SEISMIC EVALUATION AND RETROFIT OF MASONRY SCHOOL BUILDINGS IN THE HIMALAYAN STATE OF UTTRAKHAND, INDIA

Abstract

Himalayan region is one of the most seismic areas of the world. However, similar to many other seismically active regions of the world, there is a large numbers of unreinforced masonry buildings, most of which have not been designed for seismic loads. Recent Earthquakes have shown that such buildings are highly vulnerable to earthquakes. Retrofitting of these masonry buildings is the most perceived issue of the present times. The most common method of strengthening of masonry buildings are surface treatment, grout and epoxy injections, micro concrete in splint and bandage and strengthening of existing members by FRP or RC jacketing. Many times these techniques are used as out of box solutions as analysis of masonry structures is a complex task. Unreinforced masonry walls are very weak in out-of-plane bending due to lack of tensile strength. These are generally not capable of bearing out-of-plane bending moment, even resulting from their own inertia. These walls act as shear-walls in their in-plane action and possess sufficient in-plane strength, if not weakened by too many openings. While adopting strategy of retrofitting for this building, care has been taken to ensure integral box action by suitable means.

Most of the government school buildings in rural areas of North India are constructed of unreinforced masonry. These school buildings are socially important structures and most vulnerable in region of high level seismicity. As part of collaboration between the Indian Institute of Technology Roorkee (IITR) and Nanyang Technological University (NTU) Singapore, supported by Temasek Foundation, Singapore, ten schools have been retrofitted in five cities of India. In this paper a case study of four schools in Uttrakhand state has been presented. All the school buildings are evaluated for expected seismic hazard, as per Indian code and retrofit design has been implemented with welded wire mesh and micro concrete in form of horizontal bandage, and vertical splints at corners and junctions of walls. The paper presents the analysis and design methodology along with implementation issues.

Keywords: Earthquake, Design, Retrofit, Unreinforced Masonry, School Buildings.

1. Introduction

Unreinforced masonry (URM) buildings constitute a significant part of the existing building inventory worldwide. URM buildings are vulnerable to lateral loads such as those caused by earthquakes or high speed winds. Most of these were built with little or no seismic loading considerations, and these are not capable of resisting the expected seismic action. Several techniques are available to improve the seismic performance of existing URM walls. Some of them are Stitching & Grout/Epoxy Injection, Re-pointing, Bamboo Reinforcement, Post-Tensioning using Rubber tyres and various types of mesh reinforcement and some of the advanced materials like FRPs which is efficient though costly. Polypropylene Packaging Strip Mesh Reinforcement method of reinforcement uses polypropylene packaging strips that can be found with many packaged items. The strips are intertwined to produce a mesh that is then attached to the wall by drilling through it and using ties. This method effectively improves the shear resistance under static loading. However, mesh snapping at corners is a problem in this method.

IIT Roorkee together with Nanyang Technological University (NTU-Singapore) and the Disaster Mitigation and Management Centre (DMMC) of Uttarakhand, have selected four masonry schools building in the northern state of Uttrakhand (India) for seismic evaluation and retrofit. All these school consists of several blocks constructed in traditional masonry. All the schools are 25-50 years old and have been constructed with burnt clay brick/concrete block masonry in cement mortar. All the walls are load bearing with rigid slab at top. The most crucial issue in seismic retrofitting is availability of drawing. For all four building these drawings were not available with school authority. So the first important task carried out to prepare detailed drawing of all blocks together and layout drawing of school building as reconnaissance survey. The buildings have been evaluated for the expected seismic action as per Indian Standard (IS 1893:2002) and found to be inadequate, particularly under out-of-plane action of walls and bending tension in in-plane actions. Accordingly a seismic retrofit scheme based on strengthening using welded wire mesh has been designed and executed.

