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Design Method of Reinforced Concrete Shear Wall Using EBCS

Dr. Suresh Borra[1], P.M.B.RajKiran Nanduri[2], Sk. Naga Raju[3]

[1](Associate Professor, Department of Civil Engineering, BahirdarUniversity, Ethiopia)
[2](Lecturer I in Structural Engineering, School of Building & Civil Engineering, Fiji National University, Fiji)
[3](Assistant Professor, Department of construction Management, AdigratUniversity, Ethiopia)

ABSTRACT :*Concrete shear walls or structural walls are often used in multistory buildings to resist lateral loads such as wind, seismic and blast loads. Such walls are used when the frame system alone is insufficient or uneconomical to withstand all the lateral loads or when partition walls can be made load bearing, replacing columns and beams. The analysis and design of buildings with shear walls became simple using commercially available computer programs based on the finite element method (FEM) and subsequent implementation of stress integration techniques to arrive at generalized forces (axial, shear, and moments). On the other hand, design engineers without such facilities or those with computer facilities lacking such features use simple method of analysis and design by taking the entire dimensions of the walls. This is done by considering the shear walls as wide columns of high moment of inertia and following the same procedure as for columns. The primary purpose of this paper is believed that structural engineers working in the analysis and design of high-rise buildings will be benefited from the design shear wall by using EBCS: 2-1995 and EBCS:8-1995codes and its results.*

KEYWORDS-*concrete shear wall, Ethiopian building code standard (EBCS), lateral loads, moment of inertia, stress integration techniques.*

I.   INTRODUCTION

Shear walls are deep relatively thin vertically reinforced concrete beams .They are commonly used in the structures to resist the effects of gravity loads and storey shears. Shear walls are vertical elements in the lateral force resisting system that transmit lateral forces from the diaphragm above to the diaphragm below or to the foundation . Shear walls may also bearing walls in the gravity load system or they may be components in dual system framed so as to resist only lateral loads.

Walls may be subjected to both vertical (gravity) and horizontal (Wind or Earthquake) forces. The horizontal forces are both in plane and out of plane. When considered under their in plane loads walls are called shear walls. When considered under their out of plane loads they are called normal walls. Walls will be designed to withstand all vertical loads and horizontal forces both parallel to and normal to the flat surface with due allowance for the effect of any eccentric loading or overturning forces generated. Any wall whether or not intended as part of the lateral force resisting system is subjected to lateral forces unless it is isolated on three sides(both ends and top) in which case it is classified as non structural. Any wall that is not isolated will participate in shear resistance to horizontal forces parallel to the wall. Since it tends to deform under stress when the surrounding framework deforms. The distribution of lateral loads on shear walls varies with their height. For example under lateral wind loading this distribution may vary from nearly uniform on a wall in a tall building to a single concreted force on a wall in a low building.

B.K. Thakkar [1] developed a methodology for generation of interaction diagrams for shear walls, thereby enabling a more accurate description of the capacity of the shear wall cross section under the effect of axial compression and lateral bending.ManojS. Medhekar and SudhirK.Jain [5] presented a worked example on shear wall design conformed to IS: 456-1978 and IS: 4326-1976. As IS: 456 and IS: 4326 do not give the specifications for the same,

A detailed commentary is included in their study regarding behavior and strength characteristics of R.C shear wall to explain the basis of theIS: 456-1978 and IS:4326-1976 specifications.M.Tomii and F.Esaki [6] presented a design method of reinforced concrete monolithic framed shear walls whose columns do not fail in shear by an earthquake and whose horizontal shear capacity is dominated by slip failure of their in filled panel wall.Under the large overturning effects caused by earthquake forces, edges of shear walls experience high compressive and tensile stresses. To ensure that shear walls behave in a ductile way concrete in the wall end regions must be reinforced in a special manner to sustain these load reversals without losing strength. End regions of a wall with increased confinement are called boundary elements. The special confining transverse reinforcement in boundary elements is similar to that provided in columns of R.C. frames. Sometimes the thickness of shear wall in these boundary elements is also increased. R.C walls with boundary elements have substantially higher bending strength and horizontal shear force carrying capacity and are therefore less susceptible to earthquake damage than walls without boundary elements.Special design checks are required to ensure that the net cross section area of wall at an opening is sufficient to carry the horizontal earthquake force. Door or window openings can be provided in shear walls but their size must be small to ensure least interruption to force flow through walls. Moreover openings should be symmetrically located. If opening are very small their effect on the overall state of stress in a shear wall is minor. Large openings have more pronounced effect and if large enough result in a system in which typical frame action predominates. Openings commonly occur in regularly spaced vertical rows throughout the height of the wall and the connection between the wall sections is provide by either connecting beams (or Spandrels) which form a part of the wall or floor slabs or a combination of both. If the openings are do not line up vertically and (Or horizontally) the complexity of the analysis is greatly increased. In most cases a rigorous analysis of a wall with openings is not required. Strut and tie procedure s that depict shear walls as consisting of compression struts and tension ties are useful tools for the evaluation of shear walls with openings.

