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COST-EFFECTIVE SEISMIC ISOLATION RETROFIT OF HERITAGE CATHEDRALS IN HAITI

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Abstract

Historic and heritage 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 are typically comprised of unreinforced masonry stone/rubblewalls. 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 Miragoane Cathedrals in Haiti is presented construction.

Keywords: Heritage Buildings, Seimic isolation, seimic retrofit, haiti Earthquake, Structural interventions

1 Introduction

1.1 Descriptions of buildings

Saint John Baptist Cathedral of 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. Saint Jacques et Saint Philippe Cathedral of Jacmel; see Figure 2 was originally constructed in 1859 and is one of the oldest Cathedrals in Jacmel— a coastal town approximately 80 km southwest of Port-au-Prince, the capital of Haiti.

The buildings were constructed using concrete floors with an unreinforced masonry and stonewalls over stone masonry foundations. The roof structures were assembled with trusses that combine both wood and steel. The roofs are supported by the walls on the exterior and by uniformly placed columns along the interior. The front entrances of the cathedral have bell towers high. The tower is constructed with steel frames above the walls.

Both Cathedrals suffered minor damage, primarily minor cracking during the 2010 Haiti Earthquake. The damage to the building was minor because they were not located near the epicenter of the 2010 earthquake

Fig. 1 Miragoane Cathedral

Fig. 2 Jacmel Cathedral

1.2 Architectural features

Fig. 3 and Fig. 4 present the interior of the cathedrals. As seen in these figure, these structures comprise a large number of architectural and heritage features. The retrofit solution for the buildings was selected to preserve the architectural features of the buildings. In addition. It was critical to maintain low accelerations for the buildings to reduce potential damage to such features. Seismic isolation retrofit was selected to achieve both of these goals.

Fig. 3 Miragoane Cathedral

Fig. 4 Jacmel Cathedral

2 Seismic evaluation and retrofit

2.1 Overview

ASCE/SEI 41 [1] served as the principal document used for retrofit evaluation. To achieve the design objectives and parameters, it was 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 option [2]. Using this option, alterations of the superstructure is significantly reduced or eliminated.

2.2 Performance objectives

For the Cathedral, the ASCE 41 dual basic safety objectives were used. The seismic hazard coefficients for the site were obtained from the USGS [3] are listed in Table 1.

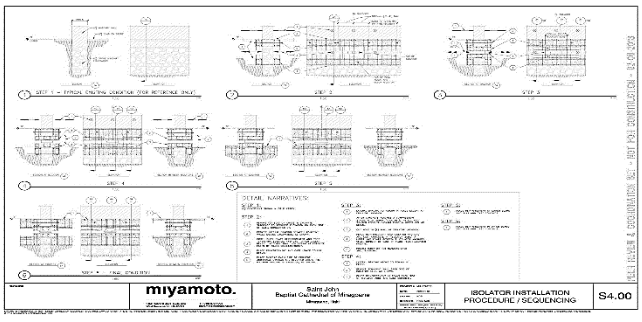
Table 1 - Site seismicity for Cathedrals

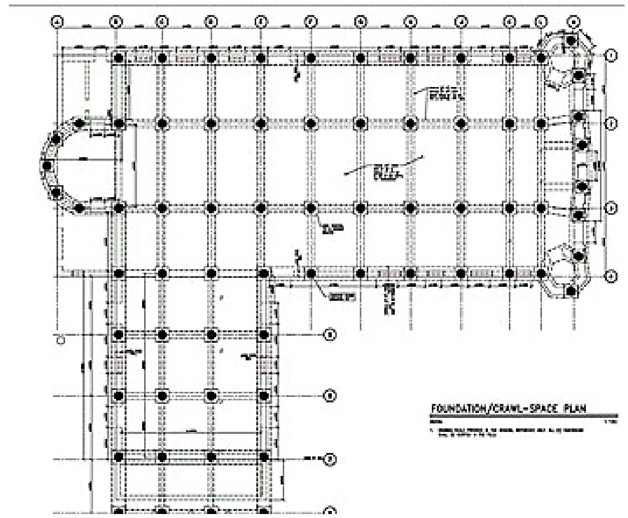
Cathedral	**Ss, g**	**S1, g**
Miragoane	1.62	0.6
Jacmel	0.82	0.29

The geotechnical reports indicate that the cathedrals were built on limestone rock with an allowable bearing pressure of 1 MPa. The site condition was classified as soil class C and D for the two structures, respectively, using the data from the 2012 log of boring data.

