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Numerical Study on Seismic Performance of Stone Masonry Building in Cement Mortar

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Abstract
Stone masonry buildings, on their own, without seismic provisions are found vulnerable during earthquakes. Hence, various seismic provisions such as inclusion of different horizontal bands and vertical reinforcements are proposed by several national/international guidelines for the strengthening of the structure to resist seismic action. This thesis focuses on the analytical methodology of seismic performance assessment of those stone masonry buildings with cement mortar through a case study of building prototype presented in Department of Urban Development and Building Construction (DUDBC) catalogue. The work analyzes the effectiveness of several strengthening components of masonry model with comparison to simple unreinforced masonry model. Stepwise procedures of seismic assessment are analyzed through non-linear pushover analysis in DIANA FEA and fragility functions are generated to define the vulnerability level of the studied buildings. The fragility analysis highlights the vulnerability of the unreinforced buildings over the reinforced counterparts.

Keywords
Stone Masonry, Finite Element, Fragility

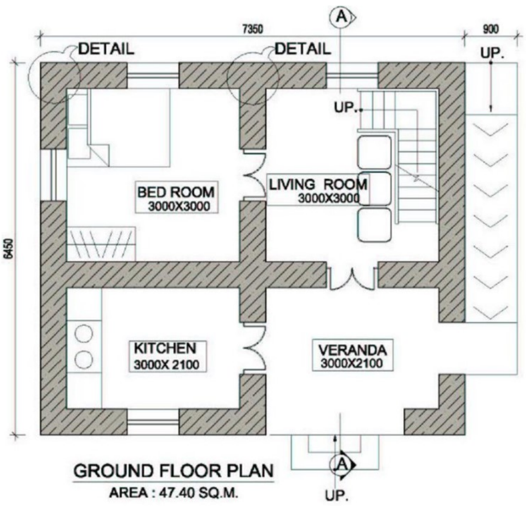
1. Introduction

Being a seismically active geological feature Nepal was hit by many devastating earthquakes with magnitudes greater than Mw7.5 in its history as; 1255, 1408, 1505, 1833, 1934, and 2015 [1]. These earthquakes led to various damages to the structure, enormous losses in economy and lives. As reported by the National Planning Commission (NPC)-2015, Nepal Gorkha earthquake caused over 8790 fatalities and 22300 injuries [2]. According to the NRA (2021), out of 1,047,261 damaged houses surveyed, 78.4% were of low-strength masonry, 7.87% cement-mortared masonry and only 3.57% were reinforced concrete houses. Low-strength masonry houses were greatly affected in comparison to other typologies in the recent Gorkha earthquake in 2015 [3]. Masonry structures not built according to the design guidelines are the most prominent type of construction in the rural areas of Nepal which were hit hard by those seismic actions [4]. After the Gorkha earthquake, it is realized that the seismic resistant design of masonry structures is most important to minimize the damages and losses during the earthquake. The Government of Nepal proposed the Build Back Better philosophy which emphasized for the provision of different strengthening component for the construction of earthquake resistant masonry buildings. NBC 202:2015 made a provision of different horizontal band at different level and vertical reinforcement. Considering those provision, Department of Urban Development and Building Construction (DUDBC) also made the design catalogue for reconstruction of earthquake resistant houses presenting different model of houses. These model buildings consist of horizontal band at different level (plinth band, sill band, lintel band, stitches, roof band, gable band etc.), vertical reinforcement at corner and junction of wall, through stone etc [5] [6]. The provision of seismic bands hold the walls together and ensure the integral box action of an entire building [7]. During reconstruction, those types of buildings are constructed in large numbers. Thus it is necessary to measure how vulnerable those structures are to damage when the ground shakes severely. But the limited research have been performed to know the seismic performance of those provision. So, to evaluate the seismic performance of these strengthened buildings and to quantify their seismic vulnerability, a research work which includes finite element modeling of a case study stone masonry building in cement mortar (SMC-2.2 model) [6] taken from DUDBC Catalogue is presented here. The study focus on the modeling in DIANA FEA, non-linear static analysis, capacity spectrum method for performance point and fragility curve generation to check the seismic vulnerability of two cases: Reinforced and Unreinforced masonry building.

