



## SEISMIC ISOLATION OF 200 YEAR OLD HERITAGE CATHEDRALS IN HAITI

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### ABSTRACT

Historic and heritage building present a unique class of structures. These buildings form a cultural pillar of the communities and thus require preservation. However, given their vintage, they have sustained severe damage and collapse in recent earthquakes, including in Italy (2009), Haiti (2010) and New Zealand (2011). The main vertical and lateral load bearing members for these buildings is typically comprised of unreinforced stone masonry (URSM) walls. These walls have experienced both in-plane and out-of-plane failures, leading to the collapse of the structures. Given that the walls have little lateral capacity, it is critical to limit the input forces acting on them. In addition, these structures do not have a well-defined load path or diaphragm for seismic loading. A proposed mitigation strategy combining seismic isolation and superstructure intervention is discussed to address these deficiencies. Advanced nonlinear global and local finite element analysis is used to assess the efficiency of the proposed retrofit. The proposed method significantly reduces the level of seismic excitation acting on the existing walls and limits the superstructure retrofit, and thus preserves the historical features of the structures. Application of this technique to two Cathedrals in Haiti is presented.

### INTRODUCTION

#### Building description

Saint John Baptist Cathedral of Miragoane (hereafter referred to as the Cathedral) was originally constructed in 1880 and is one of the oldest Cathedrals in Miragoane— a coastal town approximately 80 km west of Port-au-Prince, the capital of Haiti. Figure 1 shows a photograph of the building looking east. The building has an area of approximately 580 m<sup>2</sup> and is nearly rectangular shape. The building is constructed using concrete floors with an unreinforced masonry and stone walls over stone masonry foundations. There is a ground floor, and a mezzanine with access to the upper tower that houses the bell. The roof structure is assembled with trusses that combine both wood and steel and is approximately 13.9 m tall at its peak. The roof is supported by the walls on the exterior and by uniformly placed columns along the interior. The front entrance of the cathedral has a bell tower that stands approximately 30.5 m high. The tower is constructed with steel frames above the walls. There is a concrete mezzanine that sits about 7m above the finished floor of the cathedral. The walls along the perimeter vary from 500 mm to 750 mm in thickness and are the primary gravity and lateral load resisting members. Figure 2 presents elevation and plan views of the building.

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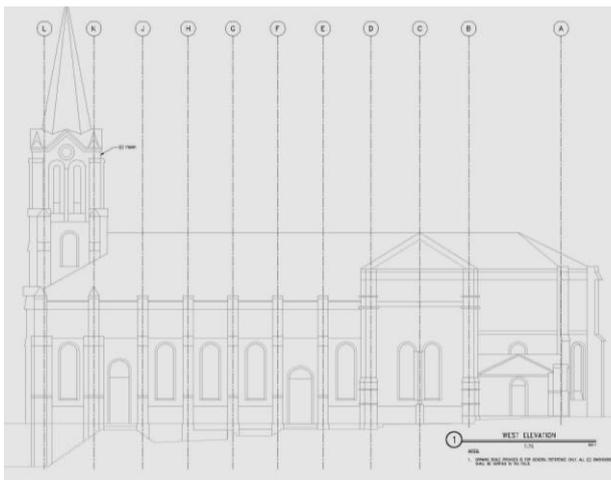
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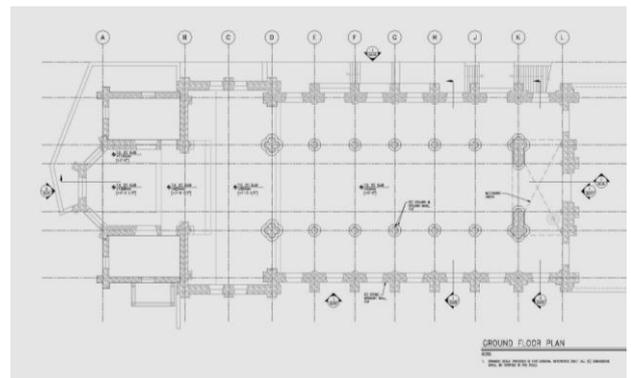
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Figure 1. Photograph of the Cathedral, looking east



Front elevation



Plan view

Figure 2. Architectural drawings of the Cathedral

The Cathedral suffered minor damage, primarily minor cracking during the 2010 Haiti Earthquake. The damage to the building was minor because it was not located near the epicenter of the 2010 earthquake

