Report Summary Sheet Date Reviewed: 26 September 2011

Group	Mission	Building	Type	Category	Document	Date of Report	Consultant
AMS-B	Bridgetown, Barbados	Chancery	Owned	1 Immediate Occupancy (IO)	Phase 1/2 Seismic Evaluation	October 2010	JL Richards

Standard Used	ASCE Structure Type	NRC Structure Type	SPI
ASCE 31-03 as modified by DFAIT and NBCC 2005	Steel Braced Frames with Stiff Diaphragms (S2)	Steel Braced Frame (SBF)	N/A

Purpose: Determine seismic performance of the structure and outline 3 seismic retrofit options to provide a Seismic Force Resisting System (SFRS) with sufficient capacity for the design seismic event.

Description of Building:

- Constructed in 1984
- 2 storeys above grade with one partial basement level
- Steel columns, steel joists, concrete floor slabs on metal decks and reinforced concrete (RC) footings
- Finished with mortar or plaster panels and glazing
- Interior finishes are varied and consist of glazing, concrete masonry and gypsum wall boards partitions
- Steel braced frames with Hollow Structural Section (HSS) columns, structural steel beams, structural steel diagonal braces, and deck diaphragms are metal deck of composite steel decking with RC diaphragms
- Foundations and Basement Level:
    - Foundations not visible, basement slab and foundation walls visible in crawl space at basement level
    - Basement walls consist of concrete masonry block, HSS columns rest on base plates secured with 2 anchor bolts to foundations
    - Concrete slab in crawl space in good condition
    - Efflorescent staining in concrete masonry walls
    - Corrosion of steel beam and open web steel joist (OWSJ) in basement supply room thought to be caused by moisture from air conditioning unit
- Ground Floor, Second Floor and Roof:
    - The structural steel columns, beams, bracing and steel deck of floor and roof were observed by selectively removing suspended acoustic ceiling panels

  - No corrosion/deterioration
- Exterior Façade:
  - Consists of glazing and concrete or masonry panels
  - No visible damage/deterioration

Type of Soil: Site Class C assumed

Lateral Force Resisting System: Braced frames made up of steel HSS and W-section beams with diagonal L-shaped angle braces.

Lateral forces applied to structure at centre of mass and are functions of displacement of structure (acceleration) and inertial weight of structure. Lateral forces follow load path from floor or roof diaphragms through SFRS to foundations

- *Diaphragms:*
  - Structural drawings make no reference to seismic consideration and lateral braces are specifically referred to as wind braces. In seismic design, storey diaphragms and diaphragm connections more significant to performance of building
  - No lateral support in N-S direction in upper clear storey roof requires additional lateral bracing members along E and W edges. (analysis based on assumptions that additional braces will be installed)
  - Welded roof diaphragm connections do not have acceptable pattern of spacing to joists for transfer of lateral loads (should say spacing MORE than 300mm to not be considered), large spacing between welds results in less stiff element with lower shear capacity
  - In-situ steel deck would not perform satisfactorily during design seismic event
  - Connection pattern for second floor diaphragm not acceptable by Canadian standards for transfer of lateral loads, and would not perform satisfactorily during design seismic event
  - Existing perimeter chord member does not have adequate capacity to transfer compression and tension forces from diaphragm to SFRS
  - Existing deck diaphragms do not have adequate capacity to transfer shear forces in diaphragm to SFRS, specifically around diaphragm openings, where diaphragm width is reduced or in roof where diaphragm connected with light steel beams. (Retrofit to improve shear resistance of steel deck diaphragms is required)

- *HSS Steel columns:* Assumed that seismic forces could be transferred to SFRS from steel diaphragms

- *Diagonal Braces:* Tensile capacity based solely on member strength since no connection detail provided. According to Slenderness checks, no bracing members within allowable range for carrying compression forces, so considered to be purely tension members.

Seismicity	PGA at 10%/50yr (m/s^2)	Estimated PGA at 2%/50yr (g)	Significant Earthquakes
High	N/A	0.377*	• Cayman Islands M6.8, 2004 • Martinique M7.4, 2007 • Haiti M7.0, 2010.