If the design of the wall with openings the deformations must be visualized in order to establish some approximate method for analyzing the stress distribution to the wall. The major points that must be considered are the lengthening and shortening of the extreme sides (boundaries) due to deep beam action , the stress concentration at the corner junctions of the horizontal and vertical components between openings and the shear and diagonal tension in both the horizontal and vertical components.The ease of methods of analysis for walls with openings is greatly dependent on the relative rigidities of the piers and spandrels as well as general geometry of the building. Reinforced concrete and reinforced masonry shear walls are seldom simple walls. Whenever the walls has the doors, windows or other openings the wall must be considered as an assemblage of relatively flexible components (Column segments and wall piers) and relatively stiff elements (wall segments).A column segment is a vertical member whose height exceeds three times its thickness and whose width is less than two and half times its thickness. Its load is predominantly axial. Although it may contribute little to the lateral force resistance of the shear wall its rigidity must be considered. When a column is built integral with a wall the portion of the column that projects from the face of the wall is called pilaster. A wall pier is a segment of a wall whose horizontal length is between two and half to six times its thickness and whose clear height is at least two times its horizontal length. And wall segments are components that are longer than wall piers. They are the primary lateral load resisting components in the shear wall.There are many types of reinforced concrete shear walls which are used in buildings to carry forces. Shear walls are oblong in cross section that is one dimension of the cross section is much longer than the other while rectangular cross section is common; L and U shaped sections are also used. Thin walled hollow R.C shafts around the elevator core of the buildings also acts as shear walls and should be taken advantage to resist earthquake forces. Shear walls ranges from simple rectangular types and the flanged walls(called the bar bell type walls with boundary elements) These are formed by columns and wall in between, coupled shear walls , rigid frame shear walls, framed walls with in-filled frames, column supported shear walls and core type shear walls. And based on behavior shear walls also be classified as Shear –Shear walls usually low rise shear walls, Ordinary –moment shear walls usually high rise shear walls to resist high winds and cyclones and ductile moment shear walls meant for seismic regions which have good energy dissipation characteristics under reversed cyclic loads.

Horizontal forces at any floor or roof level are generally transferred to the ground (foundation) by using the strength and rigidity of shear walls and partitions. A shear wall may be considered analogous to a cantilever plate girder standing on end in a vertical plane where the wall performs the function of a plate girder web the pilasters or floor diaphragms function as web stiffeners and the integral reinforcement of the vertical boundaries functions as flanges.

Axial , flexural and shear forces must be considered in the design of shear walls .The tensile forces on shear wall elements resulting from the combination of seismic uplift forces and seismic overturning moments

must be resisted by anchorage into the foundation medium unless the uplift can be counteracted by gravity loads mobilized from neighboring elements.In foundation effects the rotation at the foundation can greatly influence the overall rigidity of shear wall because of rigidity nature of the shear wall itself .However the rotational influence on relative rigidities of walls for purposes of horizontal force distribution may not be as significant. Considering the complexities of soil behavior a quantitative evaluation of the foundation rotation is generally not practical but a qualitative evaluation will be provided.In framework effects the relative rigidity of concrete or unit masonry walls with nominal openings is usually much greater than that of the building framework. Therefore the walls tend to resist essentially all or a major part of the lateral force. For a building with rigid diaphragms there is a torsional moment and a rigidity analysis is required. It is necessary to make a logical and consistent distribution of story shears to each wall. An exact determination of wall rigidities is very difficult but is not necessary because only relative rigidities are needed. Approximate methods in which the deflections of portions of walls are combined usually are adequate.

