Seismic behaviour, design and detailing of RC shear walls, Part I: Behaviour and strength

Manoj S. Medhekar and Sudhir K. Jain

Shear walls offer an economic means to provide lateral load resistance in multi-storey buildings. Their seismic behaviour, modes of failure, and the factors influencing their structural response are discussed. Expressions are developed to estimate the flexural strength of slender rectangular shear wall sections with uniformly distributed vertical reinforcement. These expressions are consistent with the provisions of IS:456-1978. The axial load-moment interaction diagram for the section is also developed from these expressions. An alternative numerical method is also outlined for the same.

Reinforced concrete (RC) shear walls are used in buildings to resist lateral forces due to wind and earthquakes. They are usually provided between column lines, in stair wells, lift wells, and in shafts that house other utilities. Shear walls provide lateral load resistance by transferring the wind or earthquake loads to the foundation. Besides, they impart lateral stiffness to the system and also carry gravity loads. A well-designed system of shear walls in a building frame improves its seismic performance significantly. This is evident from studies on the comparative behaviour of framed buildings and building frames with shear walls in past earthquakes[1].

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IS:456-1978 incorporates some provisions for design of reinforced concrete walls[2]. However, no explicit provisions are given for calculating the flexural strength of shear wall sections which are quite different from beam sections as they have reinforcement distributed along their whole length in plan. Extensive experimentation has been carried out abroad to assess the strength and behaviour of RC shear walls under monotonic and reversed cyclic loading. The results have been used in arriving at appropriate design and detailing provisions in the building design codes used in the U.S.A.[3,4,5]

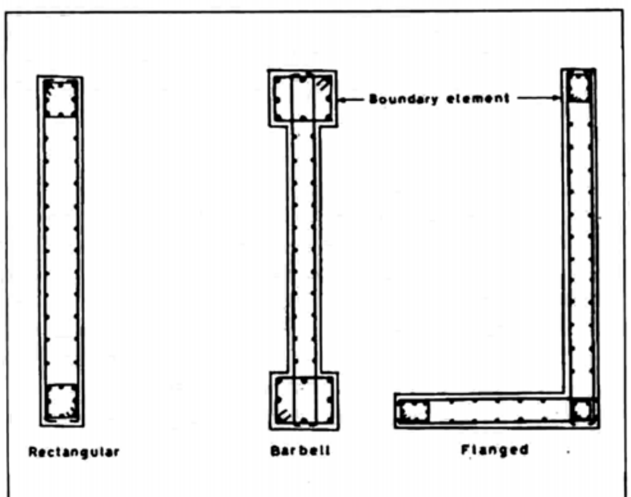

Fig 1 Types of wall section

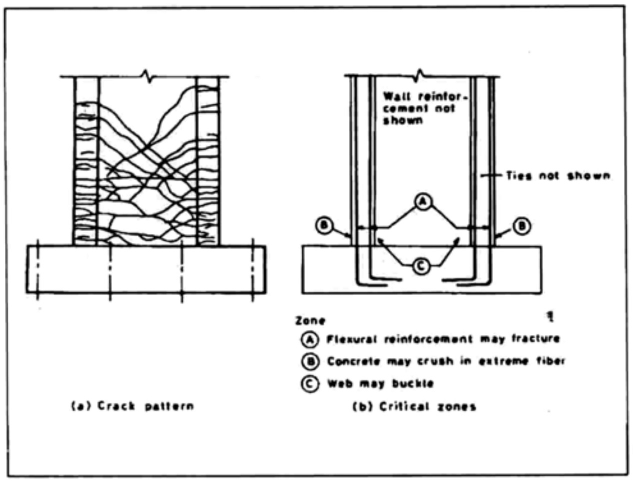

Fig 2 Crack pattern and critical zones in slender walls

Canada[6], and New Zealand[7]. However, these codes use a different design philosophy and hence cannot be directly adopted for use in India.

This paper summarises the behaviour of reinforced concrete shear walls under lateral loading. Based on available literature, the modes of failure and the factors influencing the structural response of shear walls are discussed. A method is described to calculate the flexural strength and generate the axial load-moment interaction curve for slender rectangular shear walls with uniformly distributed vertical reinforcement. An alternative numerical approach is also outlined for the same. Part II of this paper describes the design and detailing provisions for ductile earthquake-resistant shear walls along with one worked out example[8].

Behaviour under seismic loading

Shear walls usually have a rectangular cross section, *Fig* 1. When a wall is provided monolithically between two columns, a barbell shape results. The columns that are present at either ends of the wall are termed as 'boundary elements'. They increase the strength of the wall in flexure and shear significantly. Flanged wall sections result due to intersecting walls.