2. Seismic Evaluation and Retrofit Design

Analysis of masonry structures is a complex task. In this study a simplified analysis using pier method has been performed for evaluation. In the pier method, the perforated walls are considered as assemblages of piers for in-plane safety. For out-of-plane safety evaluation the walls have considered as vertically spanning members between floors/foundation/roof. The following sections provide the details of method used for in-plane and out-of-plane analysis of walls:

2.1 In-Plane Safety of Walls

Different walls are considered as consisting of different piers and equivalent stiffness of the wall is evaluated using spring analogy. Before calculated this equivalent stiffness in plane stiffness of each pier should be calculated .The in plane stiffness of pier is a function of aspect ratio of pier (h/L), thickness of pier (t), elastic modulus of masonry (E_m) and boundary condition.

For cantilever pier stiffness is expressed as

$$Ri = \frac{E_m t}{4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)} \quad (1)$$

For fixed end piers, pier stiffness is expressed as

$$Ri = \frac{E_m t}{\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)} \quad (2)$$

Stiffness of individual walls can be obtained by series and parallel combination of pier stiff nesses and calculating equivalent stiffness of springs.

2.2 Estimation of Design Seismic Actions

The period of vibration of the building has been calculated using the formula given in IS 1893-2002, which gives the approximate fundamental time period of the of vibration

$$T_a = \frac{0.09h}{\sqrt{d}} \quad (3)$$

where,

h = height of building, in m and

d = base dimension of the building at the plinth level, in m, along the considered direction of the lateral force.

The design horizontal seismic coefficient Ah is determined by the following expression, as per IS 1893 (Part 1): 2002.

$$A_h = \frac{Z}{2}\frac{I}{R}\frac{Sa}{g}B(\xi) \quad (4)$$

where,

A_h = Design horizontal seismic coefficient for the structure Z = Zone factor, for maximum Considered Earthquake (MCE). The factor 2 in the denominator of Z is used so as to reduce the MCE zone factor to the factor for Design basis Earthquake (DBE).

I = Importance factor, depending upon the functional use of the structures, characterized by hazardous consequences of its failure, post-earthquake functional needs, historical value, or economic importance = 1.5 (Table 6 IS 1893 (Part 1): 2002.

R = Response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. However, the ratio (I/R) shall not be greater than 1.0.

Sa/g = Average response acceleration coefficient for rock or soil sites as given by Fig. 2 of IS 1893 (Part 1): 2002 based on fundamental period of vibration of structure.

B(ξ) = Damping Factor. A value of 0.8 has been taken considering the damping ratio of 10% in the masonry building.

2.3 Design Seismic Base Shear

The design lateral force along the direction of earthquake force has been determined using the following expression:

$$V_b = A_h W \tag{5}$$

where,

A_h = Design horizontal seismic coefficient for a structure as per Cl: 6.4.2 using the fundamental time period

W=Seismic weight of the building as per Cl: 7.4.2 of IS: 1893-2002. Here it has been assumed to be equal to the weight of roof and walls. Live load on roof has been ignored, considering the access conditions of the roof.

2.4 In-Plane Bending Moment

The in-plane bending moment in individual walls is determined by considering that the lateral forces acting on the wall which includes sum of earthquake force and torsion force. This lateral force is again redistributed in corresponding pier as per its stiffness.

$$M_{IP} = Fi \frac{h_i}{2} \tag{6}$$

Where,

M_{IP}= In plane bending moment in *Nm*

F_i= Lateral load on Pier in *N*

h_i =Height of the wall in *m*.

2.5 Out of Plane Bending Moment

For estimating the out of plane bending moment in walls, the walls have been considered as a simply supported at the ends (Top and bottom). With this assumption, the bending of wall occurs in vertical plane due to uniformly applied pressure due to inertia force in horizontal direction. The behavior of wall in out of plane failure is assumed to be that of simply supported beam subjected to uniformly distributed load.

$$M_{OP} = \frac{ph^2}{8} \tag{7}$$

Where,

M_{OP} = Out of plane bending moment in *Nm*,

h = Height of wall in m,

p = Out of plane pressure in kN/m^2 due to inertia force, Calculated as,

$$p = A_h \gamma t \tag{8}$$

where,

A_h = Design horizontal seismic coefficient,

γ = Unit weight of masonry wall and

t=thickness of the masonry wall in *m*.