Wall deflections: The rigidity of a wall is usually defined as the force required causing a unit deflection. Rigidity is expressed in Kips per inch. The deflection of a concrete shear wall is the sum of the shear (Fig. 1) and flexural deflections (Fig. 2). In case of solid wall with no openings the computations of deflection are quite simple. However when the shear has openings as for doors and windows the computations for deflection (Fig. 3) and rigidity are much more complex. An exact analysis considering angular rotation of elements, rib shortening etc is very time consuming .For this reason several short cut approximate methods have been developed. These do not always give constant or satisfactory results. A conservative approach and judgment must be used.
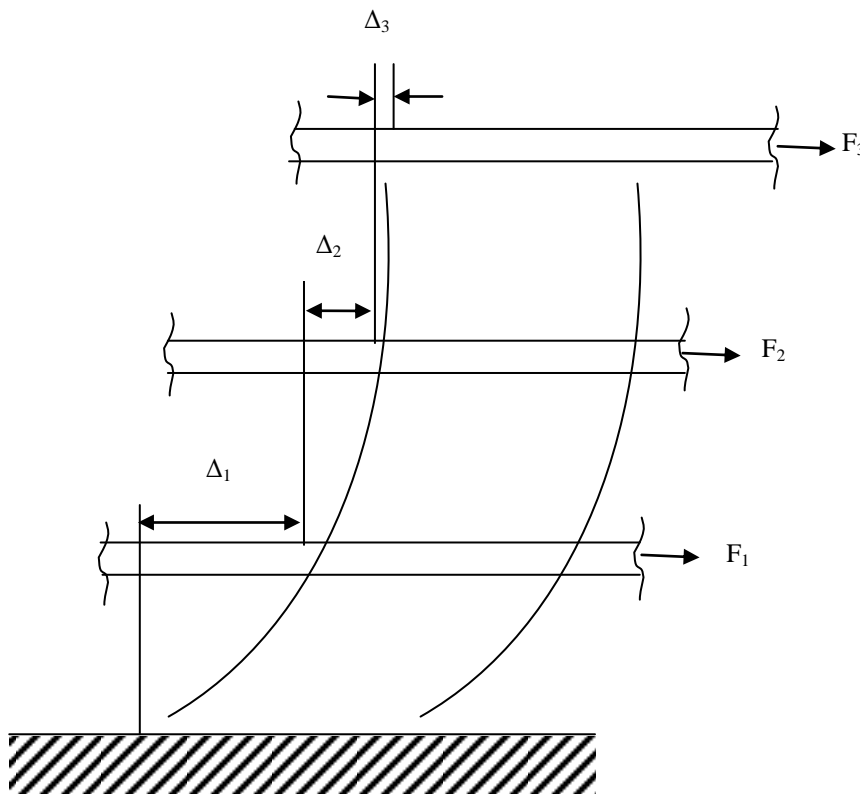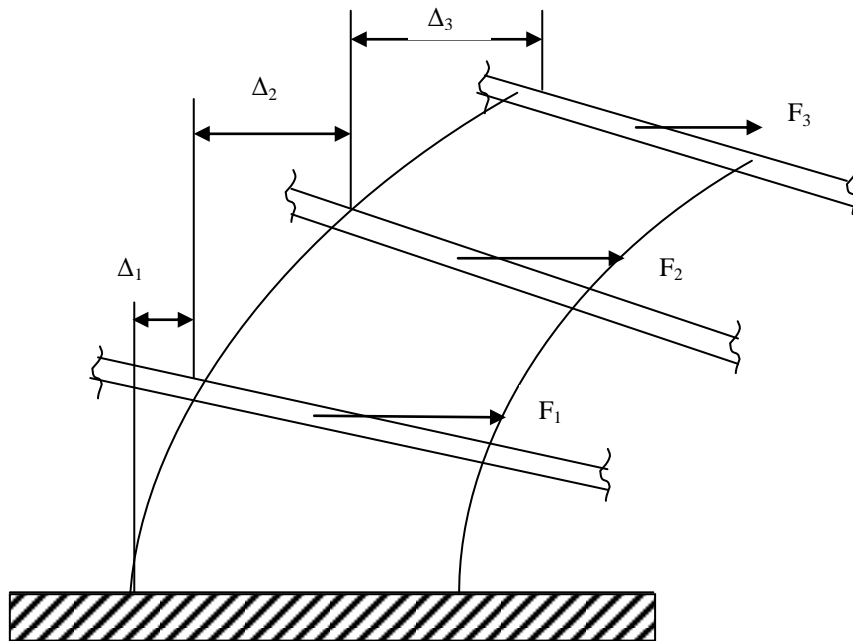

Fig. 1 Shear Deformation

Fig. 2 Flexural Deformation

Diagonal tension cracks in spandrels may occur at the top or at other elevations depending on vertical shear distribution.

Diagonal tension cracks in piers usually most severe at bottom where horizontal shear is largest.
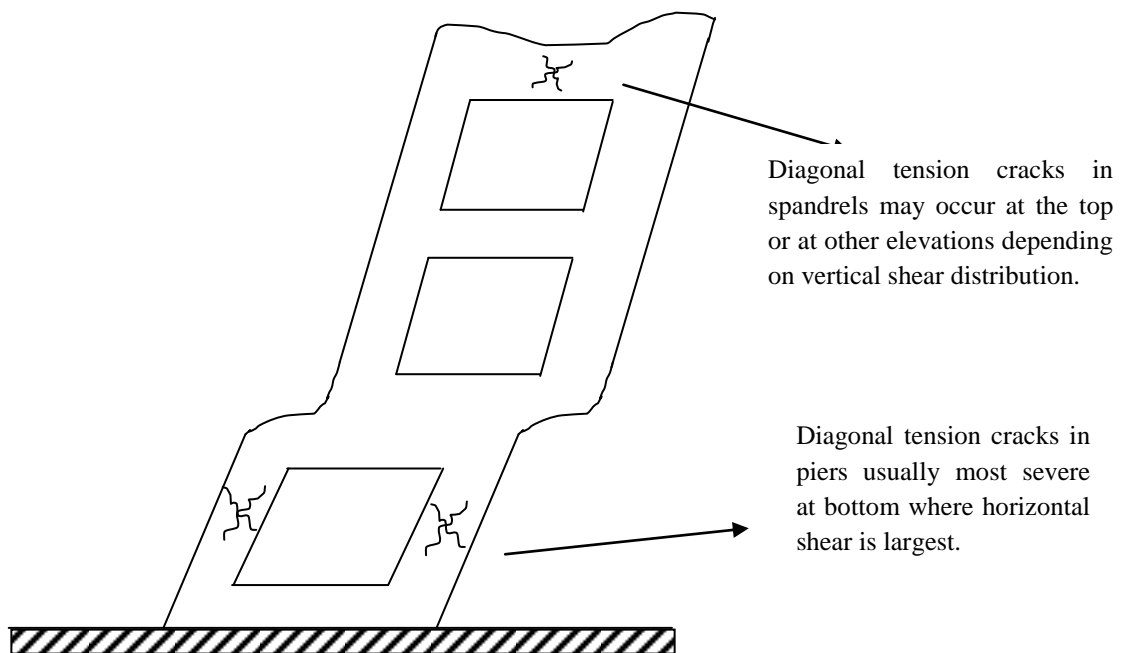
Fig. 3 Deformation shear wall with opening