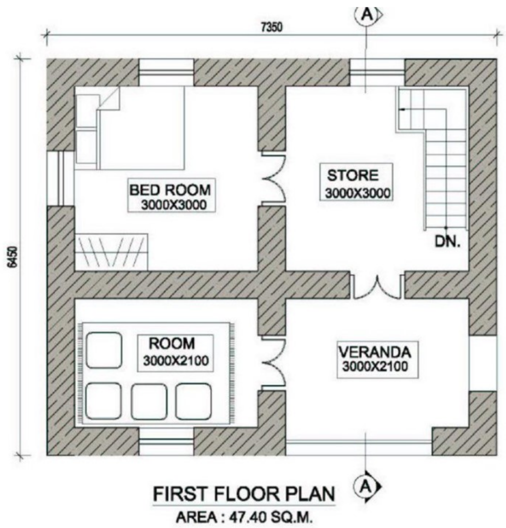
2. Case Study on Stone Masonry Building

2.1 Building Description

DUDBC proposed different masonry building typologies for the reconstruction for different type of construction material such as: Stone Masonry in Cement Mortar (SMC), Brick Masonry in cement Mortar (BMC), Stone Masonry in Mud mortar (SMM) and Brick Masonry in Mud Mortar (BMM). Among these, the stone masonry in cement mortar (SMC-2.2) building is taken as case study model in this research works which represents the rural stone masonry buildings. SMC-2.2 model is 2 story building provided with reinforced concrete band at plinth level, sill level, lintel level, roof level and at stitches. The building is also provided with vertical reinforcement at corner and junction of the wall. The plan of the building is shown in Figure-1.



(a)



(b)

Figure 1: Plan of Case Study Building a) Ground Floor Plan b) First Floor Plan (Source: DUDBC Catalogue)

The wall thickness and height of the building is 450 mm and 5.3 m respectively. The thickness of the different horizontal band and reinforcement provided are tabulated in Table-1. The size of reinforcement for both horizontal band and vertical reinforcement is 12 mm.

Table 1: Band Detail

Band	Thickness (mm)	No. of Reinforcement
Sill	75	2
Lintel	150	4
Stitches	75	2
Roof	100	2

2.2 Finite Element Modeling

The case study building is modeled in DIANA FEA 10.5 [8] considering two different cases as: I) Unreinforced Model Case (URM) and II) Reinforced model Case (RM), shown in Figure 2 and 3 respectively. Reinforced model consist of vertical reinforcement at corner and junction of walls and horizontal bands; sill band, lintel band and stitches in both floors and roof band at top level of building. While unreinforced model excludes all the horizontal bands and vertical reinforcement. Figure 4 shows the reinforcement modeled in this building.

