Addis Ababa University

School of Graduate Studies

Department of Civil Engineering

Evaluation of Analysis and Design Results of Reinforced Concrete Walls Carried out using ETABS

Submitted as a partial fulfillment of the requirements of Master of Science in Civil Engineering (Structures Major)

Wondimu Kassa

March 2008

Acknowledgment

I would like to give my heart felt thanks to my thesis supervisor, Dr.-Ing. Girma Zerayohannes, for his continuous support in completing this thesis work. My deepest thanks also extend to my parents and friends for their help and encouragement.

List of Tables

Content

Abstract

The use of commercial application programs is well established in the local structural design offices working in Ethiopia. Among the most widely used programs are SAP2000 and ETABS by CSI, California, Berkeley.

User confidence in respect of analysis results is reasonably high for building structures or bridges whose lateral force resisting systems comprise 3-D rigid frames. For buildings with wall or dual wall-frame systems, however, the comparison is not so straight forward, and has not yet been systematically investigated and reported.

More recently some users while accepting the drawbacks in respect of the design of columns tend to accept the design results of walls as valid. Clearly, response of structural walls to axial load and biaxial bending is much more involved than their column counterpart.

In an effort to show the proximity and/or divergence of the results of ETABS with other soft wares and analysis methods the following processes has been performed.

1. Lateral load distribution results of ETABS were compared with those of the approximate elastic analysis procedure for a system of walls.
2. Column reinforcement results of ETABS were compared with those from interaction charts of EBCS 2 Part2.
3. Reinforcement of the walls from ETABS was checked by software specially developed for checking capacities of structural walls of any shape under normal load and biaxial bending.

From the lateral load distribution comparison result, it was observed that there were differences between the outputs of ETABS and the approximate elastic analysis. The percentage difference varies from one wall arrangement type to another and wall section to wall section. In case of the design results, column reinforcement areas obtained from ETABS were larger than those from interaction charts of EBCS 2 Part 2. For the simple rectangular structural walls considered, the reinforcement from ETABS was found to be insufficient to resist the design actions on it. The same thing was observed on the L-shaped walls. Where as for the C-shaped walls designed by ETABS the capacity to demand ratio were nearly 1.

Based on the comparison results of the design of a limited number of columns and walls, it may be concluded that the design of walls and columns performed by ETABS may lie on the safe side or the unsafe side.

1.1. Introduction

The use of commercial application programs is well established in the local structural design offices working in Ethiopia. Among the most widely used programs are SAP2000 and ETABS by CSI, California, Berkeley.

User confidence in respect of analysis results is reasonably high for building structures or bridges whose lateral force resisting systems comprise 3-D rigid frames, because of many frame analysis tests which have been analyzed using different methods ranging from analysis by hand (Kani iteration) to small in-house developed programs based on the direct stiffness method. This was specially the case for planar frame analysis. Confidence on solutions of 3-D lateral force resisting frames has developed as a result of comparison of analysis results using equivalent 2-D models. For buildings with wall or dual wall-frame systems, however, the comparison is not so straight forward, and has not yet been systematically investigated and reported.

Moreover these programs provide the users with the possibility to design individual members of their structures based on the Euro-Code whose result may also be taken as approximately applicable according to the Ethiopian code, as the two codes are similar in many respects.

Again, while the design results for beams using ETABS are good enough approximations, the design results for columns under biaxial bending are different from the Ethiopian Code results. Thus the user uses the design results of flexural members and designs his columns using the charts of the local code.

More recently some users while accepting the drawbacks in respect of the design of columns tend to accept the design results of walls as valid. Clearly, response of structural walls to axial load and biaxial bending is much more involved than their column counterpart. Therefore, such blind acceptance of the results may not be warranted in the face of failure of the Softwares to deliver satisfactory results for much simpler column sections with simple reinforcement arrangement.

To this end the user of this structural analysis and design software packages must explicitly understand the assumptions of the programs and must independently verify the results.

1.2. Scope and Objectives of the Thesis

Though there are a few more aspects of analysis and design of structural walls which need to be assessed, this thesis aims at evaluating design results of isolated reinforced concrete structural walls and columns and comparison of the lateral load distribution results for a system of walls carried out by ETABS and hand calculation. Design verification is done using software specially developed for checking capacities of structural walls of any shape under normal load and biaxial bending. The lateral load distribution result is verified using the approximate elastic method of analysis. The whole process is done in two phases.

1. Analyzing system of walls for a specified lateral load and comparing the results with solutions of the approximate elastic analysis.
2. After analyzing and designing typical cantilever columns and walls of different cross sections using ETABS, the section capacities of the columns are compared with the columns designed using the interaction charts of EBCS-2 Part 2. Design results of the walls is compared with the results of shear wall design program in reference 2 (R2):

Finally, it comments on reliability of using ETABS for design of reinforced concrete structural walls in our country.

The purpose of this study is therefore:

1. To compare the lateral load distribution results of ETABS for a system of walls with results of the approximate elastic analysis.
2. To check wall and column design results of the ETABS.

2. Literature Review

2.1. Introduction

Shear wall is a structural element used to resist lateral/horizontal/shear forces parallel to the plane of the wall by:

1. Cantilever action for slender walls where the bending deformation is dominant
2. Truss action for squat/short walls where the shear deformation is dominant

Shear walls function by working as a large vertical cantilever which has the ability to resist large seismic forces. They can be very efficient in resisting horizontal loads and generally provide strength much more economically than a frame structure. The reason for this extra strength is because they can be designed to have some ductility.

Over the past few years shear walls have become the primary design feature for tall buildings and an important one in smaller ones. They act as very deep beams which carry loads in shear in addition to bending and so do not suffer from the same deflections as a basic design without shear walls (8).

2.2. Structural Wall System

2.2.1. Arrangement of Structural Walls

Structural walls in buildings can have different geometric configuration, orientation, and location within the plane of the building. The positions of the structural walls within a building are usually dictated by functional requirements. These may or may not suit structural planning; the purpose of a building and the consequent allocation of floor space may dictate arrangements of walls that can often be readily utilized for lateral force resistance. Building sites, architectural interests, or clients' desires may lead, on the other hand, to positions of walls that are undesirable from a structural point of view. In this context it should be appreciated that while it is relatively easy to accommodate any kind of wall arrangement to resist wind forces, it is much more difficult to ensure satisfactory overall building response to large earthquakes when wall locations deviate considerably from those dictated by seismic considerations. The difference in concern arises from the fact that in the case of wind, a fully elastic response is expected, while during large earthquake demands, inelastic deformations will arise (3).

Generally, in choosing suitable location for lateral–force-resisting structural walls, three structural aspects should be considered.

1. For the best torsion resistance, as many of the walls as possible should be located at the periphery of the building. Such an example is shown in Fig. 2.1(b). The walls on each side may be individual cantilevers or they may be coupled to each other.

2. The more gravity load can be routed to the foundations via a structural wall, the less will be the demand for flexural reinforcement in that wall and the more readily can foundations be provided to absorb the overturning moments generated in that wall(3).

3. In multistory buildings situated in high-seismic-risk areas, a concentration of the total lateral force resistance in only one or two structural walls is likely to introduce very large forces to the foundation structure, so that special enlarged foundations may be required.

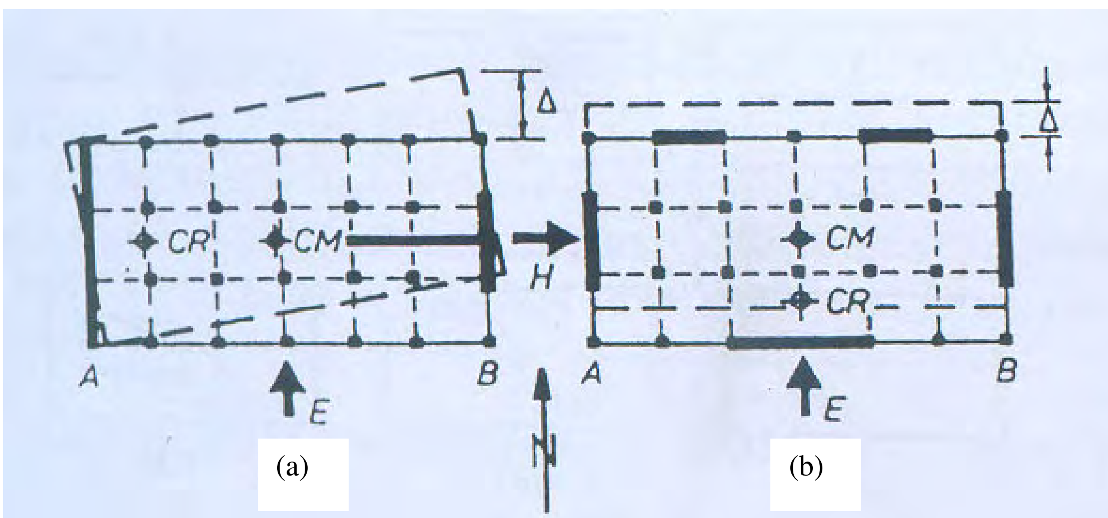


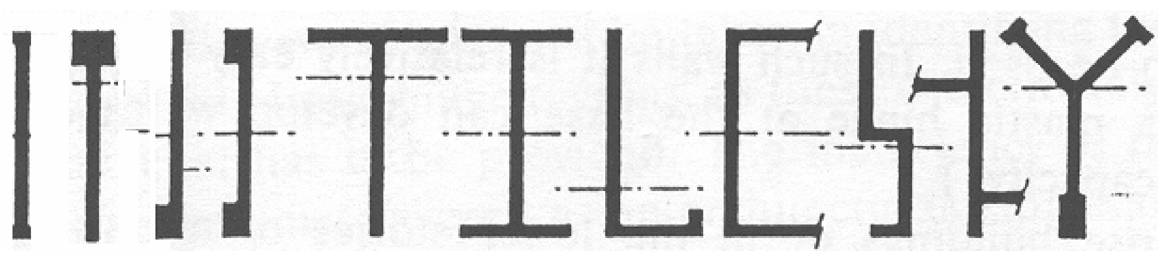
Fig 2.1 Torsional stability of inelastic wall systems

2.2.2. Sectional Shapes of Walls

Individual structural walls of a group may have different sections. Some typical shapes are shown in fig. 2.2. The thickness of such walls is often determined by code requirements to ensure workability of wet concrete or to satisfy fire ratings. When earthquake forces are significant, shear strength and stability requirements may necessitate an increase in thickness.

Boundary elements, such as shown in fig. 2.2 (b) to (d), are often present to allow effective anchorage of transverse beams. Even without beams, they are often provided to accommodate the principal flexural reinforcement, to provide stability against lateral buckling of a thin–walled section and, if necessary, to enable more effective confinement of the compressed concrete in potential plastic hinges.

Walls meeting each other at right angles will give rise to flanged sections. Such walls are normally required to resist earthquake forces in both principal directions of the building. They often possess great potential strength.



(a) (b) (c) (d) (e) (f) (g) (h) (i) (j) (k)

Fig 2.2 Common sections of structural walls

2.2.3. Variations in Elevation

In medium-sized buildings, particularly apartment blocks, the cross section of a wall, such as shown in Fig. 2.2, will not change with height. This will be the case of simple prismatic walls. The strength demand due to lateral forces reduces in upper stories of tall buildings, however. Hence wall sizes, particularly wall thickness, may then be correspondingly reduced.

More often than not, walls will have openings either in the web or the flange part of the section. Some judgment is required to assess whether such openings are small, so that they can be neglected in design computations, or large enough to affect either shear or flexural strength. In the latter case due allowance must be made in both strength evaluation and detailing of the reinforcement. It is convenient to examine separately solid cantilever structural walls and those that are pierced with openings in some pattern.

(a) **Cantilever walls without Openings**: Most cantilever walls, such as shown in Fig.2.3(a), can be treated as ordinary reinforced concrete beam-columns. Lateral forces are introduced by means of a series of point loads through the floors acting as diaphragms. The floor slab will also stabilize the wall against lateral bucking, and this allows relatively thin wall sections, such as shown in Fig 2.2, to be used. In such walls it is relatively easy to ensure that when required, a plastic hinge at the base can develop with adequate plastic rotational capacity (3).

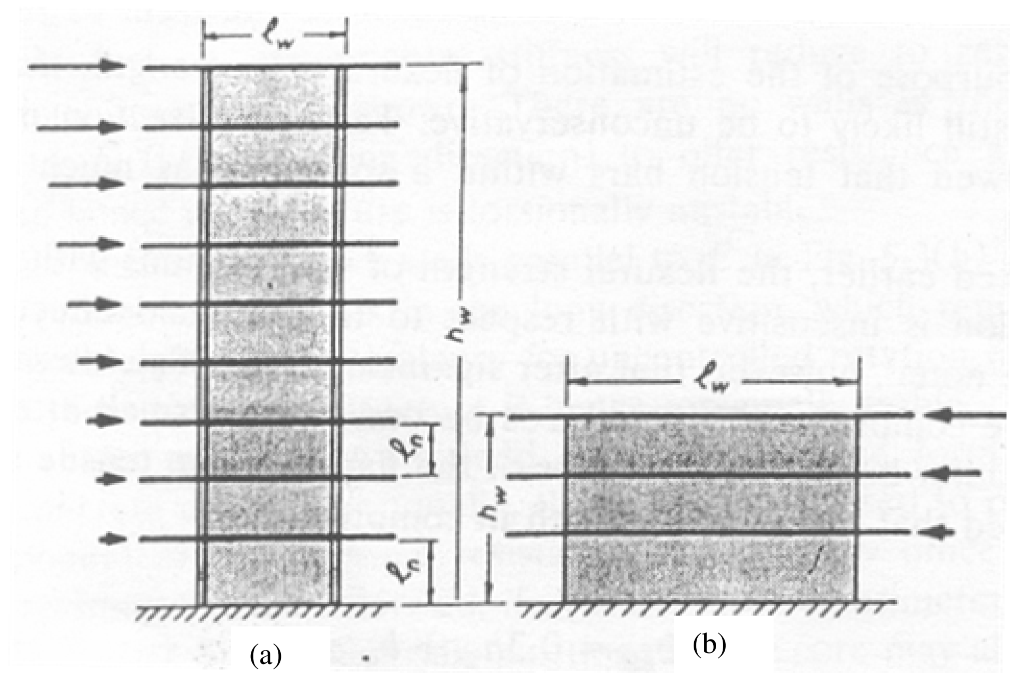


Fig 2.3 Cantilever structural walls

In low-rise buildings or in the lower stories of medium-to high-rise buildings, walls of the type shown in fig. 2.3(b) may be used. These are characterized by a small height-to-length ratio, h_w/l_w. The potential flexural strength of such walls may be very large in comparison with the lateral forces, even when code-specified minimum amounts of vertical reinforcement are used. Because of the small height, relatively large shearing forces must be generated to develop the flexural strength at the base. Therefore, the inelastic behavior of such walls is often strongly affected by effects of shear. It is possible to ensure inelastic flexural response. Energy dissipation, however, may be diminished by effects of shear. Therefore, it is advisable to design such squat walls for larger lateral force resistance in order to reduce ductility demands (3).

To allow for the effects of squatness, it has been suggested that lateral design force specified for ordinary structural walls be increased by the factor Z_1, where

$$1.0 < Z_1 = 2.5 - 0.5h_w/l_w < 2.0 \qquad (2\text{-}1)$$

(Reference R 3)

It is seen that this is applicable when the ratio $h_w/l_w < 3$. In most situations it is found that this requirement does not represent a penalty because of the great inherent flexural strength of such walls.