Depending on the height-to-width ratio, a shear wall may behave as a slender wall, a squat wall, or a combination of the two. Slender shear walls usually have a height-to-width ratio greater than 2. They behave like a vertical slender cantilever beam. The primary mode of deformation is bending; shear deformations are small and can be neglected. Flexural strength usually governs the design of such walls. Squat shear walls usually have a height-to-width ratio less than half. They show significant amount of shear deformation as compared to bending deformation. Shear strength usually governs the design of such walls.

During a severe earthquake, a shear wall that has very high strength may respond in a fully elastic manner. However, it is uneconomical to construct such walls. Ideally, shear walls should respond in a ductile manner. This can be achieved by proper detailing so as to make them capable of undergoing large inelastic deformations and dissipating seismic energy. Sometimes, the wall foundation may not be anchored adequately. This will limit the lateral load capacity of the wall to its overturning capacity. Such walls may 'rock' on their foundation during severe ground shaking.

Slender and squat shear walls have distinct modes of failure. These failure modes are described below.

Failure modes in slender walls

Slender walls are governed by their flexural strength. They are usually subjected to low nominal shear stress. They develop a predominantly horizontal crack pattern in the lower hinging region after a few cycles of inelastic deformation, *Fig* 2a. After yield of the vertical reinforcement, shear is resisted by interface friction across cracks and dowel action of the vertical reinforcement. The flexural strength of such walls is limited by fracture of main flexural reinforcement that is provided near the wall edges, concrete crushing in the compression zone, or lateral instability of the compression zone, *Fig* 2b.

Fracture of the main flexural reinforcement takes place due to low cycle fatigue caused by alternate tensile yielding and inelastic compressive buckling of the bars. Bauschinger effect also makes the compression steel buckle earlier than in comparable monotonically loaded specimens. The provisions of adequate transverse confining reinforcement to the main flexural reinforcement near the outer edges of the wall and in boundary elements can delay the onset of buckling. It will also contain the cracked concrete and prevent it from falling away, thereby preventing loss of lateral support to the main flexural reinforcement.

Failure due to concrete crushing occurs when the crushing strain of concrete is exceeded in the compression zone of the wall. This is usually accompanied by buckling of the main flexural reinforcement. Wall sections are usually under-reinforced and so concrete crushing is not expected. However, such a failure can occur in slender rectangular walls that have a large percentage of vertical reinforcement and carry a large axial load. Unsymmetrical wall sections, such as *T*, *L*, etc., are heavily stressed in the compression zone and may also fail by this mode.

Reversed cyclic loading may cause the effective moment of inertia of the wall section to reduce to that of the steel area alone. This will reduce stability of the wall against out-of-plane displacements and may cause lateral buckling of the compression zone.

Failure modes in squat walls

Squat walls are governed by their shear strength. They are usually subjected to high nominal shear stress. They develop inclined cracks in the web that form a diagonal compression strut system for each direction of loading, *Fig* 3a. Shear transfer takes place by the truss action which provides a stiffer system than that for slender walls. The shear strength of such walls is limited by diagonal tension, sliding shear, or diagonal compression *Fig* 3b.

A diagonal tension failure takes place when there is inadequate horizontal shear reinforcement to arrest the growth of diagonal tension cracks that are caused by the shear force.

Failure due to sliding shear occurs when cracks that develop under reversed cyclic loading intersect to form predominantly horizontal planes along which shear transfer is lost after several cycles of inelastic loading. The tendency to fail in sliding shear increases with an increase in nominal shear stress and with a reduction in axial compression and height-to-width ratio of the wall. Provision of well confined vertical boundary elements along the wall edges and the use of diagonal reinforcement in the web (*Fig* 4) will prevent this type of failure[9].

Squat walls may also fail due to diagonal compression or web crushing. The inclined cracks that develop under load reversals form diagonal compressive strut system in each direction of loading. As the load reversals progress, a small zone of high compressive stress develops in the web where crushing of concrete takes place. This is at the focal point of the diagonal struts. Web crushing is a function of concrete compressive strength and maximum shear distortions applied to the wall. By imposing a limit on the maximum shear stress developed in the web, a ductile failure mode involving yielding of vertical flexural reinforcement can be achieved prior to crushing of the web[9].

Factors influencing structural response of walls

The various parameters that influence the response of shear walls are as follows:

Height-to-width ratio

As already discussed, walls with height-to-width ratio exceeding 2 behave like a bending beam while those with a smaller height- to-width ratio behave like a shear beam. For walls having a height-to-width ratio less than 1.0, vertical reinforcement is seen to be more effective than horizontal reinforcement in resisting the horizontal shear force.