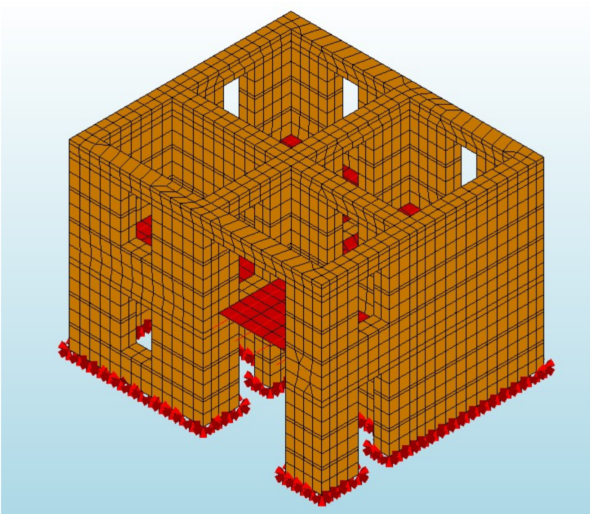


Figure 2: FE Model for Unreinforced Case

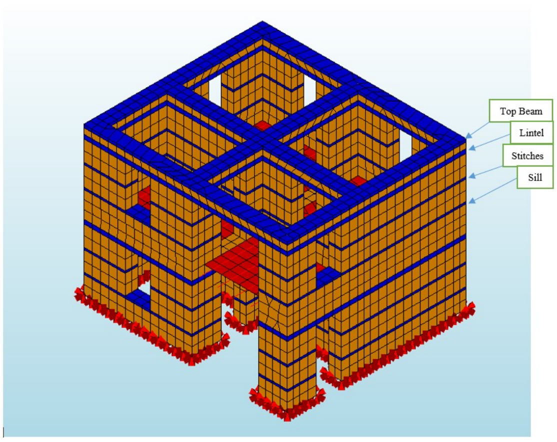


Figure 3: FE Model for Reinforced Case showing bands at different level

A 3-D macro-modeling approach is adopted to model the building. The wall components and band elements are modeled as solid elements, floor beams and reinforcements are modeled as line elements, and the timber floors are modeled as thin shell area elements. For this research work, the roof is not modeled but the dead load of roof is applied to the wall. The area load of 2 kN/m^2 is applied at floor level. The foundation is supposed to be rigidly fixed at ground level. The total strain crack model in Diana is used for simulations of failure of brittle materials such as masonry and concrete [8]. The total strain-based crack model follows a smeared cracking approach which consider cracking as a distributed effect with directionality. The tensile and compressive behavior are taken as exponential softening and parabolic softening respectively. Rot's element based method is used to compute the crack bandwidth for the calculation of crack width output. Reinforcements are modeled as fully embedded in the elements in which they are located and are fully coupled, i.e. they do not allow relative slip. Also, the reinforcement bars are modeled as Von Mises Plasticity for which strain hardening is considered. The timber element for wooden floor and floor beam are modeled as linear material.

Material properties taken for modeling are tabulated in Table-2. The strain hardening considered for reinforcement is shown in Figure 5.

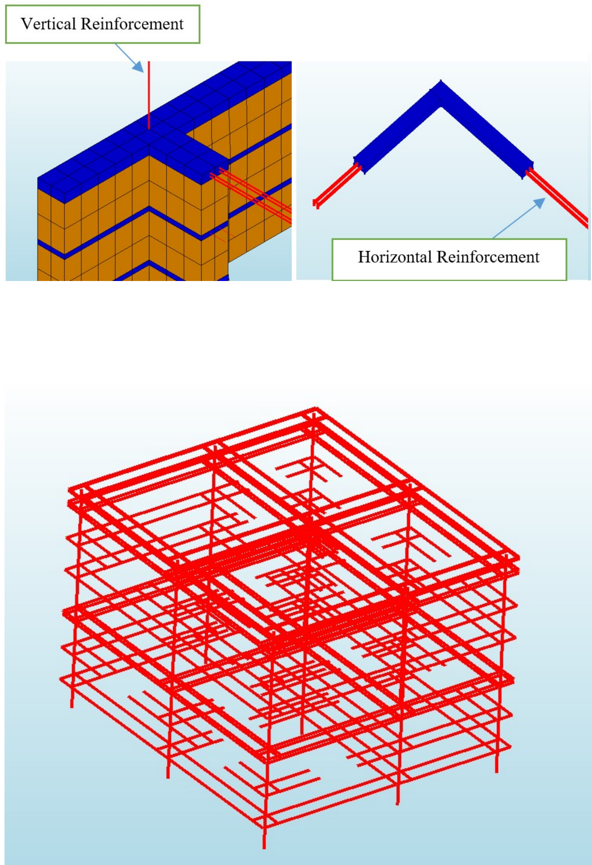


Figure 4: Reinforcement Details

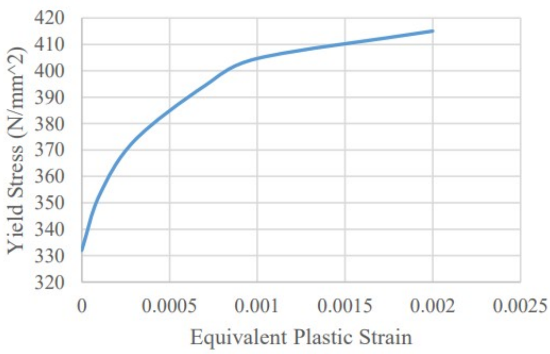


Figure 5: Stress-strain diagram for reinforcement

Table 2: Material properties taken for modeling

Material Properties	Value	Unit
Masonry		
-Unit Weight (IS 875)	23.24	kN/m^3
-Elasticity (FEMA 368)	1800	N/mm^2
-Poisson's Ratio	0.25	
-Tensile Strength	0.1	N/mm^2
-Compressive Strength	2.4	N/mm^2
Concrete-M20 (IS 456)		
-Unit Weight	25	kN/m^3
-Elasticity	22360.7	N/mm^2
-Poisson's Ratio	0.2	
-Tensile Strength	2.71	N/mm^2
-Compressive Strength	20	N/mm^2
Timber (Sal wood)		
-Unit Weight	7.9	kN/m^3
-Elasticity	12670	N/mm^2
-Poisson's Ratio	0.3	
Steel		
-Elasticity	200000	N/mm^2