## SEISMIC RETROFIT METHODOLOGY

### Overview

ASCE/SEI 41-06 (2006) [1] served as the principal document used for retrofit evaluation. To achieve the design objectives of the standard, it is proposed to seismically isolate the building. This retrofit option was selected because it provides reliable seismic performance, while preserving the historical features of this cultural heritage building and minimizing retrofit of the superstructure. For historical or essential facilities, base isolation provides an attractive retrofit (De Luca et al., 2000) [3]. Using the seismic isolation retrofit option, minimizes or eliminates alterations of the superstructure. Instead, the

structure is de-coupled at the foundation level, since isolators are installed beneath the existing columns or walls. In the past two decades, many buildings in the United States, New Zealand, Japan, and Europe have used this technique. Base isolation relies on the concepts of structural dynamics to modify the response of the building and reduce the seismic demands on the structural and nonstructural members. For isolated structures, the structural period is shifted away from the high-energy portion of the typical ground motions because the isolation plane is considerably softer than the superstructure. The isolation system also introduces effective supplementary damping to the structures since the force-deformation relation is nonlinear. There are two basic isolation systems: elastomeric rubber (either high-damping rubber or lead-core), and metallic sliding surfaces (flat or pendulum sliders).

### **Design objectives and performance goals**

The design objective for seismic strengthening of Cathedral was to provide global and local performances that exceeded the requirements of ASCE/SEI 41-06 (2006) [1]. The enhanced global performance targets at design earthquake (DE) and maximum considered earthquake (MCE) are:

- a) DE (475 year): Performance of between immediate occupancy (IO) and life safety (LS), and
- b) MCE (2475 year): Performance of between LS and collapse prevention (CP) .

The current common seismic retrofit practice targets of ASCE/SEI 41-06 (2006) [1] are to obtain LS and CP for DE and MCE, respectively. Locally, accelerations and drift ratios were reduced to level below the values initiating the in-plane and out-of-plane failure of vulnerable URSM walls. In addition, the displacement of the isolation system was monitored to ensure that it does not exceed the capacity of the system.

### **Seismic hazard**

The design spectrum for the extreme event (or maximum considered earthquake, MCE) is defined by the International Building Code (2006) [6] as the probabilistic event with a return period of 2475 years (or 2% probability of exceedance in 50-years). The seismic hazard coefficients for the site were obtained from the USGS (2012) [13] are: short period spectral acceleration ( $S_S$ ) of 1.62g and a 1-sec spectral acceleration ( $S_I$ ) of 0.6g. These values are consisted with recently published reports [11] . The geotechnical report [7] wrote that the Cathedral was built on limestone rock with an allowable bearing pressure of 1 MPa. The site condition was classified as soil class C using the data from the 2012 log of boring data. Thus the design response spectrum for the Cathedral can be constructed using short period ( $S_{DS}$ ) and 1 sec ( $S_{DI}$ ) spectral accelerations of 1.08g and 0.52g, respectively. The MCE spectrum ordinates were 1.5 times the design spectrum. The analysis of the Cathedral was based on the nonlinear response history analysis. Records from three stations recorded from large earthquakes (Landers, Northridge, and Loma Prieta) in the United States deemed to be representative for the site, were selected. Next, tight spectrum matching of the records was used such that their response spectra closely matched the target spectrum for the site. For analysis, the model was subjected to the pairs of records at design and MCE levels. The records were applied in 0 and 90-degree orientations. No vertical component of acceleration was applied. The responses were then selected as the envelope of the maxima of responses obtained from analysis.

## **SEISMIC RETROFIT**

### **Seismic isolation system**

For the Cathedral seismic retrofit, the state-of-the-art triple pendulum (TP) [4] isolation system was selected. The TP pendulum system is an adaptive self-centring system that provides isolation at functional, design, and maximum earthquakes (approximately 100, 500, and 2500 year return periods) by successive sliding of concave inner and outer surfaces. In addition, for very large earthquakes, the system designs restraints overly large deformation. Finally, due to its adaptive sliding surfaces, the footprint of the TP bearing is one-half of single-stage concave sliders. For the Cathedral, the isolation system parameters were chosen to obtain an approximate effective period of 4 sec. and equivalent supplementary damping of 30% at MCE. Given, the large shift in period, and additional damping, it is expected that only minor retrofit of the URSM walls would be required. The isolation plane is selected

to occur just below the ground level of the building. The isolators will consist of a combination of 54 TP bearing. The geometric arrangement of the isolators has been selected to preserve the current load path in the URSM walls to avoid introducing additional concentrated loads to these vulnerable components. To install the isolators, the existing walls will be reinforced either side by permanent shoring beams, above and below the isolation plane. Next, a wall section will be removed and isolators installed. Finally, the remaining wall is cut in order to complete the isolation plane.

