Seismic Performance of a Retrofitted Foundation: A Case Study in Chandigarh

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Abstract

Chandigarh is a major city situated in northern part of India. The city has predominantly alluvial soil comprising of clay, silt and sand with piedmont deposits in the north-eastern fringe. The case study described in this paper consists of a masonry building with strip foundation built over predominantly clayey soil with variable plasticity. The city falls under seismic zone IV, according to IS1893:2002, which experiences earthquake of moderate to high intensities. Hence owing to past seismic activities and insufficient bearing capacity, the building has undergone considerable differential settlement, leading to severe cracks in the superstructure. The process of rehabilitation of the building included a damage assessment study, following by retrofitting measures of the entire structure. As it is difficult to improve the bearing capacity of the soil underlying the existing structure, an attempt has been made to retrofit the existing foundation system with innovative construction scheme in order to improve the uniformity in load distribution of the building, thus increasing the safe bearing capacity to prevent further differential settlement. The results from a comparative study of responses between the finite element models of the existing and retrofitted foundation have shown an improved performance when subjected to lateral loading.

Keywords: Retrofitting, Foundation, Seismic Performance, Finite Element Model

1. Introduction

The case study comprises of a double-storied load bearing masonry structure used for residential purposes in the premises of CSIR-Central Scientific Instruments Organization (CSIO), Chandigarh. The age of construction is approximately 40 years, and a damage assessment study has revealed severe cracks in the walls and the roofs of the structure. The main cause of the distresses was identified to be differential settlement of the building. The city has predominantly alluvial soil comprising of clay, silt and sand with piedmont deposits in the north-eastern fringe [1]. In addition to that, an investigation of older city maps revealed that the building was constructed on a land where a drainage canal had existed previously, and hence it could be inferred that the moisture content of the soil was more. Chandigarh also experiences moderate to high seismic events and falls under Zone IV of the seismic map of India [2]. Hence, both these factors contributed to the differential settlement which manifested itself in the severe cracking of the building elements. Since it was difficult to improve the bearing capacity of the underlying soil, an attempt was hence made to improve the bearing pressure of the foundation with retrofitting measures.

2. Description

The foundation system of the building consists of masonry strip foundation as shown in Fig.1, extending till a depth of 600 mm. In order to improve the safe bearing pressure, the following retrofitting measures were suggested:

- Increasing the footprint of the foundation
- Increasing the area of the foundation with concrete filling
- Integrating the new portion with the old foundation with the help of post-tensioned steel bars
- Confining the entire section with steel plate to improve the cohesive behavior of the entire section

The schematic diagrams of the old and retrofitted foundations are given in Fig.2 and Fig.3 respectively.

Fig.1 Existing Strip Foundation

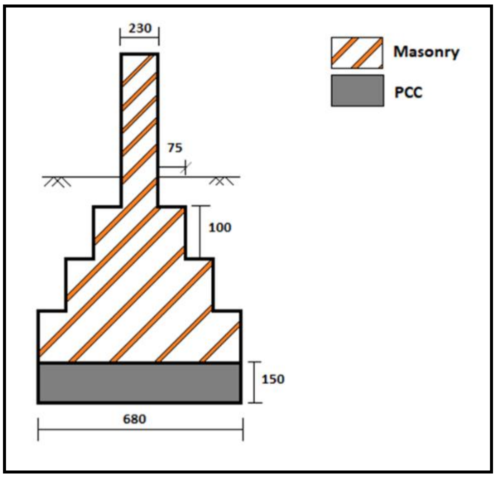


Fig.2 Existing foundation system

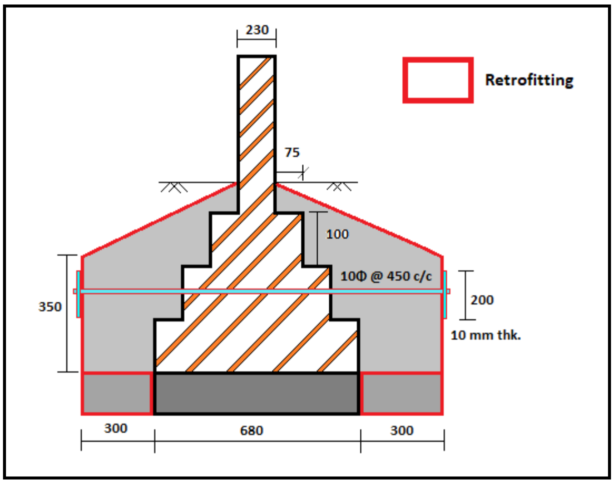


Fig.3 Retrofitted foundation system

3. Methodology

The finite element models of the foundations are built in the software suite ABAQUS [3], and subjected to an out-of-plane lateral displacement comparable to a situation when the structure is subjected to seismic forces. Consequently, the structural responses are measured and interpreted for the improvement in performance of the foundation.