(b) **Structural walls with openings**: In many structural walls a regular pattern of openings will be required to accommodate windows or doors or both. When arranging openings, it is essential to ensure that a rational structure results. The designer must ensure that the integrity of the structure in terms of flexural strength is not jeopardized by gross reduction of wall area near the extreme fibers of the section. Similarly, the shear strength of the wall, in both the horizontal and vertical directions, should remain feasible and adequate to ensure that its flexural strength can be fully developed.

Extremely efficient structural systems, particularly suited for ductile response with very good energy-dissipation characteristics, can be conceived when openings are arranged in a regular and rational pattern. Examples are shown in fig. 2.4, where a number of walls are interconnected or coupled to each other by beams. For this reason they are generally referred to as coupled structural walls. The implication of this terminology is that the connecting beams, which may be relatively short and deep, are substantially weaker than the walls. The walls, which behave predominantly as cantilevers, can then impose sufficient rotations on these connecting beams to make them yield. If suitably detailed, the beams are capable of dissipating energy over the entire height of the structure. Two identical walls [fig.2.4(a)] or two walls of differing stiffnesses [Fig.2.4 (b)] may be coupled by a single line of beams. In other cases a series of walls may be interconnected by lines of beams between them, as seen in fig. 2.4(c). The coupling beams may be identical at all floors or they may have different depths or widths. In service cores, coupled walls may extend above the roof level, where lift machine rooms or space for other services are to be provided. In such cases walls may be considered to be interconnected by an infinitely rigid diaphragm at the top, as shown Fig 2.4(d).

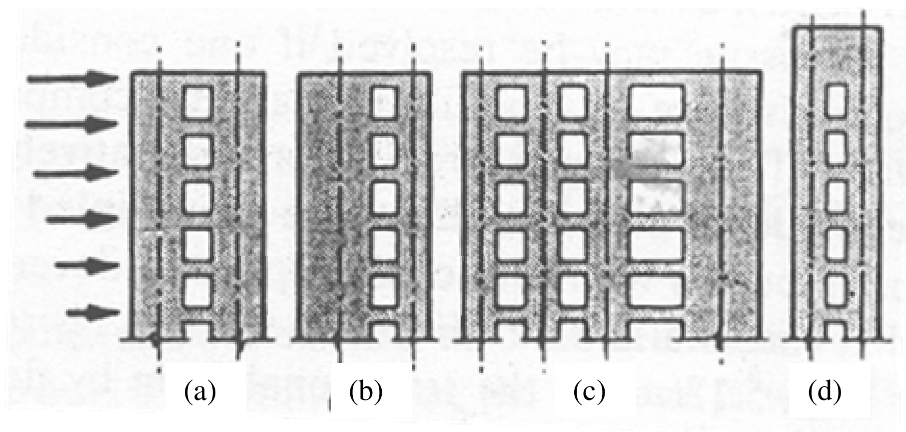


Fig 2.4 Types of coupled structural walls

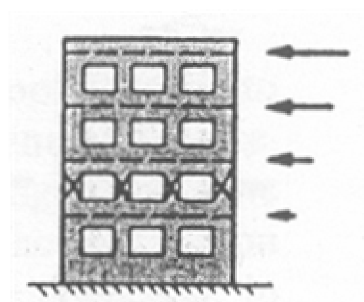


Fig 2.5 Undesirable pierced walls for earthquake resistance

2.3. Analysis Procedures

2.3.1. Modeling Assumptions

a) **Member stiffness**: To obtain reasonable estimates of fundamental period, displacements and distribution of lateral forces between walls, the stiffness properties of all elements of reinforced concrete wall structures should include an allowance for the effects of cracking. Displacement (0.75Δy) and lateral force resistance (0.75Si), relevant to wall stiffness estimate, are close to those that develop at first yield of the distributed longitudinal reinforcement (3).

i. The stiffness of cantilever walls subjected predominantly to flexural deformations may be based on the equivalent moment of inertia I_e of the cross section at first yield in the extreme fiber, which may be related to the moment of inertia I_g of the uncracked gross concrete section by the following expression:

$$I_e = \left(\frac{100}{f_y} + \frac{P_u}{f_c^{'} A_g} \right) I_g \qquad (2\text{-}2)$$

Where P_u is the axial load considered to act on the wall during an earthquake taken positive when causing compression and f_y is in MPa.

ii. For the estimation of the stiffness of diagonally reinforced coupling beams with depth h and clear span l_n,

$$I_e = 0.4 I_g / \left[1 + 3 (h / l_n)^2 \right] \qquad (2\text{-}3)$$

For conventionally reinforced coupling beams or coupling slabs,

$$I_e = 0.2 I_g / \left[1 + 3 (h / l_n)^2 \right] \qquad (2\text{-}4)$$

In the expression above, the subscripts e and g refer to the equivalent and gross properties, respectively.

iii. For the estimations of the stiffness of slabs connecting adjacent structural walls, the equivalent width of slab to compute I_g may be taken as the width of the wall b_w plus the width of the opening between the walls or eight times the thickness of the slab, whichever is less. The value is supported by tests with reinforced concrete slabs, subjected to cyclic loading (3).

iv. Shear deformations in cantilever walls with aspect ratios, h_w/l_w, larger than 4 may be neglected. When a combination of "slender" and "squat" structural walls provide the seismic resistance, the latter may be allocated an excessive proportion of the total lateral force if shear distortions are not accounted for. For such cases (i.e., when $h_w/l_w < 4$) it may be assumed that

$$I_w = \left(\frac{I_e}{1.2 + F} \right) \qquad (2\text{-}5)$$

Where $F = 30 I_e / h_w^2 b_w l_w$ $F = 30 I_e / h_w^2 b_w l_w$ (2-6)

Deflections due to code- specified lateral static forces may be determined with the use of the equivalent sectional properties above. However, for consideration of separation of nonstructural components and the checking of drift limitations, the appropriate amplification factors that make allowance for additional inelastic drift, given in codes, must be used.

b) **Geometric Modeling**: For cantilever walls it will be sufficient to assume that the sectional properties are concentrated in the vertical centerline of the wall (Fig. 2.4). This should be taken to pass through the centroidal axis of the wall section, consisting of the gross concrete area only. When cantilever walls are interconnected at each floor by a slab, it is normally sufficient to assume that the floor will act as a rigid diaphragm. By neglecting wall shear deformations and those due to torsion and the effects of restrained warping of an open wall section on stiffness, the lateral force analysis can be reduced to that of a set of cantilevers in which flexural distortions only will control the compatibility of deformations. Such analysis, based on first principles, can allow for the approximate contribution of each wall when it is subjected to deformations due to floor translations and torsion, as shown in section 2.3.2 below.

Such an elastic analysis, however approximate it might be, will satisfy the requirements of static equilibrium, and hence it should lead to satisfactory distribution also of internal actions among the walls of an inelastic structure (3).

When two or more walls in the same plane are interconnected by beams, as is the case in coupled walls shown in Fig. 2.4 and 2.5, in the estimation of stiffnesses, it will be necessary to account for more rigid end zones where beams frame into walls. Such structures are usually modeled as shown in Fig. 2.6. Standard programs written for frame analyses may then be used.

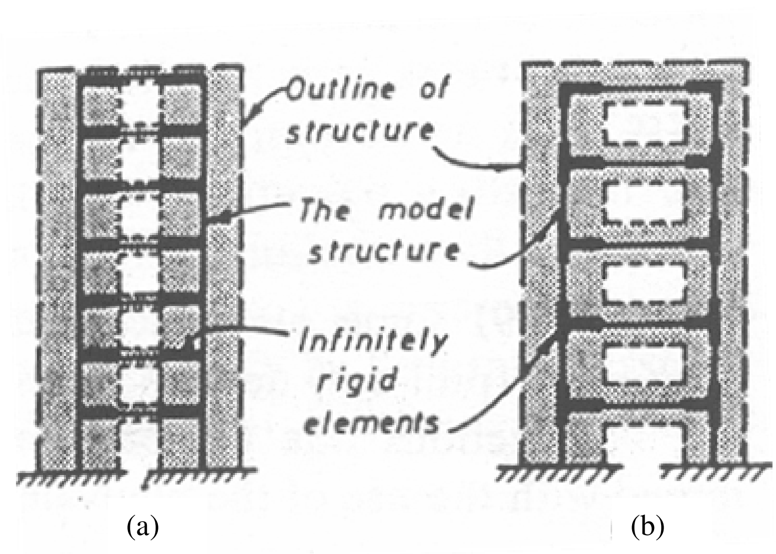


Fig 2.6 Modeling of deep-membered wall frames

Figure 2.7 illustrates the difficulties that arise. Structural properties are conventionally concentrated at the reference axis of the wall, and hence under the action of flexure only, rotation about the centroid of the gross concrete section is predicted, as shown in Fig 2.7, by line 1. After flexural cracking, the same rotation may occur about the neutral axis of the cracked section, as shown by line 2, and this will result in elongation Δ, measured at the reference axis. This deformation may affect accuracy, particularly when the dynamic response of the structure is evaluated. However, its significance in terms of inelastic response is likely to be small. It is evident that if one were to attempt a more accurate modeling by using the neutral axis of the cracked section as a reference axis for the model (Fig.2.6), additional complications would arise. The position of this axis would have to change with the height of the frame due to moment variations, as well as with the direction of lateral forces, which in turn might control the sense of the axial force on the walls. These difficulties may be overcome by employing finite element analysis techniques. However, in design for earthquake resistance involving inelastic response, this computational effort would seldom be justified (3).

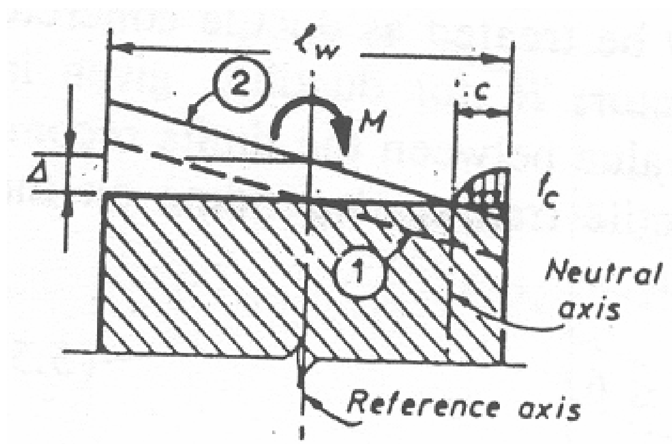


Fig 2.7 Effects of curvature on uncracked and cracked wall sections

c) **Analysis of wall sections**: The computation of deformations, stresses, or strength of a wall section may be based on the traditional concepts of equilibrium and strain compatibility, consistent with the plane section hypothesis. Because of the variability of wall section shapes, design aids, such as standard axial load-moment interaction charts for rectangular column sections, cannot often be used. Frequently, the designer will have to resort to the working out of the required flexural reinforcement from first principles. Programs to carry out the section analysis can readily be developed for minicomputers. Alternatively, hand analyses involving successive approximations for trial sections may be used.

The increased computational effort that arises in the section analysis for flexural strength, with or without axial load, stems from the multilayered arrangement of reinforcement and the frequent complexity of section shape .A very simple example of such a wall section is shown in Fig .2.8.It represents one wall of a typical coupled wall structure, such as shown in Fig .2.4. The four sections are intended to resist the design actions at four different critical levels of the structure. When the bending moment (assumed to be positive) causes tension at the more heavily reinforced right-hand edge of the section, net axial tension is expected to act on the wall. On the other hand, when flexural tension is induced at the left-hand edge of the section by (negative) moments, axial compression is induced in that wall.

The moments are expressed as the product of the axial load and the eccentricity, measured from the reference axis of the section, which, as stated earlier, is conveniently taken through the centroid of the gross concrete area rather than through that of the composite or cracked, transformed section. It is expedient to use the same reference axis also for the analysis of the cross section. It is evident that the plastic centroids in tension or compression do not coincide with the axis of the wall section. Consequently, the maximum tension or compression strength of the section, involving uniform strain across the entire wall section, will result in axial forces that act eccentrically with respect to the reference axis of the wall. These points are shown in Fig .2.8 by the peak values at the top and bottom meeting points of the four sets of curves (3). This representation enables the direct use of moments and forces, which have been derived from the analysis of the structural system, because in both analyses the same reference axis has been used.

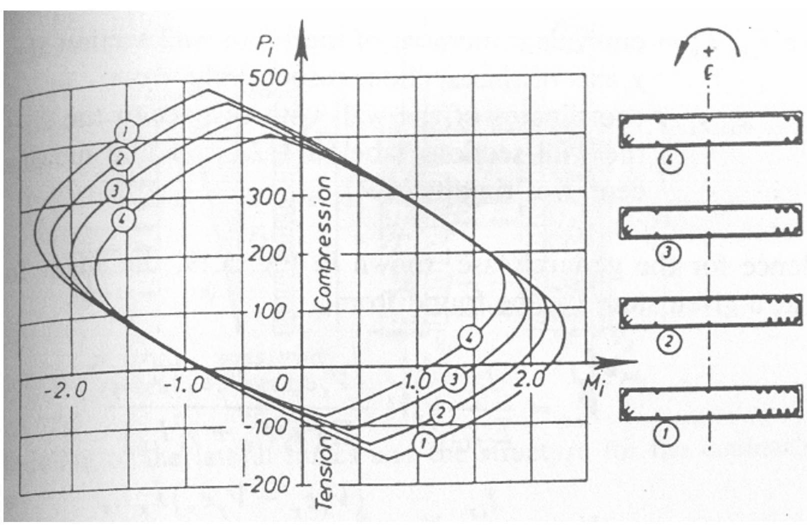


Fig 2.8 Axial load-moment interaction curves for unsymmetrically reinforced rectangular wall section

2.3.2. Analysis for Equivalent Lateral Static Forces

Generally, the choice of lateral design force level is determined based on site seismicity, structural configurations and materials, and building functions. The analysis to determine all internal design actions may then be carried out for the above lateral force level. The outline of analysis for Interacting cantilever structural wall systems is given in the following section.

The approximate elastic analysis for a series of interacting prismatic cantilever walls, such as shown in Fig. 2. 9, is based on the assumption that the walls are linked at each floor by infinitely rigid diaphragms which, however, has no flexural stiffness. Therefore, the three walls shown and so linked are assumed to be displaced by identical amounts at each floor. Each wall thus share in the resistance of a story force, F, or story shear, V, or overturning moment, M, in proportion to its own stiffness thus:

$$F_i = \frac{I_i}{\sum I_i} F \text{ or } V_i = \frac{I_i}{\sum I_i} V \text{ or } M_i = \frac{I_i}{\sum I_i} M \qquad (2\text{-}7)$$

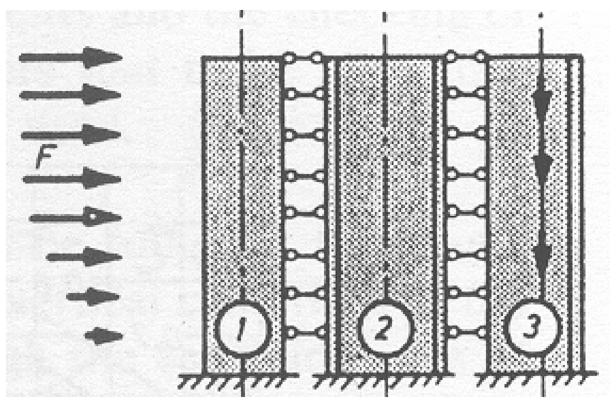


Fig 2.9 Model of interacting cantilever walls

The stiffness of rectangular walls with respect to their weak axis, relative to those of other walls, is so small that in general it may be ignored. It may thus be assumed that as for wall 1 in Fig 2.10 no lateral forces are introduced to such walls in the relevant direction .A typical arrangement of walls within the total floor plan is shown in Fig 2.10. The shear force, V, applied in any story and assumed to act at the point labeled CV in Fig 2.10 may be resolved for convenience into components V_x and V_y. Uniform deflection of all the walls would occur only if these component story shear forces acted at the center of rigidity (CR), the chosen center of the coordinate system for which the following conditions are satisfied:

$$\sum x_i I_{ix} = \sum y_i I_{iy} = 0 \qquad (2\text{-}8)$$

Where I_{ix},I_{iy} = equivalent moment of inertia of wall section about the x and y axis of that section, respectively

x_i,y_i= coordinates of the wall with respect to the shear centers of the wall sections Labeled 1,2…i and measured from the center of rigidity (CR)

Hence for the general case, shown in Fig. 2.10, the shear force for each wall at a given story can be found from (3) and (7)

$$V_{ix} = \frac{I_{iy}V_x}{\sum I_{iy}} + \frac{(V_x e_y - V_y e_x) y_i I_{iy}}{\sum (x_i^2 I_{ix} + y_i^2 I_{iy})} \qquad (2\text{-}9)$$

$$V_{iy} = \frac{I_{ix}V_y}{\sum I_{ix}} + \frac{(V_x e_y - V_y e_x) x_i I_{ix}}{\sum (x_i^2 I_{ix} + y_i^2 I_{iy})} \qquad (2\text{-}10)$$

Where ($V_x e_y$-$V_y e_x$) is the torsional moment of V about CR,$\sum(x^2_i I_{ix}+y^2_i I_{iy})$ is the rotational stiffness of the wall system, and e_x and e_y are eccentricities measured from the center of rigidity (CR) to the center of story shear (CV). Note that the value of e_y in Fig 2.10 is negative.

