ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

FACULTY OF TECHNOLOGY

DEPARTMENT OF CIVIL ENGINEERING

CORRELATION BETWEEN ACTUAL REINFORCED CONCRETE WALL BEHAVIOR AND ITS CENTERLINE MODEL

A thesis submitted to the school of Graduate Studies in Partial fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering (Structures)

By

Medhanye Biedebrhan

Advisor: **Dr. Shifferaw Taye**

June 2003

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Approved by Board of Examiners

Advisor	Signature	Date
External Examiner	Signature	Date
Internal Examiner	Signature	Date
Chairman	Signature	Date

DECLARATION

I, the undersigned, declare that this thesis is my work and all sources of materials used for the thesis have been duly acknowledged.

Name	Medhanye Biedebrhan
Signature	____________________
Place	Addis Ababa University
	Faculty of Technology
Date of submission	July 09, 2003

This work is dedicated to

My sister Tsega Biedebrhan

And

My advisor Dr. Shifferaw Taye

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NOTATION

A_w	wall section area
γ	angle of shear deformation
θ	foundation rotation angle
τ_{max}	maximum shear stress
δ_S	deformation due to shear
δ_B	deformation due to bending
δ_R	deformation due to foundation rotation
δ_D	deformation due to foundation displacement
Q, Q_F, Q_w	total shear force, shear force in frame, and shear force in wall, respectively.
P, P_F, P_w	total uniform lateral load , lateral load in frame, lateral load in wall, respectively.
κ	shear stress factor
G	shear modulus
h	storey height
K	numerical factor
E	modulus of elasticity
k_w	wall stiffness
D_w	total shear distribution coefficient of wall
D_B	shear distribution coefficient of wall from bending
D_S	shear distribution coefficient of wall from shear
D_R	shear distribution coefficient of wall from foundation rotation
D_D	shear distribution coefficient of wall from foundation displacement
β	ratio of reduction of rigidity due to inelastic deformation
β_o	inverse of β
R	rotation angle
G_F	total stiffness of frame
H	total height of system
a	stiffness ratio
l_a	length from centerline of wall to wall end
l	length of adjacent beam
k_B	beam stiffness
k_{Be}	modified beam stiffness

ABSTRACT

Key words: concrete shear walls, dual systems, ETABS, lateral loads, shear deformation.

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 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 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, a structural modeling approach that leads to incorrect results.

Therefore, it is the primary purpose of this research work to correlate the results obtained from the above simple but incorrect method of analysis and those from true wall behaviors with an ultimate goal of providing the analysts and the designers with correction factors in order to implement the centerline method of analysis for the analysis of walls. It is believed that structural engineers working in the analysis and design of high- rise buildings will be benefited from the results.

1. INTRODUCTION

Structural analysis is the process of determining the response of a structure due to specified loading; and structural design is the process of proportioning structures under loads in order to satisfy essential requirements of function, safety, economy, and aesthetics. Simply defined, structural analysis is a mathematical process by which the engineer verifies the adequacy of structure with respect to its strength and stiffness. It is not always possible or necessary to obtain rigorous mathematical solutions for building engineering problems. In fact, rigorous analytical solutions can be obtained only for certain simplified cases. High-rise structural problems, like most other practical engineering problems, involve complex material property, loading, and boundary conditions. The engineers introduce assumptions and idealizations deemed necessary to make the problem mathematically manageable, but still capable of providing sufficiently accurate solutions and satisfactory results from the point of view of safety and economy. They establish a link between the real physical system and the mathematically feasible solution by providing an analytical model which is the symbolic designation for the substitute idealized system, including all the assumptions imposed on physical problems. Modeling techniques, therefore, can be defined as a way to reduce, synthesize, and properly represent the structural system.

Preliminary Hand Calculation

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 with in the limits of acceptability. Starting the process with the best possible selection of members results in a rapid convergence of the iterative process to the desired solution.

Second, because of the ever-increasing cost of labor and building materials, 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. Of the myriad structural systems which represent themselves as possibilities, only two or three schemes may be worthy of further refinement requiring full-blown computer solutions. Approximate methods are all that may be required to logically arrive at cost figures and to sort out the few final contenders from among the innumerable possibilities. It is very time consuming, costly, and indeed unnecessary to undertake a complete sophisticated analysis for all the possible schemes. Preliminary designs are therefore very useful in weeding out the weak solutions.

Sophisticated computer analysis are indispensable in reducing the number of inaccuracies caused by hand analysis techniques and are being used routinely in everyday engineering practice. Although such computer analyses may intimidate the structural engineer by virtue of their unbelievable amount of documentation and out put, the prudent engineer will always verify the reasonableness of the computer analysis by using approximate hand-calculated values for forces, moments, and deflections. Approximate analysis is, therefore, a powerful tool in providing the engineer

1. *A basis for preliminary sizing of members,*
2. *An orderly method for evaluating several scheme to select the most likely one for further study, and*
3. *Methods for obtaining approximate values of forces, moments, and deflections to check on the validity of computer solutions.*

2. LATERAL LOAD RESISTING SYSTEMS

The structural systems mainly used as earthquake (or generally, lateral load) resistant structures are:

- Frame systems
- Wall systems
- Dual systems, i.e. shear walls acting with frames
- Tubes

2.1. Rigid frame systems

Rigid frame skeletons generally consist of a rectangular grid of horizontal beams and vertical columns connected together in the same plane by means of rigid joints. Because of its continuity, the rigid frame responds to lateral loads primarily through flexure of the beams and columns. This continuous character of the rigid frame is dependent on the rotational resistance of the member connections not to permit any slippage.

In other words, a rigid frame derives its resistance to lateral loads from the rigidity of its joints. It has no diagonal members, and once the joints are assumed to be rigid, the stiffness of the frame becomes a function of the stiffness of the beams and columns constituting the frame.

The strength and stiffness of the frame is proportional to the beam and column size and inversely proportional to story height and column spacing. That is, the load capacity of the frame relies very much on the strength of the individual beams and columns; and its capacity decreases as story height and column spacing become larger.

Under lateral loads, a rigid frame deforms in a “shear mode”. This lateral deflection of rigid frames is caused by deflection due to bending of beams and columns resulting in shear lag.

In such systems the relative displacements are proportional to the shear forces; this is the reason why these systems are called ‘shear systems’ in many literatures. The deformation of these systems is such that they present a concave form on the side of the loading as shown in Fig. 2.1., below.

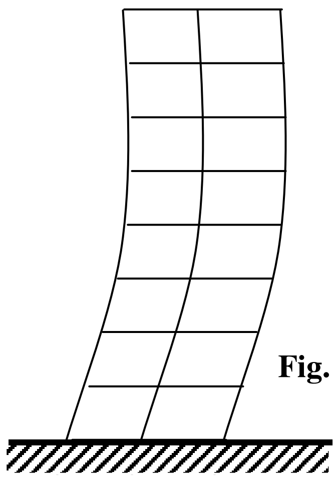


Fig. 2.1 Frame shear mode deformation

2.2. Shear wall systems

Outwardly, a shear wall building is in no way different from an ordinary framed building. However, it differs significantly when it comes to transference of lateral loads. Shear walls are vertical stiffening elements designed to resist lateral forces exerted on a building by wind or earthquakes. Floors acting as horizontal diaphragms transmit lateral loads to the shear walls.

Great structural advantage may be taken from reinforced concrete shear walls in aseismic construction, provided they are properly designed and detailed for strength and ductility. Favorably positioned shear walls can be very efficient in resisting horizontal wind and earthquake loads. The considerable stiffness of shear walls not only reduces the deflection demands on other parts of the structure, such as beam-column joints, but may also help to insure development of all available plastic hinge positions throughout the structure prior to failure. It is recognized that the deflection of a shear wall, which is primarily due to bending, may be significantly affected by rotation of a foundation that is on compressible soil. This effect should be included in the deflection calculations, since it would affect load distribution. This is particularly true for distribution between shear walls and frames, since frames deflect mainly in shear and would not be affected to the same extent by foundation rotation.

Shear walls behave as cantilevers under the action of lateral forces. The shear distribution is proportional to the moments of inertia of the cross-sections of the walls. The relative displacements of the floors result from bending deformation of the walls and therefore they present a convex form on the side of the loading, Fig. 2.2.

In many cases walls carry a major part of the seismic base shear in the case of earthquakes, while the existing frames are designed primarily to act as a second line of defense against earthquakes, after extensive cracking and/or failure of walls.

The main advantage offered by earthquake resisting reinforced concrete walls is the significant increase in the stiffness of the building, which leads to a reduction of second-order effects and a subsequent increase of safety against collapse, as well as a reduced degree of damage to non structural elements, whose cost is often higher than that of the structural elements. Furthermore, the significant reduction of psychological effects on the inhabitants of high rise buildings subjected to earthquake induced displacements should be pointed out.

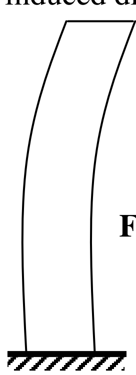


Fig. 2.2. Bending effect of a cantilever shear wall

2.3. Rigid frame-shear wall (Dual) systems

Using only shear walls to respond to lateral loads is impractical for very high buildings. The lateral rigidity is then greatly improved by using not only the shear wall system but also the rigid frame to resist lateral forces.

The structural behavior of shear walls and frames being distinctly different, interaction between them produces a mean deflection pattern and the total deflection of the interacting shear wall and rigid frame systems is obtained by superimposing the individual modes of deformation, i.e.

- rigid frame shear mode deformation, and
- shear wall bending mode deformation

The diagram shown below, Fig. 2.3., shows the shear wall-frame interaction and the distribution of total lateral load to the individual shear walls and frames as given by this simple interaction diagram is valid only if one of the following two conditions is satisfied [Muto, 1974].

1. Each shear wall and frame must have constant stiffness properties throughout the height of the building.
2. If stiffness properties vary over the height, the relative stiffness of each wall and frame must remain unchanged throughout the height of the building.

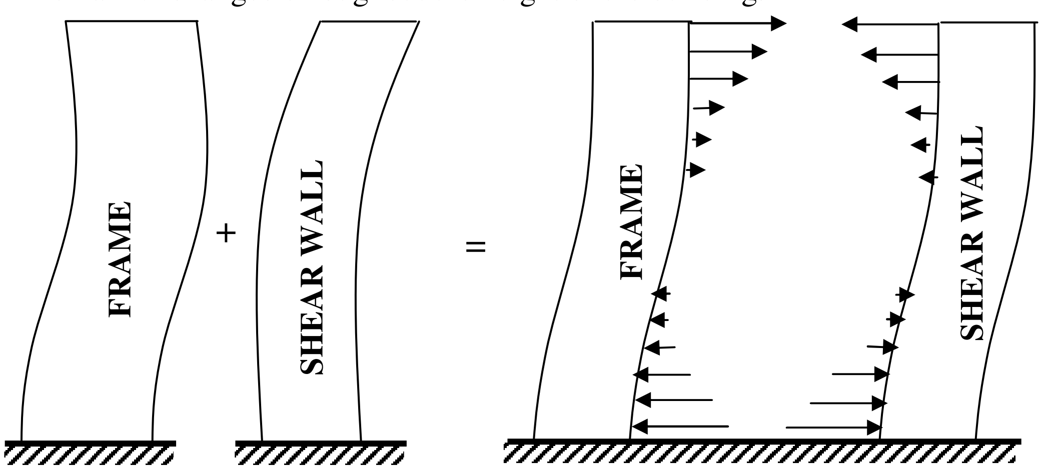


Fig. 2.3. Shear wall – frame interaction

Compatibility of the horizontal deflection introduces interaction between the two systems which tends to impose a reverse curvature in the deflection pattern of the system. Generally speaking, shear walls and frames have different modes of deformation under lateral loads; and if both exist in a building, each tries to obstruct the other from developing its natural mode; and a compromise behavior occurs with redistribution of loads between the frame and the shear wall.

It is not always easy to differentiate between the two modes of deformation. For example, under lateral loads a frame consisting of closely spaced columns and deep beams tend to behave more like a shear wall responding predominantly in a bending mode. Similarly, a shear wall weakened by a row or rows of openings may tend to act more like a frame by deflecting in a shear mode. The combined structural action, therefore, depends on the relative rigidities of different elements used in the makeup of the lateral-load-resisting system. The coupling of the two systems into a dual system under lateral loading, because of the completely different deformation shape of the individual components, results in interaction forces that alter the moment and shear diagrams of both the frame and the wall. The characteristic of this combination is that in the lower floors the wall retains the frame while in the upper floors the frame inhibits the large displacements of the wall. As a result the frame exhibits a small variation in storey shear between the first and the last floors [Muto, 1974].

2. DEFORMATIONS OF SINGLE-STORIED SHEAR WALLS

Before going to the discussion of multi-storied shear walls, discussion on single-storied walls will be made first. The single-storied sandwiched shear wall shown in Fig. 2.1., will be taken as an example. The deformations can be classified in this case as δ_S from shear deformation, δ_B from bending, δ_R from rotation, due to insufficiency of fixing of the foundation and δ_D from displacement of the whole wall due to displacement of the foundation where the ground is especially soft.

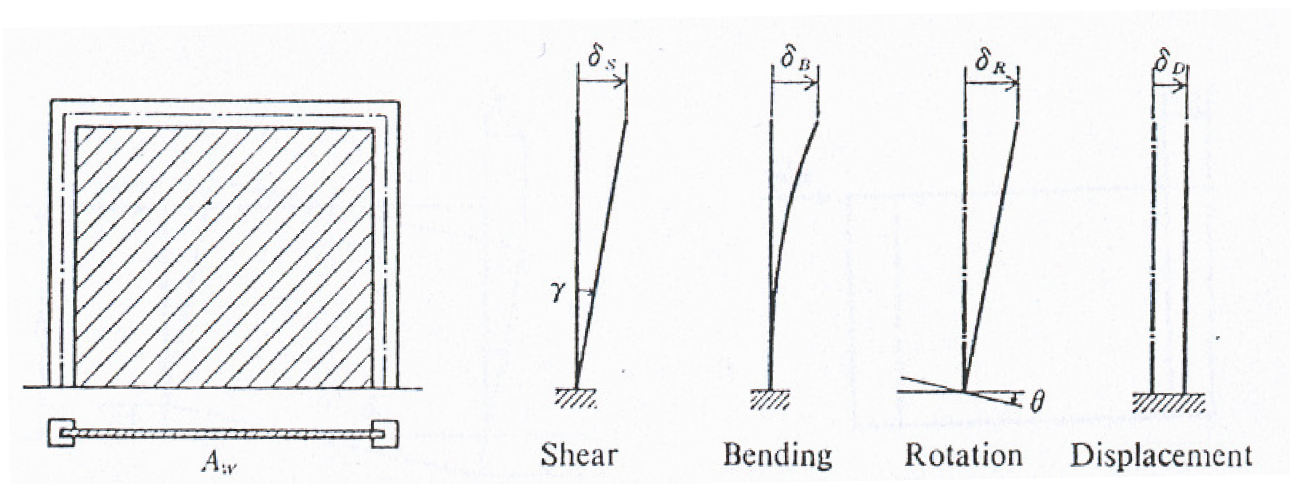


Fig. 2.4. Deformations of single-storied shear wall.

Shear Deformation: The shear stress, τ, of a wall will be distributed on the effective area, A_w, of the wall, and the shape of the stress distribution will be in accordance with the characteristics of I-shaped members (*see Fig. 2.2.*).

As shown in Fig. 2.2., when the shapes of the end columns or flanges are large, the stresses will be distributed equally within the cross section of the web, A_w, and τ_{max}, the stress at the center line, will be the average shear stress as shown in the following equation

$$\tau_{max}=Q/A_w$$

If the sectional areas of the end columns are extremely small, it will become close to the parabolic distribution of rectangular webs and is greatest at the center line being 1.5 times the average.

$$\tau_{max}=1.5Q/A_w$$

Ordinarily, the actual cases will be in between and this is expressed as κ in the following equation

$$\tau_{max}= \kappa Q/A_w \qquad (2\text{-}1)$$

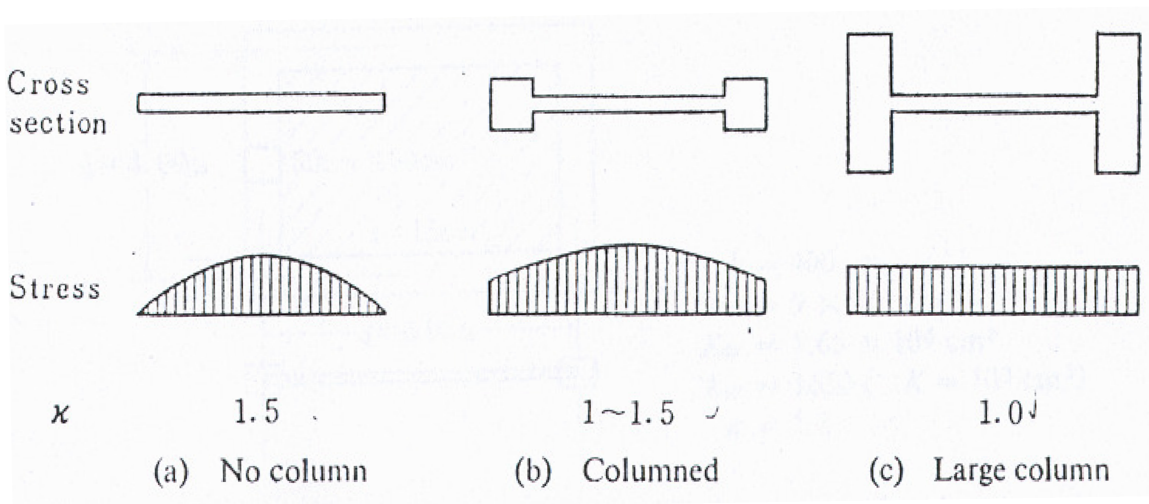


Fig. 2.5. Distribution of shear stress.

With this shear stress the wall will show a shear deformation angle and deform in a complicated manner, but δ_S, the amount of deflection due to shear, can be obtained by assuming simply that the wall is inclined at the maximum angle of deformation, γ, at the center line as shown on the right side of Fig. 2.1., where the lines represent the centerline of the shear wall.

$$\gamma = \tau_{max}/G = \kappa Q/GA_w$$

Therefore, shear deflection is

$$\delta_s = \gamma h = \kappa Qh/GA_w \qquad (2\text{-}2)$$

expressing this with a unit [$12EK/h^2$] [Muto, 1974].

$$\delta_S = \kappa \frac{12EK}{GA_W h} .Q. \frac{h^2}{12EK} = \kappa \frac{27.6K}{A_W h} .Q. \frac{h^2}{12EK} \quad \text{………………………………(2-3a)}$$

To obtain D-value (shear distribution coefficient) taking only shear deformation into consideration,

$$D_S = \frac{1}{\kappa \frac{27.6K}{A_w h}} = \frac{A_w h}{\kappa * 27.6K} \quad \text{……………………………….................(2-4)}$$

and therefore

$$\delta_S = \frac{Q}{D_S} . \frac{h^2}{12EK} \quad \text{……………………………………(2-3b)}$$

Bending Deformation: A one-storied shear wall is too wide to be considered as being a cantilevered column, but as multi-storied shear walls are also discussed later on, the bending deformation is calculated from the ordinary equation for bending, taking the case for a cantilever column with concentrated lateral load at the free end, Q,

$$\delta_B = \frac{h^2}{3EK_W} Q \quad or \quad \delta_B = \frac{4}{K_W} . \frac{h^2}{12EK} Q \quad \text{……………………….................(2-5a)}$$

Obtaining the D-value for bending only

$$D_B = K_w/4 \text{………………………………………………………(2-6)}$$

Therefore

$$\delta_B = \frac{Q}{D_B} . \frac{h^2}{12EK} \quad \text{……………………………......................(2-5b)}$$

Taking both shear and bending deformation into consideration, the equation below for the total deflection of the wall is obtained.

$$\delta_w = \delta_S + \delta_B = \left(\frac{1}{D_S} + \frac{1}{D_B} \right) Q \left[\frac{h^2}{12EK} \right] \text{……………………………(2-7)}$$

Then the D-value will be

$$\frac{1}{D_W} = \frac{1}{D_S} + \frac{1}{D_B} \quad \text{..(2-8)}$$

The inverse of the D-value means the deformation.

Rotation and Displacement of Foundation: When there is rotation of the foundation and there are upward and downward displacements of s at both ends, Fig. 2.3.,

$$\delta_R = \theta h \text{..(2-9a)}$$

where

$$\theta = 2s/l \text{..(2-10)}$$

Expressed with the common unit and to obtain the D-value from rotation, the following equation is used.

$$\delta_R = \frac{12EK\theta}{Qh} Q \left[\frac{h^2}{12EK} \right] \qquad (2-9b)$$

$$\textit{and} \qquad D_R = \frac{Qh}{12EK\theta} \qquad (2-11)$$

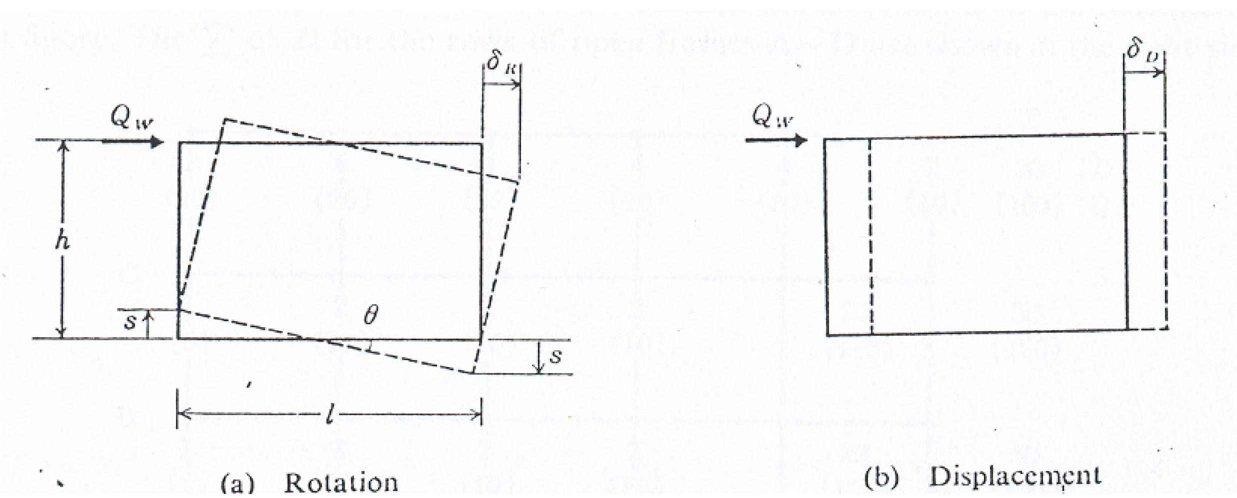


Fig. 2.6. Rotation and displacement of foundation

When there is a horizontal displacement, δ_D, of the foundation, expressed with the common unit this is

$$\delta_D = \frac{12EK\delta_D}{Qh^2} Q\left[\frac{h^2}{12EK}\right] \qquad (2-12)$$

$$and \qquad D_D = \frac{Qh^2}{12EK\delta_D} \qquad (2-13)$$

Taking all deformation elements into consideration the final formulae will be [Muto, 1974].

$$\frac{1}{D_W} = \frac{1}{D_S} + \frac{1}{D_B} + \frac{1}{D_R} + \frac{1}{D_D} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2\text{-}14)$$

$$\delta_W = \frac{Q}{D_W}\left[\frac{h^2}{12EK}\right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2\text{-}15)$$

3. DEFORMATIONS OF MULTI-STORIED SHEAR WALLS

The shear walls of actual high-rise buildings are often connected with frames; but unlike a single-storied building, the influence (boundary effect) of the connection between shear wall and frame is great and its behavior is considerably different from that of a free-standing shear wall. Handling of the problem, which is described later, is relatively complicated. However, deflection properties can be readily inferred from the deflections of free-standing shear walls and the method of calculation for free-standing shear walls is the basis of the method of calculation for shear walls with boundary effect. For this reason, the deflection characteristics of a free-standing shear wall will first be studied.

Similarly to the case of a single-storied shear wall, the deflection of a multi-storied shear wall may be separated into

-Bending deformation

-Shear deformation

-Deformation due to foundation rotation and base movement.

Of the three, deformations from bending and foundation rotation become governing factors when it comes to multiple stories. The deflection characteristics differ considerably from the deflection characteristics of a frame, or in other words, shear-type deformation. The relative displacements of upper stories of the shear wall are much larger than those of the lower stories, whereas in a frame, the relative displacement is of similar magnitude for upper and lower stories.

