Strengthening of Brick Masonry with GI wire mesh

Ravi Dhital
Department of Earthquake Enginering
IOE Thapathali Campus
Kathmandu, Nepal
nepaliaswin@gmail.com

***Abstract*— This paper presents the result of an analytical study, in terms of seismicity, of a single masonry wall before and after the use of a strengthening technique: Wall Jacketing. The study idealizes an inferior material quality and non-engineered plain brick masonry wall for seismic evaluation. The wall was modelled in SAP 2000 and analyzed for the stresses induced and its distribution. As the stresses induced upon seismic loading was beyond its strength, Wall jacketing was introduced as a seismic strengthening measure, and designed. The new analysis showed improved performance. Through the study, it has been concluded that GI wire mesh wall jacketing significantly increases lateral strength and deformability of the seismically deficit low strength masonry structure. It also improves the in-plane strength of the wall, and the structural integrity of the whole structure in terms of in-plane and out-of-plane forces.**

Keywords*— *Wall jacketing, Retrofitting, Seismic performance, Brick Masonry, GI wire mesh

I. INTRODUCTION

Masonry is one of the oldest, traditional, and most common medium of housing construction in Nepal. It consists of fieldstone, fired brick, concrete blocks, adobe or rammed earth, wood or a combination of locally available traditional materials. The construction system is informal with the use of local masons with only little intervention by professional experts, so our masonry practice is non-engineered construction. Although, masonry is most often preferred and employed construction, it is not perfect with regard to seismic efficiency. Experiences show that the collapse of non-engineered construction is one of the largest contributors of losses and casualties during an earthquake. In compared to other modes of construction such as reinforced concrete and steel, masonry has low seismic resistance and is a sister cause of huge devastation during earthquake.

A proper adherence to recommended earthquake resistant measures as per NBC or IS code may avoid such heavy loss of life and property. However, for existing buildings that are in use there exists a threat to future damage due to an earthquake. Retrofitting of existing buildings emerges as a possible solution that implies incorporation of earthquake resistant measures in either seismically deficit or earthquake damaged parent construction. The original structural inadequacies, material degradation over time, and alterations carried out over time such as making new openings, addition/removal of new/old parts inducing dissymmetry in plan and elevation are responsible for affecting the seismic behavior of old buildings. Retrofitting is important for we still have many seismically deficit buildings, and in case of an earthquake event, the immediate shelter requirements cannot be met; all the buildings cannot be replaced or rebuilt. This saves cost, time and most importantly lives of occupants. Selection of any retrofitting technique depends on the existing fragility of the masonry structure. Although masonry shows localized behavior in most cases because of variation in type of materials used, type of construction, site of construction, structural typology, almost all observations show uniform modes of failure of wall during seismicity. The two most common, and dominant, modes of failure are out-of-plane and in-plane failure of walls.

a. Out-of-plane failure: The structural walls perpendicular to seismic motion are subjected to out of plane bending resulting in out of plane failure. This is also due to inadequate anchorage of the wall into the roof diaphragm, and limited tensile strength of masonry element as well as mortar. The flexural stress exceeds the tensile strength of masonry resulting in rupture followed by collapse. This is dominant in long span diaphragms causing excessive horizontal flexure. Out of plane movement and failure characterization (Zuccaro and Papa, 1999) follow as:
    i. Vertical cracks in corners and/or T-walls
    ii. Horizontal cracks along the façade
    iii. Partial collapse of an external wall
    iv. Wythe Separation
    v. Cracks at lintel and top of slender piers
    vi. Cracks at the level of the roof
b. In-plane failure: The walls parallel to seismic motion suffer in plane bending and shear causing horizontal and diagonal cracks in the wall known as In-plane failure. This is most common in un-reinforced masonry structures due to excessive

bending of shear as is evident from double diagonal (X) shear cracking. In-plane failure Characterization (Pasquale and Orsmi, 1999);

i. Vertical cracks on openings
ii. Diagonal shear cracks on parapets and in doors and window lintels
iii. Diagonal shear cracks in the masonry piers between openings
iv. Crushing of corners of walls due to excessive compression stress
v. Horizontal flexure cracks on top and/or base of masonry piers
vi. Vertical cracks at wall intersections
vii. Separation and expulsion of the intersection zone of two corner walls

c. The other masonry failure are diaphragm failure, pounding, connection failure, and failure of non-structural members.