Type of loading

Slender shear walls show less flexural strength and deformation under reversed cyclic loading as compared to monotonic loading. Flexural strength is reduced due to the effect of inelastic buckling and Bauschinger effect on the tensile capacity of the reinforcement. Squat shear walls do not show any significant difference in behaviour under monotonic and reversed cyclic loading. This is due to formation of the diagonal compressive strut system that does not deteriorate appreciably in shear stiffness under reversed loading. Cracking and energy dissipation increase in both types of wall under cyclic loading.

Flexural reinforcement

The vertical reinforcement present in the wall determines the flexural capacity and hence the maximum shear force to which it can be subjected. The maximum shear force that can be developed depends on the actual flexural capacity which may be significantly higher than the design flexural capacity due to strain hardening of the vertical reinforcement. The distributed vertical reinforcement may also have been neglected in the calculation of the design flexural capacity. Thus, shear force that can be induced during inelastic response may be significantly higher than expected.

For the same amount of vertical reinforcement, walls having reinforcement concentrated in the boundary elements at their edges develop higher flexural capacity and ultimate curvature than walls having uniformly distributed reinforcement.

Shear reinforcement

Horizontal shear reinforcement is provided to prevent diagonal tension failure. It improves the inelastic response of walls subjected to high nominal shear stress by reducing shear deformation. It is ineffective in resisting sliding shear and does not influence the web crushing strength significantly.

Diagonal reinforcement

The use of diagonal reinforcement in the web of walls (*Fig* 4) reduces shear distortion and resists sliding shear. It is particularly useful in squat shear walls[9,10]. It also contributes to flexural strength and results in increased energy dissipation capacity.

Special transverse reinforcement

This is required in boundary elements that lie in the potential hinging region of the wall where large inelastic rotations are likely to occur. Tests on slender walls indicate that this hinging region extends above the wall base for a height equal to the horizontal length of the wall[11]. It may also occur at discontinuities resulting from strength taper, changes in geome-

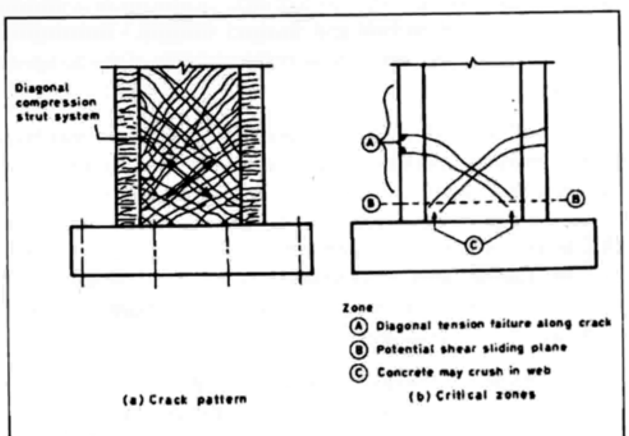

Fig 3 Crack pattern and critical zones in squat walls

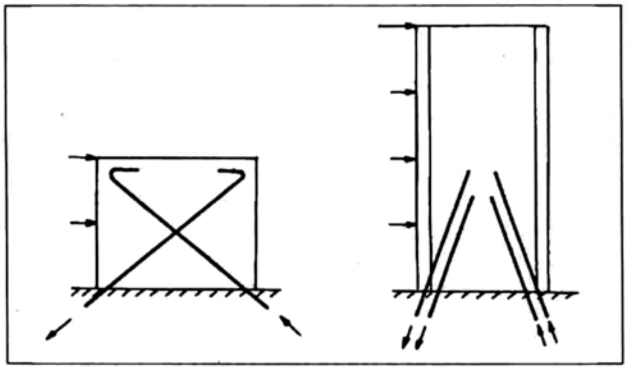

Fig 4 Diagonal reinforcement in walls

try, or the effect of higher mode inertial forces. Special transverse reinforcement serves four functions :

- *(i)* it provides lateral confinement to concrete in the boundary elements, thereby increasing the crushing strain of concrete
- *(ii)* it supports the vertical reinforcement against inelastic buckling,
- *(iii)* it forms a 'basket' along with the vertical bars which holds the cracked concrete core together, and
- *(iv)* it improves the shear capacity and stiffness for dowel action of the boundary elements.

Concrete strength

This affects the extreme fibre compressive capacity, web crushing capacity, and shear strength of concrete. A low concrete strength results in low deformation capacity of the wall.