Therefore, in an actual building, when lateral force is carried by the shear wall and frame, the proportions taken up by upper and lower stories should be different. According to Kiyoshi Muto, a fundamental study of the nature of shear force by wall and frame is presented in the paper, *Theoretical Study of Lateral Force Distribution of Multi-Storied Shear Walls* (Transactions of the Architectural Institute of Japan, No. 46, 1953) by Kiyoshi Muto and others. In the conclusion of the paper, the following points are made clear:

(1) The practical hypothesis that there is no relation between the shear distribution coefficient of a wall and the location of a story applied heretofore must not be used.

(2) The shear distribution coefficient is influenced by the characteristics of bending rigidity of the wall, and in general, the shear force carried at upper stories is greatly reduced. Particularly, when the wall is tall and slender and the bending rigidity is small, the capacity to carry shear force not only is lost but a reverse effect is caused in some cases and the shear forces of columns are increased.

(3) There are cases when it can be considered more effective to limit narrow shear walls to the middle stories and refrain from extending them to upper levels.

(4) When the degree of fixture of the base is not sufficient the shear distribution coefficient is lowered.

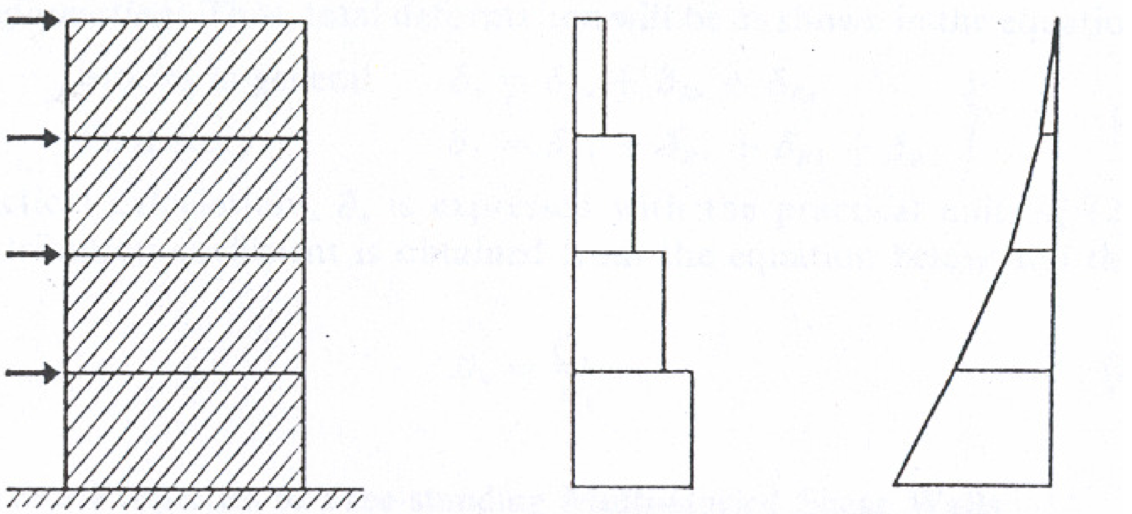
Until around 1950, in determining the shear distribution coefficient of a shear wall, it was the concept to define it as the ratio by which shear due to seismic force is distributed, and without any relation to the actual rigidity of the wall, values such as 10 times or 20 times of columns were assumed. Kyoshi Muto with others pointed out the dangers of such assumptions and stated the desirability of determining shear distribution coefficient by obtaining basic figures through calculations based on rigidity.

As the first step of analysis, the method of calculation for a free-standing shear wall will now be discussed.

3.1. Deformation of Free-standing Multi-storied Shear Walls

3.1.1. Deformation and Rigidity of Free-Standing Shear Walls

When a free-standing shear wall carries seismic forces, the stresses in the wall become as shown in Fig. 3.1. The Q-diagram shows all stories with rectangular distribution and the M-diagram with trapezoidal (triangular at top story) distribution. In the calculations, the shear and bending deformations of the wall are first considered, after which rotation at the base and movement of the base are figured in.



External force　　　Shear force diagram　　　Bending moment diagram

Fig. 3.1. shear wall forces

Shear Deformation: A case of n stories will be described. As in the case for a single story, a uniform shear stress is imposed and the angle of shear deformation, γ_n, and the relative displacement, δ_{sn}, are (*see Fig. 3.2.*).

$$\gamma_n = \frac{\tau_{max}}{G} = \kappa \frac{Q_n}{GA_{wn}}$$

$$\delta_{sn} = \gamma_n h_n = \kappa \frac{Q_n h_n}{GA_{wn}} \quad \text{……………………………………….(3-1a)}$$

Since shear walls also show inelastic deformation, the above expression for δ_{sn}, after inserting ratio of reduction of rigidity due to inelastic deformation, β, will be

$$\delta_{sn} = \kappa \frac{Q_n h_n}{\beta GA_{wn}} \quad \text{…………………………………………..(3-1b)}$$

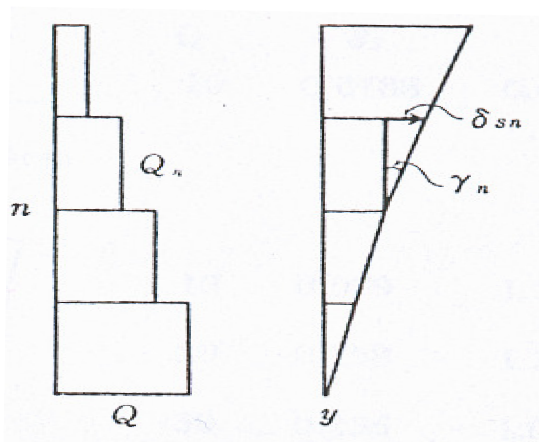


Fig. 3.2. Shear deformation

Bending Deformation: Bending deformation is calculated from the M/EI diagram using beam theory. The relative displacement of story n is divided into deflection, $\delta_{\theta n}$ due to angle of rotation, θ_n at the bottom of story n, and δ_{Mn} due to bending deformation of that story (*see Fig. 3.3.*).

$$\delta_{\theta n} = \theta_n h_n$$

where, θ_n: area of M/EI diagram of stories below ($\sum F_n$ of diagram)

$$\delta_{Mn} = F'_n \, x_n$$

where, F'_n: area of M/EI diagram of story n

x_n: distance from top end to center of gravity of F'_n

From the notations in the Fig. 3.3., the equation obtained is as follows.

$$\delta_{Bn} = \left(\sum_{i=1}^{n-1} \frac{M_i h_i}{EI_i} \right) h_n + \frac{M_n h_n}{EI_n} x_n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\text{(3-2)}$$

Bending deformation is small at lower stories, but at upper stories the influence of the bending deformation of the lower stories (first term on right-hand side in Eq. (3-2)) becomes large. For this reason, rigidity is markedly reduced at upper stories. This must be noted as a characteristic of shear walls.

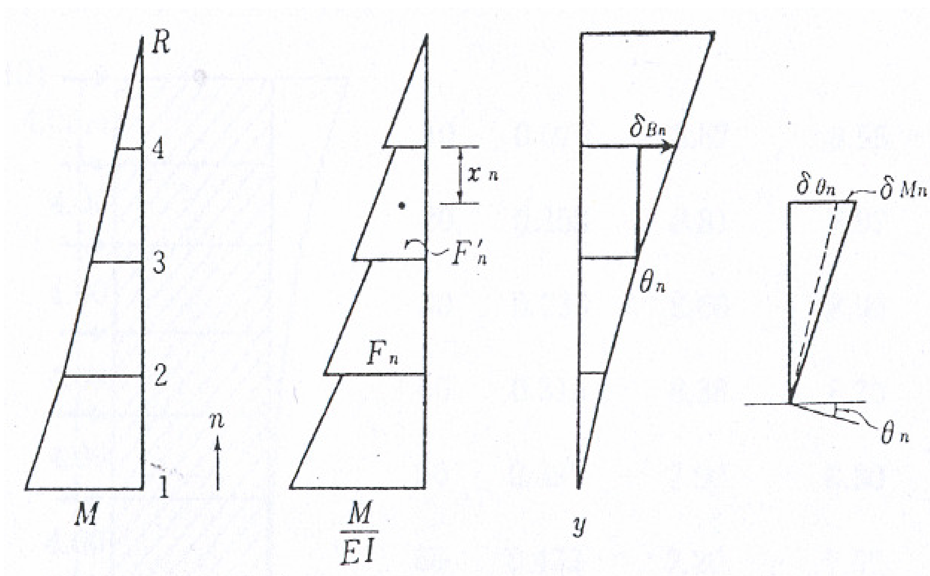


Fig. 3.3. Bending deformation

Foundation Rotation: When there is a rotation of θ at the base, the relative displacement of story n becomes a multiple of h_n. (Fig. 3.4.).

Movement of Base: When there is a movement of the base as illustrated in Fig. 3.5., the displacements of all stories are the same and there will be no relative displacements at upper stories. Only the first story incurs relative displacement against the original point on the ground.

The relative displacement of the first story due to movement of the base is

$$\delta_{D1} = \delta_0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\text{(3-3)}$$

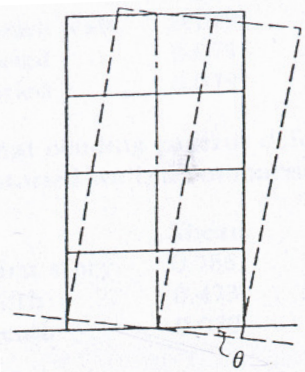


Fig. 3.4. Foundation rotation

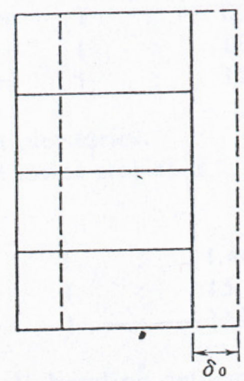


Fig. 3.5. Base movement

Relative displacement does not occur at the upper stories (Fig. 3.5.).

$$n \geq 2, \qquad \delta_{Dn} = 0$$

In other words, movement of the base increases relative displacement of the first story with the original point on the ground as reference, but does not affect the upper stories.

Total Deformation: Thus, total deformation of a story with reference to a story below it, will be as shown in the equation below.

$$\left.\begin{aligned} &\text{Stories in general} \quad \delta_n = \delta_{sn} + \delta_{Bn} + \delta_{Rn} \\ &\text{First story} \quad \delta_1 = \delta_{s1} + \delta_{B1} + \delta_{R1} + \delta_{D1} \end{aligned}\right\} \quad \text{...........................(3-4)}$$

For practical calculations of the shear distribution coefficients, D_n, the deformation, δ_n, considering only shear, bending, and foundation rotation is expressed with the practical unit, $h_n^2/12EK$, and the shear distribution coefficient is obtained from the equation below and the unit is $12EK/h_n^2$.

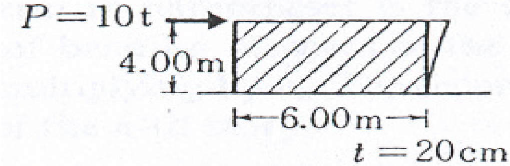
$$D_n = \frac{Q_n}{\delta_n} \quad \text{..(3-5)}$$

3.1.2. Deflection Properties of Free-standing Multi-storied Shear Walls:

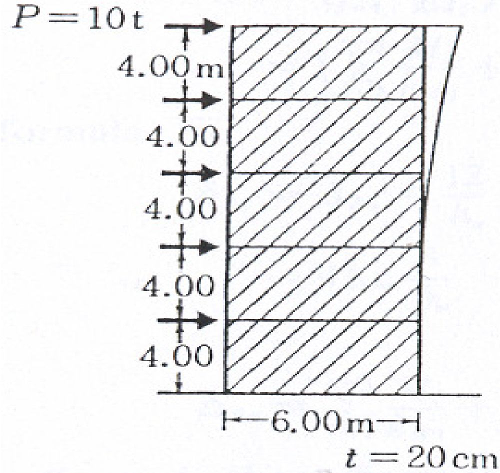
As explained previously, the deformations of a wall are divided into those due to bending, shear, rotation and movement of base. Of these, the deformations due to elastic deflection of the wall are the bending and shear deformations, and their properties will be discussed here.

Shear deformation is proportional to the shear force to which a story is subjected and therefore the properties are clear-cut. Calculations are simple since only the particular story is considered. In contrast, bending deformation is related to the shear force applied to the particular story and the bending moment from upper stories, and is further affected by rotation angles due to bending of lower stories so that the characteristics are complex. The deformation is thus governed by the location of the story and the condition of distribution of external forces above and below the story. The effect of bending deformation becomes overwhelmingly great in walls of multiple stories and rigidity is markedly reduced at upper stories.

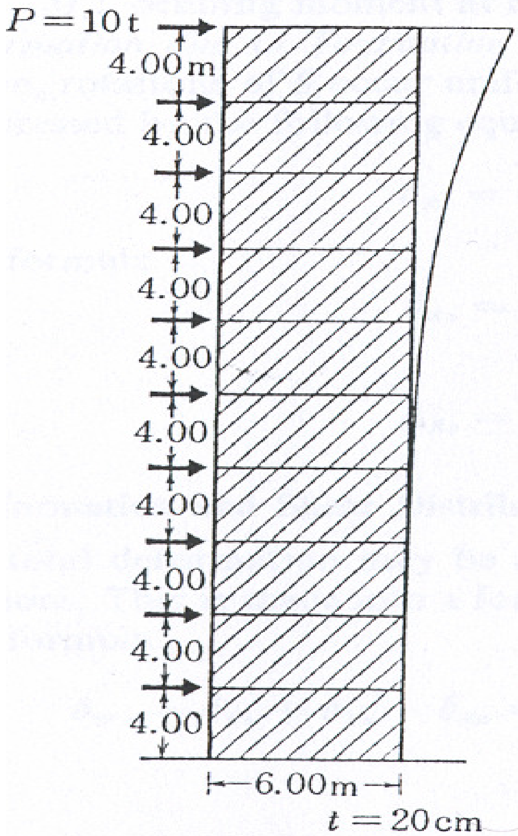
In Fig. 3.6., it is shown (as given by Muto) how distributions of bending and shear deformations differ for one-storied, five-storied and ten-storied free-standing walls of identical dimensions with each story subjected to lateral force of 10 tons. It should be noted that with multiple stories the proportion of bending deformation to shear deformation is extremely large, especially in the upper stories.



Q	δ_S	δ_B	δ_W
10	0.0788	0.0154	0.0942



Q	δ_S	δ_B	δ_W
10	0.079	1.26	1.34
20	0.158	1.20	1.36
30	0.236	1.05	1.29
40	0.315	0.76	1.08
50	0.395	0.29	0.689



Q	δ_S	δ_B	δ_W
10	0.079	8.87	8.95
20	0.158	8.81	8.97
30	0.236	8.66	8.90
40	0.315	8.38	8.70
50	0.395	7.90	8.30
60	0.473	7.20	7.67
70	0.552	6.22	6.77
80	0.632	4.92	5.55
90	0.710	3.25	3.96
100	0.788	1.15	1.94

Fig. 3.6. Examples of deformations of walls

For example, at the first story of the three types, the ratio of bending deformation to shear deformation is:

	Shear		Bending				
1-storied wall	0.079	:	0.015	=	1	:	0.19
5-storied	0.395	:	0.29	=	1	:	0.73
10-storied	0.788	:	1.15	=	1	:	1.46

showing fair amounts of difference, and it is clear that the ratio becomes larger with greater number of stories.

Next, to make a comparison of top stories

	Shear		Bending				
1-storied wall	0.079	:	0.015	=	1	:	0.19
5-storied	0.079	:	1.26	=	1	:	16
10-storied	0.079	:	8.87	=	1	:	112

And it is seen that bending governs deformation in multiple stories.

For a 10-storied wall, a comparison of upper and lower stories is

	Shear		Bending				
First story	0.788	:	1.15	=	1	:	1.46
Fifth	0.473	:	7.20	=	1	:	15
Tenth	0.079	:	8.87	=	1	:	112

and it is seen that in upper stories of a multi-storied wall, bending deformation governs while the shear deformation is negligible.

It should be noted that the higher the number of stories, the markedly greater are the relative displacements of the upper stories compared with those of the lower stories, and that rigidities are greatly reduced. According to Muto, in consideration of this characteristic, it can be shown that a slender (e. g. 6m) free-standing shear wall used in a multi-storied building (e. g. 10 stories) does not possess the ability to withstand seismic forces due to the reduction in rigidity at upper stories. In order to obtain a strong shear wall, it must be made a broad wall or the aid of boundary effect of a continuous frame must be utilized.

3.2. Lateral Rigidity of Shear Walls.

In the study of deformation of a free-standing shear wall, it has been found that deformation is large and rigidity is markedly reduced when there is influence of bending deflection. This section discusses how shear force is carried when a shear wall is joined to a frame in order that these points may be better understood.

The case of a shear wall and frame joined at each floor and showing identical horizontal displacements is taken assuming that the wall shows bending and shear deformation. It is considered that the deformation of the frame is represented by the D-value. The case of uniform distribution of horizontal load is taken and the characteristics of the distribution of shear force between wall and columns of frame are studied mathematically. The equation is expressed with a difference equation, but for convenience of handling, a differential equation is used as this is thought to be adequate for study in the case of multiple stories.

As a basis, the differential equation for a joined wall and frame is derived. For a case of both wall and frame having uniform cross sections, solutions are sought assuming fixed base.

Fundamental Equations

(1) Fundamental Equation for Frames (Uniform Cross Section)

Using the notations indicated in Fig. 3.7., the rotation angle of the column can be written as follows:

$$R = \frac{\delta}{h} \rightarrow \frac{\Delta_y}{\Delta_x} \rightarrow \frac{dy}{dx}$$

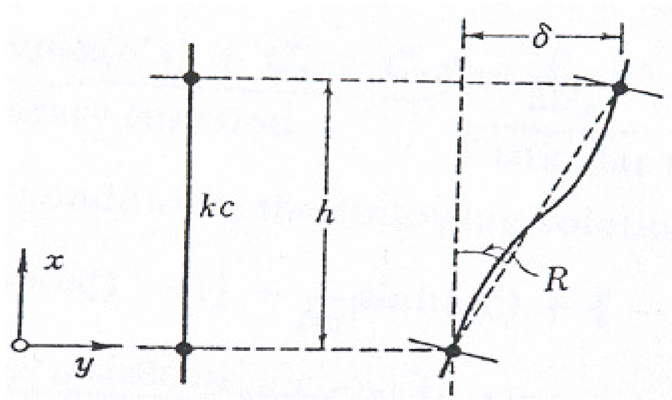


Fig. 3.7. Deformations of frame

Then $$\frac{dy}{dx} = \frac{Q_F}{G_F} \quad \text{..(3-6)}$$

By differentiation, the following equation is obtained

$$\frac{d^2 y}{dx^2} = -\frac{P_F}{G_F} \quad \text{..(3-7)}$$

Where F: subscript expressing frame

$$G_F = \sum D_F = \sum ak_c \frac{12EK}{h} : \text{ Rigidity of frame..(3-8)}$$

$Q_F = P_F$ (h-x)

P_F = uniformly distributed lateral load

h = story height

The rigidity of frame is based on the shear distribution coefficient by Muto's approximate method of calculation. D_F is proportional to the stiffness ratio of the column and is related to the sum of the stiffness ratios of the beams on the left and right.

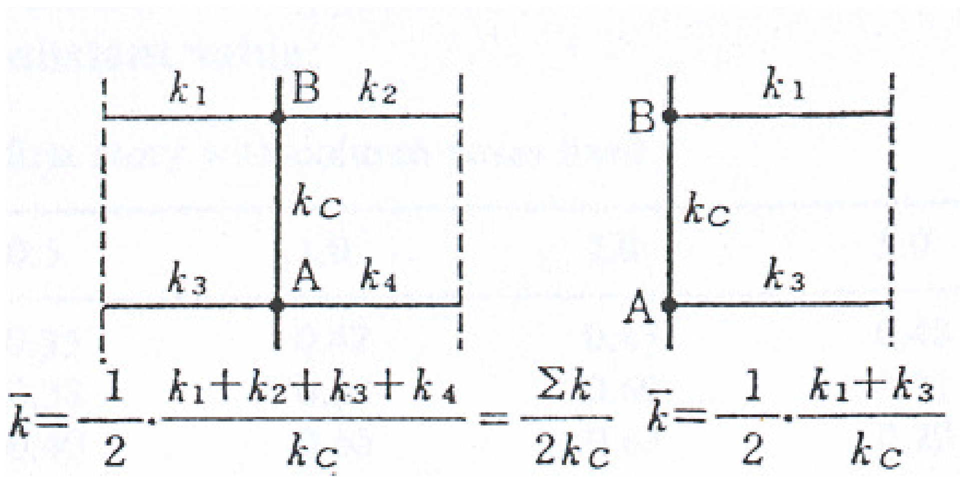


Fig. 3.8. Effective stiffness ratios

If $\bar{k}$ represents the total sum of stiffness ratios of beams above and below the column divided by $2k_c$ (Fig. 3.8.), the approximate formula for general cases is derived using this $\bar{k}$ obtaining the coefficient of rigidity, *a*, from

$$a = \bar{k}/(2+\bar{k}) \qquad \text{(3-9)}$$

Fixed Column Base: The above formula for cases of fixed column base is

$$a = \frac{(0.5+\bar{k})}{(2+\bar{k})} \quad and \ \bar{k} = \frac{\sum k_{top}}{k_c} \qquad \text{(3-10)}$$

The relative deflection, δ, of the top end when uniformly distributed lateral load p, is carried by the frame only is expressed by the equation below

$$\delta_F = \frac{pH^2}{2G_F} \qquad \text{(3-11)}$$

(2) Fundamental equation for shear walls

The differential equation for shear walls derived considering both bending and shear deformation at the same time is as follows, Fig. 3.9.

$$\frac{dy}{dx} = \left(\frac{dy}{dx}\right)_M + \left(\frac{dy}{dx}\right)_Q \quad \text{...(3-12)}$$

where

$$\left(\frac{dy}{dx}\right)_Q = \frac{\beta_0 \kappa Q_W}{GA} \text{ (Shear deformation angle of shear wall, } \gamma \text{)} \quad \text{...(3-13)}$$

Where, β_0 is the coefficient of inelastic shear deformation (inverse of β).

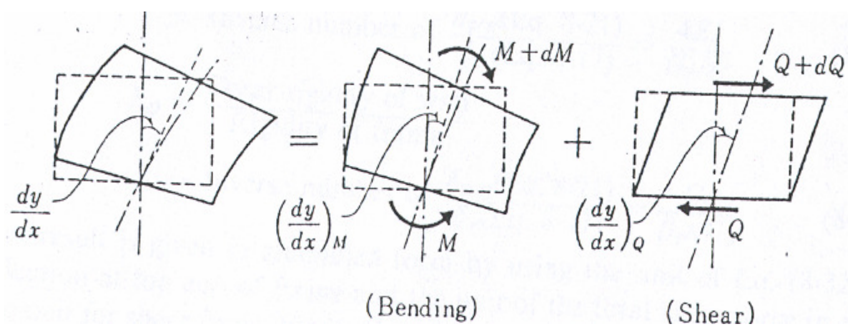


Fig. 3.9. Deformations of wall segment.

Differentiating this further, the equation below is derived (see *derivation 1. below*).

$$\left(\frac{d^2 y}{dx^2}\right) = -\frac{M}{EI} - \frac{\beta_0 \kappa}{GA} p_W \quad \text{...(3-14)}$$

When uniformly distributed load, p, is carried only by the wall, the relative deflection, δ_w, at the top end is given as shown below as the sum of the relative deflections, δ_{WM} and δ_{WQ}, due to bending and shear.

$$\delta_W = \delta_{WM} + \delta_{WQ} = \frac{pH^4}{8EI} + \frac{\beta_0 \kappa p H^2}{2GA} \quad \text{...(3-15)}$$

Derivation 1.

