LATERAL LOAD ANALYSIS OF SHEAR WALL-FRAME STRUCTURES

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ABSTRACT

LATERAL LOAD ANALYSIS OF SHEAR WALL-FRAME STRUCTURES

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The purpose of this study is to model and analyze the nonplanar shear wall assemblies of shear wall-frame structures. Two three dimensional models, for open and closed section shear wall assemblies, are developed. These models are based on conventional wide column analogy, in which a planar shear wall is replaced by an idealized frame structure consisting of a column and rigid beams located at floor levels. The rigid diaphragm floor assumption, which is widely used in the analysis of multistorey building structures, is also taken into consideration. The connections of the rigid beams are released against torsion in the model proposed for open section shear walls. For modelling closed section shear walls, in addition to this the torsional stiffness of the wide columns are adjusted by using a series of equations.

Several shear wall-frame systems having different shapes of nonplanar shear wall assemblies are analyzed by static lateral load, response spectrum and time history methods where the proposed methods are used. The results of these analyses are compared with the results obtained by using common shear wall modelling techniques.

Key words: Shear Wall; Shear Wall-Frame Structures; Wide Column Analogy

ÖZ

PERDE DUVAR-ÇERÇEVE SİSTEMLERİN YANAL YÜK ANALİZİ

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Bu çalışmanın amacı perde duvar-çerçeve tipi yapılardaki düzlemsel olmayan perde duvarların modellenmesi ve analizidir. Açık ve kapalı kesit perde duvar tipleri için üç boyutlu iki ayrı model geliştirilmiştir. Bu modeller, düzlemsel bir perde duvarın bir kolon ve kat seviyelerindeki rijit kirişlerle modellendiği geniş kolon benzeşimi yöntemine dayanmaktadır. Çok katlı yapıların analizinde sıkça kullanılan rijit diyafram kat kabulü de bu çalışmada gözönünde bulundurulmuştur. Açık kesit perde duvarlar için önerilen modelde, kat seviyelerindeki rijit kirişlerin birbiriyle bağlantıları burulmaya karşı serbest bırakılmıştır. Kapalı kesit perde duvarlar için ise buna ek olarak, geniş kolonların burulma rijitlikleri bir dizi denklem kullanılarak modifiye edilmiştir.

Önerilen modeller kullanılarak farklı tipte düzlemsel olmayan duvarlara sahip perde duvar-çerçeve sistemleri statik yanal yük, spektrum ve zaman tanım alanı yöntemleriyle analiz edilmiştir. Bulunan sonuçlar, farklı modelleme teknikleri kullanılarak yapılan analizlerden elde edilen sonuçlarla karşılaştırılmıştır.

Anahtar sözcükler: Perde Duvar; Perde Duvar-Çerçeve Yapılar; Geniş Kolon Benzeşimi

To My Family

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CHAPTER 1

INTRODUCTION

1.1 Introduction

The primary purpose of all kinds of structural systems used in the building type of structures is to support gravity loads. The most common loads resulting from the effect of gravity are dead load, live load and snow load. Besides these vertical loads, buildings are also subjected to lateral loads caused by wind, blasting or earthquake. Lateral loads can develop high stresses, produce sway movement or cause vibration [1]. Therefore, it is very important for the structure to have sufficient strength against vertical loads together with adequate stiffness to resist lateral forces.

In Turkey, a considerable number of buildings have reinforced concrete structural systems. This is due to economic reasons. Reinforced concrete building structures can be classified as [2]:

1. Structural Frame Systems: The structural system consist of frames. Floor slabs, beams and columns are the basic elements of the structural system. Such frames can carry gravity loads while providing adequate stiffness.

2. Structural Wall Systems: In this type of structures, all the vertical members are made of structural walls, generally called shear walls.

3. Shear Wall–Frame Systems (Dual Systems): The system consists of reinforced concrete frames interacting with reinforced concrete shear walls.

Most of the residential reinforced concrete building structures in Turkey have shear wall-frame systems. A typical floor plan of a shear wall-frame building structure is given in Figure 1.1. It is a fact that shear walls have high lateral resistance. In a shear

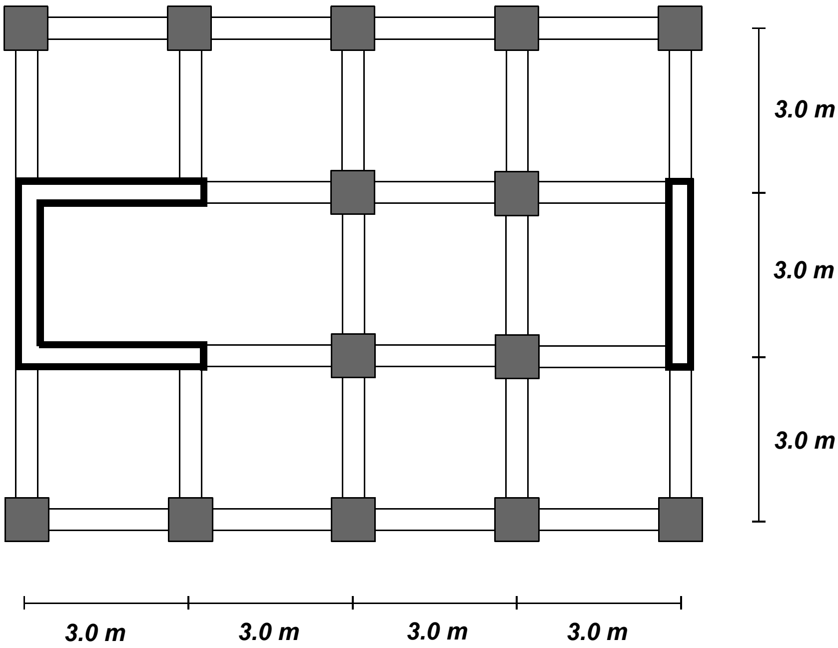


Figure 1.1 Typical Floor Plan of a Shear Wall - Frame Building Structure

wall-frame system, this advantage can be used by placing shear walls at convenient locations in the plan of the building.

In general, shear walls are in planar form in the plan of the building. However, some combinations of planar walls are also used in the structural systems. Typical non-planar shear wall sections used in the building structures are given in Figure 1.2.

The analysis of shear wall-frame structures is more complicated than frame systems. In order to reflect the actual behavior of the shear walls, several models have been developed. Wide column analogy, braced frame analogy and shell element derived by using finite element formulation are the most popular models. In the first two models, frame elements are used and in the last model, plane stress elements are used.

Another important point for the lateral load analysis of building structures is modelling the structural system. A common method which is widely used in design offices is to perform analysis on a two dimensional model obtained from the actual three dimensional system by using some simplifying assumptions. The total number of degrees of freedom is reduced significantly through this method. Some computer programs which model the buildings in series of two dimensional frames in two orthogonal directions use the same logic. The displacement compatibility is established by

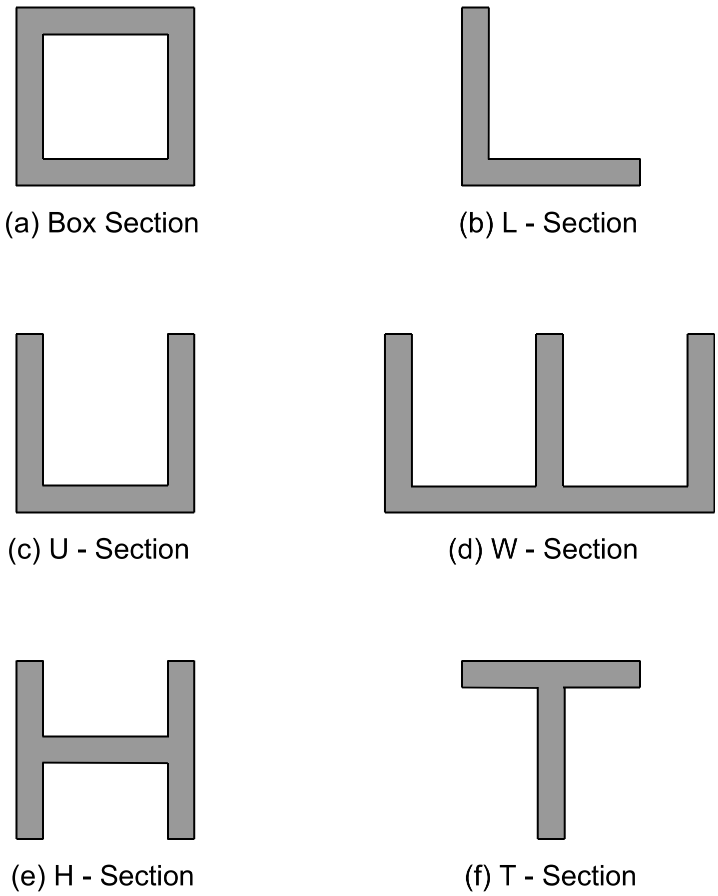


Figure 1.2 Typical Shear Wall Sections

infinitely rigid slabs at floor levels. However, this method, which is also called pseudo 3-D modelling [3], is not appropriate in lateral load analysis of some buildings, especially those having non-planar shear walls. Due to the complexity of the system, three dimensional analysis should be performed for such building structures. This is also valid for dynamic analysis of these kind of structures, so three dimensional analysis should be performed.

In three dimensional analysis, there are some factors that influence how fast results can be obtained and how accurate they are. The two most important factors are the amount of required data and computer running time. These two should be optimized in such a way that sufficient results can be obtained by entering less data and having a relatively short computing time. The computer running time is mostly affected by the total number of degrees of freedom in the system. It can be decreased by

(a) a reduction in the total number of elements used in the analysis and

(b) the use of elements having less degrees of freedom.

Frame element and shell element are the two basic structural elements used in the three dimensional analysis of structural systems. A general frame element has less total degrees of freedom when compared with a shell element. Modelling shear walls with frame members instead of shell elements can reduce the total degrees of freedom, which results a significant decrease in computer running time.

1.2 Object and Scope of the Study

As stated previously, the majority of the residential building structures in Turkey have shear wall-frame systems. Proper analysis and design of building structures that are subjected to static and dynamic loads is very important. Another important factor in the analysis of these systems is obtaining acceptable accuracy in the results.

The object of this study is to model and analyze shear wall-frame structures having non-planar shear walls. In order to reduce the required time and capacity for the analysis of the structural systems, frame elements are used instead of plane stress elements in modelling the shear walls. Two two-dimensional shear wall models, based on the conventional wide column analogy, are developed for modelling (a) open and (b) closed section non-planar shear walls The proposed models can be used in both static and dynamic elastic analysis of shear wall-frame structures.

The accuracy and the efficiency of the proposed models are tested by performing static lateral load analysis, response spectrum analysis and time history analysis on single shear walls and shear wall-frame systems. In order to check the validity of the proposed models, the same analyses are performed on the considered structural systems, in which shear walls are modelled by shell elements of SAP2000[4] and wall elements of ETABS[5]. In addition, comparisons are made with several methods and experimental results from the literature.

In the first part of the static lateral load analyses, single shear walls having different cross- sections are taken into consideration. They are subjected to point loads acting at floor levels. Two different loading conditions are applied on the structure:

(a) Axisymmetric lateral loading

(b) Pure floor torsions

Translations and rotations at floor levels are obtained for different shear wall models. In the second part, the behavior of the shear walls located in shear wall-frame building structures are investigated. Building structures having different floor plans and a different number of storeys are subjected to axisymmetric lateral loads and pure floor torsions. The performance of the proposed models is tested by comparing floor displacements and total resultant forces on shear walls at the floor levels. In the last part of the static analyses, the results of analysis and experiments of some previous studies are compared with the proposed models.

The natural vibration periods of single shear walls and shear wall-frame buildings, in which different shear wall models are used, are obtained in the first part of the response spectrum analyses. In the second part, response spectrum analysis is performed for the shear wall-frame building structures considered in static analyses and results obtained from different modelling methods are compared with each other. In the last part of the response spectrum analyses, the results of past studies on the natural vibration periods of different structures are compared with the results obtained using the proposed models.

In time history analyses, the sample shear wall-frame structures are subjected to an earthquake excitation at the base. The behavior of the structures, in which different shear wall models are used, are compared by considering

(a) top floor displacement history and

(b) total base shear force history.

In the next chapter, a literature survey of the analysis techniques of building structures against lateral loads is presented. The three types of analysis methods (equivalent lateral load method, response spectrum method and time history method) are also explained briefly. In Chapter 3, the shear wall models that appear in the literature are presented.

The proposed models are explained in Chapter 4, where basic assumptions about materials, members and building systems are also given. Verification studies of the proposed models are presented in Chapter 5, which contains the comparisons of the various shear wall models Finally, in the last chapter, the validity and the limits of the proposed models in view of the static and dynamic comparative studies are discussed.

CHAPTER 2

MODELLING AND ANALYZING BUILDING STRUCTURES

The two most important factors in the analysis and design of building structures are choosing an appropriate structural modelling method which reflects the actual behavior of the system and deciding on the analyzing technique to be performed on the structure. In the first part of this chapter, a literature survey of the modelling techniques used in the analysis of structures is presented. These approaches can be divided into two parts: two dimensional modelling and three dimensional modelling.

The most common analyzing techniques, equivalent lateral force method, modal superposition method and time history method, are summarized in the second part of this chapter.

2.1 Modelling Building Structures

As stated previously, forming a realistic mathematical model that reflects the actual behavior of the structural system is very important in analysis. In engineering practice, structural analysis of a reinforced concrete building is generally performed in the elastic range. However, in actual cases, the behavior of the structural system may be in the nonlinear range. This nonlinearity can be approximated and converted to a linear structural behavior by making a series of assumptions which simplify the problem significantly.

Modelling techniques that are proposed in the literature for building structures can be investigated in two main groups as follows [6,7]:

1. Modelling with a series of two dimensional systems.

2. Modelling with three dimensional systems.

2.1.1 Modelling Building Structures with a Series of Two Dimensional Systems

There are several methods that reduce the three dimensional building structure to a two dimensional system. In this part, the most common approaches that are often cited in the literature are presented.

The most widely applied technique for two dimensional modelling is connecting all bents of the structure at storey levels by rigid links, which simulate the in-plane rigidity of the floors. The lateral deflections of columns and shear walls can be defined in terms of the slab's horizontal translation and this allows the possibility of representing a three dimensional structure by a two dimensional model [1,8]. An example of a two dimensional model of a building structure is given in Figure 2.1.

In Figure 2.1, it can be observed from the floor plan of the building that there is a symmetry in the loading direction. In addition to this symmetry in the plan, the resultant of the distributed lateral load, *w,* is axisymmetric due to the floor plan. In view of these conditions, no floor torsion takes place and the structure undergoes simple translation only. In the model, two repeated units (Unit 1 and Unit 2) are connected by rigid links at floor levels and half of the total load, *w,* is applied to the system.

In this method, beams and columns are modelled by two dimensional frame elements, which have three degrees of freedom at each end. A typical two dimensional frame element is shown in Figure 2.2. Shear walls in the structural system are modelled by one of the methods mentioned in Chapter 1. These methods are discussed in detail in the following chapters.

The stiffness method may be used to solve the reduced system. This technique may be applied only to structures that do not twist, since the forces in the vertical and horizontal members of the structure obtained after analysis do not depend on their locations in the plan of the building. In other words, in the model, the torsional effect of lateral loading is not taken into consideration.

Another method was presented by Rutenberg and Eisenberger [10, 11] for the planar analysis of building structures. In their approach, shear force-axial force and torque-

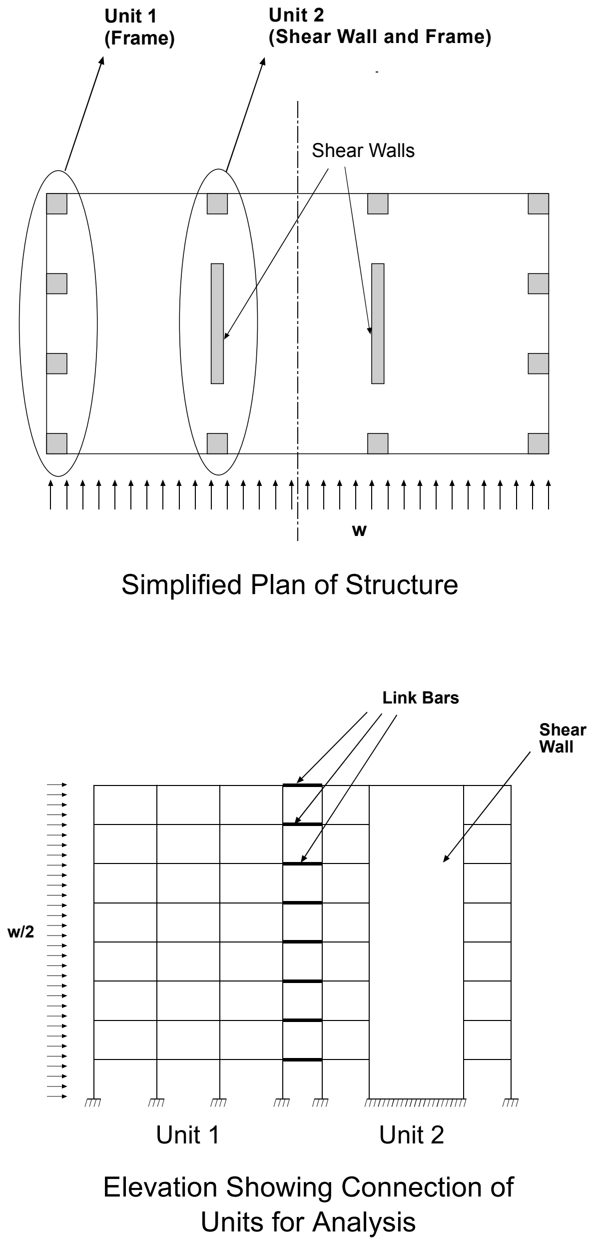


Figure 2.1 Two Dimensional Model of a Building Structure [9]

bending moment analogies are used to model the three dimensional behavior of the structures. The combined effects of bidirectional shear due to lateral and torsional displacements on columns of two orthogonal frames are considered in their model.

In the method developed by Smith and Cruvellier [12], the actual three dimensional system is reduced to a two dimensional planar system by using "governing nodes" and rigid links to represent translation and twist action.

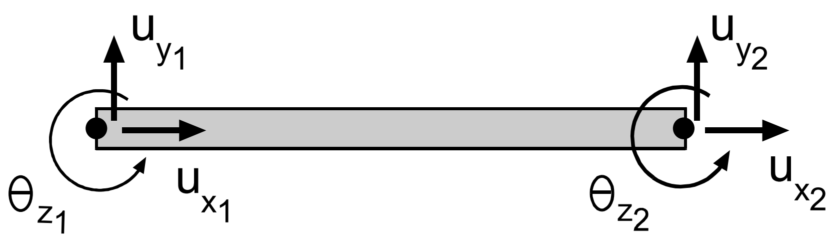


Figure 2.2 A Typical Two Dimensional Frame Element

Their proposed method starts with locating an arbitrary origin at the left of and below the lower left-end corner of structural plan. Then, a two dimensional model is formed by assembling all bents in the same plane with the x–direction bents in one group and y-direction bents in the other. Next, a set of governing nodes, which are constrained against vertical displacements, is established in the model. These governing nodes are connected to their corresponding floor level nodes by rigid vertical links, which have rotational releases at the floor level nodes. The lateral load acting on a storey is transformed into a lateral axisymmetric load and a torque acting at the origin of the storey. The governing nodes are used to apply this lateral force and torque for each bent. The reduced system can be solved using a computer program based on the stiffness method.

In the same study, another method was introduced in which, instead of separate sets of governing nodes, only a single set is used for all bents of the structure. This condensed model gives the same results as the previous one, but it reduces the computer storage and running time. An example of the condensed two dimensional model of a building structure is given in Figure 2.3. In another study by the same authors [13], the performance of their proposed method in the dynamic analysis of building structures was investigated.

These two models are effective in the analysis of structures having planar walls that translate and twist, but the application of the models on buildings having nonplanar shear walls with different shapes (L, U, etc.) may give incorrect results.

The conventional continuum approach, which is based on the closed form solution of the characteristic differential equation of the structural system, can also

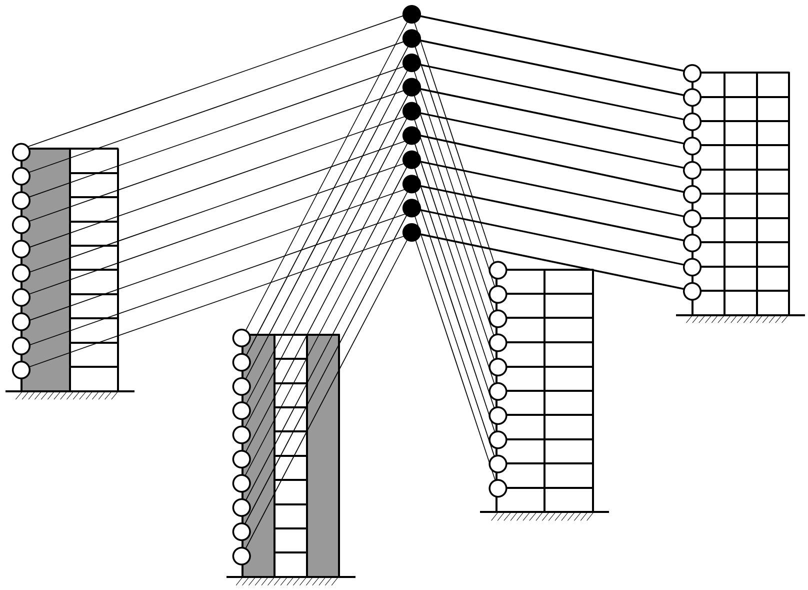


Figure 2.3 Condensed Two Dimensional Model [13]

be categorized as a two dimensional modelling method. In this method, in contrast to the above discrete member models, the horizontal slabs and beams connected to the columns and shear walls are assumed to form a continuous connection medium having equivalently distributed stiffness properties. The continuum method is limited to structures having uniform structural properties at each floor and it is considered to be a good method for understanding the overall behavior of the structural system [1].

2.1.2 Modelling Building Structures by Three Dimensional Systems

In a typical three dimensional system, the frame elements that are used in modelling beams and columns have six degrees of freedom per node: three translations and three rotations. An example of a three dimensional frame member is given in Figure 2.4. If the building structure has shear walls, probably a mesh of rectangular plane stress elements having six degrees of freedom at every corner should be used for modelling each single shear wall (a typical rectangular plane stress element having 24 degrees of freedom is shown in Figure 2.5). If the whole system is considered, there will be too many unknowns and a large system of equations would have to be solved in order to obtain results from such an analysis.