2.3 Seismic isolation system

The isolation system parameters were chosen to obtain an approximate effective period of 3 to 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 buildings. The isolators will consist of a combination of 54 and 69 bearings for the Miragoane and Jacmel cathedrals. 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 (Fig. 5). 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; see Fig. 5.





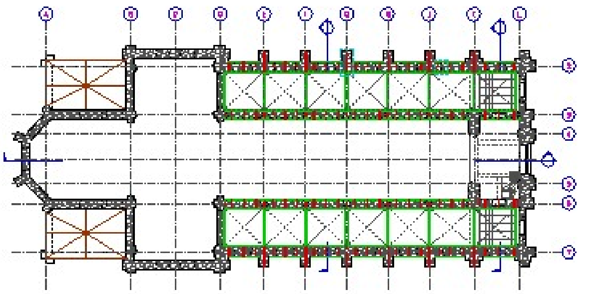
Elevation Plan (Jacmel)

Fig. 5 Typical detail of isolation plane

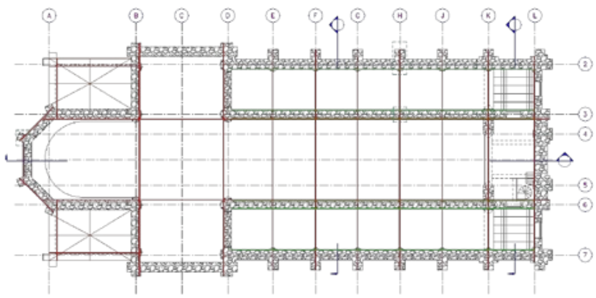
2.4 Structural load path intervention

The cathedrals in their existing configuration lack 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. For the seismic isolation system to be effective, this type of failure need to be precluded. 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 [4] for retrofit of historic buildings.

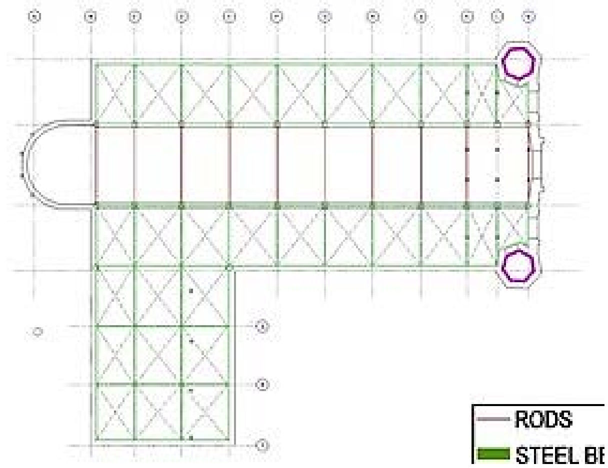
Fig. 6 presents the plan and elevation view of the cathedrals showing the added steel members. Less intervention was required for the Jacmel Cathedral since it is subjected to smaller seismic demand. As shown in the figures steel members are added: a) Horizontal tie-down steel rods are added to the building 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 cathedrals, 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.



Steel bracing



Steel Rods



Miragoane Cathedral Jacmel Cathedral

Fig. 6 Typical structural intervention details

3 Structural capacity of the walls

The Cathedrals' unreinforced stone masonry (URSM) walls are the load bearing elements resisting the applied vertical and lateral load applied to the building. Fig. 7 depicts exposed sections of the walls with the wall plaster removed for investigation for the two structures. 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

Irregular-shaped stones placed in the mortar Rectangular-shaped stone and debris in mortar

Fig. 7 Typical composition of exposed unreinforced masonry stone walls

The nominal strength of the URSM walls was based on the provisions of the Italian seismic code for unreinforced walls [5]. 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

