International Journal of Engineering and Advanced Technology (IJEAT)
ISSN: 2249 – 8958, Volume-3 Issue-5, June 2014

Retrofitting of a Damaged School Building: A Case Study

Shujaat Hussain Buch, Muhammad Dilawar Bhat

Abstract— A three story damaged school building which has developed multiple cracks in floor slabs is investigated and analyzed. The building vertical load paths are determined and failure patterns studied. The retrofitting (strengthening) measures are worked out. The building is provided with suitable strengthening features to limit the damage and prevent future damages.

Index Terms— Cracks, retrofit, strengthening, yield-line.

I. INTRODUCTION

The large numbers of school buildings particularly in India are constructed of brick masonry and unfortunately many of them are non-engineered structures and typical representative of traditional construction, as a result of which many of them are vulnerable to some serious kind of damage particularly in case of earthquakes. Keeping in view these facts, it was decided to evaluate and rectify a school building with structural deficiencies and fortunately we were able to locate one such school. The school building is located 7 km's from Srinagar city center. It is a three story load bearing masonry structure with an overall floor area of 131.6 m^2. It is 10 years old construction. The building is complex with RCC slabs at both levels with many overhanging projections. Though the school building looks safe from outside, but the cracks that were described by the owner and later on observed during the inspection compels for thorough evaluation and immediate retrofitting. Besides this, the building has many props that were installed after the construction. These props are mainly provided under the cantilever beams and overhang projections resulting in conversion of member from cantilever to simply supported, leading to reversal of stresses. The plan of the school building is also irregular. The school consists of large openings and during inspection many structural cracks were found at overhang projections that hint towards inadequate negative reinforcement at supports. Many other structural checks were performed that determined analysis and retrofitting of the building.

II. METHODOLOGY

The methodology of evaluating the un-reinforced masonry buildings is described by FEMA 307 [1].

1. *Inspection* is done by visual examination of the building, and the overall information about the structural system is obtained and possible errors regarding the structural layout construction and maintenance are identified. The condition of the structural and non-structural elements is verified and possible damage documented and categorized.

Manuscript Received on June 2014.

Shujaat Hussain Buch, Civil Engineering, Islamic University of Science and Technology, Srinagar, India.

Muhammad Dilawar Bhat, Engineer, Sarva-Shiksha-Abhiyan, Srinagar, India.

2. *Monitoring* is done in case where the cause of damage, observed during visual inspection of the building are not evident, long term observations of the building's behavior are many times needed to know the actual reason, not all observed damage can be attributed to a single cause. For this purpose the structure is instrumented with displacement strain and vibration transducers and used to monitor the dynamic effects of structure. Settlement and tilting of the structure are measured with geodetic methods. The closings of the cracks are measured with deformeters; whereas velocity transducers are in most cases used for monitoring the dynamic effects.

3. *Analysis is done*- Evaluation and the analysis are started adopting the suitable methods. The various analytical methods available for evaluation of masonry structures are mainly governed by the masonry design codes IS: 1905 and SP: 20 the two BIS (Bureau of Indian standards). FEMA- 232 is a beautiful illustration of Homebuilders guide [2].

a. Vertical load on walls at various walls is calculated first. If load on the wall at level 1 is 'w_1' then pressure on solid masonry wall 'p_1' at various levels for a thickness of 't' is given by:

$$p_1 = {}^{w_1}/_{tx1} \tag{1}$$

Pressures at various levels are:

$$\sum p = {}^{w_1}/_{t\,x\,1} + {}^{w_2}/_{t\,x\,1} + \cdots \tag{2}$$

b. Horizontal load analysis is performed for earthquake load by equivalent static method adopted by IS-1893 [3]; whereby, base shear 'V_B' is

$$V_B = \frac{Z \times I \times S_a}{2 \times R \times g} \tag{3}$$

Z = Zone Factor, I = Importance Factor, S_a/g = Acceleration coefficient, R= Reduction Factor.

Lateral Load distribution 'Q_i' is given by:

$$Q_i = V_B \times \left[\frac{W_i H_i^2}{\sum W_i H_i^2}\right] \tag{4}$$

W_i = Seismic Weight of story 'i', H_i = Story 'i' Height.

c. Slab Analysis is performed by 'Yield line Analysis', which is based on the external energy expended is equal to internal energy dissipated. Ultimate moment along the yield line for slabs is 'm:

$$m = \frac{nL^2}{2\left(\sqrt{(1+i_1)} + \sqrt{(1+i_2)}\right)^2} \tag{5}$$

n = ultimate load on slab, L= span, i_1 and i_2 = ratios of supports to mid-span moments in two directions for one-way slab.

