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Study on Earthquake Resisting Behaviour of Low Rise Confined Masonry Building

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ABSTRACT:

Confined masonry is older construction method and found acceptable performance in past history of seismic regions. The main purpose of this dissertation is to study the earthquake resisting behavior of low rise confined masonry (CM) building.

RCC and CM are investigated by nonlinear static push over analysis by modeling various planes for RCC, CM and taking results i.e. roof displacement and base shear on Software SAP 2000 (14 Version) as per Eurocode 6 & 8 as well as IS 1893. From the investigation and calculation the response modification factor for CM is found between 2 to 3 which is as per the European standard and for RCC is found 3.12 which is nearly as per Indian standard (IS1893).

Also Response modification factor of CM with opening is reduced by 13% with CM without opening as per European standard.

KEYWORDS: *Seismi cbehavior; Confined Masonary G+2 building; response factor; Seismic design*

1. INTRODUCTION

Masonry is one of the oldest construction materials providing against environmental and natural hazard. Masonry has been used in different forms in different regions of the World, such as brick masonry, stone masonry, unreinforced brick and concrete block masonry and recently as reinforced and confined brick or block masonry. Masonry is also used extensively in construction as infill in the frame structure as a partition walls only. The masonry would continue to be used in the low to medium rise buildings because of its low cost, environmental insulation and good vertical and lateral load resistance.

In the recent 2005 Kashmir earthquake more than 4, 50,000 buildings were partially or fully damaged. Most of the buildings were non-engineered, un-reinforced masonry, rubble stone, concrete block and brick masonry buildings. Most of the deaths and injuries were the direct results of collapse of buildings. Structural configurations of low quality of masonry materials, workmanship and lack of confinement of the masonry walls were responsible for the wide spread building damage.[1]. The construction currently being practiced is considered to be non-engineered as no proper analysis and design has been carried out.

The basic feature of confined masonry structures are the vertical, reinforced-concrete or reinforced-masonry bonding elements tie-columns, which confine the walls at all corners and wall intersections as well as along the vertical borders of door and window openings. In order to be effective, tie-columns are well connected with the bond-beams along the walls at floor levels. It is generally believed that tie-columns prevent disintegration and improve the ductility of masonry when subjected to severe seismic loading. In a way, similar behavior of confined masonry is expected as in the case of reinforced concrete frames with masonry infill. However, in the case of confined masonry, tie-columns do not represent the load-bearing part of a structure. According to the requirements of recent Euro code, no contribution of vertical confinement to vertical and lateral resistance should be taken into account in the calculation. The amount of reinforcement is determined arbitrarily on the basis of experience, and depends on the height and size of the building.

2. CONFINED MASONRY

2.1 Introduction

Confined masonry construction consists of masonry walls (made either of clay brick or concrete block units) and horizontal and vertical RC *confining members* built on all four sides of a masonry wall panel.

Vertical members, called *tie-columns* or *practical columns*, resemble columns in RC frame construction except that they tend to be of far smaller cross-section. Horizontal elements, called *tie-beams*, resemble beams in RC frame construction. To emphasize that confining elements are not beams and columns, alternative terms horizontal ties and vertical ties could be used instead of tie-beams and tie-columns.

2.1.1 The structural components of a confined masonry building are as follows:-

- ***Masonry walls*** – Transmit the gravity load from the slab (s) above down to the foundation. The walls act as bracing panels, which resist horizontal earthquake forces. The walls must be confined by concrete tie beams and tie-columns to ensure satisfactory earthquake performance.
- ***Confining elements*** (tie-columns and tie-beams) – Provide restraint to masonry walls and protect them from complete disintegration even in major earthquakes. These elements resist gravity loads and have important role in ensuring vertical stability of a building in an earthquake.
- ***Floor and roof slabs*** – Transmit both gravity and lateral loads to the walls. In an earthquake, slabs behave like horizontal beams and are called diaphragms.
- ***Plinth band*** –Transmits the load from the walls down to the foundation. It also protects the ground floor walls from excessive settlement in soft soil conditions.
- ***Foundation*** – Transmits the loads from the structure to the ground.