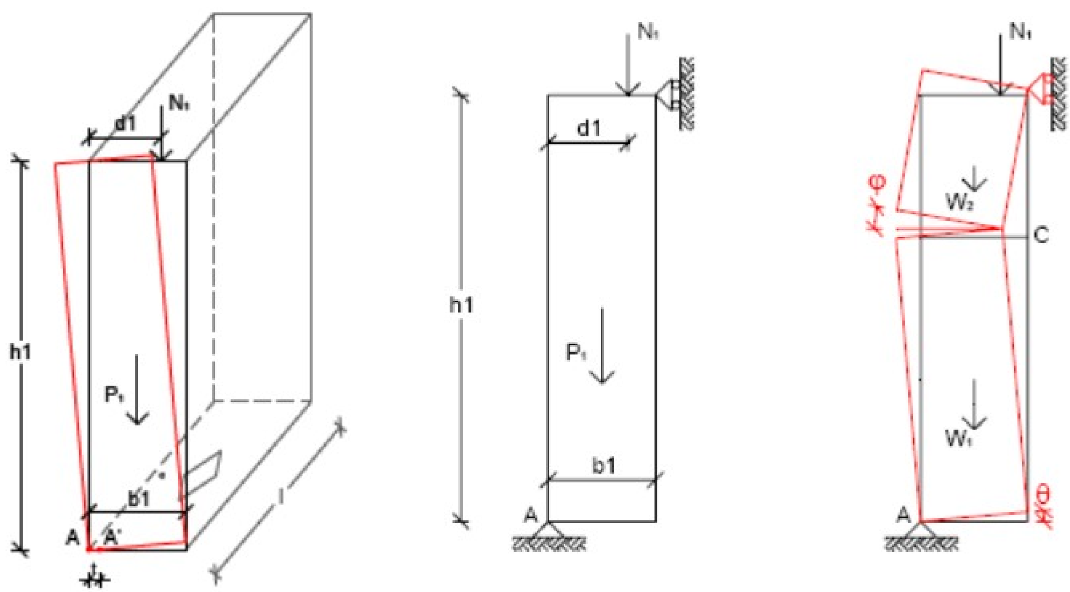
Italy. The URSM walls have the lowest mechanical properties of approximately 1.0 MPa for compressive strength

3.1 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 [6] was used to determine the out-of-plane capacity of the walls.

3.2 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 Fig. 8. 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 Fig. 8). 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 [7].



Existing Strengthened

Fig. 8 Out-of-plane failure modes for a typical wall segment

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. The computed capacities are further adjusted by two factors: Knowledge factor to account for uncertainties in material properties, construction details, and geometric characteristics. Behavior factor 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. Equilibrium kinematic analyses of various walls of the two cathedral were conducted. Table 2 summarizes the findings. As long as the isolated buildings acceleration demands are less than the governing values, out-of-plane failure will not occur.

Table 2 - Computed out-of-plane capacity (g) of Cathedral walls

Wall segment	Miragoane	Jacmel
Typ. between windows	0.39	0.19
Transept end wall	0.25	0.19
Central walls	0.44	0.26
Apse	0.82	0.21
Upper masonry above windows	0.73	0.20
Upper transept end wall	1.45	0.19
Bell tower	0.52	0.21

3.3 In-plane capacity walls

For the Miragoane Cathedral, the walls at the perimeter of the bell tower are strengthened by adding eight (8) #5 (16 mm) reinforcement on each side. Holes will be drilled the height of the wall and vertical (longitudinal) reinforcement will be grouted in the holes. The reinforcement will be hooked at the base into the foundation to ensure that the reinforcing bars are developed and thus the full moment capacity of the walls can be achieved.

Cross-sectional analysis was conducted to develop the axial force-bending moment interaction diagram for the bell tower and main cathedral walls The capacity of the bell tower walls was determined using static pushover analysis using plastic hinges whose properties were obtained from interaction analysis and program 3Muri [8]. Both flexural and shear failure modes were accounted for in the nonlinear analysis. 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.