II.   METHODS OF ANALYSIS

Approximate methods for analyzing walls with openings are generally acceptable. For the extreme cases the procedure is straight forward. For other cases a variety of assumptions may be used to determine the most critical loads on various elements thus resulting the conservative design. When however the reinforce requirements or the resulting stresses of this approach appear excessively large the strut and tie procedure or a more rigorous analysis may be justified.

Resistance of shear walls to in-plane lateral loading may be determined according to one of the methods presented below. Each method is briefly summarized and the assumptions involved in formulation of

the methods are presented. The appropriate method should be determined by the building designer or wall designer in accordance with the provisions. Analysis methods are presented with sufficient detail to allow the user to implement each method without consulting other sources. The first three methods such as Perforated Shear Wall Method, Segmented Shear Wall Method, Ni-Karacabeyli's Method   need input of unit shear resistance values. Alternatively, the shear wall resistance can be estimated using analytical methods such as Shear through Panel Rotation, Alternate Rational Analyses.   The term shear wall rather misleading as such walls behave more like flexural members. However the most important property of shear walls for seismic design as different design for wind is that it should have good ductility under reversible and repeated overloads. In planning shear walls we should try to reduce bending tensile stresses due to lateral loads as much as possible by loading them with as much gravity force as it can safely take .They should also be laid symmetrically to avoid tensional stresses. Determination of the forces in these walls is part of structural analysis. Therefore here the concern if design .In the design general dimensions, vertical m, horizontal reinforcements, shear strength requirements, adequacy of boundary elements, flexural strength, required development anchorage, openings in shear walls, discontinuity in shear walls and shear strength of construction joints are dealt.

III.   DESIGN PROBLEM

A shear wall of length 4 m and thickness 300 mm is considered and subjected to the following forces with the grade of concrete f_{cd}=25 N/mm^2 and strength of steel f_{yd}=415 N/mm^2

Table 1: Forces considered for design of shear wall

Loading	Axial Load (KN)	Moment (KNm)	Shear force (KN)
DL+LL	2500	1000	80
EQ	400	6000	900

Calculation of material strength

Height of the wall = 4m
Using EBCS:2-1995 code provision
Design strength of materials (Steel and Concrete):

Concrete:

$$\gamma_c = 1.5 \text{ for class} - 1 \text{ workmanship EBCS} - 2 \text{ Table } 3.1$$

$$f_{cd} = \frac{0.85 f_{ck}}{\gamma_s} = \frac{0.85(25 mpa)}{1.5} = 14.17 \text{ mpa}$$

$$f = 0.21 \frac{f_{ck}^{2/3}}{\gamma_c} = 0.21 \frac{(25)^{2/3}}{1.5} = 1.197 mpa$$

E_{cm}=29Gpa

Steel:

γ_s= 1.15for class-1 workmanship

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{415 mpa}{1.15} = 360.87 n/mm^2$$

Design Load calculations: The given load combinations are used for the design according to EBCS:2-1995:

Combination 1= 1.3DL+1.6LL
Combination 2= 0.75 Combination 1+EQX
Combination 3= 0.75 Combination 1-EQX
Combination 4= 0.75 Combination 1+EQY
Combination 5= 0.75 Combination 1-EQY

The calculated axial load, moment and shear force to be considered for the design by using the above combinations are shown in the following table:

<div align="center">**Table 2: Calculation of Axial load, Moment and shear force using load combinations**</div>

Load combination	Axial Load (KN)	Moment (KNm)	Shear force (KN)
Combination 1	2500	1000	80
Combination2	2275	6750	960
Combination3	1475	-5250	-840
Combination4	2275	6750	960
Combination5	1475	-5250	-840

From the five load combinations the maximum values obtained are:

$$N_{sd}=2500 \text{ kN}$$
$$M_d=6750 \text{ kN}$$
$$V_d=960 \text{ kN}$$

Determination of design eccentricity in both directions:

$$e_{tot}=e_a+e_o+e_2$$

Accidental (additional) eccentricity due to various imperfections:

- $e_a=L_e/300 \geq$ **20mm**, EBCS: 2-1995 section 4.4.3
  Where: Le=is the effective buckling length of the wall.
  Assuming the top end of the shear wall to be simply supported.

