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EXPERIENCE ON SEISMIC VULNERABILITY ASSESSMENT AND RETROFITTING OF SUPREME COURT BUILDING

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Abstract

The Supreme Court of Nepal is the highest court in Nepal. The Supreme Court of Nepal is an important building, built of brick in mud masonry structure and over 54 years of age which got moderate structural damage due to the recent Gorkha earthquake 2015. The need for safety of the building lying at high seismic zone in Nepal, the Seismic Vulnerability Assessment and Retrofit design was carried out to improve the building response in future earthquakes. The seismic vulnerability of the building was assessed after the following:(a) historical investigation about the building, (b) detailed geometrical investigation, (c) identification of materials by means of surveys and literature indications, (d) Detailed Intrusive Tests, (f) Detail linear static analysis of the building by means of a Finite Element (FE) model. After these steps, the FE model was used to assess the safety level of the building by means of linear static analyses and identifying a proper retrofitting strategy for this building. Both side wall jacketing and splint and bandage in some inner walls using the bar wire mesh was carried out for retrofitting this building.

Keywords: *Historical Building, seismic vulnerability, FE modeling, Intrusive test, Retrofit*

1. Introduction and Background

The Supreme Court of Nepal is the highest court in Nepal. The Supreme Court of Nepal is an important building, built of brick in mud masonry structure and over 54 years of age which got moderate structural damage due to the recent Gorkha earthquake 2015.

The detail seismic analysis was done based on the best engineering judgment arrived at from the site observation, destructive and non-destructive test carried out at site. Hilti PS 200 Ferroscan detector is used at few possible locations to identify the presence of lintel band on walls. All possible efforts have been made to provide an accurate and authoritative seismic vulnerability assessment and retrofit design of the building in the given circumstances of information provided by the client and limited number of field-tests. Therefore, the accuracy, completeness, or usefulness of the statements made is highly dependent on the accuracy of the information provided.

The detail seismic evaluation for retrofitting design, is carried out based on the first step evaluation of preliminary qualitative assessment of the building by the design team. If the qualitative approach identifies the seismic deficiencies in the building; and possible seismic performance is not up to the acceptable level/criteria, retrofitting design or demolition of the building is suggested. The second step involves the detail seismic evaluation followed by design for seismic strengthening measures as modifications to correct/ reduce seismic deficiencies identified during the evaluation procedure in first step. This is commonly known as seismic retrofitting of the building. Seismic retrofit becomes necessary if the building does not meet minimum requirements of the current Building Code and may suffer severe damage or even collapse during a seismic event [1].

2. Assessment of the Building

The Supreme Court Building is a four-storey brick in mud masonry building constructed on 2019 B.S. The building is almost of E shape, however in the east portion, small wing is projected in the middle. The floor of the building is rigid with reinforced concrete slab and the roof is flexible with CGI sheet covering as well as tile works. The building has many openings and there are very small piers in between openings. The summary of building is given below Table 1.

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Table 1:Summary of Building Description

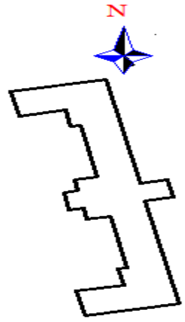
<table>
<tr><td>Building Name</td><td>The Supreme Court of Nepal</td><td rowspan="7">
Figure 2.a) South view of the building</td></tr>
<tr><td>Location</td><td>Ramshah Path, Kathmandu</td></tr>
<tr><td>G.P. S</td><td>27° 41’ 48.67” N, 85° 19' 18.65" E</td></tr>
<tr><td>Terrain type</td><td>Plain Land</td></tr>
<tr><td>Age of the building</td><td>Building was constructed in 2019 B.S.</td></tr>
<tr><td>Type of structure</td><td>Brick in Mud Masonry Building</td></tr>
<tr><td>No of stories</td><td>Four Storied</td></tr>
<tr><td>Plan configuration</td><td>Irregular (Nearly E-shaped in plan with slight projection at back side)</td><td rowspan="5">
Figure 2.b) West View of the Building</td></tr>
<tr><td>Vertical configuration</td><td>Regular</td></tr>
<tr><td>Position of the building block</td><td>Attached with other buildings with northeast corner and southeast corner</td></tr>
<tr><td>Building dimension</td><td>Refer attached drawing</td></tr>
<tr><td>Total Plinth Area</td><td>1610 sq. meters</td></tr>
<tr><td>Storey height</td><td>3.5 m Ground floor and Second floor; 3.25 m first floor; and 3.1 m at the top floor on average</td><td rowspan="6">