Section shape

Rectangular walls have less flexural strength as compared to equivalent flanged or barbell shaped walls. This is because limited vertical reinforcement can be placed near the edges of the wall. As a result, for the same moment-to-shear ratio, the level of shear stress in the web of rectangular walls is usually less than that in barbell and flanged sections. Rectangular walls are also more prone to lateral instability of the compression zone.

Barbell-shaped walls have boundary elements that have large in-plane and out-of-plane stiffness. Greater resistance is provided against sliding shear due to dowel action of the boundary elements. Such walls have greater flexural strength as more vertical reinforcement is concentrated at the wall edges. Consequently, high shear stresses can develop in such sections. The shear resistance of such sections is usually limited by web crushing.

Flange-shaped walls may also develop high shear stresses. Web crushing usually limits the shear capacity of such sections. The compression boundary element in such walls tends to shear through after web crushing.

Axial compressive stress

The presence of moderate axial compressive load on a wall that is loaded monotonically or under reversed cyclic loading results in an increase in its flexural capacity and shear strength. Axial compression reduces shear distortion and increases the shear stiffness of the hinging region.

Construction joints

Sliding shear failures have been observed in poorly constructed joints. They perform satisfactorily if the surface of previously cast concrete is roughened and cleaned so as to remove loose particles and laitance.

Coupled shear wall system

A shear wall may contain one or more vertical rows of openings. Shear cores used in multistorey buildings typically have openings for access to service shafts. The walls on either side of the openings are thus 'coupled' together by beams. Such a system is called a coupled shear wall, *Fig* 5.

When lateral loads are applied to a coupled shear wall system, moment reactions are developed at the base of each wall. In addition, a couple is also formed by the axial forces in each wall that are carried across the coupling beams by vertical shear. The coupling beams usually have small span-to-depth ratio making the shear deformations in them significant. In a severe earthquake, the ends of the coupling beam are subjected to large rotational and vertical displacements. These sections need to have adequate ductility. Properly detailed coupling beams can dissipate large amount of energy by formation of plastic hinges at their ends before any hinging occurs at the wall base.

Coupling beams that contain inadequate shear reinforcement fail in diagonal tension. Such a brittle failure results in

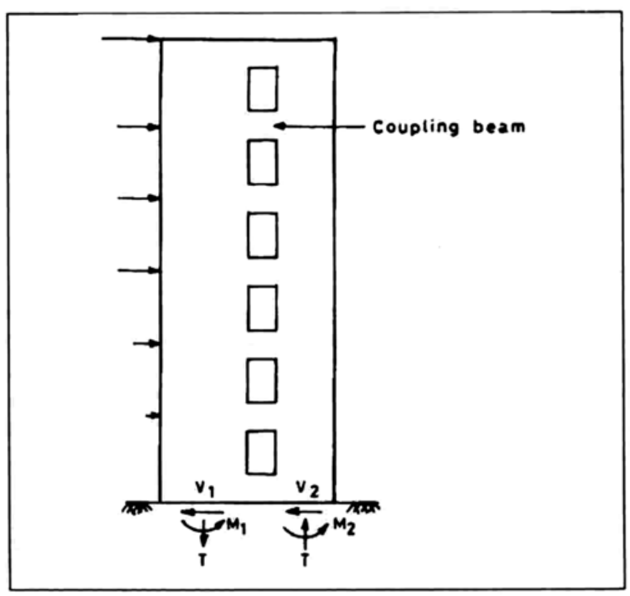

Fig 5 Coupled shear wall

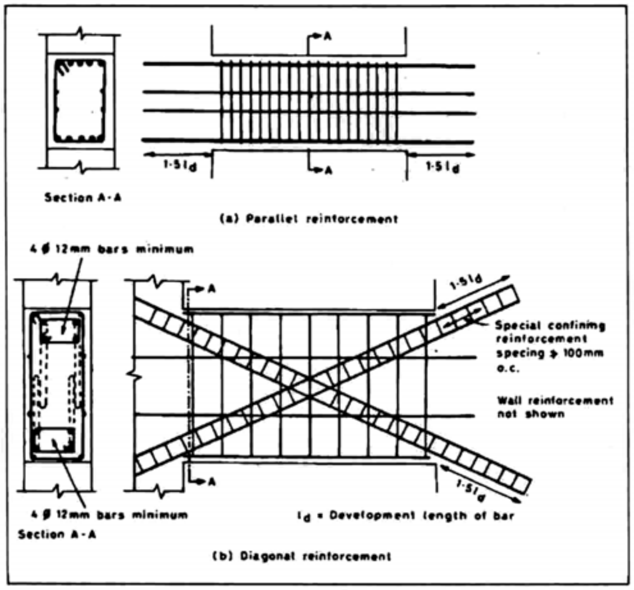

Fig 6 Reinforcement details for coupling beam

rapid strength degradation under cyclic loading. The shear strength of the coupling beam must be greater than the shear force that can be developed when plastic hinges form at its two ends[12]. This requires a limit to be imposed on the tension steel content in such beams. Coupling beams that have a small span-to-depth ratio and are provided with adequate shear reinforcement in the form of vertical stirrups may also fail due to sliding shear along the critical support section.