Several methods and computer programs have been developed for the analysis of building systems in which the total number of unknowns are reduced by some

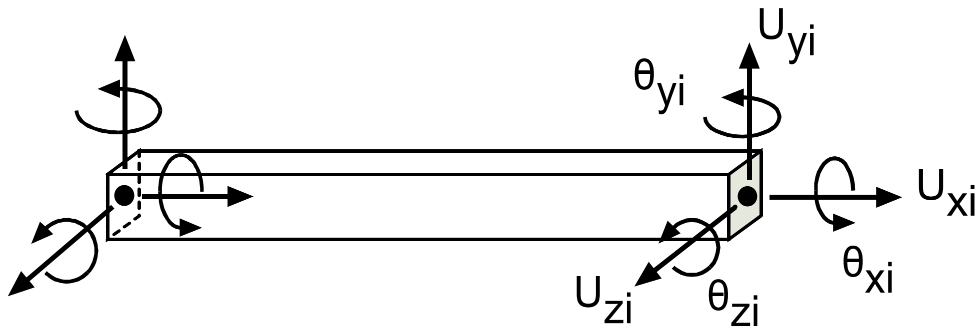


Figure 2.4 A Typical Three Dimensional Frame Element

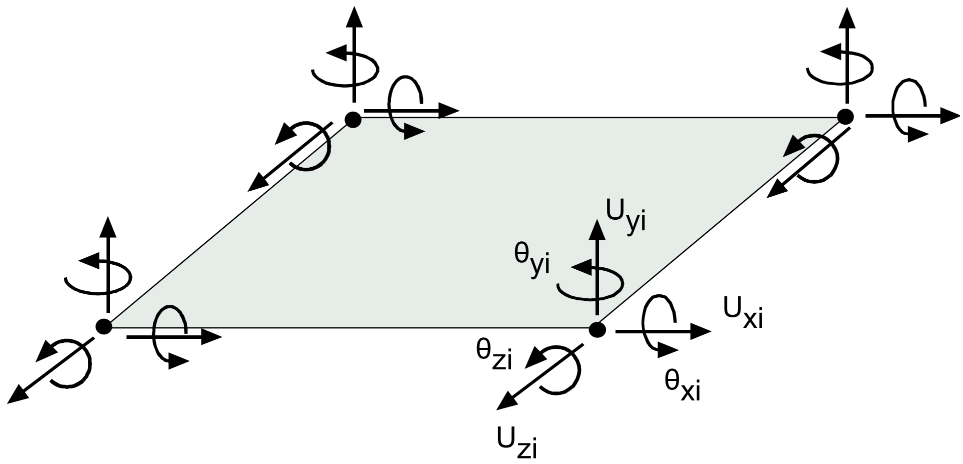


Figure 2.5 Three Dimensional Plane Stress Element Having Six Degrees of Freedom at Each Node

assumptions. In addition to these methods, generalized 3D computer programs are also available. In this section, a review of these methods and computer programs that model the building structures using three dimensional systems is reviewed.

At the beginning of the 1970s, with the developments in computer technology, several computer programs were designed for the analysis of building type of structures. TABS [14], which was developed by Wilson and Dovey in 1972, is one example of such programs. In the first version of TABS, a three dimensional building structure is reduced to a series of planar rectangular frames and each frame is treated as an independent structure. The structural stiffness matrix is formed under the assumption that all frames are connected at each floor level by a diaphragm, which is rigid in its own plane. In the program, the lateral loads are transferred to the columns and shear walls through these rigid floor diaphragms. In addition to three degrees of freedom at each floor level (translation in x and y directions and rotation about the vertical axis), at column and shear walls there is additional vertical displacement and a rotation. TABS was followed by TABS 80 [15], TABS 90 [16], and ETABS [5]. The first versions, referred to as pseudo 3-D programs, have limited applications and do not give adequate results, especially in the lateral load analysis of building structures having L, U, etc. shaped shear walls [3].

A series of generalized structural analysis programs were developed by the same group. SAP [17] and SOLID SAP [18] are the two general purpose finite element programs for three dimensional analysis of structures. They were followed by SAPIV [19], SAP90 [20] and SAP2000 [4]. For modelling the members of structural systems, two different types of elements are available in SAP: frame element and shell element. Both were derived by finite element formulation. SAP90 and SAP2000 are widely used in design offices for the analysis and design of building structures.

Ghali and Neville [21] formulized a three dimensional analysis of shear wall structures having three degrees of freedom at each floor. They derived the structural stiffness matrix for a typical shear wall and used rigid floor assumption in order to reduce the degree of kinematic indeterminacy. In their proposed method, the reduced system is solved using the stiffness method. In Figure 2.6, reduced degrees of freedom at the floor of a single-storey building structure is shown.

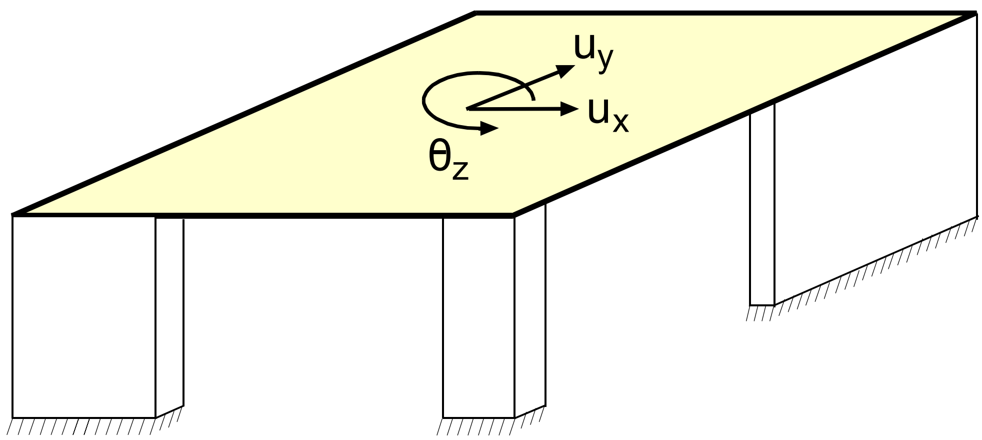


Figure 2.6 Reduced Degrees of Freedom at the Floor of a Single Storey Building Structure

In 1984, Swaddiwudhipong et al. [23] presented a computer program for the analysis of asymmetric shear wall-frame structures. In the proposed method, the continuum approach was adopted to model the structure as a shear–flexure cantilever and the Galerkin technique was used for the solution of differential equations. Thambiratnam and Irvine [24] suggested a method for the analysis of torsionally coupled multistorey buildings based on the assumption of rigid floors, each with three degrees of freedom. In their method, the potential and kinetic energies of the building are calculated and an equation of motion is derived using Lagrange equations. Zeng and Wiberg [25] suggested a generalized coordinate method for the reduction of unknowns in the three dimensional linear elastic analysis of tall buildings using the displacement method. The proposed reduction technique is based on three assumptions:

1. In-plane rigid floors.

2. A two dimensional polynomial approximation for the out-of-plane displacement of floors.

3. A one dimensional polynomial approximation of displacements with the height of the building.

In 1989, Li [26] presented a three dimensional frame analysis program, TAP-86, for tall building structures. This program, which is based on stiffness method, has two basic elements:

1. Space rectangular beams

2. Thin-wall column elements.

In this program, the torsion and warping effect of shear walls are modelled by the proposed thin-wall elements. The program is capable of performing both lateral load analysis and dynamic analysis.

Syngellakis and Younes [27, 28] proposed an analysis technique based on the transfer matrix method for shear wall-frame systems. This method is based on basic beam relations and is claimed to be suitable for the analysis of structures having a non-branching chain of members. Öztorun [3] developed a computer program, TUNAL, for the analysis of shear wall building structures. In this program, based on the finite element technique, a four-noded finite element with 24 degrees of freedom is formulated.

Recently, Hoenderkamp [29, 30] presented a simplified hand method for estimating forces in asymmetric multi-bent structures subjected to horizontal loading. His method is developed from coupled-wall deflection theory based on nondimensional structural parameters. Closed form solutions of coupled differential equations for translation and rotation are obtained in his method.

Three dimensional analysis of multistorey building structures is investigated by many other authors. There are also several commercial computer programs used in design offices [31, 32].

2.2 Analyzing Methods

The decision about which method to use in analyzing building structures is no less important than choosing an appropriate modelling technique. As stated above, linear and nonlinear analysis are the two basic methods. Linear elastic analysis is generally used for multistorey structures due to its simplicity.

Linear elastic analysis of building structures can be performed by using static or dynamic approaches. Briefly, static analysis is performed by considering the building structure as stationary and the loads acting on the structure as constant and not time dependent. The effects of all kinds of loads are idealized and simplified in this approach. For example, two lateral loads, wind and earthquake load, are assumed to act at the floor levels of structures. The equivalent lateral force method, which is recommended by most of the earthquake codes [33, 34, 35, 36], is a static method widely used in the elastic analysis of multi-storey structures subjected to earthquake loads.

In contrast to static analysis, dynamic analysis is based on the behavior of the structural system in a time domain. The modal superposition method and the time history method are the dynamic analysis methods most commonly suggested by earthquake codes.

These three methods (the equivalent lateral force method, the modal superposition method and the time history method) are summarized in the following sections.

2.2.1 Equivalent Lateral Force Method

The equivalent lateral force method is commonly preferred by design engineers because of its simplicity. It is based on the following assumptions [37]:

1. The effects of yielding on the building structure are approximated using elastic spectral acceleration reduced by a modification factor.

2. A linear lateral force distribution can be used to represent the dynamic response of the building structure.

The following procedure is used for the analysis of building structures using the equivalent lateral load method:

1. Determination of the first natural vibration period.
2. Determination of the total equivalent seismic load.
3. Determination of design seismic loads acting at storey levels.
4. Determination of points of application of design seismic loads.
5. Analysis of the structural system.

A building structure subjected to lateral forces obtained by the equivalent lateral force method is shown in Figure 2.7. A triangular distribution of equivalent lateral loads with zero loading at the base of the structure is considered in the analysis.

More detailed information about the equivalent lateral load can be found in [38] and [39].

2.2.2 Modal Superposition Method

Modal superposition is a method in which the equations of motions of floor slabs are transformed from a set of "n" simultaneous differential equations to a set of "n" independent equations by making use of normal coordinates. The solutions of these equations for each independent mode of vibration give the corresponding displacements and forces. The actual elastic response of the structure under earthquake force is obtained by superposing the evaluated individual solutions [40]. One of the most important concepts in this method is the combination of the individual solutions. In the literature, several combination methods are presented. Among them, SRSS (square-

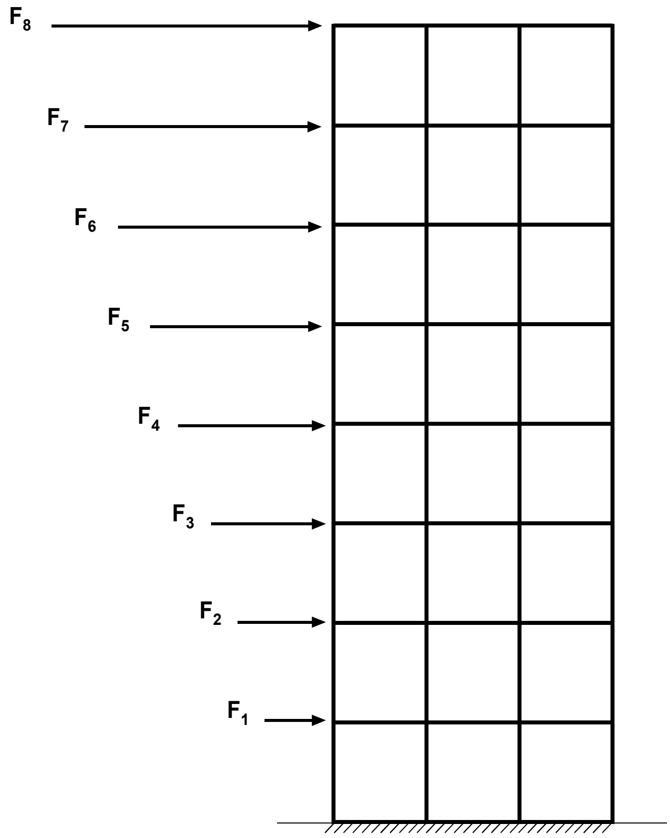


Figure 2.7 Building Structure Subjected to Equivalent Lateral Loads

root-sum-of-squares) [41] and CQC (complete quadratic combination) [42] are the two most common modal combination techniques. The basic steps of the modal superposition method are as follows [2]:

1. Selection of design spectrum.

2. Determination of mode shapes and periods of vibration.

3. Determination of the level of response from the design spectrum for the period of each of the modes considered.

4. Calculation of the participation of each mode corresponding to the single – degree – of – freedom response read from the curve.

5. Addition of the effects of modes to obtain combined maximum response.

6. Conversion of combined maximum response into shears and moments.

7. Analysis of the building for resulting moments and shears in the same manner as for static loads.

Detailed information about the modal superposition method and the mode combination techniques can be found in [22], [40] and [43].

2.2.3 Time History Method

In this method of analysis, a selected earthquake motion is applied directly to the base of the structure. For the full duration of the earthquake, instantaneous stresses throughout the structure are evaluated at small intervals. The maximum stress in any member can be obtained using the output records. The time history method is not widely used as an analysis method due to its long computer running time and relative cost [2]. Detailed information about the application of the time history method can be found elsewhere [1, 22, 43].

The main steps of time history analysis are as follows [2]:

1. Selection of the earthquake record.
2. Digitization of the record as a series of small time intervals.
3. Setting up of the mathematical model of the structure.
4. Application of the digitized record to the model.
5. Determination of the maximum member stresses by using the output records.

CHAPTER 3

MODELLING OF SHEAR WALLS

According to Turkish Earthquake Code [33], a shear wall is defined as a vertical structural member having a length of seven or more times greater than its thickness. Being the major lateral load resistant units in multistorey building structures, shear walls have been studied experimentally and theoretically over the last fifty years.

In the lateral load analysis of building structures having shear walls, proper methods should be used for modelling planar and nonplanar shear wall assemblies. Shear wall models in the literature can be divided into two:

1. Models developed for elastic analysis of building structures.

2. Models developed for nonlinear analysis of building structures.

The investigation of nonlinear shear wall models is beyond the scope of this study. Examples of such models can be found in [44], [45] and [46].

In this chapter, shear wall models developed for the lateral load analysis of multistorey structures in elastic region are presented. Since the methods for modelling building structures are analyzed separately (two dimensional modelling and three dimensional modelling are presented in Chapter 2) shear wall modelling studies can also be investigated in according to the two and three dimensional approaches.

3.1 Two Dimensional (Planar) Shear Wall Models

The literature mentions several shear wall models that were developed for two dimensional elastic analysis of multistorey building structures. In this part, a review of these models is given.

3.1.1 Equivalent Frame Model (Wide Column Analogy)

The equivalent frame model was developed by Clough et al. [47], Candy [48] and MacLeod [49] for the analysis of plane coupled shear wall structures. The model was limited to lateral load analysis of rectangular building frames without torsion. It was improved in the 1970's by Mcleod [50, 51] and McLeod and Hosny [52] for the analysis of nonplanar shear walls.

In the equivalent frame method, which is also known as wide column analogy, each shear wall is replaced by an idealized frame structure consisting of a column and rigid beams located at floor levels. The column is placed at the wall's centroidal axis and assigned to have the wall's inertia and axial area. The rigid beams that join the column to the connecting beams are located at each framing level [8]. A sample model is shown in Figure 3.1. In this method, the axial area and inertia values of rigid arms are assigned very large values compared to other frame elements.

Due to its simplicity, the equivalent frame method is especially popular in design offices for the analysis of multistorey shear wall-frame structures.

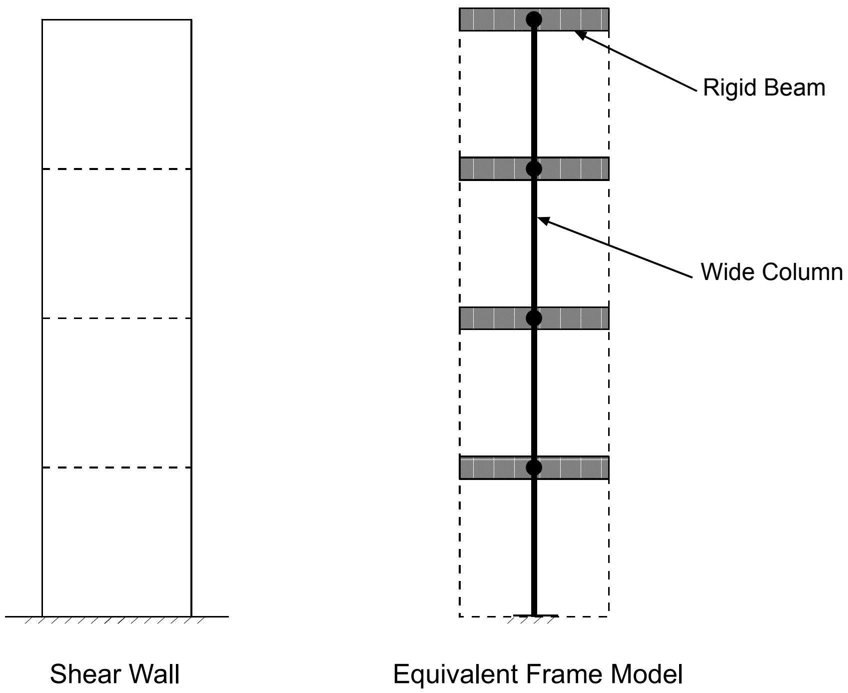


Figure 3.1 Equivalent Frame Model of a Shear Wall

3.1.2 Analogous Frame Method

This alternative method, proposed by Smith et al.[53], was developed for modelling planar and nonplanar shear walls. The purpose of their study was to overcome the artificial flexure and excessive shear deformations due to discrete modelling of continuous vertical joints between adjacent planar wall units in the conventional equivalent frame method. In their study, they proposed two different frame models for shear wall analysis: the braced wide column analogy and the braced frame analogy.

The braced wide column analogy is similar to the conventional wide column analogy presented above, but with diagonal braces. A single module consists of rigid horizontal beams, equal in length to the width of the wall, connected by a single central column. Hinged-end diagonal braces connect the ends of the beams [53]. A typical braced wide column module is shown in Figure 3.2. A planar shear wall modelled by braced wide column analogy is given in Figure 3.3.

The stiffness properties of the column (I_c, moment of inertia of the column and A_c, area of the column) and braces (A_d, axial area of the diagonal brace) are determined by the following three equations:

$$I_c = \frac{tb^3}{12} \tag{3.1}$$

$$\frac{12EI_c}{h^3} + \frac{2EA_d \cos^2\theta}{l} = \frac{btG}{h} \tag{3.2}$$

$$\frac{EA_c}{h} + \frac{2EA_d \sin^2\theta}{l} = \frac{EA_w}{h} \tag{3.3}$$

These equations are based on the simulation of the bending, shear and axial stiffnesses of corresponding wall segments. In the equations, t is the thickness and b is the width of the shear wall, E is the modulus of elasticity, h is the height of the shear

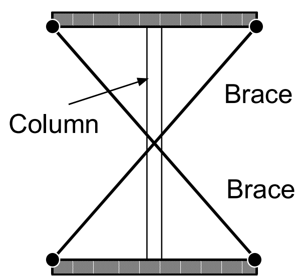


Figure 3.2 Braced Wide Column Module

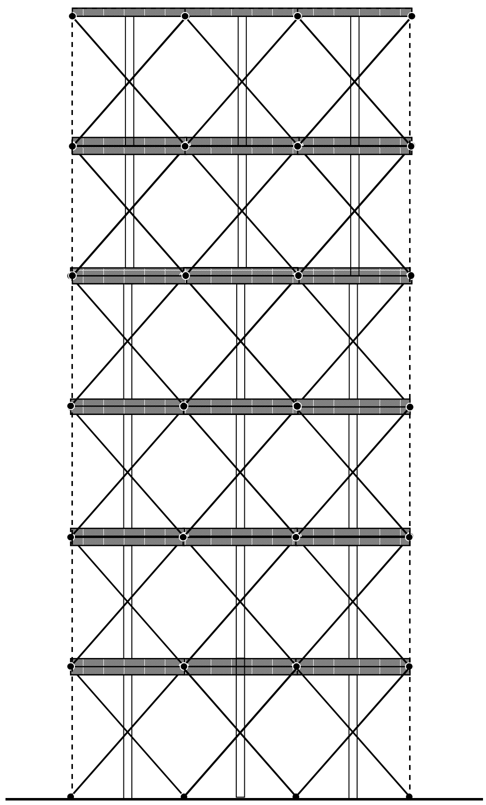


Figure 3.3 A Planar Shear Wall Modelled by Braced Wide Column Analogy

wall, θ is the slope of the diagonal, l is the length of the diagonal brace, G is the shear modulus and A_w is the sectional area of the shear wall.

In braced frame analogy, the module is asymmetric and consists of a column on the left hand side connected to the rigid beams, a hinged-end on the right hand side and diagonal braces. The left-hand end of the beam and the ends of the column rotate with the nodes, while the right-hand end of the beam and the link are rotationally released from the nodes [53]. Similar equations (Eqn.3.1, 3.2 and 3.3) are used for obtaining the stiffness properties of the column, braces and the link. In Figure 3.4, a sketch of a braced frame module is given.

One of the deficiencies of the two analogies is the probability of obtaining negative stiffness values for the column and braces for certain aspect ratios of the framework modules. Since most of the frame analysis computer programs cannot perform analysis with negative area and inertia values, these methods may be ineffective.

Koumousis and Peppas [54] derived the stiffness matrices for the two proposed

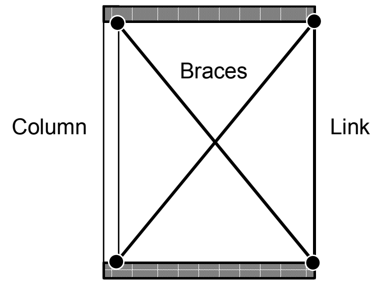


Figure 3.4 Braced Frame Module

analogies. In their study, the two dimensional braced wide column module and the braced frame module are presented as modified three dimensional modules which can be introduced to other structural analysis programs. Then negative stiffness value problem is solved by creating new shear wall geometry, in which the original shear wall is divided into a series of adjacent horizontal shear walls having positive stiffness values.