4. Finite element Model

The foundation is modeled as 3D deformable solid body. The masonry unit is macro-modeled as a homogenous material with the behavior equivalent to damaged plasticity model of concrete under cyclic loading. According to the experimental data, the average compressive strength of brick used in the structure is 9.29 MPa. For cement mortar, an intermediate mortar of 1:4/5 ratio is considered for structural works [4]. A cement mortar of the above ratio is designated as MM5 according to IS2250 [5] and has an average strength of 6.25 MPa. The compressive and the tensile strength of masonry is calculated by homogenizing the masonry as single material based on the brick and mortar strength [6, 7, 8]. The stress-strain curve as derived for the masonry is given in Fig.4. The homogenized masonry, being similar in its behaviour to concrete, is assumed to follow the Concrete Damaged Plasticity (CDP) model with parameters taken from literature [9].The retrofitted concrete section of grade M20 is defined to follow the CDP model with parameters taken from literature scaled to Indian standard codes [10, 11]. The material properties of PCC, rebar steel and steel plate is taken according to Indian Standard codes [11, 12, 13, 14]. The summary of the different parts and material properties is given in Table 1 and Table 2 respectively.

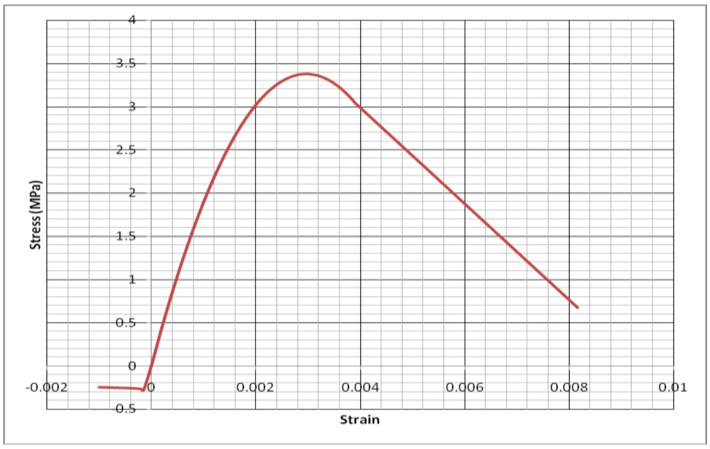


Fig. 4 Stress-strain curve for Masonry

Table 1 - Description of different parts of the Foundation

Part	Material
Base	Plain Cement Concrete (PCC of grade M15)
Strip Foundation	Masonry
Filled portion	Concrete (M20)
Post-tensioned tendon	Steel (Fe415) 10φ mm
Plate	Steel (Fe410) 200 mm x 10 mm

Table 2 - Material Properties

<table>
<tr><th>Material</th><th>Properties</th></tr>
<tr><td>Steel bar</td><td>Density = 7850 kg/m^3
Young's Modulus (E) = 2.1x 10^{11} Pa
Poisson's Ratio = .3
Yield stress = 415 MPa
Ultimate stress = 485 MPa
Maximum elongation = 14.5%</td></tr>
<tr><td>Steel plate</td><td>Density = 7850 kg/m^3
Young's Modulus (E) = 2.1x 10^{11} Pa
Poisson's Ratio = .3
Yield stress = 250 MPa
Ultimate stress = 410 MPa
Maximum elongation = 23 %</td></tr>
<tr><td>PCC</td><td>Density = 2400 kg/m^3
Young's Modulus (E) = 19364 MPa
Poisson's ratio = .25
Yield stress = 15 MPa</td></tr>
<tr><td>Concrete</td><td>Density = 2400 kg/m^3
Young's Modulus (E) = 22360.68 MPa
Poisson's ratio = .2
Material Model = Concrete Damaged Plasticity (CDP)
• Dilation angle =38º
• Eccentricity = .1
• fb0/fc0 = 1.16
• K = .667
<table>
<tr><th colspan="3">Compressive Behaviour</th></tr>
<tr><th>Stress (MPa)</th><th>Crushing Strain</th><th>Damage Parameter</th></tr>
<tr><td>5.999077102</td><td>0</td><td>0</td></tr>
<tr><td>8.077878899</td><td>7.47307E-05</td><td>0</td></tr>
<tr><td>11.99839777</td><td>9.88479E-05</td><td>0</td></tr>
<tr><td>16.11903265</td><td>0.000154123</td><td>0</td></tr>
<tr><td>20</td><td>0.000761538</td><td>0</td></tr>
</table></td></tr>
</table>