The study focuses on seismic strengthening of wall against in-plane and out-of-plane failure using one of the simplest retrofitting technique: wall-jacketing using GI mesh in cement mortar (Ferro cement Jacketing). Jacketing: This simple method consists of a galvanized iron mesh fixed to the walls through nails or connector links (anchors) through the wall thickness, and the mesh is covered by the cement sand mortar in the ratio 1:3. For full effect and improved tensile strength, the jacketing is recommended on both faces throughout the length. This method is suitable for up to 4 story. It induces extensive changes in architecture, takes more time, and is costly. However, the performance improvement can be achieved up to life safety (LS) to Immediate Occupancy (IO) levels (up to MMI IX). The study considers wall as structural elements, and structural safety is kept the prime objective.

II. METHODOLOGY

There is no closed form solution available for the computation of stresses in walls due to combined action of bending and axial compression. The analysis and design presented here is approximate and is in very simplified approach. A typical common style, single brick masonry wall 9 in (230 mm) thick, 10 ft (3048 mm) high and 14 ft (4267 mm) long with opening (5 x 6) ft. or (1524 x 1829)mm is considered for the study.

Seismic Base Shear has been calculated as per NBC,
Seismic Zoning Factor (Z =1),
Importance factor (I=1),
Structural performance factor (K=2.5),
Period, T= (0.09 X H)/ (D^0.5) =0.13,
C=0.08 for subsoil type II.
Seismic Coefficient, C_d = CZIK =0.08 x 1.0 x 1 x 2.5 = 0.2.
Similarly, other loads in the wall are Dead load due to slab self-weight, slab and floor finish. The live load used is 1.5 KN/m^2.

The other parameters adopted for wall modeling are:
Unit Weight = γ = 19 KN/m^3
Poisson's Ratio = ν = 0.15
Modulus of Elasticity (E) =968 N/mm^2

Slenderness ratio (SR) = Maximum of $\frac{3.048}{0.23}$ and $\frac{4.267}{0.23}$ = 13.25 or 18.50 = 20 taken
Eccentricity of wall = 0
Height to width ratio = $\frac{3.048}{4.267}$ = 0.71 < 0.75

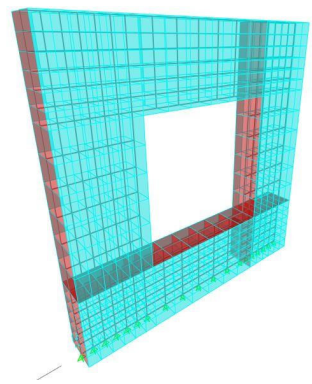

Fig. 1. 3D Model of wall in SAP2000

The wall was modeled using SAP 2000 and analyzed for stress concentration and distribution. The resultant stresses: shear, axial compression and tension were checked against permissible stresses and upon failure, the strengthening measure was designed. Wall jacketing, using GI wire meshing in cement sand mortar, was considered for strengthening of the wall. The wall was modelled using layered shell design providing equivalent thickness of steel throughout the wall. The resultant stresses induced after strengthening were analyzed and checked against permissible stresses for the composite section.

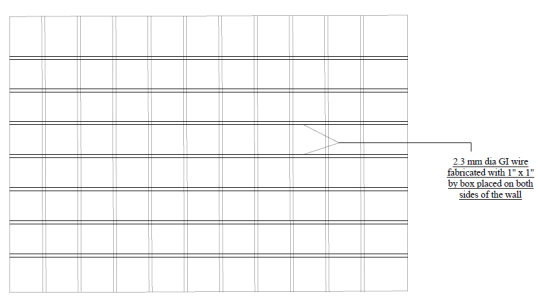

Fig. 2. Detailing of Jacketing 2.3mm @ 25 mm bothways

A. *Permissible Stresses and Checks:*

The permissible stresses check were maintained as per IS 1893: 2000. The permissible stresses for retrofitted section were computed as :

Permissible Compressive Strength ($F_{retrofittd\ wall}$)

$F_{retrofittd\ wall} = F_{wall} + F_{steel}$ (1)