Free standing shear wall:

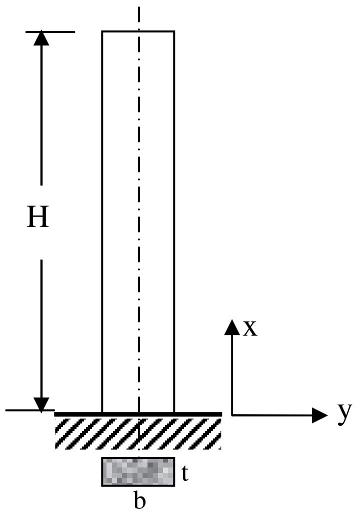


Fig. 3.10. Free standing shear wall

Total deformation of wall = Bending defromation + Shear deformation.

$$y(x)_{Total} = y(x)_{Bending} + y(x)_{Shear}$$

$$\Rightarrow \left(\frac{dy}{dx}\right)_{Total} = \left(\frac{dy}{dx}\right)_{Bending} + \left(\frac{dy}{dx}\right)_{Shear}$$

where $\left(\frac{dy}{dx}\right)_{Shear} = \frac{\beta_0 \kappa Q_w}{GA}$

For uniform lateral load, P

$Q_w = p(H-x)$

$$\left(\frac{d^2y}{dx^2}\right)_{Shear} = \frac{d}{dx}\left(\frac{\beta_o \kappa p(H-x)}{GA}\right) = -\frac{\beta_0 \kappa p}{GA}$$

and

$$\left(\frac{d^2y}{dx^2}\right)_{Bending} = -\frac{M}{EI} \text{ . . . . .Beam theory}$$

$$\Rightarrow \left(\frac{d^2y}{dx^2}\right) = -\frac{M}{EI} - \frac{\beta_0 \kappa p}{GA}$$

where

$$M(x) = -\left(\frac{p(H-x)^2}{2}\right) = -\frac{p}{2}\left(H^2 - 2Hx + x^2\right)$$

$$\left(\frac{d^2y}{dx^2}\right) = -p\left[-\frac{1}{2EI}\left(H^2 - 2Hx + x^2\right) + \frac{\beta_0 \kappa}{GA}\right]$$

$$\left(\frac{dy}{dx}\right) = -p\left[-\frac{1}{2EI}\left(H^2 x - Hx^2 + \frac{x^3}{3}\right) + \frac{\beta_0 \kappa}{GA} x\right] + c_1$$

$$y(x) = -p\left[-\frac{1}{2EI}\left(\frac{H^2x^2}{2} - \frac{Hx3}{3} + \frac{x^4}{12}\right) + \frac{\beta_0\kappa}{2GA}x^2\right] + c_1x + c_2$$

Boundary conditions:

$At \quad x = 0, \qquad y = 0$

$$\Rightarrow c_2 = 0$$

$$\frac{dy}{dx} = \left(\frac{dy}{dx}\right)_{shear} = \frac{\beta_0\kappa Q_W}{GA} = \frac{\beta_0\kappa pH}{GA} = c_1$$

Hence, $$y(x) = p\left[\frac{1}{2EI}\left(\frac{H^2x^2}{2} - \frac{Hx3}{3} + \frac{x^4}{12}\right) - \frac{\beta_0\kappa}{GA}\left(\frac{x^2}{2} - Hx\right)\right] \qquad (3\text{-}16)$$

And $$\left(\frac{dy}{dx}\right) = p\left[\frac{1}{2EI}\left(H^2x - Hx^2 + \frac{x^3}{3}\right) - \frac{\beta_0\kappa}{GA}(x - H)\right] \qquad (3\text{-}17)$$

(3) Equation for Connection

The frame and shear wall are joined in one equation from the condition that the sum of the lateral forces carried by the two should be equal to the external force, Fig. 3.11.

$$P = P_W + P_F \qquad (3\text{-}18)$$

Using this equation and Eqs. (3-7) and (3-14) together [Muto, 1974]

$$EI\left(1 + \frac{\beta_0\kappa G_F}{GA}\right)\frac{d^4y}{dx^4} - G_F\frac{d^2y}{dx^2} = p \qquad (3-19)$$

or

$$\frac{d^4y}{dx^4} - a^{12}\frac{d^2y}{dx^2} = \frac{a^{12}}{G_F}p \qquad (3-20)$$

where

$$a^{12} = \frac{G_F}{bEI},\ b = \left(1 + \frac{\beta_0\kappa G_F}{GA}\right)$$

This is the relation between displacement and load of the connected wall and frame. Obtaining a general solution from the particular solution and complimentary function for a given p and applying the boundary condition, the displacement is determined and the loads, shear forces and bending moment for the frame and wall are obtained from the following equations:

Wall:

$$\begin{cases} p_w = bEI\dfrac{d^4y}{dx^4} \\ Q_w = -bEI\dfrac{d^3y}{dx^3} \\ M_w = -EI\left(b\dfrac{d^2y}{dx^2} + \dfrac{\beta_0\kappa}{GA}p\right) \end{cases} \quad \text{(3-21)}$$

Frame:

$$\begin{cases} p_F = -G_F\dfrac{d^2y}{dx^2} \\ Q_F = G_F\dfrac{dy}{dx} \end{cases} \quad \text{(3-22)}$$

Case of Uniform Sections and *Completely Fixed Base*

Boundary condition :

x = 0, $\quad y = 0 \quad$ (3-23)

$$\frac{dy}{dx} = -\frac{\beta_0\kappa}{GA}bEI\frac{d^3y}{dx^3} \quad \text{(3-24)}$$

x=H,

$$M_w = -EI\left(b\frac{d^2y}{dx^2} + \frac{\beta_0\kappa}{GA}p\right) = 0 \quad (3-25)$$

$$Q = Q_w + Q_F = -bEI\frac{d^3y}{dx^3} + G_F\frac{dy}{dx} = 0 \quad (3-26)$$

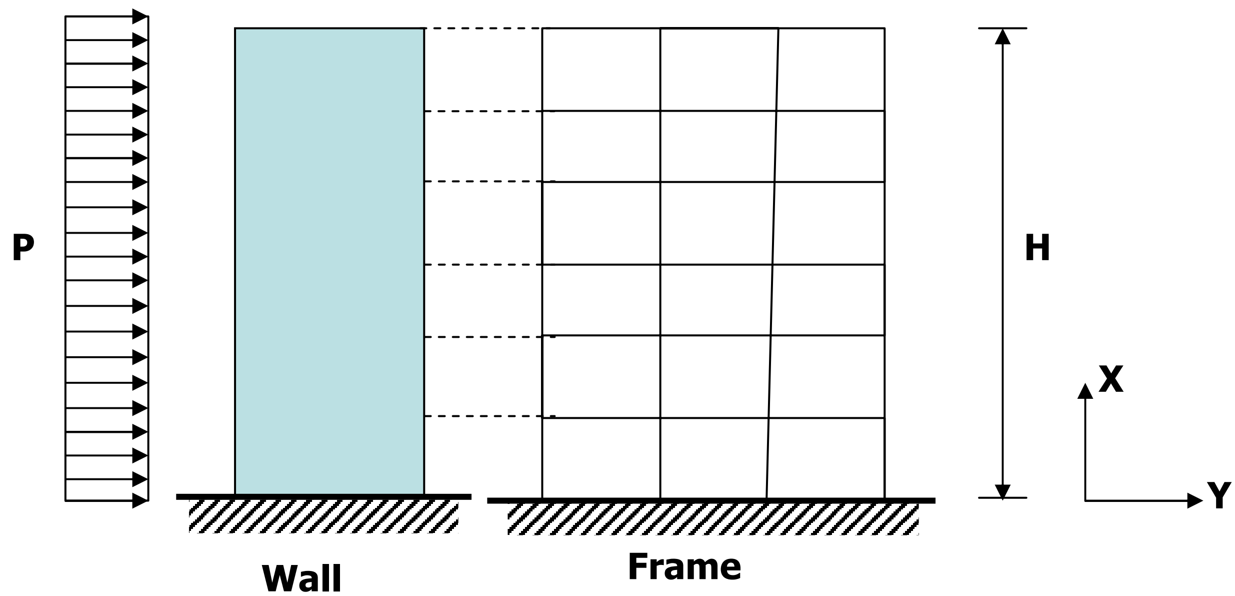


Fig. 3.11. wall-frame system

Solution: The following general solution is obtained from the complimentary function and particular solution of Eq. (3-20).

$$y = \underbrace{A\sinh(a'x) + B\cosh(a'x) + c_1 + c_2 x}_{\text{Complementary function}} - \underbrace{\frac{p}{2G_F}x^2}_{\text{Particular solution}} \quad \text{(3-27)}$$

From the above boundary conditions, the following solution is obtained

$$y = 2\left[\frac{1 + a\sinh(a)}{ba^2\cosh(a)}\{\cosh(a\xi) - 1\} - \frac{1}{ba}\sinh(a\xi) + \xi - \frac{\xi^2}{2}\right]\left[\frac{pH^2}{2G_F}\right] \quad (3-28)$$

$$Q_w = \frac{1}{b}\left[\cosh(a\xi) - \frac{1 + a\sinh(a)}{a\cosh(a)}\sinh(a\xi)\right][pH] \quad (3-29)$$

Where

$$\xi = \frac{x}{H} \tag{3-30}$$

$$b = \left(1 + \frac{\beta_0 \kappa G_F}{GA}\right) = 1 + \frac{1}{R_Q} \tag{3-31}$$

$$a^2 = \frac{G_F H^2}{EI\left(1 + \frac{\beta_0 \kappa G_F}{GA}\right)} = \frac{4}{R_M\left(1 + \frac{1}{R_Q}\right)} = (a' H)^2 \tag{3-32}$$

$$\begin{aligned} R_M &= \frac{Bending\ rigidity\ of\ wall}{Rigidity\ of\ frame} \\ &= Inverse\ number\ of\ \frac{\delta_{WH}}{\delta_F} = \frac{4EI}{G_F H^2} \end{aligned} \tag{3-33}$$

$$\begin{aligned} R_Q &= \frac{Shear\ rigidity\ of\ wall}{Rigidity\ of\ frame} \\ &= Inverse\ number\ of\ \frac{\delta_{WQ}}{\delta_F} = \frac{GA}{\beta_0 \kappa G_F} \end{aligned} \tag{3-34}$$

The last results, Eqs. (3-28) and (3-29), are given in simplified forms by using the unit [$PH^2/2G_F$] as the unit for deflection at top end of frame and [PH] as the unit for shear force distribution. The coefficients, a, and b, in the equation are determined from only bending stiffness ratio, R_M, and shear stiffness ratio, R_Q, of wall to frame and are in forms which make it easy to grasp their physical meanings.

4. MODELING OF SHEAR WALL SYSTEMS

The first step in the analysis of building structures is to idealize the structure in to a three dimensional assemblage of vertical columns and horizontal beams at each floor. In common with other modeling techniques, the analogous model must be able to simulate the significant characteristics of the prototype. The modeling of columns and beams is a straightforward procedure, but particular care is needed in modeling shear walls. In the case of shear walls it is necessary to duplicate the bending, shear, and axial stiffness of the corresponding wall segment.

It can be seen that in the case of a coupled shear wall system subjected to horizontal loads, each wall has a tendency to rotate about its base as a vertical cantilever, producing relative displacements between adjacent wall panels. The interconnecting beams or slabs which react to diminish the relative displacement are subjected to shear forces and bending moments. In a broad sense, the action of the shear wall system is thus similar to that of the moment resisting frame. Appreciation of this similarity of behavior lead to the development of the equivalent frame concept nearly three decades ago, which even today is one of the most popular methods for idealizing shear walls.

To simulate the shear wall system as an equivalent frame it is necessary to assume the following characteristics:

1. Line element of the equivalent frame extended through the center of gravity of the wall panels and beams
2. The cross sectional properties (except moment of inertia) of the columns are identical to those of the wall panels

3. In the wide column analogy with rigid offsets, it is assumed that in representing the beams of the adjacent frame, the portions of the beam falling with in the wall limits are considered as haunches with large areas and moments of inertia. The purpose of stiff haunches is to safeguard the deflection and rotation of beam ends without bending within the wall panel. The properties of the beams adjacent to the wall panel are made the same as those of the corresponding beams. The equivalent beam in the frame thus has a flexible length having the same property as the corresponding beam up to the wall panel and infinitely stiff haunches occurring within the limits of the wall.

In modeling the wall as a wide column with haunches, the actual value of sectional areas and flexural stiffness of the walls are assigned to the column. In the case of centerline columns with modified (or effective) moments of inertia, i.e. in the method which is going to be developed at the end of this work, the sectional areas of walls are assigned to the columns, but the rigid haunches and the flexible parts of the adjacent beams are replaced by their equivalent beam members with modified stiffnesses. In other words, the effect of the beam rigidity with in the wall limits is accounted for in the stiffness of the adjacent beams and a modified stiffness is calculated for the beams. Hence, in this model, particular care is required in handling the rigid beams with in the limits of the shear wall. And it is relatively easy to consider the shear deformation of the wall element by assigning equivalent shear areas to the column. All these three properties represent well all the bending, axial, and shear deformation of the wall element.

4.1. Shear Walls Connected with Frames

A case of frames connected to a shear wall will now be studied. The deflection characteristics in such a case can be considered as deformations from bending and foundation rotation in a free-standing shear wall being restrained by beams connected to the wall. However, the restraint in this case is generally not as strong as for coupled shear walls. The fact that large stresses occur at the beams connected to the shear wall is the same as for the case of coupled shear walls, but besides this, there are also concentrations of stress at adjoining columns due to forced deformation from the wall so that special calculations are necessary for these portions. In this section, examples of stresses and deformations in this type of shear wall are first shown after which a general method of analysis will be described.

4.2. Stress and Deformation of Shear Wall Connected to Frames

Calculations of forces and deformations obtained by an approximate method for a shear wall connected with frames as in Fig. 4-1 will be shown.

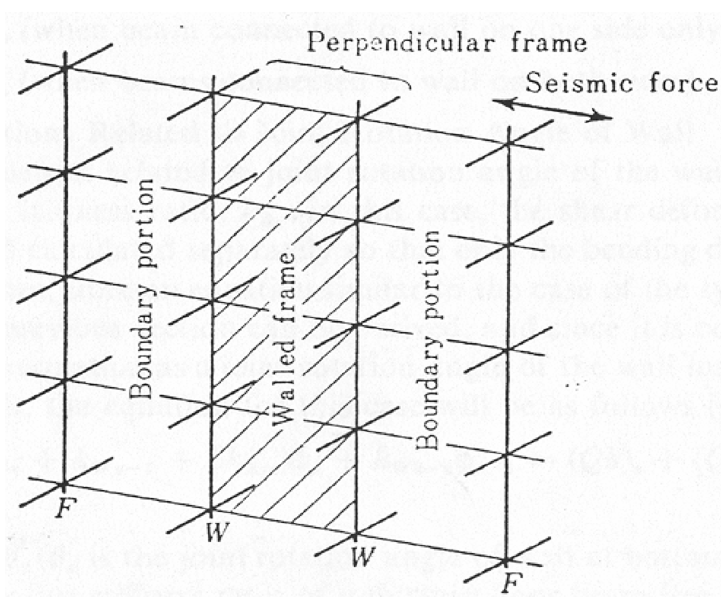


Fig. 4.1. Shear wall connected to frame

Approximate method of handling resistance of coplanar frame:

When lateral force is applied to framework as shown in Fig.4.1., the deformations and moments will be as given in Fig.4.2., and the wall is subjected to restraint from non walled frames connected around it. This restraint is due to resistance from the coplanar frame and the perpendicular frame, of which, for the coplanar frame an approximate method as indicated below is used to simplify the analysis of the wall.

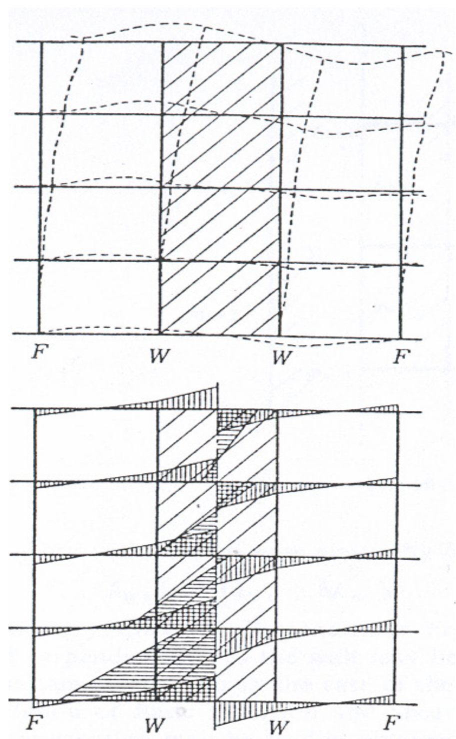


Fig. 4.2. Deformations and moments of wall and coplanar frame.

In effect, the resistance of the coplanar frame is considered to be the action of beams directly connected to the wall, and in regard to these beams:

"*the rotation angles and the deformations in the vertical direction at joints of columns adjoining the wall are neglected.*" [Muto, 1974]

By doing so, the relation between the rotation angle of the wall (on the centroid line) and the resisting moment of the beam is obtained in simplified form. The resisting moment, M_R, of an adjoining beam when there is a rotation angle, θ, at the wall centerline will be as follows (*see Fig. 4.3.*):

$$M_R = M_A + Q_{AB} * l_a$$

$$= M_A + (M_A + M_B) * l_a/l \quad \text{......(4-1)}$$

(l: length of beam, l_a: distance between centroid line and wall end)

where

$$M_A = 2EKk_B(2\theta_A - 3R_{AB}) \quad \text{......(4-2a)}$$

:bending moment at wall-side joint of beam

(k_B: stiffness ratio of beam)

$$M_B = 2EKk_B(\theta_A - 3R_{AB}) \quad \text{......(4-2b)}$$

:bending moment at opposite-side joint of beam

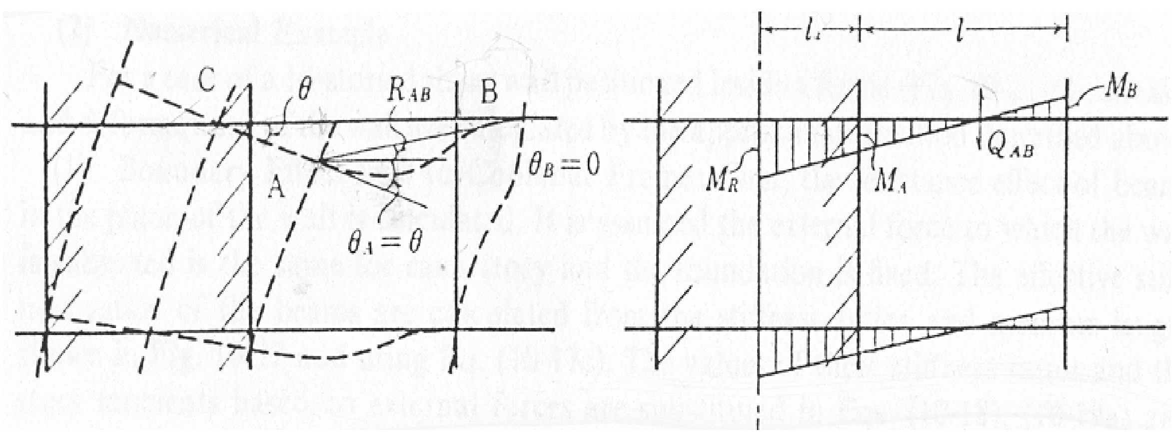


Fig. 4.3. Deformations Moments

θ_A is the rotation angle at point A and is equal to rotation angle, θ, of the wall (rotation angle of point C), while R_{AB} is the rotation angle of the beam and there is a relation of

$$R_{AB} = - l_a/l * \theta \quad \text{......(4-3)}$$

so that inserting these in Eq. (4-2a) and further substituting them into Eq. (4-1), the following equation is obtained:

$$M_R = 4EK\theta * k_{Be} \qquad (4\text{-}4)$$

where

$$k_{Be} = 1.5 * \left\{ \frac{2}{3} + 2 * \left(\frac{l_a}{l} \right) + 2 * \left(\frac{l_a}{l} \right)^2 \right\} * k_B \qquad (4\text{-}5)$$

when $l_a = l/2$

$k_{Be} = 3.25 k_B$

In other words, the effects of the rigid haunches with in the limits of the wall, Fig. 4.4., are considered to modify the stiffnesses of the adjacent beams.

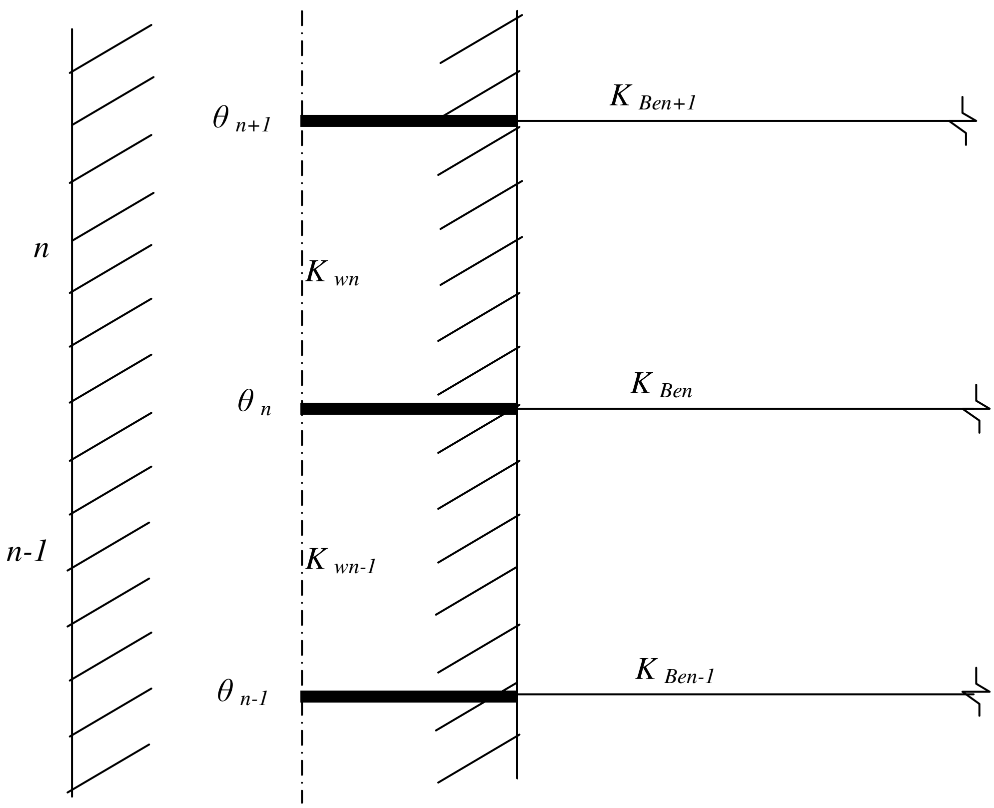


Fig. 4.4. Rigid beams within wall limits.

5. EFFECTIVE STIFFNESS OF SHEAR WALLS CONNECTED TO FRAMES

As a basis, the equivalent center line column for a free standing shear wall subjected to any type of loading can be taken as a column with all its section properties equal to the properties of the wall. This is true assuming the wall to be restrained laterally. Hence the whole cross-section of the wall can be taken as a reference and as if it is effectively resisting the applied loads.

However, in cases where the wall is used in dual systems, some part of its stiffness is shared by the attached frame, in the sense that the connected frame resists some part of the external load. As a result, the stiffness of the wall is not effectively used in resisting the externally applied load. This reduction in stiffness of the wall is directly proportional to the resistance offered by the frame which in turn is directly proportional to the reduction in rotation and lateral drift from that of a free standing shear wall.

In other words, assuming the deformations to be induced by the shear wall, the resistance by the frame, which is the load exerted by the shear wall through deflection, makes the wall not to effectively use its stiffness. This resistance by the frame is directly proportional to the reduction in lateral and rotational deflections of the shear wall. And this shows that there is a direct relationship between the reduction in stiffness of the wall and the reduction in deflections of the wall.

From the derived relation in previous sections of this report, for a shear wall acting in dual systems, it can be seen that the lateral drift and rotation for wall-beam junction of each story can be calculated.

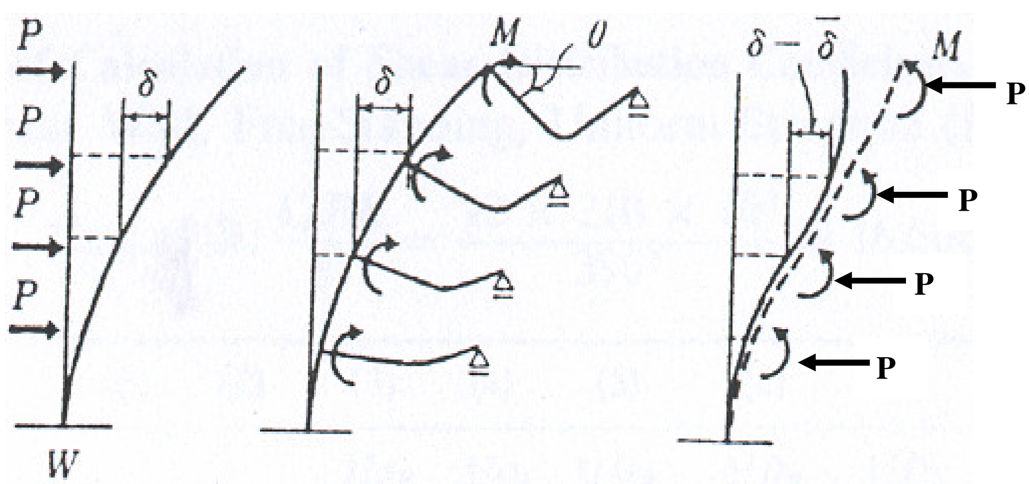
Calculation of correction factors

To calculate the correction factors to be applied to shear walls, the following assumptions are made:

- All the deformations are considered as if they are forced deformations by the shear wall.
- Each shear wall panel at each story is taken independently when applying the correction factors.