2.3 Analysis

Eigen value analysis is carried out for two different cases: RM and URM to compare their time period. The non-linear static analysis, pushover analysis, is performed to obtain the capacity curve by selecting the control node at floor level. By using the capacity spectrum method the structural performance point is obtained by intersecting the capacity curve with the demand curve from the response spectrum of NBC105 (2020) following the procedure proposed by Otani et. el.(2000) [9]. The different value of ground motion parameters at performance point is correlated to damage state, expressing the vulnerability of a structure to a user-defined intensity measure. The fragility curve is generated using the methodology proposed by Wen et. al.(2004)[10] considering the different threshold value for different damage state as shown in Table 3.

Table 3: Threshold value for damage states

Damage State	URM [11]	RM [12]
Slight Damage	1/750	1/500
Moderate Dame	1/500	3/500
Extensive Damage	1/250	3/200
Collapse	1/100	1/50

3. Results and Discussions

For RM and URM the fundamental time period are 0.1046 sec 0.1151 sec respectively from modal analysis. The time period for reinforced models are comparatively lesser than unreinforced model i.e. the unreinforced model is flexible than reinforced model. Presence of horizontal band increases the structural integrity which makes the reinforced model more stiff than unreinforced model.

From the pushover analysis the maximum base shear for RM and URM are 1844 kN and 1434 kN which occurs at drift level of 0.43% and 0.33% respectively i.e. reinforcing the building with horizontal band and vertical reinforcement increases the capacity of the building. The unreinforced masonry reaches to its maximum capacity at earlier stage of loading than reinforced masonry. The pushover curve is shown in Figure 6.

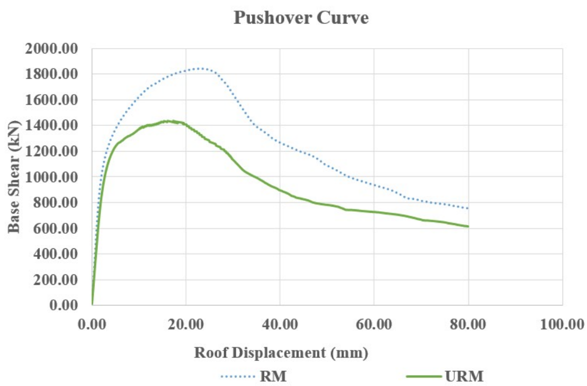


Figure 6: Pushover curve along the direction of Fundamental Mode

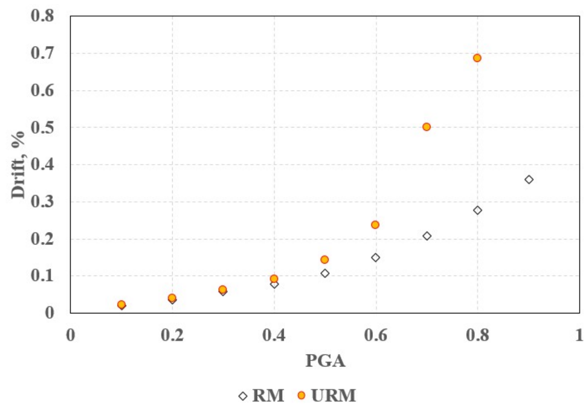


Figure 7: Variation of Drift with PGA

The drift corresponding to spectral displacement as obtained from the performance point for different PGA level for RM and URM are compared as shown in Figure 7. At 0.5g PGA, the drift for RM and URM are 0.108% and 0.143% respectively. At higher level of PGA (i.e. greater than 0.6 g) the drift for URM is much higher than that for RM. The presence of bands in reinforced model delays the damages that occurs in unreinforced model for the same level of ground motion.