$F_{wall\ only} = f_b \times K_s \times K_a \times K_p$ (2)

$F_{steel} = f_s \times$ Ast provided (3)

Where, for M2 mortar, 5 Mpa Brick masonry

Basic Compressive strength (f_b) = 0.44 N/mm^2

Stress reduction factor (K_s) = 0.62

Area Reduction Factor (Ka) = 0.7 + 1.5 A =1 for A being greater than 0.2 m^2

Shape Modification Factor (K_p) =1

Similarly, permissible Shear Strength of retrofitted wall ($\tau_{c\ (retrofitted\ wall)}$) was computed as,

$\tau_{c\ (retrofitted\ wall)} = \tau_{c\ \ original\ wall\ only} + \tau_{c\ \ (steel)}$) (4)

$\tau_{c\ original\ wall\ only} = 0.1 + f_d/6$ (5)

Where,

f_d = compressive stress due to dead load in N/mm^2

τ_c = permissible shear stress in N/mm^2

$\tau_{c\ (steel)}$ computed from Table 23 of IS 456:2000

B. Design of Strengthening technique: GI wire meshing in cement sand mortar

Wall Jacketing using GI wire mesh in cement mortar 1:3 is considered for retrofitting. The wall jacketing is considered on both face of the wall to consider for full effect(benefit). Sample calculation for bandage of jacket for unit length on both directions is presented herewith.

1) Inertial force carried by the bandage, q,

q = C x γ x t x h x 1= 0.43 x 19 x 0.23 x 1 x 1 =1.88 KNm

This bandage is designed for horizontal bending of the wall,

Span (ℓ) = 4.2672 m

Bending moment (M) = q x $\ell^2/10$ =1.88 x $4.2672^2/10$ =3.423 KNm

2) Resisting bending moment calculations:

T=C= Allowable stress x steel area

The allowable stress is increased by 25% for earthquake loading.

Resisting Bending Moment, M = T x Z

Allowable stress in steel = 1.25 x 0.56 x fy = 1.25 x 130 = 162.5 N/mm^2

M = 1.25 x 130 x Ast x Z

For, lever arm Z = 280 mm,

Ast = $\frac{3.423}{1.25 \times 130 \times 280}$ x 106 = 75 mm^2

Using galvanized wire 2.3 mm @ 25 mm and width 750 mm throughout the wall. (Ast provided= 170 mm^2)

For modeling in SAP 2000, the Ast provided is modeled as a thin layer of equivalent steel placed as a layer throughout the wall section. It is computed as:

$t_s = \frac{Ast\ Provided}{Length}$ = 170/1000 =0.17 mm

3) Shear force in the added section:

Shear Force (V) = q x ℓ/2 + (M+ M)/ ℓ=1.88 x 4.2672/2 + 2 x 3.423/4.2672 = 5.615 KN

Assuming unit m strip of bandage on either side of wall, percentage of tensile steel, Pt is,

$$Pt = \frac{170 \times 1000}{50 \times 1000} = 0.34\%$$

Permissible Shear Stress (τ_C) = 0.265 N/mm^2 (IS 456: 2000, Table 23)

Considering all shear carried by the bandage,

Induced shear stress = $\frac{5.615 \times 1000}{2 \times 50 \times 1000}$ = 0.051 < 0.265 N/mm^2 SAFE!

4) Interface Bond between Wall and Bandage:

Interface Bonding Force = T = C = Ast x 0.56 x fy

Bonding force = 170 x 130 /1000 = 22.1 KN

Assuming that unit meter strip of bandage (50 x 1000 x 4276) mm on either side of wall,

Bond Stress = $\frac{Interface\ Bonding\ force}{Wall\ Length \times Bandage\ Depth}$ = $\frac{22.1 \times 1000}{1000 \times 4276}$ = 0.0052 N/mm^2

Considering all shear carried by band interface,

The shear resisting capacity between brick and band = 0.1 N/mm^2 > 0.026 N/mm^2

The design is safe in bond without anchor!Bond Failure is brittle so we keep anchor.

5) Design of Anchor:

In case of bond failure, the anchor should resist the interface force.