According to the above assumptions, the shear wall and beam correction factors are obtained by following the procedure below.

1. Considering bending and shear deformations, the deformation, δ, and rotation, θ, as a free standing shear wall are first calculated neglecting the boundary effects of adjacent frames, Eqs. (3-16) and (3-17), respectively.
2. Next, the beam end moment, M, and the horizontal beam reaction, P, when a boundary beam is forcibly attached to this deformed condition are considered and the corrected deformations, due to both M and P are calculated, Eq. (3-28) and its first derivative. (*see Fig. 5.1.*)



(a) (b) (c)

Fig. 5.1. Deformation of shear wall.

3. The differences of deformations obtained in steps (1) and (2) above are the reduction in deformations due to the resistance of the adjacent frame. Then, using Eq. (2-8) of chapter-two, i.e. $\frac{1}{D_W} = \frac{1}{D_S} + \frac{1}{D_B}$,

Where D_W is the total wall stiffness

D_S is the total wall shear stiffness

D_B is the total wall bending stiffness

And the change in stiffness can be written as:

$$\frac{1}{\Delta D_W} = \frac{1}{\Delta D_S} + \frac{1}{\Delta D_B} \quad \text{......(5-1)}$$

$$\Rightarrow \Delta D_W = \frac{\Delta D_S * \Delta D_B}{(\Delta D_S + \Delta D_B)} \quad \text{......(5-2)}$$

4. The stiffness reduction ratios can be related with the deformation reduction ratios as follows

$$\frac{\Delta D_W}{D_W} = \frac{\Delta \delta_W}{\delta_W}$$
$$\frac{\Delta D_S}{D_S} = \frac{\Delta \delta_S}{\delta_S} \quad \text{......(5-3)}$$
$$\frac{\Delta D_B}{D_B} = \frac{\Delta \delta_B}{\delta_B}$$

and the stiffness reduction ratio, $\frac{\Delta D_W}{D_W}$ can further be related as follows

$$\frac{\Delta D_W}{D_W} = \frac{\left(\frac{\Delta D_S}{D_S}\right) * \left(\frac{\Delta D_B}{D_B}\right)}{\left[\left(\frac{\Delta D_S}{D_S}\right) + \left(\frac{\Delta D_B}{D_B}\right)\right]} \quad \text{.......................................(5-4)}$$

5. The corrected stiffness of the wall can now be written as

$$\begin{aligned} \overline{D}_W &= D_W - \Delta D_W \\ &= \left(1 - \frac{\Delta D_W}{D_W}\right) * D_W \quad \text{...(5-5)} \end{aligned}$$

Substituting the expression for $\frac{\Delta D_W}{D_W}$ in Eq. (5-5), from expressions in Eqs. (5-3) and (5-4), the following expression is obtained.

$$\overline{D}_W = \left(1 - \frac{\left(\frac{\Delta \delta_S}{\delta_S}\right) * \left(\frac{\Delta \delta_B}{\delta_B}\right)}{\left[\left(\frac{\Delta \delta_S}{\delta_S}\right) + \left(\frac{\Delta \delta_B}{\delta_B}\right)\right]}\right) * D_W \quad \text{..(5-6)}$$

where

δ_S and δ_B are the shear and rotational deformations of the free standing shear wall, respectively, and

$\Delta\delta_S$ and $\Delta\delta_B$ are the changes in shear deformation and rotation of the dual system due to the resistance of frame, respectively.

6. As a last step, the correction factor obtained in Eq. (5-6) is directly applied to the flexural stiffness of the wall, i.e. moment of inertia of the wall, taking the direct relationship between D_W and flexural stiffness of the wall into consideration.

6. ANALYSIS EXAMPLES

As an example, different cases of dual (mixed) building systems and their equivalent frames are analyzed using Extended Three dimensional Analysis of Building Systems (ETABS) and the results are included in this report which is believed to be self explanatory where the results are written for members in order from left to right and from top to bottom. The examples taken include from a single- story, single-span dual system up to a system with six stories and three bays. In the analysis example made, wall widths of 2m and 6m with a thickness of 0.2m are taken. A uniform lateral load of 100 KN is applied at each story.

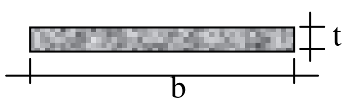
The following are the various parameters used in the analysis example.

Material:

Concrete: Modulus of elasticity, $E = 24.821$ GPa

Poisson's ratio, $\mu = 0.2$

Shear wall section:

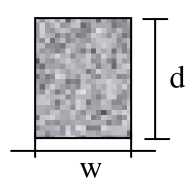


Where $t=0.2$m and

Case-1: $b=2$m , i.e. $A_w=0.4\ m^2$, $A_{ws}=0.333333\ m^2$, and $I_w=0.13333333\ m^4$

Case-2: $b=6$m, i.e. $A_w=1.2\ m^2$, $A_{ws}=1.0\ m^2$ and $I_w=3.6\ m^4$

Beam and column sections:



Where $w=0.3$m and

$d=0.5$m , i.e. $A_b= A_c =0.15\ m^2$, $A_{bs}= A_{cs} =0.125\ m^2$, and $I_b= I_c =0.003125\ m^4$

while, A_b, A_c, A_w are axial areas of beam, column, and shear wall, respectively

A_{bs}, A_{cs}, A_{ws} are shear areas of beam, column, and shear wall, respectively

I_b, I_c, I_w are moments of inertia about the major axes of beam, column, and shear wall, respectively.

In the different dual systems considered in the analysis example, W_{lmn} and F_{lmn} stand for wall and frame with '*l*' number of stories, '*m*' number of bays of frame attached to wall, and with wall width '*n*' in meters, respectively.

Out of the many examples taken, detailed calculations of the stiffness correction factors and bending moment, shear force and axial force diagrams are included in this report for examples W and F112; W and F 432; W and F436; and W and F 616.

In the preparation of the table for the correction factors, the following terms are included:

I_w, A, and b are moment of inertia, area, and width of the wall, respectively.

H, X, GF, B, and AW are as defined in Eq. (3-28) where AW = *a* in this same equation.

Y and ROT. THETA are lateral deflection and rotation of the dual system, respectively, Eq. (3-28). And YP = Y/(pH^2/GF) and TET = ROT. THETA/(pH/GF).

YF and FREE ROT are the deflection and rotation of the free standing shear wall, Eqs. (3-16) and (3-17).

FR-RT = FREE ROT-ROT. THETA

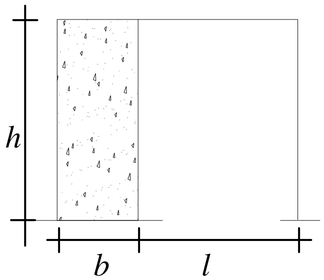
YF-Y = the difference between free deflection and deflection of the dual system.

DRIFT YF-Y = Lateral story drift.

(YF-Y)/YF = Lateral story drift ratio with respect to YF.

(FR-RT)/FR = Rotation ratio with respect to FR.

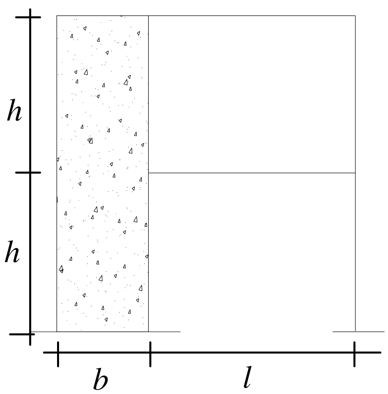
CORR. FACT. is the correction factor to be applied to the shear wall stiffness.



Wall 112, 116, i.e. wall system with one story, one bay frame connected, and wall width 2m and 6m, respectively.

$G_F=1.2837393*10^{-3}*E$

B=1.0115537 for b=2m and 1.0038512 for b=6m

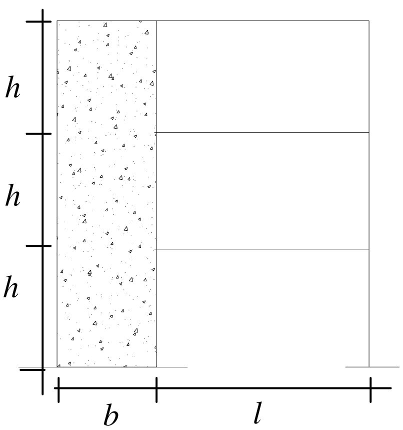


$G_{F,top}=0.691244*10^{-3}*E$

B_{top}=1.0062211 for b=2m and 1.0020737 for b=6m

$G_{F,bot}=1.2837393*10^{-3}*E$

B_{bot}=1.0115537 for b=2m and 1.0038512 for b=6m

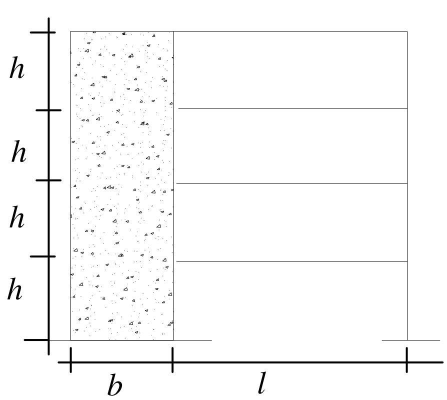


$G_{F,2nd}= G_{F,1st} =0.691244*10^{-3}*E$

$B_{2nd}= B_{1st}$ =1.0062211 for b=2m and 1.0020737 for b=6m

$G_{F,bot} =1.2837393*10^{-3}*E$

B_{bot}=1.0115537 for b=2m and 1.0038512 for b=6m



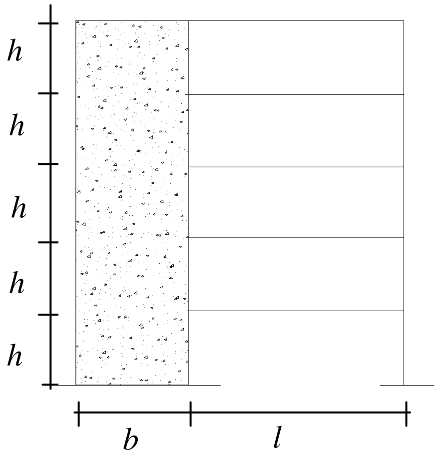
$G_{F,3rd} =G_{F,2nd}= G_{F,1st} =0.691244*10^{-3}*E$

$B_{3rd} =B_{2nd}= B_{1st}$ =1.0062211 for b=2m

and 1.0020737 for b=6m

$G_{F,bot} =1.2837393*10^{-3}*E$

B_{bot}=1.0115537 for b=2m and 1.0038512 for b=6m



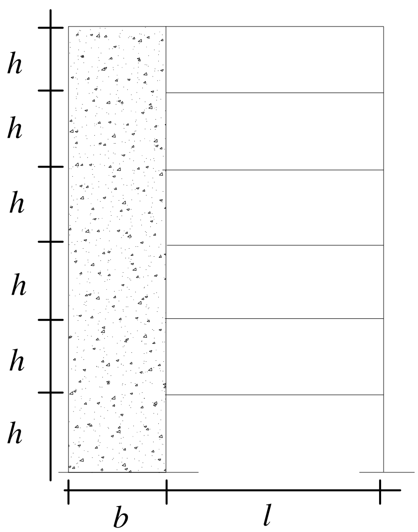
$G_{F,4th} = \ldots = G_{F,1st} = 0.691244 * 10^{-3} * E$

$B_{4th} = \ldots = B_{1st} = 1.0062211$ for b=2m

and 1.0020737 for b=6m

$G_{F,bot} = 1.2837393 * 10^{-3} * E$

$B_{bot} = 1.0115537$ for b=2m and 1.0038512 for b=6m



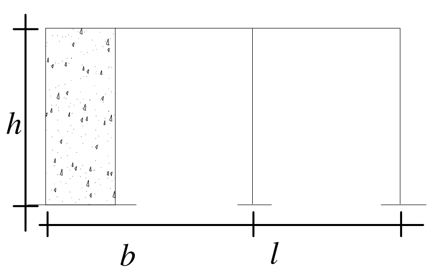
$G_{F,5th} = \ldots = G_{F,1st} = 0.691244 * 10^{-3} * E$

$B_{5th} = \ldots = B_{1st} = 1.0062211$ for b=2m

and 1.0020737 for b=6m

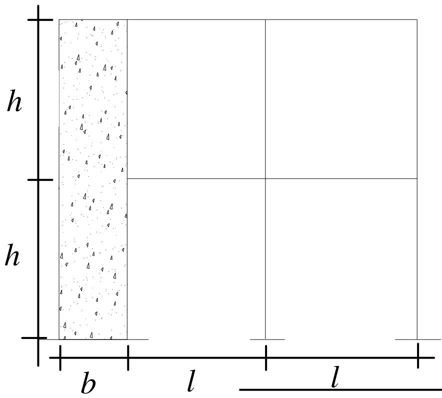
$G_{F,bot} = 1.2837393 * 10^{-3} * E$

$B_{bot} = 1.0115537$ for b=2m and 1.0038512 for b=6m



$G_F = 2.8949098 * 10^{-3} * E$

B=1.0260541 for b=2m and 1.0086847 for b=6m

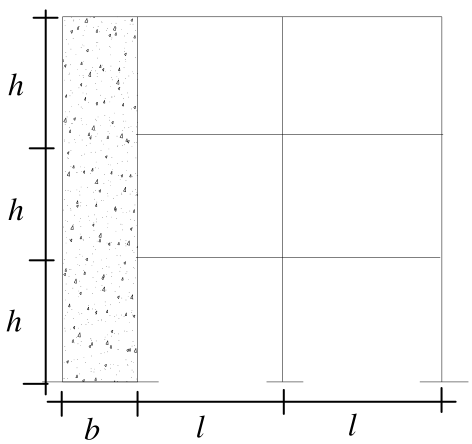


$G_{F,top}$=1.8190636*10^{-3}*E

B_{top}=1.0163715 for b=2m and 1.0054571 for b=6m

$G_{F,bot}$=2.8949097*10^{-3}*E

B_{bot}=1.0260541 for b=2m and 1.0086847 for b=6m

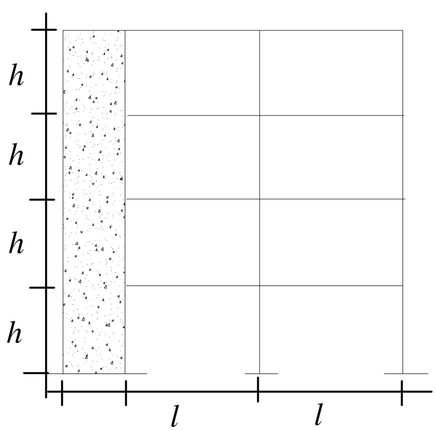


$G_{F,2nd}$= $G_{F,1st}$ =1.8190636*10^{-3}*E

B_{2nd}= B_{1st} =1.0062211 for b=2m and 1.0020737 for b=6m

$G_{F,bot}$ =1.2837393*10^{-3}*E

B_{bot}=1.0260541 for b=2m and 1.0086847 for b=6m



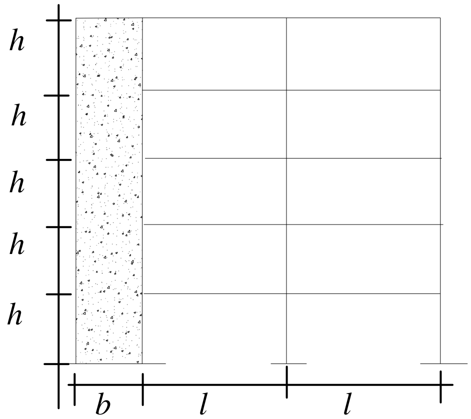
$G_{F,3rd}$ =$G_{F,2nd}$= $G_{F,1st}$ =1.8190636*10^{-3}*E

B_{3rd} =B_{2nd}= B_{1st} =1.0062211 for b=2m

and 1.0020737 for b=6m

$G_{F,bot}$ =1.2837393*10^{-3}*E

B_{bot}=1.0260541 for b=2m and 1.0086847 for b=6m



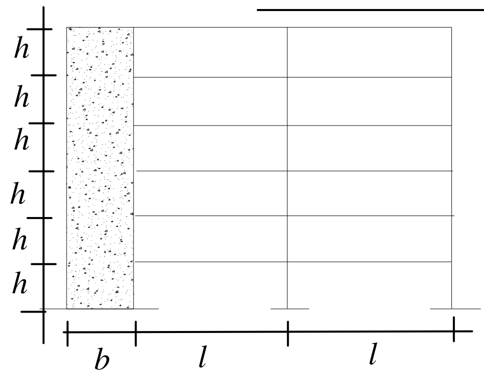
$G_{F,4th} = \ldots = G_{F,1st} = 1.8190636 * 10^{-3} * E$

$B_{4th} = \ldots B_{1st} = 1.0062211$ for b=2m

and 1.0020737 for b=6m

$G_{F,bot} = 1.2837393 * 10^{-3} * E$

B_{bot}=1.0260541 for b=2m and 1.0086847 for b=6m



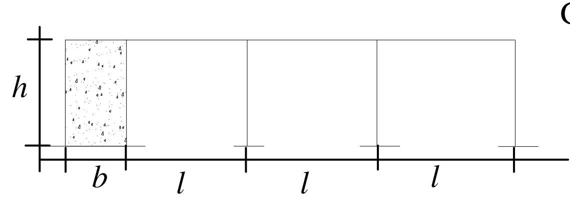
$G_{F,5th} = \ldots = G_{F,1st} = 1.8190636 * 10^{-3} * E$

$B_{5th} = \ldots = B_{1st} = 1.0062211$ for b=2m

and 1.0020737 for b=6m

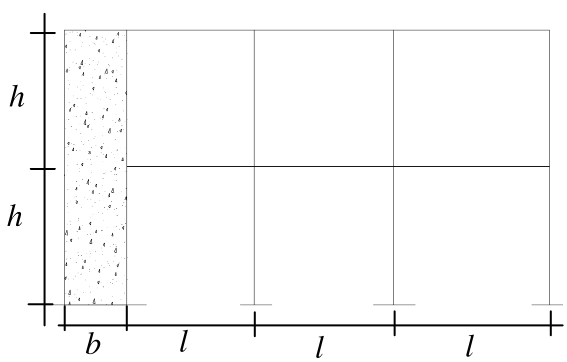
$G_{F,bot} = 1.2837393 * 10^{-3} * E$

B_{bot}=1.0260541 for b=2m and 1.0086847 for b=6m



$G_F = 4.5060802 * 10^{-3} * E$

B=1.0405547 for b=2m and 1.0135182 for b=6m

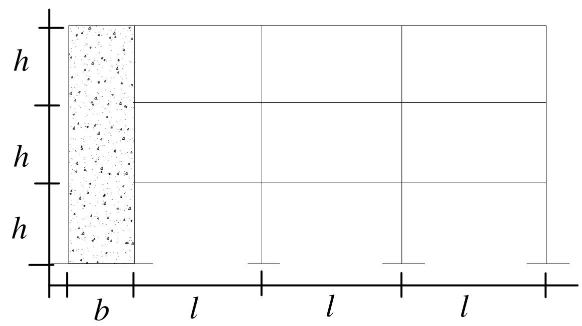


$G_{F,2nd} = G_{F,1st} = 4.5060802 * 10^{-3} * E$

$B_{2nd} = B_{1st} = 1.0265219$ for b=2m and 1.0088406 for b=6m

$G_{F,bot} = 1.2837393 * 10^{-3} * E$

B_{bot}=1.0405547 for b=2m and 1.0135182 for b=6m

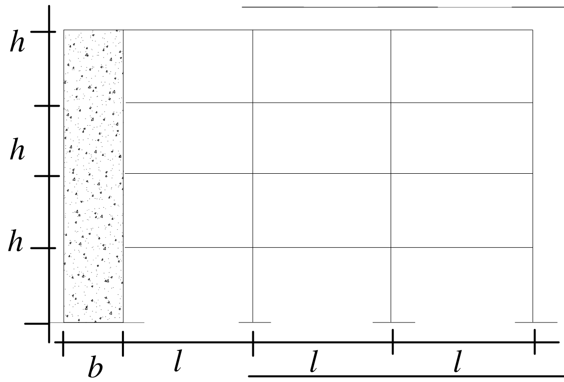


$G_{F,2nd} = G_{F,1st} = 4.5060802*10^{-3}*E$

$B_{2nd} = B_{1st} = 1.0265219$ for b=2m and 1.0088406 for b=6m

$G_{F,bot} = 1.2837393*10^{-3}*E$

$B_{bot} = 1.0405547$ for b=2m and 1.0135182 for b=6m



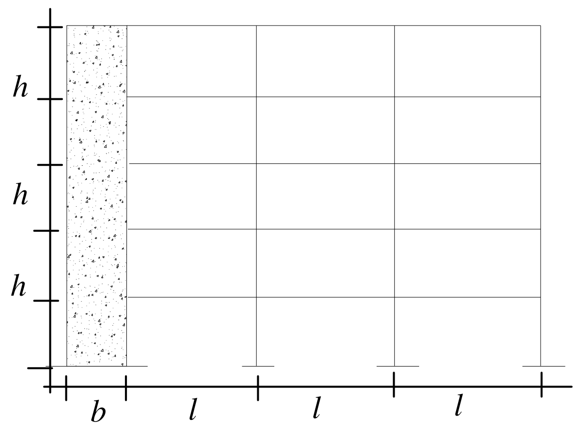
$G_{F,3rd} = G_{F,2nd} = G_{F,1st} = 4.5060802*10^{-3}*E$

$B_{3rd} = B_{2nd} = B_{1st} = 1.0265219$ for b=2m

and 1.0088406 for b=6m

$G_{F,bot} = 1.2837393*10^{-3}*E$

$B_{bot} = 1.0405547$ for b=2m and 1.0135182 for b=6m



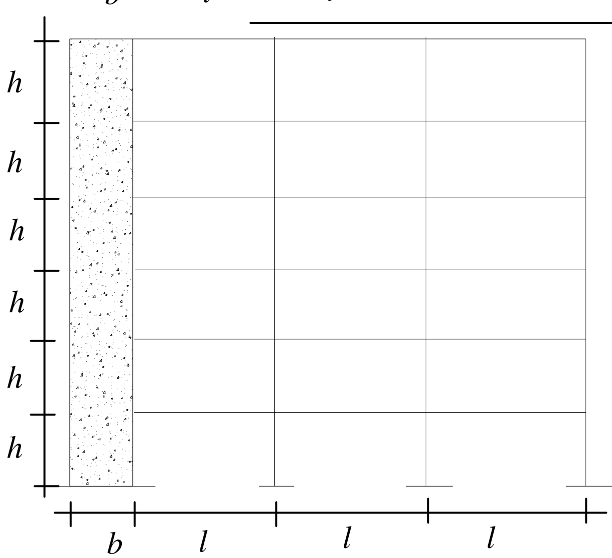
$G_{F,4th} = \ldots = G_{F,1st} = 4.5060802*10^{-3}*E$

$B_{4th} = \ldots B_{1st} = 1.0265219$ for b=2m

and 1.0088406 for b=6m

$G_{F,bot} = 1.2837393*10^{-3}*E$

$B_{bot} = 1.0405547$ for b=2m and 1.0135182 for b=6m



$G_{F,5th} = \ldots = G_{F,1st} = 4.5060802*10^{-3}*E$

$B_{5th} = \ldots = B_{1st} = 1.0265219$ for b=2m

and 1.0088406 for b=6m

$G_{F,bot} = 1.2837393*10^{-3}*E$

$B_{bot} = 1.0405547$ for b=2m and 1.0135182 for b=6m

Table 6.1. The following are the calculation details made for some of the examples, after which the corrected values for the moment of inertias of the wall at each story are applied to the equivalent frame with center line column.