Figure 2.c) Building plan</td></tr>
<tr><td>Wall thickness</td><td>All peripheral and main load bearing walls are 530 mm thick however there are walls at few locations having thickness 250mm and 115mm.</td></tr>
<tr><td>Building condition</td><td>Damaged and not occupied after 25th April 2015 Gorkha Earthquake</td></tr>
<tr><td>Floor structure</td><td>Reinforced Concrete Slab with 150mm thick</td></tr>
<tr><td>Roof Structure</td><td>CGI Sheet on steel truss, Clay tiles on steel truss and RC slab at some portion</td></tr>
<tr><td>Local hazard</td><td>No possibilities of rock fall on the site.
Not build on infill soil.</td></tr>
</table>

2.1 Damage identification due to Gorkha Earthquake

The first site survey was done on 10th June 2015 and there have been frequent visits after that to prepare the as built drawings as well as to identify and locate cracks/damages in the building. It has been identified that the building has visible cracks in the periphery walls. Most of the peripheral walls have gone diagonal cracking due to in plane action of earthquake forces. Some of the walls have deep vertical and horizontal cracks as well. There are no visible problems of settlement, tilting and cracking in the foundation. There is also a markedly visible deep crack almost along the middle length to the full height of the building. This crack extends from one face to its opposite face (front face to back face). No falling hazard was seen in the site.

Internal walls have also suffered minor to moderate cracks at different locations. At few locations, there is very deep cracks in the slab which needs serious attention. Slab at these portions have gone cracking wider than 10 mm and reinforcements have buckled. The damage picture due to Gorkha Earthquake are shown below in Fig. 2.

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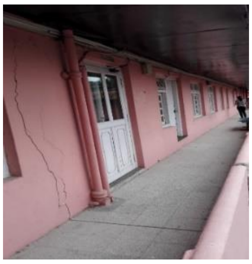
(a)

(b)

(c)

(d)

(e)



(f)

Figure 2. a) Heavy vertical crack in the pier; b) Vertical crack in the pier; c) Deep crack in the first floor slab;d) Crack in the slab; e) Crack width of about 20 mm in the wall; f)Vertical crack in the Wall

2.2 Field Investigation

In Situ In-plane Shear Test

Reliable information on shear resistance is needed when performing retrofits and seismic upgrades of masonry buildings. The shear strength of a masonry wall is difficult to measure without resorting to large-scale testing. We cannot carryout destructive tests for evaluating the shear strength of the whole masonry wall of existing buildings. As an alternative, less destructive in-situ tests of single masonry units provide a comparative figure that can be correlated to full-scale wall behaviour. This less destructive alternative is more economical than large-scale testing and is desirable when a building's historic integrity must be maintained.

The in-situ shear test is also known as the push test. It provides a direct measurement of the shear resistance of mortar joints in masonry. The test is suitable for masonry that has relatively strong units and weak mortar so that shear cracks form in the typical stair step pattern along mortar joints and the units remain un-cracked. In this type of construction, the shear strength of the mortar joints limits the shear strength of the masonry wall. Five test locations were selected based on internal and external locations. The test was carried out at 3 locations on the ground floor and 2 locations at first floor which is shown in Fig.3a).

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Figure 3 a) Conducting In-situ Shear Test on Wall

The test locations were prepared by removing the brick, including the mortar on one side of the brick to be tested. The head joint on the opposite side of the brick to be tested was also removed. This was done with caution that the mortar joint above or below the brick to be tested is not damaged. The hydraulic ram was inserted in the space where the brick was removed. A steel loading block was placed between the ram and the brick to be tested so that the ram will distribute its load over the end face of the brick. The dial gauge was inserted in the space. The brick was then loaded with the ram until the first indication of cracking or movement of the brick. The ram force and associated deflection on the dial gage were recorded. From the observation, final corrected shear strength of brick masonry is obtained as per ASTM standard and IITK- GSDMA Guideline and the corrected minimum shear strength obtained are 0.054 N/mm2 and 0.036 N/mm2 respectively. The difference in these values from two standards, ASTM[8] and IITK-GSDMA Guideline-EQ06[2], is due to the coefficient of friction between the brick and mortar is assumed as per the site conditions. Being on the conservative side, the value obtained from the GSDMA-EQ6 is used for checking the shear strength capacity.