2.6 Retrofit Design

The analysis shows that the almost all the walls of the considered buildings are unsafe in i out-of-plane action and most of the piers are also unsafe in tension resulting due to bending in in-plane action. Therefore, strengthening of walls in both the actions is required, which has been achieved in the present study using the welded wire mesh reinforcement on both faces of the walls, arranged in the form of splints and bandages. Splints are the vertical strips of reinforcement provided along the jambs of opening and along corners/joints of walls. Bandages are the horizontal bends of reinforcement provided at lintel level. This technique is preferable to other retrofit techniques due to addition of relatively small thickness, low cost and ease in application. The mesh reinforcement is galvanised to protect it from corrosion, and micro-concrete of 40 *mm* thickness on both sides of the wall is applied to cover and integrate the wire mesh with the wall. Connection between brick masonry wall and the added wire mesh is critical for transfer of shear at the interface. To accomplish satisfactory transfer of forces at the interface, connectors have been designed to resist the shear force, which develops at the interface of masonry and concrete from the out-of-plane bending of the walls. It has been observed that 6 *mm* connectors at a spacing of 450 *mm* c/c in both directions are adequate for this purpose. Another major issue is the anchorage of added reinforcement at foundation and roof/floor. The mesh reinforcement has been extended down to 300 *mm* from the plinth level and is properly anchored there using through anchors in the walls. At the intermediate floors, the reinforcement in splints is continued through the holes made in the floor slabs and in the roof the reinforcement is anchored to slab.

As mentioned earlier, the walls have been found to be safe in shear, but unsafe in bending tension due to in-plane and out-of-plane action. The splints take care of the tensile stresses in in-plane action, while the bandage is designed as a horizontally spanning composite beam resulting in reduction of vertical span of walls. The amount of reinforcement required in splints and bandages is obtained considering the composite action of the masonry and welded wire mesh. For this purpose the working stress method prescribed by IS 1905: 1987 has been used and the masonry has been assumed to carry tensile stress. Further, a perfect bond between the masonry and reinforcement, facilitated by connectors has been assumed. The design and construction procedure has been explained using a case study of a typical block of one of the selected school buildings.

3. Case Study

A typical case study of one of the block of school at Fakot is presented here. Plan of the block of classrooms has been shown in Fig.1. As discussed earlier, equivalent spring model of wall 2 has been developed and is shown in Fig. 2. The analysis of this block has been carried out as per methodology explained earlier. Table 1 shows results of in-plane analysis of Wall 2. It can be observed that all the piers of the considered wall are safe in shear and bending compression, but these are unsafe in bending tension. Table 2 shows the results of out-of-plane analysis, of Wall 1. In out-of-plane action also, the compressive stresses are within the permissible limits, but the tensile stresses exceed the permissible values. Accordingly the vertical and horizontal reinforcement to take care of the tensile stresses has been estimated and summarized in Tables 3 and 4. The estimated reinforcement has also been compared with the nominal reinforcement provided by IS 13935: 2009. Typical details of provided reinforcement are presented in Figs3 and 4.