Figure 1 shows the all component of confined masonry building,

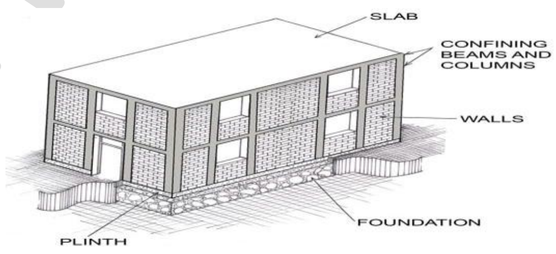

Figure 1 Components of Confined Masonry

2.2 CODE RECOMMENDATIONS FOR THE CONFINED MASONRY

In most of these countries the specification for confined masonry is part of their code or country guidelines. The specifications of the confined masonry are developed after the past earthquakes. It has been seen that the confined masonry improves both ductility and seismic resistance of the structure.

2.2.1 Specifications of Eurocode 6 & 8

According to the **Eurocode 6**:- Design of masonry structures gives some basic rules for the confined masonry as discussed below; and **Eurocode 8**:- Gives the design provisions for earthquake resistance of structures.

2.2.1.1 Construction Technique

According to Eurocode the confined masonryelement i.e. tie column (vertically) and tie beam (horizontally) should be provided to the masonry wall so that they act together during lateral

action. Concrete for confining elements should be cast after the construction of masonry wall. The confining elements should be provided at the following locations:

- At all free edges of the structural walls,
- At the walls intersection,
- Tie columns should be placed at a maximum spacing of 4 m,
- At both sides of opening having an area of more than 1.5 m2
- Tie beams should be provided at every floor level and at a vertical spacing of 4 m.

2.2.1.2 Geometric requirements in the confining masonry and area of reinforcement

1. In CM every opening having an area of more than **1.5** m2 and the maximum spacing of both horizontal and vertical is **4** m. Confining elements should have a cross-sectional area not less than **0.02** m2 with a minimum dimension of **150** mm in the plan of the wall.

2. Longitudinal reinforcements with a minimum area equal to **0.8** % of the cross-sectional area of the confining element, but not less than **200** mm2. Stirrups not less than **6** mm diameter, spacing not more than **300** mm c/c should also be provided.
3. According to additional requirements in Eurocode 8 (section 9.5.3), the minimum area of reinforcement is 300 mm2 or 1% of the cross-sectional area of the confining element .The stirrup should be provided by 5mm diameter at 150 mm c/c. The bars should be spliced at length of 60 times diameter of bar.
4. The minimum thickness of the wall should be 240 mm. The minimum effective height to thickness ratio of the wall should be 15 and length of wall to clear height of the opening (adjacent to the wall) should be 0.3.

2.2.1.3 Material Strength

The minimum compressive strength of masonry units should be not less than values as follows:

- Normal to the bed face: *f*b, min= **5** N/mm2
- Parallel to the bed face in the plane of the wall: *f*bh, mim = **2** N/mm2.

And a minimum strength is required of mortar *f*m, mim = **5** N/mm2 for confined masonry.

2.2.2 Number of stories and wall density ratio

Depending on the product ag·S means Acceleration at site and the type of construction the allowable number of storeys above ground (*n*) should be limited and walls in two Orthogonal directions with a minimum total cross-sectional area (*A*min) in each direction should be provided. The minimum cross-sectional area is as a minimum percentage (*p*A, min) of the total floor area per storey. Eurocode 8 recommends minimum number of stories depending on the seismicity of the area and wall density ratio. Table 1 gives the number of stories corresponding to minimum wall density ratio and the maximum ground acceleration.

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Table 1 Wall density ratio

Acceleration at site $a_g.S$		$\leq$0.07 k.g	$\leq$0.10 k.g	$\leq$0.15 k.g	$\leq$0.20 k.g
Type of construction	Number of storeys (n)	Minimum sum of cross-sections areas of horizontal shear walls in each direction, as percentage of the total floor area per storey ($p_{A,min}$)			
Confined masonry	2	2.0%	2.5%	3.0%	3.5%
	3	2.0%	3.0%	4.0%	n/a
	4	4.0%	5.0%	n/a	n/a
	5	6.0%	n/a	n/a	n/a

** n/a means “not acceptable”.*

In the table, k is a corrective factor based on minimum unit strength of 5 MPa for confined masonry. Where k = 1+ (lav-2)/4 $\leq$ 2 for buildings having 70% of the shear walls under consideration are longer than 2 m, however, for all other cases k = 1. In the expression lav is average wall length.