	16.09196041	0.002557559	.195402
	8.093190944	0.005675431	.596382
	2.102699321	0.011733119	.894865
	Tensile Behaviour		
	Stress (MPa)	Cracking Strain	Damage Parameter
	2.201844	0	0
	3.130495	3.33E-05	0
	2.059617	9.74E-05	.406411
	0.950299	0.000182	.69638
	0.249221	0.000556	.920389
	0.062319	0.00095	.980093

Masonry

Density = 1920 kg/m^3
Young's Modulus (E) = 1856.8 MPa
Poisson's ratio = .25
Material Modelling = Concrete Damaged Plasticity (CDP)

- Dilation angle = 30^o
- Eccentricity =.1
- fb0/fc0 = 1.16
- K = .667

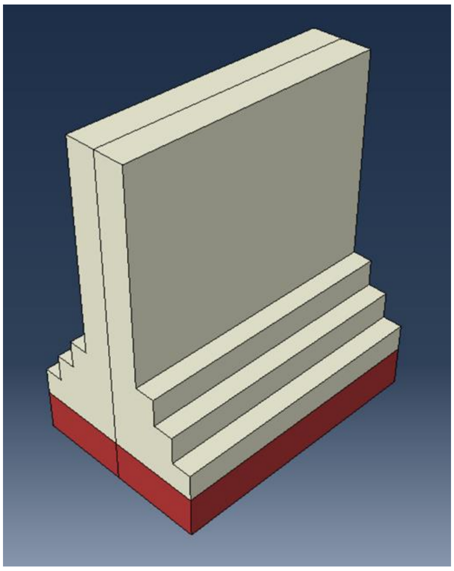
Compressive behavior		Tensile behavior	
Stress (MPa)	Crushing Strain	Stress (MPa	Cracking Strain
2.85267	0	0.28	0
3.01639	0.000182	0.26462	5.00E-05
3.14952	0.000382	0.25819	0.00015
3.252	0.000582	0.25281	0.00035
3.32385	0.000782	0.24375	0.00085
3.36506	0.000982		
3.376	0.001151		
3.37563	0.001182		
3.35556	0.001382		
3.30486	0.001582		
3.22351	0.001782		
3.11153	0.001982		
3.0384	0.002089		

		2.9869	0.002182	
		2.4319	0.003182	
		1.8769	0.004182	
		1.3219	0.005182	
		0.766896	0.006182	
		0.6752	0.006347	

5. Model properties

The model consists of a meter of the wall and the connecting foundation system, built as 3D deformable solid with dimensions as measured in the case study. Two models have been built, Model -1 as the existing foundation and Model- 2 as the proposed retrofitted foundation as shown in the Fig.5. Both the models have been meshed with hexahedral C3D8R elements and convergence check has been run, as shown in Fig.6.

The base of the foundation has been taken as fixed support and the interacting surfaces of PCC, masonry, concrete and steel plate have been tied to reduce the computational time of the analysis. The steel bars have been embedded in the retrofitted portion at a depth of 525 mm from the GL at an interval of 450 mm c/c. The bars have been post-tensioned and bolted to a steel plate of 200 mm x 10 mm along the length of the retrofitted section. It has been assumed that after post-tensioning, a pre-stress equal to 45% of the yield stress is present in the bars at the start of the analysis.