**Structural load path intervention**

The building in its existing configuration lacks a well-defined load path for seismic forces. For example, the existing floors are not designed or detailed to serve as diaphragms and they do not have adequate connection to the perimeter walls to transfer lateral forces to these vertical members. In the absence of such load path, the vulnerable unreinforced walls will act as cantilevers, (unsupported at the top) and are susceptible to out-of-plane failure. In the past earthquakes including the 2011 Christchurch Earthquake [5] , this was a common observed failure mode of many older and historic unreinforced masonry buildings.

For the seismic isolation system to be effective, this type of failure need to be precluded, as such, it is important to connect structural elements and provide a robust path for the transfer of seismic forces. In the United States, this type of failure is mitigated and the seismic load path is developed by addition of either wood or concrete diaphragms to the existing buildings. Since such approach was not feasible in Haiti, the strengthening was provided by a series of steel rods and beams (channels and angles) serve to connect the wall elements and provide horizontal bracing (diaphragm) and vertical bracing. The horizontal bracing prevent out of plane mechanisms and connect the internal columns to the external walls. Such approach has been used extensively in Europe and especially in Italy and Greece [10] for retrofit of historic buildings. Figure 3 presents the plan and elevation view of the Cathedral showing the added steel members. As shown in the figures steel members are added: a) Horizontal tie-down steel rods are added to the building to connect the perimeter walls in both directions, b) System of steel truss works is added to provide diaphragm action in plane and vertical bracing, c) Bracing is added to reinforce the tower and to connect this segment to the rest of the Cathedral, d) Vertical tie-down rods are added to reinforce the walls at tower base and to improve its flexural capacity, and e) Steel longitudinal reinforcement is added to the tower walls

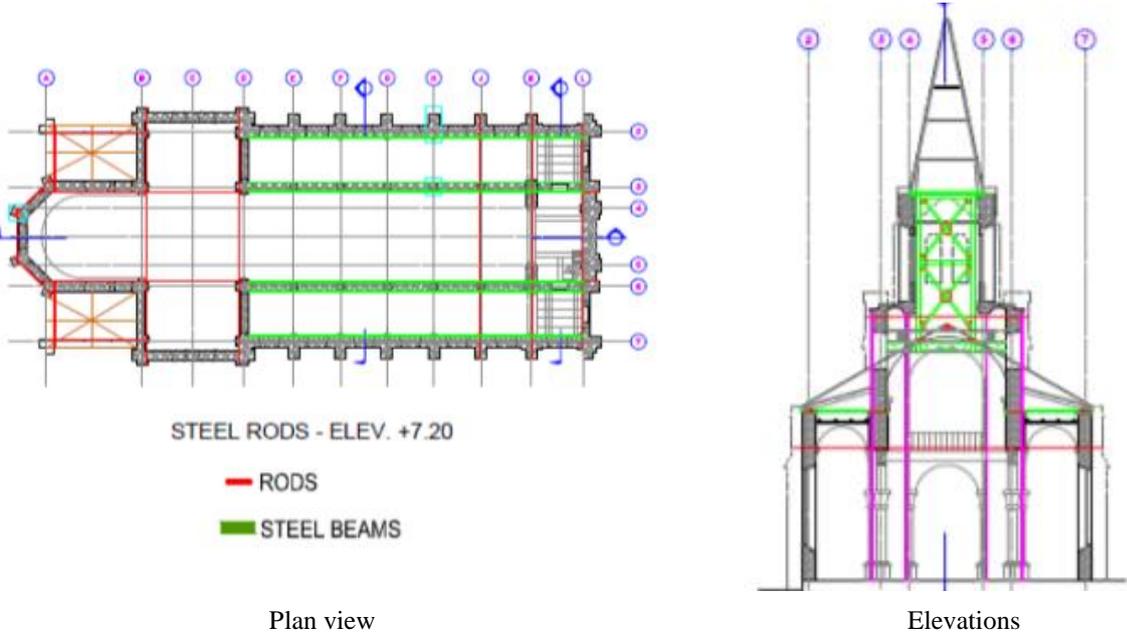


Figure 3. Typical structural intervention details

## STRUCTURAL CAPACITY OF THE WALLS

### Material properties of URSM walls

The Cathedral's unreinforced stone masonry (URSM) walls are the load bearing elements resisting the applied vertical and lateral load applied to the building. The composition of the wall is that of unreinforced masonry with irregular-shaped stones or with rectangular-shaped stones and debris placed in the mortar.