An improved wide column-frame analogy was proposed by Kwan [55] in 1991 in order to overcome the artificial flexure problem of the conventional wide column-frame analogy. It is an alternative method of the braced wide column and braced frame analogies developed by Smith et al., in which the shear deformation factor of the wall elements are adjusted to compensate for the errors in deformation due to artificial flexure. In another study by the same author [56], the analogous frame modules developed by Smith et al. [53] and the improved wide column frame module that he developed later [55] are shown to be equivalent to each other.

The three dimensional applications of the proposed models are discussed in the following parts.

3.1.3 Finite Element Models

In the finite element modelling of a two dimensional shear wall, the wall is divided into smaller elements having finite size and number. These elements may be triangular, rectangular or quadrilateral. The most common plane stress element used for modelling shear walls is the two dimensional shell element. It has three degrees of freedom at each node (two translation and one rotation). The finite element method is widely used not only in modelling multistorey structures but also for all kinds of engineering problems. In Figure 3.5, a finite element model of a coupled shear wall is

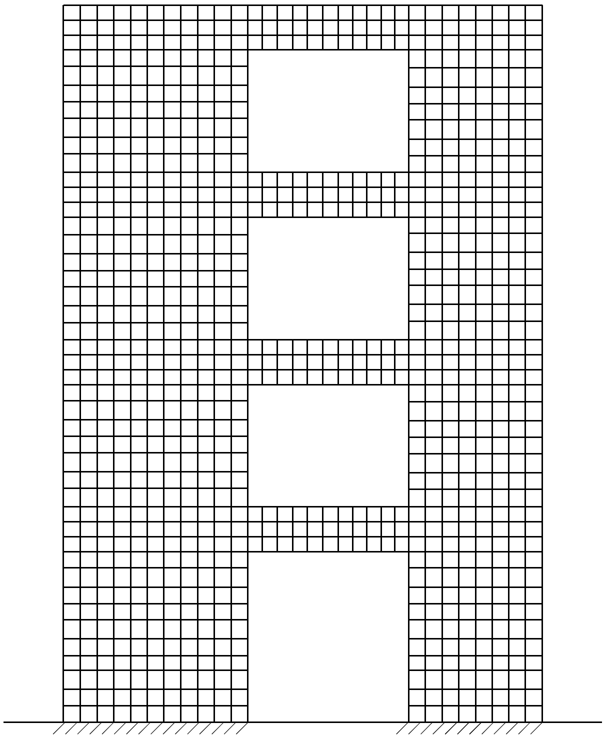


Figure 3.5 Finite Element Model of a Coupled Shear Wall

given. A rectangular shell element is given in Figure 3.6.

For the planar analysis of shear wall–frame structures, two dimensional frame members with three degree of freedom at each node (Figure 2.2) are used to model beams and columns and two dimensional plane stress elements are used for modelling shear walls.

An important factor in finite element analysis is the decison on the total number of

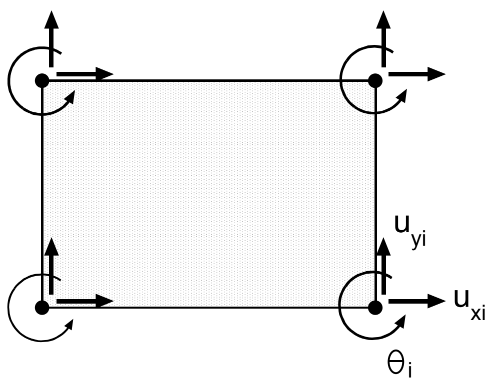


Figure 3.6 A Rectangular Shell Element with Three D.O.F. at Each Node

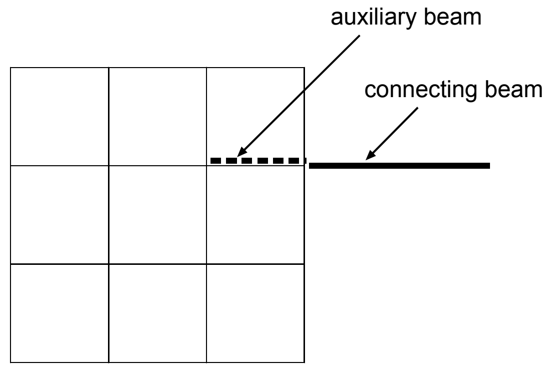


Figure 3.7 Plane Stress Elements with Horizontal Auxiliary Beam

elements that will be used in modelling the shear walls. More accurate results can be obtained with a finer mesh, but the total running time may be longer. An optimum number of finite elements should be included in analysis.

Starting in the late 1960's, different forms of finite elements were developed for the analysis of shear walls [57, 58, 59, 60, 61]. A review of these methods can be found in [62]. In order to improve the efficiency of the finite element method and to deal with the parasitic shear problem, finite strip elements [63] and high order elements were developed [64] for modelling shear walls.

3.1.4 Plane Stress Element with Auxiliary Beam

The suggested model is a combination of two type of elements, a rectangular plane stress element with two translation degrees of freedom at each node, which is also defined as a membrane element, and a frame element with three degrees of freedom at each end. This was proposed by Smith and Coull [1] for modelling shear wall-beam connections. The rotation of the wall and the moment are transferred to the external beam by the rigid auxiliary beam, which can be located horizontally or vertically (Figure 3.7 and 3.8).

In the same study, another shear wall model, which is a combination of membrane elements with continuous rigid auxiliary beams located at floor levels for transferring rotation and moments, is proposed. An example of this model is given in Figure 3.9. A single membrane element is used to model a shear wall module between two floor levels. In the model, a fictitious column is located at one edge of the wall assembly

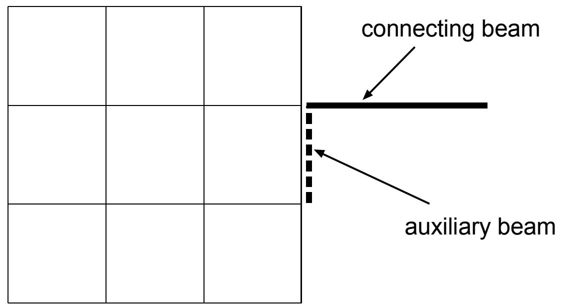


Figure 3.8 Plane Stress Elements with Vertical Auxiliary Beam

to represent the torsional stiffness of the system. A torsion constant, which is equal to the sum of the individual walls' torsional constants, is assigned to the column and all other stiffness values are designated as zero.

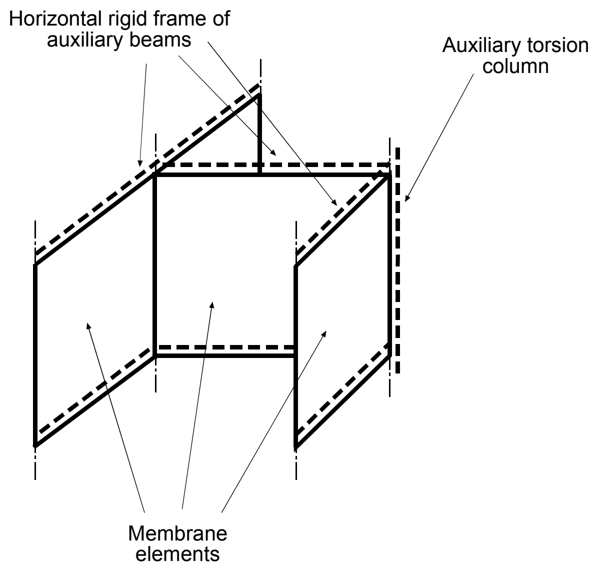


Figure 3.9 Membrane Elements with Rigid Beams at Floor Levels and Auxiliary Torsion Column

3.2 Shear Wall Models for Three Dimensional Analysis

Shear wall models used in the three dimensional analysis of structures are generally the modified versions of two dimensional models. In the following pages, the most common of these models are reviewed.

3.2.1 Equivalent Frame Model

A two dimensional (planar) equivalent frame model is used by many as a three dimensional model especially for the analysis of tall buildings having reinforced concrete cores. MacLeod [50, 51, 65], MacLeod and Hosny [52] and Lew and Narow [66] studied the equivalent frame model for the analysis of shear wall cores of tall buildings. Ghuneim [67] and Dikmen [68] used the equivalent frame model in the three dimensional analysis of tunnel form buildings.

The model is identical to the two dimensional equivalent frame with the additional requirement of the vertical compatibility of the intersecting walls. In Figures 3.10 and 3.11, a triangular reinforced concrete core and its equivalent frame model are shown [66].

Smith and Girgis [69] determined that the conventional wide column model has some deficiencies, especially in analyzing closed or partially closed core walls subjected to torsion. They reported that when these type of walls are subjected to shear stresses, the column elements used to model the walls are afflicted by parasitic moments. Due to this, in a closed or partially closed section core modelled by wide column analogy, relatively high values are obtained in shear deformations and rotations when compared with finite element modelling. Kwan [55] also stated the sources of errors in the application of wide column analogy to the three dimensional analysis

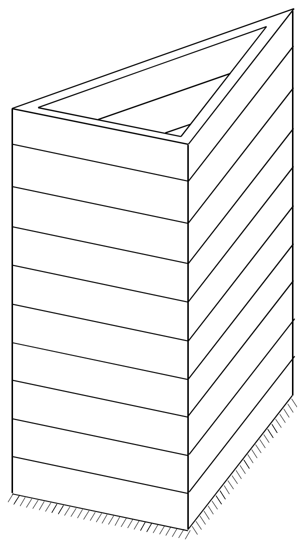


Figure 3.10 Triangular Core [66]

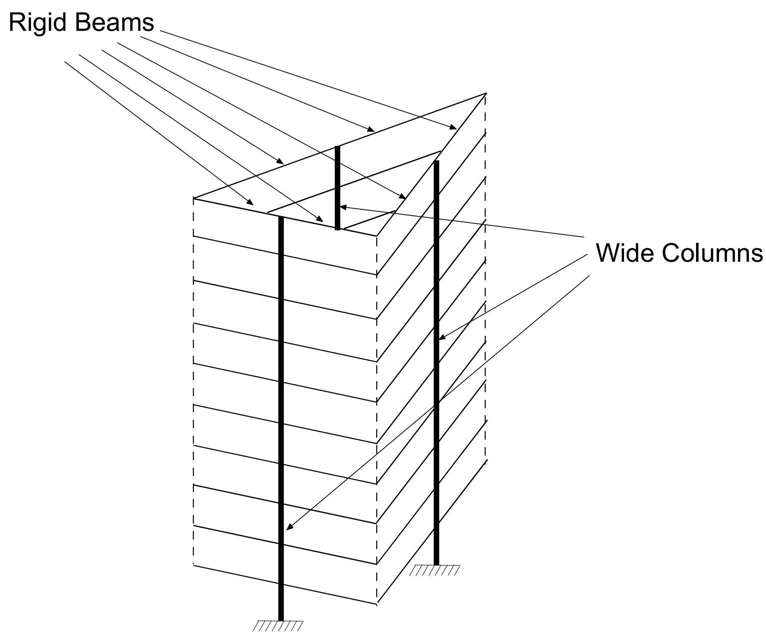


Figure 3.11 Equivalent Frame Model of a Triangular Core [66]

of core structures.

3.2.2 Braced Frame Analogy

Smith et al.[53] applied their braced frame analogy to the three dimensional analysis of core structures. Each wall is divided into a coarse mesh of wall width storey height modules. In three dimensional modelling, they placed a torsional column to represent the torsional behavior of the core structure. The braced frame model of an elevator core is given in Figure 3.12. In this method, column elements are not afflicted by parasitic moments.

This model was developed especially for analyzing open, partially open and closed cores. The application of the model to three dimensional shear wall-frame building structures has not been reported in any study. This may be due to the complexity and time consuming process of the method.

3.2.3 Two – Column Analogy

Two – column analogy was proposed by Smith and Jesien [70] for the analysis of single core walls or cores which are the parts of larger surrounding structures subjected to lateral loading. The model consists of two columns (representing the warping and

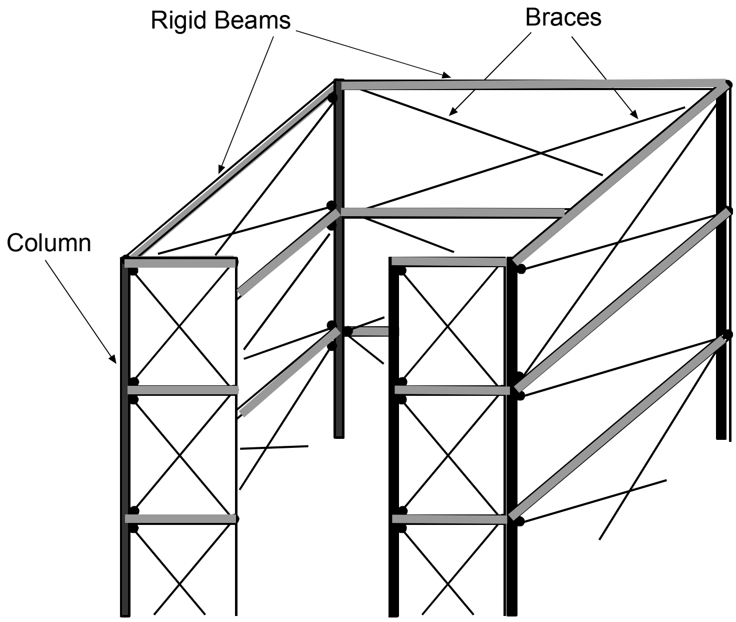


Figure 3.12 Three Dimensional Model of a Core with Braced Frame Analogy [53]

St. Venant torsional modes) placed on one of the core's principal bending axes and located on opposite sides of the shear center. The properties of the core are shared between the two columns. An example of this analogy for a U-shaped core is given in Figure 3.13. It was reported that [70] the deflections and stresses obtained by the proposed method were within 10% and 20% respectively when compared with the results obtained from shell elements.

This method is limited in analyzing single core walls and is generally used for understanding the overall behavior of the structural system.

3.2.4 Single Warping - Column Model

Single warping – column model was developed by Smith and Taranath [71] in order to represent the warping behavior of cores, especially having closed or open sections. A single column model having seven degrees of freedom for each node is used. The warping column element, which is located on the shear centre of the core, has a 14x14 stiffness matrix. A typical element of warping column model with considered degrees of freedom is shown in Figure 3.14. In Figure 3.15, a closed core wall modelled with single warping column is given.

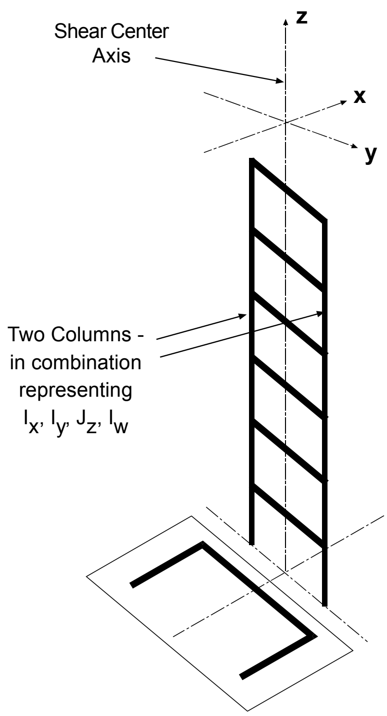


Figure 3.13 Two-Column Model of a U Shaped Core [1]

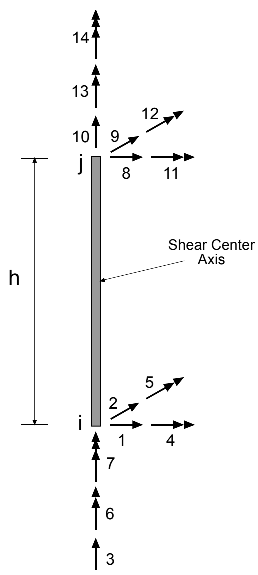


Figure 3.14 Typical Element of Warping Column Having 14 D.O.F.

Similar to the two-column model, it is a simple model developed for fast analysis of tall building cores and it may cause significant errors.

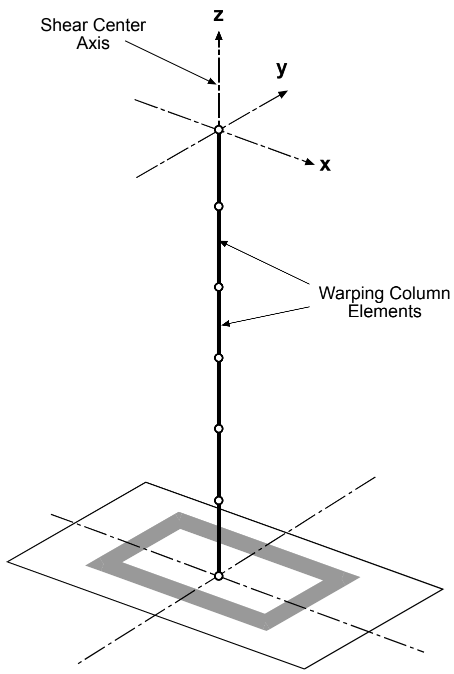


Figure 3.15 A Closed Core Model Using the Single Warping Column Model

3.2.5 Finite Element Models

The finite element method is widely used in three dimensional analysis of building structures. Various types of finite elements, which differ in shape and the number of degrees of freedom at the nodes, have been developed. A detailed review of these studies can be found in [3] and [62].

SAP2000 [4] is the most commonly used finite element program for three dimensional analysis of building structures. The shell element of SAP2000, which is a combination of a membrane element and a plane stress element, is a quadrilateral element with six degrees of freedom at each node (Figure 3.16). It is widely used in modelling planar and nonplanar shear wall assemblies. A example of a three dimensional shear wall assembly modelled with SAP2000 is given in Figure 3.17.

Oztorun [3] developed a rectangular finite element having six degrees of freedom at each node to analyze tunnel form buildings. Modelling shear walls with wall elements of ETABS [5] can also be categorized as a three dimensional finitie element modelling.

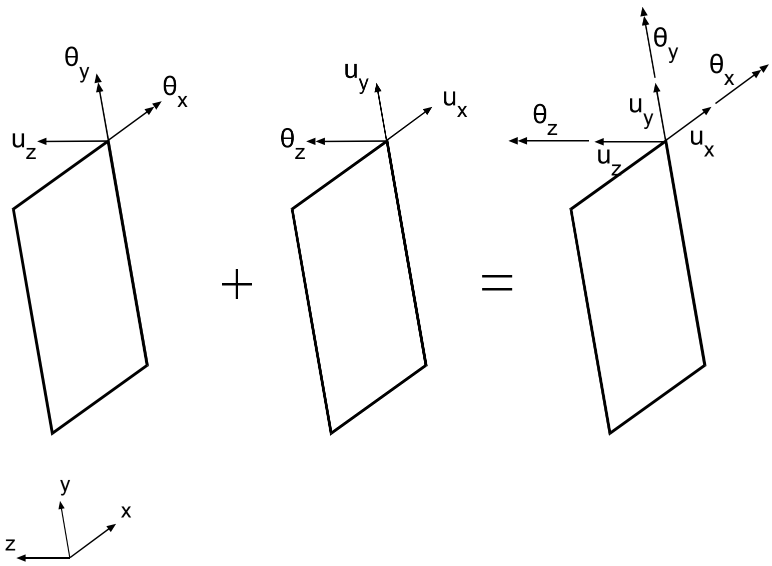


Figure 3.16 Combination of Membrane Element and Plane Stress Element to Form Shell Element

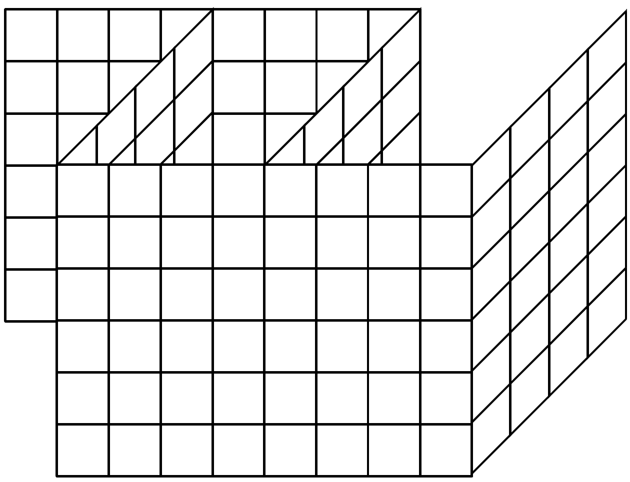


Figure 3.17 Three Dimensional Finite Element Model of a Nonplanar Shear Wall Assembly

CHAPTER 4

THREE DIMENSIONAL MODELLING OF SHEAR WALLS IN THE LATERAL LOAD ANALYSIS OF SHEAR WALL-FRAME STRUCTURES

As stated in Chapter 1, the main objective of this study is to model nonplanar shear wall assemblies in a realistic and feasible way for the analysis of shear wall–frame structures. The modelling studies are based on rigid diaphragm floor assumption and the three dimensional equivalent frame method, in which a planar shear wall is modelled using an equivalent column and rigid beams at floor levels. A generalized three dimensional finite element program, SAP2000, is used in the studies.

In the first part of this chapter, the basic assumptions used in the modelling studies are presented. These assumptions are divided into three categories:

1. Material behavior
2. Element behavior
3. Structural behavior

In the second part, the models developed for nonplanar shear walls having open and closed sections are presented. The comparison of the proposed models with the other shear wall modelling methods (using SAP2000 shell element, ETABS wall element and conventional equivalent frame method) in lateral load analyses are made in the last part of the chapter.

4.1 Basic Assumptions

In the analysis of all kinds of structures, a number of assumptions should be made in order to reduce the size of the actual problem. As stated above, these assumptions can be divided into three categories: material behavior, element behavior and structural behavior. In this part, the assumptions used in the modelling studies are presented.

4.1.1 Material Behavior

The behavior of the materials in this study is assumed to be linear elastic. Linear elasticity is the most common material model for analyzing structural systems and is based on the following assumptions [8]:

1. The material is homogenous and continuous.

2. The strain increases in a linear portion as stress increases.

3. As stress decreases, the strain decreases in the same linear portion.

4. The strain induced at right angles to an applied strain is linearly proportional to the applied strain, which is called Poisson's ratio effect.

In addition, the effects of cracking, creep, shrinkage and temperature on the material are not taken into consideration.

4.1.2 Element Behavior

Two different structural elements are used in the analyses. The three dimensional frame element, which is presented in Chapter 2 (Figure 2.4), is used for modelling the beams and columns of the structural systems. It is assumed to have six degrees of freedom at each end. The elements of the equivalent frame model (equivalent wide column and rigid beams) are also modelled using three dimensional frame element.

The three dimensional shell element, which is used for modelling shear walls in verification studies, is assumed to have six degrees of freedom at each node. A typical shell element was given in Figure 3.16.