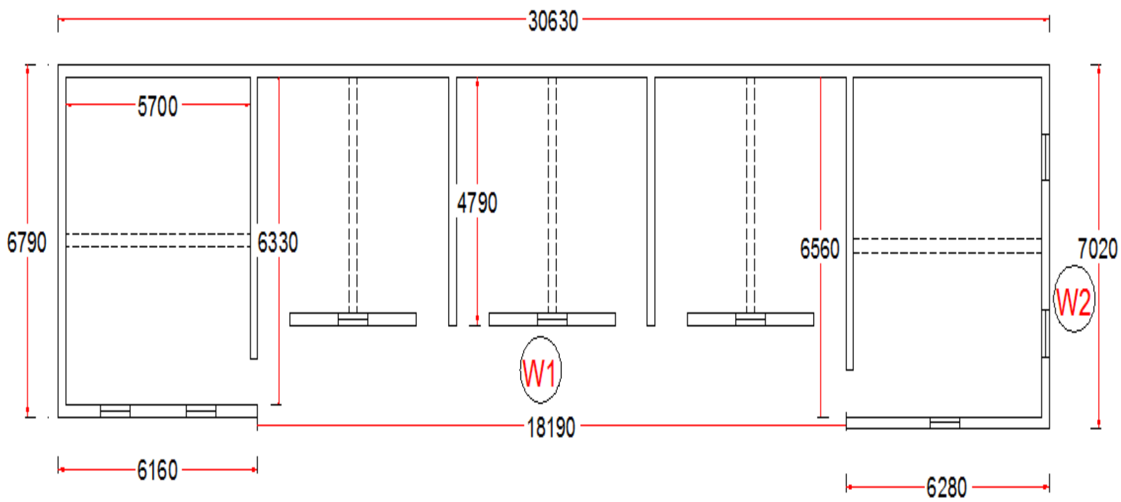

Fig. 1 Typical Block Plan of Classroom

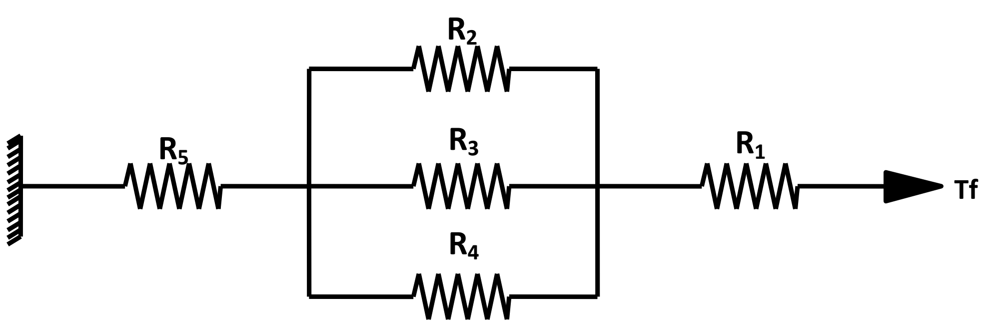

Fig. 2 Equivalent Spring Model for Wall 2

Table 1 Results of In-Plane Analysis of Wall 2

Pier No.	Total Vertical Load on Pier *(kN)*	Moment in Pier *(kN-m)*	Tensile Stress *(N/mm²)*		Compressive Stress *(N/mm²)*		Shear Stress *(N/mm²)*	
			Actual	Allowed	Actual	Allowed	Actual	Allowed
Pier 1	168.13	45.01	-0.082	0.05	0.133	1.17	0.064	0.122
Pier 2	51.02	13.83	0.052	0.05	0.405	1.17	0.079	0.167
Pier 3	101.75	34.06	-0.035	0.05	0.319	1.17	0.097	0.153
Pier 4	49.83	13.30	0.057	0.05	0.413	1.17	0.079	0.168
Pier 5	230.72	45.52	-0.122	0.05	0.173	1.17	0.065	0.128

Table 2 Results of Out of Plane Analysis of Wall 1

Total Vertical Load *(kN/m)*	Horizontal Force (P) *(kN/m²)*	Compressive Stress *(N/mm²)*		Tensile Stress*(N/mm²)*	
		Actual	*Allowed*	*Actual*	*Allowed*
10.550	*1.104*	*0.046*	*1.17*	*0.095*	*0.05*

Table 3 Summary of In-Plane Retrofit Design of Wall 2

Pier No.	Tensile Stress		Design Tensile Force *(N)*	Area of Steel Reqd. *(mm²)*	Nominal reinforcement as per IS:13935 *(mm²)*
	Actual	Allowed			
Pier 2	0.052	0.05	860.15	4.31	116.18
Pier 4	0.057	0.05	953.1	4.77	116.18