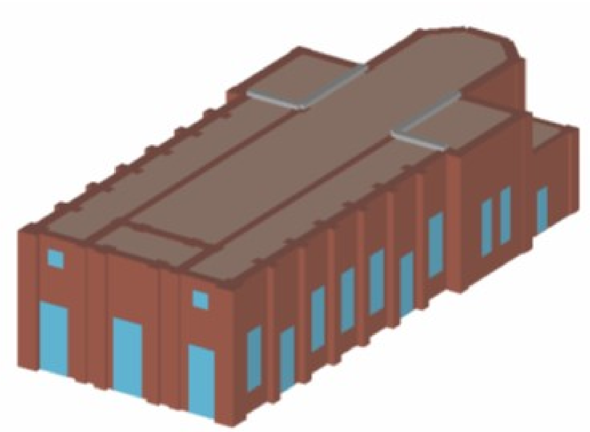
Shown in Fig. 9 and Fig. 10 are the state of the cathedrals at their limit state. In the figures:

- Green denote wall segments that remain elastic
- Pink corresponds to flexural yielding
- Red designates flexural failure
- Ivory indicates shear yielding
- Light blue represents traction failure

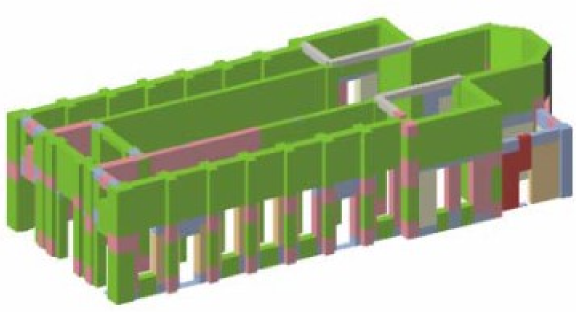
For the cathedrals, no shear failure were developed and limit state was reached when walls reach their ductile flexural capacity. The failure states were reached at a base spectral acceleration of 0.12 and 0.16 g, respectively. The progression of the nonlinear response in is listed in Table 3.

Table 3 - Progression of nonlinear response in the cathedrals

State	Miragoane	Jacmel
Flexural yielding , g	0.04	0.09
Shear yielding, g	0.05	0.09
Flexural failure, g	0.10	0.13
limit state, g	0.12	0.16

