Role of Masonry in Seismic Strengthening of Newly-Built Pharmaceutical Plant Buildings

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SUMMARY:
Seismic retrofitting of newly built buildings is no less challenging and their construction feasibility and economical viability can be significantly enhanced by including the contribution of all structural components that qualify to resist seismic loads. This is certainly true for the existing masonry which may help reduce the overall structural intervention rather than adding to its seismic vulnerability as observed in seismic strengthening of a pharmaceutical plant building which was deemed severely deficient due to non-compliance with seismic codes and higher seismic hazard revealed after a seismic assessment. The building was a mixed construction of unreinforced brick masonry and reinforced concrete frames and required extensive strengthening, rendering the retrofitting economically unviable. In-place shear and laboratory tests on bricks and simulated masonry verified that the existing masonry was of fair quality and could be relied on to resist a significant amount of in-plane loads and was shown to meet the out-of-plane stability and strength criteria due to arching action. Considering the enhanced contribution of masonry based on in-situ strength, the amount of retrofit required was significantly reduced.

Keywords: Masonry infill, Seismic strengthening, Industrial building, Concrete shear wall

1. INTRODUCTION

The use of unreinforced masonry (URM) as infills in RC frames is a common practice throughout India for practically all kinds of buildings. They are easy to construct and provide economical partitions and barrier against exterior environment. Masonry infills add significantly to the seismic mass of the structure and also to its seismic vulnerability, especially when they are weak and slender. However, they can have beneficial effect in enhancing the lateral stiffness and adding to the lateral load carrying capacity of the structure (Murty and Jain, 2000). This positive aspect of the infill masonry was fully utilized in developing seismic strengthening of the pharmaceutical building which was rendered severely seismic deficient as it lacked the adequate level of seismic detailing required. Moreover, the seismic assessment revealed that the seismic hazard of the site was underestimated by a large margin.

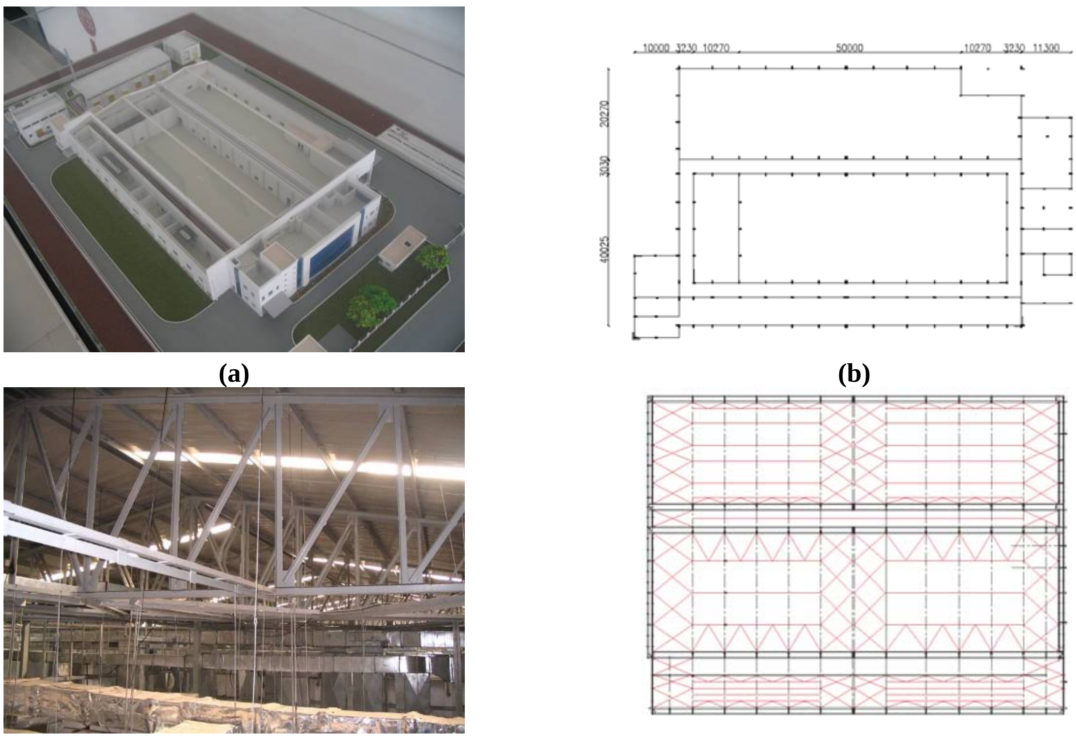
A condition assessment and detailed analysis of the structure was then carried out which deemed it severely deficient with respect to the provisions of FEMA 356 (2000). Initially, in the absence of field data, conservative properties of brick masonry were used in the evaluation analyses. The demand capacity ratio (DCR) thus obtained for the structure showed a large number of structural members failing which indicated the building to be highly deficient in resisting seismic forces. The retrofit scheme devised to eliminate these deficiencies was too extensive, difficult to execute and uneconomical. Various in-situ (shove tests) and laboratory tests confirmed a higher strength of the masonry and its ability to withstand both in-plane and out-of-plane loads. Allowing a higher contribution of existing masonry in seismic response of the structure a significant quantitative reduction in the strengthening process was achieved making the retrofit more economically viable.