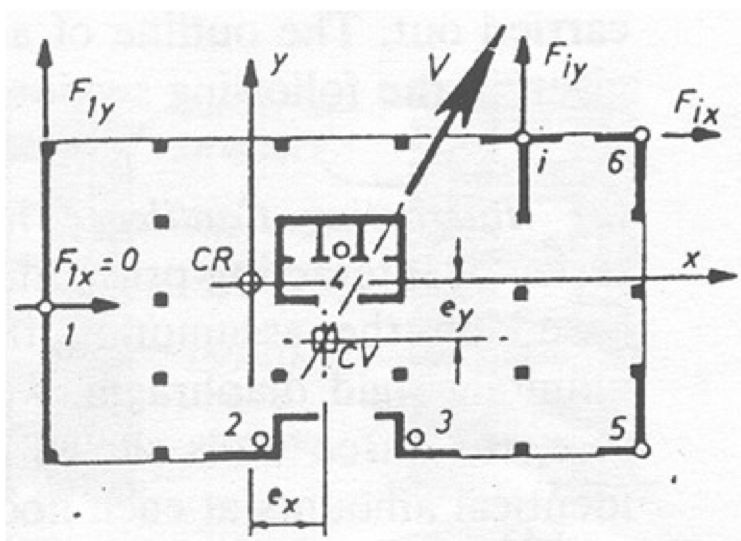


Fig 2.10 Plan layout of interacting cantilever walls

2.4. Design of Wall Elements for Strength and Ductility

2.4.1. Failure Modes in Structural Walls

A prerequisite in the design of ductile structural walls is that flexural yielding in clearly defined plastic hinge zones should control the strength, inelastic deformation, and hence energy dissipation in the entire structural system. As a corollary to this fundamental requirement, brittle failure mechanisms or even those with limited ductility should not be permitted to occur. As stated earlier, this is achieved by establishing a desirable hierarchy in the failure mechanics using capacity design procedures and by appropriate detailing of the potential plastic regions (3).

The principal source of energy dissipation in a laterally loaded cantilever wall (Fig 2.11) must be the yielding of the flexural reinforcement in the plastic hinge regions, normally at the base of the wall, as shown in Fig. 2.11(b) and (e). Failure modes to be prevented are those due to diagonal tension [Fig. 2.11 (c) or diagonal compression caused by shear, instability of thin walled sections or of the principal compression reinforcement, sliding shear along construction joints, shown in Fig 2.11(d), and shear or bond failure along lapped splices or anchorages. An example of the undesirable shear-dominated response of a structural wall to reversed cyclic loading is shown in Fig .2.12. Particularly severe is the steady reduction of strength and ability to dissipate energy.

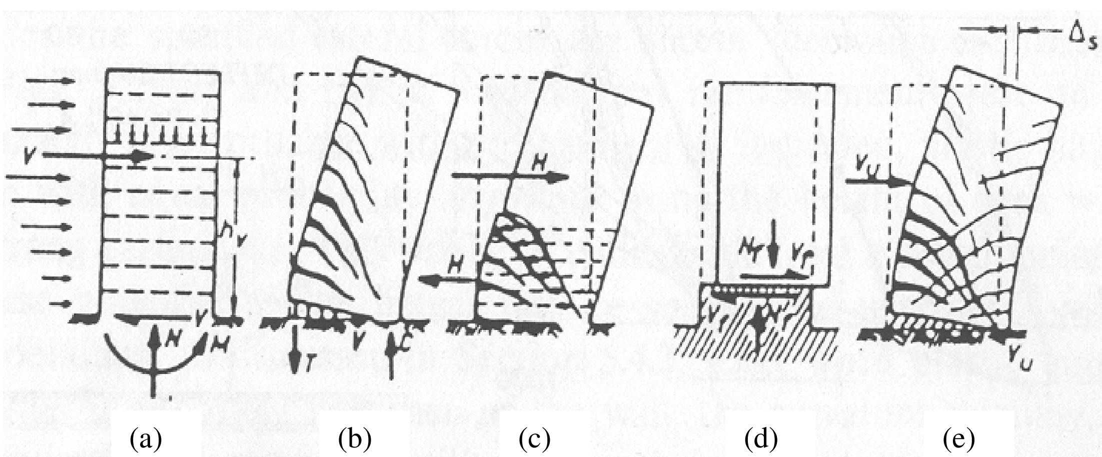


Fig 2.11 Failure modes in cantilever walls

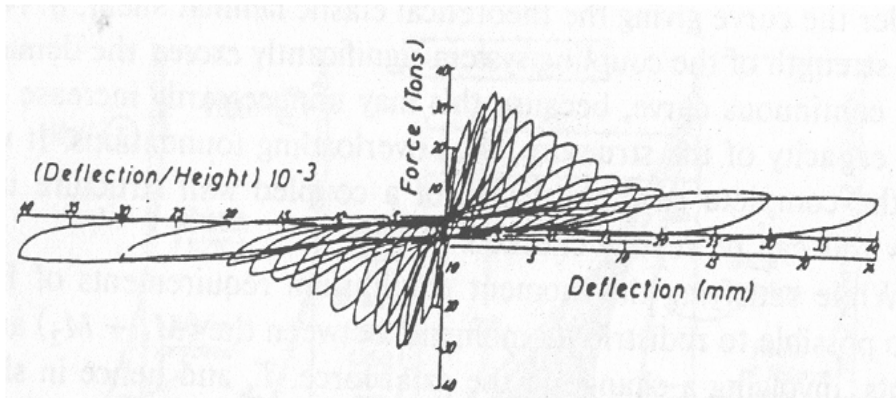


Fig 2.12 Hysteretic response of structural wall controlled by shear strength

2.4.2. Flexural Strength

a) **Design for Flexural strength:** Because of the multilayered arrangement of vertical reinforcement in wall sections, the analysis for flexural strength is a little more complex than that for beam sections. Therefore, in design, a successive approximation technique is generally used. This involves initial assumptions for section properties, such as dimensions, reinforcement content, and subsequent checking (i.e., analysis) for the adequacy of flexural strength. This first assumption may often be based on estimates which in fact can lead close to the required solution, and this is illustrated here with the example wall section shown in Fig. 2.13 (3).

Wall dimensions are generally given and subsequently may require only minor adjustments. Moment M and axial load P combinations with respect to the centroidal axis of the wall section are also known, thus the first estimate aims at finding the approximate quantity of vertical reinforcement in the constituent wall segments, such as 1, 2, and 3 in Fig 2.13. The amount of reinforcement in segment 2 is usually nominated and it often corresponds to the minimum recommended by codes. However, this assumption need not be made because any reinforcement in area 2 in excess of the minimum is equally effective and hence will correspondingly reduce the amounts required in the flange segments of the wall. By assuming that all bars in segment 2 will develop yield strength, the total tension force T_2 is found. Next we may assume that when $M_a = e_a P_a$, the center of compression for both concrete and steel forces C_1 is in the center of segment 1. Hence the tension force required in segment 3 can be estimated from

$$T_3 \approx \frac{x_a P_a - x_1 T_2}{x_1 + x_2} \qquad (2\text{-}11)$$

and thus the area of reinforcement in this segment can be found. Practical arrangement of bars can now be decided on. Similarly, the tension force in segment area 1 is estimated when $M_b = e_b P_b$ from

$$T_1 \approx \frac{x_b P_b - x_2 T_2}{x_1 + x_2} \qquad (2\text{-}12)$$

Further improvement with the estimates above may be made, if desired, by checking the intensity of compression forces. For example, when P_a is considered, we find that

$$C_1 = P_a + T_2 + T_3 \qquad (2\text{-}13)$$

and hence with the knowledge of the amount of reinforcement in segment 1, to provide the tension force T_1, which may now function as compression reinforcement, the depth of concrete compression can be estimated.

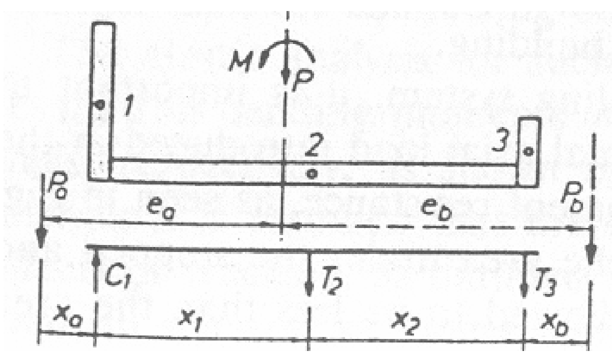


Fig 2.13 Example wall section

With these approximations the final arrangement of vertical bars in the entire wall sections can be made.

b) **Control of Shear:**

i. **Determination of Shear Force**: To ensure that shear will not inhibit the desired ductile behavior of wall systems and that shear effects will not significantly reduce energy dissipation during hysteretic response, it must not be allowed to control strength. Therefore, an estimate must be made for the maximum shear force that might need to be sustained by a structural wall during extreme seismic response to ensure that energy dissipation can be confined primarily to flexural yielding.

   For details on reduction factors for base shear one can refer specialized books.

ii. **Control of Diagonal Tension and Compression**: in inelastic regions it must be recognized that shear strength will be reduced as a consequence of reversed cyclic loading involving flexural rigidity. However, the uniform distribution of both horizontal and vertical reinforcement in the web portion of wall sections is considered to preserve better the integrity of concrete shear-resisting mechanisms (3).

3. Evaluation of Lateral Load Distribution, Column and Wall Design Results of ETABS

3.1. ETABS, the Software

General

ETABS (Extended Three-Dimensional Analysis of Building Systems) is special purpose analysis and design program developed specially for buildings.

Original development of TABS 30 years back led to the development of the today's ETABS. Early releases of ETABS provided input, output and numerical solution that took into consideration the characteristics unique to building type structures, providing a tool that offered significant savings in time and increased accuracy over general purpose programs.

As computers and computer interfaces evolved, ETABS added computationally complex analytical options such as dynamic nonlinear behavior, and powerful CAD-like drawing tools in a graphical and object-based interface.

The philosophical approach put forward by this program are namely:

- Most buildings are straightforward geometrically with horizontal beams and vertical columns. Although any building configuration is possible with ETABS, in most cases, a simple grid system defined by horizontal floors and vertical column lines can establish building geometry with minimal effort.
- Many of the floor levels in buildings are similar. This commonality can be used numerically to reduce computational effort.
- The input and output conventions used correspond to common building terminology. With ETABS, models are defined logically floor-by-floor, column-by-column, bay-by-bay and wall-by-wall and not as a stream of non-descript nodes and elements as in general purpose programs. Thus the structural definition is simple, concise and meaningful.
- In most buildings, the dimensions of the members are large in relation to the bay widths and story heights. Those dimensions have significant effect on the stiffness of the frame. ETABS corrects for such effects in the formulation of the member stiffness, unlike most general-purpose programs that work on centerline-to-centerline dimension.

- The results produced by the programs should be in a form directly usable by the engineer. General-purpose computer programs produce results in a general form that may need additional processing before they are usable in structural design.

ETABS offers the widest assortment of analysis and design tools available for the structural engineer working on building structures. The following list represents just a portion of the types of systems and analysis that ETABS can handle easily (6).

- Multi-story commercial, government and health care facilities
- Parking garages with circular and linear ramps
- Staggered truss buildings
- Buildings with steel, concrete, composite or joist floor framing
- Buildings based on multiple rectangular and/or cylindrical grid systems
- Flat and waffle slab concrete buildings
- Buildings subjected to any number of vertical and lateral load cases and combinations, including automated wind and seismic loads
- Multiple spectrum load cases, with built-in input curves
- Automated transfer of vertical loads on floors to beams and walls
- P-Delta analysis with static or dynamic analysis
- Explicit panel-zone deformations
- Construction sequence loading analysis
- Multiple linear and nonlinear time history load cases in any direction
- Foundation/support settlement
- Large displacement analysis
- Nonlinear static pushover
- Buildings with base isolators and dampers
- Floor modeling with rigid or semi-rigid diaphragms
- Automated vertical live load reductions

Physical Modeling Terminologies in ETABS

In ETABS objects, members, and elements are often referred. Objects represent the physical structural members in the model. Elements, on the other hand, refer to the finite elements used internally by the program to generate the stiffness matrices (6). In many cases objects and physical members will have a one-to-one correspondence, and it is these objects that the user draws in the ETABS interface.

In ETABS, objects or physical members drawn by users, are typically subdivided into the greater number of finite elements needed for the analysis model, without user input.

Structural Objects

ETABS uses objects to represent physical structural members. The following objects are available in ETABS:

- Point objects
- Line objects and
- Area objects

Area objects are used to model walls, slabs, decks, planks, and other thin-walled members. Area objects will be meshed automatically into the elements needed for analysis if horizontal objects with the membrane definition are included in the model; otherwise, the user should specify the meshing option to be used (6).

ETABS Design Settings

ETABS offers the following integrated design postprocessors:

- Steel frame design
- Concrete frame design
- Composite beam design
- Steel joist design
- Shear wall design

ETABS Shear Wall Modeling

In ETABS shear wall design is available for objects that are defined as piers and spandrels by the user. One must assign a pier or spandrel element a label before he can get output forces for the element or before one can design the element.
Pier labels are assigned to vertical area objects (walls) and to vertical line objects (columns).
Spandrel labels are assigned to vertical area objects (walls) and to horizontal line objects (beams) (6).
After a wall pier has been assigned a label and an analysis has been run, forces can be output for the wall pier and it can be designed. Wall pier forces are output at the top and bottom of wall pier elements. Also, wall pier design only performed at stations located at the top and bottom of wall pier elements.

ETABS Shear Wall Analysis and Design Sections

In this software analysis sections are simply the objects defined in the model that make up the pier or spandrel section. The analysis section for wall piers is the assemblage of wall and column sections that make up the pier. Similarly, the analysis section for spandrels is the assemblage of wall and beam sections that make up the spandrel.

The analysis is based on these section properties, and thus, the design forces are based on these analysis section properties.