2.3 COMPARISON OF RCC FRAME AND CONFINED MASNORY CONSTRUCTION.

The appearance of a finished confined masonry construction and a RC frame construction with masonry infills may look alike to lay people, however these two construction systems are substantially different. The main differences are related to the construction sequence, as well as to the manner in which these structures resist gravity and lateral loads. These differences are summarized in Table 3.4 and are illustrated by diagrams in Figure 3.22. Examples of RC frame and confined masonry construction from Cambodia and Mexico respectively are shown in Figure 3.21

Table 2 A comparison between the confined masonry and RC frame construction.

	Confined masonry construction	**RC frame construction**
Gravity and lateral load	Masonry walls are the main load bearing elements and are expected to	RC frames resist both gravity and lateral loads through their

resisting system	resist both gravity and lateral loads. Confining elements (tie-beams and tie columns) are significantly smaller in size than RC beams and columns.	relatively large beams, columns, and their connections. Masonry infills are not load-bearing walls.
Foundation construction	Strip footing beneath the wall and the RC plinth band	Isolated footing beneath each column
Superstructure construction sequence	1. Masonry walls are constructed first. 2. Subsequently, tie-columns are cast in place. 3. Finally, tie-beams are constructed on top of the walls, simultaneously with the floor/roof slab construction.	1. The frame is constructed first. 2. Masonry walls are constructed at a later stage and are not bonded to the frame members; these walls are non-structural, that is, non-load bearing walls.

3. RESPONSE MODIFICATION FACTOR

3.1 DEFINITION OF R FACTOR AND ITS COMPONENTS

As already discussed, R factors are essential seismic design tools, which defines the level of inelasticity expected in structural systems during an earthquake event. The commentary to the provisions defines R factor as "…factor intended to account for both damping and ductility inherent in structural systems at the displacements great enough to approach the maximum displacement of the systems." This definition provides some insight into the understanding of the seismic response of buildings and the expected behavior of a code-compliant building in the design earthquake. R factor reflects the capability of structure to dissipate energy through inelastic behavior. R factor is used to reduce the design forces in earthquake resistant design and accounts for damping, energy dissipation capacity and for over-strength of the structure. Conventional seismic design procedures adopt force-based design criteria as opposed to displacement-based. The basic concept of the latter is to design the structure for a target displacement rather than a strength level. Hence, the deformation, which is the major cause of damage and collapse of structures subjected

Hence, the role of the force reduction factor and the parameters influencing its evaluation and control are essential elements of seismic design according to codes. The values assigned to the response modification factor (R) of the US codes are intended to account for both reserve strength and ductility. Some literature also mentions redundancy in the structure as a separate parameter. But in this study, redundancy is considered as a parameter contributing to over strength, contrary to the proposal of, splitting R into three factors: strength, ductility and redundancy. The philosophy of earthquake resistant design is that a structure should resist earthquake ground motion without collapse, but with some damage. Consistent with this philosophy, the structure is designed for much less base shear forces than would be required if the building is to remain elastic during severe shaking at a site**.** Such large reductions are mainly due to two factors: (1) the ductility reduction factor (Rμ), which reduces the elastic demand force to the level of the maximum yield strength of the structure, and (2) the over strength factor, (Ω), which accounts for the overstrength introduced in code-designed structures. Thus, the response reduction factor (R) is simply Ω times Rμ. See Figure 4.2.