2. BUILDING DESCRIPTION

The building is an industrial structure comprising of rectangular RC frames with brick infills at selected bays. Fig. 2.1 shows the general arrangement of building components, its plan view and steel truss diaphragm. It measures 98.3 m in longer direction having 12 bays and 57.5 m in shorter direction having 10 bays. The building was originally provided with expansion joint in the middle to cater for the thermal stresses. The light steel truss roof directly supported on RC columns lacked adequate horizontal bracing and suffered from poorly implemented and weak steel to concrete as well as steel to steel connections. These two units though being dynamically independent faced the issue of pounding during a high-level earthquake.

Locally produced, hand made, and coal-based kiln fired solid bricks laid in English bond pattern with 1:5 cement-sand mortar was used for constructing infill masonry of thickness of 280 mm including cement plaster on both faces. The brick masonry was built in the plane of the concrete frame and was in full contact with the columns and beams. Engineering properties of the masonry were not available initially.

Concrete and reinforcement steel properties data obtained from the tests conducted during the construction confirmed its compliance with design specifications. The grade M20 concrete having cube strength of 20 MPa was used and reinforcing bars were Fe415 (HYSD) with design yield strength of 415 MPa. All structural steel used was Fe10W A and Fe10W B type meeting the requirements of IS 2062 (BIS, 2006) and having minimum yield stress of 250 MPa.



(a) (b) (c) (d)

Figure 2.1 (a) A 3D model of the building, (b) its plan view, (c) the steel roof truss and (d) arrangement of the horizontal bracing

3. SEISMIC HAZARD

The building site is located in the Himalayan foothills, a highly seismic region surrounded by a number of active faults and susceptible to large magnitude earthquakes (M_w >7). The site is placed in Zone IV of the Indian seismic code IS 1893 (BIS, 2002) with PGA of 0.24*g*. However, after further investigation of neighboring fault system and seismicity of the region, it was established that a PGA of 0.4g is more appropriate. In addition, near source factors N_a = 1.5 and N_v = 2.0 were to be considered for the proximity with faults as suggested in UBC 97 (ICBO 1997). Shear wave velocity measurements were taken to identify the soil type at the site. Originally, the test data was sufficient to assume soil to be type D, however, later tests reclassified the soil to type C with seismic coefficients of C_a = 0.6 and C_v = 1.12 according to UBC 97. In Fig. 3.1, the design response spectrum used in the original design of IS 1893 is compared with the revised design response spectrum derived as per UBC 97 provisions which shows 1.5 times increase in the short-period spectral acceleration.

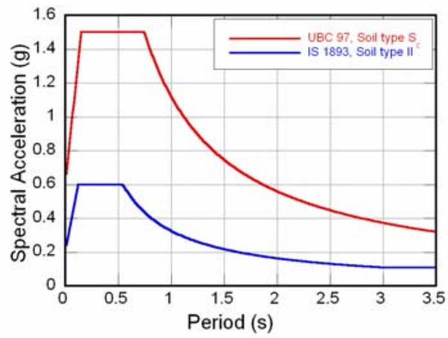
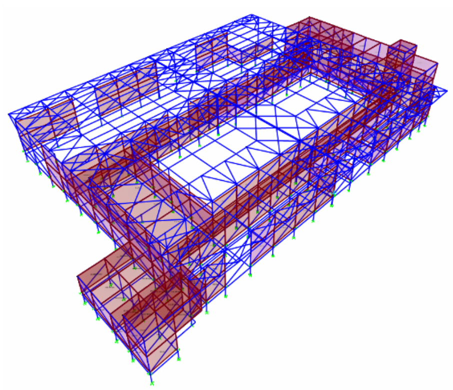