The nominal strength of the URSM walls was based on the provisions of the Italian seismic code for unreinforced walls [9]. The code provides average tabulated values for different types of masonry. The tabulated average values were developed based on the material data available from the large pool of historical buildings in Italy. The URSM walls have the lowest mechanical properties, whose values are listed in Table 1. For evaluation, the lowerbound values reduced by a safety factor, listed in the table were used to determine the capacity of the walls

Table 1. Average nominal properties for URSM walls

Property	$f_m$ MPa	$\tau_o$ kPa	$E$ GPa	$G$ GPa	$w$ (kN/m <sup>3</sup> )
Lowerbound	1.0	20	0.69	0.23	19
Upperbound	1.8	32	1.05	0.35	

Where:

$f_m$  = average compression strength,  $\tau_o$  = average shear strength,  $E$  = average (uncracked) elastic modulus,  $G$  = average (uncracked) shear modulus, and  $w$  = average unit weight

### Strength of URSM walls

The addition of the tie rods and steel members increase the capacity of the existing walls. This effect was accounted for in calculation of the in-plane and out-of-plane strength of these walls. Linear (code-based) and nonlinear (static pushover) analyses methods can be used to determine the in-plane strength of the walls. The equilibrium kinematic approach (see for example [14]) was used to determine the out-of-plane capacity of the walls.

### Out-of-plane capacity of walls

The wall failure in the original configuration will be comprised of the rigid motion (rocking) of the wall about its base; see Figure 4. This kinematic condition is possible since the top of the wall is not attached to a diaphragm. The lateral load (acceleration) required to initiate failure is resisted by the vertical load acting on the wall. Once the steel members are added, the diaphragm action excludes this mode of failure. Instead, the failure mechanism will include formation of a hinge along the height of the wall (see Figure 4). A larger lateral force (acceleration) will be required to initiate this higher mode failure. In addition, the tie-down rods provide additional resistance to overturning and thus serve to increase the lateral load required to initiate out-of-plane failure of walls [8]. The key parameter for development of the out-of-plane strength is the lateral acceleration at the base of the wall and perpendicular to its plane that initiates failure. This is because, the seismic load is considered as static force (wall mass times spectral acceleration at the base of the wall), and acceleration is assumed to be constant along the height of the wall ( $a_o$ ). The computed capacities are further adjusted by two factors: a) Knowledge factor  $\kappa$  to account for uncertainties in material properties, construction details, and geometric characteristics, and b) Behavior factor  $q$  to account that the limited ductility of the walls and constraint by adjacent elements to provide restraint to the out-of-plane rotation of the wall segment under consideration. At the time of analysis, no field test data was available, a  $\kappa$  factor of 0.75 (1/1.35) was used as prescribed in the Italian seismic code [9]. Similarly, the Italian code recommends using a value of 2.0 for  $q$  when a simplified linear procedure is utilized.