Force to be resisted by anchors = T = C =22.1 KN

Use 4.75 mm (Fe 415) rebar as anchor, then shear area of each anchor = 17.71 mm^2

Allowable shear stress in anchor = 0.4 x 415 = 166 N/mm^2

Shear resistance per anchor = $\frac{17.71 \times 166}{1000}$ = 2.94 KN

Number of anchors = $\frac{22.1}{2.94}$ = 7.5 or 8 nos.

Spacing of anchors = $\frac{4267}{8-1}$ = 609 mm,

Use 600 mm.

6) Check for vertical Bending:

Vertical height (h) = 3.048 m

For q = 1.88 KN,

Moment (M) = q x $h^2/12$ = $\frac{1.88 \times 3.048 \times 3.048}{12}$ = 1.455 KNm/m

Bending Stress (fb) = $\frac{M}{Z}$ = $\frac{1.455 \times 10^6}{1000 \times \frac{230 \times 230}{6}}$ = 0.165 N/mm^2

Vertical Loads:

P= wall + slab + Finish = 19 x 4.2672 x 0.23 x 3.048 + 4.522 x 0.1 x 24 + 4.522 x 1 = 72 KN

Axial Stress (fa) = $\frac{P}{A}$ = $\frac{72 \times 1000}{230 \times 4267.2}$ = 0.0733 N/mm^2

7) Check for wall stresses:

Direct Compression = 0.0733 N/mm^2

Bending Stress = 0.165 N/mm^2

Maximum Stress = fa + fb = 0.0733 + 0.165 = 0.2383 N/mm^2

Minimum Stress = fa - fb = 0.0733 – 0.165 = -0.0917 N/mm^2

The wall is in tension as well.

We provide longitudinal reinforcement in vertical direction for tension.

Tensile stress (σt) = 0.0917 N/mm2

Tension (T) = σt x A = 0.0917 x 230 x 1000= 21091 N = 21.091 KN

Also, T = 1.25 x 0.55 x fy x Ast

Then, Ast = $\frac{T}{1.25 \times 130}$ = $\frac{21091}{1.25 \times 130}$ = 130 mm^2

Ast provided = 170 mm^2 per m length in both directions. Hence, steel takes tensile stress

III. Results and Discussion

The seismic analysis of the wall was carried out considering earthquake in two directions. The design forces for retrofitting were determined by considering direct and torsional forces due to lateral loads, axial load due to overturning in addition to dead and live loads. The seismic design includes determination of steel for GI mesh in both directions for resisting the compression, shear and flexure forces of the wall.

TABLE I. Output Comparision Table

Parameters	Unit	Control Model	Retrofitted Model	Remarks
Length	m	4.267	4.267	Wall Jacketing with GI wire mesh 2.3 mm @ 25 mm both ways with 50 mm thick M20 mortar all over the wall.
Height	m	3.048	3.048	
Thickness	m	0.23	0.33	
Permissible compressive stresses	N/mm²	0.273	1.157	Capacity has increased
Model S22	N/mm²	0.639	0.316	Axial Compression decreased
Permissible Shear Stress	N/mm²	0.112	0.368	Shear Strength has increased.
Model S12	N/mm²	0.423	0.359	Shear Stress decreased
Inter Story Drift	mm	0.504	0.069	Displacement Decreased
Base Shear	KN	13.2	18.2	Base shear capacity Increased