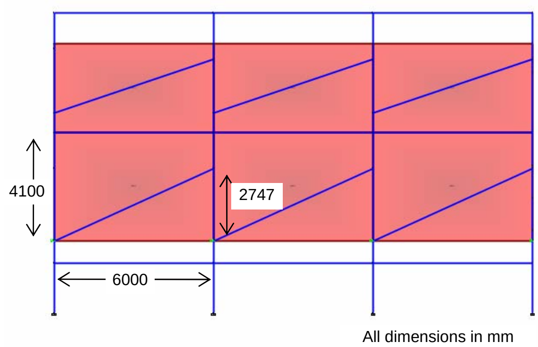
Figure 3.1 Response spectrum with 5% damping for soil type C (UBC 97) and soil type II (IS 1893)

4. SEISMIC EVALUATION

Linear dynamic procedure (LDP) analysis using SAP 2000 (CSI 2009) was performed to analyze the existing building according to FEMA 356 (2000). The RC frames were modeled as plane frame with a series of 1-D beam elements and infills masonry were added as eccentric equivalent diagonal struts. The eccentricity was incorporated to calculate additional shear demand induced in the RC columns due to the horizontal component of the masonry strut force. The roof trusses were modeled as equivalent line elements which together with the horizontal roof bracing in the plane of the truss bottom chord formed the roof diaphragm. Component effective stiffness values were adopted to account for the effects of cracking of the RC sections under earthquake loads. Fig. 4.1 represents the SAP model for the structure with axial force elements representing the equivalent diagonal struts for masonry infills.



(a)



(b)

Figure 4.1 (a) FE model in SAP environment for the existing building and (b) close-up view of eccentric diagonal members used to model infill masonry

The analysis assumed the foundation to be relatively rigid at the base of columns. Due to presence of significant amount of masonry, the seismic force demand for the building was found to be controlled by the short-period region of the design spectrum. The structure was assessed using the basic load combinations of gravity and earthquake loads. Forces from the elastic response spectrum analysis were compared to the expected capacities of the individual element force components with the use of appropriate element demand modifiers *m* of FEMA 356 for their acceptability. Allowable seismic drifts of 2% for the concrete frame and 0.5% for unreinforced masonry were taken. Acceptance criteria for the members comprising the roof diaphragm and for reinforced concrete frame were in accordance FEMA 356. The capacity of the masonry in the evaluation phase was determined based on the expected in-plane panel shear strength and out-of-plane strength as given in FEMA 356.

The DCR is taken as the ratio of section demand over section capacity for a structural action. DCR values at critical sections were calculated for both shear and flexure for the existing building. The DCR values indicated that 100% RC columns failed in shear while 98% columns failed in flexure, and 97% of beams failed in shear while 21% of failure in the beams was controlled by flexure. The assessment found that nearly all the frame members were inadequate, the diaphragm was deficient, critical steel to concrete connections were failing and there was a pounding issue in the existing building. These findings clearly identified the need for structural intervention.