Reinforcement in the coupling beam may consist of parallel reinforcement or full length diagonal reinforcement, *Fig* 6. Parallel reinforcement when adopted must be well anchored into the wall. Closely spaced stirrups must be provided over the full length of the coupling beam to confine the concrete and provide adequate shear strength. Diagonal reinforcement is more effective in coupling beams as compared to parallel reinforcement[13]. The diagonal bars must also be well anchored into the wall and restrained over their full length to prevent buckling.

Flexural strength of rectangular wall sections

The flexural strength of slender rectangular shear wall sections containing uniformly distributed vertical reinforcement and subjected to axial load can be derived by using the same assumptions as for reinforced concrete beams[14]. The stress-strain curves assumed herein are shown in *Fig* 7. Consider the cross section of a rectangular wall that is subjected to combined uni-axial bending and axial load (*Fig* 8a). The distributed vertical reinforcement is represented by an equivalent steel plate along the full length of the wall. Two types of flexural failure may take place in the section. A flexural tension failure takes place when the tension steel yields prior to crushing of concrete in the extreme compression fiber. A flexural compression failure takes place when the tension steel does not yield while the concrete crushes at the extreme compression fibre.

(a) Flexural tension failure: The assumed strain distribution for this condition is shown in *Fig* 8b. This implies that the axial load producing failure is smaller than that which produces simultaneous crushing of concrete and yielding of tension steel in the extreme fibres. The various forces and their lever arm with respect to the bottom fibre are as follows (*Figs* 8c and 8 d).

Force	*Lever arm*
$C_c = 0.36 f_{ck}\, x_u\, t_w$	$l_w - 0.416\ x_u$
$C_1 = 0.87 f_y\, t_w\, \rho\, x_u\, (1-\beta)$	$l_w - 0.5\ x_u\ (1-\beta)$
$C_2 = 0.435\ f_y\, t_w\, \rho\, \beta\, x_u$	$l_w - x_u\ (1 - \frac{2}{3}\beta)$
$T_1 = 0.435\ f_y\, t_w\, \rho\, \beta\, x_u$	$l_w - x_u\ (1 + \frac{2}{3}\beta)$
$T_2 = 0.87 f_y\, t_w\, \rho\, [\, l_w - x_u\, (1+\beta)\,]$	$0.5\, [\, l_w - x_u\, (1+\beta)\,]$
P_u	$0.5\ l_w$

where f_{ck} = characteristic strength of concrete cube; f_y = yield stress of reinforcement; l_w = horizontal length of wall; t_w = thickness of web; x_u = depth of natural axis from extreme compression fibre; ρ = vertical reinforcement ratio = $A_{st}\ /(t_w\ l_w)$; A_{st} = area of uniformly distributed vertical reinforcement; β = 0.87 f_y /(0.0035 E_s); E_s = elastic modulus of steel; and P_u = axial compression on wall.

For equilibrium of forces

$$C_c + C_1 + C_2 = T_1 + T_2 + P_u \tag{1}$$

By substituting the necessary expressions into equation (1), the non-dimensional depth of the neutral axis is obtained as

$$\frac{x_u}{l_w} = \left(\frac{\varphi + \lambda}{2\varphi + 0.36} \right) \tag{2}$$

