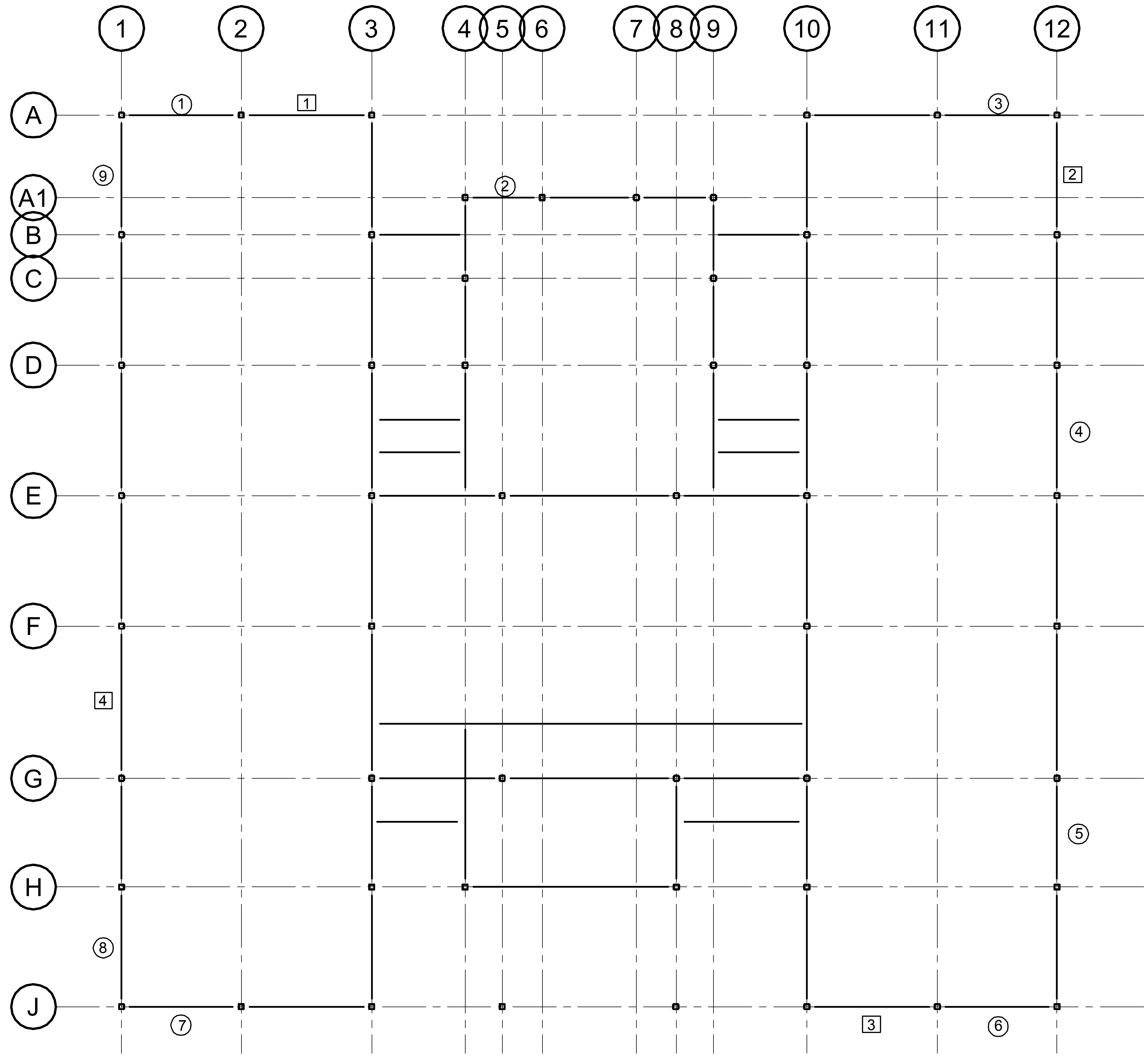
Vérifié par: e.d. Approuvé par: j.-c.g.

Date pour soumission Date pour construction

L'Entrepreneur devra vérifier toutes les dimensions avant de débuter les travaux.

Dessin n°: 8 de 8

APPENDIX F
BRACE LAYOUT DRAWING



(#) - EXISTING BRACED FRAME

[#] - PROPOSED ADDITIONAL BRACED FRAME

NOTES:

ALL BRACED FRAME MEMBERS AS FOLLOWS:

-COLUMNS
 - HSS 6x6x13
-BEAMS
 - W12x16
-DIAGONAL BRACES BAYS 1,3-9
 - L3x3x¼"
-DIAGONAL BRACES BAY 2
 - L2x2x¼"

PROJECT:	DRAWING:			DESIGN:	DRAWING NO.:
SEISMIC EVALUATION OF BRIDGETOWN CHANCERY	SECOND FLOOR STEEL FRAMING AND BRACED BAY LAYOUT	J.L.Richards ENGINEERS·ARCHITECTS·PLANNERS	J.L. Richards & Associates Limited 864 Lady Ellen Place Ottawa, ON Canada K1Z 5M2 Tel: 613 728 3571 Fax: 613 728 6012	DRAWN: DP CHECKED: DP PLOTTED: Jul 07, 2010	S1 JLR NO: 23423-03

File Location: P:\23000\23423-03 DFAIT Seismic - B Americas\Bridgetown\Chancery\Report\Appendix E - Structural Drawings\Drawing 1.dwg