Calculations for determining the correction factors for W112 and F112

IW	A	wall width, b	H	X	GF	B	AW	YP	Y	TET	ROT THETA	YF	FREE ROT	FR-RT	YF-Y	DRIFT YF-Y	(YF-Y)/YF	(FR-RT)/FR	CORR. FACT.	CORR. IW
0.133	0.4	2.00	3.50	3.50	0.0013	1.0116	0.3415	0.0195	186.029	0.0182	49.7538	195.80	53.59	3.83	9.7788	9.7788	0.0499	0.0716	0.9706	0.1294

Calculations for determining the correction factors for W432 and F432

IW	A	wall width, b	H	X	GF	B	AW	YP	Y	TET	ROT THETA	YF	FREE ROT	FR-RT	YF-Y	DRIFT YF-Y	(YF-Y)/YF	(FR-RT)/FR	CORR. FACT.	CORR. IW
0.13	0.4	2	14	14	0.0029	1.0265	2.054	0.214	14215.8	0.213	1012.7	36897	3430	2417.3	22681.2	8302	0.61	0.7047	0.6717	0.0896
0.13	0.4	2	14	10.5	0.0029	1.0265	2.054	0.158	10504.6	0.238	1128.4	24884	3407.9	2279.5	14379.2	7297.9	0.58	0.6689	0.69	0.092
0.13	0.4	2	14	7	0.0029	1.0265	2.054	0.095	6335.55	0.258	1228.1	13417	3064.3	1836.2	7081.26	4832.8	0.53	0.5992	0.7194	0.0959
0.13	0.4	2	14	3.5	0.0045	1.0406	2.523	0.045	1935.91	0.271	842.5	4184.3	2077.5	1235	2248.42	2248.4	0.54	0.5945	0.7178	0.0957

Calculations for determining the correction factors for W436 and F436

IW	A	wall width, b	H	X	GF	B	AW	YP	Y	TET	ROT THETA	YF	FREE ROT	FR-RT	YF-Y	DRIFT YF-Y	(YF-Y)/YF	(FR-RT)/FR	CORR. FACT.	CORR. IW
3.6	1.2	6	14	14	0.00295	1.0088	0.399	0.0229	1525.865	0.0245	116.435	1627.89	127.037	10.602	102.024	36.564	0.0627	0.0835	0.96421	3.47114
3.6	1.2	6	14	10.5	0.00295	1.0088	0.399	0.0166	1101.162	0.0264	125.47	1166.62	135.552	10.082	65.4594	32.437	0.0561	0.0744	0.96802	3.48486
3.6	1.2	6	14	7	0.00295	1.0088	0.399	0.0099	659.8962	0.0261	123.939	692.919	132.157	8.2182	33.0227	18.773	0.0477	0.0622	0.97302	3.50287
3.6	1.2	6	14	3.5	0.00451	1.0135	0.492	0.0059	255.0584	0.0314	97.6387	269.309	104.943	7.3046	14.2502	14.25	0.0529	0.0696	0.96994	3.49178

Calculations for determining the correction factors for W616 and F616

IW	A	wall width, b	H	X	GF	B	AW	YP	Y	TET	ROT THETA	YF	FREE ROT	FR-RT	YF-Y	DRIFT YF-Y	(YF-Y)/YF	(FR-RT)/FR	CORR. FACT.	CORR. IW
3.6	1.2	6	21	21	0.00069	1.0021	0.291	0.0112	7171.3	0.01354	411.285	7414.3	428.75	17.465	243	60.6	0.0328	0.041	0.9818	3.535
3.6	1.2	6	21	17.5	0.00069	1.0021	0.291	0.009	5714.6	0.01383	420.269	5897	437.265	16.996	182.41	57.2	0.0309	0.039	0.9828	3.538
3.6	1.2	6	21	14	0.00069	1.0021	0.291	0.0066	4242.1	0.01377	418.352	4367.4	433.87	15.518	125.21	50.23	0.0287	0.036	0.9841	3.543
3.6	1.2	6	21	10.5	0.00069	1.0021	0.291	0.0044	2812.8	0.01296	393.643	2887.7	406.656	13.013	74.982	39.71	0.026	0.032	0.9857	3.548
3.6	1.2	6	21	7	0.00069	1.0021	0.291	0.0024	1527.2	0.011	334.195	1562.4	343.713	9.5177	35.275	18.31	0.0226	0.028	0.9876	3.555
3.6	1.2	6	21	3.5	0.00128	1.0039	0.396	0.0015	520.37	0.01368	223.828	537.33	233.131	9.3028	16.967	16.97	0.0316	0.04	0.9824	3.537

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EXTENDED THREE DIMENSIONAL ANALYSIS OF BUILDING SYSTEMS
VERSION  5.41
BY
ASHRAF HABIBULLAH
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STRANCOM International                                        PAGE    1

                                   PROGRAM:ETABS/FILE:w112.FRM
UNITS:  KN-M

COLUMN  FORCES AT LEVEL ROOF      IN FRAME

 COL OUTPUT OUTPUT      MAJOR      MAJOR      MINOR      MINOR      AXIAL TORSIONAL
  ID     ID  POINT     MOMENT      SHEAR     MOMENT      SHEAR      FORCE    MOMENT
   3 CASE 1    TOP      -7.94       6.55        .00        .00      -3.94       .00
            BOTTOM      11.72                   .00

BEAM FORCES AT LEVEL ROOF      IN FRAME

 BAY OUTPUT OUTPUT      MAJOR      MAJOR      MINOR      MINOR      AXIAL TORSIONAL
  ID     ID  POINT     MOMENT      SHEAR     MOMENT      SHEAR      FORCE    MOMENT
   2 CASE 1  END-I      12.42       3.94        .00        .00        .00       .00
             END-J     -10.23                   .00

WALL FORCES AT LEVEL ROOF      IN FRAME

WALL OUTPUT OUTPUT      MAJOR      MAJOR      MINOR      MINOR      AXIAL TORSIONAL
  ID     ID  POINT     MOMENT      SHEAR     MOMENT      SHEAR      FORCE    MOMENT
   1 CASE 1    TOP     -16.36      93.45        .00        .00       3.94       .00
            BOTTOM     310.70                   .00

                                        PROGRAM:ETABS/FILE:f112.FRM
UNITS:  KN-M

COLUMN FORCES AT LEVEL ROOF      IN FRAME

 COL OUTPUT OUTPUT      MAJOR      MAJOR      MINOR      MINOR      AXIAL TORSIONAL
  ID     ID  POINT     MOMENT      SHEAR     MOMENT      SHEAR      FORCE    MOMENT
   1 CASE 1    TOP     -15.34      92.59        .00        .00       3.83       .00
            BOTTOM     308.74                   .00

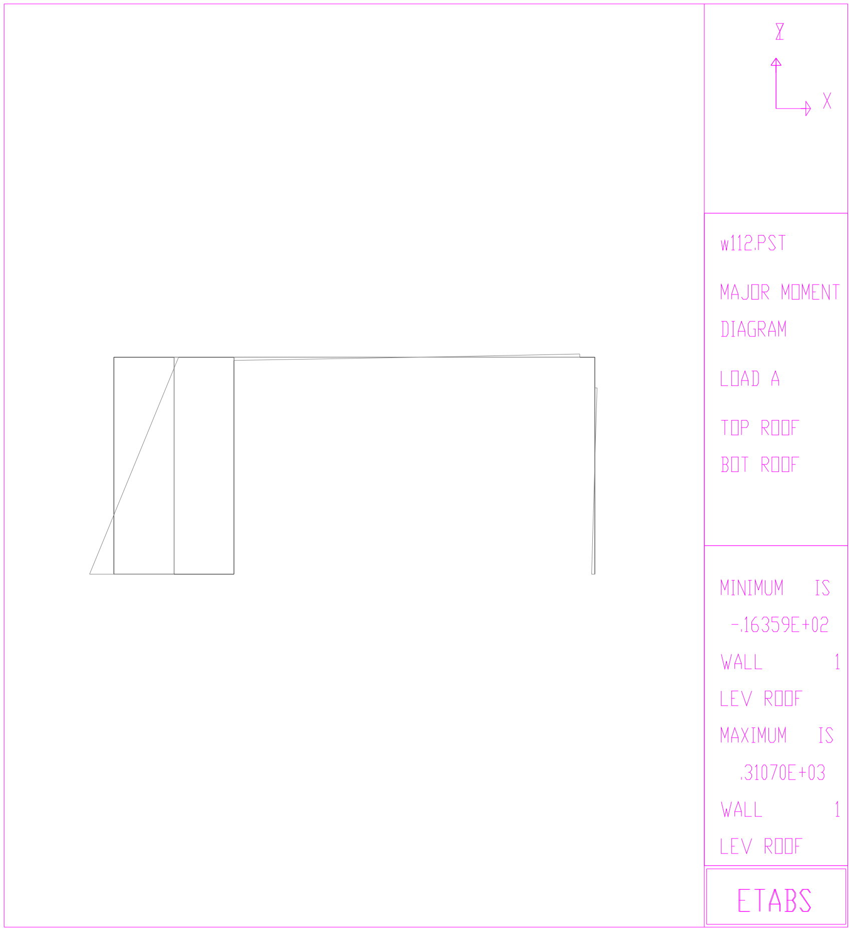
   2 CASE 1    TOP     -11.50       7.41        .00        .00      -3.83       .00
            BOTTOM      14.42                   .00

BEAM FORCES AT LEVEL ROOF      IN FRAME

 BAY OUTPUT OUTPUT      MAJOR      MAJOR      MINOR      MINOR      AXIAL TORSIONAL
  ID     ID  POINT     MOMENT      SHEAR     MOMENT      SHEAR      FORCE    MOMENT
   1 CASE 1  END-I      15.34       3.83        .00        .00        .00       .00
             END-J     -11.50                   .00
```

Table 6.2. Comparison of results for W112 and F112

	Level	Dual system (W112)				Equivalent frame (F112)				Difference in %			
		Bending moment		Shear force	Axial force	Bending moment		Shear force	Axial force	Bending moment		Shear force	Axial force
		Top	Bot.			Top	Bot.			Top	Bot.		
Shear wall	Roof	-16.36	311	93.45	3.94	-15.34	309	92.59	3.83	6.23	0.63	0.92	2.792



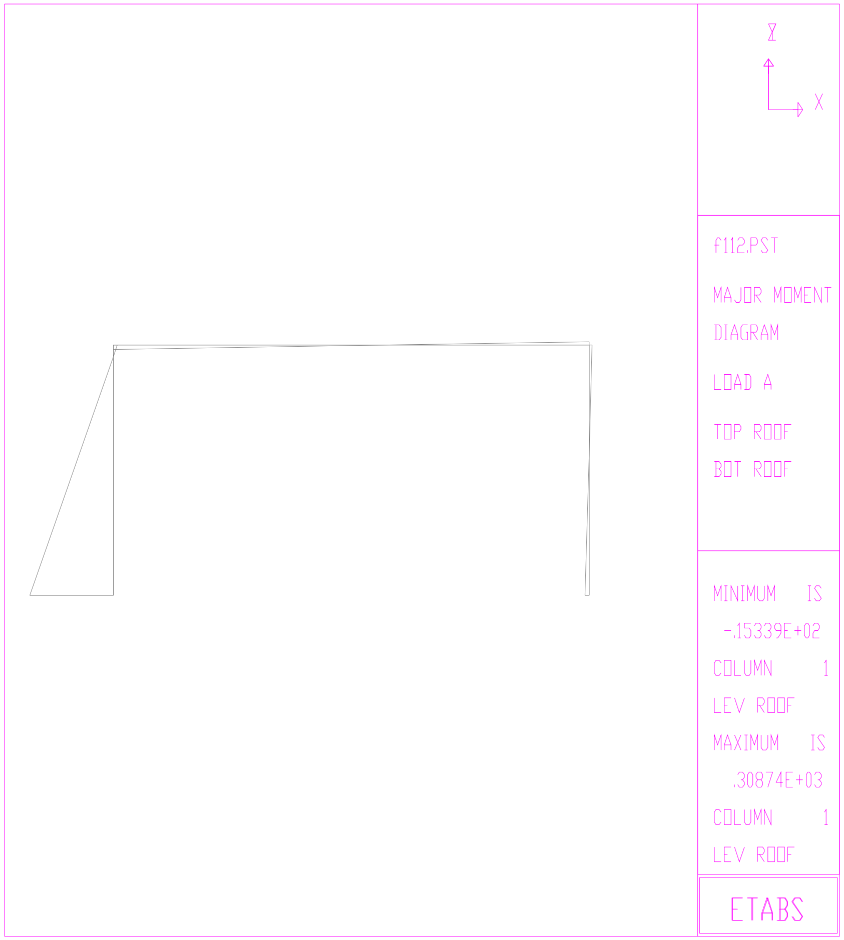
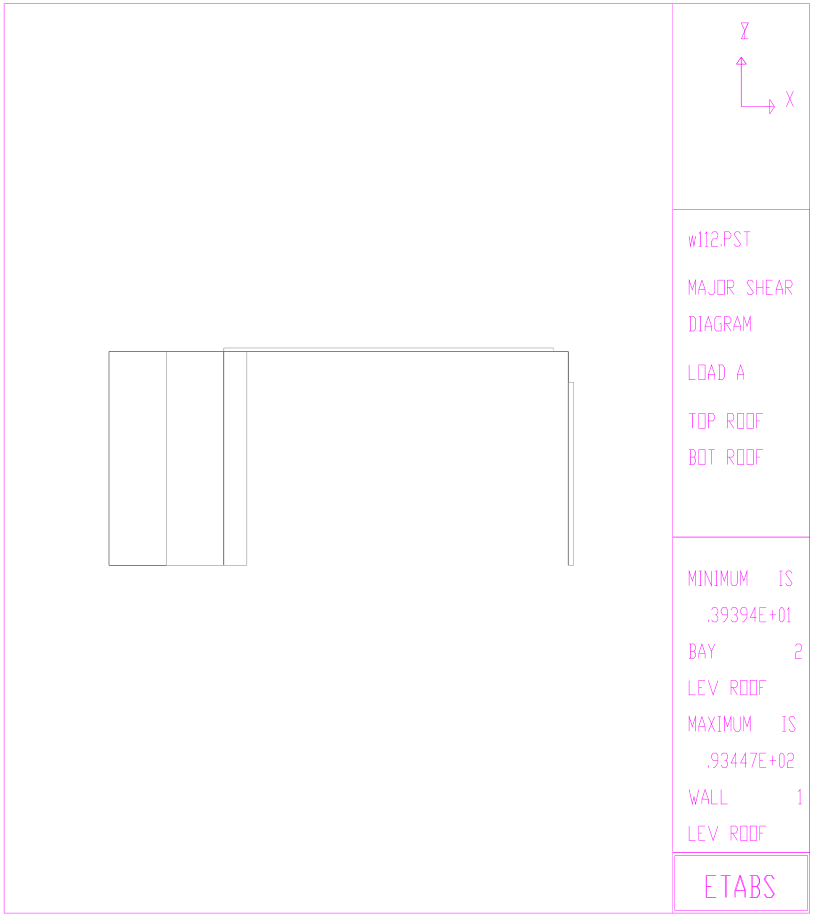
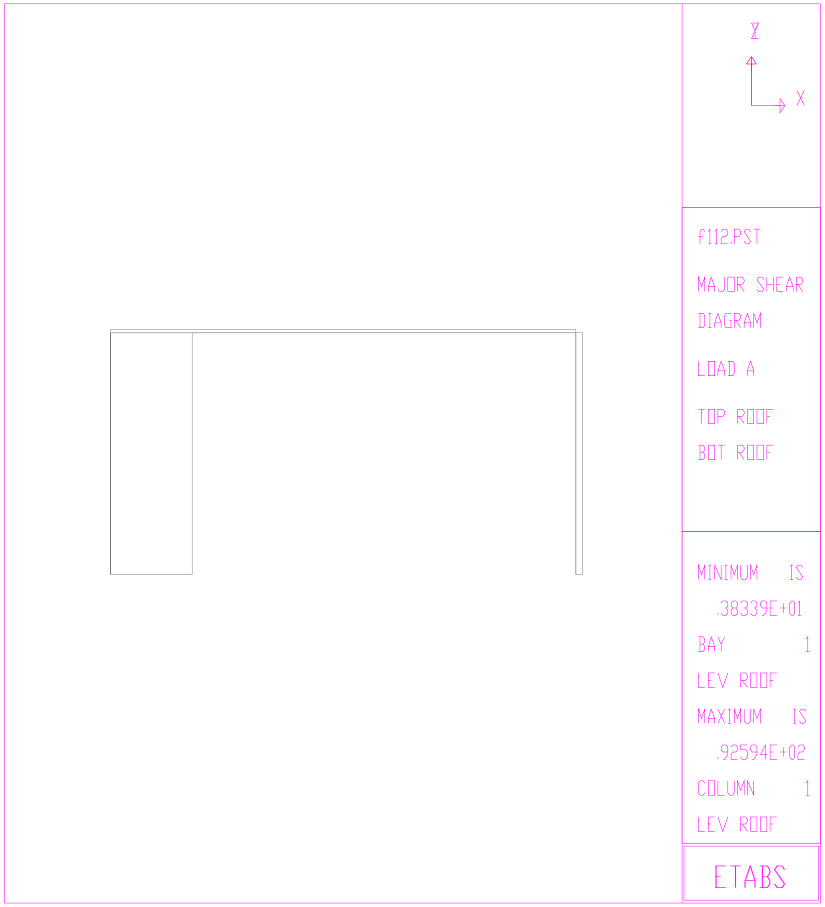
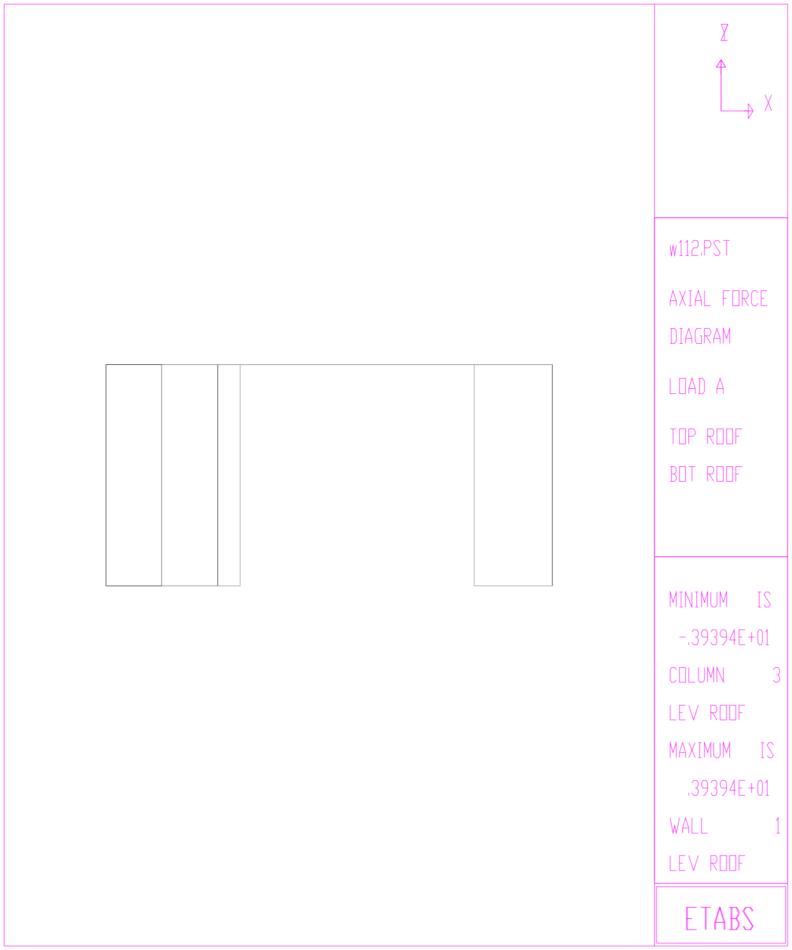


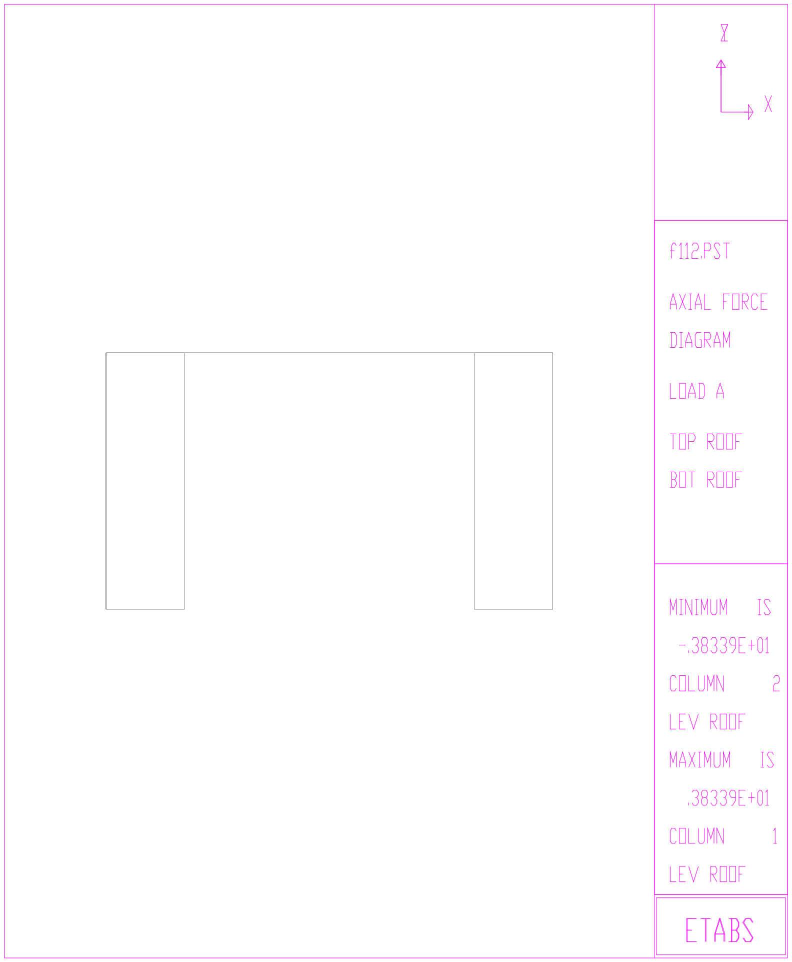
Fig. 6.1. (a) Bending moment diagram of wall *(W112)* (left) and equivalent frame *(F112)* (right), respectively



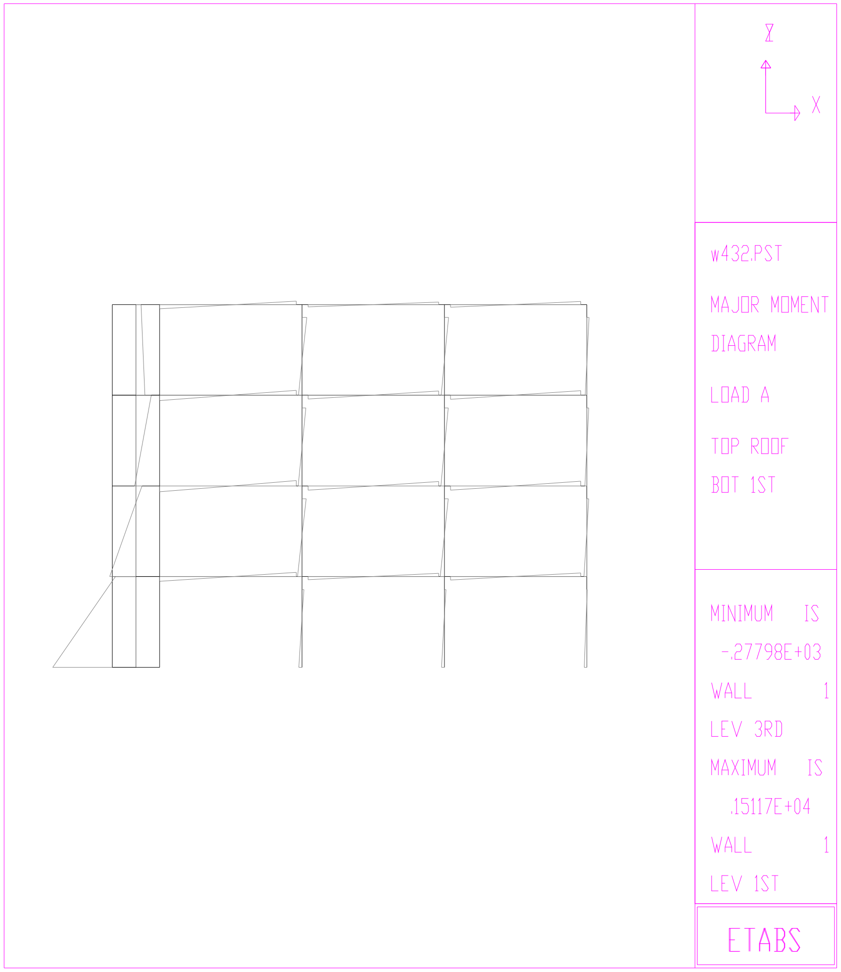


(b) Shear force diagram of wall *(W112)* (left) and equivalent frame *(F112)* **(right), respectively**





(c) Axial force diagram of wall *(W112)* (left) and equivalent frame *(F112)* (right), respectively



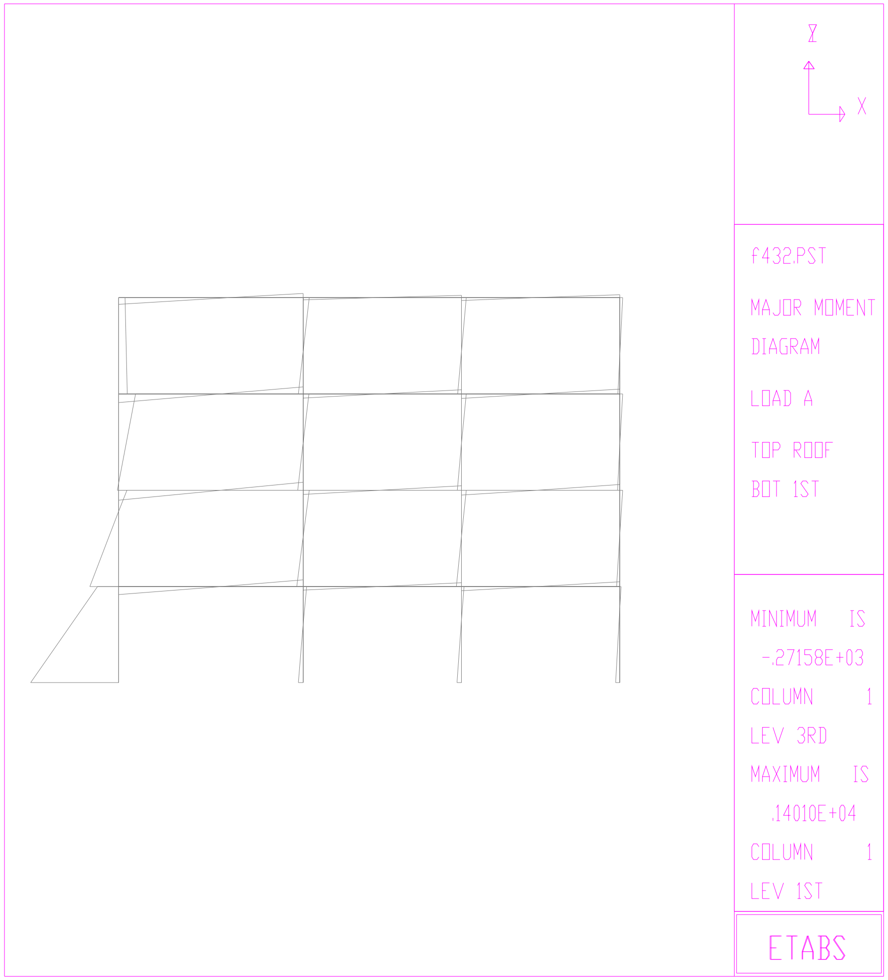
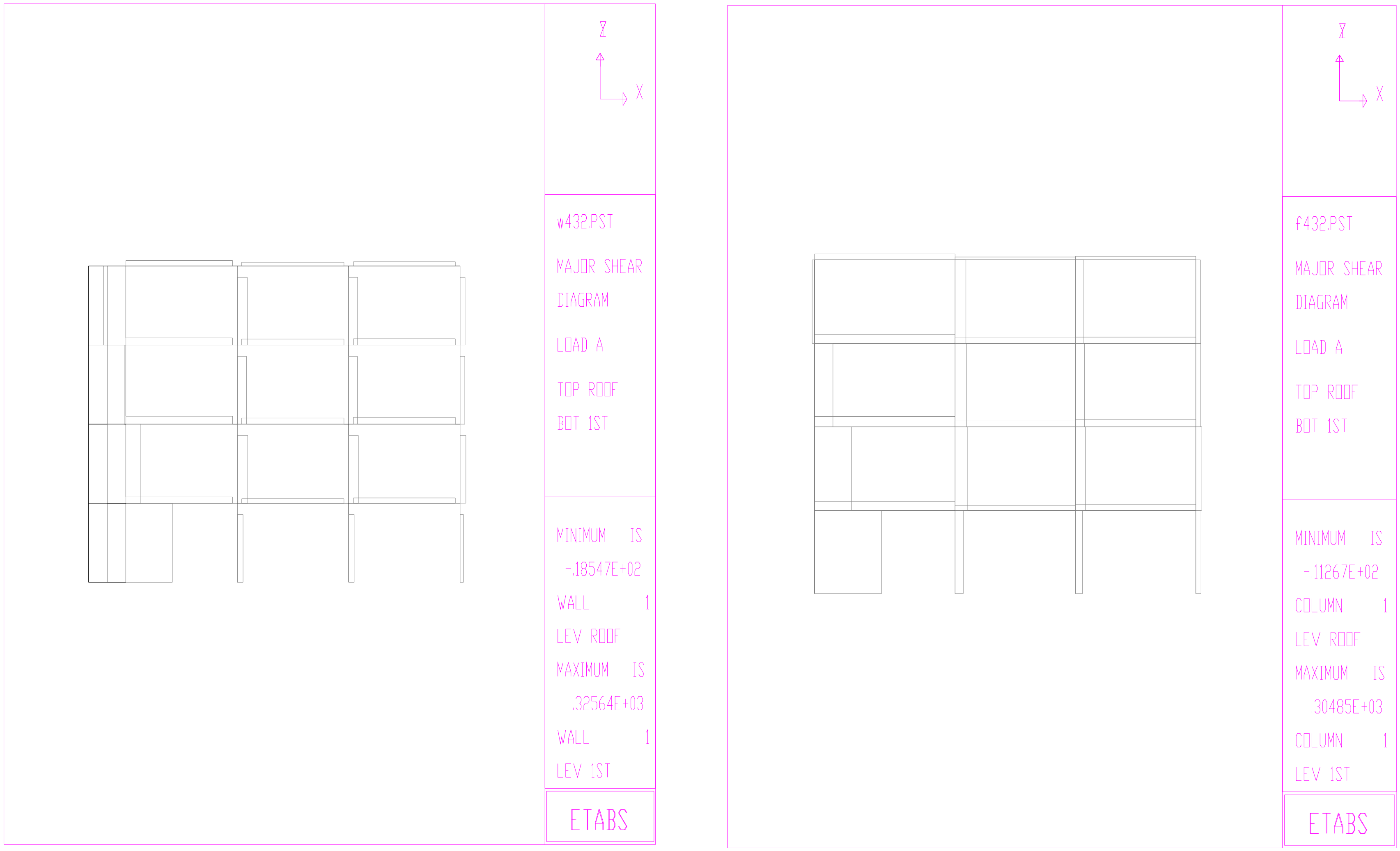
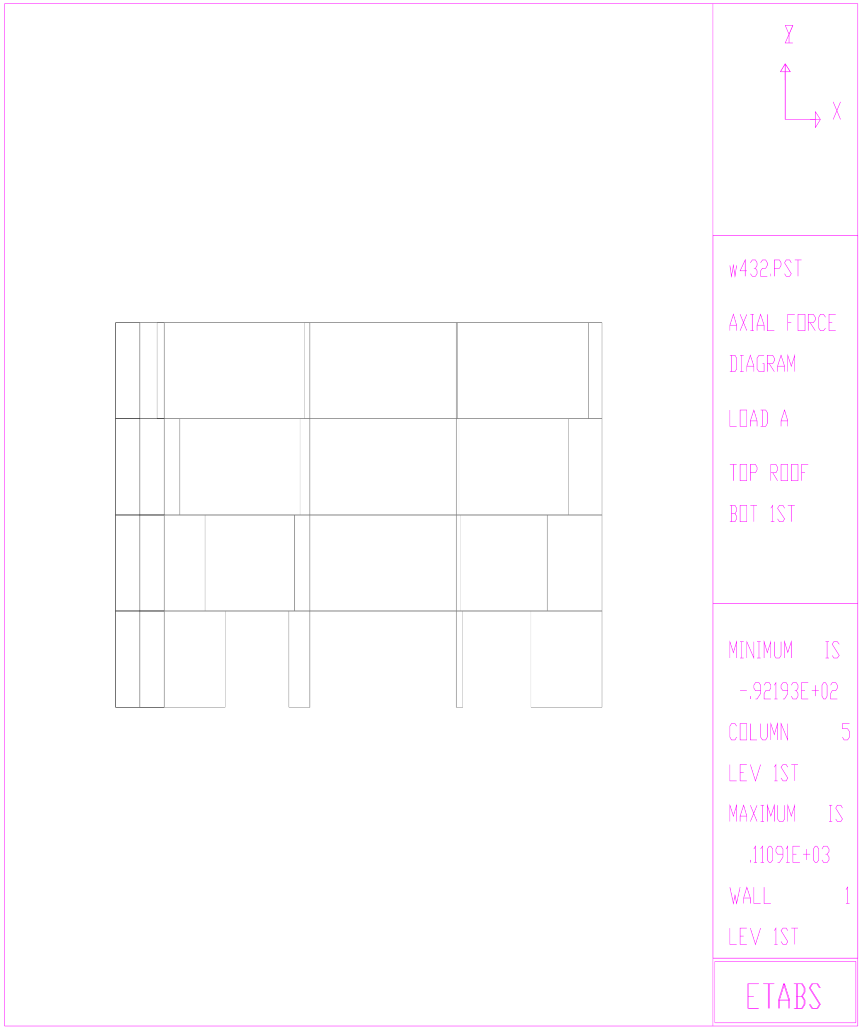
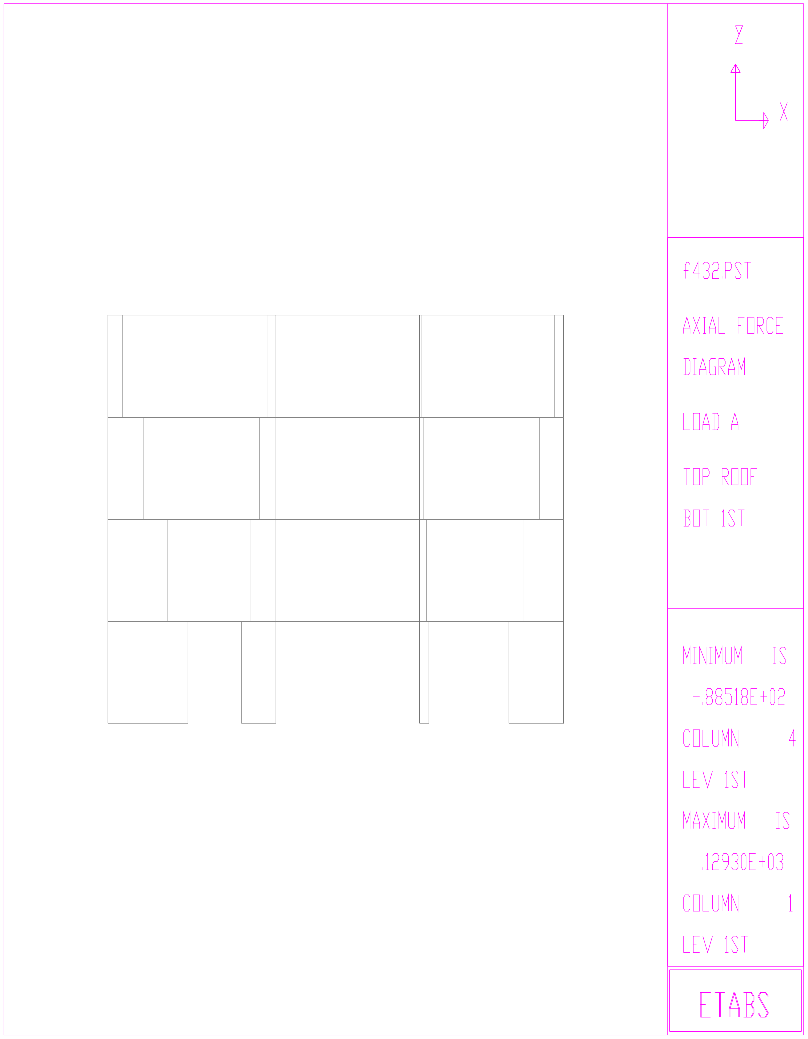


Fig. 6.2. (a) Bending moment diagram of wall *(W432)* (left) and equivalent frame *(F432)* (right), respectively



(b) Shear force diagram of wall *(W432)* (left) and equivalent frame *(F432)* (right), respectively





(c) Axial force diagram of wall *(W432)* (left) and equivalent frame *(F432)* (right), respectively

Table 6.3. Comparison of results for W432 and F432

	Level	Dual system (W432)				Equivalent frame (F432)				Difference in %			
		Bending moment		Shear force	Axial force	Bending moment		Shear force	Axial force	Bending moment		Shear force	Axial force
		Top	Bot.			Top	Bot.			Top	Bot.		
Shear wall	Roof	-93.74	-159	-18.6	22.43	-99.98	-139	-11.27	23.75	-6.66	12.13	39.25	-5.88
	3rd	-278	23.5	86.12	52.06	-271.6	20.7	83.52	58.07	2.30	11.56	3.019	-11.5
	2nd	-109	480	168.2	84.68	-129.6	461	168.66	96.77	-18.9	3.96	-0.28	-14.3
	1st	371.98	1512	325.6	110.91	334.07	1401	304.85	129.3	10.19	7.32	6.384	-16.6