The design section is completely separate from the analysis section. Three types of pier design sections are available (6). They are:

- **Uniform Reinforcing Section**: For flexural design and/or checks the program automatically (and internally) creates a Section Designer pier section of the same shape as the analysis section pier. Uniform reinforcing is placed in this pier. The Uniform Reinforcing Section pier may be planar or it may be three-dimensional.
- **General Reinforcing Section**: For flexural designs and/or checks, the pier geometry and the reinforcing is defined by the user in the Section Designer utility.

The pier defined in Section Designer may be planar or it may be three-dimensional.

- **Simplified Pier Section**: This pier section is defined in the pier design overwrites. The simplified section is defined by a length and a thickness. The length is in the pier 2-axis direction and the thickness is in the pier 3-axis direction.

For shear design, in all the above flexural design sections, the program automatically (and internally) breaks the analysis section pier up into planar legs and then performs the design on each leg separately and reports the results separately for each leg.

The program designs wall piers at stations located at the top and bottom of the pier only. To design at the mid-height of a pier, break the pier into two separate “half-height piers.

ETABS Shear Wall Design for BS8110 97

Flexural Design for the Uniform Reinforcing Section

Interaction surface

In this program, a three-dimensional interaction surface is defined with reference to the P, M_2, M_3 axes. The surface is developed using a series of interaction surfaces that are created by rotating the direction of the pier axis in equally spaced increments around a 360 degree circle. For example, if 24 PMM curves are specified in the program (the default), there is one curve every 360^0/24 curves = 15^0. Figure 3.1 illustrates the assumed orientation of the pier neutral axis and the associated sides of the neutral axis where the section is in tension (designate T in the figure) or compression (designated C in the figure) for various angles (6).

Each PMM interaction curve that makes up the interaction surface is numerically described by a series of discrete points connected by straight lines. The coordinates of these points are determined by rotating a plane of linear strain about the neutral axis on the section of the pier. By default, 11 points are used to define a PMM interaction curve. One can change this number in the preferences.

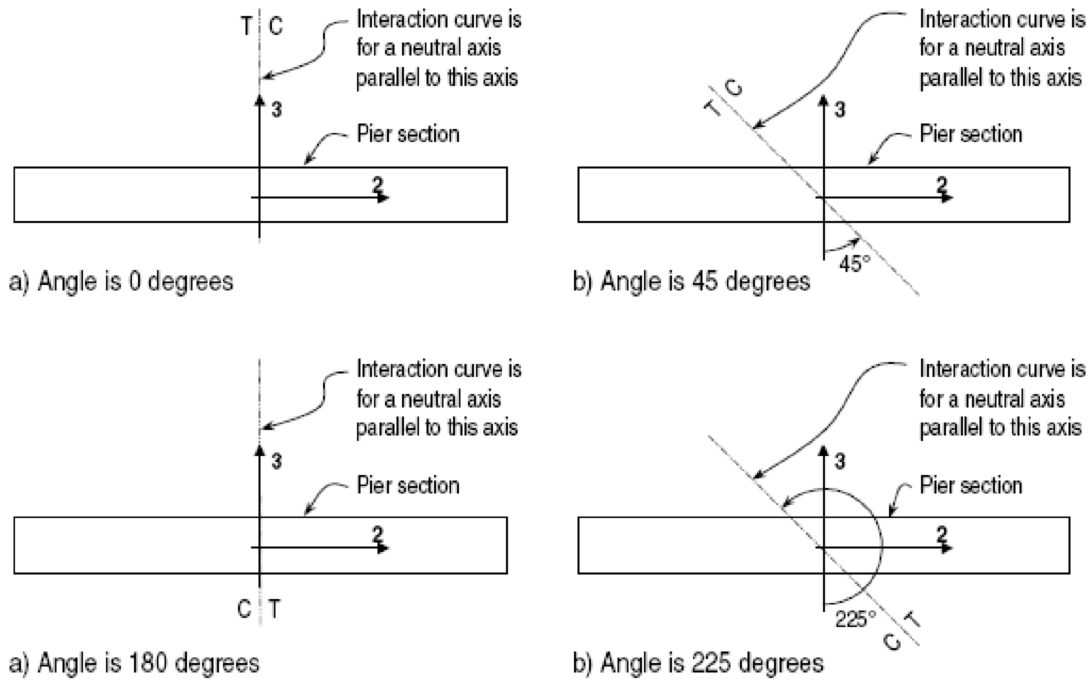


Figure 3.1: Orientation of the Pier Neutral Axis for Various Angles

Formulation of the Interaction Surface

The formulation of the interaction surface in this program is based consistently on the basic principles of ultimate strength design given in BS 8110-97 code.

The program uses the requirements of force equilibrium and strain compatibility to determine the design axial load and moment strength (P_r, M_{2r}, M_{3r}) of the wall pier. For the pier to be deemed adequate, the required strength (P, M_2, M_3) must be less than or equal to the design strength, as indicated in the following equation.

$$(P,\ M_2\ M_3) \leq (P_r,\ M_{2r},\ M_{3r}) \qquad (3\text{-}1)$$

The effects of the partial safety factors for concrete (g_c=1.50) and for steel (g_s=1.05) are included in the generation of the interaction curve.

g_c= Partial safety factor for concrete.

g_s= Partial safety factor for reinforcing steel.

The theoretical maximum compressive force that the wall pier can carry is designated $P_{r,max}$ and is given by:

$$P_{r,max}= 0.67(f_{cu}/\gamma c)(A_g-A_s)+(f_y/\gamma_s)A_s \quad (3\text{-}2)$$

The theoretical maximum tension force that the wall pier can carry is designated $P_{t,\ max}$ and is given by:

$$P_{t,max} =(f_y/\gamma_s)\mathbf{A_s} \qquad (3\text{-}3)$$

If the wall pier geometry and reinforcing is symmetrical in plan, the moments associated with both $P_{r,\ max}$ and $P_{t,\ max}$ are zero. Otherwise, there will be a moment associated with both $P_{r,\ max}$ and $P_{t,\ max}$.

In addition to $P_{r,max}$ and $P_{t,max}$, the axial load at the balanced strain condition, i.e.,P_b, is also determined. In this condition, the tension reinforcing reaches the strain corresponding to its specified yield strength modified by the corresponding partial factor of safety, $f_y/\ \gamma_s$, as the concrete reaches its assumed ultimate strain of 0.0035.

Details of the Strain Compatibility Analysis

As previously mentioned, the program uses the requirements of force equilibrium and strain compatibility to determine the deign axial capacity and moment strength (P_r, M_{2r}, M_{3r}) of the wall pier. The coordinates of these points are determined by rotating a plane of linear strain on the section of the wall pier (6).

Figure 3.2 illustrates varying planes of linear strain such as those that the program considers on a wall pier section for a neutral axis orientation angle of 0 degrees. In these planes, the maximum concrete strain is always taken as -0.0035 and the maximum steel strain is varied from -0.0035 to plus infinity. When the steel strain is -0.0035, the maximum compressive force in the wall pier, $P_{r,max}$, is obtained from the strain compatibility analysis. When the steel strain is plus infinity, the maximum tensile force in the wall pier, $P_{t,\ max}$, is obtained. When the maximum steel strain is equal to the yield strain for the reinforcing, P_b is obtained.

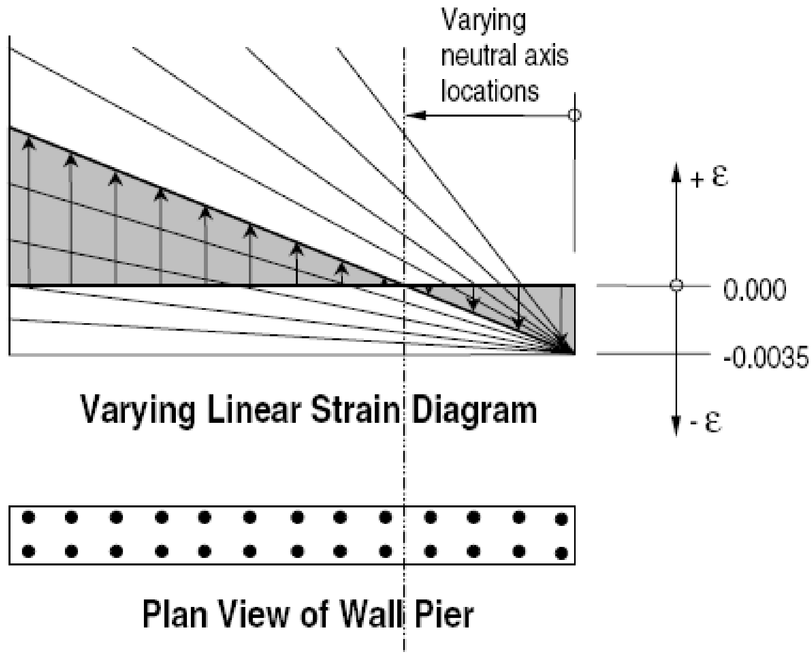


Figure 3.2: Varying Planes of Linear Strain

Figure 3.3 illustrates the concrete wall pier stress- strain relationship that is obtained from a strain compatibility analysis of a typical plane of linear strain shown in Figure 3.2.

In Figure 3.3 the compressive stress in the concrete, C_c, is calculated using the following equation:

$$C_c = 0.67\,(f_{cu}/\gamma_c)(0.9 * t_p) \qquad (3\text{-}4)$$

In Figure 3.2, the value for maximum strain in the reinforcing steel is assumed. Then the strain in all other reinforcing steel is determined based on the assumed plane of linear strain. Next the stress in the reinforcing steel is calculated using the following equation, where ε_s is the strain, E_s is the modulus of elasticity, σ_s is the stress, and f_y is the characteristic yield strength of the reinforcing steel.

$$\sigma_s = \varepsilon_s E_s \leq f_y/\gamma_s$$

The force in the reinforcing steel (T_s for tension or C_s for compression) is calculated using the following equation:

$$T_s \text{ or } C_s = \sigma_s A_s \qquad (3\text{-}5)$$

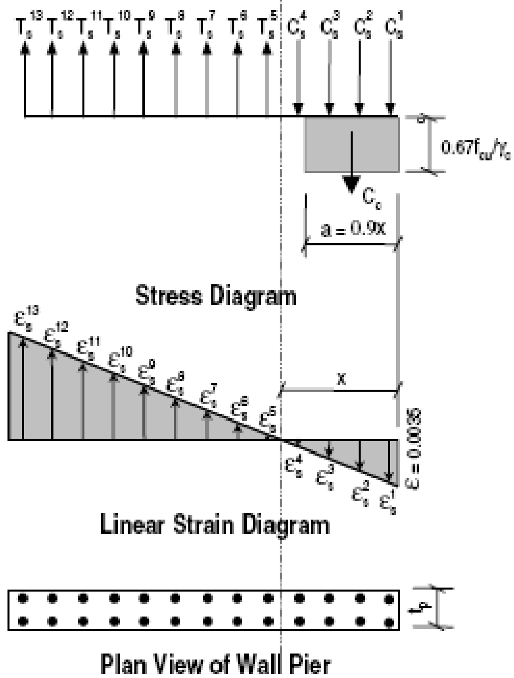


Figure 3.3: Wall Pier Stress-Strain Relationship

For the given distribution of strain, the value of Pr is calculated using the following equation:

$$P_r = (\Sigma T_s - C_c - \Sigma C_s) \leq P_{max} \qquad (3\text{-}6)$$

In the previous equation, the tensile force T_s and the compressive forces C_c and C_s are all positive. If P_r is positive, it is tension, and if it is negative, it is compression. The term P_{max} is taken as $P_{r,\,max}$ if P_r is compressive, and as $P_{t,\,max}$ if P_r is tensile.

The value of M_{2r} is calculated by summing the moments resulting from all of the forces about the pier local 2-axis .Similarly, the value of M_{3r} is calculated by summing the moments resulting from all of the forces about the pier local 3-axis. The forces whose moments are summed to determine M_{2r}, M_{3r} and P_r, are C_c, all of the T_s forces and all of the C_s forces.

The P_r, M_{2r} and M_{3r} values calculated as described above make up one point on the wall pier interaction diagram. Additional points on the diagram are obtained by making different assumptions for the maximum steel stress; that is considering a different plane of linear strain, and repeating the process.

When one interaction curve is complete, the next orientation of the neutral axis is assumed and the points for the associated new interaction curve are calculated. This process continues until the points for all of the specified curves have been calculated.

Determine the Concrete Shear Capacity

Given the design force set P, M and V acting on a wall pier section, the shear stress carried by the concrete, V_c, is calculated as follows:

$$V_c^{'} = V_c + 0.6\frac{N}{A_c}\frac{V_d}{M} \leq V_c\sqrt{1+\frac{N}{A_c V_c^{'}}} \qquad (3\text{-}7)$$

Where

$$V_c = R_{LW}\frac{0.79K_1K_2}{\gamma_m}\left(\frac{100A_s}{bd}\right)^{1/3}\left(\frac{400}{d}\right)^{1/4} \qquad (3\text{-}8)$$

R_{LW} is a shear strength reduction factor that applies to lightweight concrete. It is equal to1 for normal weight concrete. This factor is specified in the concrete material properties (6).

K_1 is the enhancement factor for support compression and taken conservatively as 1,

$$K_2 = \left(\frac{f_{cu}}{25}\right)^{1/3}$$

γ_m=1.25,

$$0.15 \leq \frac{100A_s}{bd} \leq 3,$$

$$\frac{400}{d_v} \geq 1,$$

$$\frac{Vd_v}{M} \leq 1,$$

$f_{cu} \leq 40$ N/mm^2,

As is area of tensile steel and it is takes as half the total reinforcing steel area, and

d_v is the distance from extreme compression fiber to the centroid of the tension steel. It is taken as $0.8L_p$.

If the tension is large enough that V_c' results in a negative number, V_c is set to zero.

Determine the Required Shear Reinforcing

Given V and V_c, the following procedure provides the required shear reinforcing in area per unit length (e.g., mm^2/mm or optionally cm^2/m) for wall piers.

- Calculate the design shear stress from

$$v = \frac{V}{A_{CV}} \tag{3-9}$$

$$v_{\max} = \min\left\{0.8 R_{LW}\sqrt{f_{cu}}, 5MPa\right\}$$

$$v \le 0.8 R_{LW}\sqrt{f_{cu}}$$

$$v \le 5N/mm^2 \quad v \le 5N/mm^2, \text{ and}$$

$$A_{CV} = t_p d_V$$

- If v exceeds $0.8\ R_{Lw}\ f_{cu}^{1/2}$ or 5 N/mm^2, the concrete section area should be increased. In that case, the program reports an overstress.
- If $v \le v_c' + 0.4$, provide minimum links defined by:

$$\frac{A_{SV}}{S_V} \ge \frac{0.4 t_p}{0.95 f_y}$$

- Else if $v \le v_c' + 0.4 < v < v_{max}$, provide links given by

$$\frac{A_{SV}}{S_V} \ge \frac{\left(v - v_c'\right) t_p}{0.95 f_y}$$

and else if $v > v_{max}$, a failure condition is declared.

A_{sv}/S_v is the horizontal shear reinforcing per unit vertical length (height) of the wall pier.

In shear design, f_y cannot be taken as greater than 460 MPa. If f_y for shear rebar is defined as greater than 460MPa, the program deigns shear rebar based on f_y equal to 460 MPa.

Note:

1. The program reports an over stress message when shear stress exceeds $8R_{LW}\, f_{cu}^{1/2}$ or 5 N/mm^2.
2. One can set the output units for the distributed shear reinforcing in the shear wall design preferences.