The analytical fragility curves are obtained from fragility analysis. The fragility curves for URM (Figure 8) and RM (Figure 9) are compared which are shown in Figure 10-13. At 0.5 g PGA, for RM and URM, the probability of exceedance for slight damage are 45% and 75%, for moderate damage 16% and 61%, for extensive damage 4% and 35% and for collapse 3% and 10% respectively. This shows that the URM is more vulnerable than RM at same level of earthquake loading, that means the reinforcing element such as band and vertical reinforcement enhance the capacity of the building.

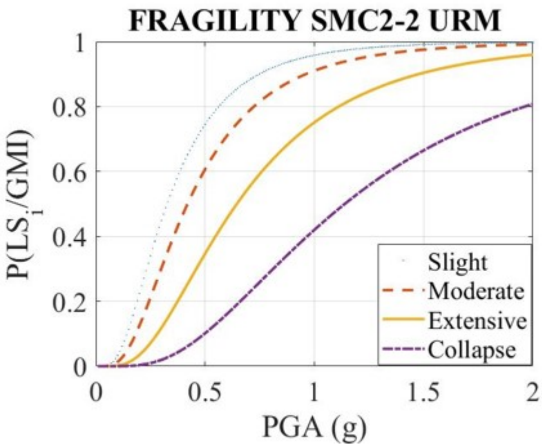


Figure 8: Fragility Curve for URM

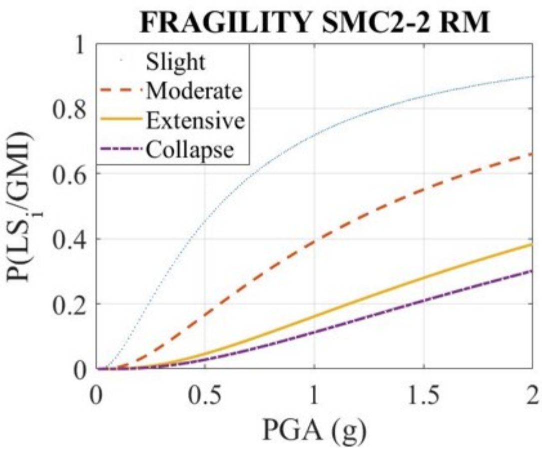


Figure 9: Fragility Curve for RM

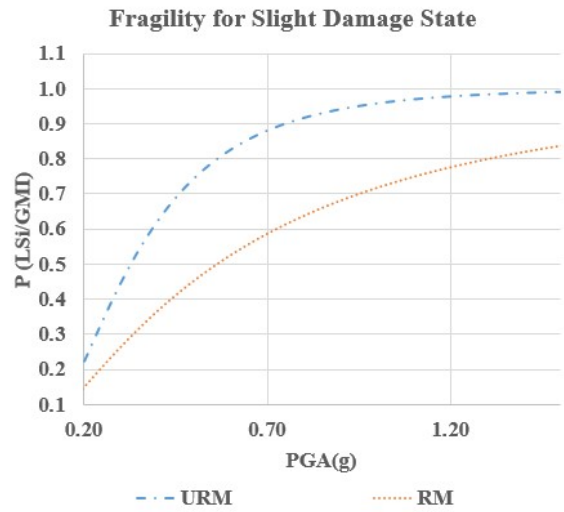


Figure 10: Fragility Curve for Slight Damage State

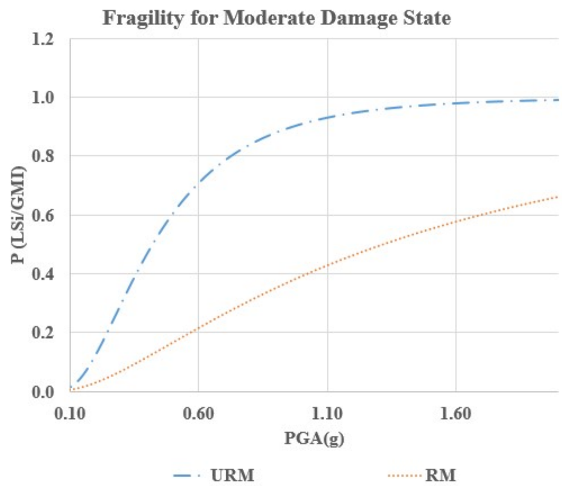


Figure 11: Fragility Curve for Moderate Damage State

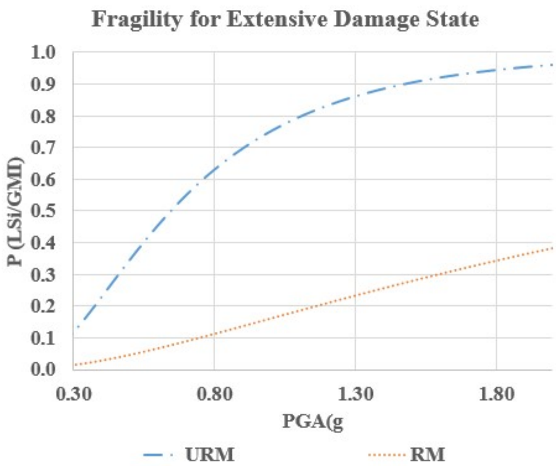


Figure 12: Fragility Curve for Extensive Damage State

