REPORT ON PHASE 1
SEISMIC RE-EVALUATION
CANADIAN CHANCERY
BRIDGETOWN, BARBADOS

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1.0 BACKGROUND

This report presents the results of a re-evaluation of the Canadian Chancery in Bridgetown, Barbados. The original report was submitted in November 2011; this report presents the results of a re-analysis and detailed structural evaluation based on the new geotechnical investigation findings.

This report was prepared in accordance with the Standing Offer Agreement SO-ARP-AMS-SEISO-7074B and Call-Up Number ARD 154069/TBSS-100-CU12, and J.L. Richards & Associates Limited's proposal dated August 3, 2012.

The scope of work for this project is as follows:

- Re-Analysis and Evaluation based on the new Geotechnical findings (Site Class A);
- Linear Dynamic Analysis and 3-D model;
- Analysis and Evaluation in accordance with the 2010 National Building Code of Canada (NBCC) and associated material codes;
- Analysis of the structure at both normal and post-disaster importance levels; and
- Preparation of a technical memorandum presenting the findings of the re-analysis signed and sealed by a professional engineer licensed to practice in the province of Ontario.

DFAIT will determine if Component 2, options re-analysis, will proceed, based on a review of this report.

1.1 Description of Building and Previous Evalaution Results

The Bridgetown High Commission is owned by DFAIT and the level of performance is specified as Immediate Occupancy (IO). 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. The façade consists of glazing, concrete masonry and gypsum wall board partitions.

The Bridgetown High Commission is a steel braced frame structure consisting of steel hollow structural section (HSS) columns, steel beams and diagonal steel braces. The results of the original analysis indicated that the seismic force resisting system (SFRS) does not have adequate capacity to perform to the IO performance objective during the design seismic event in accordance with the 2010 NBCC and ASCE/SEI 31-03. The diagonal steel braces were found to have demand/capacity ratios greater than 1.0, the foundation uplift capacity was inadequate for the design loads, and the steel deck diaphragms of the second storey and the roof were inadequate to transfer seismic loads to the SFRS. The combination of these three issues indicated that the SFRS 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 inoperation after the design seismic event, and potential injury to building occupants.

2.0 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.

- Geotechnical Report prepared by Golder Associates, dated April 2011.

In the previous evaluation a Seismic Site Class C was assumed. However, the report prepared by Golder Associates recommended a Seismic Site Class A be used. Table 1 presents the Seismic Site Class and design spectral acceleration values used. The design spectral accelerations provided in Table 1 include the F_a and F_v factors.

Table 1: Seismic Hazard Data

Site Class	C	A
Fa	1	0.8
Fv	1.3	0.8
Spectral Period	Design Spectral Acceleration	Design Spectral Acceleration
0.2	1.263	1.01
0.5	1.263	0.804
1	0.672	0.414
2	0.373	0.23
PGA	0.545	0.436

The design seismic hazard curve for the spectral acceleration coordinates provided by DFAIT Site Class C and Site Class A, are presented in Figure 1.