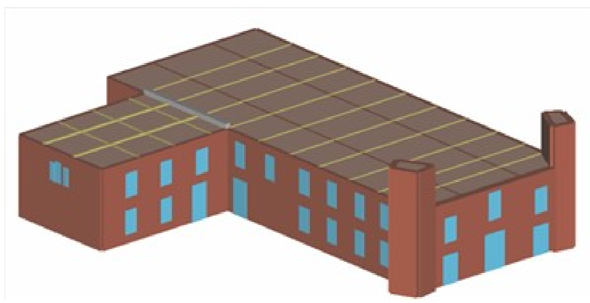


Mathematical model

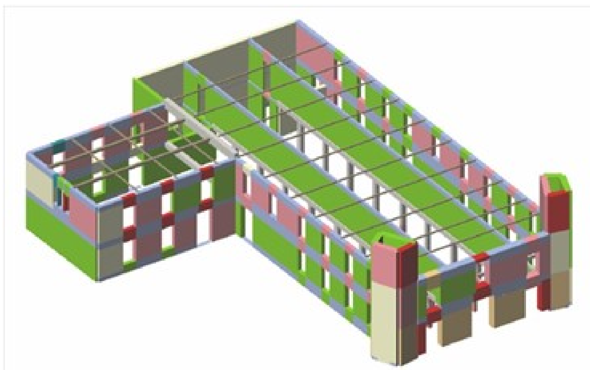


Pushover results

Fig. 9 Miragoane Cathedral



Mathematical model



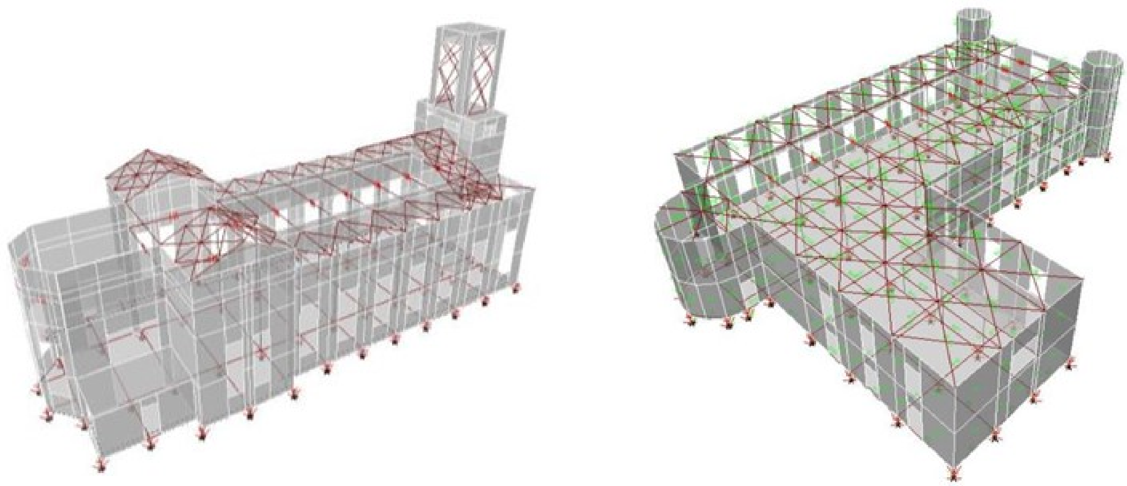
Pushover results

Fig. 10 Jacmel Cathedral

4 Analytical model and results

4.1 Overview

Three-dimensional analytical model of the cathedrals were prepared; see Fig. 11. The isolation system and new steel members are highlighted for clarity. The total inertial mass of the structure is estimated at 2,800 and 3,100 Mg, respectively.



Miragoane Jacmel

Fig. 11 Analytical model of the building

4.2 Drift ratios

Table 4 presents the computed story drift ratios above the isolation. The maximum story drift ratios are 0.42% and 0.49% ate the DE and MCE levels, respectively. The maximum computed drifts were 0.11% and 0.13% for Miragoane and Jacmel cathedrals, respectively.

Table 4 - Computed story drift ratios, Miragoane

Story	DE		MCE	
	X-	Y-	X-	Y-
TOWER ROOF7	0.11%	0.05%	0.12%	0.06%
TOWER FLR6	0.10%	0.06%	0.11%	0.07%
HIGH ROOF5-1	0.09%	0.10%	0.10%	0.10%
HIGH ROOF5	0.30%	0.20%	0.34%	0.25%
ROOF4	0.14%	0.20%	0.17%	0.27%
ROOF3	0.16%	0.24%	0.19%	0.31%
MEZZANINE2	0.24%	0.26%	0.29%	0.30%
LOW ROOF1	0.20%	0.42%	0.25%	0.49%

For unreinforced masonry non-infill walls, ASCE 41 [1] limits drift ratios to 0.6% and 1% for life safety and collapse prevention, respectively. Therefore, for the retrofitted structure, at both DE and MCE levels, performance of between IO and LS are obtained

4.3 Story shear

Fig. 12 and Fig. 13 present the distribution of shear force (normalized with respect to the building seismic mass) along the height of the structures. The effective base shear for the two buildings are approximately 12% and 7% g, respectively at the DE intensity. The addition of the isolation system has served to reduce significantly the demand on the structure.