(a) Model 1

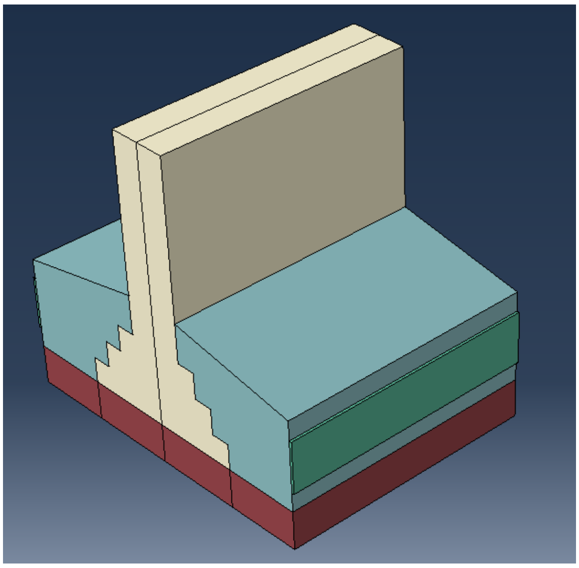
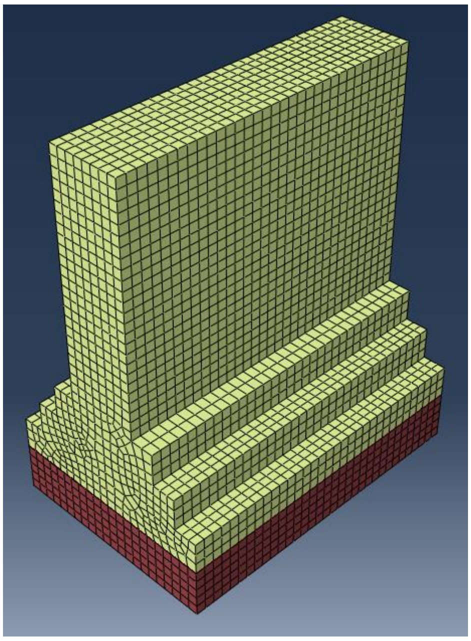
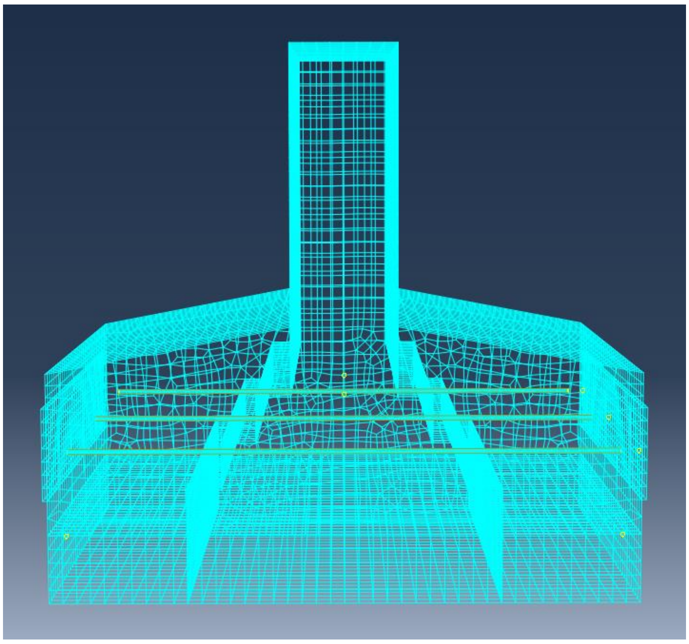