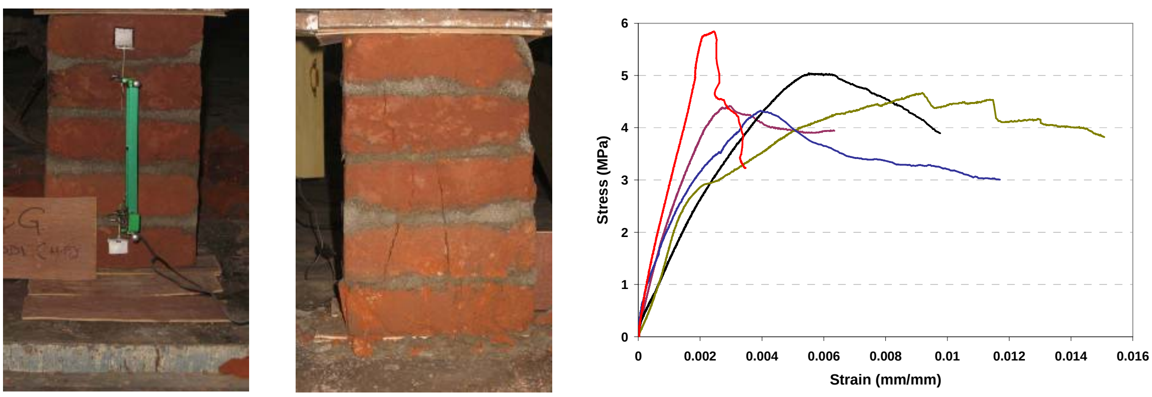
5. STRUCTURAL QUALIFICATION OF EXISTING MASONRY

The structural strengthening scheme developed for the building required extensive work. The high direct and indirect disruption and closure costs were the driving force to revise the retrofit scheme re-assessing the contribution of each structural component. The strength of existing masonry used in the previous assessment was re-examined and it was identified that project specific data was not available. This identified the need for physical testing of the masonry.

A batch of bricks from the site was tested, according to the applicable Indian Standards, as shown in Table 5.1. Laboratory tests were conducted on plain brick and five-brick stack bonded prisms with mortar joints using surrogate mortar of 1:5 cement: sand mix. A total of five prisms were tested on the 28th day which failed in the expected tensile splitting mode. The measured stress-strain curves are plotted in Fig. 5.1. These properties compared well with bricks commonly available in northern India (Kaushik et al., 2007).

Table 5.1 Summary of laboratory tests conducted on bricks and prisms using surrogate mortar

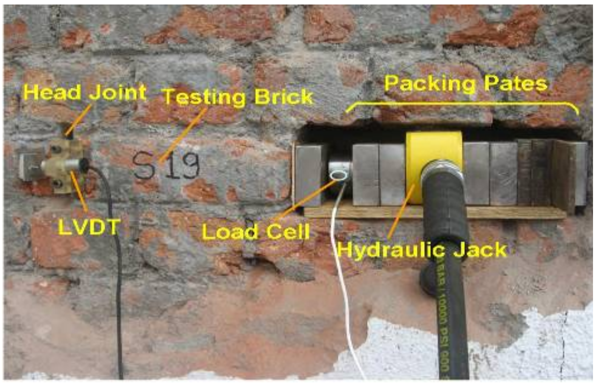
Test	Test Method	Average Values	COV (%)
Dimensions of bricks	IS 1077 :1992	231 x 115 x 71 mm	-
Water absorption of bricks	IS 3495 : 1992 (1)	12.6%	26.8
Compressive strength of bricks	IS 3495 : 1992 (2)	19.86 MPa	18.0
Compressive Strength of surrogate mortar	IS 2250:1981	6.87 MPa	6.5
Compressive strength of masonry prisms, f'_m	IS 1905 :1987	4.32 MPa	12.7
Elastic Modulus in compression, E_m	UBC 97	2150 MPa	30.7
E_m/f'_m		499	28.6



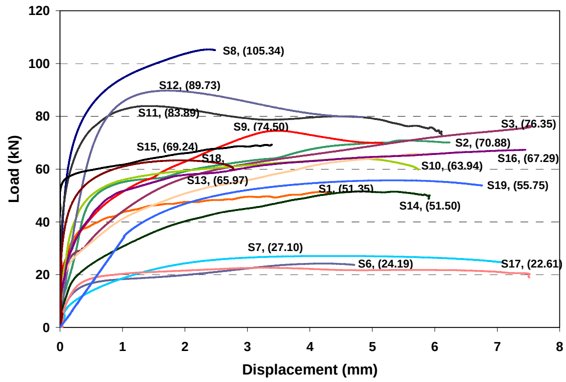
(a) (b) (c)

Figure 5.1 Compression test of masonry prisms (a) Stack bonded 5-brick prism in 1:5 cement:sand mortar, (b) Characteristic vertical splitting failure and (c) observed stress-strain curves

In-situ shear (shove) tests were conducted as per method B of ASTM C 1531-03 to find the mortar joint shear strength. The test involved displacing a single masonry unit horizontally using a hydraulic jack as shown in Fig. 5.2, which also shows the movement of bed-joint with applied load. A displacement transducer was installed across the head joint to measure the movement of the test bricks. The correction for pre-compression (axial load) was calculated as the vertical stress due to self-weight of masonry above the test location, assuming weight density of masonry to be 18 kN/m^3. An average test value of 1.11 MPa with COV of 40.7% was determined for a total of 16 measurements. A large scatter in the test data was noted, however, a large test data set employed in this study is an acceptable sample size even for such a large scatter. The results of the in-situ and laboratory tests were used to derive material properties for use in the seismic retrofit design and are summarized in Table 5.2.