Additional assumptions about the element behavior are as follows:

1. Shear deformations in the structural elements are ignored.

2. Frame elements and shell elements have uniform cross-sections throughout the length.

4.1.3 Structural Behavior

The multistorey building systems analyzed in this study are considered to be rigid frame structures. In such systems, all structural elements of the system are assumed to have infinitely rigid moment resistant connections at both ends.

Another assumption about the structural system is the linear elastic structural system behavior, in which the deformations are proportional to the loads. It is widely used in structural analysis and leads to a very important simplification called superposition. In superposition, if a linear elastic structure is subjected to a number of simultaneously applied loads, the overall response can be determined by summing the responses of the structure to the loads applied at one time [72]. Based on this assumption, the behavior of the structural system under eccentric lateral loads can be determined by superposing the behavior under the considered lateral loads, which are applied axisymmetrically, and the behavior under the pure torsion produced by these eccentric lateral loads.

In the analysis performed in this study, it is assumed that only the structural components participate in the overall behavior. The effects of structural components, such as non-structural walls, are assumed to be negligible in the lateral load analysis.

One of the most important assumptions in this study is the 'rigid diaphragm floor' assumption, a common assumption which simplifies the problem significantly and reduces computing time. The rigid diaphragm floor assumption is based on the rigidity of the floors in their own plane. Field measurements on a large number of building structures verified that in-plane deformations in the floor systems are small compared to the inter-storey horizontal displacements [43]. With the use of rigid floor diaphragms, the horizontal lateral loads acting at the floor levels of a building structure are directly transferred to the vertical structural elements (columns and shear walls). This results in three displacement degrees of freedom at each floor level (translations in two orthogonal directions and rotation about vertical direction), and in-plane displacements of the diaphragm can be expressed in terms of these displacements [14].

In modelling building structures, the rational way is to define a 'master node' at each rigid floor. Having three degrees of freedom (two translations and a rotation), the master node is located at the center of gravity of each floor. All the other nodes on that floor are called slaved nodes and their three displacement components (translation in x-direction, translation in y-direction and rotation about z-direction) can be represented using the displacements of the master node and the distance to the master node as in the following equations:

$$u_x^{(i)} = u_x^{(m)} - y^{(i)} u_{\theta z}^{(m)} \tag{4.1}$$

$$u_y^{(i)} = u_y^{(m)} + x^{(i)} u_{\theta z}^{(m)} \tag{4.2}$$

$$u_{\theta z}^{(i)} = u_{\theta z}^{(m)} \tag{4.3}$$

In the above equations, $u_x^{(i)}$, $u_y^{(i)}$ and $u_{\theta z}^{(i)}$ are the three displacement components of the slaved node, $u_x^{(m)}$, $u_y^{(m)}$ and $u_{\theta z}^{(m)}$ are the displacement components of the master node and $x^{(i)}$ and $y^{(i)}$ are the components of distance between master and slaved node at that floor. This method is called the master–slave technique [1]. A sketch of master and slaved nodes on a floor is given in Figure 4.1.

In dynamic analyses, it is assumed that the mass of each floor is lumped at a single node on the floor, which is generally the master node. The mass, m, is defined only in three degrees of freedom due to the constraining effect of the floor diaphragms. This approach is suggested by Chopra [22].

4.2 Modelling Nonplanar Shear Walls

Due to deficiencies in the two dimensional and the ´pseudo´ three dimensional lateral load analyses that are discussed in Chapter 2, three dimensional analysis should be performed for shear wall-frame structures having nonplanar shear wall assemblies. In this study, the modelling of nonplanar shear walls is examined in two parts:

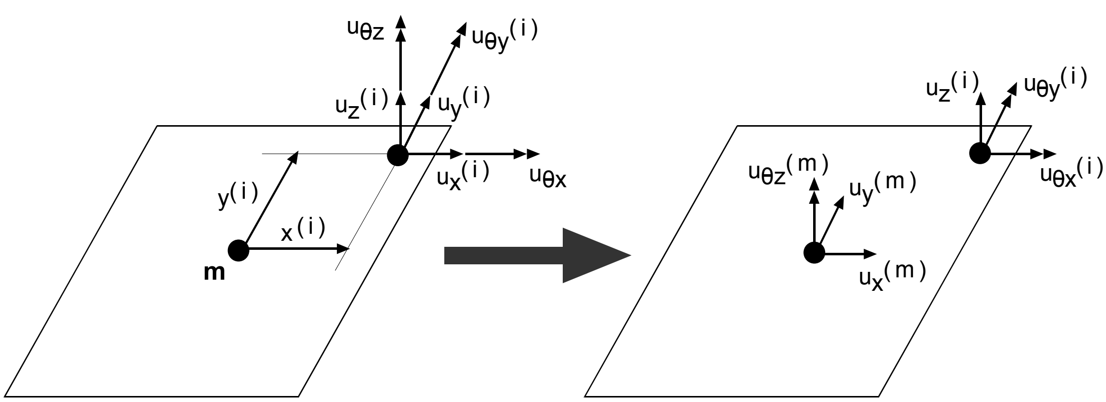


Figure 4.1 Master and Slaved Nodes at a Floor Level [43]

1. Open sections

2. Closed sections

Two different modelling methods are developed for open and closed section shear wall assemblies. These methods are based on the behavior of the assemblies in shear wall-frame structures subjected to lateral loads for which the rigid diaphragm floor assumption is valid. In the modelling studies, the conventional equivalent frame model is used with significant modifications. The translational and rotational response of the shear wall assemblies are considered separately and the actual behavior of the structure subjected to eccentric lateral loads is assumed to be obtained by the superposition of the two responses. In the modelling and verification studies, SAP2000 software is used. However, the proposed models can be implemented in any three dimensional frame analysis program having constraint option.

In the following two parts, the proposed models developed for open and closed section shear wall assemblies are presented. In the last part, the performance of these models is investigated by comparing the responses of the assemblies in static and dynamic loading.

4.2.1 Modelling of Open Section Shear Walls

It is a common assumption that due to the high in-plane stiffness of floor slabs, open section shear walls can be considered as thin-walled beams of non-deformable contour [73]. In modelling open section shear walls, each planar wall in the assembly is replaced with a column having the same mechanical properties of the wall as in the equivalent frame method. In order to ensure the vertical compatibility of the displacements, the rigid beams at floor levels are rigidly connected to each other at the corners. In addition, the ends of the rigid beams that are connected to each other are released (disconnected) from the connection joint only for torsional moments. In another words, the transfer of torsional moments between rigid beams is prevented. In Figure 4.2, the connection details of two orthogonal shear walls are given.

In three dimensional analysis of open shear wall assemblies modelled by the conventional equivalent frame model, serious errors occur especially in the analysis of assemblies subjected to torsion. The stiffness of the structural system becomes stiffer

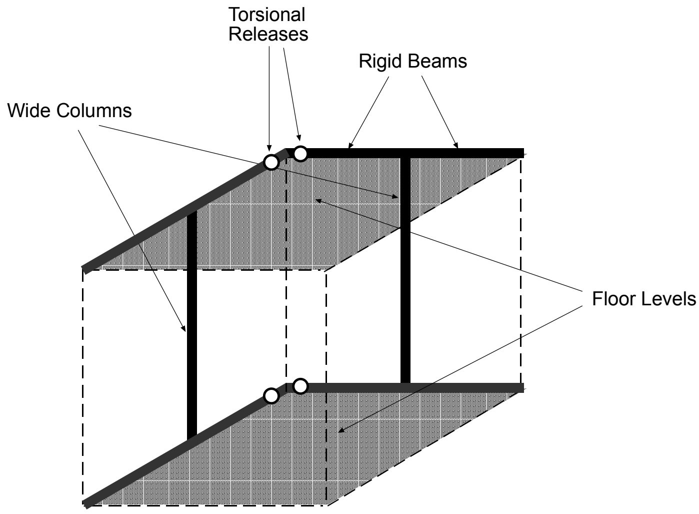


Figure 4.2 Connection Details of Two Orthagonal Shear Walls

than with finite element modelling. In the studies, it is observed that releasing the ends of the rigid beams from the connection joint decreases the torsional stiffness of the shear wall assembly significantly. This difference can be seen in the comparison studies of open section shear walls presented in the last part.

Several modelling studies are performed on the open section shear wall assemblies. The plans of the analyzed open section shear walls and the corresponding models developed by the proposed method are given in Figure 4.3 to 4.7.

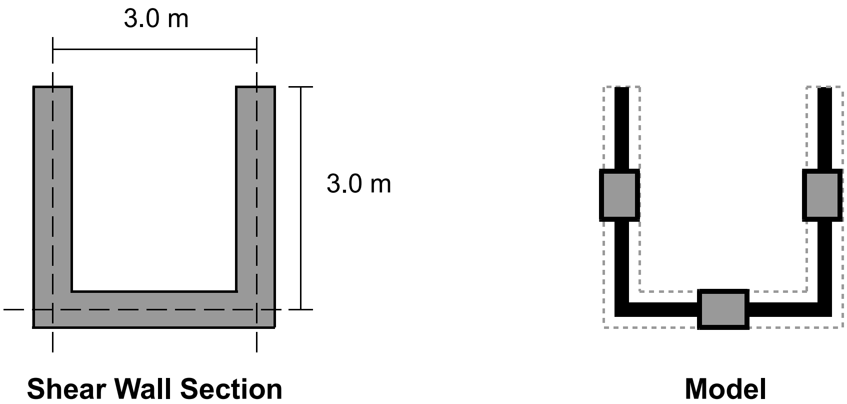


Figure 4.3 Plan of a U-Shaped Shear Wall Assembly and Its Proposed Model

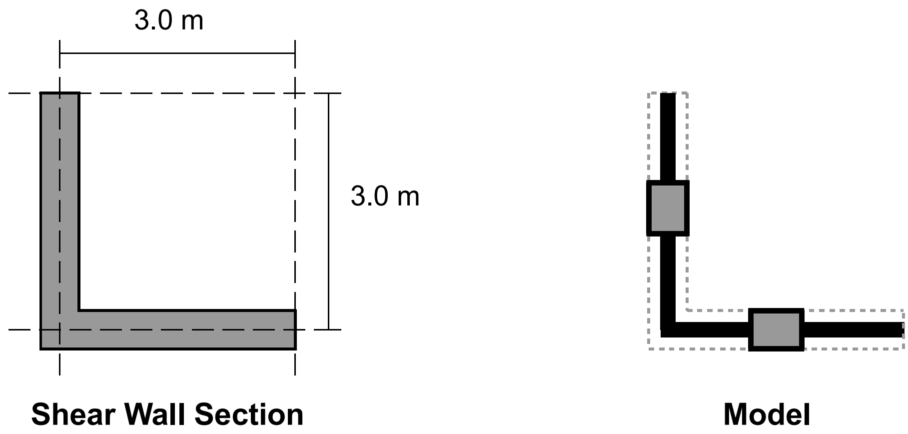


Figure 4.4 Plan of an L-Shaped Shear Wall Assembly and Its Proposed Model

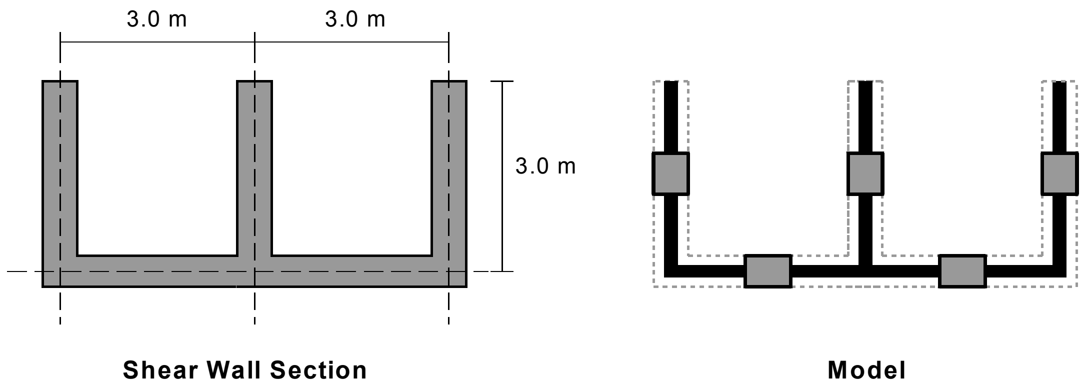


Figure 4.5 Plan of a W-Shaped Shear Wall Assembly and Its Proposed Model

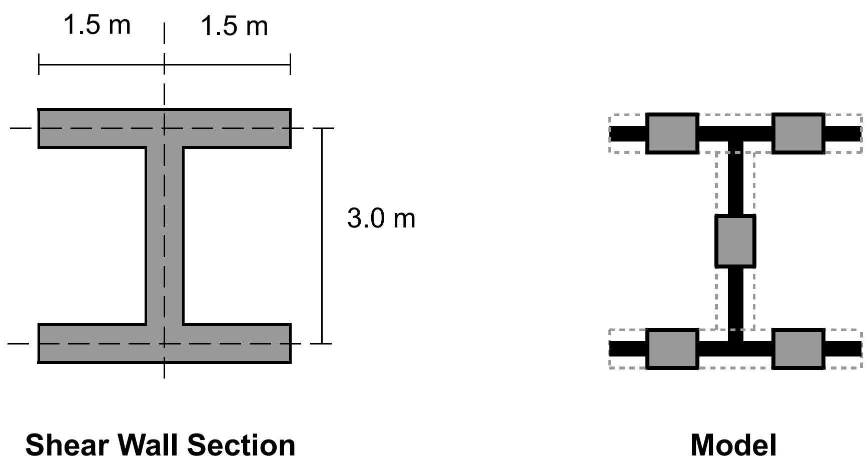


Figure 4.6 Plan of an H-Shaped Shear Wall Assembly and Its Proposed Model

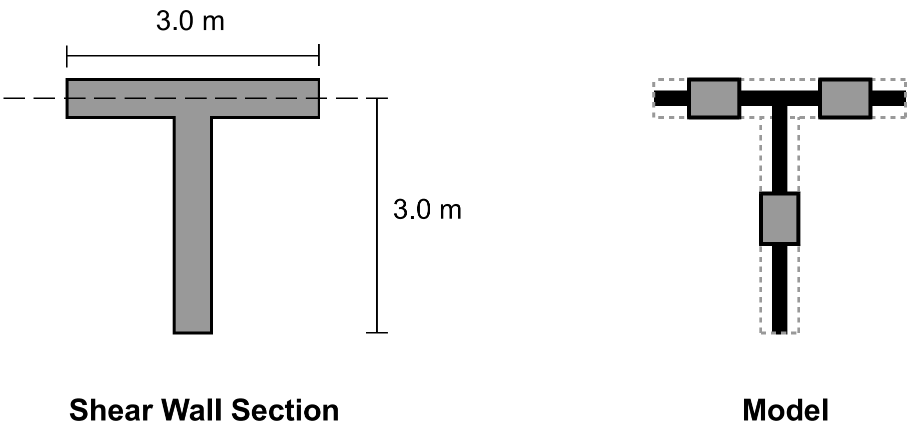


Figure 4.7 Plan of a T-Shaped Shear Wall Asembly and Its Proposed Model

4.2.2 Modelling of Closed Section Shear Walls

The proposed model for closed section shear wall assemblies is similar to the model developed for open section shear wall assemblies. The columns are placed at the walls' centroidal axes and assigned to have the same mechanical properties of the walls. Rigid beams are located at the floor levels and make rigid connections with each other. Similar to the previous model, the ends of the rigid beams are released at the connections only for torsional moments.

In the case of pure torsion, due to the rigid floor assumption, it is observed that rigid beams behave independently from the wide columns and make closed loops at floor levels. For this reason, the torsional stiffness of the model becomes much smaller than the torsional stiffness of the actual closed section assembly, as it is a summation of the torsional stiffnesses of disconnected wide columns in the model. This problem is also stated by Smith and Girgis [69]. They reported that the closed section shear walls modelled by the equivalent frame method become less stiff than with the finite element method.

The proposed model solves this problem by modifying the torsional constants of the wide columns by using the torsional constant of the shear wall section considered. The procedure has three steps:

1. Calculation of the torsional constant of the closed section (J_c)
2. Calculation of the torsional constants of the wide columns (J_i)

3. Calculation of the modified torsional constants of the wide columns using the following equation:

$$\bar{J}_i = \frac{J_c}{\sum_{k=1}^{n} J_k} \cdot J_i B_i \tag{4.4}$$

In the above equation, $\bar{J}_i$ is the modified torsional constant of the wide column, B_i is a constant depending on the horizontal distance between the centroid of the wide column and the centroid of the closed section and n is the total number of wide columns in the model. The value of B_i for the wide columns in a square shear wall model is 1.0. The calculation procedure of B_i values for the wide columns of a rectangular shear wall is given in Appendix.

The torsional constants (J_c) of square and rectangular solid cross sections can be evaluated by the following equations [21]:

$$J_c = 0.1406 b^4 \qquad \text{(square)} \tag{4.5}$$

$$J_c = bt^3 \left[\frac{1}{3} - 0.21 \frac{t}{b} \left(1 - \frac{t^4}{12 b^4} \right) \right] \qquad \text{(rectangle)} \tag{4.6}$$

In Eqn. 4.5, b is the dimension of one side of the square section. In Eqn. 4.6, b is the large and t is the small dimensions of the rectangular section. The plans of a square and a rectangular shear wall and their models are given in Figure 4.8 and 4.9.

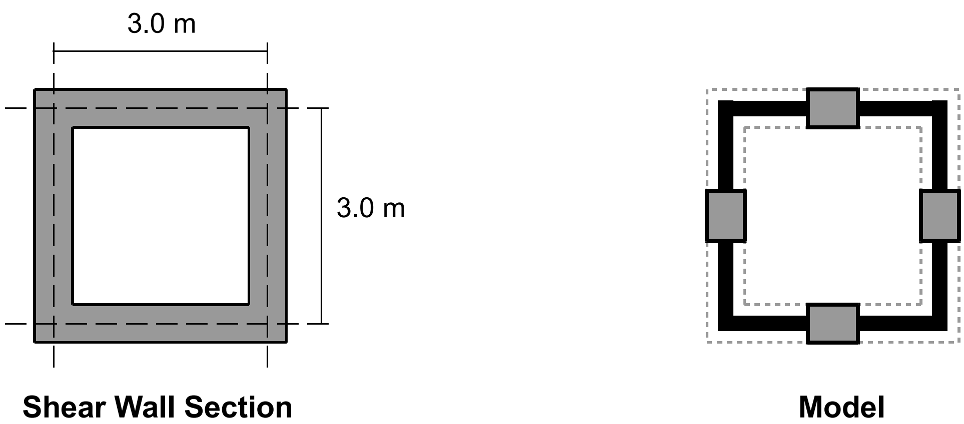


Figure 4.8 Plan of a Square Shear Wall Assembly and Its Proposed Model

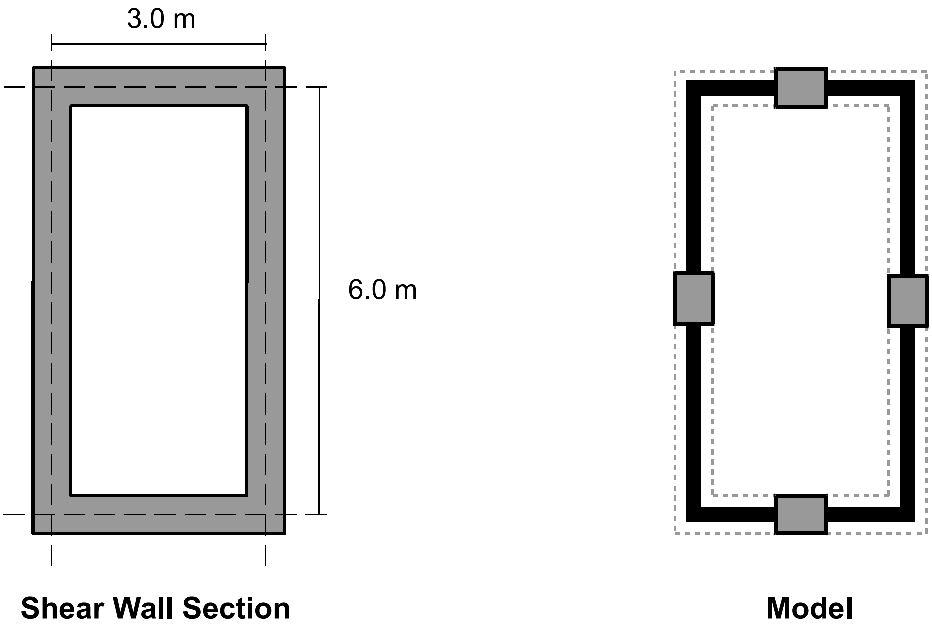


Figure 4.9 Plan of a Rectangular Shear Wall Assembly and Its Proposed Model

4.2.3 Comparison of the Proposed Models With Other Models

The performance of the proposed models are compared with the following shear wall modelling methods:

1. Modelling with SAP2000 shell elements
2. Modelling with ETABS wall elements
3. Conventional equivalent frame method

In the comparisons, the following single shear wall assemblies are taken into consideration:

1. U-shaped shear wall (SWS1) shown in Figure 4.3
2. L-shaped shear wall (SWS2) shown in Figure 4.4
3. W-shaped shear wall (SWS3) shown in Figure 4.5
4. H-shaped shear wall (SWS4) shown in Figure 4.6
5. T-shaped shear wall (SWS5) shown in Figure 4.7
6. Square shear wall (SWS6) shown in Figure 4.8
7. Rectangular shear wall (SWS7) shown in Figure 4.9

All assemblies have four stories with rigid diaphragms at the floor levels. The height of all stories is 3.0 m and the thickness of the shear walls is 0.25 m. Other dimensions of the shear wall assemblies are given in the related figures.

Two different types of analyses are performed in comparison studies of single shear wall assemblies:

1. Static lateral load analysis.
2. Dynamic analysis to obtain natural vibration periods of the assemblies.

In the static lateral load analyses, two different loading conditions are used as shown in Figure 4.10. In loading condition 1, the shear wall assemblies are subjected to axisymmetric lateral loads acting at floor levels. Each of the four loads is 100 t. In load condition 2, assemblies are subjected to pure torsions (out of plane moments) at the floor levels. The applied moments are 300 t.m. Modulus of elasticity and Poisson's ratio of concrete are taken as $2.531 \cdot 10^9$ kgf/m^2 and 0.20 respectively.