(b) Model 2

Fig.5. Finite Element Models of the Foundations



(a) Model 1



(b) Model 2

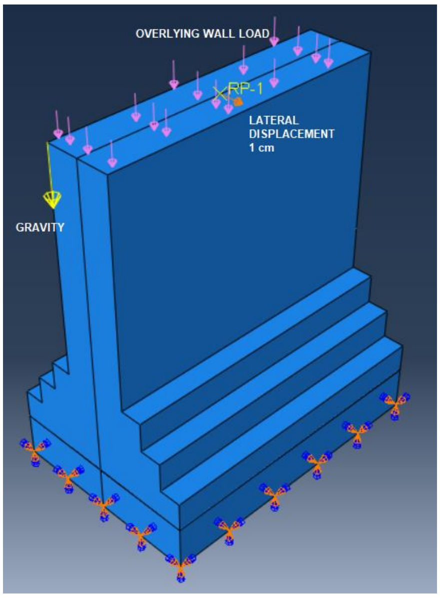
Fig.6. Meshed Foundation

6. Loads Applied

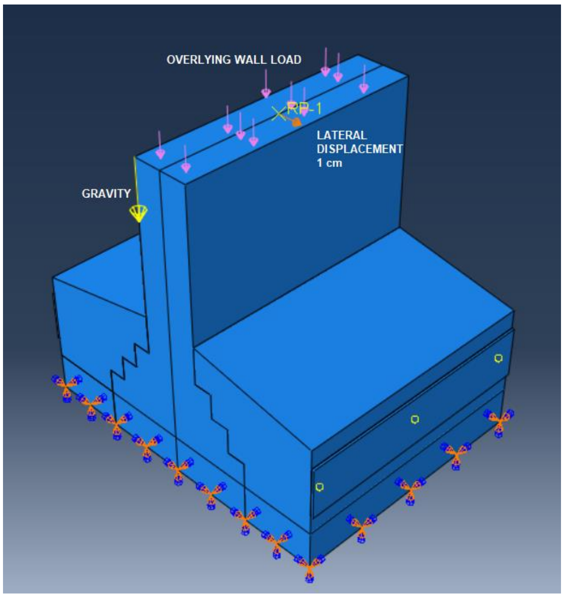
The following loads are applied to the models, as shown in Fig.7:

(1) Gravity load with an acceleration of 9.81 m/sec^2
(2) Superstructure double storied masonry wall pressure = 1920 x 9.81 x 6 x 10^{-6} = .113 MPa
(3) An out-of-plane displacement of 1 cm, applied at the mid-pt of the wall-top, in increasing steps.

There are two load cases run for the models. The first load case consists of Gravity load case, where the prestress also exists in the Model -2. The second load case consists of applying an out-of-plane displacement of 1 cm in increasing steps, applied at the mid-pt. of the top of the wall surface as shown in Fig.7.



(a) Model-1



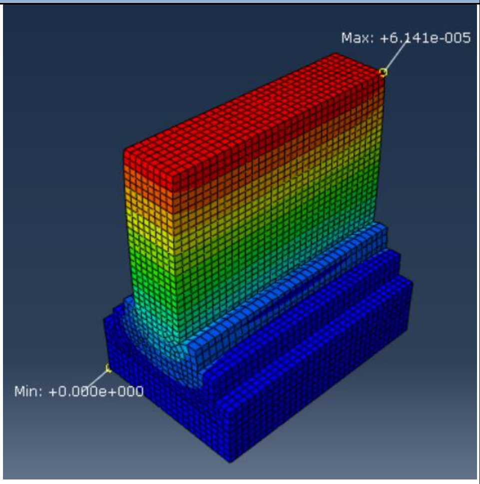
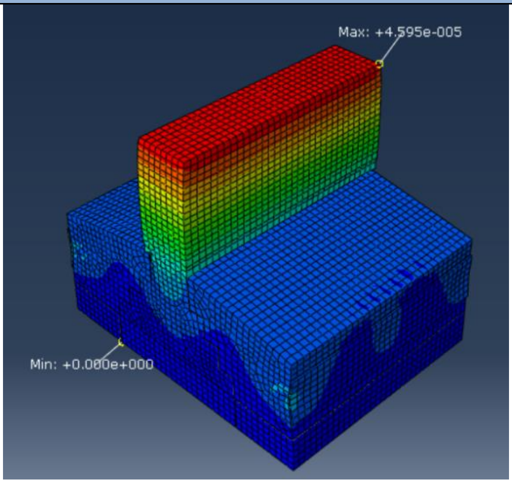
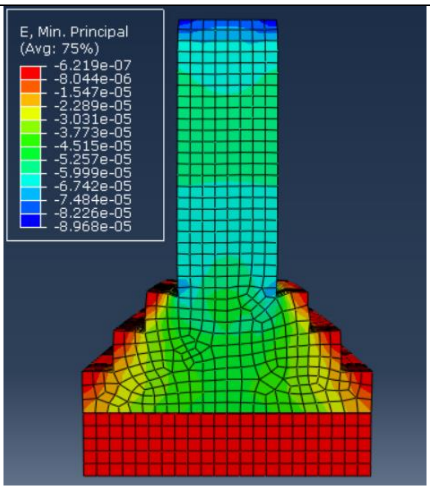
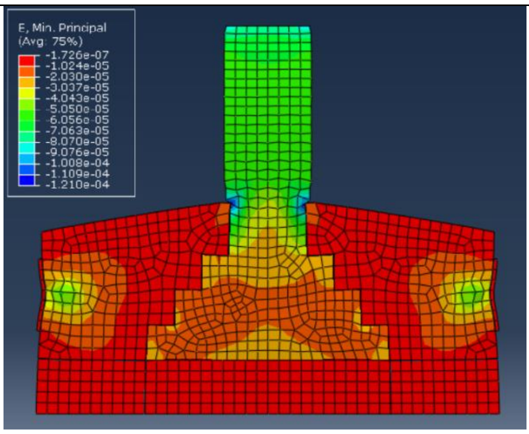
(b) Model-2