3.2. Evaluation of Lateral Load Distribution Results of ETABS

As stated in section 1.2 of this paper, one of the goals of the thesis work is to investigate the relative proximity and/or divergence of the results of lateral load distribution among walls using the approximate elastic analysis described in section 2.3.2 and the corresponding finite element solutions by ETABS. Hence, this work begins with analysis of three simple buildings that inherit their lateral-force resisting capabilities solely from stable arrangement of structural walls. The lateral load distribution result will then be extracted for comparison. Walls having simple rectangular, L, and C cross-sections will be placed at the periphery of a four story building as shown in Fig 3.4. In the first case, in order not to include effects of floor slabs to the lateral load distribution among the walls, the wall systems are modeled just with diaphragms at each floor level with out the actual floor slabs. However, in the second case the walls will be modeled with the floor slabs and the lateral load distribution result will be compared with those with floors with just diaphragms.

It is assumed that C30 concrete and S400 steel are used to construct the walls.

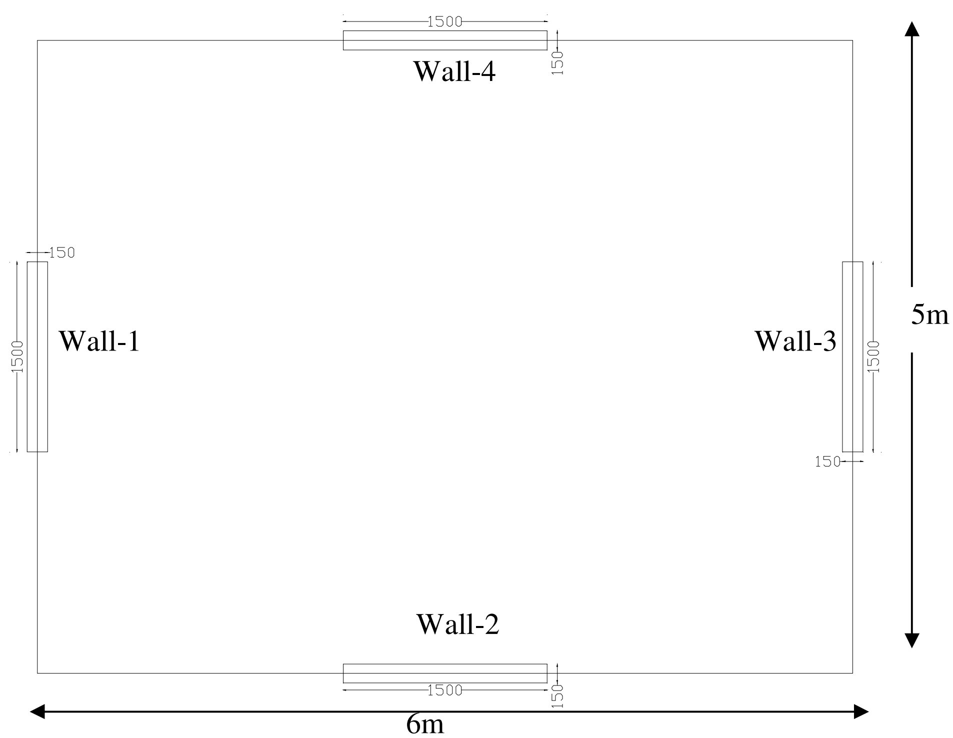


Fig 3.4(a) Wall arrangment-1

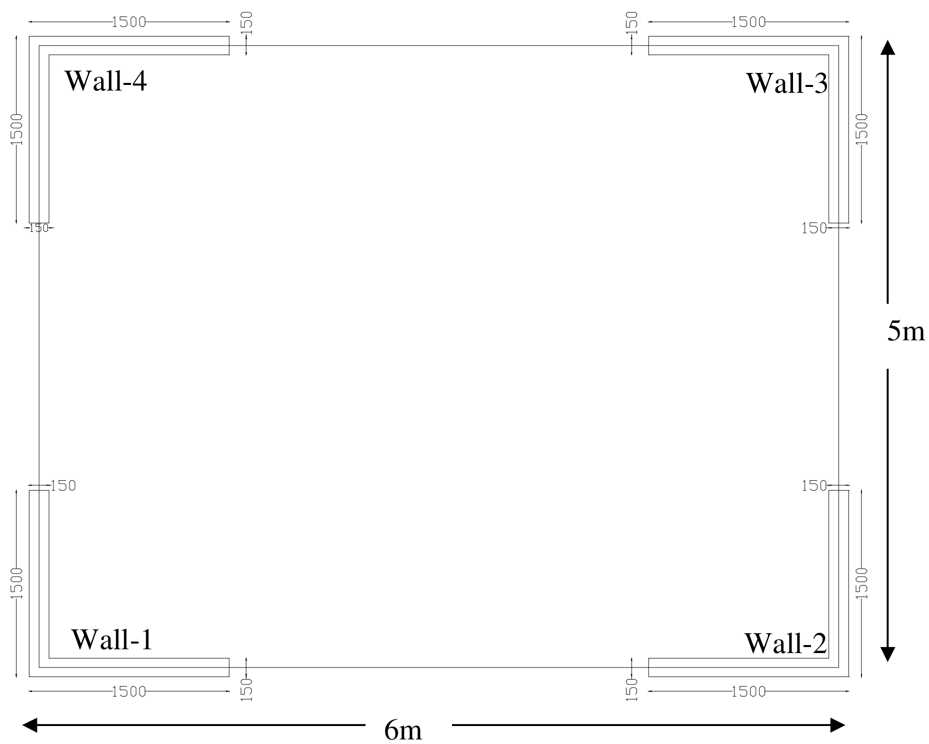


Fig 3.4(b) Wall arrangment-2

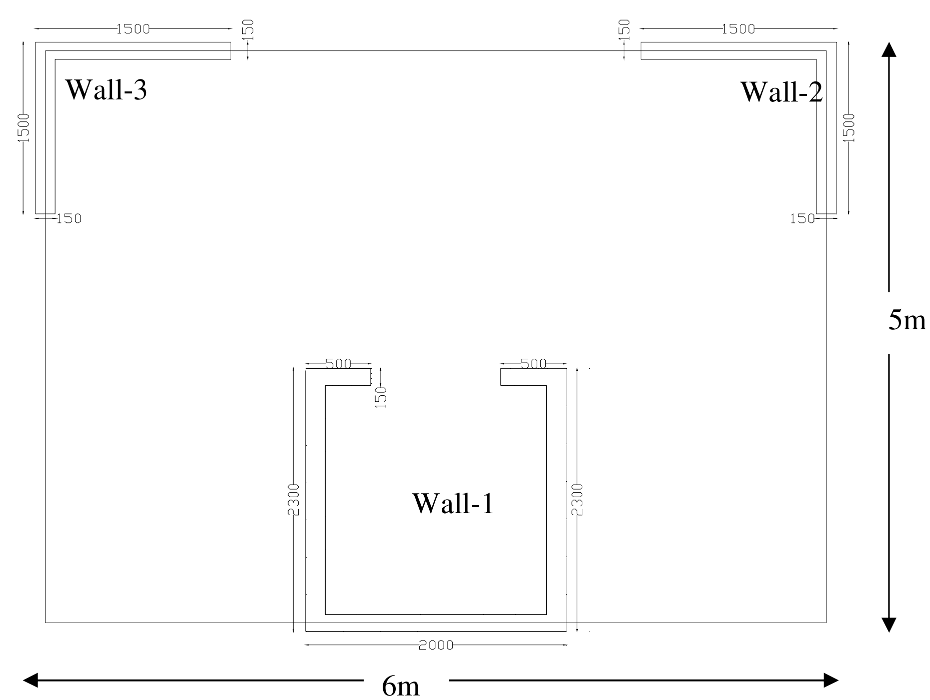


Fig 3.4(c) Wall arrangment-3

STORY DATA FOR BOTH THE MODLES

The following story data is common for all the three arrangements under consideration.

Story	Height (m)	Elevation (m)
Base	3	0
Ground	3	3
1st	3	6
2nd	3	9
Roof	-	12

Table 3.1 Story Data

LOADING DATA FOR BOTH THE MODELS

Because it is not the purpose of this study to determine the exact seismic forces on the wall system only arbitrarily assumed lateral forces are taken. The lateral loads assumed for the three wall arrangements at each story level are tabulated here under.

Arrangment-1

Story	V_x (kN)
Base	0
Ground	10
1st	40
2nd	60
Roof	80

Arrangment -2

Story	V_x (kN)
Base	0
Ground	20

1st	80
2nd	120
Roof	160

Arrangement -3

Story	V_x (kN)
Base	0
Ground	150
1st	200
2nd	250
Roof	300

Table 3.2 Load data for the three wall arrangements

Load Distribution Results of ETABS for the Model with Diaphragms only

The assumed Lateral loads given in the table above are applied at the center of rigidity of the respective wall system arranged at the periphery of the 6m by 5m grid system in Fig 3.4. The wall systems are analyzed and the lateral load distribution is determined. It is to be noted that wall forces are reported at the centroid of each analysis section. Three dimensional diagrams of this modeling assumption are shown in Fig. 3.5.

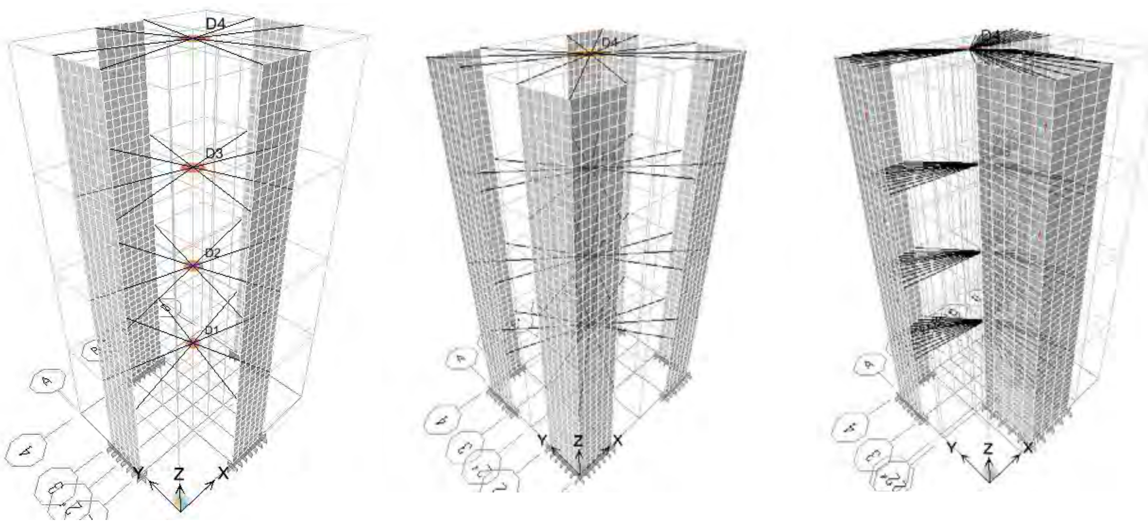


Fig 3.5 ETABS models with floor diaphragms only

Wall arrangment-1

The lateral load distribution among walls of the first wall arrangement (Fig 3.4a) is tabulated here under.

Wall No	Story	Shear, Vx (kN)	Shear, Vy (kN)
1	Base	1.29	0.00
	Ground	0.8	0.00
	1st	0.73	0.00
	2nd	0.38	0.00
2	Base	93.71	0.00
	Ground	89.2	0.00
	1st	69.27	0.00
	2nd	39.62	0.00
3	Base	1.29	0.00
	Ground	0.8	0.00
	1st	0.73	0.00
	2nd	0.38	0.00
4	Base	93.71	0.00
	Ground	89.2	0.00
	1st	69.27	0.00
	2nd	39.62	0.00

Table 3.3 Lateral load distribution among walls of the first wall arrangement for the model with diaphragm

Wall arrangment -2

The lateral load distribution among walls of the second wall arrangement (Fig 3.4b) is tabulated here under.

Wall No	Story	Shear, Vx (kN)	Shear, Vy (kN)
1	Base	95	-43.68
	Ground	90	-53.48
	1st	70	-41.56
	2nd	40	-25.31

2	Base	95	43.68
	Ground	90	53.48
	1st	70	41.56
	2nd	40	25.31
3	Base	95	-43.68
	Ground	90	-53.48
	1st	70	-41.56
	2nd	40	-25.31
4	Base	95	43.68
	Ground	90	53.48
	1st	70	41.56
	2nd	40	25.31

Table 3.4 Lateral load distribution among walls of the second wall arrangement for the model with diaphragm

Wall arrangment -3

The lateral load distribution among walls of the third wall arrangement (Fig 3.4c) is tabulated here under.

Wall No	Story	Shear, Vx (kN)	Shear, Vy (kN)
1	Base	505.53	0.00
	Ground	433.98	0.00
	1st	316.18	0.00
	2nd	175.01	0.00
2	Base	197.23	-96.101
	Ground	158.01	-121.67
	1st	116.91	-86.74
	2nd	62.50	-53.71
3	Base	197.23	96.101
	Ground	158.01	121.67
	1st	116.91	86.74

	2nd	62.50	53.71

Table 3.5 Lateral load distribution among walls of the third wall arrangement for the model with diaphragm

Lateral Load Distribution Results of ETABS for the Model with Floor Slabs

The wall systems discussed above are also modeled as a wall system with floor slabs to see effect of the slabs. The three dimensional models are as shown in Fig. 3.6.