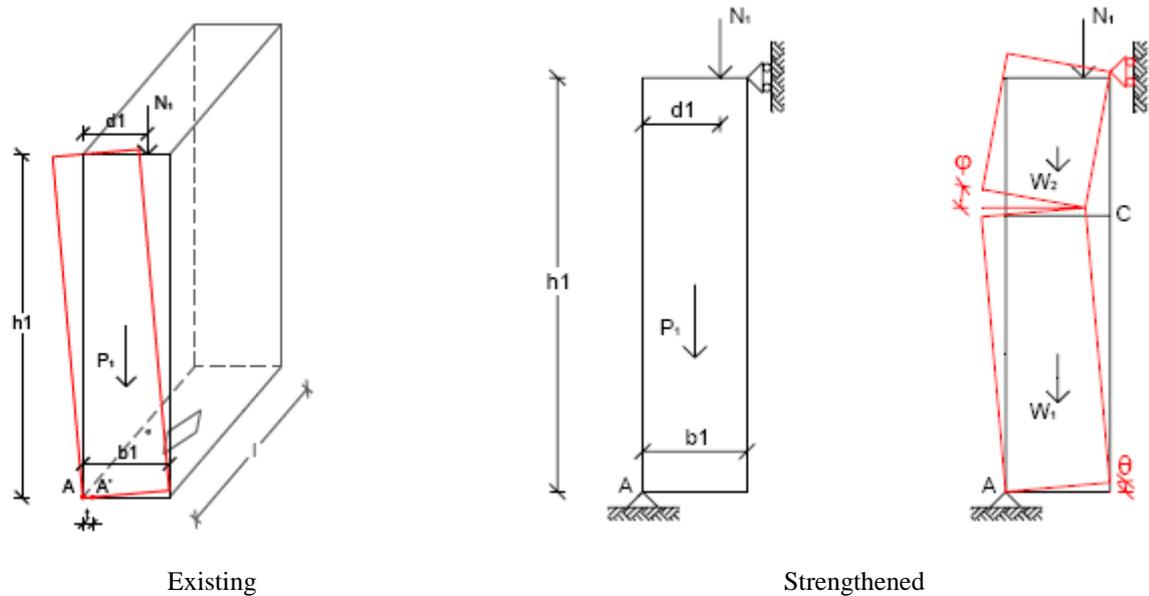


Figure 4. Out-of-plane failure modes for a typical wall segment

Equilibrium kinematic analyses of various walls of the Cathedral were conducted. The analyses accounted for the dimensions (height and thickness) of the wall, the vertical force acting on it, and the vertical tie-down restraining forces. Table 2 summarizes the findings. The highlighted values are the modified strength values.

Table 2. Computed out-of-plane capacity of Cathedral walls

Wall segment	Wall (Elev.)	Failure direction	Capacity, g	
			Computed ( $a_0$ )	Modified ( $q\kappa a_0$ )
Typ. between windows	0.6 m	Outward/inward	0.62/0.27	0.92/0.39
Transept end wall		Outward/inward	0.21/0.17	0.31/0.25
Central walls		--	0.30	0.44
Apse	6.6 m	Outward/inward	1.68/0.55	2.49/0.82
Upper masonry		--	1.66	2.46
Upper transept end wall	9.9 m	Outward/inward	0.98 /0.49	1.45/0.73
Bell tower		--	0.35	0.52

Thus as shown in the table, the critical lateral accelerations are 0.25g at the ground level and 0.52g at the roof. As long as the isolated buildings acceleration demands at these two levels are less than the governing values, out-of-plane failure will not occur.

### In-plane capacity of main cathedral building and bell tower

The capacity of the walls was determined using static pushover analysis using plastic hinges whose properties were obtained from interaction analysis and program 3Muri [12]. The observation of damages on existing structures has led to the definition of masonry as a macro-element that captures the shear behavior in its central part and the buckling behavior in the outlying areas. The kinematic model used is described by eight degrees of freedom: the six components of displacement of the end nodes and the two components of the macro-element. The overturning mechanism of the panel, caused by the absence of a significant tensile strength of the material, is represented assuming elastic contact in interfaces, while the mechanism of shear failure is schematized considering a state of uniform tension in the central module. Maximum deformations (drift) acceptable for the panels are settled to define the collapse mechanism, due to the mechanisms of shear and bending.

Shown in Figure 5 is the state of the main cathedral and bell tower at the limit state. In this figure:: a) Green denote wall segments that remain elastic, b) Pink corresponds to flexural yielding, c) Red

designates flexural failure, d) Ivory indicates shear yielding , and e) Light blue represents traction failure Note that no shear failure was developed and limit state is reached when walls reach their ductile flexural capacity. The progression of the nonlinear response in the bell tower is listed in Table 3.

Table 3. Progression of nonlinear response for ground acceleration in g

State	Flexural yielding	Shear yielding	Flexural failure	limit state
Main cathedral	0.04	0.05	0.09	0.12
Bell tower	0.04	0.05	0.11	0.11