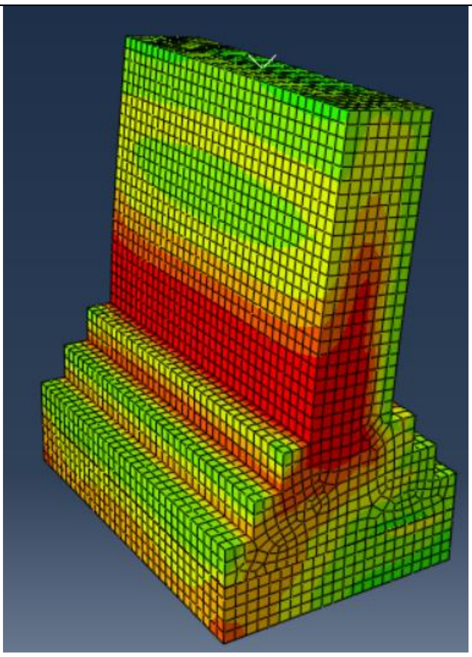
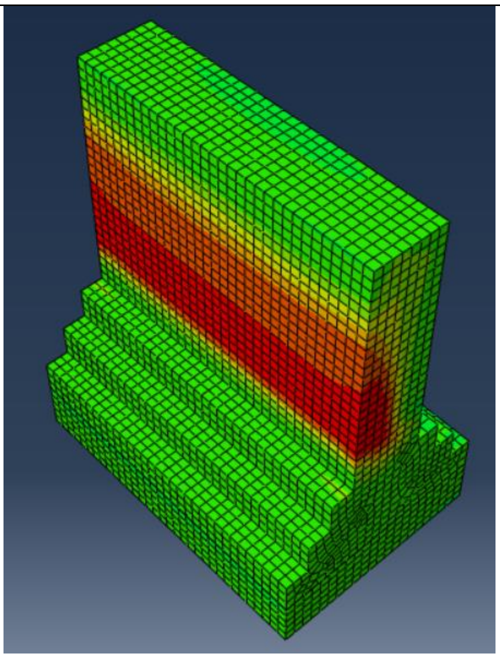
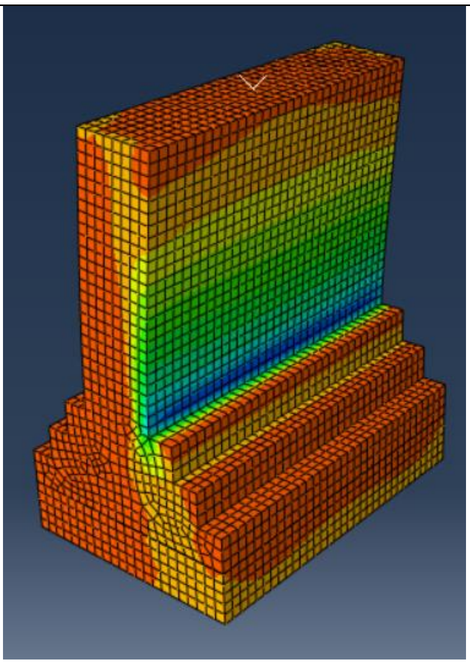
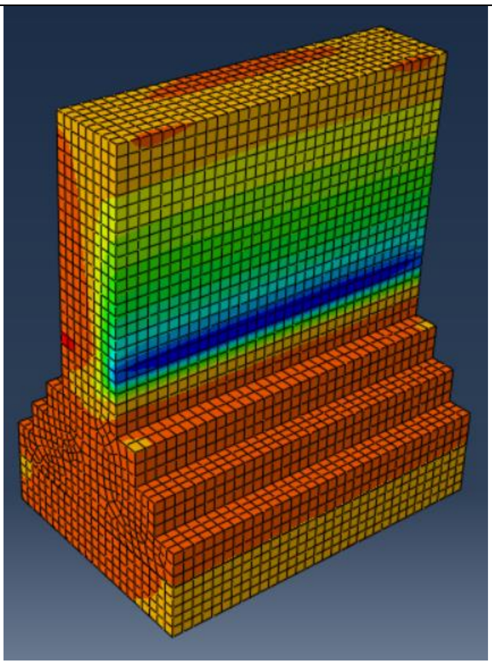
Fig.7. Load Cases applied