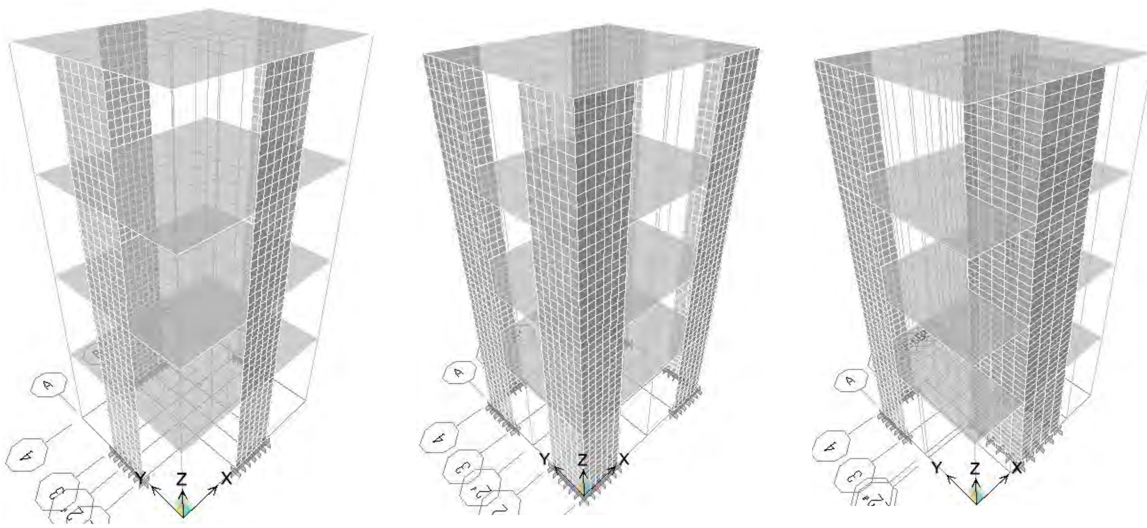
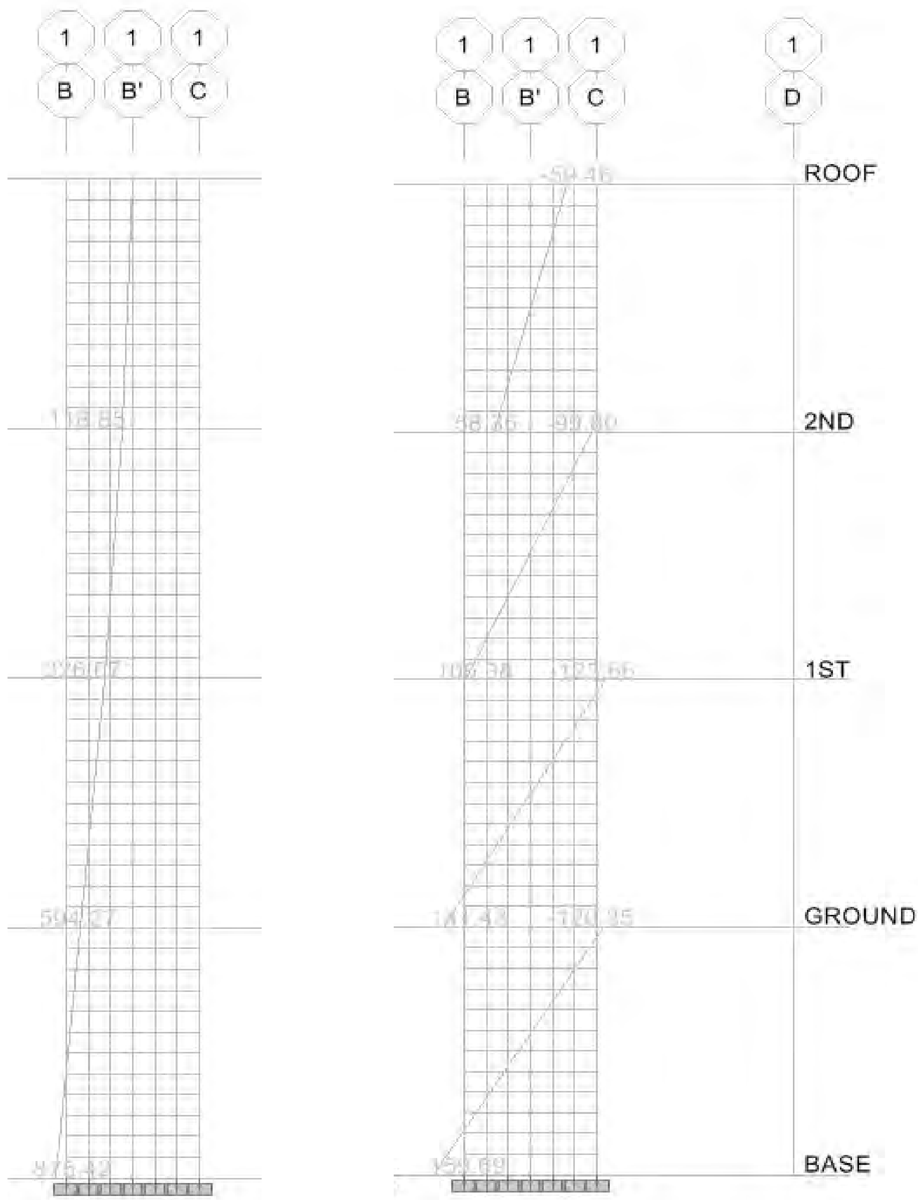


Fig 3.6 ETABS models with floor slabs

The previously assumed loads are applied and the moment diagrams for the corresponding axes are shown in Figs. 3.7 and 3.8. Comparison of the bending moment diagrams indicate the effect of coupling by the slabs.

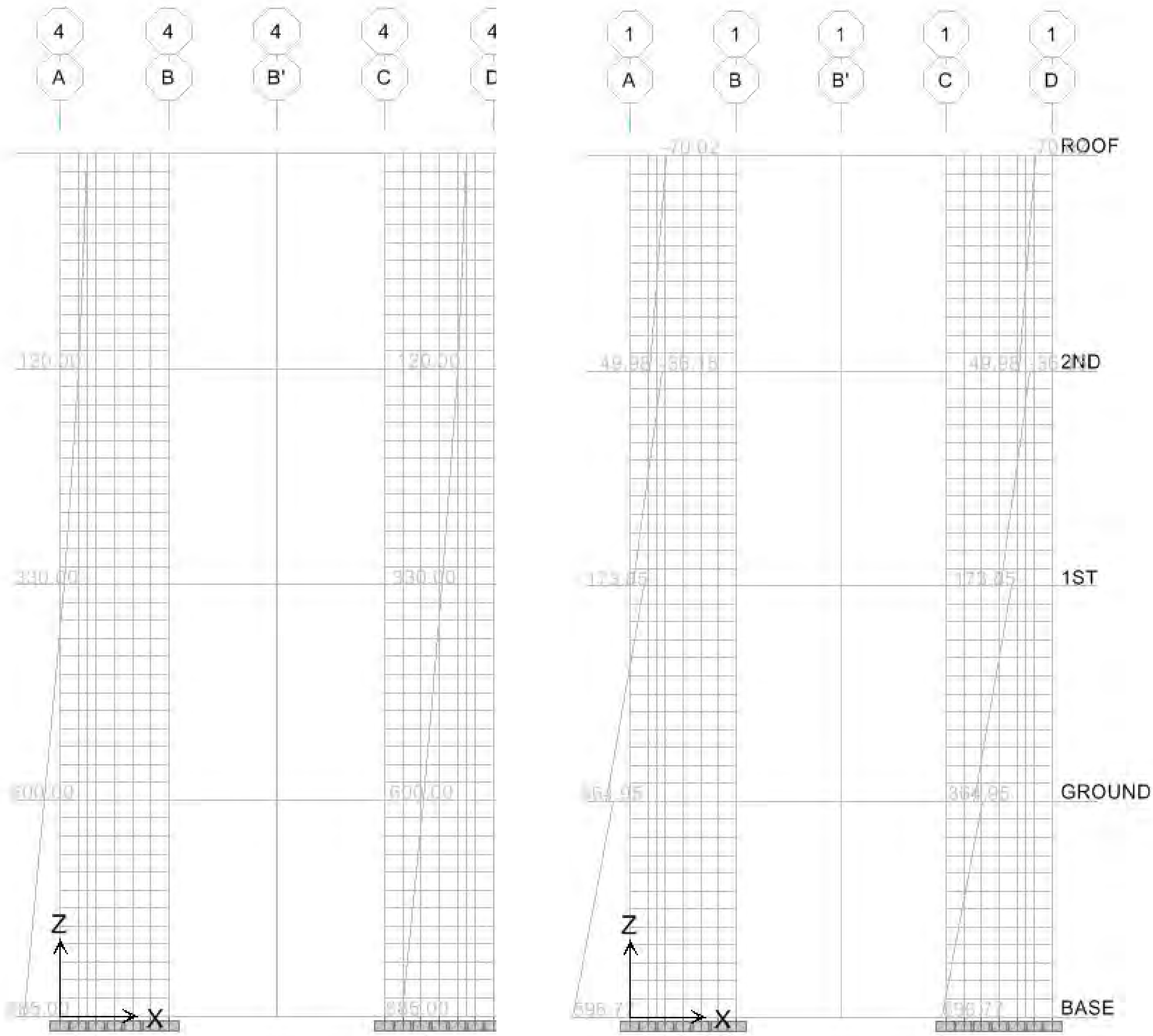
Comparison between the moment values from the two models

For the sake of visualization and comparison of the bending moment results from the two models, bending moment diagrams of typical wall will be shown side by side for the axes parallel to the plane of the applied force Fx.



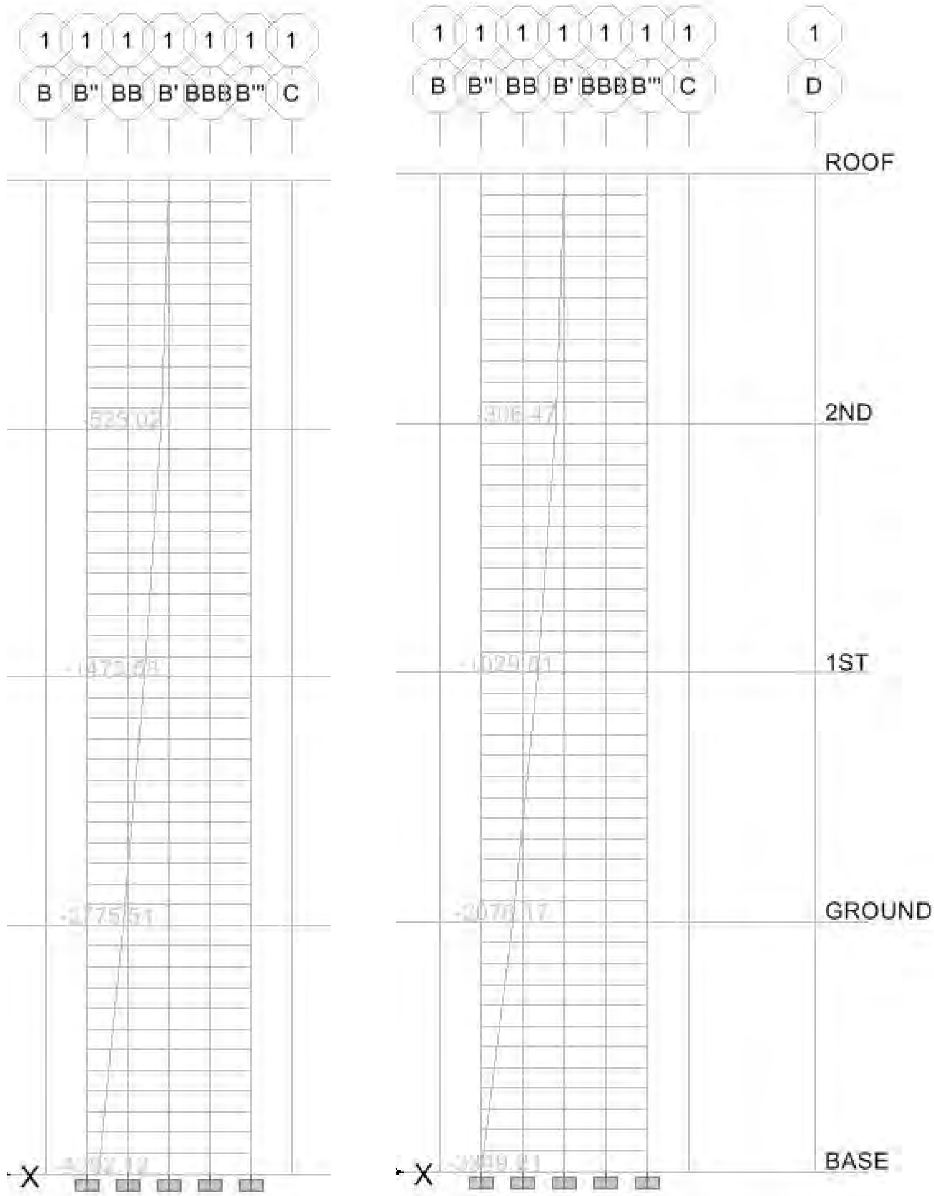
(a) From model-1 (b) From model-2

Fig. 3.7 Moment diagram for wall-1 of arrangement-1



(a) From model-1 (b) From model-2

Fig. 3.8 Moment diagram for wall-1 and 2 of arrangement -2



(a) From model-1 (b) From model-2

Fig. 3.9 Moment diagram for wall-1 of arrangement -3

Comparison of the Lateral Load Distribution of the Approximate Elastic Analysis and the FEM for the model with Diaphragms only

Those assumed Lateral loads given in the tables above and fed in to the ETABS are applied at the center of mass of the respective wall systems to perform approximate elastic analysis. The lateral load is distributed among the walls, in the relevant directions, based on their stiffness according to the approximate elastic analysis in section 2.3.2. In this case also it is to be noted that wall forces are reported at the shear center of each analysis section and the results are as given below.

Wall arrangment-1

Wall No	Story	Shear, Vx (kN)	Shear, Vy (kN)
1	Base	0.94	0.00
	Ground	0.89	0.00
	1st	0.69	0.00
	2nd	0.40	0.00
2	Base	94.06	0.00
	Ground	89.11	0.00
	1st	69.31	0.00
	2nd	39.60	0.00
3	Base	0.94	0.00
	Ground	0.89	0.00
	1st	0.69	0.00
	2nd	0.40	0.00
4	Base	94.06	0.00
	Ground	89.11	0.00
	1st	69.31	0.00
	2nd	39.60	0.00

Table 3.6 Lateral load distribution result of the ETABS for the first wall arrangement in the model with diaphragms only

Wall arrangment-2

Wall No	Story	Shear, Vx (kN)	Shear, Vy (kN)
1	Base	95.00	-56.77
	Ground	90.00	-53.78
	1st	70.00	-41.83
	2nd	40.00	-23.90
2	Base	95.00	56.77
	Ground	90.00	53.78
	1st	70.00	41.83
	2nd	40.00	23.9
3	Base	95.00	-56.77
	Ground	90.00	-53.78
	1st	70.00	-41.83
	2nd	40.00	-23.90
4	Base	95.00	56.77
	Ground	90.00	53.78
	1st	70.00	41.83
	2nd	40.00	23.90

Table 3.7 Lateral load distribution result of the ETABS for the second wall arrangement in the model with diaphragms only

Wall arrangment-3

Wall No	Story	Shear, Vx (kN)	Shear, Vy (kN)
1	Base	609.73	0.00
	Ground	508.11	0.00
	1st	372.61	0.00
	2nd	203.24	0.00
2	Base	145.14	-110.01
	Ground	120.95	-61.68
	1st	88.69	-67.23
	2nd	48.38	-36.67
3	Base	145.14	110.01
	Ground	120.95	61.68
	1st	88.69	67.23
	2nd	48.38	36.67

Table 3.8 Lateral load distribution result of the ETABS for the third wall arrangement in the model with diaphragms only

Comparison of the results

Wall arrangment-1

		ETABS		Manual		% difference	
	Story	Shear, Vx (kN)	Shear, Vy (kN)	Shear, Vx (kN)	Shear, Vy (kN)	in Vx	in Vy
Wall-1	Base	1.29	0	0.94	0	37.23	0.00
	Ground	0.8	0	0.89	0	-10.11	0.00
	1st	0.73	0	0.69	0	5.80	0.00
	2nd	0.38	0	0.4	0	-5.00	0.00
Wall-2	Base	93.71	0	94.06	0	-0.37	0.00
	Ground	89.2	0	89.11	0	0.10	0.00
	1st	69.27	0	69.31	0	-0.06	0.00
	2nd	39.62	0	39.6	0	0.05	0.00
Wall-3	Base	1.29	0	0.94	0	37.23	0.00
	Ground	0.8	0	0.89	0	-10.11	0.00
	1st	0.73	0	0.69	0	5.80	0.00
	2nd	0.38	0	0.4	0	-5.00	0.00
Wall-4	Base	93.71	0	94.06	0	-0.37	0.00
	Ground	89.2	0	89.11	0	0.10	0.00
	1st	69.27	0	69.31	0	-0.06	0.00
	2nd	39.62	0	39.6	0	0.05	0.00

Table 3.9 Comparison of the Lateral load distribution result of the Approximate Elastic Analysis and the FEM for the first wall arrangement.

Wall arrangment-2

		ETABS		Manual		% difference	
	Story	Shear, Vx (kN)	Shear, Vy (kN)	Shear, Vx (kN)	Shear, Vy (kN)	in Vx	in Vy
Wall-1	Base	95	-43.68	95	-56.77	0.00	-23.06
	Ground	90	-53.48	90	-53.78	0.00	-0.56
	1st	70	-41.56	70	-41.83	0.00	-0.65

	2nd	40	-25.31	40	-23.9	0.00	5.90
Wall-2	Base	95	43.68	95	56.77	0.00	-23.06
	Ground	90	53.48	90	53.78	0.00	-0.56
	1st	70	41.56	70	41.83	0.00	-0.65
	2nd	40	25.31	40	23.9	0.00	5.90
Wall-3	Base	95	-43.68	95	-56.77	0.00	-23.06
	Ground	90	-53.48	90	-53.78	0.00	-0.56
	1st	70	-41.56	70	-41.83	0.00	-0.65
	2nd	40	-25.31	40	-23.9	0.00	5.90
Wall-4	Base	95	43.68	95	56.77	0.00	-23.06
	Ground	90	53.48	90	53.78	0.00	-0.56
	1st	70	41.56	70	41.83	0.00	-0.65
	2nd	40	25.31	40	23.9	0.00	5.90

Table 3.10 Comparison of the Lateral load distribution result of the Approximate Elastic Analysis and the FEM for the second wall arrangement.