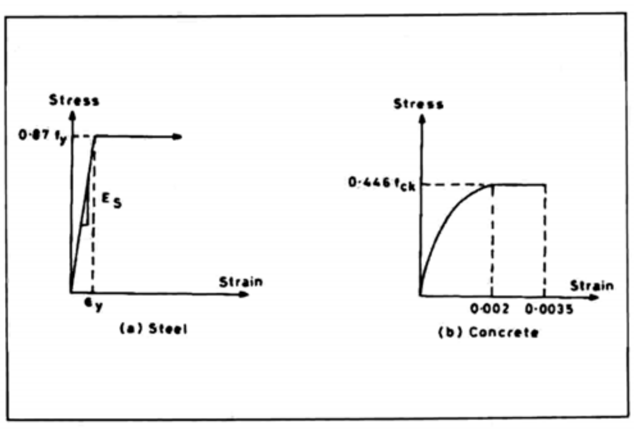

Fig 7 Stress-strain curves

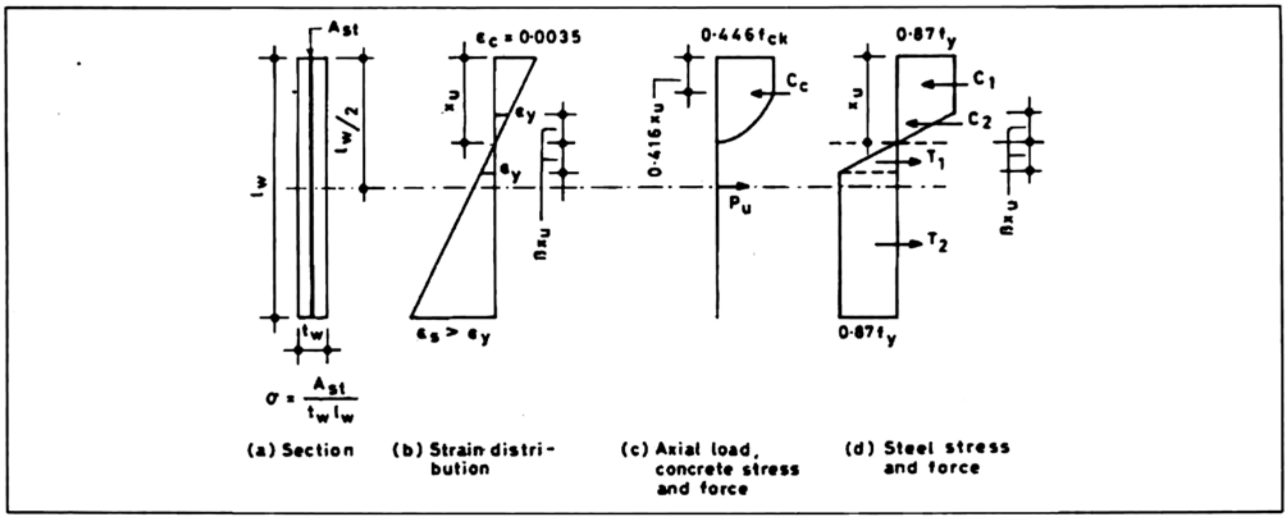

Fig 8 Flexural strength of rectangular wall section

where,

$$\varphi = \left(\frac{0.87 f_y \rho}{f_{ck}}\right); \text{ and } \lambda = \left(\frac{P_u}{f_{ck}\, t_w\, l_w}\right) \qquad (3)$$

By summing the moment of the forces about the bottom fibre, the flexural strength is obtained as

$$\frac{M_{uw}}{f_{ck}\, t_w\, l_w^2} = \varphi\left[\left(1 + \frac{\lambda}{\varphi}\right)\left(\frac{1}{2} - 0.416\frac{x_u}{l_w}\right) - \left(\frac{x_u}{l_w}\right)^2\left(0.168 + \frac{\beta^2}{3}\right)\right] \qquad (4)$$

This equation is valid when the non-dimensional depth of the neutral axis is less than $\frac{x_u^*}{l_w}$ where

$$\frac{x_u^*}{l_w} = \left(\frac{0.0035}{0.0035 + 0.87\frac{f_y}{E_s}}\right) \qquad (5)$$

This is the non-dimensional depth of the neutral axis such that simultaneous crushing of concrete and yielding of tension steel takes place in the extreme fiber of the sections

When $\frac{x_u}{l_w}$ is less than 0.5, the term containing $\left(\frac{x_u}{l_w}\right)^2$ in equation (4) may be neglected for simplicity without introducing any significant error. Thus for $\frac{x_u}{l_w} < 0.5$, the flexural strength may be expressed as

$$\frac{M_{uw}}{f_{ck}\, t_w\, l_w^2} = \varphi\left(1+\frac{\lambda}{\varphi}\right)\left(\frac{1}{2} - 0.416\,\frac{x_u}{l_w}\right) \qquad (6)$$

(b) Flexural compression failure: For this condition, the strain in the tension steel at the extreme fiber will be less than the yield strain. Thus, tension steel does not yield. The contribution of force T_2 (*Fig* 8d) should not be considered in the analysis.