(a)



(b)

Figure 5.2 (a) Loading and instrumentation system, which was, employed during the test and (b) Plots of applied load versus displacement of the test brick for in-place shear test at various locations

Table 5.2 Expected values of masonry properties for FEMA 356 analyses

Property	Value
Compressive strength of masonry, f_{me}	3.24 MPa
Elastic Modulus, E_m	500 f_{me} (MPa)
Bed-joint shear strength, v_{me}	0.41 MPa + 0.67 P_{CE}/A_n

5.1 Evaluation for In-plane loads

In-plane behaviour of masonry infills is a statically indeterminate and a complex phenomenon. The failure mode of the masonry is generally determined by its relative strength in compression and shear, along with parameters describing geometry of the infill panel, and its placement in the building, etc. Various formulations (Polyakov, 1956; Paulay and Priestley, 1992; Angel et al., 1994; Stafford-Smith and Carter, 1969; Mainstone, 1971) have been suggested to predict the lateral stiffness and strength of masonry infills. The strength is predicted considering two prominent failure modes: (a) Diagonal compression failure of the masonry (i.e., strut compression failure) and (b) Sliding failure of masonry through bed joints either single or stair-stepped (i.e., sliding shear failure). For each failure mode, strength equations have been developed and are available in documents such as FEMA 306 (1999), FEMA 356 (2000), etc., and in the literature. However, it is rather difficult to predict the failure mode by simply comparing the strength obtained from these equations, because they independently ignore many other influencing factors. In general, strength of masonry in compression and shear along with parameters describing geometry of the infill panel, etc., determine the controlling failure mode and, hence, the expected infill strength. Al-Chaar et al. (2002) provided the following criterion which holds true for the strut compression failure mode in a wall consisting of several bays and stories:

$$\left(f_m' / f_v\right) \times \left(n_1 / n_2\right) \times \left(h / w\right) > 36 \tag{5.1}$$

where, f_m' is compressive strength of masonry; f_v is shear strength masonry; h and w are height and width of the wall panels, respectively and n_1 and n_2 are total number of bays and stories. These calculations for infill frames in the long and short directions are summarized in Table 5.3 and it indicates that the compression strut failure mode is probable for infill frames in both directions. However, it should be noted that FEMA 356 specifies that the infill strength is calculated based on the bed-joint shear formula, ignoring the other possible failure modes.

Table 5.3 Calculations for controlling failure mode of masonry infills (Al-Chaar et al. 2002)

Compressive strength of masonry, f_m' (MPa)	3.24	
Shear strength masonry, f_v (MPa)	0.41	
Height of the wall, h (m)	4.1	
Typipcal wall width, w (m)	6	
	Short direction	Long direction
Number of bays n_1	9	16
Number of storeys n_2	1	1
$(f_m'/f_v)\times(n_1/n_2)\times(h/w)$	49	86
Failure mode	Compression strut	Compression strut

For modeling of the diagonal strut, FEMA 356 suggests the following relations for the equivalent strut width (Eqn. 5.2 and 5.3) to be used for the lateral stiffness of the masonry infill frame, which is based on the work of Mainstone (1971):

$$a = 0.175(\lambda_1 h_{col})^{-04} r_{\inf} \tag{5.2}$$

$$\text{and, } \lambda_1 = \left[\frac{E_{me} t_{\inf} \sin 2\theta}{4 E_{fe} I_{col} h_{\inf}}\right] \tag{5.3}$$

where, h_{col} is column height between centerlines of beams, h_{inf} is height of infill panel, E_{fe} is expected modulus of elasticity of frame material, E_{me} is expected modulus of elasticity of infill material, I_{col} is moment of inertia of column, L_{inf} is length of infill panel, r_{inf} is diagonal length of infill panel, t_{inf} is thickness of infill panel and equivalent strut, θ is angle whose tangent is the infill height-to-length aspect ratios (units are in., in.^{4}, ksi and radian).