Table 2 –Calculation of Shear Strength of Masonry Walls from Direct Shear Test

Shear test Number	**Coeff.of friction**	**Overburden pressure**	**Shear Strength for Sample**	**Corrected Shear strength (N/mm2)**	
	μ	p(N/mm2)		Va=Vte-μ*p	Va= 0.1Vte+0.15Pce/Ah
IP: ST1	0.8	0.1966	0.6437	0.486	0.093857308
IP: ST2	0.8	0.2091	0.4506	0.283	0.076428176
IP: ST3	0.8	0.1985	0.4828	0.324	0.078059031
IP: ST4	0.8	0.1339	0.1609	0.054	0.036172477
IP: ST5	0.8	0.1421	0.2897	0.176	0.050281099
Minimum Corrected Shear Strength				0.054	0.036

Brick Unit Test

Four brick samples were taken from the building wall and compressive strength and water absorption tests were carried out at Central Material Testing Laboratory (Pulchowk Campus), Institute of Engineering which is shown in Fig.3b). The average breaking strength of the three brick samples tested is was 42.51 kg/cm2 (4.17 Mpa) which indicates low strength brick. Water absorption of the three samples are 28.22%, 26.34% and 24.6%. As per the Nepal National Building Code (NBC 109), a first-class hand-made brick shall not absorb more water than 25% of its weight. The brick falls to lower category of second class [9].

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Figure 3 b) Compressive Strength Test of Brick Unit performed at CMTL

Direct shear test of soil

Soil samples were taken from two locations, ground excavated for foundation exploration (at the premise of Supreme Court building) to determine the shear strength of soil which is shown in Fig.3c). The cohesion (c) and angle of friction (φ) of the soil below 1m from the existing ground level is found in the range of 0.1 to 0.13 kg/cm2 and 24.130 to 27.140.

Figure 3 c) Soil Sampling for direct shear test of soil

Foundation Inspection

To explore the foundation details of the building, excavation was carried out at three different locations; one at the North West wing (front face) of the building, another at the North-East (back face) and the other at the East (back face) of the building. The details of the foundations is shown in Figure 3 d) and e).The building has strip footings made of brick masonry. The depth of the foundation is same at all locations. Total, depth of strip footing is 4 feet and the width is around 40 inches. There is no plinth band in the walls.

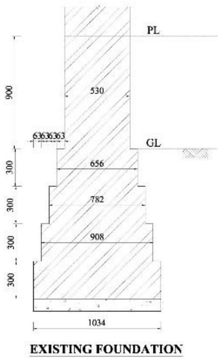

Figure 3 d) Foundation Section of Supreme Court building

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Figure 3 e) Foundation Investigation

Investigation of the Brick Masonry Wall

Status of bricks, mortar and lay pattern is important parameter to determine the status of the building. For such purpose, opening of size 450mm x 450mm x450mm were created at two places and observations were recorded. The wall is constructed using burnt clay brick with mud mortar (mixed with bajra, chaku, mash, chun). Average thickness of mortar is 10 mm.All bricks are laid properly with offset along the length and breadth of the wall using an English bond. Brick Lay Pattern in Wall is shown in Fig 3 f)

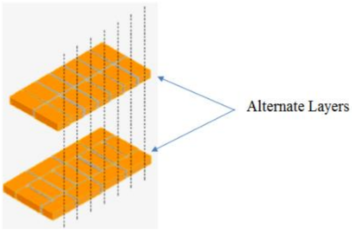

Figure 3 f) Brick Lay Pattern in Wall

2.2 Seismic intervention options for the building under study

The possible intervention options are selected based on the building typology and the expected performance of the building after retrofitting. The best applicable intervention options that are available are selected.

Following retrofitting strategy are adopted in this building:

- Some wing walls are added at strategic locations
- Both side wall jacketing and bandage in some inner walls of wall thickness 4” to provide integrity.
- Vulnerable half brick walls built out of main grid system are tied up.

3. Detailed structural analysis

Finite Element Modelling of the building of Supreme is done by using the structural analysis and design software program ETABS 2015. For the analysis of the system, whole building is modelled. Load bearing brick masonry walls and RC floor slabs are modelled as single layered shell elements. Since the roofing of the building is made of clay tiles in truss, tying element as beams are modelled.