$$R = R\mu \times \Omega \qquad (1)$$

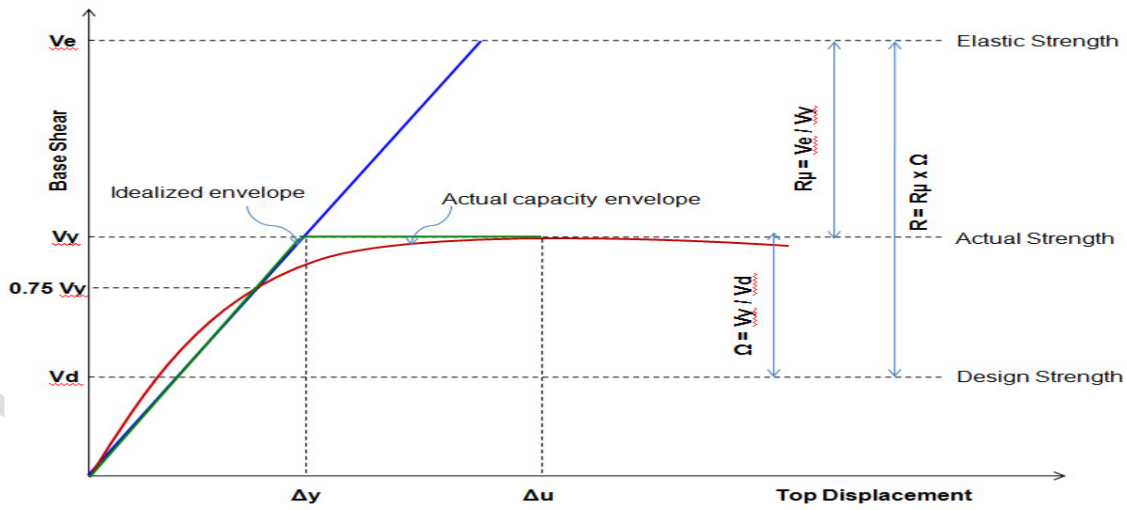


Figure 2 Relationship between force reduction factor (R), structural overstrength (Ω), and ductility reduction factor (Rμ)

3.1.1 Ductility Reduction Factor (Rμ)

The ductility reduction factor (Rμ) is a factor which reduces the elastic force demand to the level of idealized yield strength of the structure and, hence, it may be represented as the following equation:

$$R\mu = Ve / Vy \qquad (2)$$

Ve is the max base shear coefficient if the structure remains elastic. The ductility reduction factor (Rμ) takes advantage of the energy dissipating capacity of properly designed and well-detailed structures and, hence,

3.1.2 Structural Overstrength (Ω)

Structural overstrength plays an important role in collapse prevention of the buildings. The overstrength factor (Ω) may be defined as the ratio of actual to the design lateral strength:

$$\Omega = Vy / Vd \qquad (3)$$

Where Vy is the base shear coefficient corresponding to the actual yielding of the structure; Vd is the code-prescribed unfactored design base shear coefficient.

Finally Response Modification Factor (R) can be found by using following equation will becomes from Eq. 1

$$R = Ve / Vd$$

Where Ve = Maximum Base Shear

Vd = Design Base Shear.

4. METHOD OF ANALYSIS

4.1 PLAN OF RCC AND CONFINED MASONRY BUILDING

Different types of building models are taken into consideration and subjected to the Push over analysis to evaluate Base Shear and Displacement curve and after that finding R factor. Eight building models i.e. four for CM and four for RCC and their variations of G+1 story with and without opening and 2 bays respectively with height 4m according to Euro code. Plans for CM and RCC are same for G+ 1for the analysis as shown in following and details of member element are also shown. Design of member i.e. Beam and Column for RCC are as per IS 456-2000 and Eurocode. Special Provision to design member for RCC and CM are given in SAP2000.

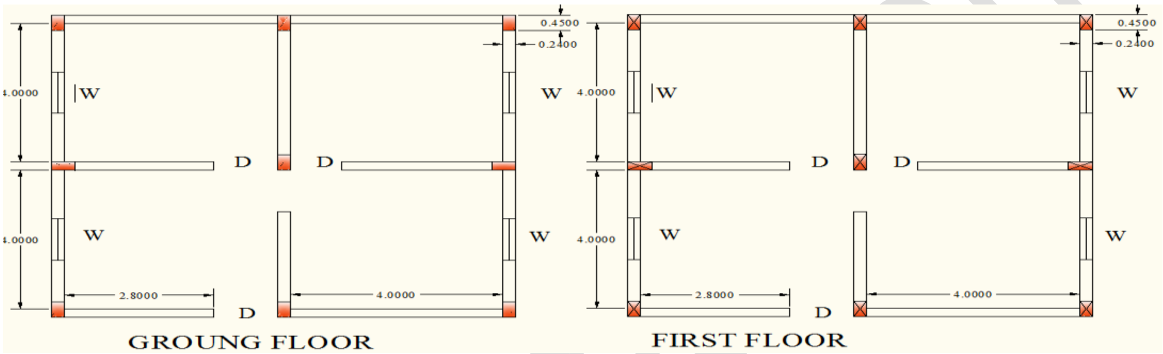


Fig 3 Typical G+ 1 Story for CM as well as RCC

4.2 NON-LINEAR STATIC ANALYSIS

Non-linear static analysis (pushover analysis), has been developed over the past years and has become a useful analysis procedure for design and performance evaluation purposes. Since the procedure is relatively simple, it does involve certain approximations and simplifications so that some amount of variation is always expected to exist in seismic demand evaluation. The function of the pushover analysis is to evaluate the expected performance of a structural system by estimating its strength and deformation demands in design earthquakes by means of a static inelastic analysis, and comparing these demands to available capacities. Pushover analysis can be viewed as a tool for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces occurring when the structure is subjected to inertia forces that no longer can be resisted within the elastic range of structural behaviour.