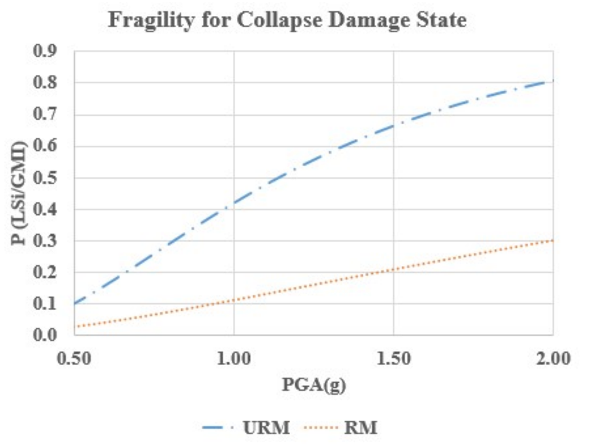


Figure 13: Fragility Curve for Collapse Damage State

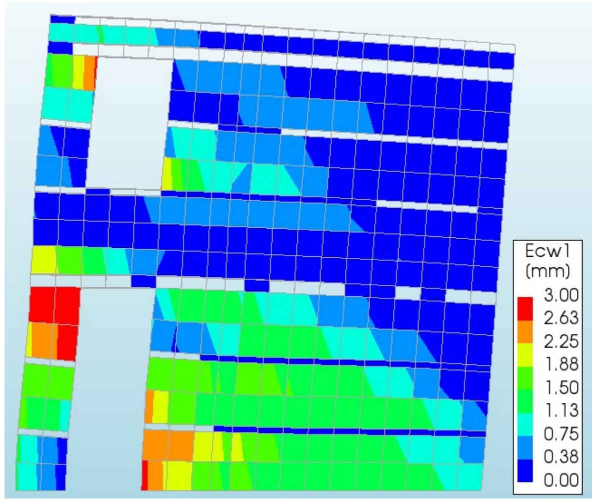


Figure 15: Crack width contour for RM at maximum base shear corresponding to drift of 1/233

The maximum crack width for RM and URM cases corresponding to their maximum base shear at drift level of 0.43% and 0.33% are 4.39 mm and 5.29 mm respectively. The maximum crack width at different drift level for RM and URM cases are compared in Figure 14. Figure 15 and 16 shows the crack distribution pattern for RM and URM cases respectively. In RM the crack is distributed within the wall while in URM the crack is concentrated at the location of opening. In RM the horizontal band broke the continuity of the crack. In URM maximum cracks formation are at first floor level but in RM cracks are also distributed in second floor level.

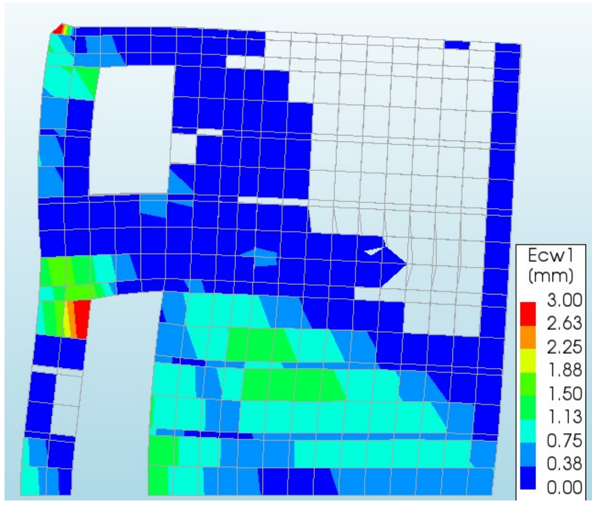


Figure 16: Crack width contour for URM at maximum base shear corresponding to drift of 1/303