- $L_e=0.7L$

     Where L =Length of wall height.
$L_e=0.7(4) =2.8m.$

Therefore, $e_a= (2800/300)=9.33 \text{ mm}<20mm$

Considered $e_a=20mm$

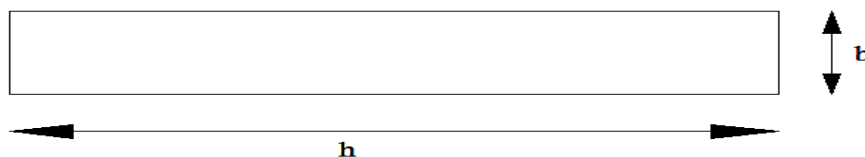**Determination of design eccentricity in H-direction:**

<div align="center">**Fig 4 : Cross section of the wall**</div>

First order eccentricity:

$$e_o=\frac{M_d}{N_{sd}} = \frac{6750}{2500} = 2.7m = 2700mm$$

Second order eccentricity:

- *Checking the slenderness of the wall:*A wall may be considered as short walls when the ratio of its effective height to its thickness does not exceed 7. Otherwise it shall be considered slender.

$$\frac{H_e}{t} \leq 7, \text{Short wall}$$

$$\frac{H_e}{t} >, Slend \text{ } or \text{ } wall$$

Where He =Effective height of the wall.

- *Effective height of the wall:*

The effective height of reinforced concrete walls in non-sway mode is given by:

$He= \beta H$

Where: H= is the Story height of the wall.

β=is the coefficient which is determined from the following equation.

$$\beta = \begin{cases} 1.00 \text{ for walls with two edges restrained} \\ \frac{1}{1+\frac{L}{(3b)^2}} \geq 0.3 \text{ , for walls with three edges restrained.} \\ \text{For walls with four edges restrained} \begin{cases} = \frac{1}{(2\frac{L}{b})}, L > b \\ \frac{1}{1+\frac{L}{(3b)^2}}, L \leq b \end{cases} \end{cases}$$

Where: b is the width of the wall measured center-to-center of the bracing walls, or width measured from the center of a bracing wall to the free edge.

In our case the wall is bar belled type i.e. the wall is restrained in with two edges with walls on each edge.

$\beta = 1$

$He= \beta H = 1(4) = 4m.$

$\frac{H_e}{t} = \frac{4000}{300} = 13.33$

> 7, so the wall is slender and we should account for second order eccentricity

$e_2 = 0.4t(\frac{L_e}{10t})^2$

Where t=thickness of the wall in our case t=300mm

$e_2 = 0.4(300)(\frac{2800}{10(300)})^2 = 104.53mm$

$e_{tot} = e_a + e_o + e_2$

$e_{tot} = 20mm + 2700mm + 104.53mm = 2824.53mm$

Relative eccentricity:

The relative eccentricity, for a given direction, is the ratio of the total eccentricity, allowing for initial eccentricity and second-order effects in that direction, to the wall width in the same direction.

$e_{rel} = \frac{e_{tot}}{h} = \frac{2824.53}{5000} = 0.56$

Determination of design eccentricity in B-direction:

- First order eccentricity: no moment is carried in this direction as it is carried by the perpendicular walls, $M_d=0$

$e_o = \frac{M_d}{N_{sd}} = 0$

- Second order eccentricity: $e_2 = 0.4t(\frac{L_e}{10t})^2$

   Where t=thickness of the wall in our case t=b

   e_2=104.53mm

$e_{tot} = e_a + e_o + e_2$

$e_{tot} = 20mm + 0 + 104.53mm = 124.86mm$

- Relative eccentricity:

The relative eccentricity, for a given direction, is the ratio of the total eccentricity, allowing for initial eccentricity and second-order effects in that direction, to the column width in the same direction.

$$e_{rel} = \frac{e_{tot}}{b} = \frac{124.86mm}{300mm} = 0.415$$

- Equivalent eccentricity: e_{eq}

$$e_{eq} = e_{tot}(1 + k \propto)$$

Where:
e_{eq}= total eccentricity in the direction of the larger relative eccentricity
k=relative eccentricity ratio.
α=is obtained from table 4.1 of EBCS: 2-/1995(which is given below) as a function of the relative normal Force,

$$v = \frac{N_{sd}}{f_{cd} \times A_c}$$

Table 3: Function of the Relative normal force

v	0	0.2	0.4	0.6	0.8	≥ 1.0
∝	0.6	0.8	0.9	0.7	0.6	0.5

- Relative eccentricity ratio: The eccentricity ratio, for a given direction, is the ratio of the total eccentricity, allowing for initial eccentricity and second-order effects in that direction, to the column width in the same direction.