At the beginning of the comparison studies, an attempt was made to find the optimum number of shell elements to be used in the analyses performed by SAP2000. The four storey U-shaped shear wall assembly, SWS1, is analyzed for the two loading

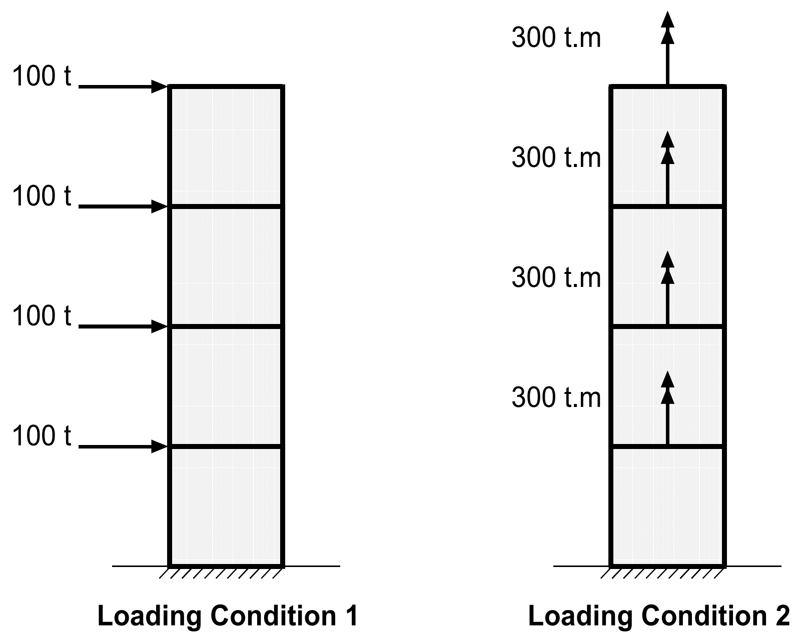


Figure 4.10 Loading conditions Used in the Analyses

Table 4.1 Top Storey Lateral Displacements of U Shaped Shear Wall Assembly for Loading Condition 1

Number of Elements in a Wall Module	Top Storey Lateral Displacement (m)
1 (1x1)	0.02139
4 (2x2)	0.02221
16 (4x4)	0.02251
64 (8x8)	0.02261
256 (16x16)	0.02264
1024 (32x32)	0.02266

conditions specified above, in which meshes having different numbers of shell elements are used. For each loading condition, top storey displacements are obtained for the considered shell element meshes. The results of the analyses are given in Table 4.1 and 4.2. It is observed that there is a convergence in lateral displacement and rotation values. In the view of the results, the optimum number of shell elements to be used in modelling a planar wall module located between two floor levels is determined as 16 (4x4).

SAP2000 software is used in the analyses of the single shear wall assemblies modelled by the proposed methods and the conventional equivalent frame method. The section properties of rigid beams (cross sectional area, moment of inertia, torsional constant, etc.) are assigned as very large values in the program.

The lateral displacements and rotations of the floors of the single shear wall assemblies obtained in the analyses by using four different modelling techniques are given in Figure 4.11 to 4.24. Good agreement is obtained between the results of the proposed models and the SAP2000 shell element model. The maximum percent difference in top storey displacement values between the proposed models and the SAP2000 shell element model for loading condition 1 is 7.26%. This value is 5.59% between ETABS

Table 4.2 Top Storey Rotation of U Shaped Shear Wall Assembly for Loading Condition 2

Number of Elements in a Wall Module	**Top Storey Rotation (rad)**
1 (1x1)	0.09977
4 (2x2)	0.10124
16 (4x4)	0.10222
64 (8x8)	0.10259
256 (16x16)	0.10270
1024 (32x32)	0.10274

wall element model and the SAP2000 shell element model. For loading condition 2, the maximum percent difference in top storey rotation values between the proposed models and the SAP2000 shell element model is obtained 11.43%, which is 3.86% between the ETABS wall element model and SAP2000 shell element model.

The deficiency in the conventional equivalent frame method is especially observed in the results of the analyses in which loading condition 2 is considered. Open section shear wall assemblies modelled by the conventional equivalent frame method have a much more rigid behavior than the other models (Figures 4.12, 4.14, 4.16, 4.18). Conversely, the closed sections modelled by the conventional equivalent frame method behave less rigid compared with the other models (Figure 4.22, 4.24).

The first natural vibration period is quite important especially in equivalent lateral load analysis of structural systems. For this reason, in dynamic analyses the first natural vibration periods of the shear wall assemblies are obtained for each modelling technique. In the analyses, the unit mass of concrete is taken as 255 kg/m^3. The rigid beams used in the proposed methods and conventional equivalent frame method are considered to be massless structural elements.