Seismic coefficient method is used to analyse the building. Indian Seismic Code IS 1893:2002 is used for the lateral load calculations and the seismic coefficient value is also compared with the Nepal National Building Code NBC 105:1994. The building plan is shown is Figure 6 while 3D view of the analytical model is shown in Figure 4 a) and b).

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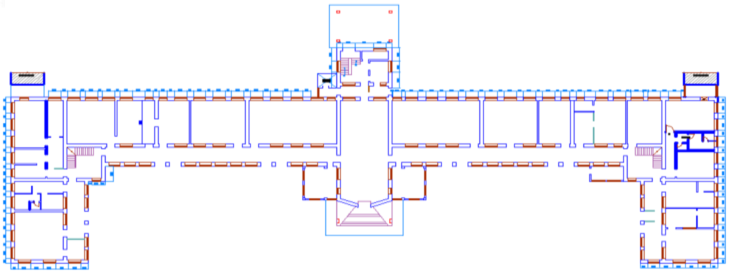

Figure 4 a) Plan of the Building

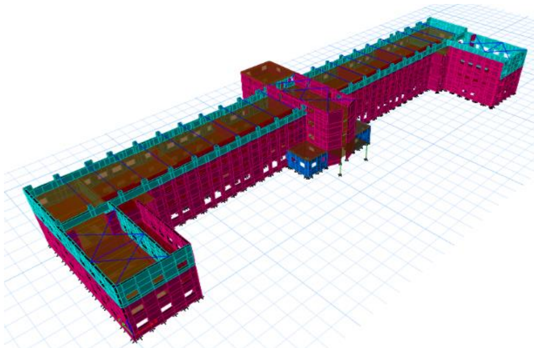

Figure 4 b) 3 D Analytical Model of the Supreme Court

2.2 Seismic analysis

The seismic analysis is a part of the detailed evaluation of an existing building. The steps involve in developing a computational model of the building include applying the external forces, calculating the internal forces in the members of the building, identifying deformations and capacity of the members and building, and finally interpreting the results. The structural analysis is carried out with the help of the available drawings and ETABS 2013, a structural analysis and design software. IS 1893:2002; criteria for earthquake resistant design of structures is used to determine the base shear in the building.

The modulus of elasticity is calculated using different formulas and codal provision from measuring the compressive strength of brick (fb') and weakest mortar suggested in NBC109. Many researchers (Deodhar 2000; Gumaste et al. 2006; Kaushik et al. 2007a; Kaushik et al. 2007b) have attempted to develop an empirical expression relating the brick unit, mortar and masonry compressive strengths as shown in Equation 1 (CEN 2005).

$$f'_m = K f'^{\alpha}_b \times f'^{\beta}_j \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \ (1)$$

Where:

f'_m is the characteristic compressive strength of masonry, in N/mm^2

K is a constant

α β are constant

f'_b is the normalized mean compressive strength of the units, in the direction of the applied action effect, in N/mm^2

f'_j is the compressive strength of the mortar, in N/mm^2

Modulus of Elasticity(Em) =550f'_m ………………………………(2)[From FEMA 306 (1999)-for existing masonry]

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Table 5 –The modulus of elasticity is calculated using different formula and codal provision

Source	K	α	β	**Compressive strength of Brick (f'b) form test(N/mm2)**	f'_m	**Modulus of Elasticity E(N/mm^2)**
Eurocode-6	0.5	0.65	0.25	4.17	1.064	585
Stress-Strain Characteristics of Clay Brick Masonry under Uniaxial Compression(Hemant B. Kaushik1; Durgesh C. Rai2; and Sudhir K. Jain, M.ASCE)	0.63	0.49	0.32	4.17	1.016	559

During the calculation from the empirical formulas, the mean compressive strength of unit is taken from the brick test result, 4.17 N/mm^2 and the weakest mortar suggested in NBC 109 having a compressive strength 0.5 N/mm^2. Value of compressive strength of masonry and modulus of elasticity of masonry obtained from the test results are used for the detailed evaluation of the building. The value of modulus of elasticity use during analysis is given below in Table 6.