```
                                                   PROGRAM:ETABS/FILE:w432.FRM
UNITS: KN-M

COLUMN FORCES AT LEVEL ROOF     IN FRAME

 COL OUTPUT OUTPUT      MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT     MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   3 CASE 1    TOP     -82.47     49.68       .00       .00     -7.44       .00
            BOTTOM      66.57                 .00

   4 CASE 1    TOP     -73.41     44.43       .00       .00      2.46       .00
            BOTTOM      59.89                 .00

   5 CASE 1    TOP     -42.82     24.43       .00       .00    -17.46       .00
            BOTTOM      30.48                 .00

BEAM FORCES AT LEVEL ROOF     IN FRAME

 BAY OUTPUT OUTPUT      MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT     MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   2 CASE 1  END-I      71.31     22.43       .00       .00       .00       .00
             END-J     -57.67                 .00

   3 CASE 1  END-I      40.29     14.99       .00       .00       .00       .00
             END-J     -42.17                 .00

   4 CASE 1  END-I      45.34     17.46       .00       .00       .00       .00
             END-J     -50.67                 .00

WALL FORCES AT LEVEL ROOF     IN FRAME

WALL OUTPUT OUTPUT      MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT     MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   1 CASE 1    TOP     -93.74    -18.55       .00       .00     22.43       .00
            BOTTOM    -158.66                 .00


COLUMN FORCES AT LEVEL 3RD      IN FRAME

 COL OUTPUT OUTPUT      MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT     MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   3 CASE 1    TOP     -70.66     46.69       .00       .00    -12.80       .00
            BOTTOM      69.40                 .00

   4 CASE 1    TOP     -66.37     43.52       .00       .00      3.95       .00
            BOTTOM      64.19                 .00

   5 CASE 1    TOP     -37.30     23.67       .00       .00    -43.21       .00
            BOTTOM      33.71                 .00