Al-Chaar et al. (2002) noted that the lateral stiffness of masonry infills is underestimated by the above relations and, thereby, the drifts are overestimated because the calculated strut width is smaller than recommended by other researchers. In this study, the stiffness was calculated by taking the strut width as 0.25d, where d is length of the diagonal as recommended by Paulay and Priestley (1992). For the infill strength calculation corresponding to the compression failure of the strut diagonal, the following relation of FEMA 306 was used, which is a modified version of the method suggested by Stafford-Smith and Carter (1969).

$$V_{cd} = a \times t_{\inf} \times f'_{me90} \cos\theta \tag{5.4}$$

where, V_{cd} is horizontal component of the diagonal strut capacity indicating strength capacity of the infill for the strut compression failure mode, a is equivalent strut width, t_{inf} is infill thickness and f'_{me90} is expected strength of masonry in the horizontal direction, which is taken as 50% of the expected stacked prism strength f'_{me}. For the infill strength corresponding to bed-joint sliding, V_{bj} the following relation as specified by FEMA 356 was used:

$$V_{bj} = A_{ni} \times f_{vie} \tag{5.5}$$

where, A_{ni} is area of net mortared section across infill panel and f_{vie} is expected shear strength of masonry infill.

The total shear strength of infills in both orthogonal directions as determined by diagonal compression strut capacities and bed-joint sliding capacities are summarized in Table 5.4, which shows that the strength corresponding to strut capacity is smaller than the sliding shear capacity. Considering an *m* value of 3, masonry infill capacity is also compared with the total shear force demand and it is clear that masonry infills alone were not adequate in the short direction.

Table 5.4 Evaluation of in-plane capacity

Seismic mass W (kN)		71841
m -factor		3
m -factor adjusted total shear force demand V_b (kN)		35920
Avg. shear strut capacity V_{cd} (kN)		618.7
Total number of bays	Long Dir.	75
	Short Dir.	35
Shear strength capacity (kN)	Long Dir.	46400
	Short Dir.	21654

5.2 Evaluation for out-of-plane loads

Infill masonry panels derive a considerable resistance against out-of-plane loads (such as wind and earthquakes) due to arching action. The compressive force developed in the masonry wall due to arching action act as a stabilizing force against the destabilizing effect of out-of-plane inertia loads. This resistance is deformation controlled and, hence, ductile and more reliable. For the given infill frame the validity of arching action has been established as per FEMA 356. For the given infill frame, the flexural rigidity, $E_{fe} \times I_f = 1.41 \times 10^{13}$ MPa, which exceeds the limiting value of 1.03×10^{13} MPa. Also, $h_{inf} / t_{inf} = 10.7$, which is less than the upper limit of 25. The out-of-plane force per unit area F_p is calculated as 3.4 kPa as shown below, where χ is factor as 0.4 for the selected performance level, S_{XS} is spectral response acceleration at short periods equal to1.6g and W is weight of the wall per unit area:

$$F_p = \chi S_{XS} W = (0.4)(1.6)(19 \times 0.28) = 3.4 \times 10^{-3} \text{ MPa} \tag{5.6}$$

The capacity calculations performed as per FEMA 356 and various other methods shown in Table 5.5 confirms that the existing infill panels had a significant margin of safety in both undamaged as well as in the in-plane damaged state, thus, giving confidence in the out-of-plane resistance of the existing URM infill walls.

Table 5.5 Evaluation of out-of-plane capacity

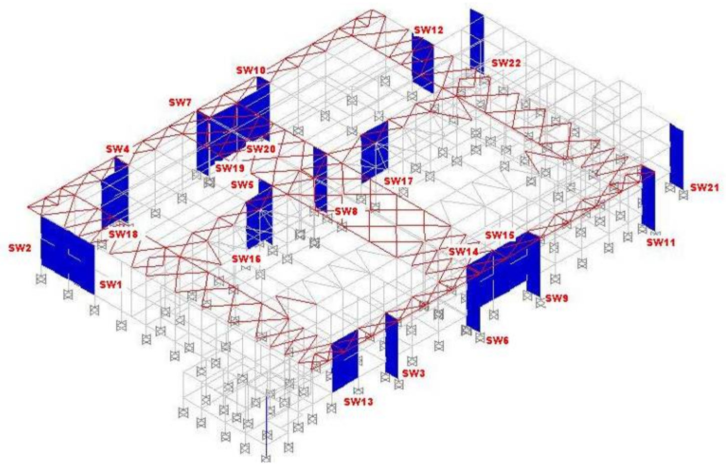
Analytical method	**Without in-plane damage**		**With in-plane damage (cracked stage $\Delta = 4\Delta_{cr}$)**	
	Capacity (kPa)	**DCR**	**Capacity (kPa)**	**DCR**
Dawe and Seah (1989)	8.8	0.39	-	-
Angel et al. (1994)	22.9	0.15	17.7	0.18
Klingner et al. (1996)	33.6	0.10	-	-
Flanagan et al. (1999)	7.9	0.41	-	-
FEMA 306 (1999)	22.9	0.15	20.3#	0.16#
FEMA 356 (2000)	9.1*	0.36*	-	-