*Approximately 90% of value of Ottawa

Building Irregularities:

- None

PHASE 1-SEISMIC EVALUATION

Demand/Capacity Ratios:

	Demand/Capacity Ratios for HSS Columns (Cf/Cr or Tf/Tr) + Mf/Mr		Demand/Capacity Ratios for Steel Beams (V_f/V_r)	Demand/Capacity Ratios for Diagonal Braces in Tension	
	60%	1.06		60%	1.25
	100%	1.11		100%	2.02
Maximum	150%	1.75	0.16	150%	3.13
	60%	0.556		60%	0.651
	100%	0.662		100%	0.996
Average	150%	0.518	0.0861	150%	1.541
	60%	0.275		60%	0.307
Standard	100%	0.330		100%	0.425
Deviation	150%	0.518	0.0575	150%	0.688

- columns from ground floor to second floor have higher D/C ratios than floor to roof
- 7/16 columns are over 1.0 (100%)
- D/C ratios for beams in shear all less than 1 and beams have negligible bending. Likely to have enough overstrength and ductility to allow for inelastic deformations required by design seismic event. No details provided for beam-column connections so adequacy of connection cannot be commented on.

Non-compliant Checklist Items:

3.7.4 Basic Structural: 2 separate diaphragms, distance between storey centre of mass and storey centre of rigidity exceeds 20% of building for smaller one. Visible rusting of steel beam and joist in maintenance supply room. Number of braced bays in each direction is less than 3 as required for IO (redundancy). Axial stress in diagonal bracing greater than maximum allowable. Steel column anchorage not able to develop uplift capacity of foundation. Details of connections from diaphragms to steel frames unknown.

3.7.4S Supplemental Structural: Slenderness ratios of diagonal elements greater then maximum value. Connection details unknown.

3.8 Geological Site Hazards and Foundations: Liquefaction and surface fault potential unknown. Ratio of horizontal dimension of LFRS at foundation level to building height less than minimum values, braced bays vulnerable to overturning.

3.9.1 Basic Non-structural Component: URM units not braced. Details of cladding anchors, cladding isolation connections, and inserts unknown. Mechanical equipment over 20 lbs. and fire suppression piping not braced. No flexible couplings noted on fire suppression system.

3.9.2 Intermediate Non-structural Component: Exterior window glazing has no safety film

3.9.3 Supplemental Non-structural Component: Tops of partitions that extend to ceiling line not laterally braced to the building structure. Edges of integrated ceilings not separated from enclosed walls by the minimum ½ inch required. Exterior glazing not laminated or heat-strengthened safety glass. No latches on some cabinet doors and drawers. Electrical equipment and associated wiring not laterally braced. Fluid and gas piping not braced. Details of required shut off valves not known.

PHASE 2—RETROFIT OPTIONS STUDY

Seismic Retrofit Options

I. Replacing Bracing Members
II. Additional Braced Bays

Initial Load Path Upgrades

Each retrofit option requires 5 basic upgrades of key elements of the load path to transfer seismic loads to SFRS and foundations. CH could be further improved by installation of friction dampers within braced bays. They will dissipate seismic energy by increasing damping of structure, and lowering force function.

1. Improved capacity of roof /Roof Diaphragm Upgrades
   a. Roof diaphragm has inadequate capacity to transfer shear forces to SFRS
   b. Options to increase capacity include: horizontal braces with additional steel diaphragms or replacement of existing diaphragm with higher gauge steel deck with a connection pattern suited to resist applied loads

2. Improved capacity of 2nd floor diaphragm/2nd Floor Diaphragm Upgrades
   a. Depending on design level and retrofit strategy, may require upgrading to increase shear capacity
   b. Same options as roof, welding steel deck to support members to create acceptable connection pattern to develop strength of diaphragm

3. Improved connectivity of deck diaphragms to SFRS/Diaphragm to SFRS Connection Upgrades
   a. No clearly defined load path from diaphragm to SFRS, indicating possibility of storey diaphragms shearing from their supports during a seismic event
   b. Replacement of existing perimeter angle with larger steel section to both transfer loads to SFRS and resist tension and compression forces generated by flexure of deck diaphragm