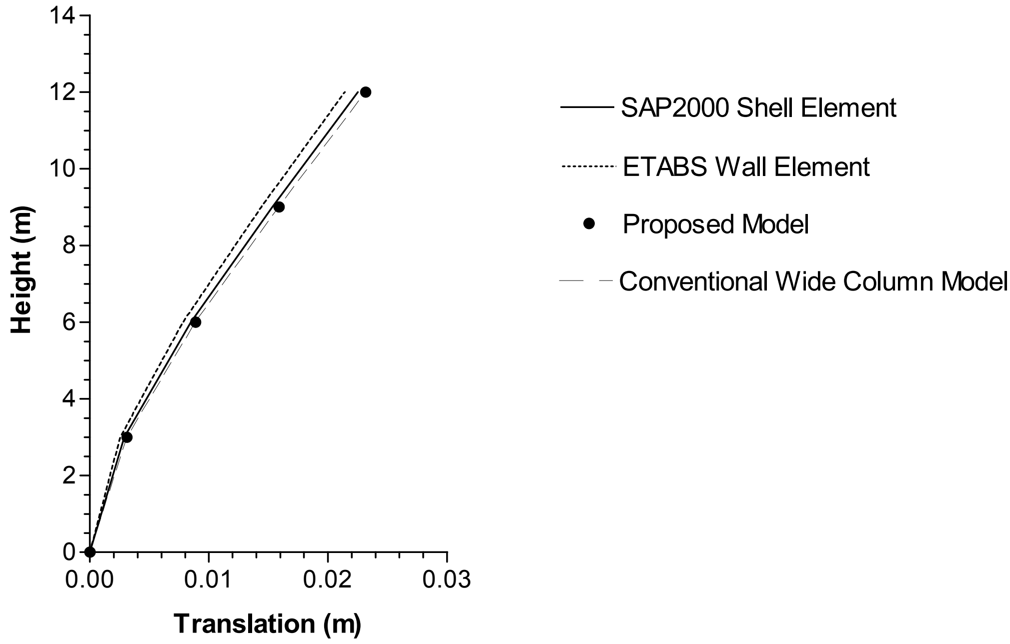


Figure 4.11 Lateral Displacement Graph SWS1 for Loading Condition 1

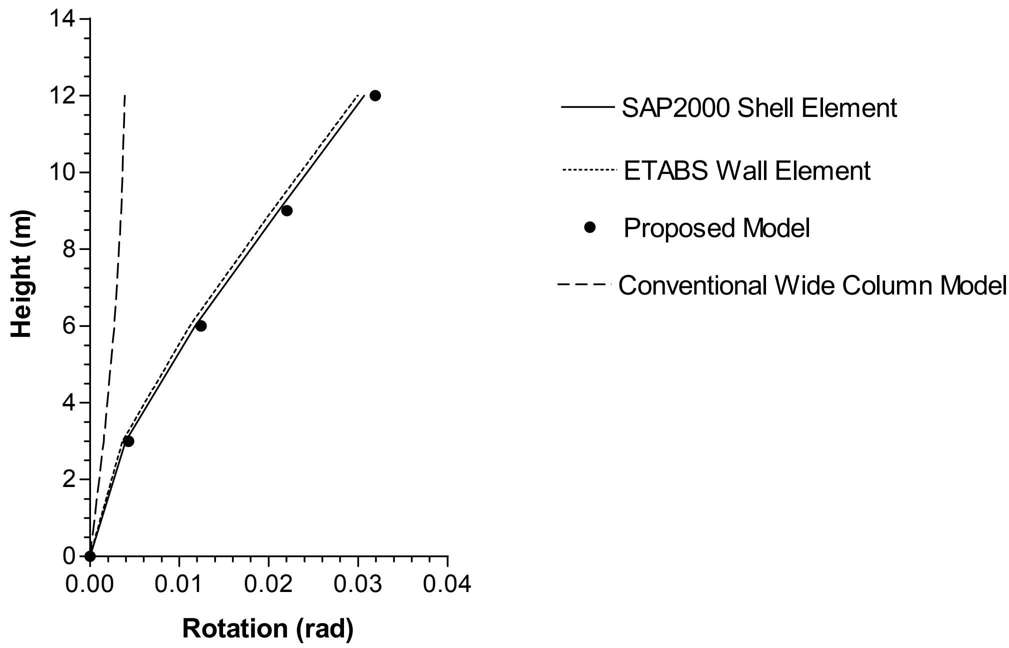


Figure 4.12 Floor Rotation Graph of SWS1 for Loading Condition 2

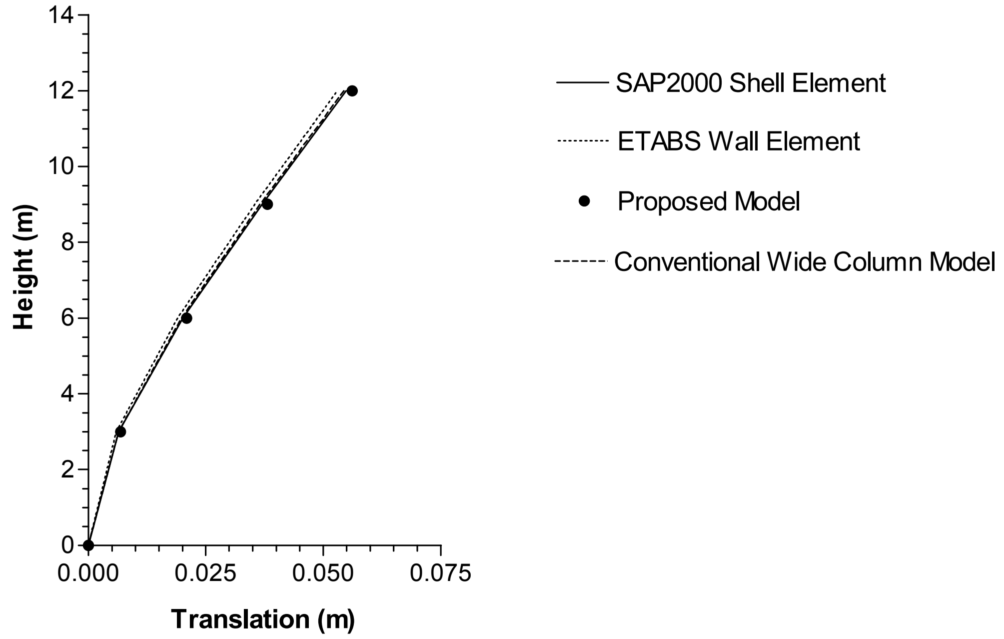


Figure 4.13 Lateral Displacement Graph of SWS2 for Loading Condition 1

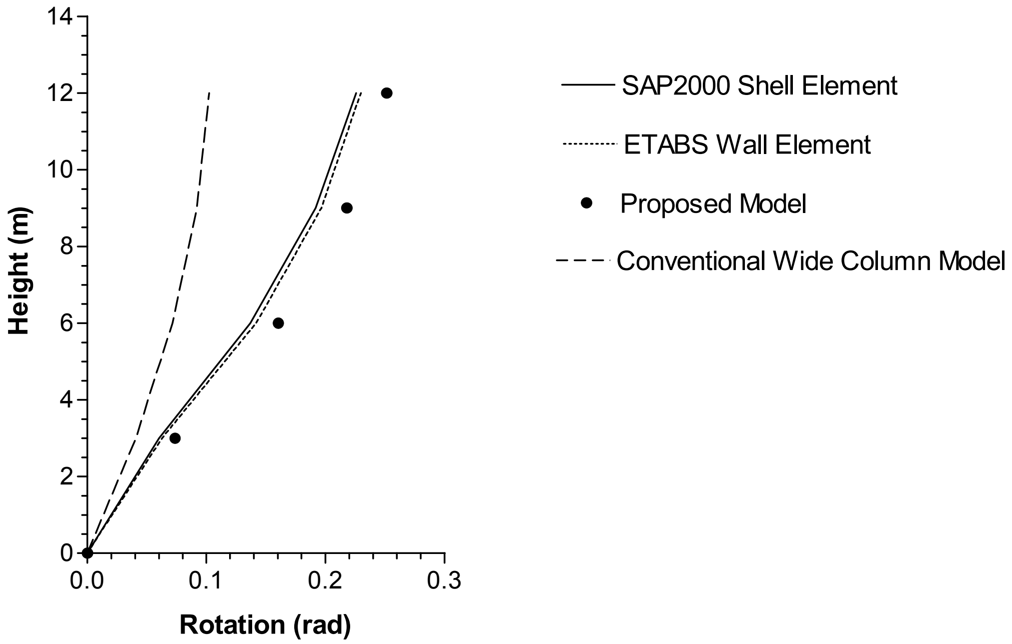


Figure 4.14 Floor Rotation Graph of SWS2 for Loading Condition 2

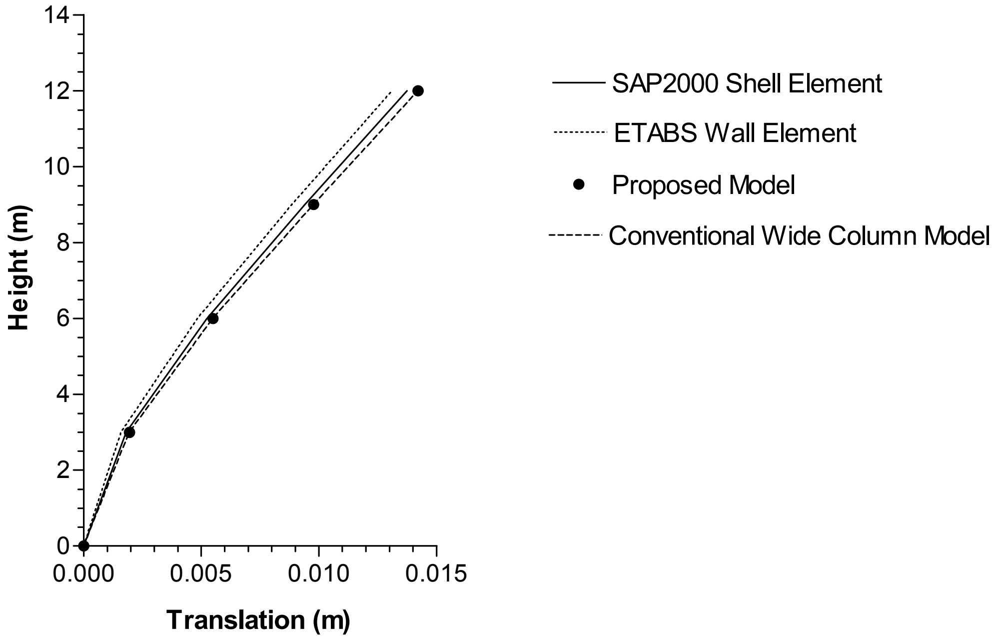


Figure 4.15 Lateral Displacement Graph of SWS3 for Loading Condition 1

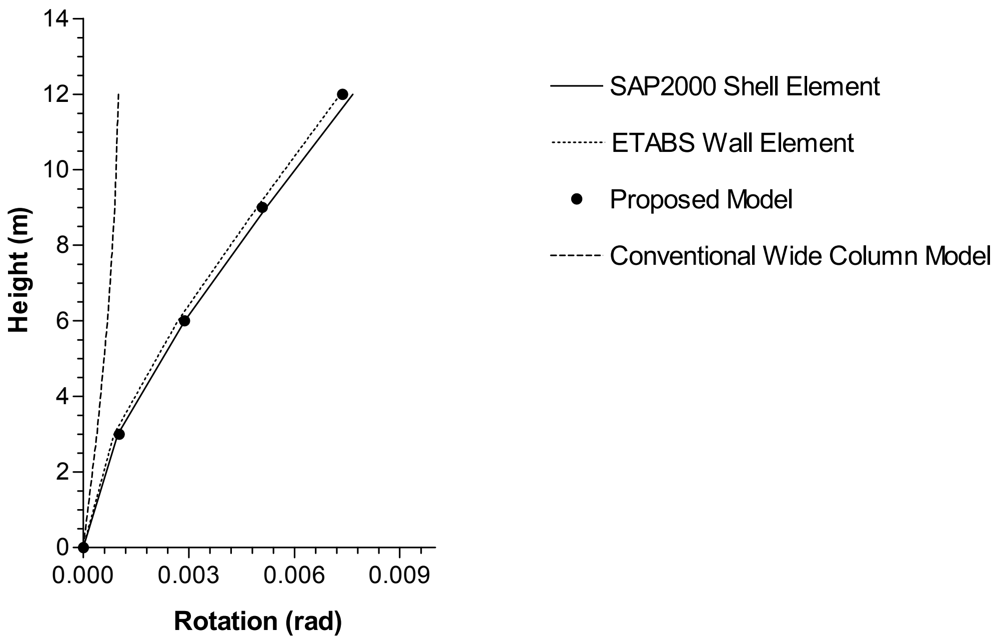


Figure 4.16 Floor Rotation Graph of SWS3 for Loading Condition 2

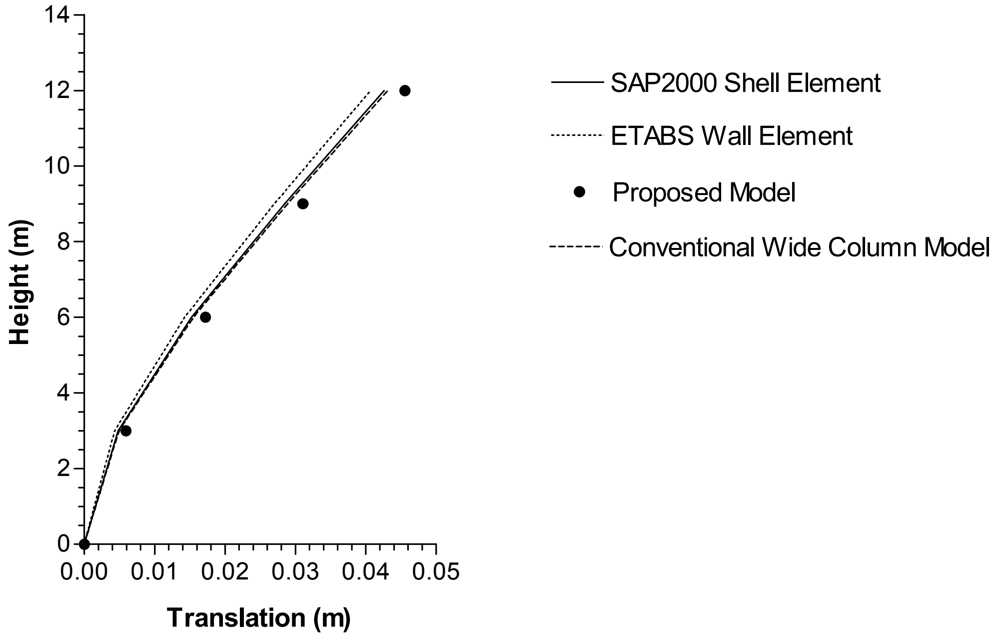


Figure 4.17 Lateral Displacement Graph of SWS4 for Loading Condition 1

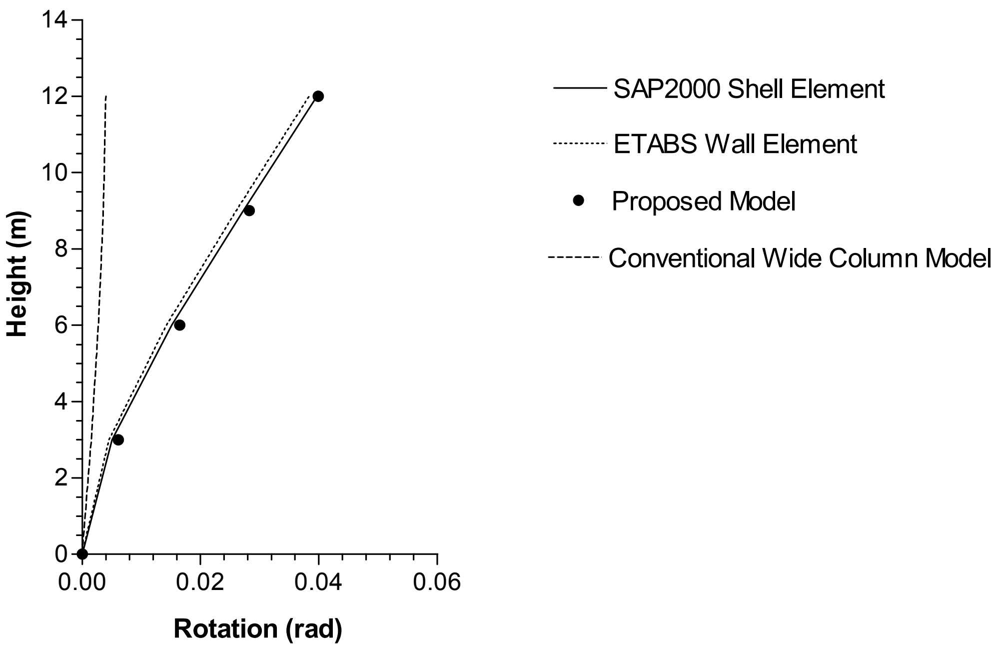


Figure 4.18 Floor Rotation Graph of SWS4 for Loading Condition 2

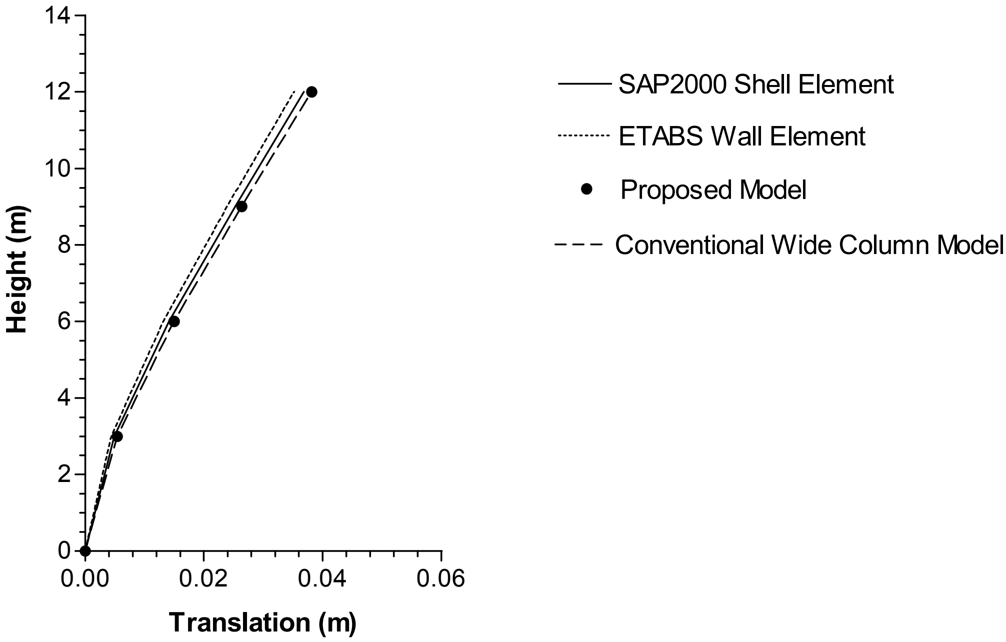


Figure 4.19 Lateral Displacement Graph of SWS5 for Loading Condition 1

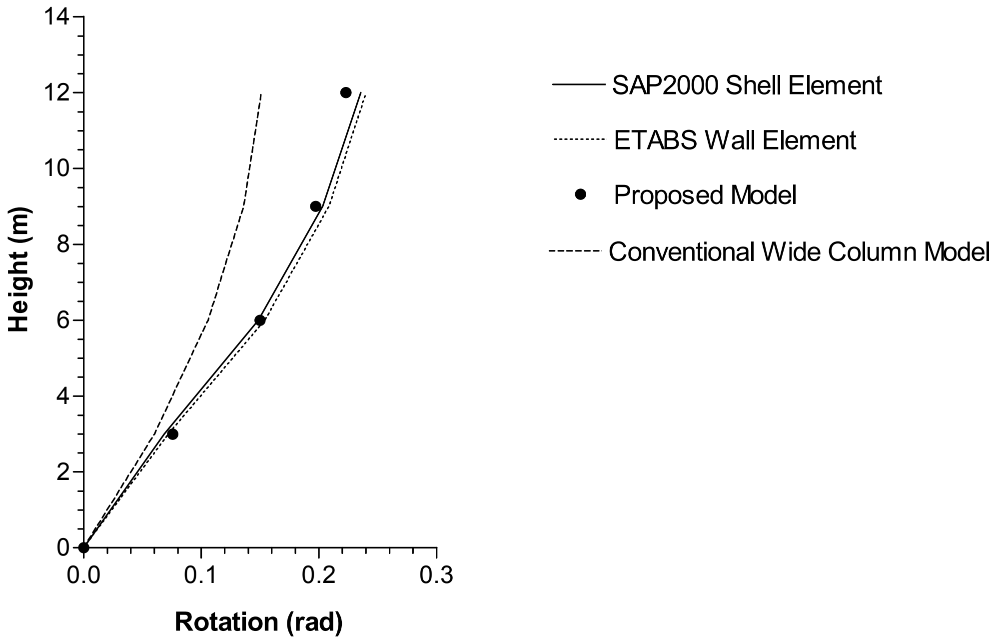


Figure 4.20 Floor Rotation Graph of SWS5 for Loading Condition 2

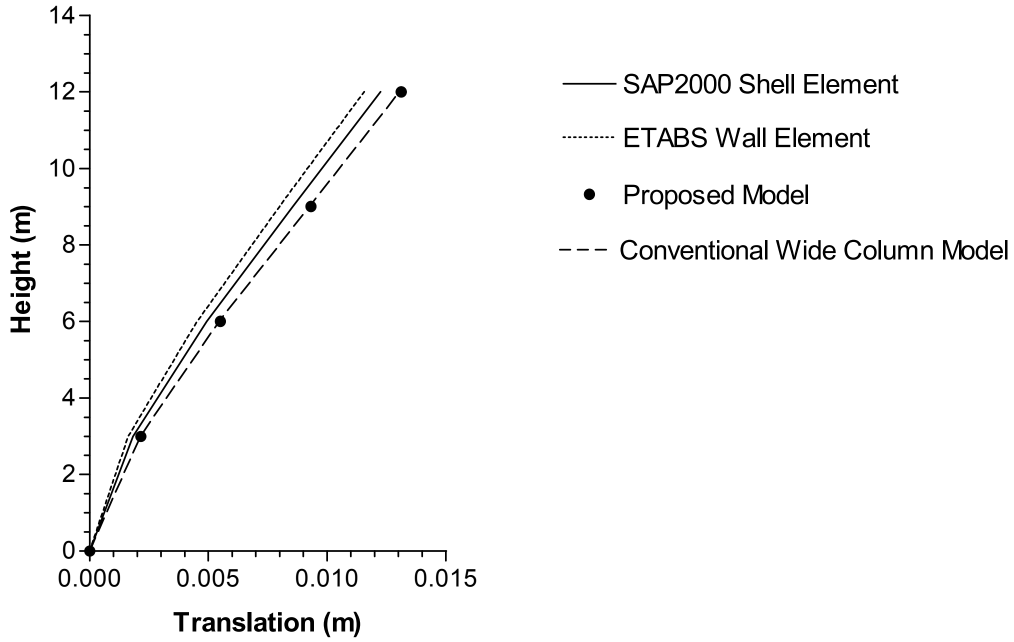


Figure 4.21 Lateral Displacement Graph of SWS6 for Loading Condition 1

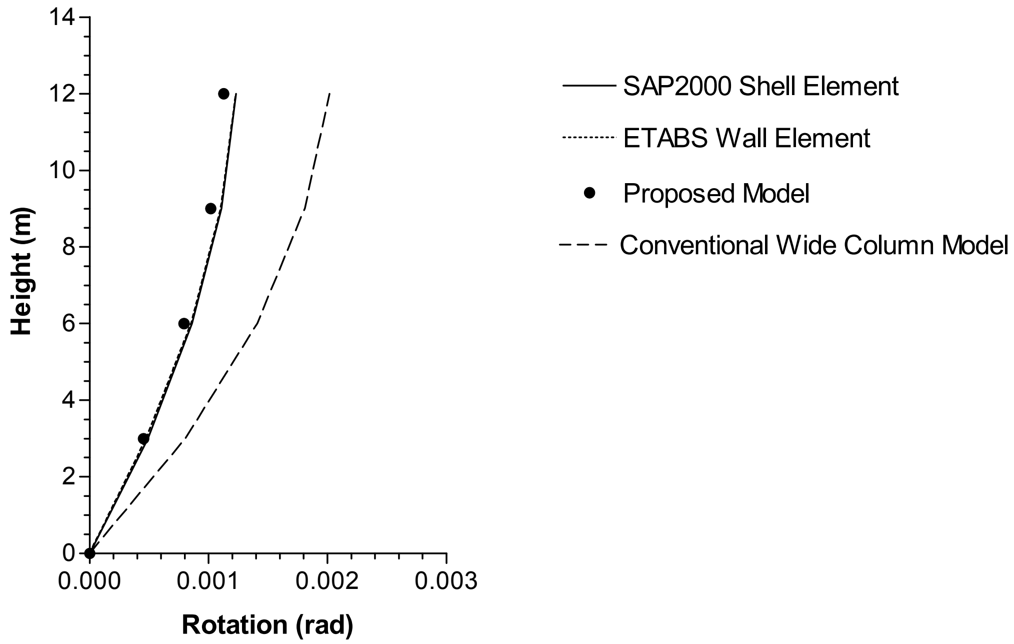


Figure 4.22 Floor Rotation Graph of SWS6 for Loading Condition 2

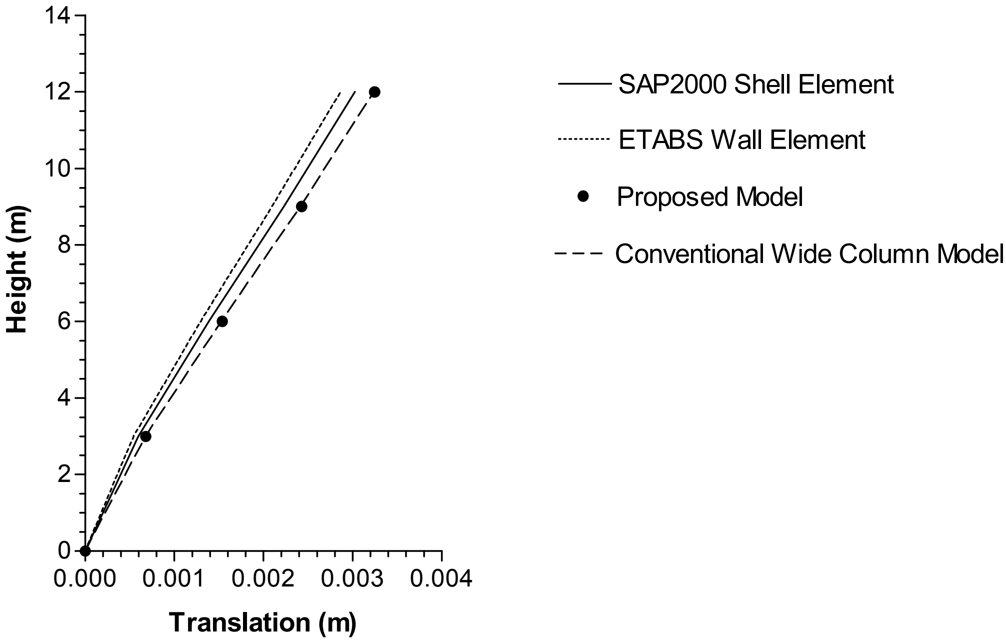


Figure 4.23 Lateral Displacement Graph of SWS7 for Loading Condition 1

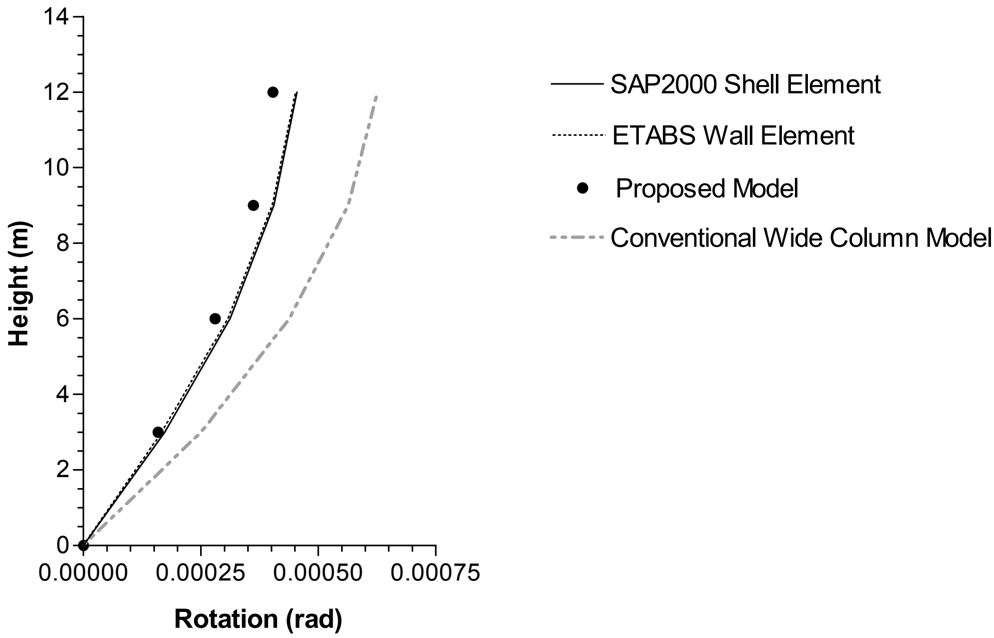


Figure 4.24 Floor Rotation Graph of SWS7 for Loading Condition 2

Table 4.3 Comparison of the First Natural Vibration Periods of Shear Wall Assemblies Modelled by the Proposed Methods and SAP2000 Shell Elements

Shear Wall	Proposed Models (s)	SAP2000 Shell El.(s)	Percent Dif.(%)
SWS1	0.170916	0.172197	0.74
SWS2	0.235385	0.254182	7.40
SWS3	0.118069	0.121619	2.92
SWS4	0.125940	0.118603	6.19
SWS5	0.201886	0.221229	8.74
SWS6	0.079409	0.075324	5.42
SWS7	0.080644	0.077200	4.46

A comparison of the first natural vibration periods of the seven shear wall assemblies (SWS1 to SWS7) modelled by the proposed models and SAP2000 shell elements is given in Table 4.3. The maximum percent difference between the two models is 8.74%, which shows good agreement. In Table 4.4, the comparison of the first natural vibration periods of the assemblies that are modelled by the ETABS wall element and the SAP2000 shell element is given. The maximum percent difference between the two models is 21.86%, which is much greater than the percent difference between the proposed models and SAP2000 shell elements. A comparison of the results obtained by the conventional equivalent frame model and SAP2000 shell element is given in Table 4.5. Similar to static lateral load analysis, the conventional equivalent frame model gives serious errors in dynamic analysis.

Table 4.4 Comparison of the First Natural Vibration Periods of Shear Wall Assemblies Modelled by ETABS Wall Elements and SAP2000 Shell Elements

Shear Wall	ETABS Wall El. (s)	SAP2000 Shell El.(s)	Percent Dif.(%)
SWS1	0.187700	0.172197	9.00
SWS2	0.309758	0.254182	21.86
SWS3	0.115558	0.121619	4.98
SWS4	0.119900	0.118603	1.09
SWS5	0.258935	0.221229	17.04
SWS6	0.075600	0.075324	0.37
SWS7	0.073673	0.077200	4.57

Table 4.5 Comparison of the First Natural Vibration Periods of Shear Wall Assemblies Modelled by the Conventional Equivalent Frame Method and SAP2000 Shell Elements

Shear Wall	Conv. Eq. Fr. M. (s)	SAP2000 Shell El.(s)	Percent Dif.(%)
SWS1	0.090002	0.172197	47.73
SWS2	0.156803	0.254182	38.31
SWS3	0.091134	0.121619	25.07
SWS4	0.121469	0.118603	2.42
SWS5	0.171584	0.221229	22.44
SWS6	0.079206	0.075324	5.15
SWS7	0.080014	0.077200	3.65

CHAPTER 5

VERIFICATION STUDIES

Verification studies are performed in two parts. In the first part, five groups of shear wall-frame building structures having different floor plans are considered. The three methods

(1) equivalent lateral load analysis

(2) response spectrum analysis and

(3) time history analysis

are used in the lateral load analysis of these building structures. Each building structure type is considered to have 3, 6, 9, 12 and 15 storeys.

The performance of the proposed shear wall models in equivalent lateral load analyses of sample building structures is determined by comparing the results obtained using the proposed models and

(a) shell elements of SAP2000

(b) wall elements of ETABS.

The validity of the proposed models in calculating the natural vibration periods of the sample building structures is determined by comparing the values obtained using SAP2000 shell and ETABS wall elements. In response spectrum and time history analyses, a comparison is made between SAP2000 shell elements and the proposed models.

In all comparison studies, the planar shear wall modules between two floor levels are divided into 16 (4x4) elements in the analyses where shear walls are modelled by SAP2000 shell elements and the results obtained in these analyses are assumed to be correct. The modulus of elasticity and Poisson's ratio of concrete are taken as $2.531 \cdot 10^9$ kgf/m^2 and 0.20 respectively.

In the second part of the verification studies, a number of shear wall and shear wall-frame structures, which were modelled and analyzed by several authors, are taken into consideration. The results of the analyses obtained in these studies are compared with the results in which the considered structures are modelled by the proposed methods.

5.1 Performance of the Proposed Models in the Analysis of Sample Shear Wall-Frame Structures

In this part, five groups of shear wall-frame building structures are taken into consideration. Their floor plans are given in Figures 5.1 to 5.5. The analyses are performed on 3, 6, 9, 12 and 15 storey sample building structures. The code BSi-j is used to represent the type of the building structure and the total number of storeys. For example, BS2-12 corresponds to the second type of building structure with 12 storeys.

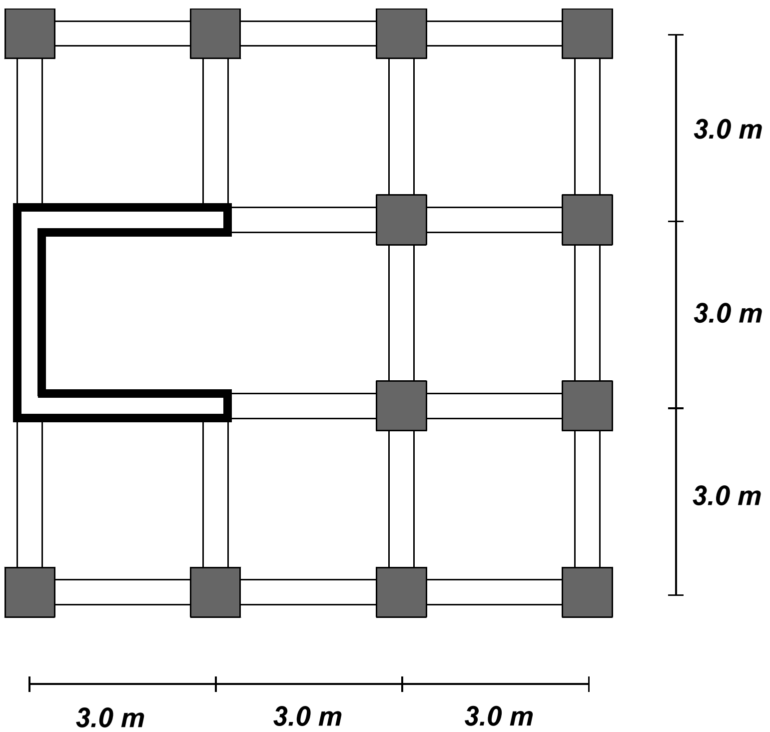


Figure 5.1 Floor Plan of Building Structure BS1

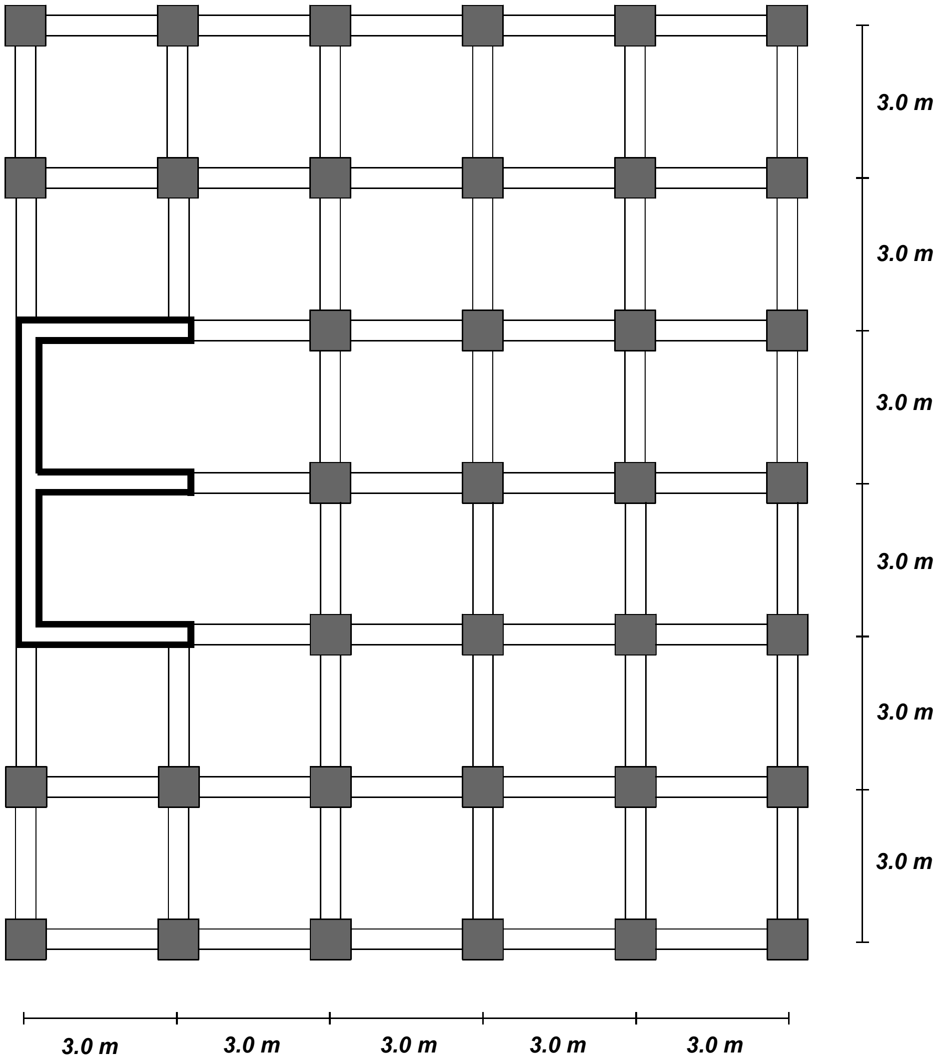


Figure 5.2 Floor Plan of Building Structure BS2

As stated above, the nonplanar shear wall assemblies of these building structures are modelled by the proposed models, shell elements of SAP2000 and wall elements of ETABS. The results of the analyses of sample building structures, in which SAP2000 shell elements are used, are assumed to give correct results.

In the comparison studies, three different analysis techniques, which are explained in Chapter 2, are considered: equivalent lateral load analysis, response spectrum analysis and time history analysis. In addition, the three natural vibration periods of the sample structures are also compared.