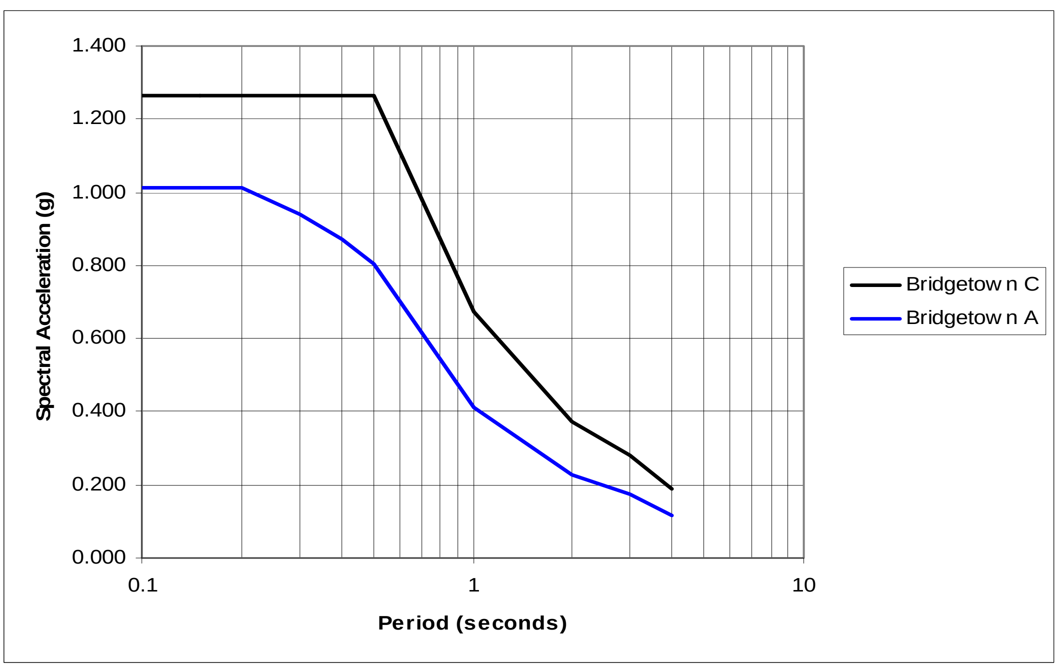

Figure 1: Seismic Hazard Curve- Bridgetown, Barbados

3.0 ANALYSIS PRODEDURE

The Bridgetown High Commission was analyzed and evaluated based on the 2010 NBCC. The SFRS was evaluated using the Dynamic Analysis Procedure, as described in the 2010 NBCC. A 3D model of the High Commission was developed to complete the required analysis. The base shear determined by the dynamic analysis were calibrated to be equal to 80% of the Equivalent Static Force Procedure (ESFP) base shears as described in the 2010 NBCC. The main parameters used to complete the ESFP are outlined below: If the parameters from this re-evaluation differ from those of the previous evaluation the parameters are presented in table format.

- 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;
  - the design spectral accelerations based on a maximum first mode period of 0.36 seconds are presented in Table 2 below.

Table 2: Design Spectral Accelerations

Site Class	$S(T_a)$
C	1.263
A	0.9

- 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 or 1.5 as indicated for normal and IO respectively.

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

- The equivalent static base shear was calculated as follows:

  V_{max} = (2/3) S(0.2) I_e W / ($R_d R_o$), V_{max} = 1,719 kN

Table 3: Base Shear Results

Site Class	Base Shear (KN)	80% Base Shear (KN)
C (I=1.0)	1,719	1,375
A (I=1.0)	1,375	1,100
A (I=1.5)	2,062	1,650

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).

Table 4: Dynamic Base Shear Results for Site Class A

Importance Factor	Dynamic Base Shear (KN)
I_E = 1.5	1,680
I_E = 1.0	1,125

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 5 below.

Table 5: Dynamic Modes Shapes and Frequencies

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

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 NBCC 2010 Structural Commentaries.

To account for torsion, the lateral loads in the equivalent static and dynamic analysis were applied at 5% eccentricity from the centre of mass, as specified in NBCC 2010 4.1.8.12 4) b).

For this re-evaluation the brace elements were considered to be “tension only” elements, which reflect the relative weakness of the slender braces in compression compared to tension. In a “tension only” analysis the compression braces are assumed to have failed and become ineffective, leaving only the tension braces to carry the lateral loads. The effect is that the brace and column loads are increased. This analysis method more accurately models the actual failure mode of the SFRS.

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 6 below.

Table 6: Material Properties

Material	Property	Value
Hollow Structural Steel	f_y	350 MPa
	E_s	200,000 MPa
Structural Steel	f_y	300 MPa
	E_s	200,000 MPa
Bolts	F_u	830 MPa

4.0 RESULTS OF EVALUATION

The structural calculations were completed in general conformance to the NBCC 2010 and the material codes referenced therein.

The following load combinations and factors were considered, as specified in the 2010 NBCC:

- 1.0D+0.5L+1.0E

4.1 Building Element Analysis

The SFRS in the Bridgetown High Commission consists of braced frames made up of steel 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.

4.2 Diaphragms

The re-evaluation results for the diaphragms at the roof and second level are relatively unchanged from the original analysis. However, the deficiencies are listed below in detail, so that DFAIT has a thorough understanding of the deficiencies. Also presented are the possible mitigation measures that could be implemented.

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 a 450 mm centre to centre spacing. 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 a 600 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 greater 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 and a 65 mm concrete topping because there is tabulated data for this case. The estimated strengths of the steel deck diaphragms were taken from the tabulated values included in the Design of Steel Deck Diaphragms (3rd Edition) by Canadian Sheet Steel Building Institute (CSSBI).

Due to the concrete topping on the second level diaphragm, it is possible that the second level diaphragm will have adequate capacity to transfer shear forces from the lateral loads.

Diaphragm Openings

Both the second level and roof have a large central diaphragm opening and two smaller openings on either side. At the second level the two smaller openings are framed by two beams and a narrow strip of deck, while at the roof level, the openings are framed by four beams. These openings present a problem for both shear transfer and the development of chord action at the perimeter of the diaphragm.

The potential hazard associated with these deficiencies is that the connection between the central portions of the deck surrounding the openings will fail (either the beams or the strips of deck). If these elements fail, the deck could collapse; which would potentially threaten the lives of the building occupants.