Table 4 Summary of Out-of-Plane Retrofit Design of Wall 1

Tensile Stress		Design Moment (kN-m)	Reinforcement Steel (mm²)	
Actual	Allowed		Estimated value	Nominal value as per IS-13935
0.095	0.05	7.08	142.263	149.383

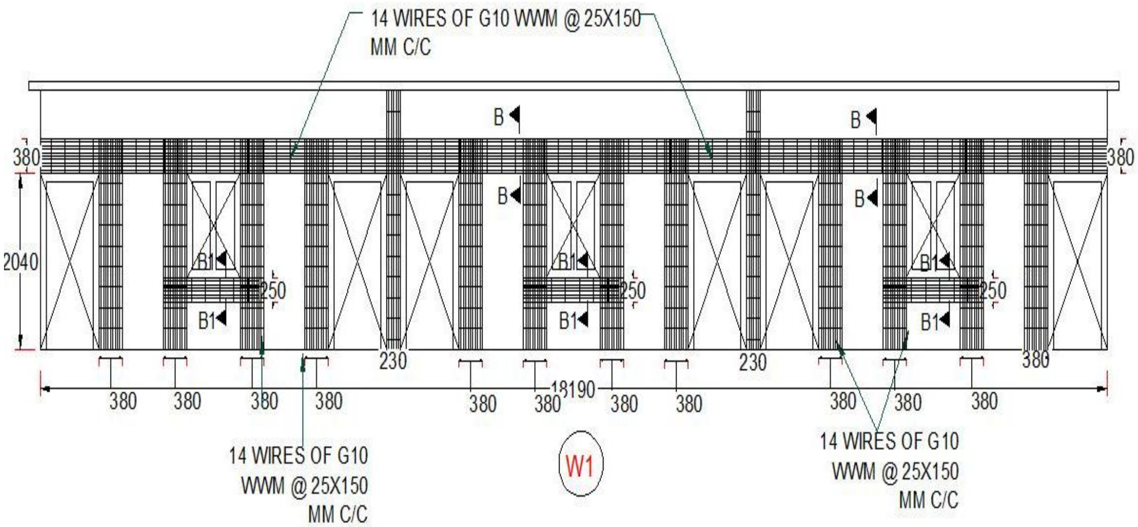

Fig. 3 Typical Retrofitting Details of Wall 1

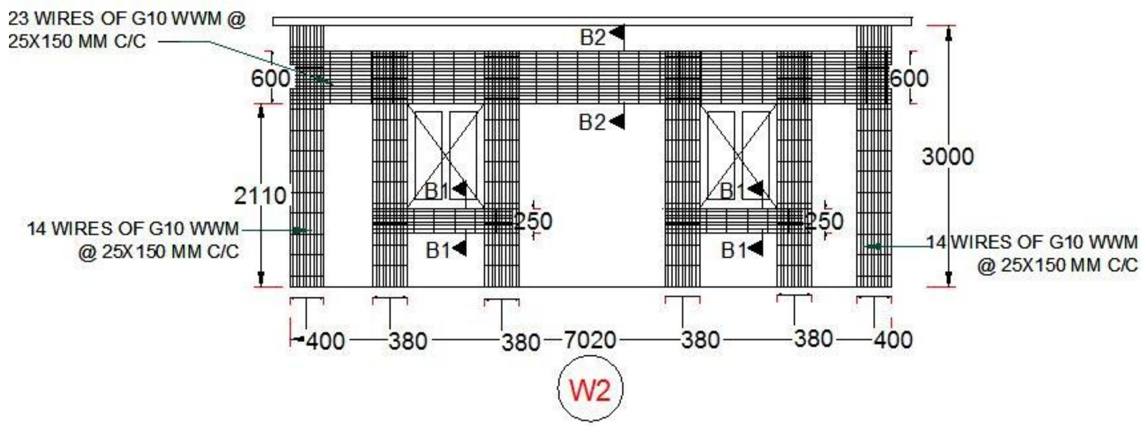

Fig. 4 Typical Retrofitting Details of Wall 2

4. Execution at Site

The execution at site includes following steps. For better understanding typical photographs of execution of retrofitting work at Dobhalwala School are presented at different stages along with equipments used at site.