Wall arrangment-3

		ETABS		Manual		% difference	
	Story	Shear, Vx (kN)	Shear, Vy (kN)	Shear, Vx (kN)	Shear, Vy (kN)	in Vx	in Vy
Wall-1	Base	505.53	0	609.73	0	-17.09	0.00
	Ground	433.98	0	508.11	0	-14.59	0.00
	1st	316.18	0	372.61	0	-15.14	0.00
	2nd	175.01	0	203.24	0	-13.89	0.00
Wall-2	Base	197.23	-96.101	145.14	-110.01	35.89	-12.64
	Ground	158.01	-121.67	120.95	-61.68	30.64	97.26
	1st	116.91	-86.74	88.69	-67.23	31.82	29.02
	2nd	62.5	-53.71	48.38	-36.67	29.19	46.47
Wall-3	Base	197.23	96.101	145.14	110.01	35.89	-12.64
	Ground	158.01	121.67	120.95	61.68	30.64	97.26

	1st	116.91	86.74	88.69	67.23	31.82	29.02
	2nd	62.5	53.71	48.38	36.67	29.19	46.47

Table 3.11 Comparison of the Lateral load distribution result of the Approximate Elastic Analysis and the FEM for the thirdt wall arrangement.

Observations

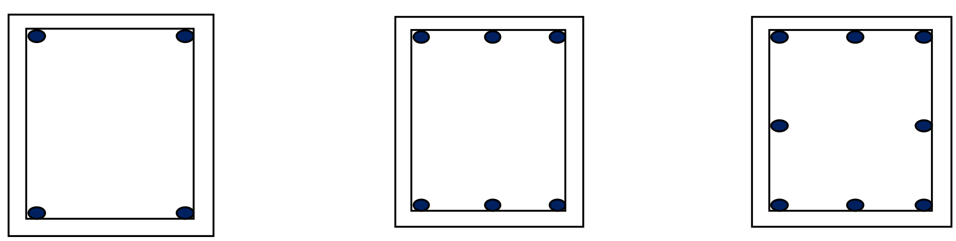
- In the first arrangment the maximum percentage difference for large shear values is 10.22 and for smaller shear values the maximum percentage difference is 37.15.
- In the second arrangment the maximum percentage difference is 23.06 and the minimum is 0.00.
- In the third arrangment the maximum percentage difference is 97.26 and the minimum is 0.00.

3.3. Verification of Design of Structural Walls and Columns Performed by ETABS

The second goal of the thesis work is to verify structural column and wall design outputs of the ETABS. To this end analysis and design of simple isolated cantilever columns and walls are carried out. The columns are rectangular while the walls are rectangular, L, and C in shapes. The walls and columns will be subjected to assumed factored lateral and gravity loads at their top ends. After analysis is made the walls will be designed for defined load combinations using BS8110 97 (EUROCODE 2- 1992 for wall design is not available in the ETABS 8.5 version) shear wall design code and the columns will be designed for the EUROCODE 2-1992.

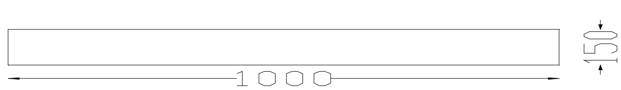
Shear wall reinforcement and section found from the ETABS will be checked by a wall design program for its capacity (R2). The column design results will be checked using the interaction charts in EBCS-2 Part 2.

The 400 X 400 (width X depth) square columns are having three different reinforcement arrangements as shown in Fig. 3.10.

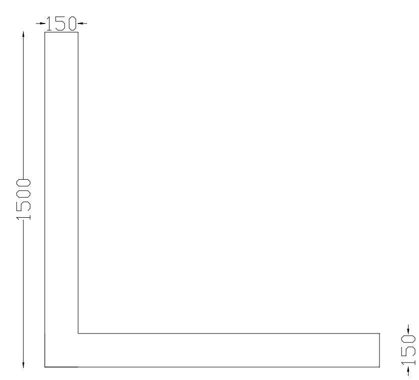


Column-1 Column-2 Column-3

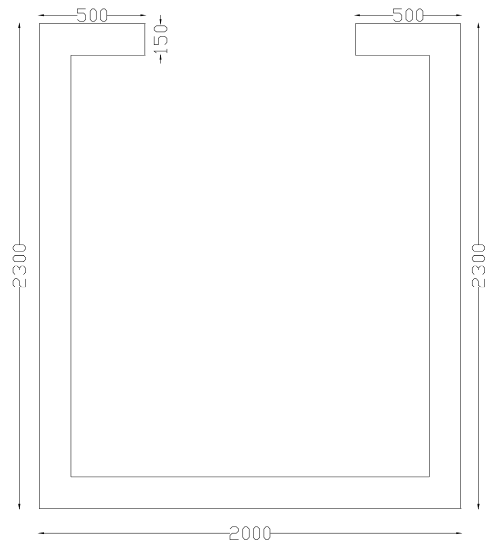
Fig. 3.10 Column sections



Rectangular section



L- Section



C- Section

Fig. 3.11 Wall sections

It is assumed that C30 concrete and S400 steel are used to construct the walls and C30 concrete and S300 steel are used for the columns.

In all the cases, height of the wall is taken to be 10m and that of the columns is taken to be 3.0m.

Loads and Load Combinations taken for the Walls

Three sets of arbitrarily assumed lateral and gravity force systems are taken. The design loads assumed for the different wall cross section types considered is summarized in the table below.

Load-1

X-Section Type	Gravity, F_z (kN)	Lateral (kN)		Torsion, M_z (kN-m)
		F_x	F_y	
Rectangular	200	15	2	0
L	300	80	80	20

C	800	300	450	30

Load-2

X-Section Type	Gravity, F_z (kN)	Lateral (kN)		Torsion, M_z (kN-m)
		F_x	F_y	
Rectangular	150	10	1	0
L	200	60	70	25
C	700	200	350	35

Load-3

X-Section Type	Gravity, F_z (kN)	Lateral (kN)		Torsion, M_z (kN-m)
		F_x	F_y	
Rectangular	180	20	0	0
L	250	70	60	15
C	850	250	400	40

Table 3.12 Load data assumed for the three wall types

Load combinations to be considered in the design of the walls are summarized in the table below.

Combination	Load Factor	
	Gravity	Lateral
COMB-1	1	1
COMB-2	0.75	1
COMB-3	1	0
COMB-4	1	-1

Table 3.13 Load combination assumed for the three wall types

Loads on the Columns

The design loads assumed to act on the columns are summarized in the table below.

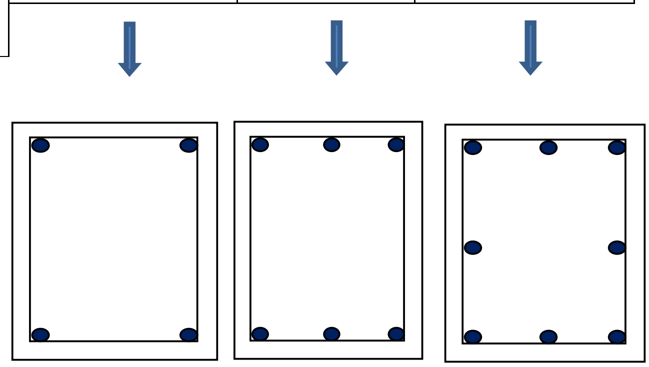
Column	Gravity, F_z (kN)	Moment (kN-m)	
		M_x	M_y
1	870.400	147.968	156.672
2	1740.800	304.640	104.448
3	435.200	95.744	139.264

Table 3.14 Load data assumed for the columns

Column Design

Table 3.15 Summary of Column Design according to EBCS-2 Part-2 and EUROCODE 2-1992 as used by the ETABS

Column Group	COL 1	COL 2	COL 3
N	870.400	1740.800	435.200
Col. Wt.	0.000	0.000	0.000
f_{cd}	13.600	13.600	13.600
M_x	147.968	304.640	95.744
M_y	156.672	104.448	139.264
b	0.400	0.400	0.400
h	0.400	0.400	0.400
N_t	870.400	1740.800	435.200
M_b	147.968	304.640	95.744
M_h	156.672	104.448	139.264
n	0.340	0.680	0.170
m_b	0.145	0.298	0.094
m_h	0.153	0.102	0.136
w	0.5	1.0	0.4
f_{yd}	260.87	260.87	260.87
A_s(mm)	4170.67	8341.33	3336.53
$A_s$final	4170.67	8341.33	3336.53
ETABS A_s	4751.00	8946.00	3767.00
(EUROCODE 2-1992)	-13.91%	-7.25%	-12.90%



Shear Wall Design

ETABS, unlike other structural analysis and design softwares, has the capability to design concrete shear walls of any shape. To this effect it is equipped with different shear wall design codes. These codes are the ACI318-99, UBC97, CSA-A23.3-94, BS8110 89 and BS8110 97.
In this thesis work, among these shear wall design codes, the BS8110 97 is chosen to design the shear walls. All the three walls above will be designed for shear and flexure at their bottom.
As discussed in section 3.1, in ETABS shear wall design there are three alternatives sections of reinforcement design for flexure: simplified design section, uniform reinforcing and general reinforcing sections.
Out of these preferences, in this paper work, the uniform reinforcing alternative is chosen. In the case of design for shear, only the simplified design alternative is available.

Rectangular wall

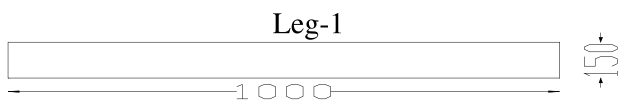


Fig 3.12(a)

The arbitrarily assumed loads given in the tables above are applied at the centroid of the simple rectangular cross section in Fig. 3.12(a) below and the wall is analyzed for gravity and lateral load cases and designed for the load combinations given.

Design for Load-1

Location Data

Pier Height	Axis Angle	Station Location	Xc Ordinate	Yc Ordinate	Zc Ordinate
10.000	0.000	Bottom	0.000	0.000	0.000
		Top	0.000	0.000	10.000

Flags and Factors

Design	RLLF	RLLF	Design

Active	Source	Factor	Type
Yes	Prog Calc	1.000	Seismic

Uniform Reinforcing Data

Edge Bar	Edge Spacing	End/Corner Bar	Clear Cover
12d	0.125	12d	2.500E-02

Pier Material and Geometry Data

Station Location	Pier Material	Pier Ag	Pier fcu	Pier fy	Pier LtWt Factor
Top	CONC	0.150	30000.000	400000.000	1.000
Bottom	CONC	0.150	30000.000	400000.000	1.000

Flexural Design Data

Station Location	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P	M2	M3
Top	0.0025	0.0136	COMB3	200.000	0.000	0.000
Bottom	0.0133	0.0136	COMB1	200.000	86.667	160.000

Pier Leg Location, Length and Thickness (Used for Shear Design)

Station Location	Xmin	Ymin	Xmax	Ymin	Length	Thickness
Top Leg 1	0.500	0.000	0.500	0.000	1.000	0.150
Bot Leg 1	0.500	0.000	0.500	0.000	1.000	0.150

Shear Design Data

Station Location	Rebar mm^2/m	Shear Combo	P	M	V	Capacity Vc	Capacity Vs
Top Leg 1	157.895	COMB2	150.000	0.000	15.000	207.131	60.000
Bot Leg 1	157.895	COMB1	200.000	150.000	15.000	157.292	60.000

Design for Load-2

Flexural Design Data

Station Location	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P	M2	M3
Top	0.0025	0.0136	COMB3	200.000	0.000	0.000
Bottom	0.0082	0.0136	COMB1	150.000	60.000	100.000

Shear Design Data

Station Location	Rebar mm^2/m	Shear Combo	P	M	V	Capacity Vc	Capacity Vs
Top Leg 1	157.895	COMB2	112.500	0.000	10.000	193.533	60.000
Bot Leg 1	157.895	COMB1	150.000	100.000	10.000	154.292	60.000

Design for Load-3

Flexural Design Data

Station Location	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P	M2	M3
Top	0.0025	0.0136	COMB3	180.000	0.000	0.000
Bottom	0.0074	0.0136	COMB2	135.000	0.000	200.000

Shear Design Data

Station Location	Rebar mm^2/m	Shear Combo	P	M	V	Capacity Vc	Capacity Vs
Top Leg 1	157.895	COMB2	135.000	0.000	20.000	201.802	60.000
Bot Leg 1	157.895	COMB1	180.000	200.000	20.000	156.092	60.000

Table 3.16 Summary of reinforcement area and spacing required at the bottom of the simple rectangular wall for flexure and shear.

Design for Load	**Thickness (mm)**	**Section Area ($10^6 mm^2$)**	**Flexure**			
			Ratio	**Area (mm2)**	**Dia.**	**Spacing (mm)**
1	150	0.15	0.0133	1995	12	113 On both faces.
2	150	0.15	0.0085	1230	10	128 On both faces.
3	150	0.15	0.0074	1110	12	204 On both faces.

Design for Load	**Leg**	**Shear**		
		Area (mm2/m)	**Dia.**	**Vertical Spacing (mm)**
1	1	157.985	8	636 On both faces.
2	1	157.985	8	636 On both faces.
3	1	157.985	8	636 On both faces.

L-Shaped wall

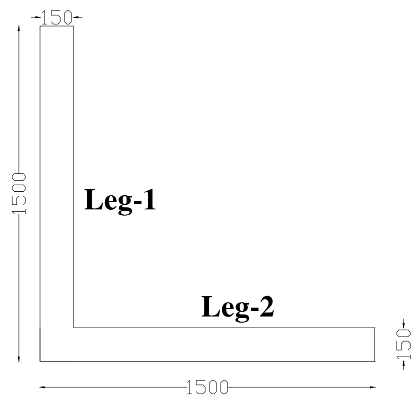


Fig 3.12(b)

The given loads are applied at the corner of the L-section wall shown.

The analysis and design results of the L-shaped wall are tabulated here under.