* Lower bound strength; # Severe in-plane damage

6. RETROFITTING SCHEME & ROLE OF MASONRY

The seismic retrofit design was undertaken in accordance with FEMA 356 and the retrofit was aimed at achieving Life Safety building performance level. The implication of this is that overall damage will be moderate and some residual strength and stiffness will be available in all stories. Gravity load bearing elements will remain functional and no out-of-plane failure of walls or tipping of parapets should occur.

The retrofitting concept adopted was focused on strengthening the weaker elements and improving the connection between the primary structural elements to enhance the load transfer mechanism and as a whole to develop a better seismic response. In order to eliminate various seismic deficiencies, the option of stiffening, strengthening, load path completion and tying the roof diaphragm across the expansion joint were formulated. The basic strengthening included introduction of new shear walls and buttresses in order to control the drift of the structure and minimize the loads acting on the brittle RC frame with low deformation capacity. The plan of the building with retrofit scheme is shown in Fig. 6.1. With these new shear walls and local retrofit of isolated members, the number of columns failing in shear and flexure reduced to 3% and 10% respectively and 1% of beams failed in shear as well as flexure, which is significant reduction in the numbers observed for the “as-is” analysis case. The displacement demands imposed on components that had inadequate ductility to resist the resulting deformations were reduced to acceptable levels.



Shear wall ID	**Shear strength (kN)**
Long direction	
SW13,14,16-19	3712
SW15,20	4640
Total of 8 walls	**31552**
Short direction	
SW1,2	3712
SW3-10	1931
SW11	2265
SW12	3218
SW21,22	1856
Total of 14 walls	**32070**

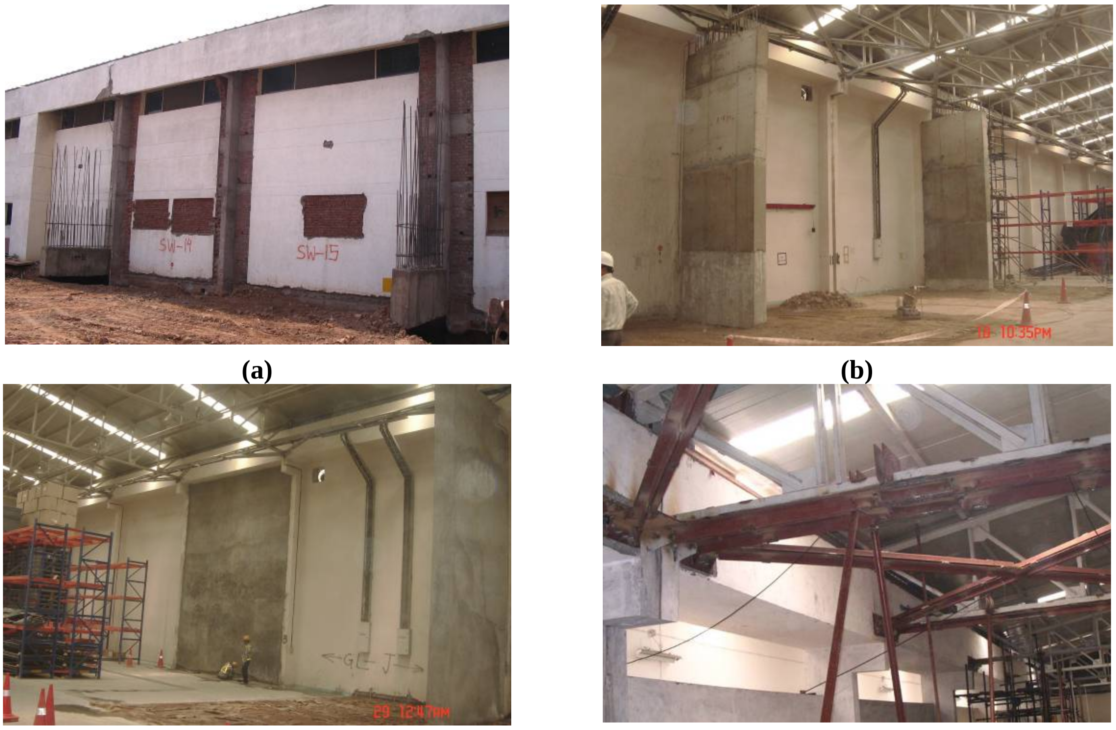
Figure 6.1 Retrofit scheme of building showing location of shear walls and diaphragm bracings