In the recent guidelines the seismic demands are computed by non-linear static analysis of the structure subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached. Both the force distribution and target displacement are based on the assumption that the response is controlled by the fundamental mode and that the mode shape remains unchanged which both assumptions are approximate after the structure yields.

4.2.1 Process of Non-linear Static Analysis

A three dimensional mathematical model of the structure which includes load-deformation relationship of all members is first created and gravity loads are applied first. A lateral load pattern which is distributed along the building's height is then applied. In this particular study the lateral load pattern is selected as the first mode shape of the structure. The lateral forces are increased in a step by step fashion until a member yields (plastic hinge occurrence). The model is then modified to account for the change in stiffness of yielded member and lateral forces are increased until additional members yield. The process is continued until the control displacement reaches a certain level or structure becomes a mechanism which is unstable. In this particular study the typical end state of the analysis was the mechanism condition as to investigate the full capacity of the system. However in some cases to prevent occurrence of further excrescent results the target displacement is, at most, kept 3% of

4.2.2 Force Deformation Relationships

In the force deformation relationships for individual members, the basic relationship is often represented by concentric plastic hinges assigned to desired locations along the frame members. As its most probable that the yielding will occur at the ends of the members which are subjected to lateral loads, the plastic hinges are assigned to those locations. Yielding and post-yielding behavior can be modeled as a moment rotation curve for flexural yielding (typical for beam members), as a three dimensional axial force – bending moment interaction for column members or as an axial force – axial deformation curve for brace members.

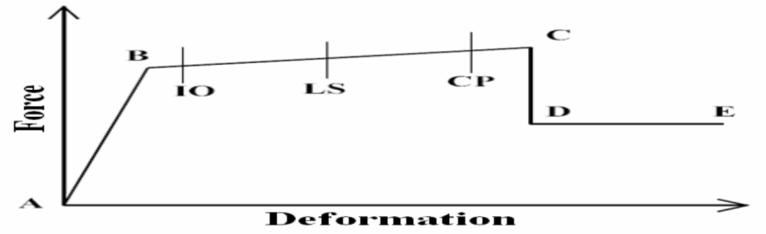


Figure 4. Component Force-Deformation Curve

A generic component behavior curve is represented in Figure 5.3. The points marked on the curve are expressed by the software vendor [69] as follows:

- Point A is the origin.
- Point B represents yielding. No deformation occurs in the hinge up to point B regardless of the deformation value specified for point B. The deformation (rotation) at point B will be subtracted from the deformations at points C, D, and E. Only the plastic deformation beyond point B will be exhibited by the hinge.
- Point C represents the ultimate capacity for pushover analysis. However, a positive slope from C to D may be specified for other purposes.
- Point D represents a residual strength for pushover analysis. However, a positive slope from C to D or D to E may be specified for other purposes.
- Point E represents total failure. Beyond point E the hinge will drop load down to point F (not shown) directly below point E on the horizontal axis.

If it is not desired that the hinge to fail this way a large value for the deformation at point E may be specified. One can specify additional deformation

measures at points IO (immediate occupancy), LS (life safety), and CP (collapse prevention). These are informational measures that are reported in the analysis results and used for performance-based design. They do not have any effect on the behavior of the structure.

4.3 DESIGN BASE SHEAR CALCULATION

The total design lateral force or design seismic base shear (V_d) along any principal direction shall be determined by the following expression as per IS 1893: 2002

$V_d = A_h W$

Where

A_h = Design horizontal acceleration spectrum value as per clause 6.4.2 using the fundamental natural period *Ta* as per clause 7.6 in the considered direction of vibration.

W= Seismic weight of the building as per clause 7.4.2.

4.3.1 The design horizontal seismic coefficient

The design horizontal seismic coefficient A_h for a structure shall be determined by the following expression:

$A_h =$

Provided that for any structure with $T \leq 0.1$ s the value of A_h will not be taken less than Z/2 whatever be the value of *I/R*

Where

Z = Zone factor given in Table 2 of IS 1893:2002, is for the Maximum Considered Earthquake (MCE) and service life of structure in a zone.