The failure mechanism involving the diaphragms will be fairly complex and dependent on the direction of the seismic waves and other factors. Nonetheless, there is a potential risk that portions of the deck may have very high forces concentrated in narrow strips due to presence of the openings. These forces will place large demands on beams and the concrete deck/roof deck that will very likely locally exceed the material strengths. This will possibly result in localized floor/roof surface failures and/or radical changes in the overall diaphragm performance to a more flexible diaphragm. This change will result in increased overall displacements of the structure, in particular the centre portion of the building. This further creates a hazard of localized failures of portions of floors and roof surfaces in the central portion of the building.

Perimeter Chord Element

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.

In addition, the perimeter chord element is not continuous and due to the large openings in the deck, there are several locations where it is not possible to transfer the shear loads required through the deck to the chord elements.

The potential hazard associated with this deficiency is the inability of the overall diaphragm to behave as a rigid body and an inadequate transfer of inertia forces from the roof to the SFRS. These deficiencies will likely lead to large local displacements, resulting in an increased risk of localized collapse or extensive damage to portions of the structure. The IO criteria would not be met and the Life Safety criteria would not be fully met.

Diaphragm Connections and Load Transfer

The second level diaphragm is expected to have adequate capacity to transfer lateral forces into the SFRS through bearing of the diaphragm on the columns and the existing connections. This is due, in part, to the concrete topping which adds additional bearing area between the diaphragm and columns.

The existing roof diaphragm does not have adequate capacity to transfer the shear forces to the SFRS, specifically around diaphragm openings, where the width is reduced and where the diaphragm is connected with light steel beams. Load transfer through the diaphragm depends on the ability of the diaphragm connections. These connections include; the connection from the deck to the joists and the joists to the beam, specifically the joist connection to the perimeter chord elements and the beams in the braced bay locations. These connection details are not indicated on the structural drawings and were not observed during the site visit. Failure of the diaphragm connections could lead to displacement of the roof; therefore any mitigation measures to the deck should include a thorough inspection of the connections. If the connections are determined to have inadequate capacity, mitigation measures would be necessary to increase the connection capacity.

The potential failure and collapse of even a portion of the diaphragm poses a threat to the building occupants and therefore should be treated very seriously. Possible options to mediate the diaphragm deficiencies include filling in some of the deck openings, using horizontal cross-bracing to stabilize the deck openings and the addition of new chord elements to stabilize the diaphragm.

4.3 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.

Columns are referenced according to their braced bay number as indicated in Figure 2. Structural steel capacities were evaluated in accordance with CAN/CSA S16-01 Limit States Design of Steel Structures.

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:

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

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

$$T_r = \phi A_g F_y$$

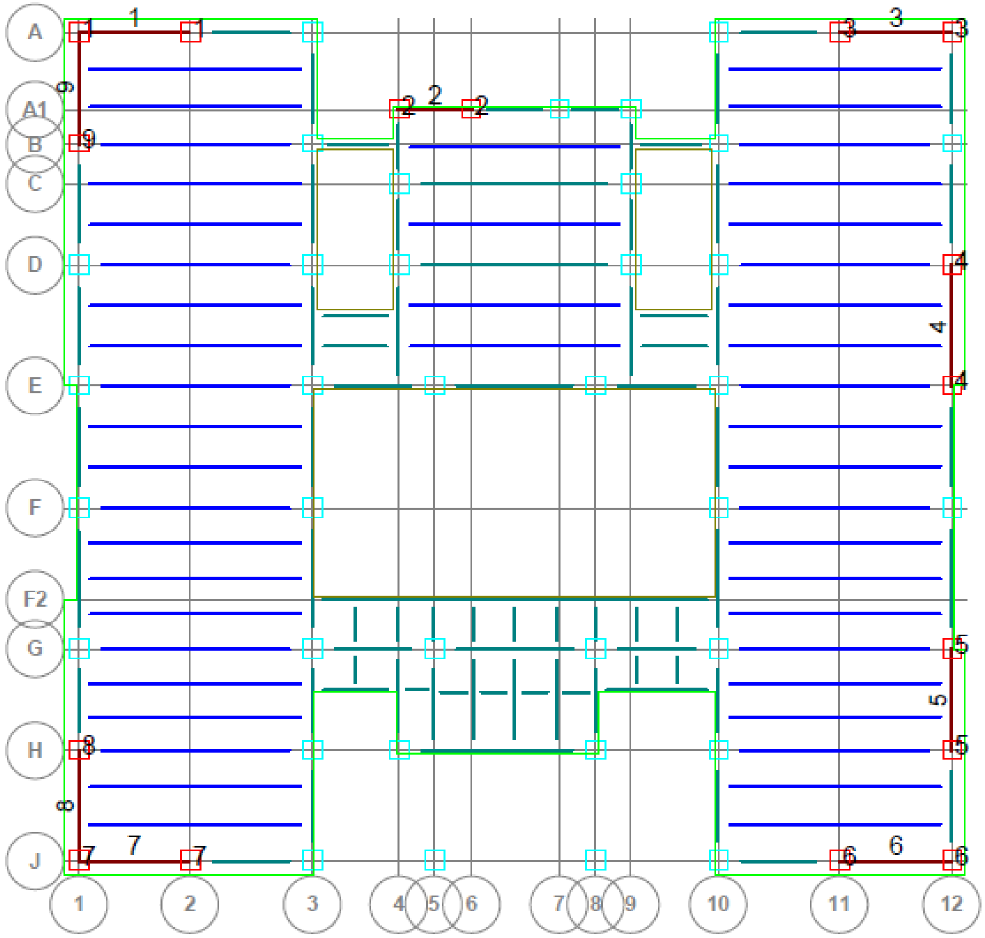

Figure 2: Braced Frame Locations (Second Floor and Roof)