For two way slab, 'm' is:

Published By:
Blue Eyes Intelligence Engineering & Sciences Publication Pvt. Ltd.

$$m = \frac{n\,a_r^2}{24} \times \left[\sqrt{3 + \left(\frac{a_r^2}{b_r^2}\right)} - \frac{a_r}{b_r}\right]^2 \qquad (6)$$

'a_r' & 'b_r'= side length of slabs.

4. *Non-Destructive testing*- Radar tomography or impact echo testing and reinforcement detector is used to detect the reinforcement and qualitative data regarding the general structure of the masonry walls.

5. *Retrofitting*- This is strengthening of the walls and the floors and their connections against lateral loads. Lateral load resistance is generally improved, continuity introduced in connections, weakness removed and brittle failure avoided. Cracks can be rectified by use of epoxy and wire meshing generally. However, local modification in walls and openings and global modification in symmetry of plan is also done to strengthen the buildings. Besides, the connections are modified by proper anchoring, new walls added and old strengthened by repair and restoration.

III. CASE

A. Top floor plan of the load bearing masonry building is given in Fig. 1. Fig. 2 and Fig. 3 depict the first floor and ground floor respectively. The preliminary investigation states following problems: a. Slab cracks in Top and first Floor (Fig. 4a), b. some minor wall cracks. This is judged due to a. Probability of less reinforcement than required in slabs, b. Overhead projection without proper structural action (Fig. 4b). So the analysis is broken into wall analysis and slab analysis.