I = Importance factor, depending upon the functional use of the structures,

R = Response reduction factor, depending on the perceived seismic damage Perfonnance of the structure, characterized by ductile or brittle deformations. However, the ratio *(I/R)* shall not be greater than 1.0. The values of *R* for buildings are given in Table 7.

Sa/*g* = Average response acceleration coefficient.

4.4 ANALYTICAL ANALYSIS

An attempt is to study behavior of CM and RCC building with and without opening. 3D models of building are developed and analysis is carrying. This study is investigate response of CM and RCC building with and without opening based on static push over analysis. Four models of CM and four models of RCC are made each model loading as per IS 875: 1987 Part I (dead load) and part II (live load) and details of member material as per following.

Beam and Column (450mm x 240mm)

Concrete M20

Weight per unit volume = 25 KN/m3

Modules of elasticity = 2 x 105 KN /m2

Poisons ratio = 0.3

Concrete compressive strength = 20 x 103 KN /m2

Steel Fe250

Weight per unit volume = 78.5 KN/m3

Modules of elasticity = 2 x 105 KN /m2

Poisons ratio = 0.15

Minimum yield stress = 250 x 103 KN / m2

Wall

Brick

Weight per unit volume = 20 KN/m3

Modules of elasticity = 5.5 x 105 KN /m2

Poisons ratio = 0.15

4.4.1 Calculation of design base shear

By considering G+1 building for RCC and CM located at zone V having dead load is 8.7 KN/m at roof/floor level according to IS 875 part I (table 02) and live load 2 KN/m at roof/floor according to IS 875 part II (table 1).

4.4.1.1 Design base shear for CM Building

Floor Area (8X8) = 64 m2

Live load = 3 KN/mDead load = 8.7 KN/m (Including self-weight)

Weight on floor W1= 64 x (8.7 + 0.25x3) = 604.8 KN

Weight on roof W2= 64 x 8.7 = 556.8 KN

Total weight on building (W) = 2W1 + W2 = 2 x 604.8 + 556.8 = 1766.4 KN

Ah = (Z/2) x (I/R) x (Sa/g)

Z = 0.36 as per clause 6.4.2

R = 2.5 as per clause 6.4.2

T = 0.09 x 8 / √ 8 = 0.25

Sa/g = 2.5 for medium soil

Ah = (Z/2) x (I/R) x (Sa/g) = (0.36/2) x (1/2.5) x (2.5) = 0.18

Design Base shear is given as

Vd = AhW = 0.18 x 1766.4 = 317.95 KN

Vd= 317.95 KN

Similarly

Design base shear for RCC Building is **Vd= 264.96 KN**

4.4.2 Calculation of yielding or maximum base shear

Yielding or maximum base shear is depend on Component of force-deformation curve i.e. A,B,IO,LS,CP,C,D and E as discussed in 5.2.2. If the maximum hinges form in between IO (immediate occupancy) to LS (life safety) then at that step yielding base shear value is be taken.

Following tables shows the all values of components of different RCC and CM with and without opening and design by IS 456 and Euro Code.

Table 3 Maximum or Yielding base shears

Condition of the building	**Types of building and code used**	**Max. Base Shear (V_e)**
With Opening	CM (Eurocode)	782.375
	CM (IS 1893)	823.006
	RCC (Eurocode)	874.42
	RCC (IS 1893)	826.993
Without Opening	CM (Eurocode)	680.061
	CM (IS 1893)	822.645
	RCC (Eurocode)	827.245
	RCC (IS 1893)	789.219