The position of the neutral axis is such that $\frac{x_u}{l_w} > \frac{x_u^*}{l_w}$. Assuming the neutral axis to lie within the section, that is, $\frac{x_u}{l_w} < 1.0$, the flexural strength is obtained as

$$\frac{M_{uw}}{f_{ck}\, t_w\, l_w^2} = \alpha_1\left(\frac{x_u}{l_w}\right) - \alpha_2\left(\frac{x_u}{l_w}\right)^2 - \alpha_3 - \frac{\lambda}{2} \qquad (7)$$

where,

$$\alpha_1 = \left[0.36 + \varphi\left(1 - \frac{\beta}{2} - \frac{1}{2\beta}\right)\right];$$

$$\alpha_2 = \left[0.15 + \frac{\varphi}{2}\left(1 - \beta - \frac{\beta^2}{2} - \frac{1}{3\beta}\right)\right]$$

$$\text{and} \quad \alpha_3 = \frac{\varphi}{6\beta}\left[\frac{1}{\left(\frac{x_u}{l_w}\right)} - 3\right] \qquad (8)$$

The value of $\frac{x_u}{l_w}$ to be used in equation (7) should be calculated from the quadratic equation

$$\alpha_1\left(\frac{x_u}{l_w}\right)^2 + \alpha_4\left(\frac{x_u}{l_w}\right) - \alpha_5 = 0 \qquad (9)$$

where,

$$\alpha_4 = \left(\frac{\varphi}{\beta} - \lambda\right); \text{ and } \alpha_5 = \left(\frac{\varphi}{2\beta}\right) \qquad (10)$$

The flexural strength of barbell and flanged shaped wall sections can also be obtained by adopting a similar procedure.

Axial load-moment interaction diagram

The equations derived in the previous section can be used to obtain the axial load-moment interaction diagram for the shear wall section under consideration. This may be done as follows:

(a) Tension region ($\frac{x_u}{l_w} < \frac{x_u^*}{l_w}$) : Assume a value of $\frac{x_u}{l_w}$ that is less than $\frac{x_u^*}{l_w}$. Equation (2) can be rearranged to give the axial load as

$$\lambda = (2\varphi + 0.36)\left(\frac{x_u}{l_w}\right) - \varphi \qquad (11)$$

Thus, λ can be calculated for a given value of $\frac{x_u}{l_w}$, f_y, f_{ck}, and vertical reinforcement ratio, ρ. The flexural strength can then be calculated from equation (4).

(b) Compression region ($\frac{x_u^*}{l_w} < \frac{x_u}{l_w} < 1.0$) : Assume a value of $\frac{x_u}{l_w}$ within the limits indicated above. Equation (9) can be rearranged to give the axial load as

$$\lambda = \alpha_1\left(\frac{x_u}{l_w}\right) + \frac{\varphi}{\beta}\left[1 - \frac{1}{2}\,\frac{1}{(x_u/l_w)}\right] \qquad (12)$$

Once λ is calculated, the flexural strength can be obtained from equation (7). Thus by varying $\frac{x_u}{l_w}$ within appropriate limits, the axial load and moment can be obtained. The axial load corresponding to uniform compression over the section is taken as

$$\frac{P_o}{f_{ck}\,t_w\,l_w} = (0.446 + \varphi) \qquad (13)$$

Fig 9 illustrates an interaction diagram for a rectangular shear wall section with uniformly distributed vertical reinforcement that was obtained by using the above mentioned procedure.

In order to verify the results obtained, an alternative numerical approach is used. The wall section is discretized into a number of elements. The strain in the extreme compression fiber of the section is taken as 0.0035, that is, crushing strain of concrete. Next, the position of the neutral axis is assumed. This defines the strain profile across the entire section. The strain at the mid-point of any particular element can be calculated. This strain is then used to obtain the stress in the concrete and steel within each element from the respective stress-strain curves. Thus, the force for each element can be calculated. The summation of such elemental forces gives the axial load on the section. The moment of resistance is obtained by summing up the moment of all elemental forces and the externally applied axial load about any convenient point. To generate the interaction curve, the position of the neutral axis is changed and the above mentioned procedure is repeated. One such curve is shown in *Fig* 9. Results by both methods agree well. The minor difference obtained is due to the fact that the former analysis assumes a bi-linear stress-strain curve for Fe 415 steel while the numerical approach uses the stress-strain curve as per IS:456-1978. No difference in result is observed in the case of Fe 250 steel.