In the determination of the performance of the proposed models in response spectrum and time history analyses of the sample structures, comparisons are made between

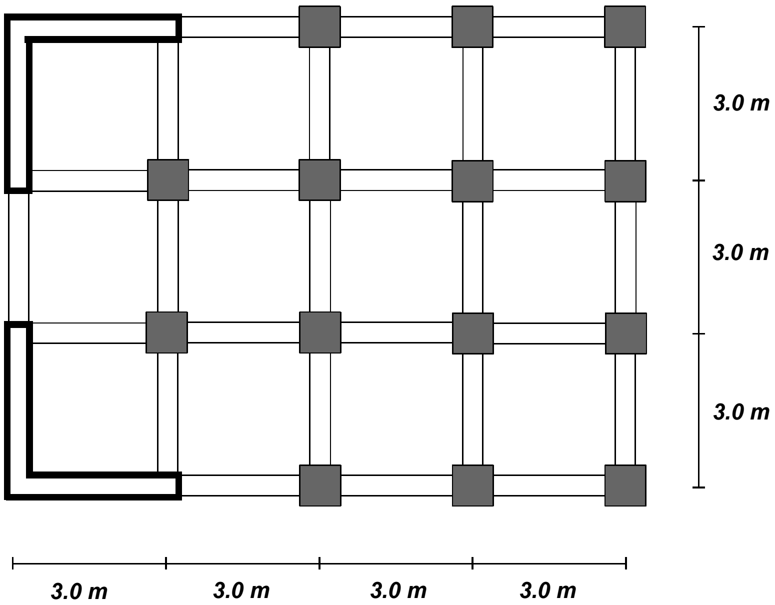


Figure 5.3 Floor Plan of Building Structure BS3

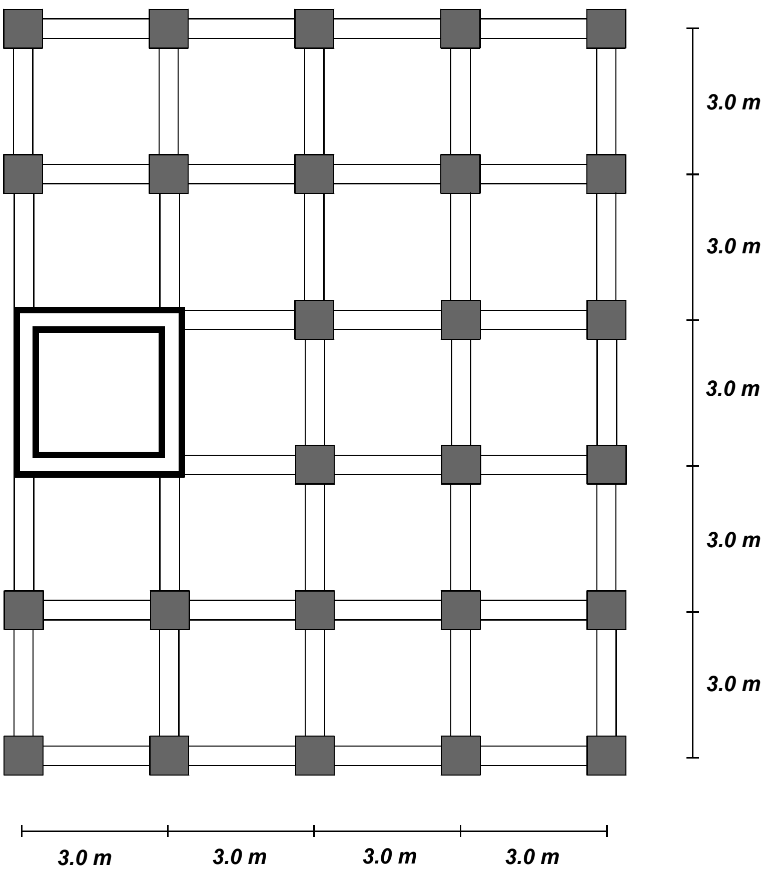


Figure 5.4 Floor Plan of Building Structure BS4

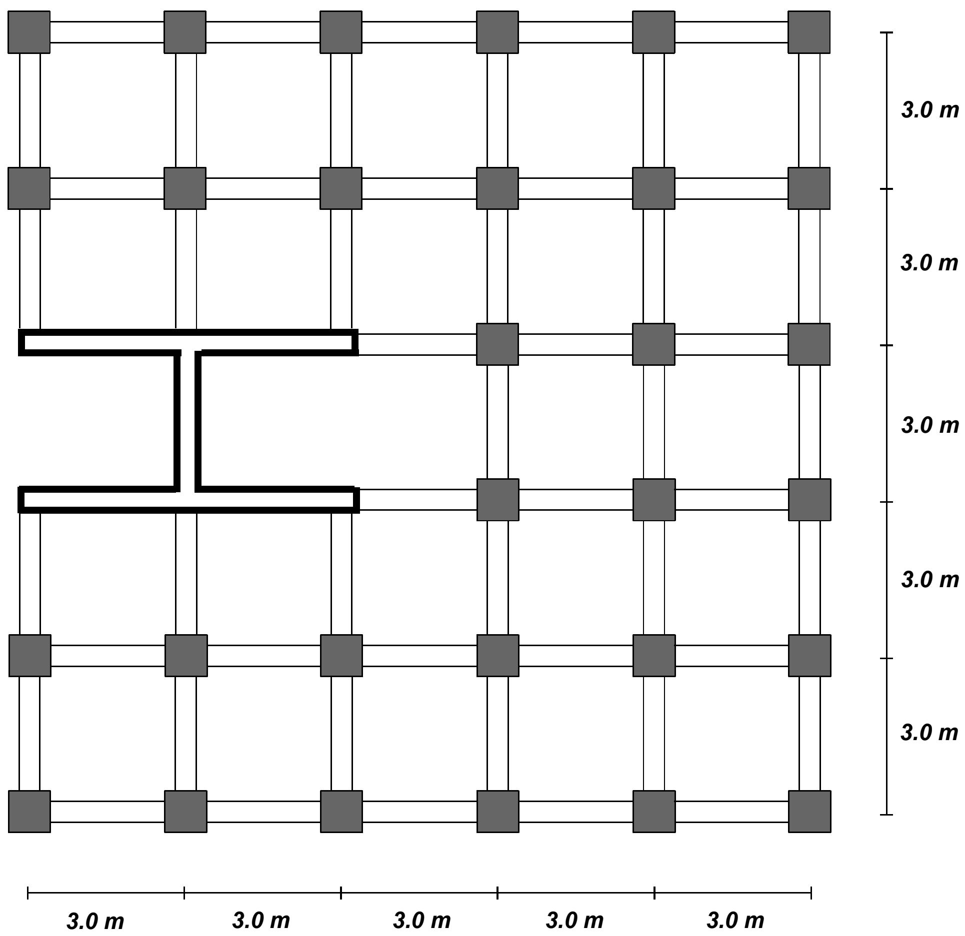


Figure 5.5 Floor Plan of Building Structure BS5

the proposed models and SAP2000 shell elements.

The next section presents a comparison of the results obtained by the equivalent lateral load analysis of the sample building structures.

5.1.1 Performance of the Proposed Models in Equivalent Lateral Load Analysis

The nonplanar shear wall assemblies of the sample building structures are modelled by SAP2000 shell elements, ETABS wall elements and the proposed models (for open and closed shear walls). In the modelling studies, the rigid diaphragm floor assumption is taken into consideration.

According to the Turkish Earthquake Code [33], the lateral loads applied at each floor level of a building structure are calculated by the following equation:

$$F_i = (V_t - \Delta F_N)\frac{w_i H_i}{\sum_{j=1}^{N}(w_j H_j)} \tag{5.1}$$

In this equation, F_i is the equivalent lateral load acting at the i-th floor, V_t is the total base shear, ΔF_N is the additional lateral load acting at the top floor (for the buildings higher than 25 m), and w_i and H_i are the weight and height of the i-th floor respectively.

In the comparisons, the above equation is used in a simpler form

$$F_i = A * \frac{w_i H_i}{\sum_{j=1}^{N}(w_j H_j)} \tag{5.2}$$

where A is taken as 600 t for 3-storey buildings, 2100 t for 6-storey buildings, 4500 t for 9-storey buildings, 7800 t for 12-storey buildings and 12000 t for 15-storey buildings. The effect of the additional lateral load for structures higher than 25 m is not taken into consideration. For all sample building structures, the height of a typical storey is taken as 3.0 m and the dimensions of all columns and beams are taken as 0.30 x 0.30 m. The thickness of the shear walls is assumed to be 0.25 m.

In the linear elastic analysis of building structures that are subjected to lateral static loads, an eccentric loading condition can be represented by the superposition of an axisymmetric translational loading and a torsional loading at floor levels. The torsion acting at a floor level can be obtained by multiplying the floor load acting at that floor with the eccentricity.

In the equivalent lateral load analyses of the sample structures, a general eccentric loading which is 3.0 m to the floor centroid, is considered. In the first loading condition, the floor loads that are obtained by Eq. 5.2 are assumed to act axisymmetrically on x-direction of the sample structures. In the second loading condition, each floor of the building structure is subjected to pure torsion and the out of plane moments acting at the floor levels are assumed to be computed by the following equation:

$$M_i = 3 \cdot F_i \tag{5.3}$$

The results of the equivalent lateral load analysis of the sample building structures

Table 5.1 Comparison of Floor Displacements of BS1-3 Obtained by the Two Loading Conditions

Floor Level	SAP2000 S.E.	ETABS W.E.	Proposed Model
	Translation(m)	**Translation(m)**	**Translation(m)**
1	0.00377	0.003276	0.00406
2	0.01074	0.009702	0.01119
3	0.01874	0.017087	0.01903
	Rotation(rad)	**Rotation(rad)**	**Rotation(rad)**
1	0.00236	0.002162	0.00256
2	0.00585	0.005536	0.00616
3	0.00892	0.008617	0.00925

are given in the following parts. For each sample building, (a) floor displacements (translation and rotation) are given for the two loading conditions and (b) the resultant shear forces and out-of-plane moments at the floor levels of the shear wall assemblies are presented. For three-storey building structures, the results are presented in tabular form. The results of analyses of the building structures having more than 3 storeys are given in graphic form. For each building type, two building structures of that type with a different number of storeys are chosen and the results of the analyses of these buildings are presented. For example, for BS1 type building structures, the comparison of the analyses results of BS1-3 and BS1-9 building structures are presented.

5.1.1.1 Building Structure Type BS1

In Table 5.1, a comparison of the floor translations and floor rotations of the three storey BS1 type building structure (BS1-3) is given for the first and second loading conditions. In Table 5.2, the comparison of the resultant shear forces and out-of-plane moments at the floor levels of the shear wall assembly obtained by the three methods are tabulated.

Table 5.2 Comparison of Resultant Shear Forces and Out-of-Plane Moments at the Shear Wall Assembly in BS1-3 for the Two Loading Conditions

Floor Level	SAP2000 S.E.	ETABS W.E.	Proposed Model
	F_x (t)	**F_x (t)**	**F_x (t)**
0	-580.995	-583.780	-579.051
1	-472.425	-474.170	-471.704
2	-263.609	-264.82	-263.045
	M_z (t.m)	**M_z (t.m)**	**M_z (t.m)**
0	-1153.681	-1244.940	-1094.180
1	-780.199	-825.509	-753.338
2	-209.095	-228.948	-200.158

In Figure 5.6, a comparison of the floor translations of the nine storey BS1 type building structure (BS1-9) is given for the first loading condition and in Figure 5.7, a comparison of the floor rotations of the considered building structure is given for the second loading condition

A comparison of the resultant shear forces in the loading direction at the floor levels of the shear wall assembly for the first loading condition is given in Figure 5.8. In Figure 5.9, a comparison of resultant out-of-plane moments at the floor levels of the shear wall assembly for the second loading condition is given.

5.1.1.2 Building Structure Type BS2

A comparison of the results of two different BS2 type structures (BS2-6 and BS2-15) is presented in this part. In Figure 5.10, a comparison of the floor translations of BS2-6 building structure is given for the first loading condition and in Figure 5.11, the floor rotations of the considered building structure are given for the second loading condition

A comparison of the resultant shear forces in the loading direction at the floor levels

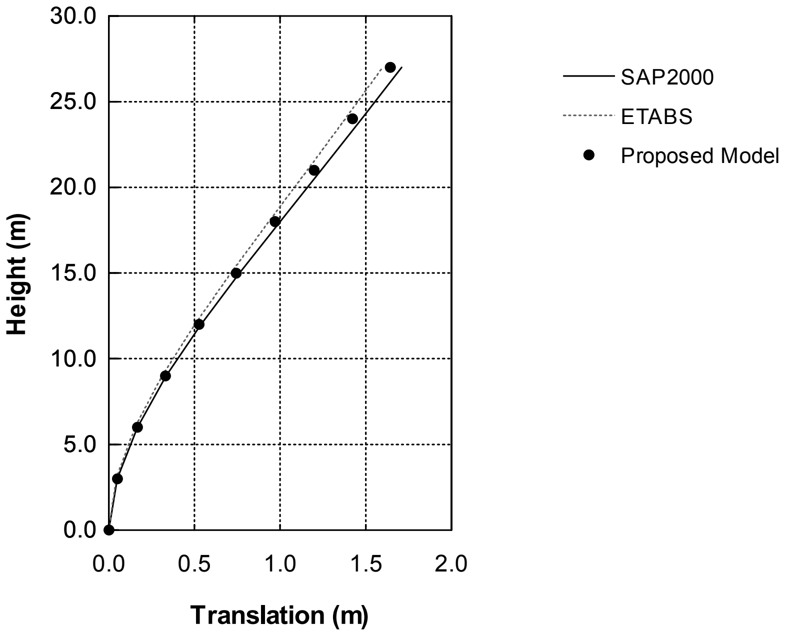


Figure 5.6 Comparison of Floor Displacements of BS1-9 for Loading Condition 1

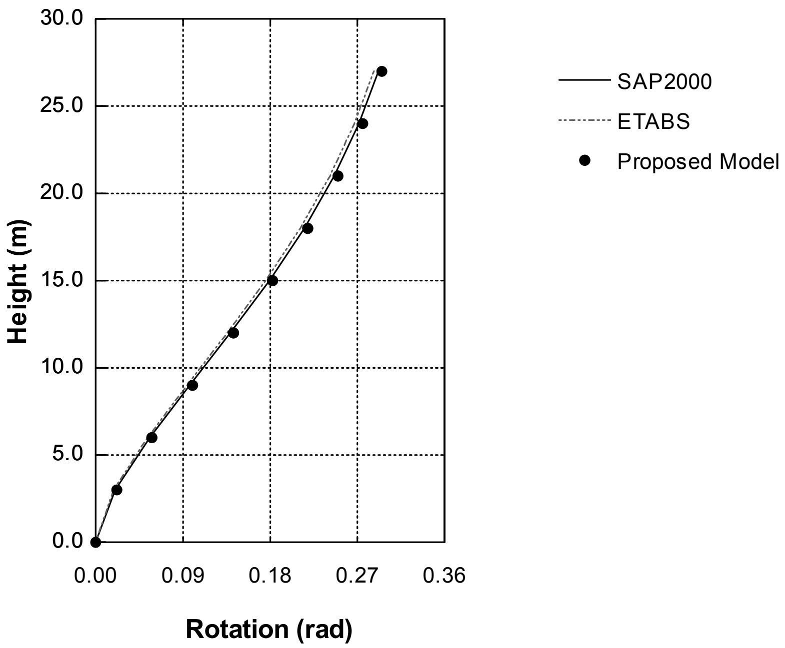


Figure 5.7 Comparison of Floor Rotations of BS1-9 for Loading Condition 2

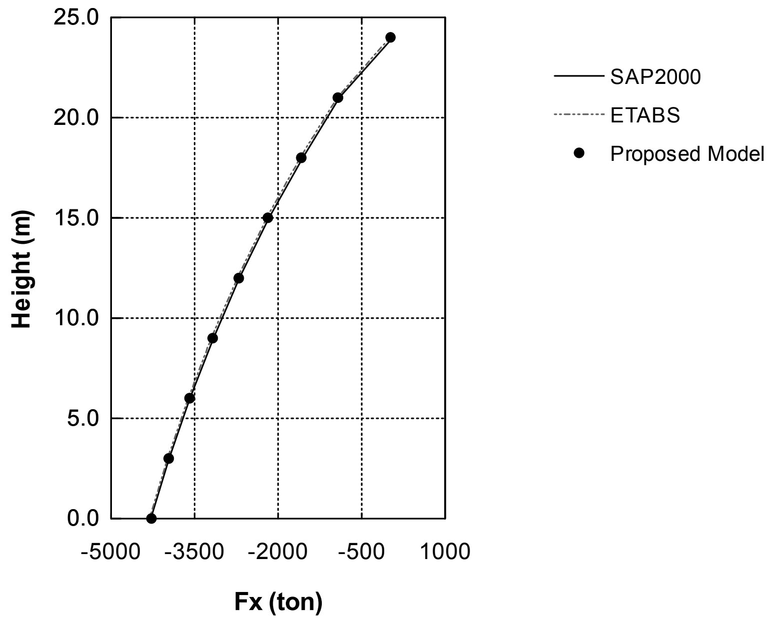


Figure 5.8 Comparison of Resultant Shear Forces at Floor Levels of the Shear Wall Assembly of BS1-9 for Loading Condition 1

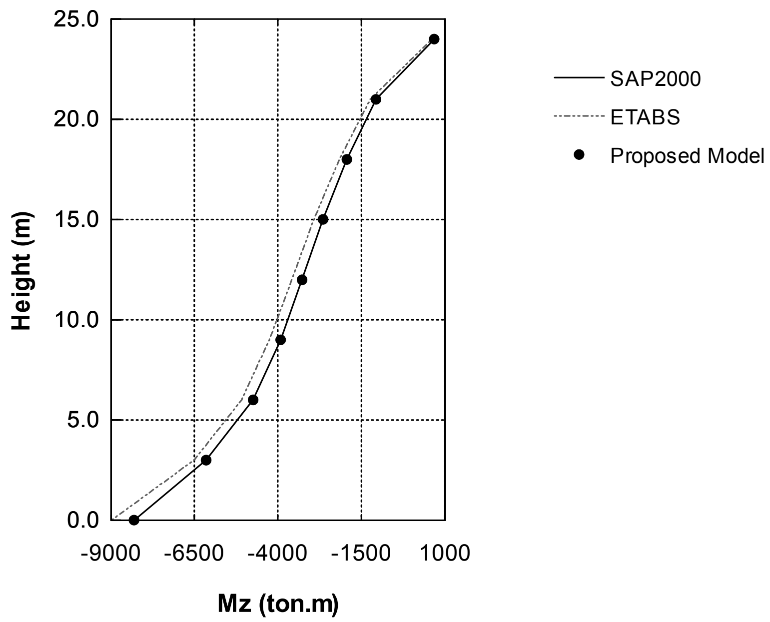


Figure 5.9 Comparison of Resultant Out-of-Plane Moments at Floor Levels of the Shear Wall Assembly of BS1-9 for Loading Condition 2

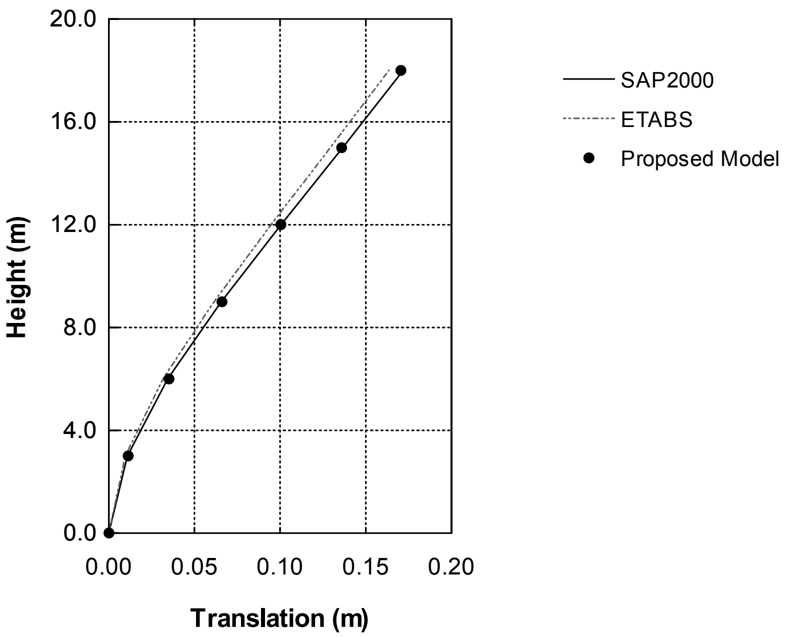


Figure 5.10 Comparison of Floor Displacements of BS2-6 for Loading Condition 1

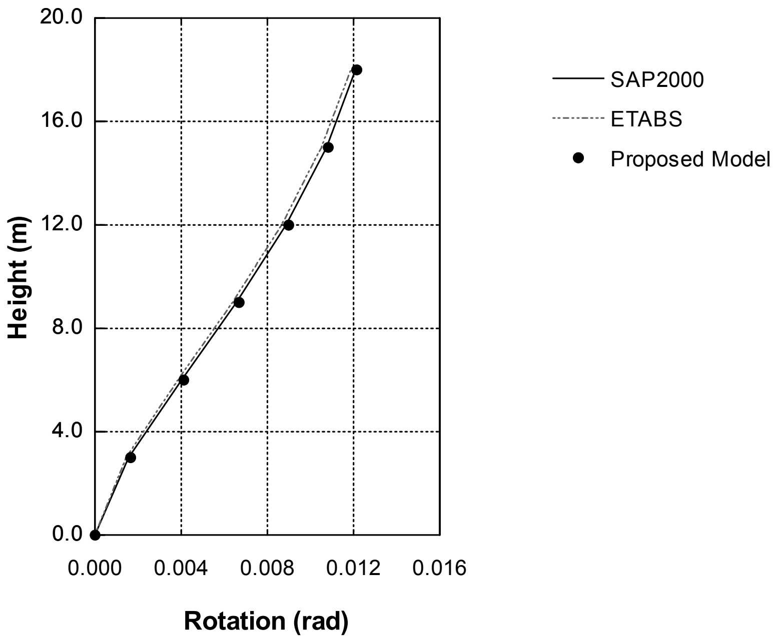


Figure 5.11 Comparison of Floor Rotations of BS2-6 for Loading Condition 2

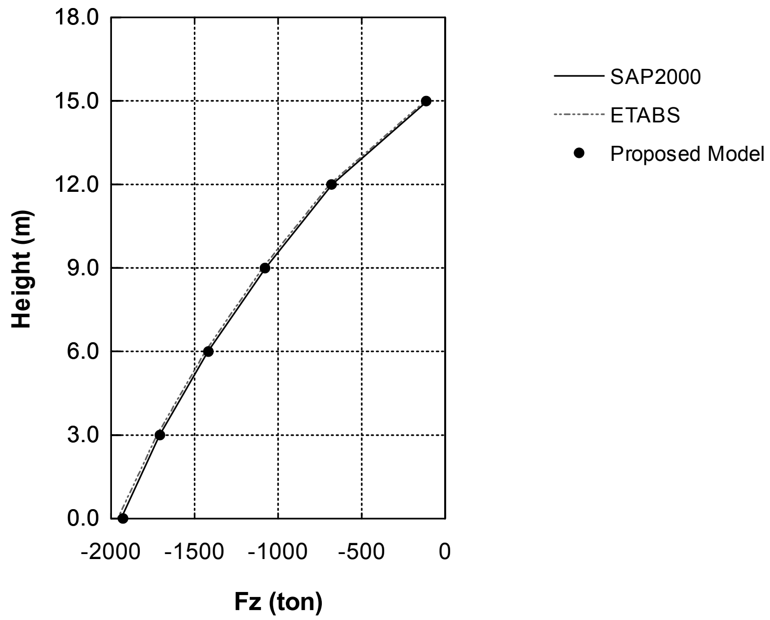


Figure 5.12 Comparison of Resultant Shear Forces at Floor Levels of the Shear Wall Assembly of BS2-6 for Loading Condition 1

of the shear wall assembly for the first loading condition is given in Figure 5.12. In Figure 5.13, a comparison of resultant out-of-plane moments at the floor levels of the shear wall assembly for the second loading condition is given.