Design for Load-1

Location Data

Pier Height	Axis Angle	Station Location	Xc Ordinate	Yc Ordinate	Zc Ordinate
10.000	0.000	Bottom	0.375	0.375	0.000
		Top	0.375	0.375	10.000

Flags and Factors

Design Active	RLLF Source	RLLF Factor	Design Type
Yes	Prog Calc	1.000	Seismic

Uniform Reinforcing Data

Edge Bar	Edge Spacing	End/Corner Bar	Clear Cover
10d	0.125	10d	2.500E-02

Pier Material and Geometry Data

Station Location	Pier Material	Pier Ag	Pier fcu	Pier fy	Pier LtWt Factor
Top	CONC	0.450	30000.000	400000.000	1.000
Bottom	CONC	0.450	30000.000	400000.000	1.000

Flexural Design Data

Station Location	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P	M2	M3
Top	0.0025	0.0091	COMB4	300.000	-118.664	-113.733
Bottom	0.0077	0.0091	COMB4	300.000	-962.500	-922.500

Pier Leg Location, Length and Thickness (Used for Shear Design)

Station Location	Xmin	Ymin	Xmax	Ymin	Length	Thickness
Top Leg 1	0.000	0.000	0.000	1.500	1.500	0.150
Top Leg 2	0.000	0.000	1.500	0.000	1.500	0.150
Bot Leg 1	0.000	0.000	0.000	1.500	1.500	0.150
Bot Leg 2	0.000	0.000	1.500	0.000	1.500	0.150

Shear Design Data

Station Location	Rebar mm^2/m	Shear Combo	P	M	V	Capacity Vc	Capacity Vs
Top Leg 1	157.895	COMB4	150.000	111.156	79.927	255.089	90.000
Top Leg 2	157.895	COMB4	150.000	111.156	79.927	255.089	90.000
Bot Leg 1	157.895	COMB4	144.509	900.918	64.703	200.227	90.000
Bot Leg 2	157.895	COMB1	155.491	679.559	71.538	205.618	90.000

Design for Load-2

Flexural Design Data

Station Location	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P	M2	M3

Top	0.0025	0.0091	COMB4	200.000	-78.226	-75.741
Bottom	0.0061	0.0091	COMB4	200.000	-808.333	-681.667

Shear Design Data

Station Location	Rebar mm^2/m	Shear Combo	P	M	V	Capacity Vc	Capacity Vs
Top Leg 1	157.895	COMB4	99.377	73.658	69.654	235.387	90.000
Top Leg 2	157.895	COMB4	100.623	74.567	59.888	235.892	90.000
Bot Leg 1	157.895	COMB4	44.923	730.527	60.897	194.527	90.000
Bot Leg 2	157.895	COMB1	58.650	555.612	55.683	196.176	90.000

Design for Load-3

Flexural Design Data

Station Location	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P	M2	M3
Top	0.0025	0.0091	COMB4	250.000	-99.381	-94.734
Bottom	0.0060	0.0091	COMB4	250.000	-808.333	-681.667

Shear Design Data

Station Location	Rebar mm^2/m	Shear Combo	P	M	V	Capacity Vc	Capacity Vs
Top Leg 1	157.895	COMB4	125.623	93.083	60.082	245.799	90.000
Top Leg 2	157.895	COMB4	124.377	92.175	69.848	245.315	90.000
Bot Leg 1	157.895	COMB4	169.095	720.488	47.194	200.855	90.000
Bot Leg 2	157.895	COMB1	177.331	563.364	62.660	208.638	90.000

Table 3.17 Summary of reinforcement area and spacing required at the bottom of the L-shaped wall for flexure and shear.

Design for Load	Thickness (mm)	Section Area ($10^6 mm^2$)	Flexure			
			Ratio	Area (mm2)	Dia.	Spacing (mm)
1	150	0.45	0.0077	3465	12	196 On both faces.
2	150	0.45	0.0061	2745	10	172 On both faces.
3	150	0.45	0.006	2700	10	175 On both faces.

Design for Load	Leg	Shear		
		Area (mm2/m)	Dia.	Vertical Spacing (mm)
1	1	157.895	8	637 On both faces.
	2	157.895	8	637 On both faces.
2	1	157.895	8	637 On both faces.
	2	157.895	8	637 On both faces.
3	1	157.985	8	636 On both faces.

	2	157.985	8	636	On both faces.

C-Shaped wall

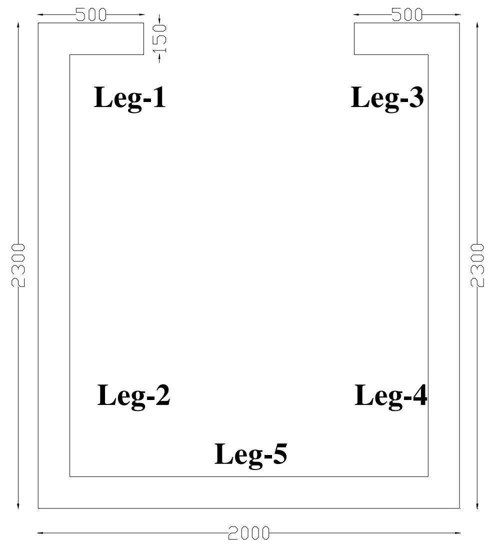


Fig 3.12(c)

The loads given are applied at the center of the web of the C-section wall.

The analysis and design results of the C-shaped wall are tabulated here under.

Design for Load-1

Location Data

Pier Height	Axis Angle	Station Location	Xc Ordinate	Yc Ordinate	Zc Ordinate
10.000	90.000	Bottom	1.000	0.999	0.000
		Top	1.000	0.999	10.000

Flags and Factors

Design Active	RLLF Source	RLLF Factor	Design Type
Yes	Prog Calc	1.000	Seismic

Uniform Reinforcing Data

Edge Bar	Edge Spacing	End/Corner Bar	Clear Cover
12d	0.110	12d	2.500E-02

Pier Material and Geometry Data

Station Location	Pier Material	Pier Ag	Pier fcu	Pier fy	Pier LtWt Factor
Top	CONC	1.140	30000.000	400000.000	1.000
Bottom	CONC	1.140	30000.000	400000.000	1.000

Flexural Design Data

Station Location	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P	M2	M3
Top	0.0025	0.0142	COMB4	800.000	0.000	-801.570
Bottom	0.0136	0.0142	COMB4	800.000	3080.702	-5316.339

Pier Leg Location, Length and Thickness (Used for Shear Design)

Station Location	Xmin	Ymin	Xmax	Ymin	Length	Thickness
Top Leg 1	1.500	2.300	2.000	2.300	0.500	0.150
Top Leg 2	2.000	0.000	2.000	2.300	2.300	0.150
Top Leg 3	0.000	2.300	0.500	2.300	0.500	0.150
Top Leg 4	0.000	0.000	0.000	2.300	2.300	0.150
Top Leg 5	0.000	0.000	2.000	0.000	2.000	0.150
Bot Leg 1	1.500	2.300	2.000	2.300	0.500	0.150
Bot Leg 2	2.000	0.000	2.000	2.300	2.300	0.150
Bot Leg 3	0.000	2.300	0.500	2.300	0.500	0.150
Bot Leg 4	0.000	0.000	0.000	2.300	2.300	0.150
Bot Leg 5	0.000	0.000	2.000	0.000	2.000	0.150

Shear Design Data

Station Location	Rebar mm^2/m	Shear Combo	P	M	V	Capacity Vc	Capacity Vs
Top Leg 1	157.895	COMB4	-4.412	1.043	5.295	69.999	30.000
Top Leg 2	157.895	COMB1	28.222	36.974	274.893	308.331	138.000
Top Leg 3	157.895	COMB4	-11.144	2.596	22.009	65.959	30.000

Top Leg 4	157.895	COMB4	21.722	0.535	189.245	305.211	138.000
Top Leg 5	157.895	COMB1	765.818	26.163	279.640	526.787	120.000
Bot Leg 1	157.895	COMB1	241.977	28.795	6.754	89.674	30.000
Bot Leg 2	258.953	COMB4	-1294.968	477.667	226.325	0.000	226.325
Bot Leg 3	180.957	COMB4	-843.180	33.892	34.382	0.000	34.382
Bot Leg 4	169.237	COMB2	-872.202	1124.571	147.913	0.000	147.913
Bot Leg 5	309.604	COMB1	-988.240	752.811	235.299	0.000	235.299

Design for Load-2

Flexural Design Data

Station Location	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P	M2	M3
Top	0.0025	0.0142	COMB4	700.000	0.000	-701.612
Bottom	0.0099	0.0142	COMB4	700.000	2070.614	-4214.296

Shear Design Data

Station Location	Rebar mm^2/m	Shear Combo	P	M	V	Capacity Vc	Capacity Vs
Top Leg 1	157.895	COMB4	-3.741	0.883	4.784	70.401	30.000
Top Leg 2	157.895	COMB1	21.000	28.461	212.869	304.862	138.000
Top Leg 3	157.895	COMB4	-8.648	2.016	16.823	67.457	30.000
Top Leg 4	157.895	COMB4	16.023	1.356	156.205	302.449	138.000
Top Leg 5	157.895	COMB1	671.965	17.753	187.376	502.315	120.000
Bot Leg 1	157.895	COMB1	227.272	18.438	7.225	99.361	30.000
Bot Leg 2	191.110	COMB4	-851.715	419.542	167.030	0.000	167.030
Bot Leg 3	157.895	COMB4	-645.241	23.846	26.109	0.000	30.000
Bot Leg 4	157.895	COMB4	850.859	1087.315	33.025	438.196	138.000
Bot Leg 5	208.860	COMB1	-722.082	487.049	158.733	0.000	158.733

Design for Load-3

Flexural Design Data

Station Location	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P	M2	M3
Top	0.0025	0.0142	COMB4	850.000	0.000	-852.117
Bottom	0.0119	0.0142	COMB4	850.000	2585.746	-4867.360

Shear Design Data

Station Location	Rebar mm^2/m	Shear Combo	P	M	V	Capacity Vc	Capacity Vs
Top Leg 1	157.895	COMB4	-4.158	0.983	5.054	70.151	30.000
Top Leg 2	157.895	COMB1	25.703	33.768	246.347	307.126	138.000
Top Leg 3	157.895	COMB4	-10.187	2.374	19.878	66.533	30.000
Top Leg 4	157.895	COMB4	20.016	0.309	175.451	304.387	138.000
Top Leg 5	157.895	COMB1	817.029	22.114	233.983	539.672	120.000
Bot Leg 1	157.895	COMB1	238.636	23.301	7.280	95.013	30.000
Bot Leg 2	221.718	COMB4	-1050.695	456.693	193.782	0.000	193.782
Bot Leg 3	162.187	COMB4	-758.522	29.481	30.816	0.000	30.816
Bot Leg 4	157.895	COMB4	1074.943	1282.463	148.759	466.612	138.000
Bot Leg 5	260.460	COMB1	-795.312	612.518	197.950	0.000	197.950

Table 3.18 Summary of reinforcement area and spacing required at the bottom of the C-shaped wall for flexure and shear.

Design for Load	Thickness (mm)	Section Area (10^6mm^2)	Flexure				
			Ratio	Area (mm2)	Dia.	Spacing (mm)	
1	150	1.14	0.0136	15504	12	111	On both faces.
2	150	1.14	0.0099	11286	10	106	On both faces.
3	150	1.14	0.0119	13566	12	127	On both faces.

Design for Load	Leg	Shear			
		Area (mm2/m)	Dia.	Vertical Spacing (mm)	
1	1	157.895	8	637	On both faces.
	2	258.953	8	388	On both faces.
	3	180.957	8	556	On both faces.
	4	169.237	8	594	On both faces.
	5	309.604	8	325	On both faces.
2	1	157.895	8	637	On both faces.
	2	191.11	8	526	On both faces.

	3	157.895	8	637	On both faces.
	4	157.895	8	637	On both faces.
	5	208.86	8	481	On both faces.
3	1	157.895	8	637	On both faces.
	2	221.718	8	453	On both faces.
	3	162.187	8	620	On both faces.
	4	157.895	8	637	On both faces.
	5	260.46	8	386	On both faces.

Capacity to Demand Ratio (C/D)

The wall sections, designed in the previous sections, using the ETABS according to the BS8110 97 will be checked for their capacity to demand ratio by the program specified in reference 2(R2). The capacity to demand ratio is summarized in the table below.

Table 3.19 Capacity to demand ratio of the wall sections designed using ETABS

Shape of wall	Load	Design Actions			Reinforcement	C/D
		P (kN)	M_x (kN-m)	M_y (kN-m)		
Rectangular	1	-200.00	86.667	160.000	Ø 12 c/c 113	0.51
	2	-150.00	60.000	100.000	Ø 10 c/c 128	0.51
	3	-135.00	0.000	200.000	Ø 12 c/c 204	1.11
L-shaped	1	-300.00	-922.500	-962.500	Ø 12 c/c 196	0.61
	2	-200.00	-808.333	-681.667	Ø 10 c/c 172	0.55
	3	-250.00	-681.667	-808.333	Ø 10 c/c 175	0.56
C-shaped	1	-800.00	5316.339	3080.702	Ø 12 c/c 111	1.02
	2	-700.00	4214.296	2070.614	Ø 10 c/c 106	0.99
	3	-850.00	4867.360	2585.746	Ø 12 c/c 128	1.05

4. Conclusions and Recommendations

After careful inspection of the comparison results of ETABS version 8.5 outputs studied in section 3.2, the following conclusions and recommendations are made.

a. Modeling of Interacting walls and lateral load distribution results of ETABS based on the assumption that the walls are connected at each floor by rigid diaphragms.
   1. For the simple rectangular walls arranged in such a way that the center of story shear and the center of rigidity are exactly the same, the story shear distribution results of the individual walls according to ETABS differ from results of the approximate elastic analysis by 10.11%.
   2. For the L-shaped walls arranged in such a way that the center of story shear and center of rigidity are exactly the same, the story shear distribution results of the individual walls according to ETABS in the direction of the applied lateral force are exactly the same as results of the approximate elastic analysis.
   3. For the L-shaped walls configured in such a way that the center of story shear and center of rigidity are exactly the same, the story shear distribution results of the individual walls according to ETABS in the direction perpendicular to the direction of the applied lateral force differ from results of the approximate elastic analysis by 23.06%.
   4. For the mixed L and C-shaped wall system considered, the story shear distribution results of the individual walls according to ETABS in the direction of the applied lateral force differ from results of the approximate elastic analysis by 35.89%.
   5. For the mixed L and C-shaped wall system considered, the story shear distribution results of the individual walls according to ETABS in the direction perpendicular to the applied lateral force differ from results of the approximate elastic analysis by 97.26%.

b. Column and Shear Wall Design Results of the ETABS based only on limited number of investigations

   1. For the rectangular column sections considered the amount of flexural reinforcement obtained from the ETABS according to EUROCODE 2-1992 is higher than those from EBCS- 2 Part-2 column design charts. Average percentage difference is about 12%.
   2. For all the three simple rectangular section walls studied, the flexural reinforcement amount obtained from ETABS according to BS8110 97 wall design code is not sufficient to resist the design actions which it was designed for in the ETABS. The average deviation below the required amount is 29%.
   3. For all the three L-shaped section walls studied, the flexural reinforcement amount obtained from ETABS according to BS8110 97 wall design code is not sufficient to resist the design actions which it was designed for in the ETABS. The average deviation below the required amount is 42.5%.
   4. In case of all the C-shaped walls considered, the flexural reinforcement area obtained from the ETABS is sufficient to withstand the design actions. Mean deviation above the exact value is only 2%.

Therefore, based on the comparison results of the design of a limited number of columns and walls, it may be concluded that the design of columns and structural walls performed by ETABS may lie on the safe side or the unsafe side.

5. References

R1 Fredrich & Lochner, Ingenieurbüro Fredrich & Lochnor, Stuttgart, Dresden, 1995.

R2 Busjaeger, D., Quast, U., “Programmgesteuerte Berechnung beliebiger Massivbauquerschnitte unter Zweiachsiger Biegung mit Längskraft”, Deutscher Ausschuss für Stahlbeton, Heft 415, Beuth Verlag, Berlin 1990.

R3 T. Paulay & M.J.N. Priestley, “Seismic Design of Reinforced Concrete & Masonry Buildings”, John Wiley & Sons, Inc., New York, 1992.

R4 Zerayhonnes, Girma, Ethiopian Building Code Standard, EBCS-2: Part 2, “Design Aid, Slabs, Beams and Columns”, Addis Ababa, 1998.

R5 Edward L. Wilson, “Three-Dimensional Static and Dynamic Analysis of Structures”, Computers and Structures Inc., California, 2002.

R6 Computers and Structures Inc., ETABS, Integrated Building Design Software, Berkeley, California, 2003.

R7 K-J. Schneider, Bautabellen für Ingenieure mit Berechnungshinweisen und Beispielen,17. Auflage, Werner Verlag, 2006.

R8 W.F. Chen (Ed), Structural Engineering Hand Book, CRC Press, LLC, 1999.