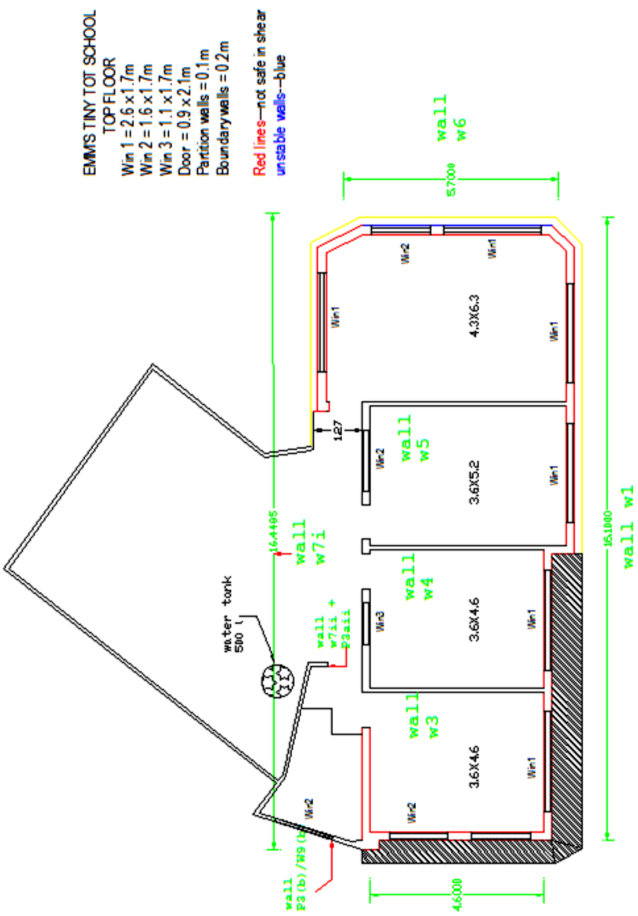

Fig. 1 Plan of Top Floor and Unstable Walls in Blue

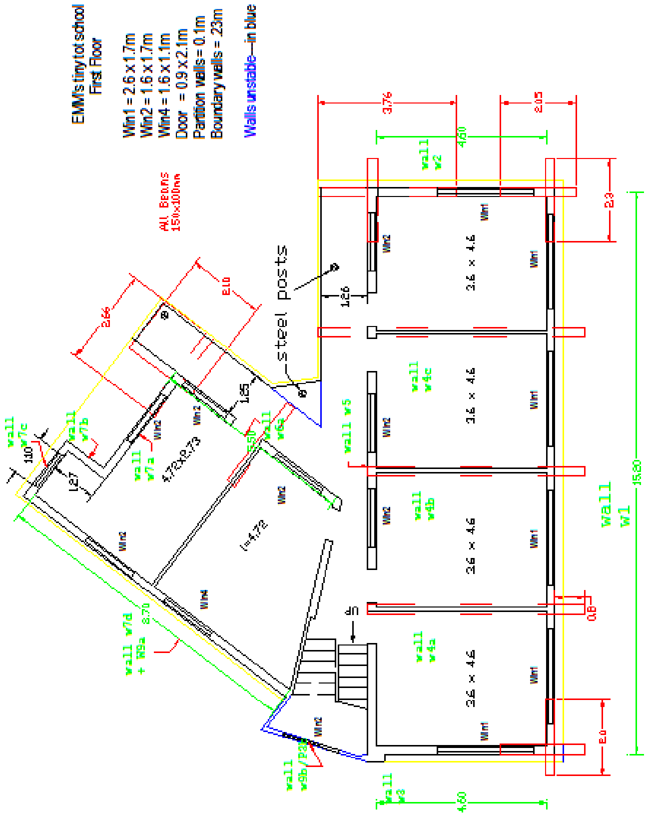

Fig. 2 Plan of First Floor and Unstable Walls in Blue

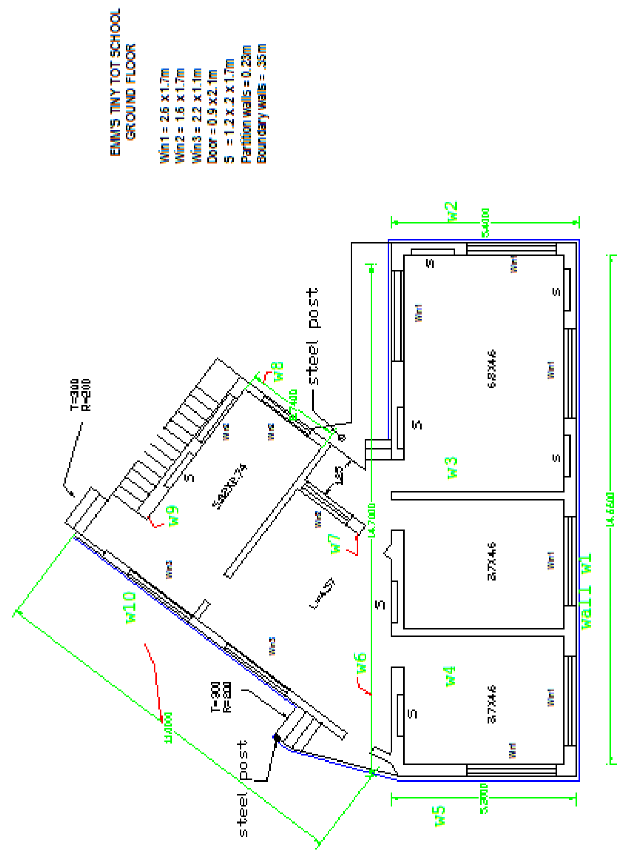

Fig. 3 Plan of Ground Floor and unstable walls in blue

Analysis for walls is performed (Stability, Earthquake considerations, compression, tension and shear analysis) and checked for failure under vertical loads, the walls which are

Published By:
Blue Eyes Intelligence Engineering
& Sciences Publication Pvt. Ltd.

A.Cracks in Slab B. Overhead Projections

Fig. 4

in red in Fig. 1 for Top floor, are susceptible to such failure and walls in blue are unstable. Similar is the case depicted in Fig. 2 and 3. From above, it is found that only 7% of walls are unsafe for vertical and lateral loads; however, majority of them don't meet the earthquake considerations. Further, about 40% of walls in Top story are liable to shear failure.

B. The slab analysis is performed by breaking the slabs into individual panels and implementing yield lines [4]. For each panel, the loaded moments are calculated (Fig. 5 and Fig. 6) and reinforcement required is determined. The reinforcement provided is checked by bar detector (Fig. 7). Thereby flexural check is performed. Besides, the shear check and deflection check is performed to gain additional information. For top slab given in Fig. 5, the panel 6 is failing in flexure while as for first slab given in Fig. 6, panel H, I, J are failing in flexure and shear after check. It is seen that overall bearing strength of soil is less than 50T/m^2 and is safe in bearing strength.