Relative eccentricity ratio, k

$$K = \frac{\text{small relative eccentricty}}{\text{larger relative eccentricity}}$$

$$K = \frac{0.415}{0.565} = 0.734$$

- Relative normal force, $v = \frac{2500KN}{(14.17 \times 10)^2 KN/m^2 (0.3 \times 0.5)} = 0.1176 \geq 1$

Therefore $v = 0.1176$
$$e_{eq} = e_{tot}(1 + k \propto);$$
$$e_{tot} = \text{total eccentricity in the direction of the larger relative eccntricity}; e_{tot} = 2824.53mm$$

Therefore $e_{eq} = 2824.53(1 + (0.734)(0.7176)) = 4312.26mm$
Design moment calculation:

Design moment, $M_{sd} = e_{eq} \times N_{sd}$

$M_{sd} = (4.31226m)(2500 kN) = 10780.65$ kN-m

Design of vertical reinforcement:
Area of reinforcement: $A_s = \frac{\omega f_{cd} A_c}{f_{yd}}$

ω is the reinforcement ratio from chart using v and μ values. $\mu = \dfrac{M_{sd}}{f_{cd}A_c h}$

Where h =depth of the cross section.

$$\mu = \frac{10780.65 Kn_m}{\left(14.17 \times 10^3 \frac{Kn}{m^2}\right)(0.3m)(5m)(5m)} = 0.1014$$

Using, $v = 1.176$

$$\mu = 0.1014$$

Take $\dfrac{d'}{h} = 0.05$

From uni-axial chart number-6 the value of ω lies under the curve with $\omega = 0$, so provide minimum reinforcement.

Check for the limiting values:

For vertical reinforcement the code provides the following limiting values, as discussed in the previous section.

$$A_{min} = 0.004 \ A_c = 0.004(300mm)(5000mm) = 6000mm^2$$

$$A_{max} = 0.04 A_c = 0.04(300mm)(5000mm) = 60,000mm^2$$

$A_s = A_{min} = 6000mm^2$

Using $\emptyset 14$:

$a_s = \dfrac{\pi d^2}{4} = \dfrac{\pi(14)^2}{4} = 153.93mm^2$
$s = \dfrac{h a_s}{A_s}$

Using two layers of bars.

$A_s = \dfrac{A_{s,total}}{2} = \dfrac{6000}{2} = 3000mm^2$

$S = \dfrac{(5000mm)(153.93mm)}{3000 \ mm^2} = 256.56 \ mm$

$S_{max} \leq \begin{cases} 2t \\ 300mm \end{cases} \ t = b = 300mm$

$S_{max} = 300mm$
Therefore provide $\emptyset 14$ at $\dfrac{c}{c} 250mm$.

Design of shear reinforcement:

- Check for the diagonal compression failure of concrete
Section resistance $Vrd = 0.25 \ f_{cd} b_w d \geq v_d$
$$Vrd = 0.25(14.17 \ mpa)(300mm)(4750mm) = 5048.06 \ K_n > V_d \ O.K$$
Note here that if V_{rd} less than V_d(i.e, Vrd<Vd) the possible measures are:

Increase the grade of concrete:

Increase the cross-section (as it is barbell type shear wall i.e. fixed in the two edges it means increasing the thickness of the wall).

- Check the section capacity, V_c

For members subjected to significant axial compression: EBCS-2/1995 Section 4.5.3.2

$$V_c = 0.25\, f_{ctd}\, k_1 k_2 b_w d + V_{cn}$$

$$V_{cn} = 0.10\, \frac{b_w d}{A_c}\, N_{sd}$$

$$f_{ctd} = 1.197\ \text{Mpa}$$

$$k_1 = 1.6 - d \geq 1, d\ \text{in m}$$

$$d = h - d'$$

$$\text{Take } \frac{d'}{h} = 0.05$$

$$d' = 0.05h = 0.05 \times 5000 = 250\text{mm}$$

$$d = h - d' = 5000\text{mm} - 250\text{mm} = 4750\text{mm}$$

$$k_1 = 1.6 - 4.570 \geq 1$$

So take $k_1 = 1$

$$k_2 = 1 + 50\rho \leq 2.0$$

$$\rho = \frac{A_s}{b_w d} = \frac{6000\ \text{mm}^2}{(300\text{mm})(4750\text{mm})} = 0.0042$$

$$k_2 = 1 + 50(0.0042) = 1.21 \leq 2.0.\ \text{O.K}$$

$$V_c = 0.25(1.197\ \text{Mpa})(1)(1.21)(300\text{mm})(4750\text{mm}) * 10^{-3}$$
$$+0.10\,\frac{(300\text{mm})(4750\text{mm})}{(300\text{mm})(5000\text{mm})} * 2500\ \text{kN} = 753.42\ \text{kN}$$

$$V_c = 753.42\text{kN} < V_d = 960\ \text{kN}$$

- So, design for shear reinforcement
  EBCS provision for area of shear reinforcement (horizontal reinforcement), According to section 6.2.1.2 of EBCS-2