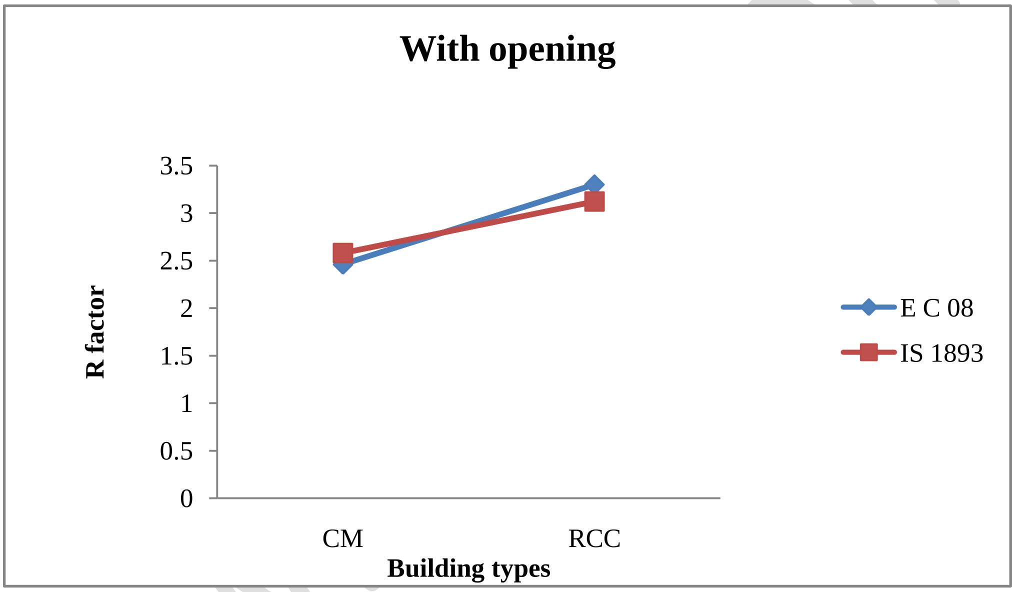
Table 4 R factor with opening

OPENING					
		Eurocode		**IS 1893**	
Building	**Design Base Shear (Vd)**	**Maximum Base shear (Ve)**	**R factor**	**Maximum Base shear (Ve)**	**R factor**
CM	317.95	782.38	2.46	823.01	2.58
RCC	264.96	874.42	3.3	826.99	3.12

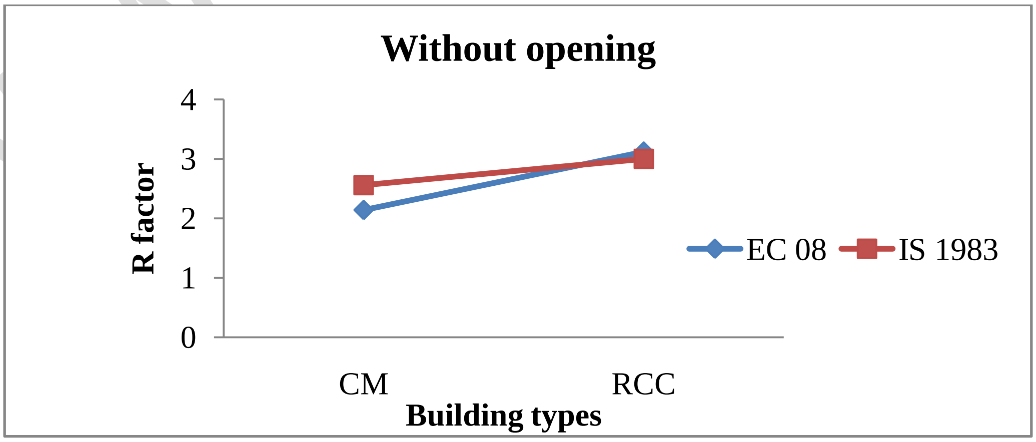
Table 5 R factor without opening

WITHOUT OPENING					
		Eurocode		IS 1893	
Building	**Design Base Shear (Vd)**	**Maximum Base shear (Ve)**	**R factor**	**Maximum Base shear (Ve)**	**R factor**
CM	317.95	680.06	2.14	822.65	2.56
RCC	264.96	827.25	3.12	789.22	3.00

Graph 6.9 R factor for both CM and RCC with opening



Graph 6.10 R factor for both CM and RCC without opening



CONCLUSION

This paper explored analytically the response modification factor (R factor) and roof-displacement curve for RCC and Confined Masonry Building

Four models of RC buildings and four models of Confined Masonry with and without opening studied to evaluate the R factor for RCC and CM in India Standard as well as Eurocode. The study involved analysis by inelastic static pushover. It was found from analysis that the R factor for both RCC and CM by varying code is nearly same.

It was also found that a single value of R factor as suggested in IS 1893:2002 for RCC may become reduced by 10%. The following are the conclusion are drawn from the study:

The response modification factor for CM is found 2.46 which satisfied European standard range and found increased base shear and maximum defection at story level due to opening, there is minimum effect of opening on R factor.