BEAM FORCES AT LEVEL 3RD      IN FRAME

 BAY OUTPUT OUTPUT      MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT     MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   2 CASE 1  END-I      89.69     29.63       .00       .00       .00       .00
             END-J     -80.66                 .00
```

3	CASE 1	END-I	66.45	24.27	.00	.00	.00	.00
		END-J	-67.03		.00			
4	CASE 1	END-I	68.49	25.76	.00	.00	.00	.00
		END-J	-73.17		.00			

WALL FORCES AT LEVEL 3RD IN FRAME

WALL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	-277.98	86.12	.00	.00	52.06	.00
		BOTTOM	23.45		.00			

COLUMN FORCES AT LEVEL 2ND IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
3	CASE 1	TOP	-75.08	53.51	.00	.00	-20.15	.00
		BOTTOM	85.44		.00			
4	CASE 1	TOP	-68.44	49.01	.00	.00	6.51	.00
		BOTTOM	78.58		.00			
5	CASE 1	TOP	-37.86	29.30	.00	.00	-71.04	.00
		BOTTOM	50.04		.00			

BEAM FORCES AT LEVEL 2ND IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
2	CASE 1	END-I	99.79	32.62	.00	.00	.00	.00
		END-J	-87.80		.00			
3	CASE 1	END-I	68.96	25.27	.00	.00	.00	.00
		END-J	-70.05		.00			
4	CASE 1	END-I	73.81	27.83	.00	.00	.00	.00
		END-J	-79.26		.00			

WALL FORCES AT LEVEL 2ND IN FRAME

WALL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	-108.97	168.19	.00	.00	84.68	.00
		BOTTOM	479.69		.00			

COLUMN FORCES AT LEVEL 1ST IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
3	CASE 1	TOP	-30.82	29.88	.00	.00	-27.74	.00
		BOTTOM	58.81		.00			
4	CASE 1	TOP	-25.63	27.15	.00	.00	9.02	.00
		BOTTOM	55.83		.00			
5	CASE 1	TOP	-6.92	17.33	.00	.00	-92.19	.00
		BOTTOM	45.07		.00			

BEAM FORCES AT LEVEL 1ST IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
2	CASE 1	END-I	81.47	26.23	.00	.00	.00	.00
		END-J	-69.34		.00			
3	CASE 1	END-I	50.65	18.64	.00	.00	.00	.00
		END-J	-51.85		.00			
4	CASE 1	END-I	55.99	21.15	.00	.00	.00	.00
		END-J	-60.33		.00			

WALL FORCES AT LEVEL 1ST IN FRAME

WALL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	371.98	325.64	.00	.00	110.91	.00
		BOTTOM	1511.72		.00			

UNITS: KN-M

COLUMN FORCES AT LEVEL ROOF IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	-99.98	-11.27	.00	.00	23.75	.00
		BOTTOM	-139.41		.00			
2	CASE 1	TOP	-96.97	50.03	.00	.00	-12.72	.00
		BOTTOM	78.13		.00			
3	CASE 1	TOP	-76.37	38.94	.00	.00	3.64	.00
		BOTTOM	59.93		.00			
4	CASE 1	TOP	-47.05	22.30	.00	.00	-14.67	.00
		BOTTOM	30.98		.00			

BEAM FORCES AT LEVEL ROOF IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	END-I	99.98	23.75	.00	.00	.00	.00
		END-J	-66.25		.00			
2	CASE 1	END-I	30.72	11.02	.00	.00	.00	.00
		END-J	-35.42		.00			
3	CASE 1	END-I	40.95	14.67	.00	.00	.00	.00
		END-J	-47.05		.00			

COLUMN FORCES AT LEVEL 3RD IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	-271.58	83.52	.00	.00	58.07	.00
		BOTTOM	20.74		.00			
2	CASE 1	TOP	-91.50	51.65	.00	.00	-26.00	.00
		BOTTOM	89.27		.00			
3	CASE 1	TOP	-74.01	41.18	.00	.00	6.63	.00
		BOTTOM	70.12		.00			
4	CASE 1	TOP	-44.06	23.65	.00	.00	-38.70	.00
		BOTTOM	38.73		.00			

BEAM FORCES AT LEVEL 3RD IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	END-I	132.17	34.33	.00	.00	.00	.00
		END-J	-108.13		.00			
2	CASE 1	END-I	61.50	21.05	.00	.00	.00	.00
		END-J	-64.79		.00			
3	CASE 1	END-I	69.15	24.03	.00	.00	.00	.00
		END-J	-75.04		.00			

COLUMN FORCES AT LEVEL 2ND IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	-129.60	168.66	.00	.00	96.77	.00
		BOTTOM	460.71		.00			
2	CASE 1	TOP	-97.11	57.24	.00	.00	-42.03	.00
		BOTTOM	103.22		.00			
3	CASE 1	TOP	-77.18	46.08	.00	.00	10.81	.00
		BOTTOM	84.10		.00			
4	CASE 1	TOP	-45.19	28.02	.00	.00	-65.55	.00

		BOTTOM	52.90		.00			

BEAM FORCES AT LEVEL 2ND IN FRAME

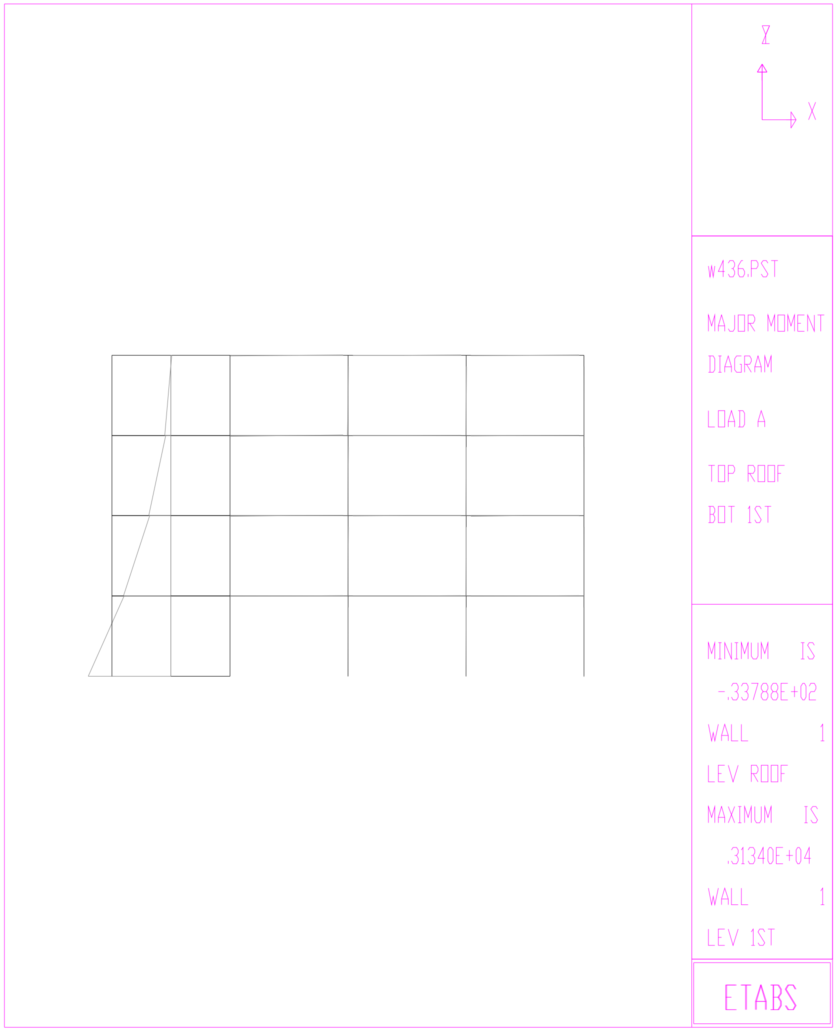
BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	END-I	150.34	38.69	.00	.00	.00	.00
		END-J	-120.50		.00			
2	CASE 1	END-I	65.88	22.67	.00	.00	.00	.00
		END-J	-70.13		.00			
3	CASE 1	END-I	77.18	26.85	.00	.00	.00	.00
		END-J	-83.91		.00			

COLUMN FORCES AT LEVEL 1ST IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	334.07	304.85	.00	.00	129.30	.00
		BOTTOM	1401.03		.00			
2	CASE 1	TOP	-52.21	38.12	.00	.00	-55.83	.00
		BOTTOM	81.22		.00			
3	CASE 1	TOP	-39.77	32.87	.00	.00	15.05	.00
		BOTTOM	75.27		.00			
4	CASE 1	TOP	-19.15	24.16	.00	.00	-88.52	.00
		BOTTOM	65.42		.00			

BEAM FORCES AT LEVEL 1ST IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	END-I	126.65	32.54	.00	.00	.00	.00
		END-J	-101.12		.00			
2	CASE 1	END-I	54.30	18.73	.00	.00	.00	.00
		END-J	-58.09		.00			
3	CASE 1	END-I	65.77	22.97	.00	.00	.00	.00
		END-J	-72.05		.00			



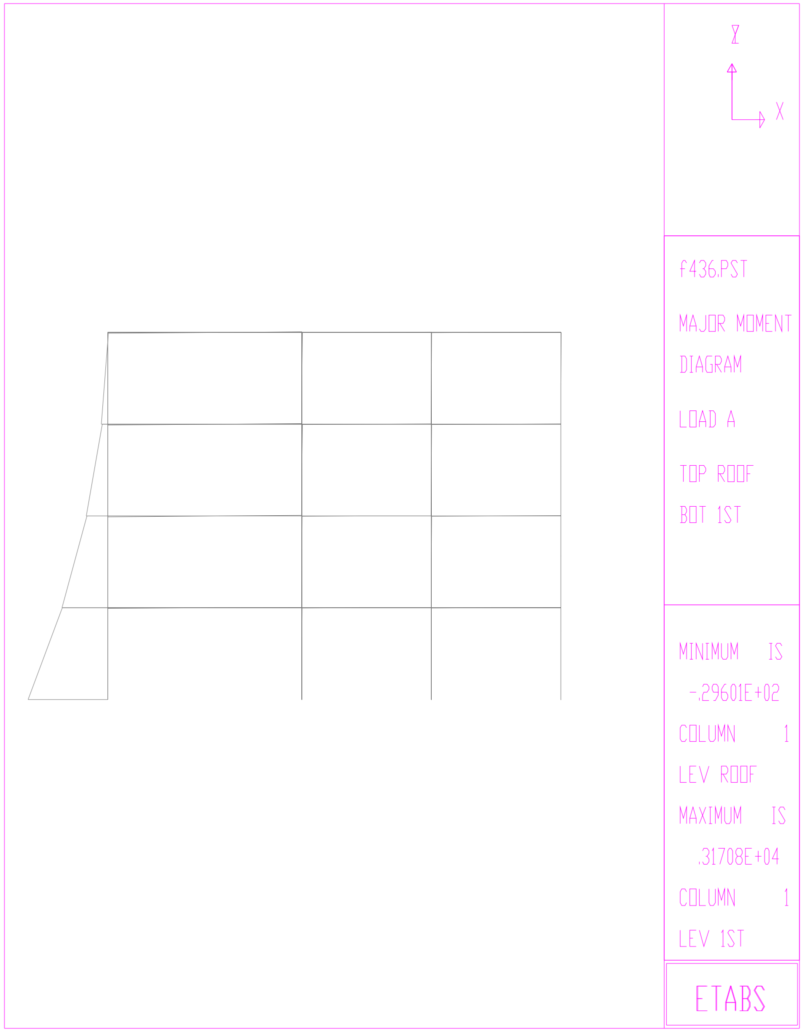
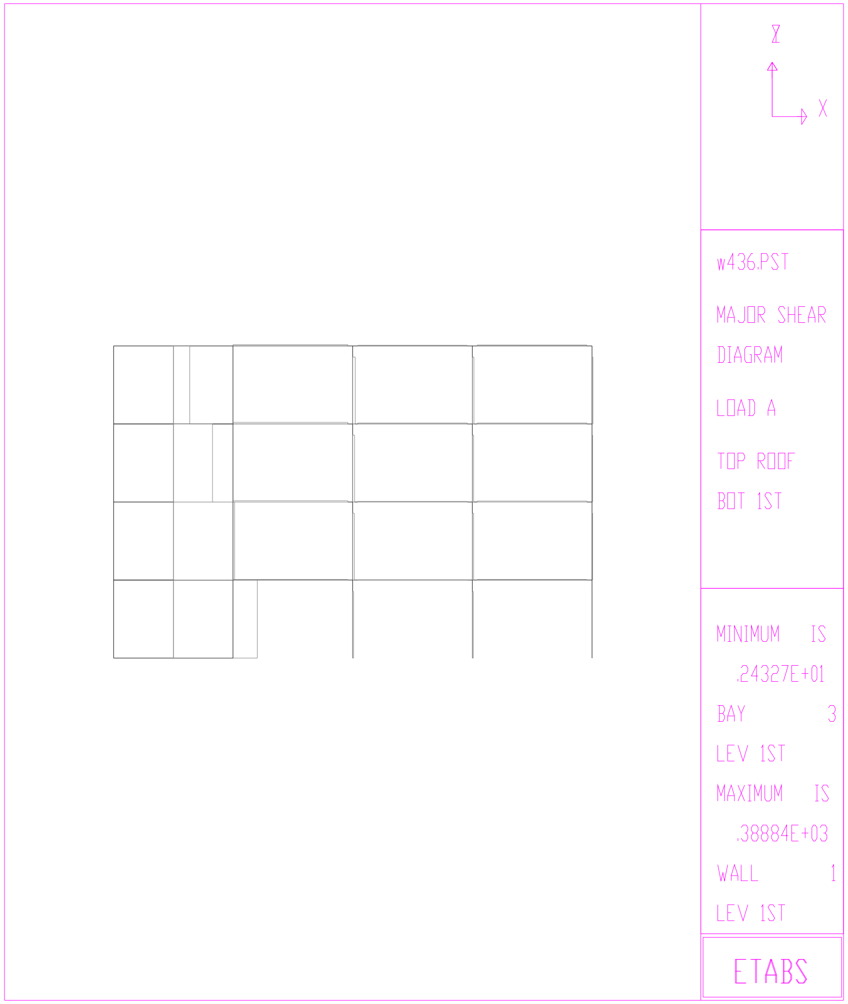
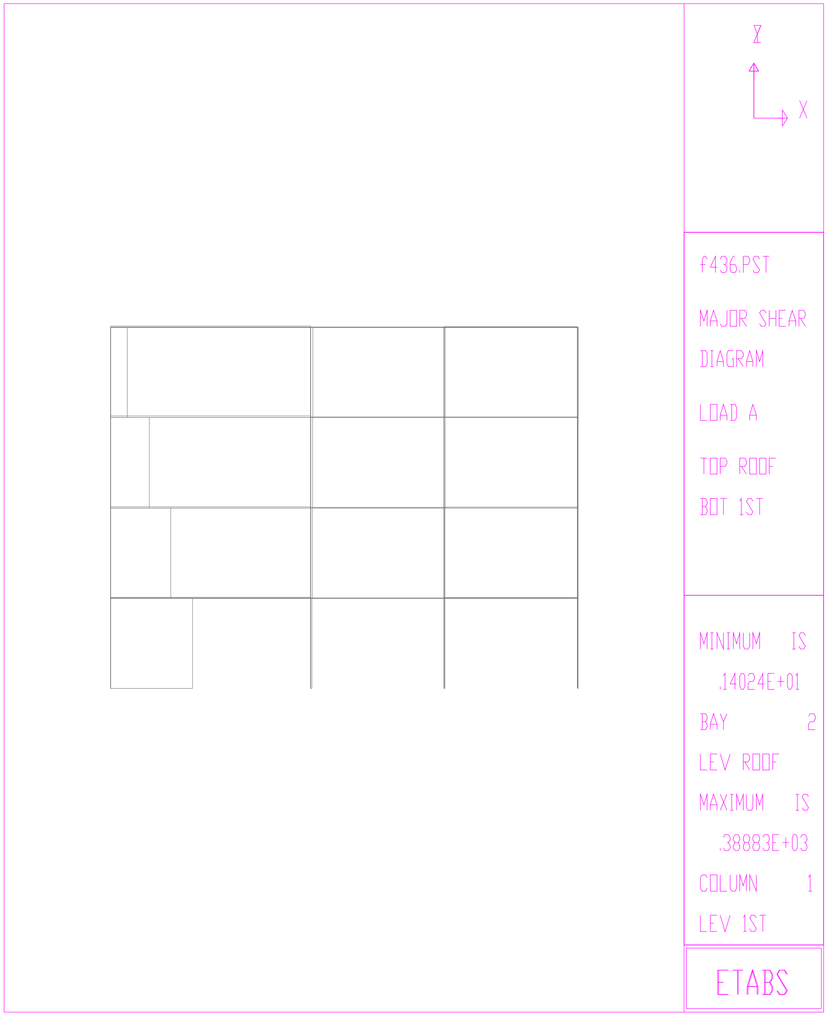
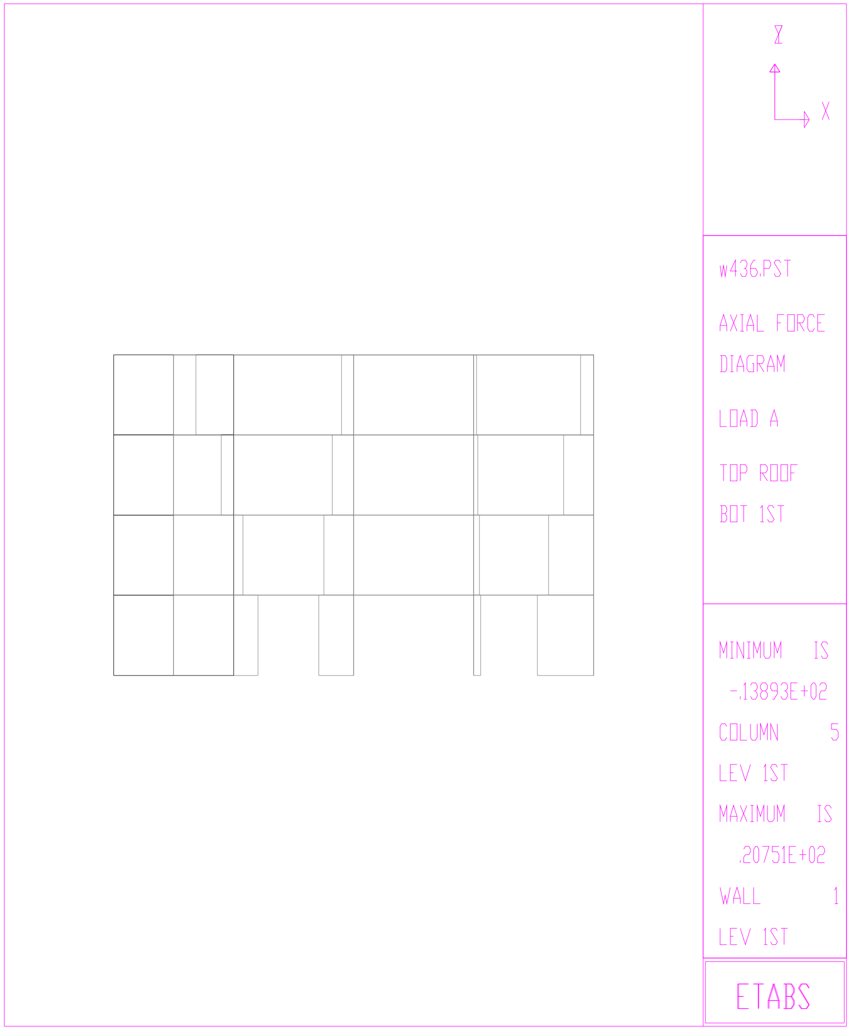


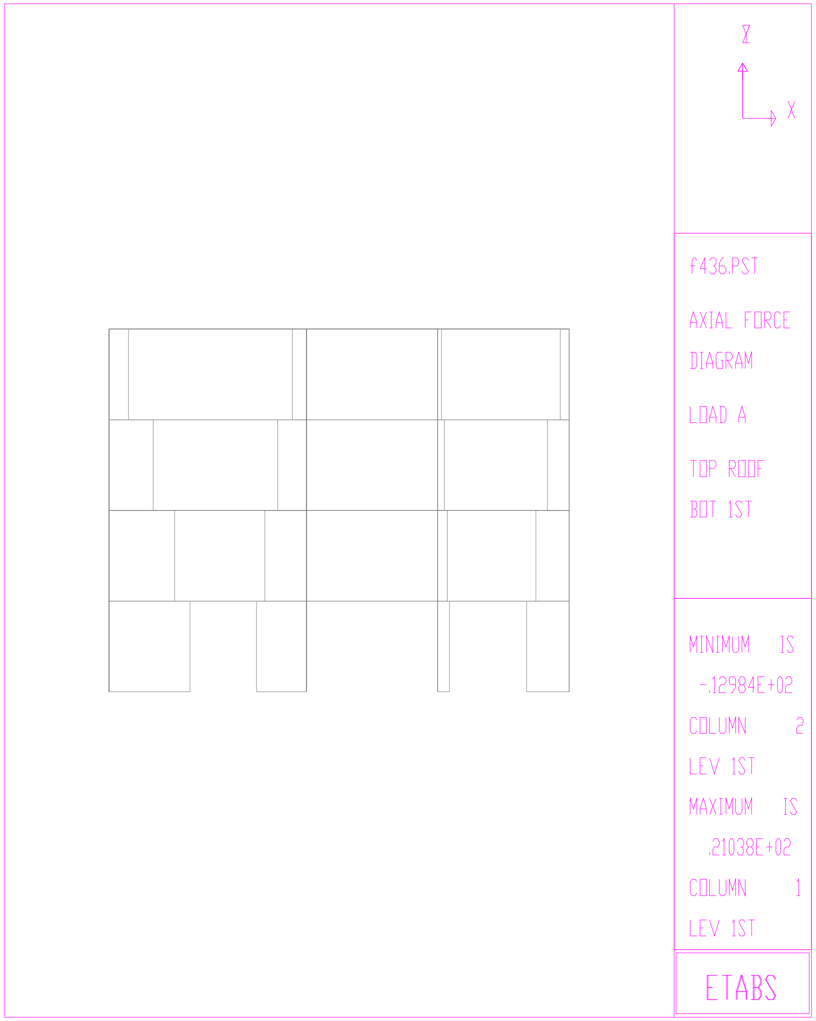
Fig. 6.3. (a) Bending moment diagram of wall *(W436)* (left) and equivalent frame *(F436)* (right), respectively





(b) Shear force diagram of wall *(W436)* (left) and equivalent frame *(F436)* (right), respectively





(c) Axial force diagram of wall *(W436)* (left) and equivalent frame *(F436)* (right), respectively

Table 6.4. Comparison of results for W436 and F436

	Level	Dual system (W436)				Equivalent frame (F436)				Difference in %			
		Bending moment		Shear force	Axial force	Bending moment		Shear force	Axial force	Bending moment		Shear force	Axial force
		Top	Bot.			Top	Bot.			Top	Bot.		
Shear wall	Roof	-33.79	233	76.33	5.49	-29.6	250	79.82	5.05	12.40	-7.02	-4.57	8.015
	3rd	196.23	834	182.1	11.69	216.53	859	183.62	11.45	-10.3	-3.08	-0.84	2.053
	2nd	801.14	1795	284	17.08	829.95	1830	285.66	17.09	-3.60	-1.94	-0.6	-0.06
	1st	1773	3134	388.8	20.75	1809.9	3171	388.83	21.04	-2.08	-1.17	0.003	-1.4

```
                                                PROGRAM:ETABS/FILE:w436.FRM
 UNITS: KN-M

COLUMN FORCES AT LEVEL ROOF      IN FRAME

 COL OUTPUT OUTPUT      MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT     MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   3 CASE 1    TOP     -17.44     10.73       .00       .00     -2.97       .00
            BOTTOM      14.76                 .00

   4 CASE 1    TOP     -13.13      8.20       .00       .00       .70       .00
            BOTTOM      11.47                 .00

   5 CASE 1    TOP      -7.79      4.73       .00       .00     -3.23       .00
            BOTTOM       6.41                 .00

 BEAM FORCES AT LEVEL ROOF      IN FRAME

 BAY OUTPUT OUTPUT      MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT     MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   2 CASE 1  END-I      17.30      5.49       .00       .00       .00       .00
             END-J     -14.29                 .00

   3 CASE 1  END-I       6.51      2.53       .00       .00       .00       .00
             END-J      -7.39                 .00

   4 CASE 1  END-I       8.40      3.23       .00       .00       .00       .00
             END-J      -9.35                 .00

 WALL FORCES AT LEVEL ROOF      IN FRAME

WALL OUTPUT OUTPUT      MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT     MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   1 CASE 1    TOP     -33.79     76.33       .00       .00      5.49       .00
            BOTTOM     233.38                 .00

 COLUMN FORCES AT LEVEL 3RD       IN FRAME

 COL OUTPUT OUTPUT      MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT     MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   3 CASE 1    TOP     -11.37      7.89       .00       .00     -5.28       .00
            BOTTOM      12.30                 .00

   4 CASE 1    TOP      -9.33      6.48       .00       .00      1.02       .00
            BOTTOM      10.11                 .00

   5 CASE 1    TOP      -4.83      3.54       .00       .00     -7.43       .00
            BOTTOM       5.80                 .00

 BEAM FORCES AT LEVEL 3RD       IN FRAME

 BAY OUTPUT OUTPUT      MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT     MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   2 CASE 1  END-I      18.57      6.19       .00       .00       .00       .00
             END-J     -17.04                 .00