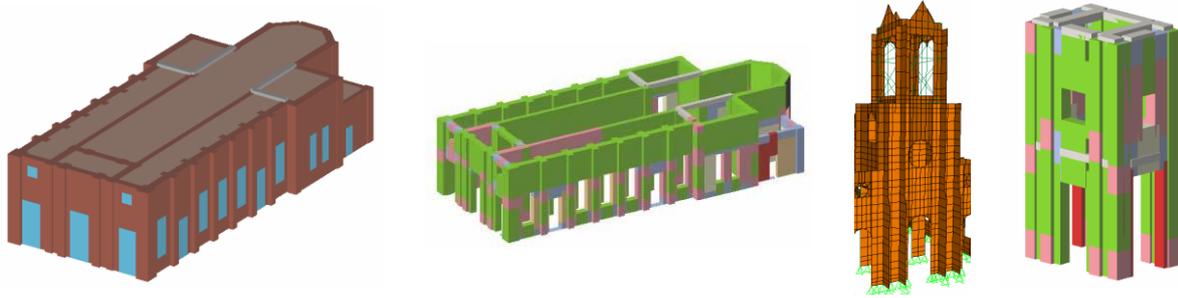


Figure 5. Mathematical model, and failure mode for main cathedral and bell tower

## ANALYTICAL MODEL

### Model properties

A three-dimensional analytical model of the building was prepared using the program ETABS [2] see Figure 6. The inertial mass of the structure is estimated at 2,800 Mg. The individual isolators were model as bilinear link elements using the friction and curvature properties provided by the TP bearing manufacturer. All calculations were performed using nominal values (no upper- and lower-bound analyses were conducted.)

### Analysis results

#### Drift requirements

Table 4 presents the computed story drift ratios above the isolation plane refer to Figure 2 for the location and elevations (m) of the stories listed in the table. The maximum story drift ratios are 0.42% and 0.49% at the DE and MCE levels, respectively.

Table 4. Computed story drift ratios

Story	DE		MCE	
	X-	Y-	X-	Y-
RF7	0.11%	0.05%	0.12%	0.06%
RF6	0.10%	0.06%	0.11%	0.07%
RF5-1	0.09%	0.10%	0.10%	0.10%
RF5	0.30%	0.20%	0.34%	0.25%
RF4	0.14%	0.20%	0.17%	0.27%
RF3	0.16%	0.24%	0.19%	0.31%
MEZZANINE2	0.24%	0.26%	0.29%	0.30%
LR1	0.20%	0.42%	0.25%	0.49%

For unreinforced masonry non-infill walls, ASCE 41 [1] has the following limitations on drift ratios: IO: Transient or permanent value Of 0.3%; LS: Transient or permanent value Of 0.6%; CP: Transient

or permanent value of 1.0%. Therefore, for the retrofitted structure, at both DE and MCE levels, performance of between IO and LS are obtained; see Figure 7 and thus the enhanced performance criteria are satisfied for drift response.

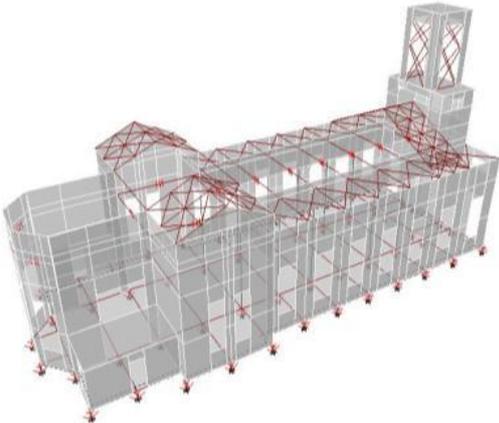


Figure 6. Analytical model of the building

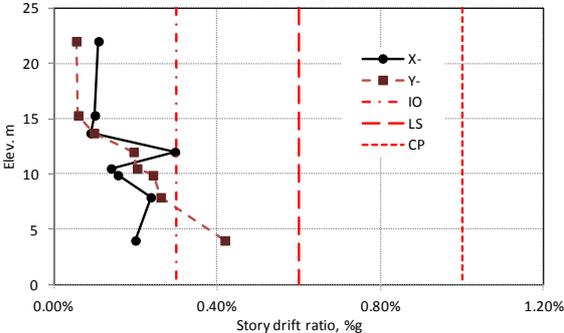


Figure 7. Story drift ratios (DE) and performance limits

**Out-of-plane accelerations**

The out-of-plane accelerations at the ground slab and roof are listed in Table 5. The computed demands at both DE and MCE are less than the capacity; no out-of-plane failure is anticipated even at the MCE.