Table 7 shows the maximum demand/capacity ratios for the columns in each of the braced frames. Demand/Capacity ratios for the columns are based on the required brace forces with R_dR_o taken as 1.3 (NBCC 4.1.8.12 7). Demand/Capacity ratios were not calculated based on the nominal brace capacity because the braces do not have adequate capacity to transfer the design loads (see Section 6.5). Demand/Capacity ratios greater than 1 indicate columns with insufficient capacity for the design seismic loading.

Table 7: Demand/Capacity Ratios for HSS Columns

Braced Frame Number	Level	Normal Occupancy I=1.0	Immediate Occupancy I=1.5
1	Roof – Second	0.46	0.66
	Second - Ground	1.11	1.62
2	Roof – Second	0.28	0.40
	Second - Ground	0.71	1.02
3	Roof – Second	0.37	0.55
	Second - Ground	0.97	1.43
4	Roof – Second	0.50	0.72
	Second - Ground	1.24	1.78
5	Roof – Second	0.51	0.73
	Second - Ground	1.29	1.85
6	Roof – Second	0.46	0.66
	Second - Ground	1.15	1.68
7	Roof – Second	0.46	0.67
	Second - Ground	1.15	1.67
8	Roof – Second	0.48	0.69
	Second - Ground	1.23	1.77
9	Roof – Second	0.47	0.68
	Second - Ground	1.22	1.76

† *Demand/capacity ratio greater than one represents an overstressed condition.*

‡ *Refer to Figure 2 for the location of the columns.*

As shown in Table 7 for normal occupancy (I=1.0), the demand/capacity ratios for the columns range from 0.28 to 1.29 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 eighteen columns are over 1.0, with 7 of 9 of the ground floor columns overstressed. 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 IO (I=1.5) condition range from 0.4 to 1.85, with all of the ground floor columns overstressed. The demand/capacity ratios indicate that the columns would probably be damaged during the design seismic event and may not meet the IO requirements; however, it is possible that the columns would still be able to carry gravity loads at these overstress levels and would not fail.

4.4 Steel Beams

The horizontal elements of the braced frames in the Bridgetown High Commission consist of steel beams. The previous evaluation indicated that the steel beams have adequate capacity for both normal occupancy and post-disaster occupancy. Based on the required capacity of the braces the beam forces were calculated with R_dR_o=1.3. The demand/capacity ratios for the steel beams located in the braced bays are presented in Table 8.

The location of these beams on the floor plan is shown in Figure 2.

Table 8: Demand/Capacity Ratios for Steel Beams

Beam‡	Level	Demand/Capacity Ratio I=1.0†	Demand/Capacity Ratio I=1.5†
1	2^{nd} - Ground	0.30	0.45
2	2^{nd} - Ground	0.30	0.45
3	2^{nd} - Ground	0.16	0.23
4	2^{nd} - Ground	0.36	0.53
5	2^{nd} - Ground	0.35	0.53
6	2^{nd} - Ground	0.34	0.52
7	2^{nd} - Ground	0.35	0.52
8	2^{nd} - Ground	0.35	0.53
9	2^{nd} - Ground	0.35	0.53
1	Roof - 2^{nd}	0.23	0.29
2	Roof - 2^{nd}	0.23	0.29
3	Roof - 2^{nd}	0.13	0.15
4	Roof - 2^{nd}	0.23	0.35
5	Roof - 2^{nd}	0.24	0.36
6	Roof - 2^{nd}	0.23	0.34
7	Roof - 2^{nd}	0.23	0.34
8	Roof - 2^{nd}	0.23	0.34
9	Roof - 2^{nd}	0.23	0.34

† *Demand/capacity ratio greater than one represents an overstressed condition.*

‡ *Refer to Figure 2 for the location of the beams.*

The demand/capacity ratios for the beams are all less than 1. 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 both design scenarios. No details are provided for the beam – column connections so the adequacy of the connection can not be commented on.

4.5 Diagonal Braces

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): Min ($T_r = \phi A_g F_y$, $T_r = \phi A_n F_u$)

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/SEI 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 structural analysis software package used for this analysis (Bentley – RAM structural analysis V8) has been updated, and now has the option to use a non-linear algorithm to track tension-only members during the solution of the structural model under applied lateral loads. This analysis option was not available during the original analysis.