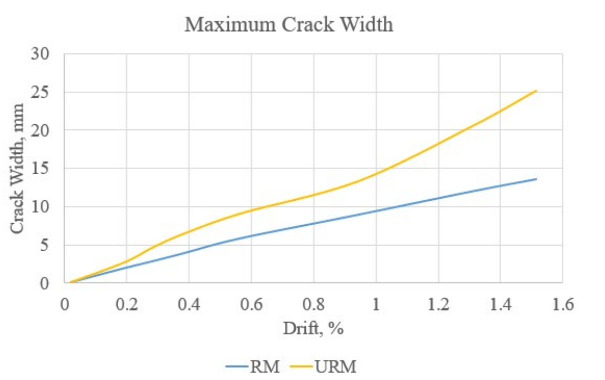


Figure 14: Maximum Crack Width at different drift level

The stress distribution on the reinforcement at maximum base shear corresponding to drift of 0.43% for RM is shown in Figure 17. The vertical reinforcement at corner yield earlier than the reinforcement at horizontal band. Stresses near to opening is higher for the band reinforcement. In RM case initially the stresses are taken by the reinforcement which enhance the capacity of the building and minimize the damages.
Hence, addition of reinforcing component as; horizontal bands and vertical reinforcement at junction of wall improves the performance of the building during the lateral load.

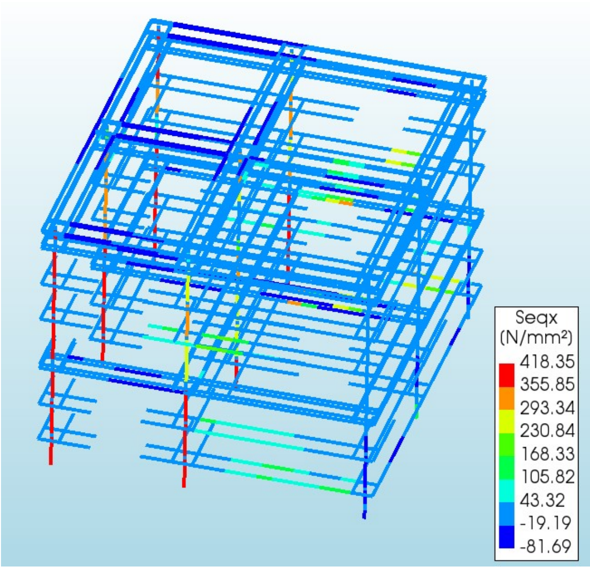


Figure 17: Stresses in Reinforcement

4. Conclusions

A case study SMC-2.2 model building from DUDBC catalogue (stone masonry in cement mortar) is analysed numerically with FE modeling in DIANA FEA. The two cases RM and URM is modeled in which RM includes vertical reinforcement and horizontal bands as: sill band, lintel band, roof band and stitches while URM excludes both horizontal band and vertical reinforcement. The result obtained from analysis and fragility generation for both the cases are compared. On the basis of the work performed following onclusions are made:

- The strengthening elements like horizontal band and vertical reinforcement enhance the performance and capacity of the RM buildings.
- The distributed crack development pattern and lower crack width for RM as compare to URM shows that horizontal band increase the structural integrity of the building.
- The fragility analysis of this buildings revealed that the reinforced masonry buildings proposed by DUDBC has lower chance of reaching collapse and extensive damage stage at earthquake with PGA lower than 0.5 g. But the unreinforced masonry has 10 to 35% chances to reach the collapse and extensive damage stage. There is high chances of slight to moderate damage (60% to 75%) of URM building at earthquake of intensity lower than 0.5 g PGA as compare to RM building (for which chances of slight to moderate damage is 16% to 45%).
- The URM buildings also have better performance in terms of damage prevention which is possibly due to greater wall thickness for regular dimension of the building.

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