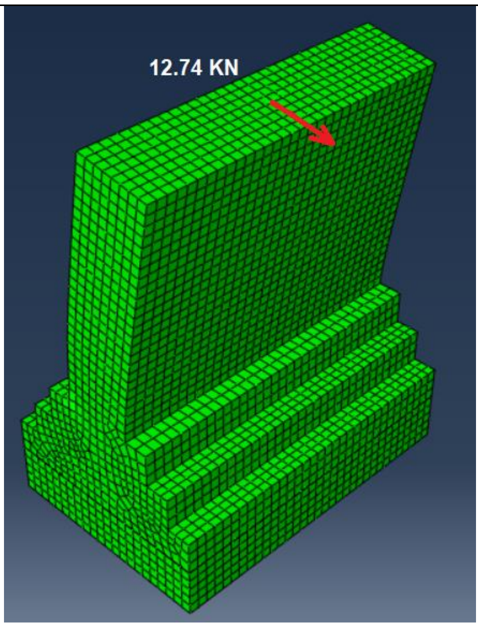
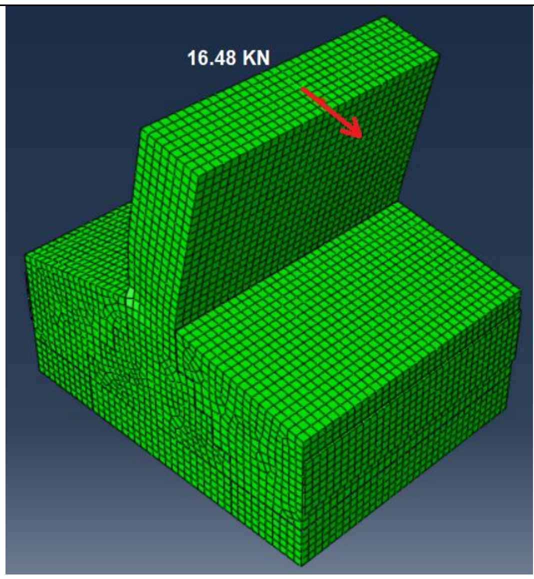
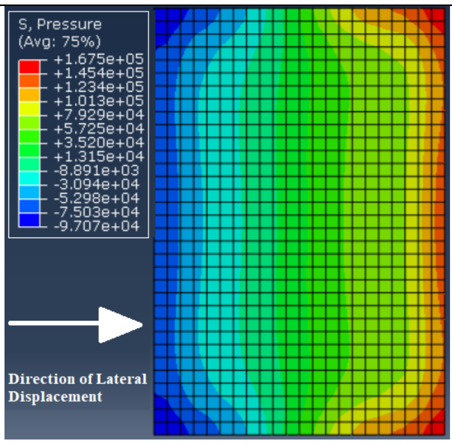
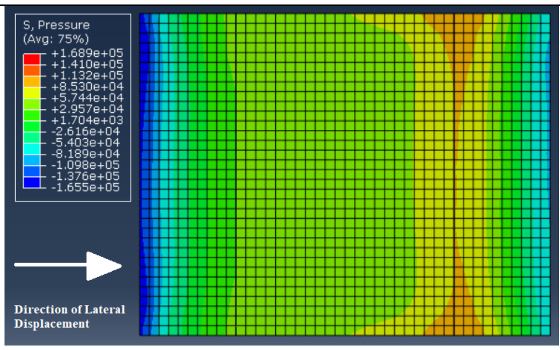
7. Results

Table 3 shows the figures of the response parameters (P) that are recorded after the analyses and Fig.8 shows the Lateral Load vs Displacement graph for both the models. The blue line signifies the behavior of the existing foundation and the red line signifies the behavior of the retrofitted foundation.