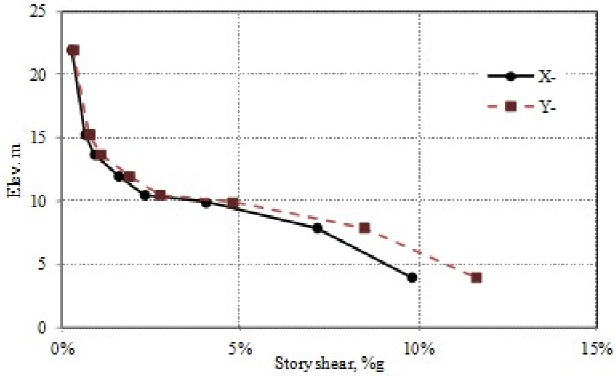


Fig. 12 Story shear, Miragoane

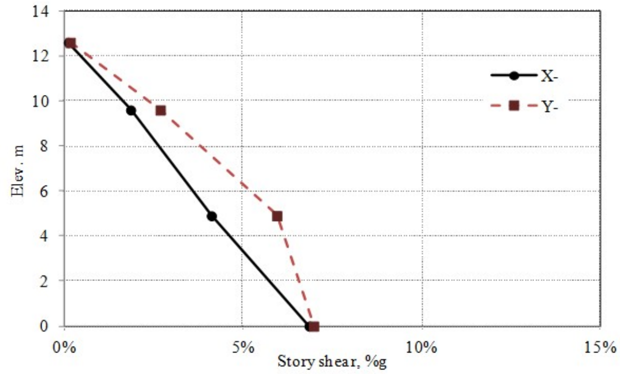


Fig. 13 Story shear, Jacmel

4.4 Out-of-plane accelerations

The out-of-plane accelerations at the ground slab and roof for Miragoane cathedral are listed in Table 5. The first column shows the capacity values computed from kinematic analysis. Columns 2 and 3 of the table show the computed demand at DE and MCE intensities. The computed demands at both DE and MCE are less than the capacity. In other words, no out-of-plane failure is anticipated even at the MCE.

Table 5 - Out-of-plane accelerations at wall base, Miragoane

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

4.5 Response of the isolation system

Fig. 14 presents the force-deformation response of a typical isolator from a MCE analysis. Also shown in the figure are the nominal bi-linear backbone curve of the isolator obtained from the manufacturer data. It is seen that the loading and unloading response closely track the nominal response. Fig. 15 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 are less than the capacity.

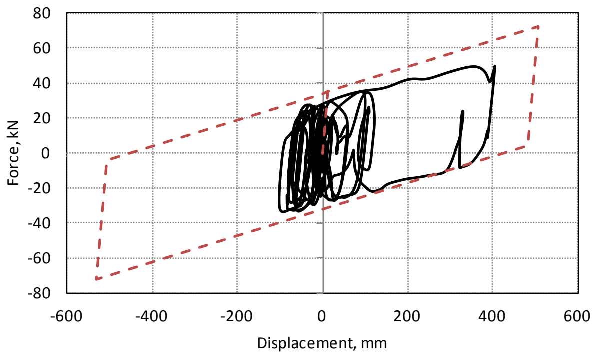


Fig. 14 Typical isolator response

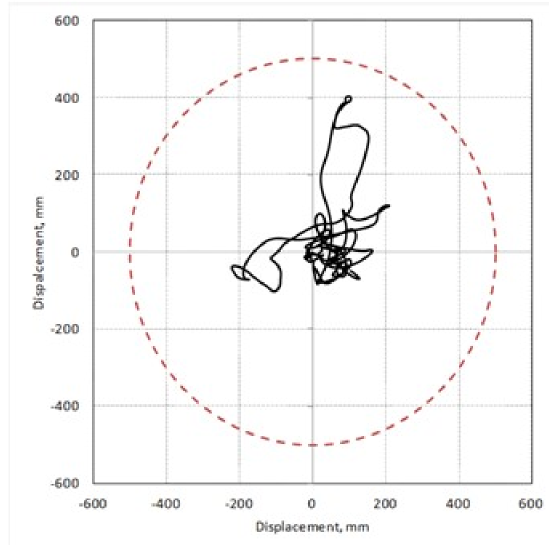


Fig. 15 Bi-directional response

5 CONCLUSIONS

The Miragoane and Jacmel Cathedrals were constructed of nonductile URSM walls and do not meet the current code requirements for seismic performance. The structures are being retrofitted with an isolation system and strengthening measures to improve their load path and the out-of-plane capacity of the walls.

- Analysis showed that the retrofit including the addition of the isolation system would significantly reduce the story drifts, accelerations, and shear.
- 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

6 Project status

For Miragoane building buildings, the structural interventions have been completed. An approximate of cost breakdown is presented in Table 6.

Table 6 - Seismic retrofit and building renovation cost estimates in $US 1000

Item	Miragoane	Jacmel*
Structural upgrade	$US470†	$US1,500‡
Renovations (doors, windows, tiles, furniture, EMP, exterior, yard)	$US730	$1,800
Additional work (roof replacement, chandeliers, etc.)	$US500	

* Estimate only
†† Excluding cost of seimic isoaltion
‡ Inclusding cost of seimic isoaltion

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