1. Height or width of desired belt (splint and bandage) based on retrofit design and reinforcement required was marked on the wall. The marking of the seismic bands to be provided at various levels were done using colour and Thread as shown in Fig 5.

Fig. 5 Marking on Masonry Wall

Fig. 6 Mechanical cutter used for cutting plaster

2. Existing plasters at the edge had been cut by mechanical cutter. On the markings which were done the "Stone Cutting Machine" was used to cut through the layer of plaster as shown in Fig 6. Equipment used is Stone Cutter (Alpha A 81012, Angle Grinder 100mm)

3. Exposed joint to the depth of 20 mm had been racked and clean with jet of water to make surface even and clean as shown in Fig.7

4. The micro concrete had been made on the site as per the specifications. The micro concrete had been chosen because minimum thickness of 20 mm is required and application of normal concrete is not possible. Micro-Concrete has been made in the proportion (1:1.5:3). Acrylic Bonding Agent and Liquid Integral Waterproofing Compound are added to it. Acrylic Bonding Agent is used as bonding agent for new to old substrates. The maximum size of aggregate is 8-10mm. Liquid Integral Waterproofing Compound is used as an additive for cement concrete, because of its plasticizing properties, makes concrete cohesive and prevents segregation. For this Pidicrete 303 MPB Acrylic Bonding Agent was used.

Fig.7 Racking of joint

Fig 8 First Layer of micro concrete

5. First layer of micro concrete had been applied for filling all raked joints fully and covering the wall with thickness of 20mm/15mm.Surface was made rough for better bond with second layer of plaster as shown in Fig.8.

6. The wire mesh (1'' x 6'') had been cut to desired width and length as per the design as shown in Fig.9 .Epoxy Zinc Primer is then applied to the wire mesh. Epoxy Zinc Primer is used for coating on steel reinforcement or steel surfaces as an anti corrosion primer. Chemicals Used: Epoxy Zinc Primer (For catholic protection to re-bars and steel surfaces).

7. Mesh or reinforcement had been fixed through nails or connector. Both faces of a wall with both bands tightly connected by a 6 mm galvanized/epoxy coated rods placed in a hole (8-10 mm dia) drilled in a wall of burnt brick @ 300 mm c/c at nodes of main and distribution steels.

Fig. 9 Preparing WWM as per Design

Fig.10 Fixing of WWM Mesh for Bandage

8. Second layer 15mm/20mm micro concrete had been applied finally. Good bondage has been achieved with the first layer as micro concrete or cement mortar was applied by a brush to the wall and the mesh just in advance of second layer of plaster. The wire mesh was then covered with Micro-Concrete. This was done by Guniting. It consists of a twin chamber gun and a twin water tank and is powered by an air motor. The material was deposited onto the desired surface through a nozzle under compressed air. However at some places Hand Vibrator was used as the use of Guniting machine is not viable. Here Equipments Used: Guniting Machine; Hand Vibrator (AKARI AOS-93A).Typical gunitting operation and final view after second layer are shown in Fig.11.

Fig.11 Gunnting operation and final view after second layer.

5. Conclusion

In this paper the procedure of seismic evaluation and retrofitting of URM buildings has been presented. A typical block of school building in the northern India has been evaluated for design earthquake using a simplified Pier Analysis Procedure. The estimated stresses have been compared with the permissible limits. The stresses in the walls are within permissible limit in compression and shear, but these are unsafe in tension in in-plane as well as out-of plane bending. A method of retrofitting using micro concrete and welded wire mesh in splints and bandages has been presented. The estimated reinforcement has been compared with the provisions of IS: 13935. The execution of the same design has been presented with typical photographs. This method of retrofitting is cost effective and is easy in its application compared to the other methods.

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