Table 6-Element type and material properties used in the FE model

Structural Member	Etabs Element	Density (kg/m^3)	Modulus of Elasticity E(N/mm^2)	Poisson's Ratio
Wall	Shell	1900	585	0.1

In analysis we have taken Permissible Compressive Stress as 0.6 N/mm^2 considering the masonry wall confined by reinforced concrete elements on both sides.

2.3 Modelling output for existing building

Initially, the existing building is modeled and in-plane stresses along with out-of-plane moments are studied. The in-plane stress and moment diagrams obtained from analysis are shown in Fig.5 a),b) and c) below.

In plane Stress

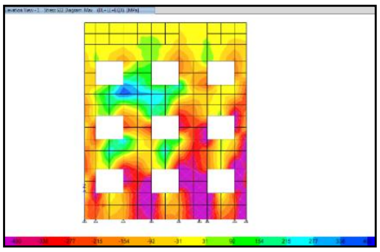

Figure 5 a) Compressive/Tensile stress (In plane stress (S22)) (X-direction: Grid 1-1: wall X1)

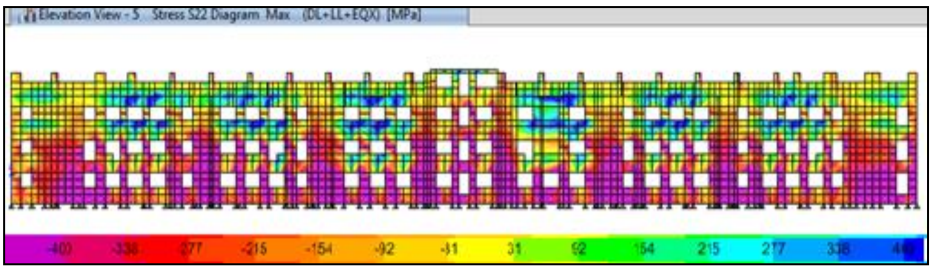

Figure 5 b) Compressive/Tensile stress (In plane stress (S22)) (X-direction: Grid 5-5: wall X5)

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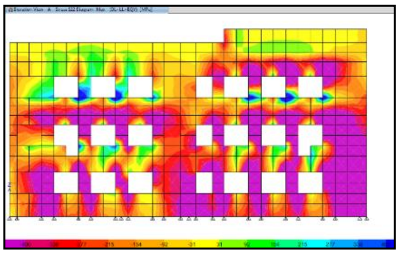

Figure 5 c) Compressive stress in wall (In plane stress (S22)) (Y-direction-A-A)

Table 7-Tensile and compressive stress in wall (In plane bending)
Walls along X/Y direction

Wall	**Stress**	**Vertical Stress S22 (average)**			
		0.7DL+EQx	**0.7DL-EQx**	**DL+LL+EQx**	**DL+LL-EQx**
		Stress	**Stress**	**Stress**	**Stress**
		N/mm^2	**N/mm^2**	**N/mm^2**	**N/mm^2**
X1	Tension	0.160	0.100	0.16	0.12
	Comp.	0.520	0.600	0.7	0.72
X4	Tension	0.170	0.190	0.28	0.27
	Comp.	0.610	0.640	0.91	0.86
X5	Tension	0.440	0.120	0.68	0.65
	Comp.	0.750	0.830	1.00	0.91
Wall	Stress	Vertical Stress S22 (average)			
		0.7DL+EQy	0.7DL-EQy	DL+LL+EQy	DL+LL-EQy
		Stress	Stress	Stress	Stress
		N/mm^2	N/mm^2	N/mm^2	N/mm^2
YA	Tension	0.480	0.480	0.68	0.65
	Comp.	0.690	0.650	0.94	0.86
YB	Tension	0.410	0.270	0.35	0.16
	Comp.	0.640	0.250	0.72	0.95
YE	Tension	0.350	0.130	0.53	0.49
	Comp.	0.860	0.680	0.85	1.01

Out of plane bending (horizontal bending):Show in Fig.5d),e)