The demand/capacity ratios for the diagonal steel braces in tension are shown in Table 9. Table 9 shows that the demand/capacity ratios for the braces in tension range from 0.80 to 1.6 for normal occupancy (I=1.0) with 15 out of 18 braces overstressed. The demand/capacity ratios for IO (I=1.5) range from 1.1 to 2.4, with all of the braces demand/capacity ratios greater than 1.0.

Table 9: Demand/Capacity Ratios for Diagonal Braces in Tension

Brace Bay Number‡	Level	Demand / Capacity Ratio†	
		Demand/Capacity Ratio I=1.0†	Demand/Capacity Ratio I=1.5†
1	2nd - Ground	1.3	2.0
2	2nd - Ground	1.3	2.0
3	2nd - Ground	1.2	1.8
4	2nd - Ground	1.5	2.3
5	2nd - Ground	1.6	2.4
6	2nd - Ground	1.5	2.3
7	2nd - Ground	1.5	2.3
8	2nd - Ground	1.6	2.3
9	2nd - Ground	1.6	2.3
1	Roof - 2nd	0.9	1.3
2	Roof - 2nd	0.8	1.3
3	Roof - 2nd	0.8	1.1
4	Roof - 2nd	1.0	1.5
5	Roof - 2nd	1.1	1.6
6	Roof - 2nd	1.0	1.5
7	Roof - 2nd	1.0	1.5
8	Roof - 2nd	1.0	1.5
9	Roof - 2nd	1.0	1.5

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

‡ *Refer to Figure 2 for location of the braced frames.*

The magnitude of overstresses for the normal occupancy performance level indicates that the braces will be damaged, particularly between the ground floor and the second floor. It is possible that the brace members may sustain damage and yield, which would increase the length of the structural period and provide a reduction in the overall seismic loads. Although the demand/capacity ratios indicate that the brace members are overstressed, it cannot be said with certainty that the brace members would fail entirely.

If the connection between the braces and the columns is insufficient, it is likely that the connection could fail under the design loads, rendering the brace ineffective. The loss of the brace connections would render the braced frames ineffective and result in additional loads being shifted to other braced bays. If all of the connections have failed, the structure would begin to perform as a pseudo moment frame structure. This structure would shift in behavior towards large displacements with an expectation of greater damage and possible localized collapse.

For the IO performance level the overstresses are large enough to cause the braces to fail. These overstresses, up to 2.4, would very likely cause the brace connections to fail, increasing the lateral loads in the columns and pose a threat to building occupants.

4.6 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 285 kN for normal occupancy and 430 kN for immediate occupancy. Uplift was calculated using R_dR_o=1.3. While this indicates that there is uplift on the braced bays the magnitude of the uplift, 255 kN to 400 kN, is easily overcome using rock anchors or similar commercially available tie down systems.

4.7 Load Transfer and Drift Limits

The NBCC specifies that for the seismic design of conventional braced bays, connections should be designed for either the seismic load multiplied by R_d (1.5) or the probable yield stress of the member (1.1 x F_y). The connection details of the SFRS elements (diaphragms, braces, columns and beams) are not known. Adequate performance of the SFRS to ensure life safety requires that the connections remain functional. Therefore to fully determine the performance of the SFRS and possible mitigation required, the connection details should be determined. Other than the diaphragm connections discussed in Section 4.2 the critical SFRS connections are between the brace elements and the columns, the beams and the columns, and the columns and the foundations. From photographs taken during the site visit, the brace to column connection appears to consist of 3-22 mm diameter A325 M bolts, with a shear capacity of 106 kN/bolt. These bolts would have an approximate shear capacity of 318 kN to transfer the seismic forces. The column to baseplate connection consists of 2-3/4” anchor bolts with a capacity of 280 kN, this connection has adequate capacity for the normal occupancy column loads but is inadequate for the post-disaster load level.

The NBCC specifies drift limits for normal occupancy buildings of 0.025 h_s and 0.01 h_s for IO buildings. The average storey displacements were determined to be approximately 45 mm and 61 mm for the roof and the second level respectively (assuming the diaphragm remains rigid). These displacements are within the limits of 85 mm and 90 mm, for the roof and the second level respectively, for normal occupancy buildings. The storey displacements exceed the limits of 34 mm and 36 mm, for the roof and the second level respectively, for post-disaster buildings. Drift limits are set to limit damage due to displacements. The higher than recommended drift values indicate that unless the displacements are reduced, the structure will probably not perform to the IO performance level. It is expected that if the diaphragm loses its rigid capacity, the displacements will be larger than the values calculated above, particularly in the weak diaphragm direction.