4. Horizontal Roof Braces
   a. Roof level requires horizontal bracing at some locations to provide connection between small portion of roof diaphragm which is separate from main diaphragm and SFRS in N-S direction
   b. Important as existing building has no defined means of laterally restraining this portion of roof in N-S direction

5. Foundation upgrades

a. Upgrading braced bay foundations
b. Installation of rock anchors to resist significant uplift on foundation from SFRS, considered necessary in all cases
c. Retrofit of column-foundation connection to withstand high tension forces present in uplift condition, would involve larger steel baseplate and concrete pier to accommodate more anchor bolts to transfer forces to rock anchors

I. Replacing Bracing Members

- Replace diagonal bracing members within braced bays with larger members to increase capacity of SFRS
- Assumption that diaphragms on second storey and roof will be upgraded to increase their shear capacities and foundation will be upgraded to resist uplift forces
- In design of braced frame buildings, it is necessary that diagonal members be the point in SFRS where yielding occurs. As strength of SFRS increases, required strength of other components must be increased as well, including member connection, shear transfer between braced frames and diaphragms, diaphragms and foundation capacity
- Seismic forces only distributed to 4-5 braced bays in N and S direction, seismic forces more concentrated which result in higher loads in deck diaphragms adjacent to braced bays and foundation supporting braced bays
- Interior finishes in area of each braced bay would be removed allowing access for existing bracing members
- End result; braced bay with diagonal bracing members of higher tensile resistance capacity

II. Additional Braced Bays

- Adding diagonal braces to currently unbraced steel frames in CH to increase capacity of SFRS
- Placement would be along perimeter of building like existing braced bays, placed so as not to interfere with architectural details, such as large windows at front entrance, placed in an arrangement such that centre of rigidity is not shifted away from center of mass. Shift would cause amplification of shear forces in building due to increased torsion
- Increasing number of braced bays in a given loading direction is an effective means of distributing seismic forces applied to a building over a greater number of SFR elements which results in smaller loads in each element and contribute to redundancy of structure (non-compliant checklist item)
- Magnitude of forces in braced bays would be distributed over more bays, therefore associated forces in diaphragms would be reduced, and reduction in level of work needed to reinforce diaphragms and foundations

- Interior finishes in area of each braced bay to be modified or removed allowing access to existing steel members, each additional bay would require installation of plates for connection details similar to those on site

Retrofit Option	Load Case	Cost Estimate (CAD)**	Estimated Duration	Level of Intrusiveness
Replace Existing Bracing Members	60%	$0.710M	8-16 months	High
	100%	$0.850M	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

**costs based on average North American (NA) labour and material

<u>Non-structural Component Upgrades</u>

Non-structural Component	Hazard	Solution	Estimated cost (CAD)
Masonry walls	During seismic event, possibility of portions of URM walls and partition walls of CH toppling into adjacent area within building	Installation of steel angle member to brace tops of these walls	$10,000
Windows	No indication that windows contain safety glass or treated with safety film Purpose of safety glass/film is to prevent pane of glass from dislodging from frame and/or shattering during a seismic or other event	Application of safety film or replacing existing windows with safety windows	$25,000 for safety film or $75, 000 for replacing windows
Gas Supply Lines	Damage to natural gas supply lines has high potential of causing a building fire	Installation of braces to gas lines and fire suppression system	$200 per brace
Fire Suppression Supply		Flexible couplings to allow for lateral movement of system without	$150 CAD per coupling

		compromising pipes	
Large furniture and Mechanical/Electrical equipment	Standing cabinets/bookshelves can pose safety hazard due to potential of toppling onto occupants or blocking means of exit	Anchoring furnishings to adjacent walls Bracing of suspended light fixtures/equipment also to be considered	N/A

Report Recommendations:

- Load path upgrades
    - Addition of horizontal braces or higher capacity steel diaphragm to roof or replacement of existing roof diaphragm with a higher gauge steel deck with a connection pattern to suit the applied loads.
    - Upgrading capacity of the second floor diaphragm using the same methods as for the roof diaphragm. Would also include welding the steel deck to the support members to create an acceptable connection pattern to develop the strength of the diaphragm.
    - Replacement of the existing perimeter angle along the perimeter of the diaphragms to adequately transfer loads to the SFRS.
    - Placement of horizontal braces at the roof level in some locations, braces are required to provide a connection between small portion of the roof diaphragm which is separate from the main diaphragm and the SFRS in the north-south direction.
    - Provision of rock anchors and other foundation upgrades to provide sufficient uplift capacity to braced frames to resist design seismic event.
- Strength of structural steel comprising SFRS be confirmed through intrusive testing (material testing)
- Addition of new braced bays and replacement of existing bracing members with friction dampers be pursued with 150% capacity threshold be met
- Incorporation of friction dampers, to reduce impact of retrofit in terms of construction schedule, cost and impact to occupants
- Retrofit non-structural components

Reviewer's Notes:

- Strength of steel deck diaphragms found to be inadequate to transfer loads to the SFRS, diagonal bracing members found to have inadequate capacity and foundations found to have inadequate uplift capacity

- Effective masses of 2 storeys checked and found to not change more than 50% as specified in ASCE
- Estimated distance between storey centre of mass and storey centre of rigidity greater than 20%, due to smaller roof diaphragm having centre of rigidity on outer edge of building
- Axial stress in columns subjected to overturning forces found to be less than capacity limit
- Axial stresses in diagonal bracing members greater than capacity limit
- Connection of steel columns in LFR brace frames unable to develop tensile capacity and uplift capacity of foundation, uplift/tension forces in all columns exceed capacity of anchorage
- Width/thickness ratios of all frame elements found to be within allowable range
- No bracing members were within allowable rage for carrying compression forces and bracing was assumed to be tension only
- Base-height ratio of LFRS in allowable range
- Foundation elements restrained by strip footing, therefore adequate
- Mass participation for first mode in both Y-direction and mass participation for second mode in X-direction meet or exceed minimum recommended by commentary J of NBCC
- Lateral loads applied at 10% eccentricity from centre of mass for equivalent static and dynamic analysis to account for torsion
- Effective masses of 2 storeys found not to change more than 50% as specified in ASCE
- Axial stress less than allowable limit
- Number of braced bays in each direction is inadequate for IO
- Demand Capacity Ratio for foundation over 17, current foundations inadequate to resist uplift caused by design seismic event
- Assumptions made for analysis:
    - Member sizes and reinforcement details as shown on structural drawings
    - Material properties as shown on structural drawings
    - Site class C, no consideration to slope failure or liquefaction of underlying subgrade
    - Not considering local/near fault affects

Reviewer's Comments and Observations:

- Additional braced bays is the cheaper option, for 150%, and would provide more material to resist lateral forces
- The report recommendation is to pursue a combination of adding new braces and replacing existing ones and adding friction dampers to existing braces, but cost not estimated for a combination

- Phase 1 results not verified with final results of geotechnical investigation for Phase 2 as geotechnical investigation was performed afterwards
    - From geotechnical report, Site class A would be appropriate for this structure and this will significantly reduce seismic forces

Report Reviewed by: Liza Rozina, Civil Engineering Student

Structural Engineer's Comments/Recommendations:

Notes:

- BDGTN is located in an area of a medium seismic risk, about 90% that of Ottawa.
- BDGTN was identified as undergoing a pending major mid-life refit.
- This building seems to be at some limited risk.
- Cost of upgrades is $420K - $1.30M.
- Maximum D/C is 2.1.
- Recently raised concerns over the presence of Karsts topography have been mitigated by extensive geotechnical investigations.
- The geotechnical report updated the Site Class to Class A.

Recommendations:

- In light of the overall seismic risk of this building, the following is recommended:
    - Maintain occupancy of current CH as is until the mid-life refit project is scheduled with the caveats mentioned below.
    - Update the report with a Site Class A analysis and include friction damper option as a high-priority.
    - Reassess these recommendations after the revised report is received.

X

Damian deKrom
Structural Engineer

1 May 2012