C. Retrofitting- The walls that are not safe in shear are recommended to be provided with bands that give particular shear strength. This Lintel band is of dimensions 10cm depth x width 23cm x full length of wall is provided as per IS: 1905 in 15% of the walls as wall shear strengthener [5].

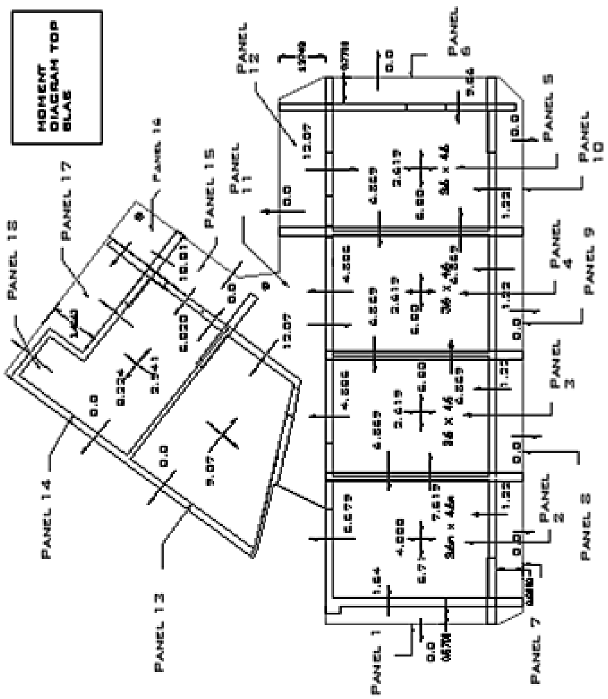

Fig. 5 Moments calculated by 'Yield Line Analysis' Top Slab

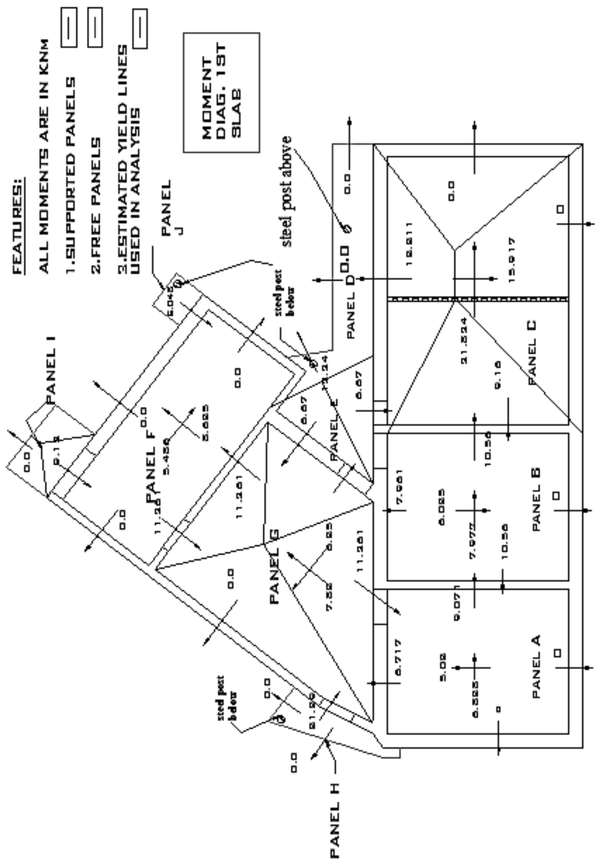

Fig. 6 Moments calculated by 'Yield Line Analysis' Bottom Slab

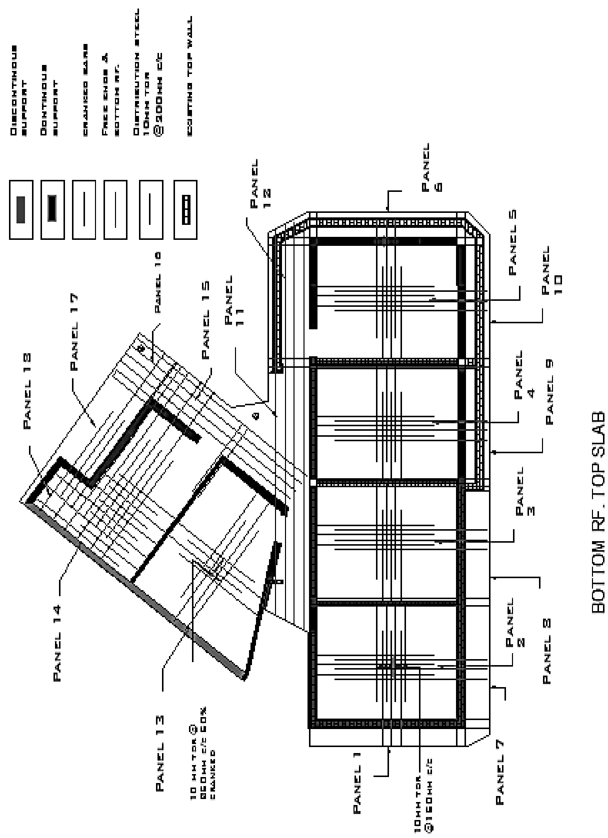

Fig. 7 Bottom Reinforcement Provided in Top Floor Slab

In 14% of the walls, corner strengthening is provided as corner reinforcement of 12mm dia. Fe415. Besides, in 65% of walls the opening stiffener is provided as Jambs of 8mm

Published By:
Blue Eyes Intelligence Engineering
& Sciences Publication Pvt. Ltd.

dia. Fe415. FEMA 547 is a commentary on rehabilitation techniques to be used in buildings [6].
The study of the Slab cracks is done in done. This is done in following stages:
1. Pre-Retrofitting analysis
2. Retrofitting and analysis
3. Post-Retrofitting Check
Various structural problems are discussed:
1. *Panel 'J' of First Slab* above ground floor has developed torsional cracks as in Fig. 6 in previous page. On preliminary investigation, it is judged that this is due to excessive unsymmetrical loading through a prop above the slab as shown in Fig. 7. The loading generated on the slab panel is shown in Fig. 8 below. It is found that the reinforcement provided is enough to support the panel, however there is eccentric load from above leading to torsion. Remedy suggested is a prop below at end of landing and provision of an angle section on two sides around the prop to a considerable distance, acting composite with the slab at the newly introduced slab. Due to this remedy, the moments in the slab are re-distributed by 5% leading to decrease in torsion of slab. Besides there is no need of any further bottom reinforcement along any direction; as