5.0 RECOMMENDATIONS

The recommendations described below are based on the following assumptions:

- Eccentricity of 5%.
- The maximum allowable period of the structure was limited to 2 x the code equation period.
- Dynamic base shears calibrated to be equal to 80% of the base shears calculated using the ESFP.
- Braces were considered to be tension only members.
- Brace forces calculated using R_dR_o=1.5 x 1.3.
- Column, beam, diaphragm and connection forces were calculated using R_dR_o=1.0 x 1.3.
- Displacement and drift were calculated using R_dR_o=1.0 x 1.0.

It is recommended that the Bridgetown Chancery be retrofit to perform to the normal occupancy level. The purpose of the retrofit would be to safeguard the life safety of building occupants, while limiting damage to the structure and reducing possible shutdown time following a seismic event.

The following steps are recommended going forward with this project:

1) Following review and approval by DFAIT, JLR will proceed with Component 2 of this project options re-analysis and the development of retrofit strategies.

2) Develop a non-linear model to be analyzed using time-history analysis techniques. A non-linear model is required to model friction dampers, but will also be useful when considering the non-linear behaviour of elements in all of the retrofit approaches.

3) Evaluate the three retrofit strategies specified by DFAIT: replacing or adding to the existing bracing elements with either a) Conventional braces, b) Buckling Restrained Braces, or c) Friction Dampers.

4) Evaluate the impact of the brace retrofit strategies on the roof diaphragm. Diaphragm deficiencies should be mitigated by the braces wherever possible.

5) Develop retrofit strategies for all components of the SFRS for each retrofit strategy in sufficient detail (including preliminary drawings) to determine the approximate cost of implementation.

6) Determine the schedule and approach to implement the retrofit strategy with the operational needs of the Mission as a guide.

6.0 SUMMARY AND CONCLUSION

A re-analysis of the Bridgetown Chancery based on the results of the Geotechnical Investigation was performed. It was determined that the building would likely not meet the IO performance objectives. Although the seismic forces were reduced by 20%, the results of the analysis remain similar to the previous study.
The configuration of the diaphragm with significant openings and minimal horizontal restraint prevents shear transfer to the SFRS and perimeter chord elements. This deficiency poses a life safety hazard to the building occupants because if the deck elements fail, portions of the deck could potentially collapse.

The columns and braces of the SFRS do not have adequate capacity to resist the design seismic forces. Demand/capacity ratios for the normal occupancy condition in the 1.5 range indicate that these elements would likely be damaged, while demand/capacity ratios greater than 2 for the IO level indicate that the elements may fail.

Additionally, uplift remains a problem for the foundation elements. The capacity of the foundation elements to resist overturning is far less than the design overturning loads. Finally the storey drifts exceed the limits for IO buildings.

While these deficiencies are serious, particularly the diaphragms, the magnitude of the overstresses combined with the construction of the Chancery itself, provide a variety of feasible retrofit approaches. It is recommended that this project proceed to the options analysis component. There are several possible options for retrofit of the Chancery structure, that could be implemented quickly, economically and with minimal disturbance while enhancing the seismic performance of the structure and greatly increasing the life safety of the building occupants.

This report has been prepared for the named client, for the stated purpose, for the named facility. Its discussions and conclusions are summary in nature and cannot properly be used, interpreted or extended to other purposes without a detailed understanding and discussions with the client as to its mandated purpose, scope and limitations. The report was prepared for the sole benefit and use of DFAIT and may not be used or relied on by any other party without the express written consent of J.L. Richards & Associates Limited. The report is copyright protected and may not be reproduced or used, other than by DFAIT for the stated purpose, without the express written consent of J.L. Richards & Associates Limited.