- Design for horizontal shear forces in the plane of the wall shall be in accordance with provisions for beams give on EBCS 2/1995 section 4.5.3

- Sections located closer to the base than a distance $\frac{b}{2}$ or, $\frac{L}{2}$ whichever is less, be designed for the shear at $\frac{b}{2}$. Or $\frac{L}{2}$

- The area of horizontal reinforcement shall not be less than one-half of the vertical reinforcement

- Spacing of shear reinforcement:

$$S = \frac{A_v d f_{yd}}{V_s},\quad V_s = V_d - V_c,\ \text{EBCS-2/1995 Section 4.5.4(4)}$$

$$S_{max} \leq \left\{ \begin{array}{l} 2t \\ 300mm \end{array} \right., t = b$$

The horizontal reinforcement shall enclose the vertical reinforcement. The horizontal bars shall enclose and be tied to the vertical bars so as to form rigid mat.

V_d=960 kN

(b/2)=150 mm< (L/2)

Position of V_c=515.98 kN

There fore $\dfrac{515.98 \text{ kN}}{x} = \dfrac{960 \text{ kN}}{4000mm}$, using the similarity of triangles, x = 2149.92mm
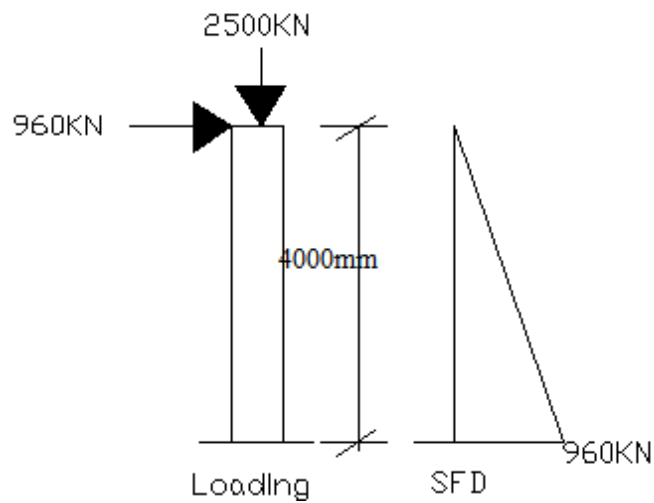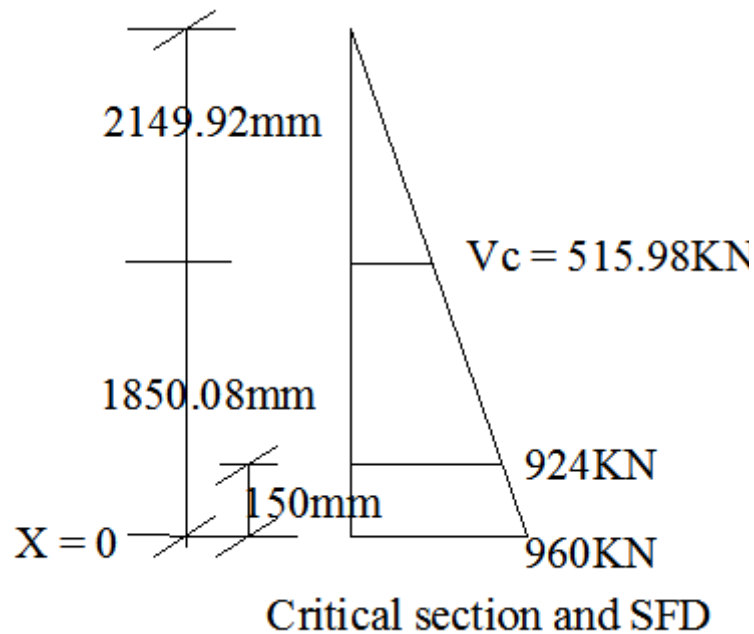

Fig 5: SFD of the Shear wall

Fig 6: Critical Section and SFD of the Shear Wall

At the critical point (@ 150 mm from the base)

$$\frac{V_d}{(4000mm - 150mm)} = \frac{960\ KN}{4000mm}\ \text{using the similarity of triangewles.}$$

Therefore V_d=924kN.
Remember here that the critical section for column is at d distance from the face of the column.For region between x=0 and x=150mm, design for shear at x=150 mm, V_d=924 kN.