already compensated by distribution steel. There is no need for any additional top reinforcement along x-direction (Fig. 7) as the moments are transferred to the composite angle sections provided. Also, there is no need for any additional top reinforcement along y-direction as moments is resisted well by the existing reinforcement. The prop is designed for base shear load of 107.38 KN and a rolled steel tube 'IS 200 Heavy' is provided. A base plate 450mm x 450mm x 30mm is provided after designing for a critical moment of 24838.04 N-mm. The complete retrofitting drawing of Panel J is shown in Fig. 9.
2. *Panel 'I' of First Slab* above ground floor has also developed torsional cracks as in Fig. 6 in previous page. On preliminary investigation, it is judged that this is due to excessive concentrated load. Moreover, on analysis, it is seen that there is not sufficient tensile reinforcement in either direction. The slab panel is cantilevered and reinforcement provided is less by 34% in one direction and 44% in other direction. Remedy suggested is a provision of two props at the extremities of the cantilever projection. These would ensure transfer of torsional moments and loads to the soil through a proper foundation. An I-Beam between the two props is also provided in order to transfer the slab load uniformly to the props. It is made composite with slab panel. After introduction of prop, the moments generated in slab panel are re-distributed and decreased. The existing reinforcement is enough to resist these re-distributed loads. So, there is no need of any further bottom reinforcement along any direction; as already compensated by distribution steel. Besides, there is no need for any additional top reinforcement along x-direction (Fig. 10). An I-beam ISLB-100 is provided to resist the loads acting on it. The prop provided is ISHT 200 and the base plate connection of beam with slab is a 450mm x 450mm x 30mm steel plate Fe415. Fig. 11 depicts the retrofitting of the Panel 'I'.