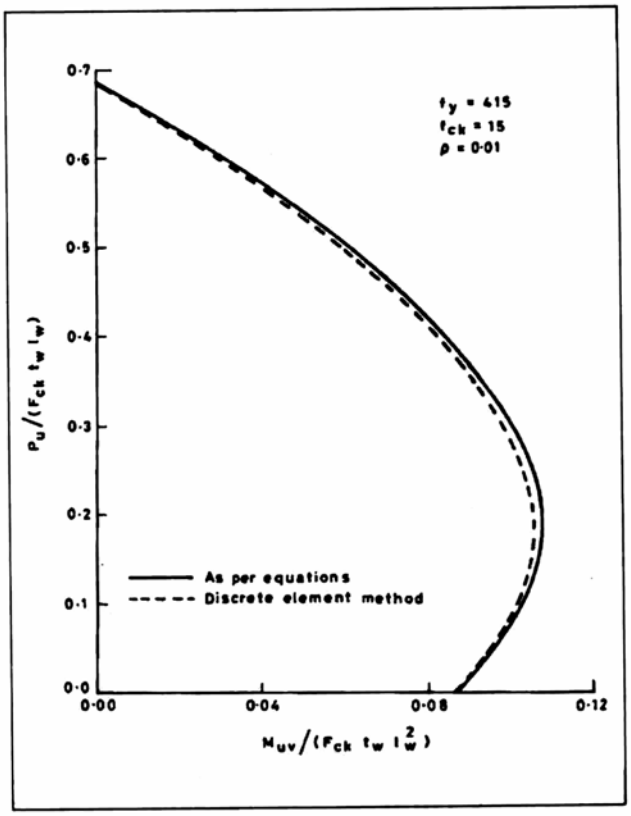

Fig 9 Axial load-moment interaction diagram

Summary and conclusions

The structural behaviour, seismic response, and modes of failure of slender, squat, and coupled shear walls are discussed. The effect of various parameter on the structural response of shear walls is also discussed. IS:456-1978 does not have provisions for estimating the flexural strength of shear wall sections; hence, algebraic expressions for the same are developed from basic principles. These expressions are applicable to slender rectangular shear wall sections having uniformly distributed vertical reinforcement. The axial load-moment interaction diagram is also generated by using these expressions. An alternative numerical approach is also outlined for the same. The results obtained by both methods compare very well. Both methods may be extended for the analysis of barbell and flanged wall sections.

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(To be continued)

References

1. FINTEL, M., 1973, *Ductile Shear Walls in Earthquake - Resistant Multistorey Buildings*, Engineering Bulletin, Portland Cement Association, USA.

2. ______*Indian Standard Code of Practice for Plain and Reinforced Concrete*, IS:456-1978, Bereau of Indian Standards, New Delhi.

3. ______*Buildings Code Requirements for Reinforced Concrete and Commentary*, ACI 318-89/318R-89, 1990, American Concrete Institute, USA.

4. ______*Uniform Building Code*, UBC-1988, International Conference of Building Officials, USA.

5. ______*Recommended Lateral Force Requirements and Commentary*, SEAOC, 1980, *Structural Engineers Association of California, USA.*

6. ______*Design of Concrete Structures for Buildings*, CAN3-A23.3-M84, 1984, *Canadian Standards Association, Canada.*

7. ______*Code of Practice for The Design of Concrete Structures and Commentary*, NZS-3101: Part-1 & 2, 1982, Standards Association of New Zealand, Wellington, New Zealand.

8. MEDHEKAR, M.S., and JAIN, S.K., 1993, Seismic behaviour design and detailing of RC shear walls, Part II: Design and Detailing, *The Indian Concrete Journal* (slated for publication in September 1993).

9. PAULAY, T., PRIESTLEY, M.J.N., and SYNGE, A.J., 1982, Ductility in earthquake resisting squat shearwalls, *Journal of the American Concrete Institute*, July/August 1982, pp.257-269.

10. PAULAY ,T., 1980. Earthquake-resisting shearwalls- New Zealand design trends, *Journal of the American Concrete Institute*, May/June, 1980, pp. 144-152.

11. OESTERLE, R.G., FIORATO, A.E., and CORLEY, W.G., 1981, *Reinforcement Details for Earthquake-Resistant Structural Walls*, PCA Research and Development Bulletin - RD073.01D, Portland Cement Association, USA.

12. MEDHEKAR, M.S., RASHAD, G., and JAIN, S.K., 1992, Shear reinforcement for aseismic design of flexural members, *The Indian Concrete Journal*, Vol. 66, pp. 319-324.

13. PARK, R., and PAULAY, T., 1975, *Reinforced Concrete Structures*, John Wiley & Sons, New York.

14. CARDENAS, A.E., and MAGURA, D.D., 1973, *Strength of High - Rise Shear Walls - Rectangular Cross Section*, Response of Multistorey Concrete Structures to Lateral Forces, SP-36, American Concrete Institute, USA, pp. 119-150.

15. ______ATC-11, *Seismic Resistance of Reinforced Concrete Shear Walls and Frame Joints: Implications of Recent Research for Design Engineers*, Applied Technology, Council, USA.

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