Table 5. Out-of-plane accelerations at wall base

Level	Capacity, g	Demand, g	
		DE	MCE
Ground slab (0.6 m)	0.25	0.08	0.11
Roof 3 (9.9 m)	0.52	0.10	0.13

**Story shear**

Figure 8 presents the distribution of shear force (normalized with respect to the building seismic mass) along the height of the structure. The effective base shear for this building is approximately 12%g at the DE intensity. The addition of the isolation system has served to significantly reduce the demand on the structure. This can be further seen in Figure 9 showing that maximum total acceleration of 0.5g is reduced to 0.18g as it travels (and thus filtered) through the isolation plane.

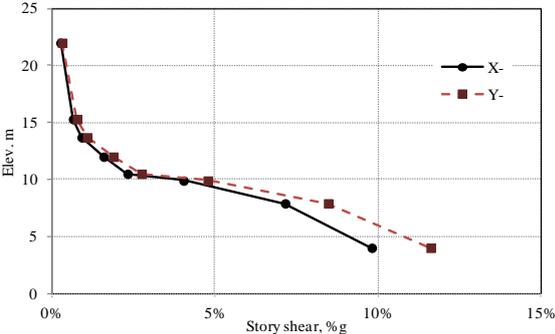


Figure 8. Distribution of story shear along building height

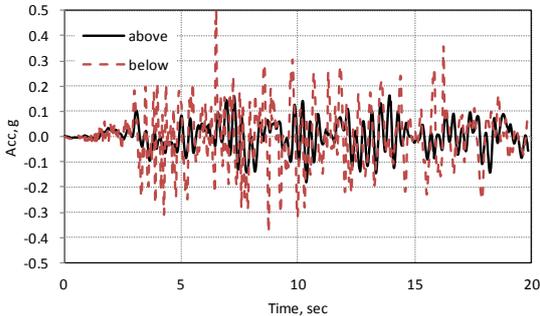


Figure 9. Computed lateral accelerations above and below isolation plane, DE, X direction

**Response of the isolation system**

Figure 10 presents the force-deformation response of a typical isolator from MCE analysis. Also shown in the figure is the nominal bi-linear backbone curve of the isolator obtained from the

manufacturer data. Figure 11 presents the bi-direction MCE displacement response of a typical isolator. Also shown in the figure is the displacement limit (500 mm) as specified by the manufacturer. As seen, the isolator MCE displacements in any direction are less than its allowed maximum motion.

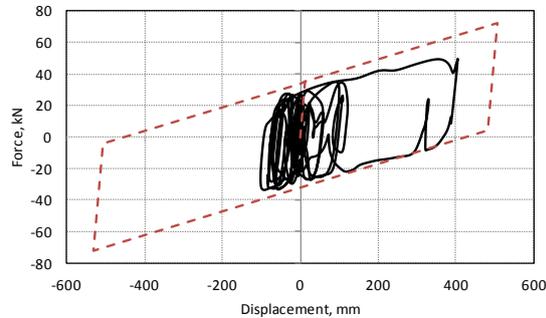


Figure 10. Typical isolator response

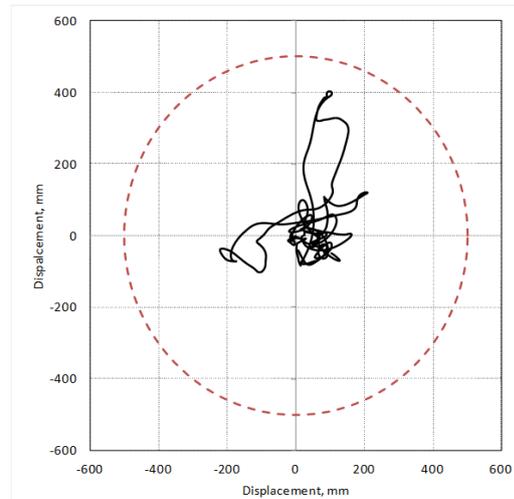


Figure 11. Bi-directional response of an isolator

## CONCLUSIONS

The Miragoane Cathedral is constructed of non-ductile URSM walls and does not meet the current code requirements for seismic performance. Analysis showed that the heritage buildings constructed with non-ductile URSM walls and retrofitted with a system of seismic isolation to reduce earthquake demand and steel members to provide load path continuity and out-of-plane restraint produced acceptable performance.

- Steel tie-downs significantly increase the out-of-plane capacity of the walls. Truss assemblage of steel members provided a reliable load path for seismic forces. Added reinforcing steel increased the flexural capacity of the tower bell walls.
- The isolation retrofit will significantly reduce the demand (drift and acceleration) on the URSM walls and the unreduced demand on the walls was reduced below member capacities

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