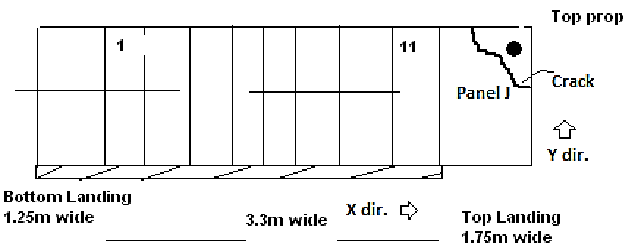

Fig. 7 Panel 'J' with Location of Prop and Cracking

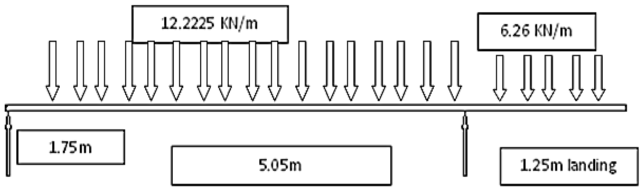

Fig. 8 Load Pattern on panel 'J'

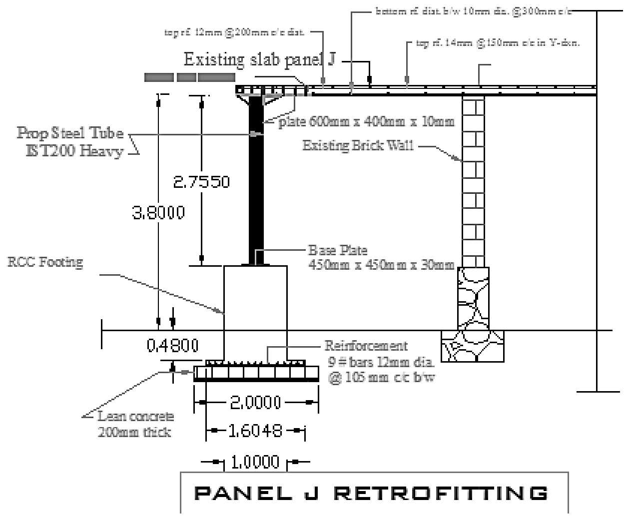

Fig. 9

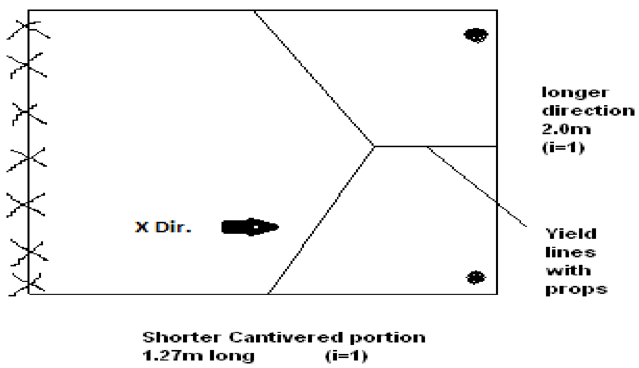

Fig. 10 Panel I with Yield Line Depiction for Props

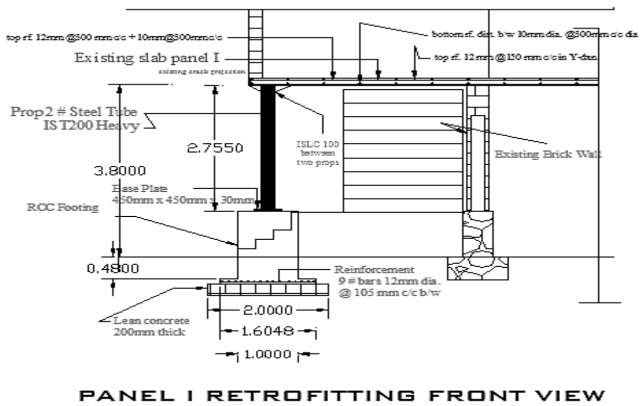

Fig. 11

Published By:
Blue Eyes Intelligence Engineering
& Sciences Publication Pvt. Ltd.

3. *Panel 'H' of First Slab* above ground floor has developed flexural cracks. Besides, by visual inspection it can be seen that there is excessive unsymmetrical concentrated load (Fig. 12). On analyzing, it was found that there is not enough tensile reinforcement to resist the flexural forces, which is short by 19% at critical location. Remedy is providing two props at the extremities of the cantilever projection. These would ensure transfer of torsional moments and a proper load transfer mechanism to through the foundation. Besides, an I-Beam along the longer dimension of the cantilever between the props which is made composite. On introduction of props, the moment resistance is re-distributed and decreased by 15% at the critical location. There is no need of any further bottom reinforcement along any direction; as already compensated by distribution steel. Besides, there is no need for any additional top reinforcement along shorter direction as moment to be resisted is by far less than moment resisted by reinforcement provided. However, props are required to transfer the torsional moments & concentrated loads. Further an I-Beam is required in order to transfer the loads uniformly as presumed in the yield line analysis. I-Beam ISMB 200 is provided and the props provided are ISHT 200 as shown in Fig. 13.