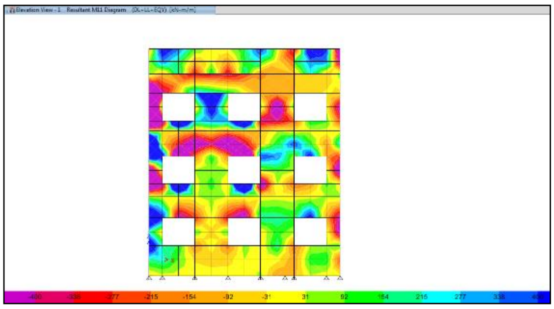

Figure 5 d) Moment Diagram (M11) for Grid 1-1

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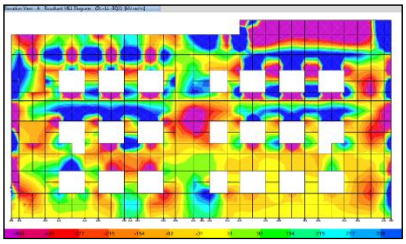

Figure 5 e) Moment Diagram M11 Obtained for Grid A-A

Table 8- Out of plane horizontal bending moment (M11)
Walls along X/Y direction

Walls	**Moments M11 in kNm/m (average)**				
	0.7 DL+ EQy	**DL+ LL+ EQy**	**0.7 DL- EQy**	**DL+ LL- EQy**	**Max.**
X1	0.835	1.506	1.573	2.478	2.478
X3	3.090	3.800	2.105	3.115	3.800
X4	1.270	1.579	0.916	0.981	1.579

Walls	**Moments M11 in kNm/m (average)**				
	0.7 DL+ EQx	**DL+ LL+ EQx**	**0.7 DL- EQx**	**DL+ LL- EQx**	**Max.**
YA	2.646	3.712	1.791	1.978	3.712
YE	6.815	7.415	4.598	5.947	7.415
YF	6.225	6.024	6.225	7.356	7.356

Out of plane bending (Vertical bending):Shown in Fig.5 f) and g)

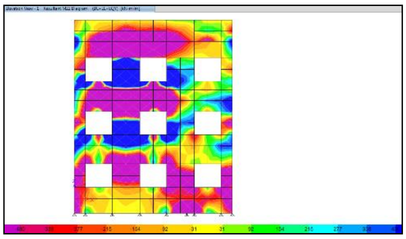

Figure 5 f) Moment Diagram (M22) for Grid 1-1

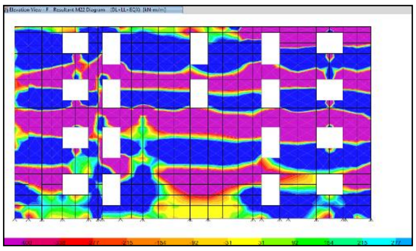

Figure 5 g) Moment Diagram (M22) for Grid F-F

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Table 2: Out of plane Vertical bending moment (M22)
Wall along X/Y direction

Walls	**Moments M22 in kNm/m (average)**				
	0.7 DL+ EQy	**DL+ LL+ EQy**	**0.7 DL- EQy**	**DL+ LL- EQy**	**Max.**
	kNm/m	**kNm/m**	**kNm/m**	**kNm/m**	
X1	1.746	2.404	2.350	3.879	3.879
X3	8.255	9.800	13.295	14.804	14.804
X4	2.980	3.30	3.848	3.811	3.848
Walls	**Moments M22 in kNm/m (average)**				
	0.7 DL+ EQx	**DL+ LL+ EQx**	**0.7 DL- EQx**	**DL+ LL- EQx**	**Max.**
YA	6.229	7.209	4.654	3.674	7.209
YE	3.664	5.608	6.673	6.876	6.876
YF	6.635	8.468	3.434	6.528	8.468

Summary of retrofit design

Modifications	Addition of wing walls Closing of openings Tying of walls outside the main grid
Retrofitting of walls	a) Steel Jacketing with 4.75 mm Ø bar @ 250 mm c/c in horizontal direction. b) Steel Jacketing with 8 mm Ø bar @ 250 mm c/c in vertical direction c) Steel Jacketing with 8 mm Ø bar @ 200 mm c/c in vertical direction
Foundation	Addition of tie beam 300 mm x 450 mm along all main walls to connect with the bars of jacketing. Addition of a tying beam 300 mm x 450 mm inserted to a length of 300 mm at an interval of 1.5 m all around the peripheral tie beams.

3. Implementation of Retrofit options

Implementation of this building was carried out which above given options. Removal of plaster in required locations, dismantling of brick work, bar mesh splint and bandage on inner and bar mesh jacketing on outer walls, micro concreting, curing, plastering were done. Few photographs of accomplished retrofit work are presented in the Fig.6 below.

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Figure 6 Implementation of Retrofit Option

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