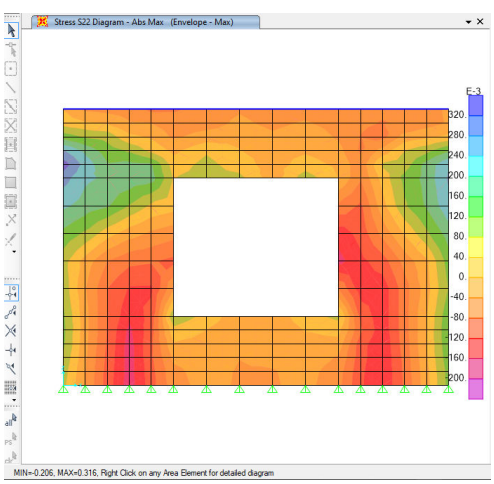

Fig. 3. Axial Stress Distribution S22 in Original Wall

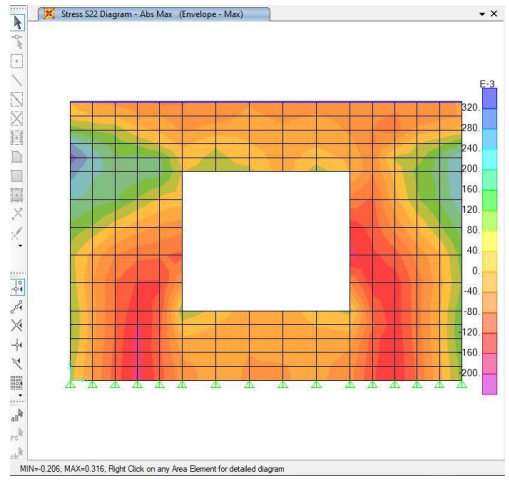

Fig. 4. Axial Stress Distribution S22 in Retrofitted Model

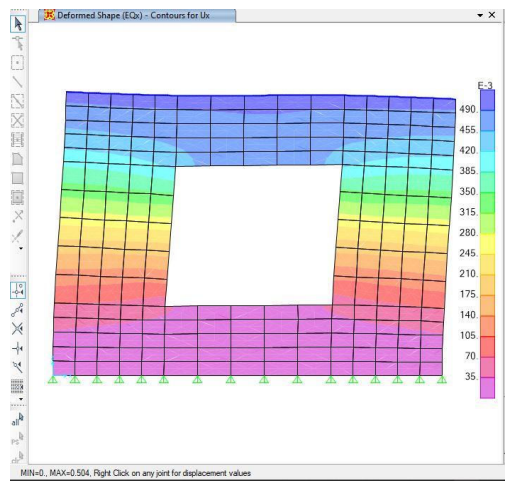

Fig. 5. Deformed Shape of Original Wall

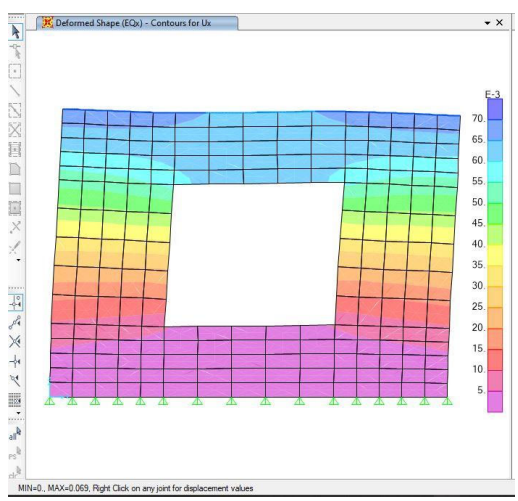

Fig. 6. Deformed Shape of Retrofitted Wall

a) The permissible compressive strength of the wall with strengthening measures is significantly higher (nearly 4.25 times) its original strength. This method of retrofitting is very suitable in case of very old masonry structures with worn out and inferior material quality.
b) The shear strength capacity of the wall increases significantly by nearly 3.25 times its original shear strength. This is due to the resistance provided by the GI wire meshing and anchorage.
c) The induced stresses decrease with an increase in area and resistance provided by the added layer of steel and concrete.
d) The inter story drift of the wall is reduced.

Limitations of the study:

a) Anchorage of the wall into the roof diaphragm has been considered integral. This frame like integrity is not achieved practically.
b) The study has not taken into consideration the contribution of concrete lining in increasing the compressive and shear strength capacity of the wall.
c) The analytical models using SAP 2000, uses equivalent steel methodology modifying the layered shell property.

IV. Conclusion

Masonry buildings have proved to be the most vulnerable to earthquake forces and have suffered maximum damage in the past earthquakes. Wall Jacketing, strengthening of plain brick masonry wall with GI mesh in M20 concrete lining, increases its flexural, compressive and shear strength significantly, and provides safety against failure in compression and shear. It shows:

1. Improvement of the existing masonry strength and deformability
2. Improvement of the in-plane strength of the wall.
3. Improvement of the structural integrity of the whole structure in terms of in-plane and out-of-plane forces that is global retrofitting.

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