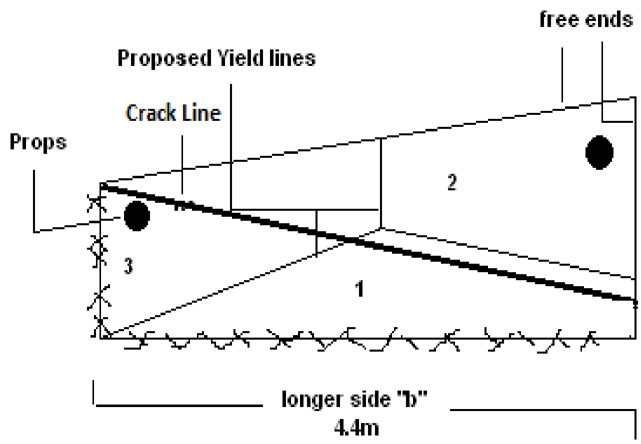

Fig. 12 Panel 'H' Crack and Yield lines

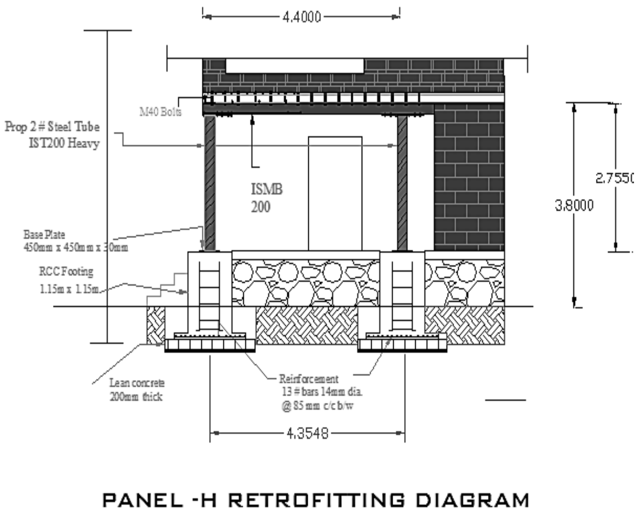

Fig. 13

IV. Conclusion

The case study discussed is a characteristic example of how to retrofit a damaged building. The building is strengthened for earthquakes and cracks are repaired and structural elements retrofitted.

V. Acknowledgment

The authors would like to thank the management of 'Emm's Tiny Tots School' for the risk analysis and retrofitting measures adopted.

References

[1] FEMA 307, *Evaluation of earthquake damaged concrete and masonry wall buildings.* ATC, California, 1998.
[2] FEMA 232, *Homebuilders' Guide to Earthquake-Resistant Design and Construction.* National Institute of Building Sciences, Washington, D.C., 2006.
[3] IS 1893, *Criteria for earthquake resistant design of structures*. Part 1, BIS, 2002.
[4] G. Kennedy, C. H. Goodchild, *Practical Yield Line Design*. The concrete Centre, 2004, ch. 1-4.
[5] S 1905, *Code of Practice for Structural use of Un-Reinforced masonry*. BIS, 1987.
[6] FEMA 547, *Techniques for the Seismic Rehabilitation of Existing Buildings*, nehrp, 2006, p. (21)1- 60.

Shujaat Hussain Buch, Department of Civil Engineering, Faculty and Assistant Professor of Engineering, Islamic University of Science and Technology, Jammu & Kashmir, India. The author is M.Tech Structural Engineering from NIT Srinagar. Research publications:
[1] S. H. Buch, J. A. Bhat, "In-Plane Behavior of Masonry Infilled Reinforced Concrete Frames with Wooden Choh-kat Openings" *IJEAT*, Vol. 2, Dec. 2012, p. 400-405.
[2] M. A. Dar, S. H. Buch, R. R. Malik, "Fly Ash Concrete - A Comparative Study" *IJEDR*, Vol. 10, Oct. 2013, p. 114-122, And is a member of ASCE.

Muhammad Dilawar Bhat, Sarva Shiksha Abhiyan, Assistant Engineer, Srinagar, Jammu & Kashmir, India. The author is M.Tech Structural Engineering from NIT Srinagar.