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PROVINCE OF ONTARIO

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APPENDIX A

COMPARISON OF UPDATED DEMAND/CAPACITY RATIOS TO VALUES FROM ORIGINAL REPORT

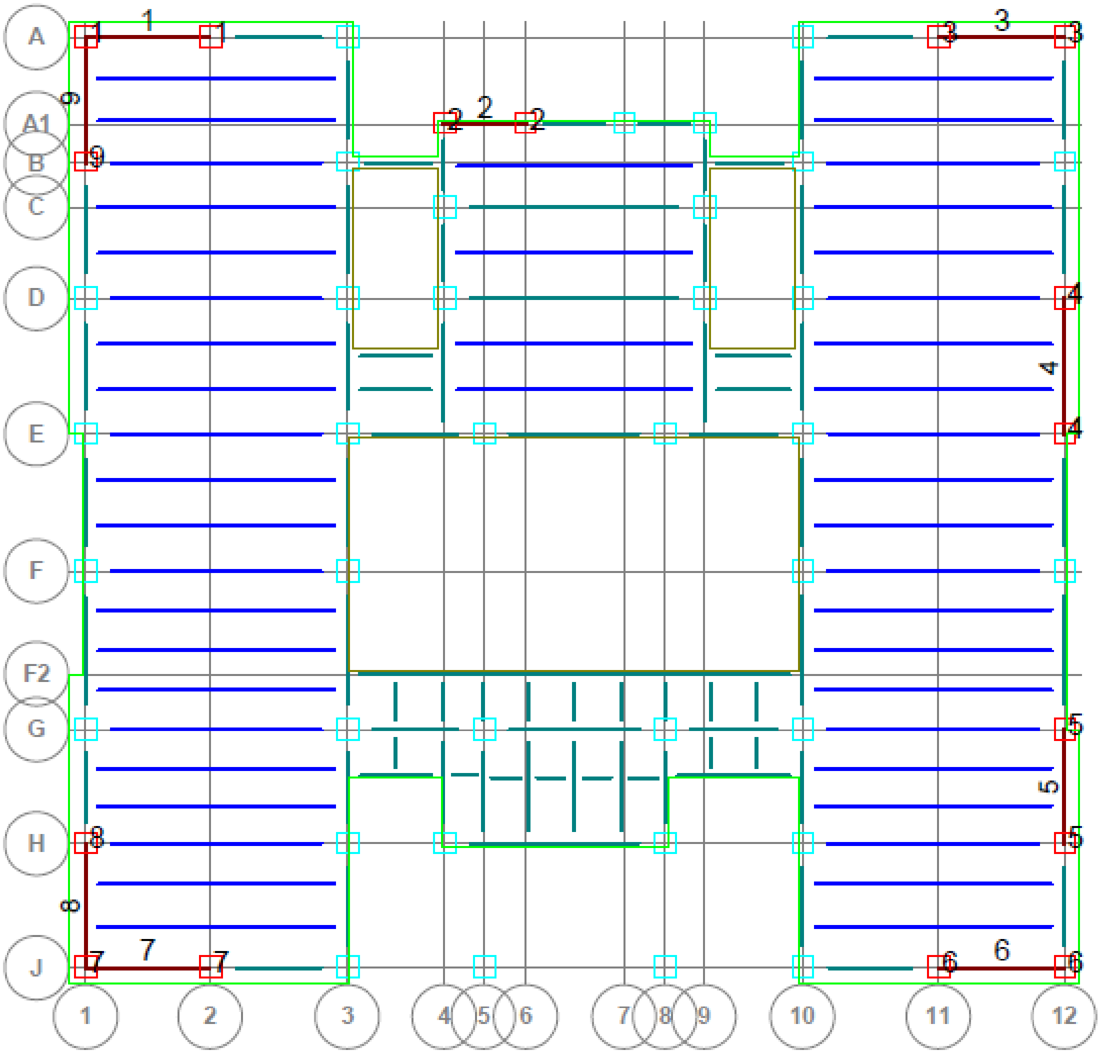

Figure 1: Braced Frame Locations (2nd Floor and Roof)

Table 1: Original Demand/Capacity Ratios for HSS Columns

Braced Frame Number‡	Level	Demand/Capacity Ratio I=1.0†	Demand/Capacity Ratio I=1.5†
1	Roof – 2nd	0.38	0.59
	2nd - Ground	1.06	1.71
2	Roof – 2nd	0.29	0.46
	2nd - Ground	0.86	1.38
3	Roof – 2nd	0.38	0.59
	2nd - Ground	1.11	1.75
4	Roof – 2nd	0.36	0.52
	2nd - Ground	0.97	1.44
5	Roof – 2nd	0.36	0.53
	2nd - Ground	1.04	1.55
6	Roof – 2nd	0.4	0.52
	2nd - Ground	1.03	1.5
7	Roof – 2nd	0.4	0.52
	2nd - Ground	0.98	1.51
8	Roof – 2nd	0.33	0.49
	2nd - Ground	0.99	1.51
9	Roof – 2nd	0.34	0.53
	2nd - Ground	1.01	1.6

† *Demand/Capacity Ratio greater than one represents an overstressed condition.*
‡ *Refer to Figure 1 for the location of the columns.*

Table 2: Updated Demand/Capacity Ratios for Site Class A for HSS Columns

Braced Frame Number	Level	Normal Occupancy (max) I=1.0	Immediate Occupancy (max) I=1.5
1	Roof – 2nd	0.27	0.42
	2nd - Ground	0.76	1.22
2	Roof – 2nd	0.21	0.33
	2nd - Ground	0.61	0.98
3	Roof – 2nd	0.27	0.42
	2nd - Ground	0.79	1.25
4	Roof – 2nd	0.26	0.37
	2nd - Ground	0.69	1.03
5	Roof – 2nd	0.26	0.38
	2nd - Ground	0.74	1.10
6	Roof – 2nd	0.29	0.37
	2nd - Ground	0.73	1.07
7	Roof – 2nd	0.29	0.37
	2nd - Ground	0.70	1.08
8	Roof – 2nd	0.24	0.35
	2nd - Ground	0.71	1.08
9	Roof – 2nd	0.24	0.38
	2nd - Ground	0.72	1.14