Using Ø8

$$a_s = \frac{\pi d^2}{4} = \frac{\pi (8)^2}{4} = 50.26\ mm^2$$

$$Smax = \begin{cases} 2t \\ 300\ mm \end{cases} t = b = 300\ mm$$

Smax=300 mm

$$s = \frac{A_v d f_{yd}}{V_s}\ EBCS - 21995, section\ 4.5.4(4)$$

$$V_s = V_d + Vc = 924\ kN - 753.42\ kN = 170.58\ kN$$

$$S = \frac{(50.26\ mm\ )(4750\ mm)\left(360.87\frac{n}{mm^2}\right)}{170.58 \times 10^3\ N} = 505.06\ mm > s_{max} = 300\ mm, o.k$$

so provide Ø8 mm at 300 $mm\frac{c}{c}$

For region between x=150mm to x=(4000mm – 2149.92 mm)= 1850. 08 mm

V_d= 924 KN

Using Ø8mm, $\therefore a_s = 50.26\ mm^2$

$$Smax = \begin{cases} 2t \\ 300\ mm \end{cases} t = b = 300mm$$

Smax =300 mm

$$S = \frac{A_v d f_{yd}}{V_s} EBCS - 2,1995, section4.5.4(4)$$

Vs=Vd+Vc=924 KN-753.42 KN=170.58 KN

$$S = \frac{(50.26\ mm^2)(4750\ mm)(360.87\frac{N}{mm^2})}{170.58 \times 10^3\ N} = 505.06\ mm > S_{max} = 300\ mm$$

So provide Ø8 mm at 300 $mm\frac{c}{c}$

For region between x=2149.92mm to x= 40`00mm, provide minimum shear reinforcement as Vd≤ V_c

$$Smax = \begin{cases} 2t \\ 300 \text{ mm} \end{cases} t = b = 300 \text{mm}$$

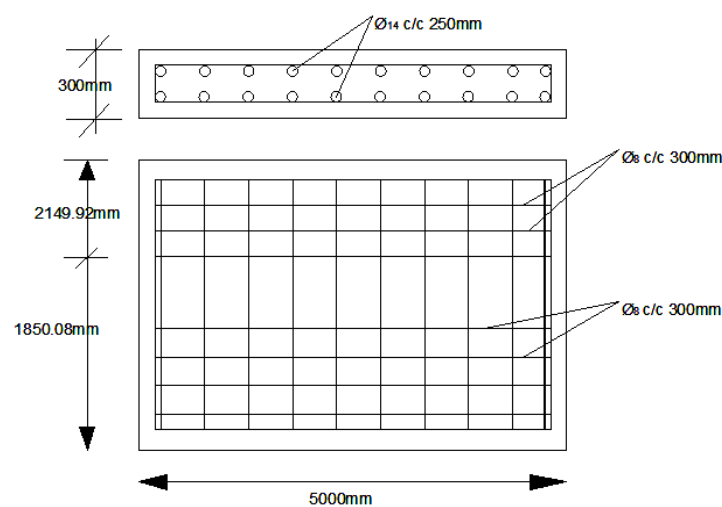Smax = 300 mm.

Reinforcement detailing:

Fig 7: Reinforcement detailing of the reinforced concrete shear wall

IV.   CONCLUSION

Even in today's high-tech computer-oriented world with all its sophisticated design capability, there still is a need to undertake approximate analysis of structures. First, it provides a basis for selecting preliminary member sizes because the design of a structure, no matter how simple or complex begins with a tentative selection of members. With the preliminary sizes, an analysis is made to determine if design criteria are met. If not, an analysis of the modified structure is made to improve its agreement with the requirements, and the process is continued until a design is obtained within the limits of acceptability. Starting the process with the best possible selection of member's results in a rapid convergence of the iterative process to the desired solution. It is almost mandatory for the structural engineer to compare several designs before choosing the one most likely to be the best from the points of view of structural economy and how well it minimizes the premium required by the mechanical, electrical, and curtain wall systems. Preliminary designs are therefore very useful in weeding out the weak solutions.

REFERENCES

[1].   B.K. Thakkar, Analysis of shear walls under compression and bending, *Current trends in technology and science (ISSN: 2279-0535)*, Vol.1, Issue 2, September 2012, *pp*.100-103.
[2].   Bungale S. Taranath, *Structural Analysis and Design of Tall Buildings* (McGraw-Hill book company)
[3].   Ethiopian Building Code of Standard, Part 2-1995, *Structural use of concrete*, Ministry of works & urban development, Addis Ababa, Ethiopia.
[4].   Ethiopian Building Code of Standard, Part 8-1995, *Structural use of concrete*, Ministry of works & urban development, Addis Ababa, Ethiopia.
[5].   Manoj S. Medhekar and SudhirK.Jain, Seismic behavior, design and detailing of RC shear walls, Part II: Design and detailing, *The Indian Concrete Journal*, Vol.67, Issue 8, September 1993, *pp*.451-457.
[6].   M.Tomii and F.Esaki, Design method of R.C framed shear walls to sustain vertical loads after shear failure , *World conference on earthquake engineering*, San Francisco, California, Article 8, Vol.5, 1984, pp.581-588.