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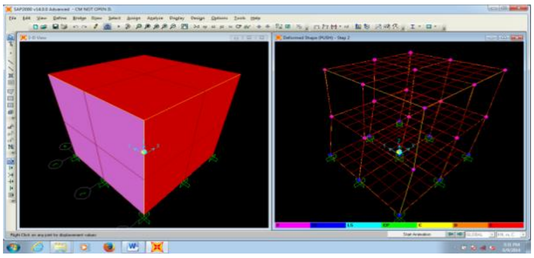
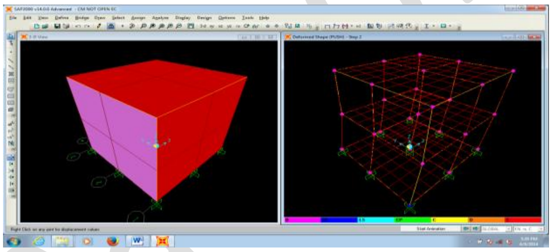
SNAPSHOT

1. RCC AND CM BUILDING SNAPSHOT

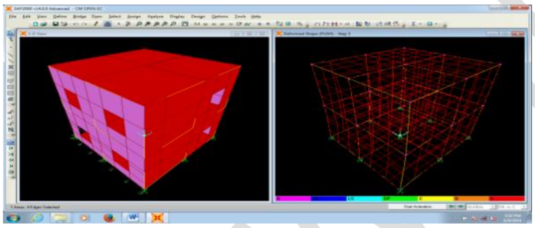
Following snapshot are shown the both regular plan (left) and Push Over analysis deformed shape (right) of different storey condition.



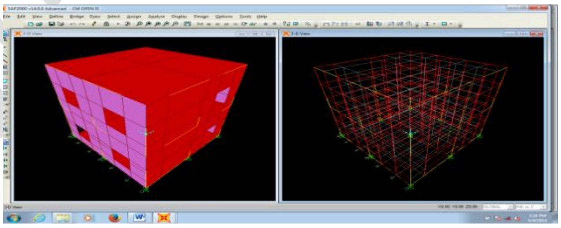
1.1 CM without opening (Eurocode)



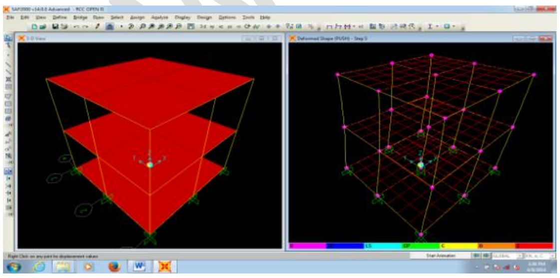
1.2 CM without opening (IS 1893)



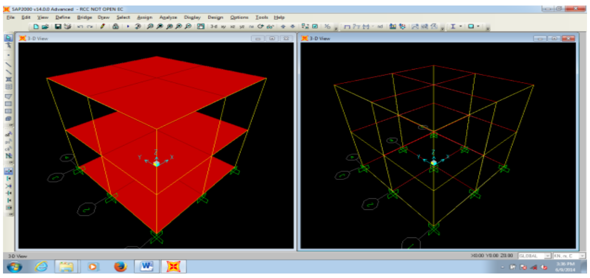
1.3 CM with opening (Eurocode)



1.4 CM with opening (IS 1893)



1.5 RCC without opening (Eurocode)



1.6 RCC without opening (IS 1893)

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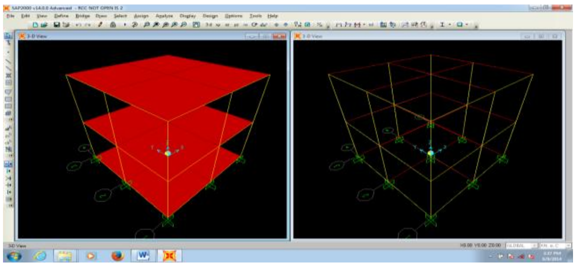
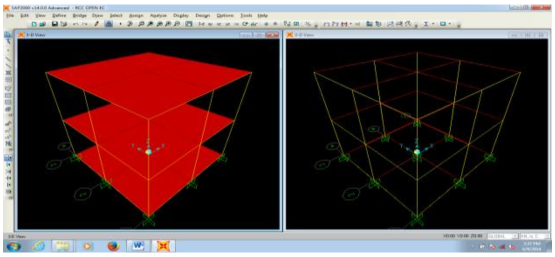


1.7 RCC with opening (Eurocode)



1.8 RCC with opening (IS 1893)