† *Demand/Capacity Ratio greater than one represents an overstressed condition.*
‡ *Refer to Figure 1 for the location of the columns.*

Table 3: Original Demand/Capacity Ratios for Steel Beams

Beam‡	Level	Demand/Capacity Ratio I=1.0†	Demand/Capacity Ratio I=1.5†
1	2nd - Ground	0.02	N/A
2	2nd - Ground	0	N/A
3	2nd - Ground	0.02	N/A
4	2nd - Ground	0.16	N/A
5	2nd - Ground	0.13	N/A
6	2nd - Ground	0.02	N/A
7	2nd - Ground	0.02	N/A
8	2nd - Ground	0.15	N/A
9	2nd - Ground	0.15	N/A
1	Roof - 2nd	0.09	N/A
2	Roof - 2nd	0	N/A
3	Roof - 2nd	0.09	N/A
4	Roof - 2nd	0.15	N/A
5	Roof - 2nd	0.13	N/A
6	Roof - 2nd	0.09	N/A
7	Roof - 2nd	0.09	N/A
8	Roof - 2nd	0.12	N/A
9	Roof - 2nd	0.12	N/A

† *Demand/Capacity Ratio greater than one represents an overstressed condition.*
‡ *Refer to Figure 1 for the location of the beams.*

Table 4: Updated Demand/Capacity Ratios for Site Class A for Steel Beams

1	2nd - Ground	0.01	N/A
2	2nd - Ground	0.00	N/A
3	2nd - Ground	0.01	N/A
4	2nd - Ground	0.11	N/A
5	2nd - Ground	0.09	N/A
6	2nd - Ground	0.01	N/A
7	2nd - Ground	0.01	N/A
8	2nd - Ground	0.11	N/A
9	2nd - Ground	0.11	N/A
1	Roof - 2nd	0.06	N/A
2	Roof - 2nd	0.00	N/A
3	Roof - 2nd	0.06	N/A
4	Roof - 2nd	0.11	N/A
5	Roof - 2nd	0.09	N/A
6	Roof - 2nd	0.06	N/A
7	Roof - 2nd	0.06	N/A
8	Roof - 2nd	0.09	N/A
9	Roof - 2nd	0.09	N/A
1	2nd - Ground	0.01	N/A

† *Demand/Capacity Ratio greater than one represents an overstressed condition.*
‡ *Refer to Figure 1 for the location of the beams.*

Table 5: Original Demand/Capacity Ratios for Diagonal Braces in Tension

Brace Bay Number‡	Level	Demand/Capacity Ratio I=1.0†	Demand/Capacity Ratio I=1.5†
1	2nd - Ground	1.1	1.7
2	2nd - Ground	1.1	1.7
3	2nd - Ground	0.6	1.0
4	2nd - Ground	1.3	2.0
5	2nd - Ground	1.4	2.1
6	2nd - Ground	1.1	1.6
7	2nd - Ground	1.1	1.6
8	2nd - Ground	1.1	1.6
9	2nd - Ground	0.7	1.0
1	Roof - 2nd	1.7	2.6
2	Roof - 2nd	1.7	2.6
3	Roof - 2nd	1.0	1.5
4	Roof - 2nd	2.0	3.0
5	Roof - 2nd	2.1	3.1
6	Roof - 2nd	1.6	2.4
7	Roof - 2nd	1.6	2.4
8	Roof - 2nd	1.7	2.5
9	Roof - 2nd	1.1	1.7

† *Demand/Capacity Ratio greater than one represents an overstressed condition.*
‡ *Refer to Figure 1 for the location of the braces.*

Table 6: Updated Demand/Capacity Ratios for Site Class A for Diagonal Braces in Tension

Brace Bay Number‡	Level	Demand/Capacity Ratio I=1.0†	Demand/Capacity Ratio I=1.5†
1	2nd - Ground	0.8	1.2
2	2nd - Ground	0.8	1.2
3	2nd - Ground	0.5	0.7
4	2nd - Ground	0.9	1.4
5	2nd - Ground	1.0	1.5
6	2nd - Ground	0.8	1.1
7	2nd - Ground	0.8	1.1
8	2nd - Ground	0.8	1.1
9	2nd - Ground	0.5	0.7
1	Roof - 2nd	1.2	1.9
2	Roof - 2nd	1.2	1.9
3	Roof - 2nd	0.7	1.1
4	Roof - 2nd	1.4	2.2
5	Roof - 2nd	1.5	2.2
6	Roof - 2nd	1.2	1.7
7	Roof - 2nd	1.2	1.7
8	Roof - 2nd	1.2	1.8
9	Roof - 2nd	0.8	1.2

† *Demand/Capacity Ratio greater than one represents an overstressed condition.*
‡ *Refer to Figure 1 for the location of the braces.*