Table 3 – Responses recorded after the Analyses

S/N	P	Model-1	Model-2
1.	Deformation under Gravity		
2.	Gravity-Initial strain		

3.	Maximum principle stress in the masonry foundation @ first localized failure	 Tensile stress = .28 MPa	 Tensile stress = .28 MPa
4.	Minimum principle stress in the masonry foundation @ first localized failure	 Compressive stress = 2.627 MPa	 Compressive stress = 1.428 MPa

5.	Lateral Force @ 1 cm horizontal displacement out-of-plane		
6.	Bearing pressure on the base		

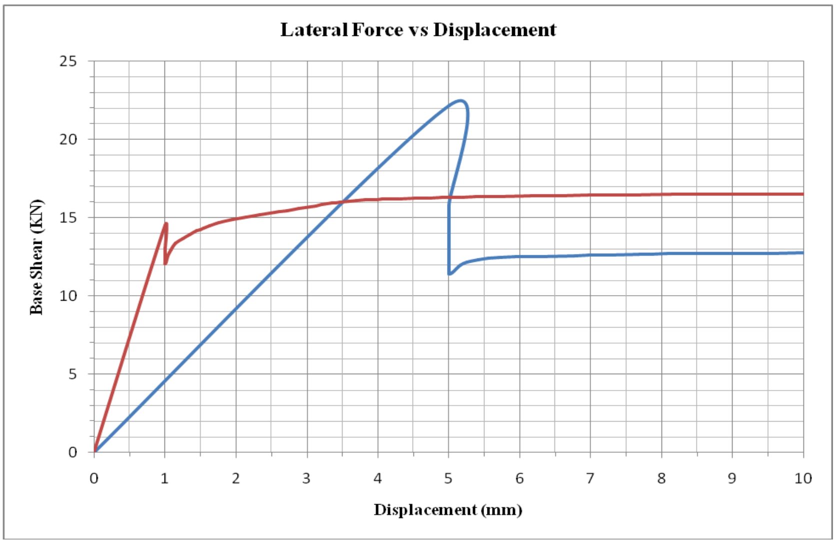


Fig. 8 Lateral Load vs Displacement graph for the foundations

8. Interpretation of results

It is seen from the graph, for lateral deformation upto approximately 3.5 mm, the force required to lateral displace the foundation system is more for the retrofitted section than the original section. Between 3.5 mm to 5 mm, the original section undergoes a peak lateral force of 22.166 KN followed by sudden drop at 5 mm, after which there is an increase in deformation without any significant increase in force. This indicates a localized failure of the structure. Even though, there is a recovery of strength after this failure, the residual strength of the structure is significantly less, which causes increase in deformation without much effort of the lateral load.

This same localized failure is noted for the retrofitted section @ 14.51 KN for a deformation of 1 mm. However, after the drop, unlike the original section, there is a steady increase in the lateral load to cause increasing lateral deformation. This behavior signifies, that there is a minor local failure in the retrofitted section at 1 mm, after which there is a steady recovery of strength till 5mm, after which there is an increase in deformation without much increase in lateral load.

It is noted from Row 3 of Table 3, that the localized failure occurs when the masonry portion of both the Model-1 and Model-2 reaches its tensile capacity of .28 MPa. At this state, the corresponding compressive stress of both the models is less than compressive strength of the masonry. However, the compressive stress in the existing foundation is more than that of the retrofitted section, as seen in Row 4 of Table 3. Therefore it can be inferred that the residual strength of masonry in the retrofitted section is more than of the existing foundation. The same phenomenon can be observed from Fig.8.

Between the two sections, the residual strength of the retrofitted section is more than the original section, as concluded from the comparatively greater lateral load value at the end of the analysis. It is also inferred that since the problem of the structured lies in excessive differential settlement, the retrofitted section being less deformable will perform better under earthquake.

Also as seen in Row 6 of Table 3, the maximum and minimum bearing pressure on the base of the foundation decreases when the original foundation is retrofitted. There is more uniformity in the bearing pressure distribution which will ultimately leads to reduced differential settlement in the event of earthquake.

9. Conclusion

From the results it can be seen there is a more equitable stress distribution in the retrofitted foundation section than the existing foundation system, which shall manifest in lesser differential settlement of the building in the event of an earthquake. However, the peak lateral load resisted by the existing foundation is seen to be more than the retrofitted section. But it is possible to improve the peak lateral load capacity of the retrofitted foundation by optimizing the various parameters of the section like amount of prestress, spacing of the post-tensioned bars and dimensions of the concrete fill, which forms the scope of future work.

10. Acknowledgement

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