An estimate of the possible reduction in the retrofit quantity by considering the strength contribution of masonry infill can be obtained by calculating the amount of new RC shear walls which will be required for the same level of lateral shear capacity as was provided in the proposed retrofit scheme. These calculations are summarized in Table 6.1 in which it can be seen that by allowing the masonry infills to share the seismic shear force demand, a reduction of about 50% in the requirement of new RC shear walls was achieved.

Table 6.1 Requirement of new RC shear walls ignoring masonry infill contribution

Total shear capacity provided by the retrofit design considering shear strength of masonry infills and shear walls		
	Long Direction	Short Direction
Shear wall Area	15.3 m^2	18.4 m^2
Shear capacity provided by shear walls	31554 kN	32070 kN
Compression strut shear capacity of masonry infill	46400 kN	21654 kN
Total shear capacity	77954 kN	53724 kN
Total RC shear wall area required for providing equivalent shear capacity without considering the strength contribution of masonry infills		
Avg. shear strength of new shear wall	1.84 MPa	
Shear wall area required in long direction	77954.6/1.84 =42.4 m^2	
Shear wall area required in short direction	53724/1.84 = 29.2 m^2	
Total shear wall area	71.6 m^2	
Net reduction in shear wall area	**53%**	

In addition to enhancing the capacity of the building’s lateral system, the roof diaphragm was strengthened to ensure load path completion in transferring the roof inertia loads to the lateral force resisting system of the building, also improving the lateral support provided at the top of the walls. Tying the roof diaphragm across the expansion joint building eliminated the pounding issue. Provisions of new shear walls helped reduced lateral deflection and amount of strengthening needed for the roof diaphragm. The spread footings and the columns perpendicular to the plane of connected masonry walls were retrofitted locally. The roof diaphragm was strengthened by the introduction of new bracing members, strengthening end connections and tying of the internal columns along the expansion joints. Figure 6.2 shows some of the on-site implementation of the proposed retrofitting work.



(a) **(b)** **(c)** **(d)**

Figure 6.2 (a) Addition of buttresses to existing RC columns on periphery, (b) New shear wall in the shorter direction, (c) New shear walls in longer direction next to the masonry infill and (d) New horizontal bracing across the expansion joint and its connection to the RC gutter beam

7. CONCLUSIONS

The use of unreinforced masonry infill in reinforced concrete frame structures is nearly a universal practice in many countries. The seismic vulnerability of such structures in the event of an earthquake can often be quite high and a thorough examination of the dynamic behavior of such buildings is required considering the potential of the infill on the frame members and the stability of the infill itself. The seismic strengthening developed for a newly built pharmaceutical industry building was extremely extensive in its scope of work and quite uneconomic due to additional costs of extended disruption and closure of the facility when the beneficial effect of the infill masonry was considered in a conservative way. Nominal investment to determine the structural properties of the masonry infill were established by various laboratory tests conducted on bricks and masonry prisms and in-situ tests. It was demonstrated that the existing infill masonry could be relied upon for resisting both in-plane and out-of-plane lateral loads developed in the structure during the design seismic event. A significant reduction in the retrofit work was achieved by appropriately accounting for the available strength of the infill masonry. In addition to the lateral strength deficiency, the structure had to be strengthened for adequate diaphragm action and robust load path for lateral loads, tying of the building halves together and providing new foundations to the new RC shear walls. In a few isolated places localized strengthening was required to address local deficiencies.

Incremental investment to determine project specific material properties is invaluable in undertaking seismic retrofit projects. It allows design teams to refine seismic retrofit designs based on evidence which reduces uncertainty and help ensure informed design decisions can be made. In this example the construction feasibility and economical viability of the project was significantly enhanced by making minimal investment in determining the in-situ masonry condition and strength which allowed the design team to significantly reduce the required overall structural intervention. In this process the need to strengthen the infill walls surrounded by RC frames was also eliminated from the seismic retrofit.

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