   3 CASE 1  END-I      10.52      3.88       .00       .00       .00       .00
```

		END-J	-10.84		.00			
4	CASE 1	END-I	11.18	4.21	.00	.00	.00	.00
		END-J	-11.96		.00			

WALL FORCES AT LEVEL 3RD IN FRAME

WALL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	196.23	182.09	.00	.00	11.69	.00
		BOTTOM	833.54		.00			

COLUMN FORCES AT LEVEL 2ND IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
3	CASE 1	TOP	-10.00	7.04	.00	.00	-7.36	.00
		BOTTOM	11.14		.00			
4	CASE 1	TOP	-7.90	5.76	.00	.00	1.41	.00
		BOTTOM	9.39		.00			
5	CASE 1	TOP	-4.02	3.24	.00	.00	-11.13	.00
		BOTTOM	5.70		.00			

BEAM FORCES AT LEVEL 2ND IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
2	CASE 1	END-I	16.23	5.39	.00	.00	.00	.00
		END-J	-14.75		.00			
3	CASE 1	END-I	8.90	3.31	.00	.00	.00	.00
		END-J	-9.30		.00			
4	CASE 1	END-I	9.83	3.70	.00	.00	.00	.00
		END-J	-10.51		.00			

WALL FORCES AT LEVEL 2ND IN FRAME

WALL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	801.14	283.95	.00	.00	17.08	.00
		BOTTOM	1794.97		.00			

COLUMN FORCES AT LEVEL 1ST IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
3	CASE 1	TOP	-4.94	4.48	.00	.00	-8.60	.00
		BOTTOM	8.50		.00			
4	CASE 1	TOP	-4.01	3.99	.00	.00	1.74	.00
		BOTTOM	7.96		.00			
5	CASE 1	TOP	-1.52	2.69	.00	.00	-13.89	.00
		BOTTOM	6.53		.00			

BEAM FORCES AT LEVEL 1ST IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
2	CASE 1	END-I	10.92	3.67	.00	.00	.00	.00
		END-J	-10.21		.00			
3	CASE 1	END-I	6.58	2.43	.00	.00	.00	.00
		END-J	-6.80		.00			
4	CASE 1	END-I	7.30	2.76	.00	.00	.00	.00
		END-J	-7.88		.00			

WALL FORCES AT LEVEL 1ST IN FRAME

WALL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	1773.03	388.84	.00	.00	20.75	.00
		BOTTOM	3133.98		.00			

UNITS: KN-M

COLUMN FORCES AT LEVEL ROOF IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	-29.60	79.82	.00	.00	5.05	.00
		BOTTOM	249.77		.00			
2	CASE 1	TOP	-19.20	10.28	.00	.00	-3.64	.00
		BOTTOM	16.80		.00			
3	CASE 1	TOP	-11.62	6.11	.00	.00	.94	.00
		BOTTOM	9.77		.00			
4	CASE 1	TOP	-7.46	3.78	.00	.00	-2.34	.00
		BOTTOM	5.78		.00			

BEAM FORCES AT LEVEL ROOF IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	END-I	29.60	5.05	.00	.00	.00	.00
		END-J	-15.82		.00			
2	CASE 1	END-I	3.37	1.40	.00	.00	.00	.00
		END-J	-5.04		.00			
3	CASE 1	END-I	6.58	2.34	.00	.00	.00	.00
		END-J	-7.46		.00			

COLUMN FORCES AT LEVEL 3RD IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	216.53	183.62	.00	.00	11.45	.00
		BOTTOM	859.19		.00			
2	CASE 1	TOP	-14.72	8.52	.00	.00	-7.45	.00
		BOTTOM	15.11		.00			
3	CASE 1	TOP	-8.38	4.96	.00	.00	1.70	.00
		BOTTOM	8.97		.00			
4	CASE 1	TOP	-4.70	2.90	.00	.00	-5.70	.00
		BOTTOM	5.45		.00			

BEAM FORCES AT LEVEL 3RD IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	END-I	33.25	6.40	.00	.00	.00	.00
		END-J	-24.40		.00			
2	CASE 1	END-I	7.12	2.60	.00	.00	.00	.00
		END-J	-8.47		.00			
3	CASE 1	END-I	9.69	3.36	.00	.00	.00	.00
		END-J	-10.48		.00			

COLUMN FORCES AT LEVEL 2ND IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	829.95	285.66	.00	.00	17.09	.00
		BOTTOM	1829.77		.00			
2	CASE 1	TOP	-12.61	7.35	.00	.00	-10.80	.00
		BOTTOM	13.14		.00			
3	CASE 1	TOP	-7.16	4.40	.00	.00	2.42	.00
		BOTTOM	8.24		.00			

4	CASE 1	TOP	-3.91	2.58	.00	.00	-8.71	.00
		BOTTOM	5.12		.00			

BEAM FORCES AT LEVEL 2ND IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	END-I	29.24	5.64	.00	.00	.00	.00
		END-J	-21.48		.00			
2	CASE 1	END-I	6.24	2.28	.00	.00	.00	.00
		END-J	-7.46		.00			
3	CASE 1	END-I	8.67	3.01	.00	.00	.00	.00
		END-J	-9.37		.00			

COLUMN FORCES AT LEVEL 1ST IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	1809.86	388.83	.00	.00	21.04	.00
		BOTTOM	3170.76		.00			
2	CASE 1	TOP	-7.38	4.85	.00	.00	-12.98	.00
		BOTTOM	9.59		.00			
3	CASE 1	TOP	-4.41	3.59	.00	.00	3.05	.00
		BOTTOM	8.17		.00			
4	CASE 1	TOP	-2.38	2.73	.00	.00	-11.10	.00
		BOTTOM	7.19		.00			

BEAM FORCES AT LEVEL 1ST IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	END-I	19.91	3.95	.00	.00	.00	.00
		END-J	-15.65		.00			
2	CASE 1	END-I	4.86	1.77	.00	.00	.00	.00
		END-J	-5.76		.00			
3	CASE 1	END-I	6.88	2.40	.00	.00	.00	.00
		END-J	-7.50		.00			

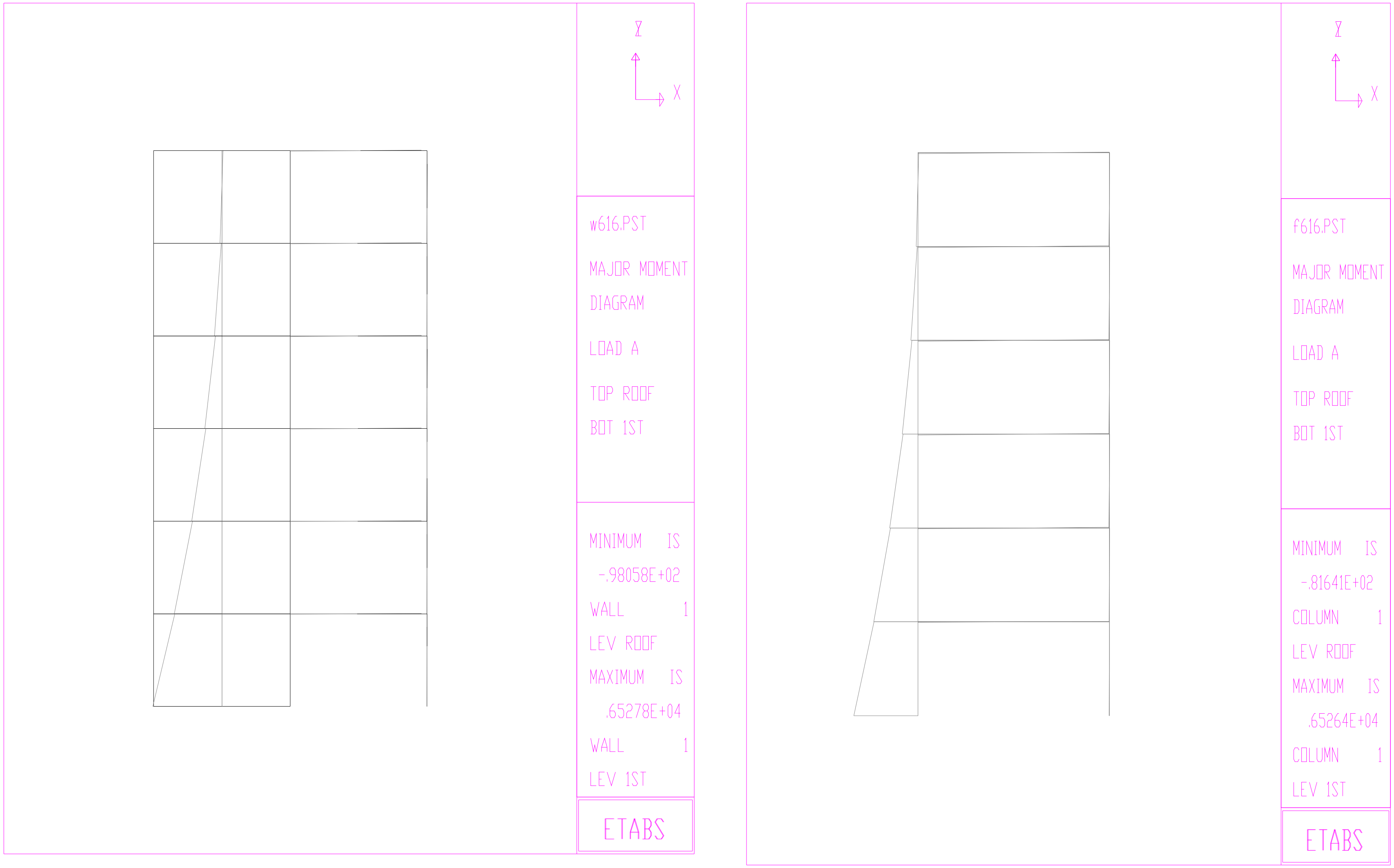
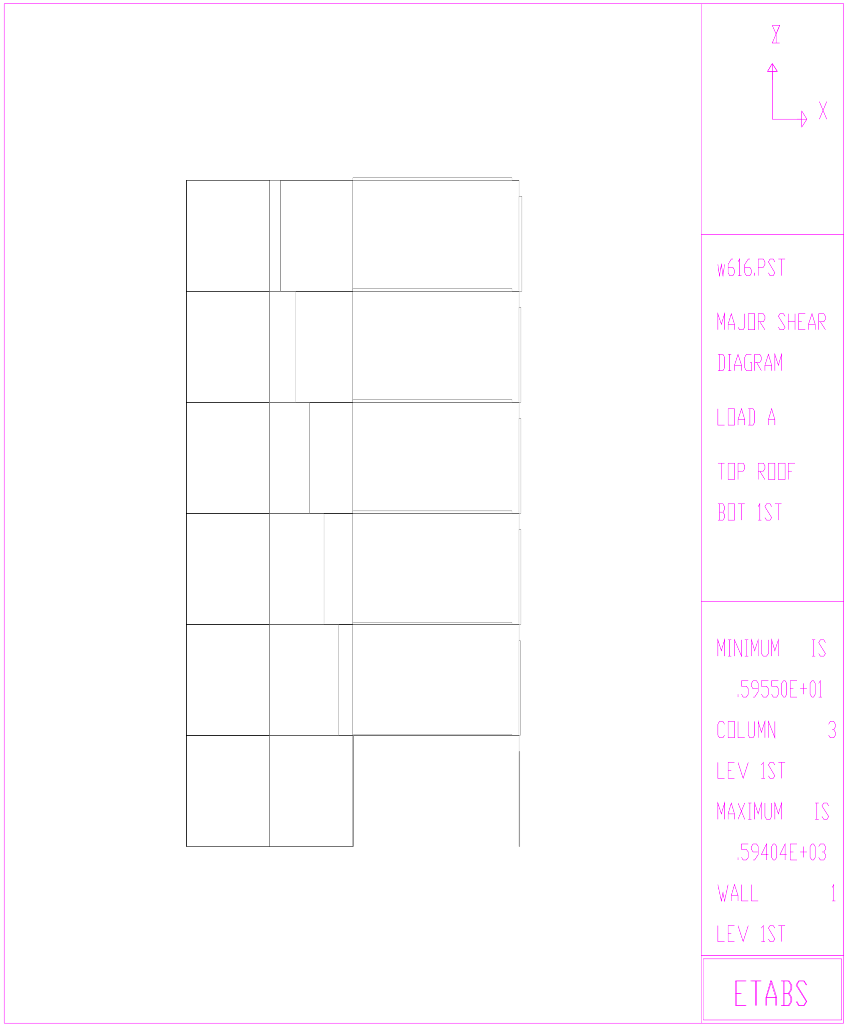
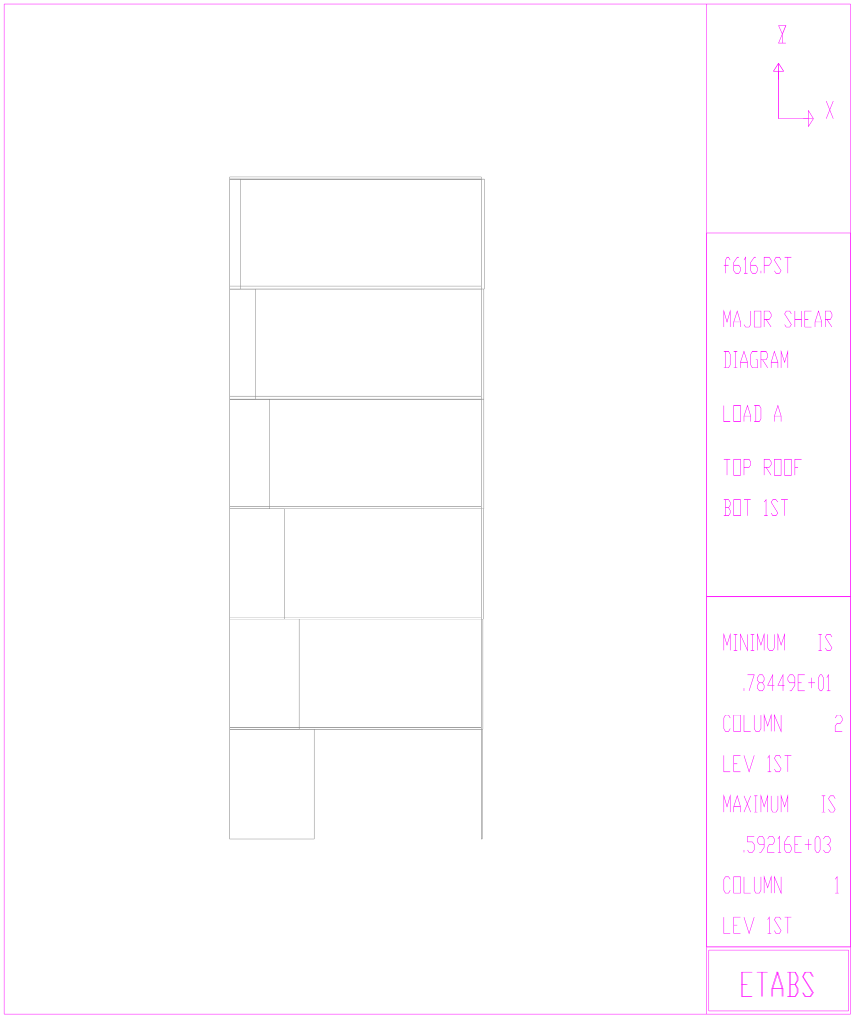
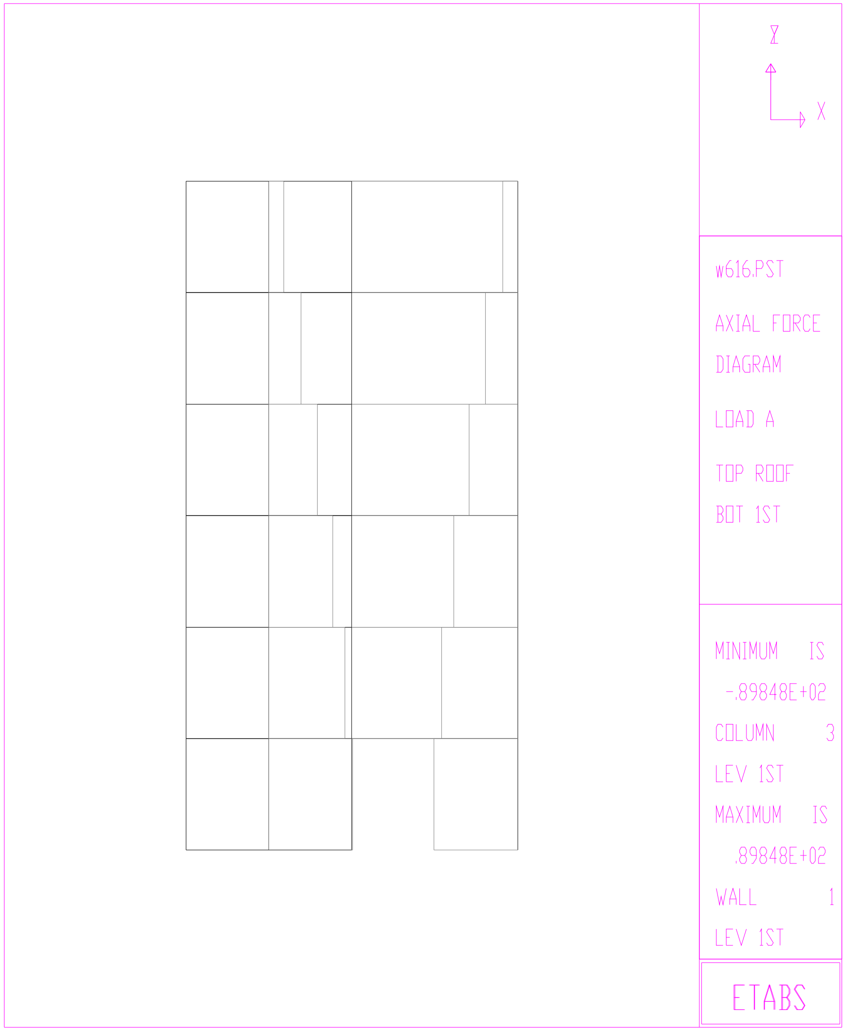


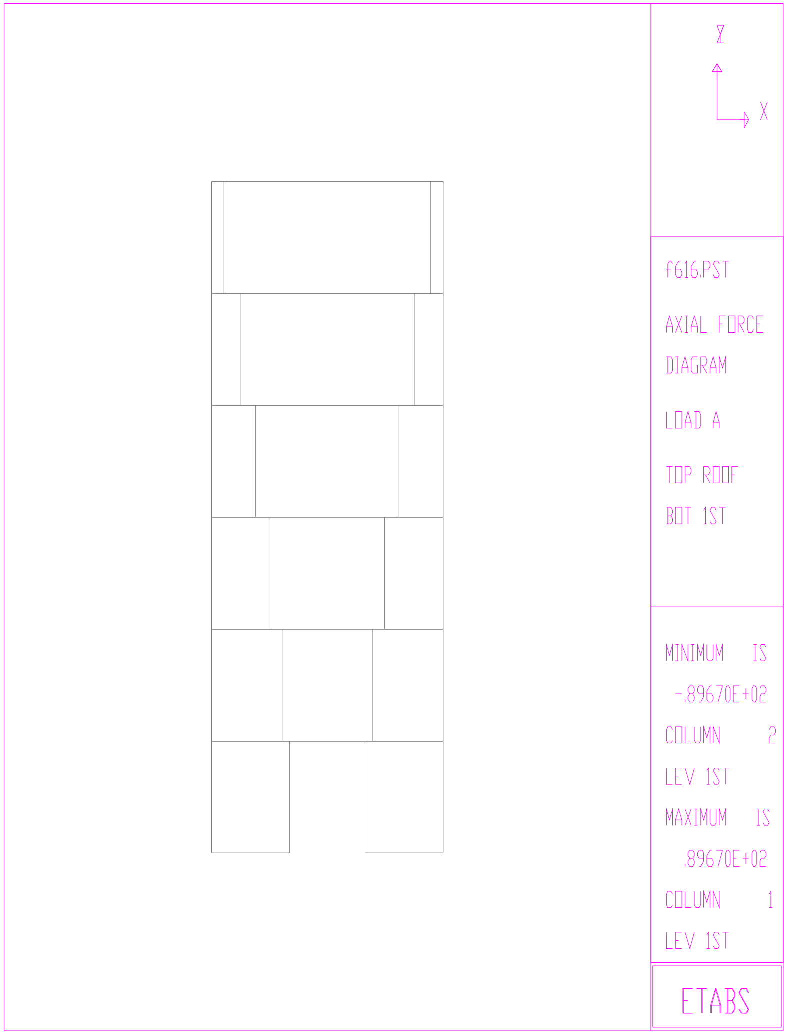
Fig. 6.4. (a) Bending moment diagram of wall *(W616)* (left) and equivalent frame *(F616)* (right), respectively





(b) Shear force diagram of wall *(W616)* (left) and equivalent frame *(F616)* (right), respectively





(c) Axial force diagram of wall *(W616)* (left) and equivalent frame *(F616)* (right), respectively

Table 6.5. Comparison of results for W616 and F616

	Level	Dual system (W616)				Equivalent frame (F616)				Difference in %			
		Bending moment		Shear force	Axial force	Bending moment		Shear force	Axial force	Bending moment		Shear force	Axial force
		Top	Bot.			Top	Bot.			Top	Bot.		
Shear wall	Roof	-98.06	176	78.34	16.13	-81.64	182	75.43	14.25	16.74	-3.52	3.715	11.66
	5th	66.64	717	185.7	34.54	87.78	719	180.26	32.92	-31.7	-0.29	2.929	4.69
	4th	611.91	1607	284.5	52.07	627.68	1609	280.29	50.72	-2.58	-0.08	1.462	2.593
	3rd	1510.5	2863	386.4	68.34	1524.6	2863	382.39	67.26	-0.93	0.00	1.035	1.58
	2nd	2783.9	4498	489.6	81.6	2794.1	4497	486.57	80.91	-0.37	0.01	0.625	0.846
	1st	4448.7	6528	594	89.85	4453.9	6526	592.16	89.67	-0.12	0.02	0.316	0.2

```
                                          PROGRAM:ETABS/FILE:w616.FRM
UNITS: KN-M

COLUMN FORCES AT LEVEL ROOF     IN FRAME

 COL OUTPUT OUTPUT     MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT    MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   3 CASE 1    TOP    -36.28     21.66       .00       .00    -16.13       .00
            BOTTOM     28.69                 .00

BEAM FORCES AT LEVEL ROOF     IN FRAME

 BAY OUTPUT OUTPUT     MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT    MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   2 CASE 1  END-I     49.67     16.13       .00       .00       .00       .00
             END-J    -43.08                 .00

WALL FORCES AT LEVEL ROOF     IN FRAME

WALL OUTPUT OUTPUT     MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT    MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   1 CASE 1    TOP    -98.06     78.34       .00       .00     16.13       .00
            BOTTOM    176.15                 .00

COLUMN FORCES AT LEVEL 5TH      IN FRAME

 COL OUTPUT OUTPUT     MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT    MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   3 CASE 1    TOP    -20.37     14.30       .00       .00    -34.54       .00
            BOTTOM     22.52                 .00

BEAM FORCES AT LEVEL 5TH      IN FRAME

 BAY OUTPUT OUTPUT     MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT    MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   2 CASE 1  END-I     54.27     18.41       .00       .00       .00       .00
             END-J    -51.60                 .00

WALL FORCES AT LEVEL 5TH      IN FRAME

WALL OUTPUT OUTPUT     MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT    MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   1 CASE 1    TOP     66.64    185.70       .00       .00     34.54       .00
            BOTTOM    716.60                 .00

COLUMN FORCES AT LEVEL 4TH      IN FRAME

 COL OUTPUT OUTPUT     MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
  ID     ID  POINT    MOMENT     SHEAR    MOMENT     SHEAR     FORCE    MOMENT
   3 CASE 1    TOP    -22.79     15.55       .00       .00    -52.07       .00
            BOTTOM     23.86                 .00

BEAM FORCES AT LEVEL 4TH      IN FRAME

 BAY OUTPUT OUTPUT     MAJOR     MAJOR     MINOR     MINOR     AXIAL TORSIONAL
```

ID	ID	POINT	MOMENT	SHEAR	MOMENT	SHEAR	FORCE	MOMENT
2	CASE 1	END-I	52.10	17.53	.00	.00	.00	.00
		END-J	-48.71		.00			

WALL FORCES AT LEVEL 4TH IN FRAME

WALL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	611.91	284.45	.00	.00	52.07	.00
		BOTTOM	1607.48		.00			

COLUMN FORCES AT LEVEL 3RD IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
3	CASE 1	TOP	-18.77	13.61	.00	.00	-68.34	.00
		BOTTOM	22.05		.00			

BEAM FORCES AT LEVEL 3RD IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
2	CASE 1	END-I	48.15	16.27	.00	.00	.00	.00
		END-J	-45.37		.00			

WALL FORCES AT LEVEL 3RD IN FRAME

WALL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	1510.53	386.39	.00	.00	68.34	.00
		BOTTOM	2862.90		.00			

COLUMN FORCES AT LEVEL 2ND IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
3	CASE 1	TOP	-13.11	10.37	.00	.00	-81.60	.00
		BOTTOM	18.02		.00			

BEAM FORCES AT LEVEL 2ND IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
2	CASE 1	END-I	39.22	13.26	.00	.00	.00	.00
		END-J	-37.03		.00			

WALL FORCES AT LEVEL 2ND IN FRAME

WALL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	2783.90	489.63	.00	.00	81.60	.00
		BOTTOM	4497.59		.00			

COLUMN FORCES AT LEVEL 1ST IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
3	CASE 1	TOP	-4.32	5.96	.00	.00	-89.85	.00
		BOTTOM	13.55		.00			

BEAM FORCES AT LEVEL 1ST IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
2	CASE 1	END-I	24.18	8.25	.00	.00	.00	.00
		END-J	-23.25		.00			

WALL FORCES AT LEVEL 1ST IN FRAME

WALL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	4448.66	594.04	.00	.00	89.85	.00
		BOTTOM	6527.82		.00			

UNITS: KN-M

COLUMN FORCES AT LEVEL ROOF IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	-81.64	75.43	.00	.00	14.25	.00
		BOTTOM	182.35		.00			
2	CASE 1	TOP	-46.62	24.57	.00	.00	-14.25	.00
		BOTTOM	39.39		.00			

BEAM FORCES AT LEVEL ROOF IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	END-I	81.64	14.25	.00	.00	.00	.00
		END-J	-46.62		.00			

COLUMN FORCES AT LEVEL 5TH IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	87.78	180.26	.00	.00	32.92	.00
		BOTTOM	718.70		.00			
2	CASE 1	TOP	-34.02	19.74	.00	.00	-32.92	.00
		BOTTOM	35.07		.00			

BEAM FORCES AT LEVEL 5TH IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	END-I	94.56	18.66	.00	.00	.00	.00
		END-J	-73.41		.00			

COLUMN FORCES AT LEVEL 4TH IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	627.68	280.29	.00	.00	50.72	.00
		BOTTOM	1608.70		.00			
2	CASE 1	TOP	-34.17	19.71	.00	.00	-50.72	.00
		BOTTOM	34.81		.00			

BEAM FORCES AT LEVEL 4TH IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	END-I	91.02	17.81	.00	.00	.00	.00
		END-J	-69.23		.00			

COLUMN FORCES AT LEVEL 3RD IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	1524.62	382.39	.00	.00	67.26	.00
		BOTTOM	2862.99		.00			
2	CASE 1	TOP	-30.00	17.61	.00	.00	-67.26	.00
		BOTTOM	31.63		.00			

BEAM FORCES AT LEVEL 3RD IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	END-I	84.08	16.54	.00	.00	.00	.00
		END-J	-64.81		.00			

COLUMN FORCES AT LEVEL 2ND IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	2794.09	486.57	.00	.00	80.91	.00
		BOTTOM	4497.07		.00			

2	CASE 1	TOP	-22.26	13.43	.00	.00	-80.91	.00
		BOTTOM	24.76		.00			

BEAM FORCES AT LEVEL 2ND IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	END-I	68.90	13.64	.00	.00	.00	.00
		END-J	-53.89		.00			

COLUMN FORCES AT LEVEL 1ST IN FRAME

COL ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	TOP	4453.85	592.16	.00	.00	89.67	.00
		BOTTOM	6526.39		.00			
2	CASE 1	TOP	-10.88	7.84	.00	.00	-89.67	.00
		BOTTOM	16.58		.00			

BEAM FORCES AT LEVEL 1ST IN FRAME

BAY ID	OUTPUT ID	OUTPUT POINT	MAJOR MOMENT	MAJOR SHEAR	MINOR MOMENT	MINOR SHEAR	AXIAL FORCE	TORSIONAL MOMENT
1	CASE 1	END-I	43.23	8.76	.00	.00	.00	.00
		END-J	-35.63		.00			

As can be seen from the analysis examples, the results obtained from the two methods are very close to each other and show very minor differences. Possible sources of these minor differences between the two results obtained from the finite element based program and its equivalent frame are:

- Approximate coefficients by Muto are used in the derivation and hence may lead to errors.
- The results obtained using ETABS are those at faces of joints, even when no rigid offsets are considered.
- The complex interaction between the shear and bending behavior of the wall, i.e. the interaction between shear and flexural stiffness of the shear wall.
- The assumption made to modify the stiffness of the adjacent beam.
- The assumption that the lateral loads at story levels are taken as if it is uniform throughout the height, even between story levels.
- Rounding of significant figures in calculating the different coefficients used to obtain the correction factors.

7. COMMENTARY

- The analysis model for a solid wall element should represent the strength, stiffness, and deformation capacity of the wall for in-plane loading. Out-of-plane behavior need not be considered, except where the wall acts as a flange for an intersecting wall element. Solid walls may be considered "slender" if their aspect ratio (height/length) is equal to or exceeds 4. Solid walls may be considered "squat" if their aspect ratio is less than or equal to 2. Slender walls usually are controlled by flexural behavior, although shear strength may be a limiting factor in some cases. Squat walls usually are controlled by shear behavior, although flexure sometimes may be a limiting factor. The response of walls with intermediate aspect ratios usually is influenced by both flexure and shear.

- Potential failure of anchorages and splices, interaction with other elements including nonstructural elements, and sliding along construction joints which limits the shear capacity of the wall may require modeling as well. Where sliding shear strength at a horizontal construction joint limits the shear capacity of the wall, this behavior can be modeled with a yielding spring in series with the wall panel. Besides, Walls can develop inelastic response associated with flexure, shear, development splices, and foundation rotations. The analytical model should represent the likely modes of inelastic response, too.

- If a wall yields in flexure, or if the foundation yields, continued lateral deformations involve plastic rotations centered near the compression toe of the wall, with uplift occurring toward the tension side. The equivalent column model cannot represent this effect, as the equivalent column is located at the wall centerline rather than the toe. This can lead to inaccuracies in representing interactions with adjacent components that may be affected by uplift.

Where interactions with adjacent elements are considered important, it may be preferable to represent the wall by using more-sophisticated techniques that represent the width of the wall. According to a literature I found from internet, multi-spring models (Otani 1980; Vulcano et al. 1989; Otani et al. 1985; Alami and Wight 1992; Charney 1991) may be considered. These models use two or three vertical springs to represent the axial and flexural stiffnesses and strengths of the wall, plus at least one horizontal or diagonal spring to represent the shear stiffness and strength of the wall. Other models that adequately account for flexural, shear, and rigid-body deformations also may be used. The model of the connection between the wall and foundation will depend on details of the wall-foundation connection and the rigidity of the soil-foundation system.

- In the equivalent frame with no rigid beams, only the boundary effects of coplanar frames are considered, however, boundary effects of frames connected perpendicularly to the wall should also be included in the three dimensional modeling of walls.

- In this study, considerations such as plane (2-D) dual systems, fixed foundation, uniform sections of walls and frames, system subjected to in-plane uniform lateral load, p, are taken. In cases where different conditions are encountered, it is relatively easy to extend the derivations to fulfill those conditions.

8. CONCLUSION

Generally speaking, as shown in this study, the center line equivalent columns of a shear wall depend on factors such as total height, width, and thickness of wall; story height; frame stiffness, i.e. beam and column stiffnesses, etc. Hence a general conclusion with regard to the effective stiffness of a shear wall can not be made, except the fact that it will be reduced when the wall is attached to coplanar frames. In order to know the effective stiffness of the shear wall, detailed but very simple calculations have to first be made.

The results obtained using this method were compared with results obtained using wide column analogy (with rigid haunches) and with results using braced-wide-column analogy (with coarse mesh elements of story-height and wall-width) and the results obtained are more close to the accurate results from ETABS than the other two.

An analysis using the aforementioned modeling technique with judicious use of equivalent center line column gives results (forces and displacements) which are very close to the accurate results obtained from the finite element-based software. Commercially available frame analysis programs can be used for this purpose without resorting to finite element programs. However, use of a finite element analysis may still be preferable for complex shear core systems.

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