In Figure 5.14, a comparison of the floor translations of BS2-15 building structure is given for the first loading condition, and in Figure 5.15 the floor rotations of the considered building structure are given for the second loading condition.

A comparison of the resultant shear forces in the loading direction at the floor levels of the shear wall assembly for the first loading condition is given in Figure 5.16 and in Figure 5.17, a comparison of resultant out-of-plane moments at the floor levels of the shear wall assembly for the second loading condition is given.

5.1.1.3 Building Structure Type BS3

The results of two different BS3 type structures (BS3-9 and BS3-12) are compared in this part. In Figure 5.18, a comparison of the floor translations of BS3-9 building structure is given for the first loading condition and in Figure 5.19, the floor rotations of the considered building structure are given for the second loading condition.

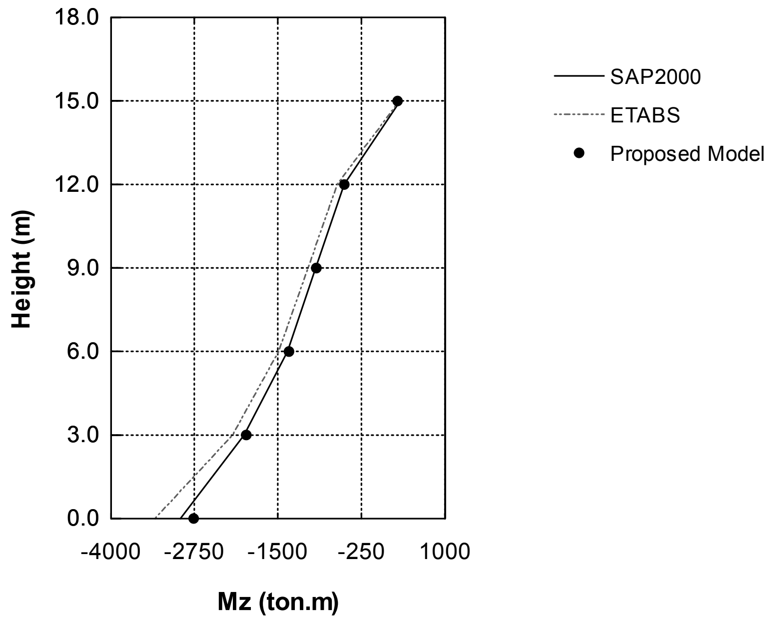


Figure 5.13 Comparison of Resultant Out-of-Plane Moments at Floor Levels of the Shear Wall Assembly of BS2-6 for Loading Condition 2

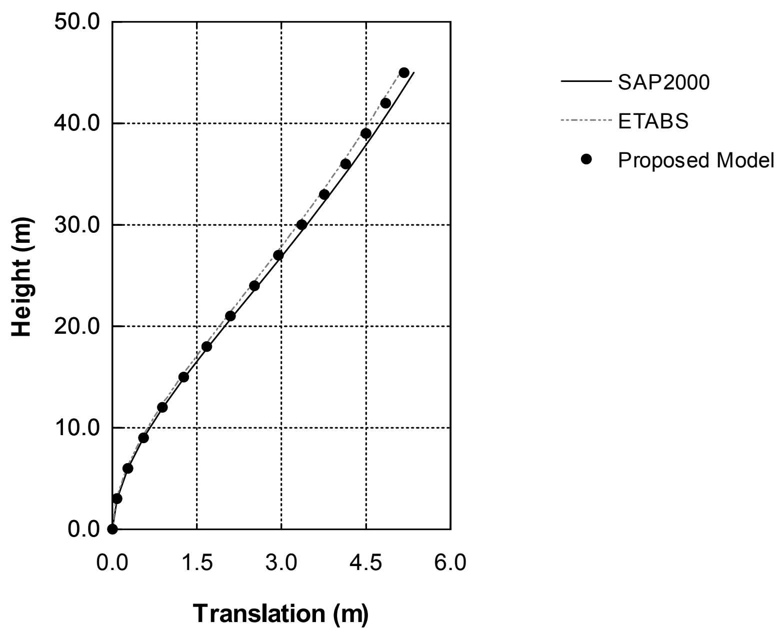


Figure 5.14 Comparison of Floor Displacements of BS2-15 for Loading Condition 1

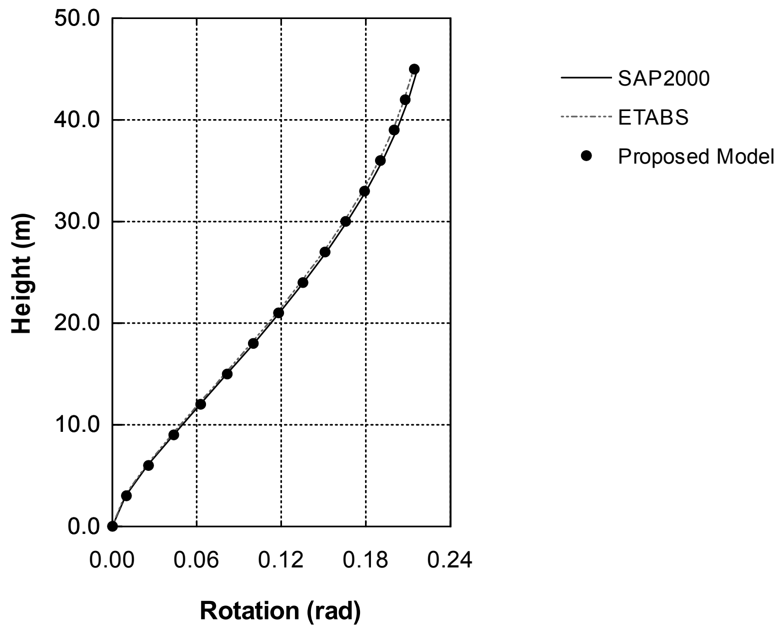


Figure 5.15 Comparison of Floor Rotations of BS2-15 for Loading Condition 2

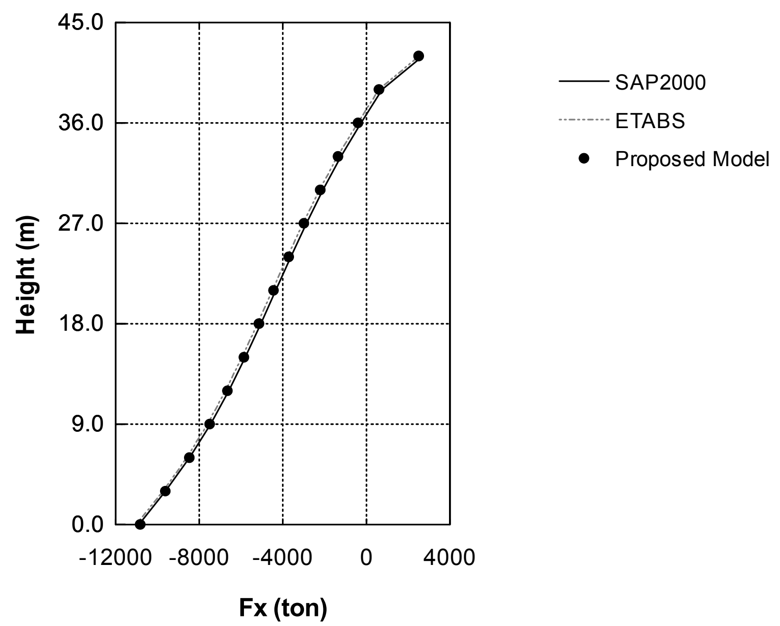


Figure 5.16 Comparison of Resultant Shear Forces at Floor Levels of the Shear Wall Assembly of BS2-15 for Loading Condition 1

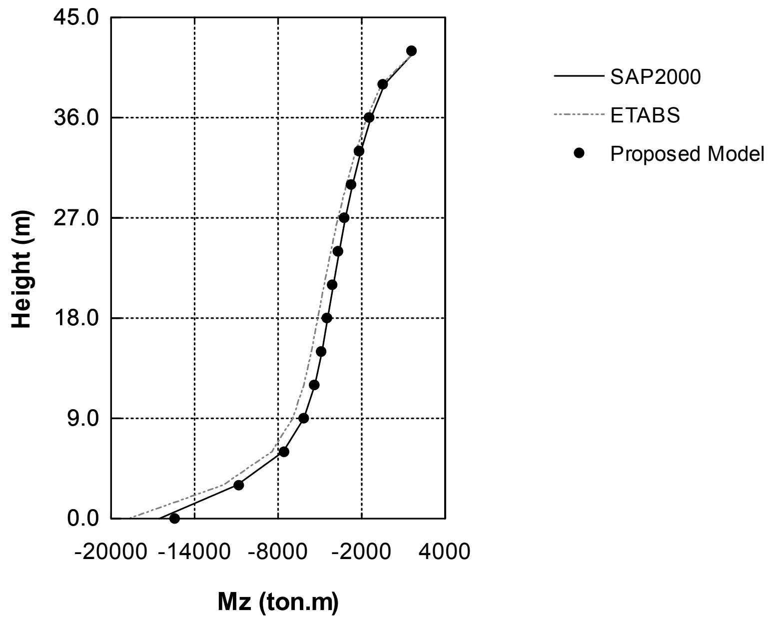


Figure 5.17 Comparison of Resultant Out-of-Plane Moments at Floor Levels of the Shear Wall Assembly of BS2-15 for Loading Condition 2

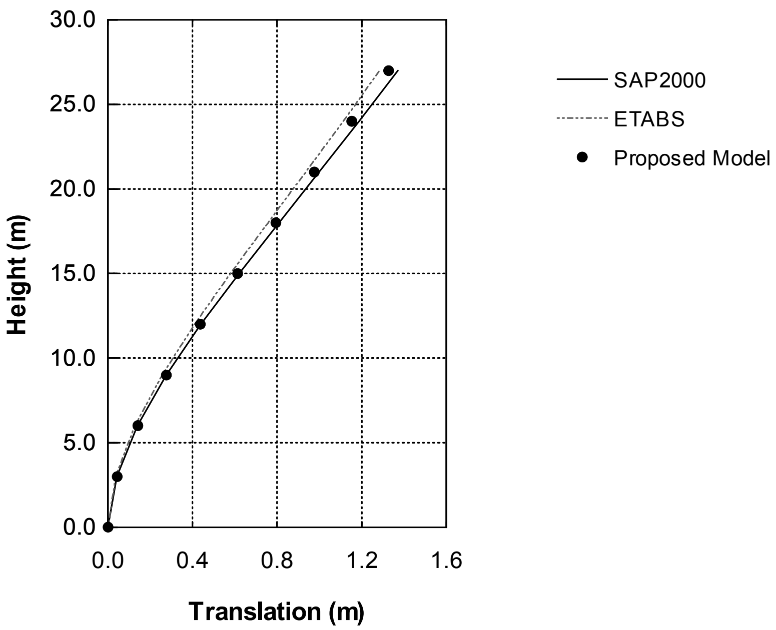


Figure 5.18 Comparison of Floor Displacements of BS3-9 for Loading Condition 1

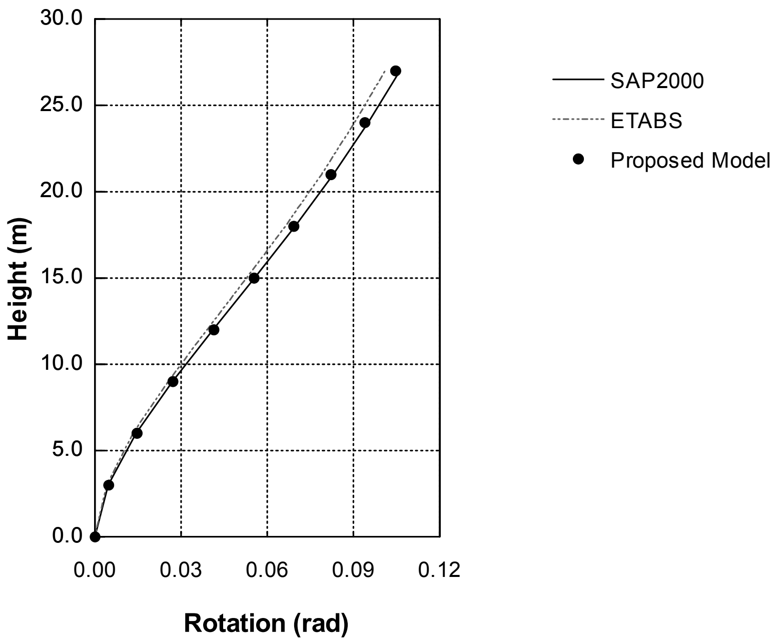


Figure 5.19 Comparison of Floor Rotations of BS3-9 for Loading Condition 2

A comparison of the resultant shear forces in the loading direction at the floor levels of the bottom-left shear wall assembly for the first loading condition is given in Figure 5.20. Figure 5.21 gives a comparison of resultant out-of-plane moments at the floor levels of the bottom-left shear wall assembly for the second loading condition.

In Figure 5.22, a comparison of the floor translations of BS3-12 building structure is given for the first loading condition and in Figure 5.23, the floor rotations of the considered building structure are given for the second loading condition.

A comparison of the resultant shear forces in the loading direction at the floor levels of the shear wall assembly for the first loading condition is given in Figure 5.24 and in Figure 5.25, a comparison of resultant out-of-plane moments at the floor levels of the shear wall assembly for the second loading condition is given.

5.1.1.4 Building Structure Type BS4

The results of the analyses of two different BS4 type structures (BS4-6 and BS4-15) are compared in this part. In Figure 5.26, a comparison of the floor translations

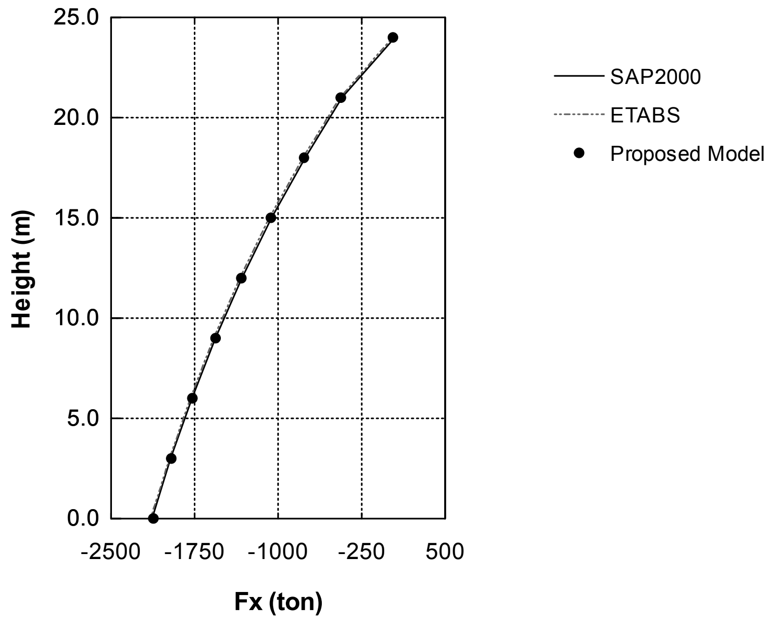


Figure 5.20 Comparison of Resultant Shear Forces at Floor Levels of the Shear Wall Assembly (bottom-left) of BS3-9 for Loading Condition 1

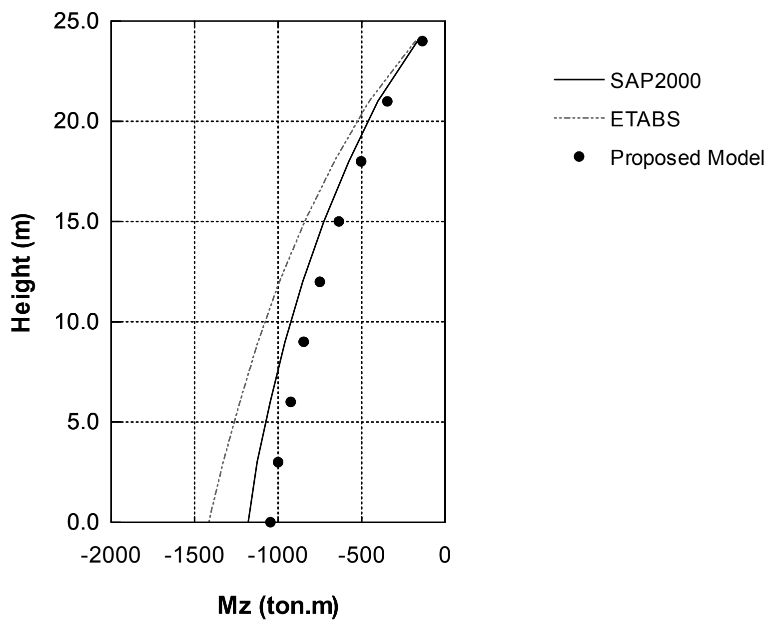


Figure 5.21 Comparison of Resultant Out-of-Plane Moments at Floor Levels of the Shear Wall Assembly (bottom-left) of BS3-9 for Loading Condition 2

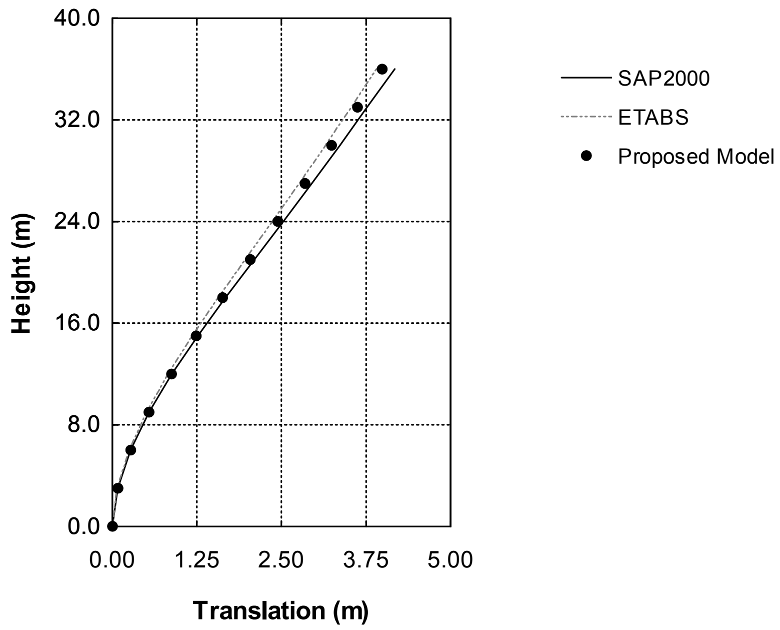


Figure 5.22 Comparison of Floor Displacements of BS3-12 for Loading Condition 1

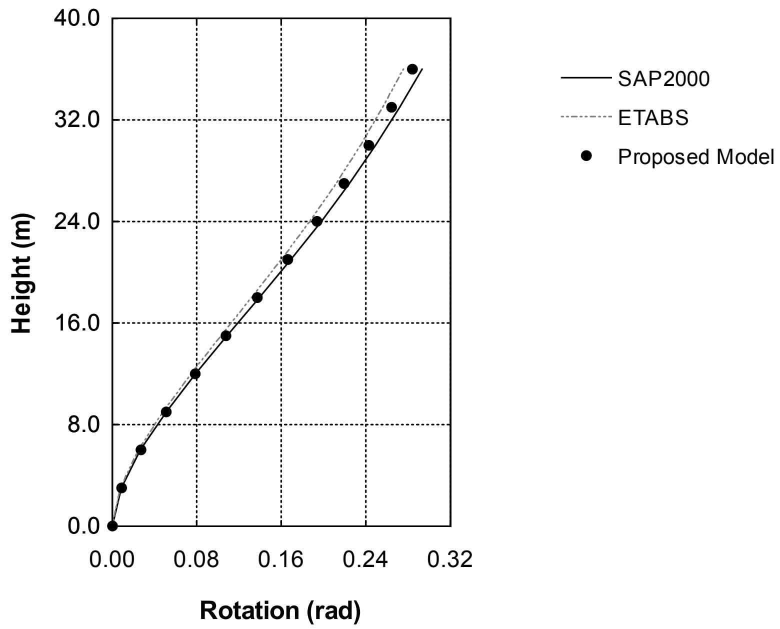


Figure 5.23 Comparison of Floor Rotations of BS3-12 for Loading Condition 2

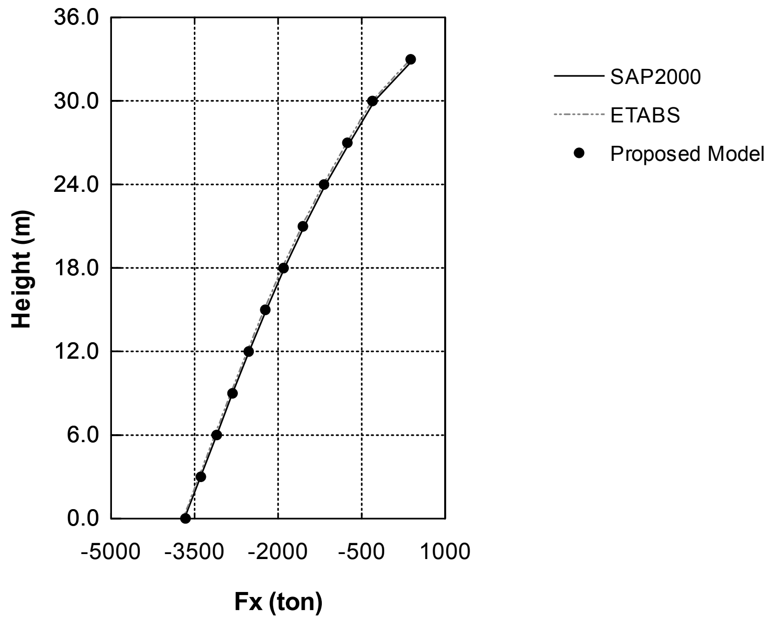


Figure 5.24 Comparison of Resultant Shear Forces at Floor Levels of the Shear Wall Assembly (bottom-left) of BS3-12 for Loading Condition 1

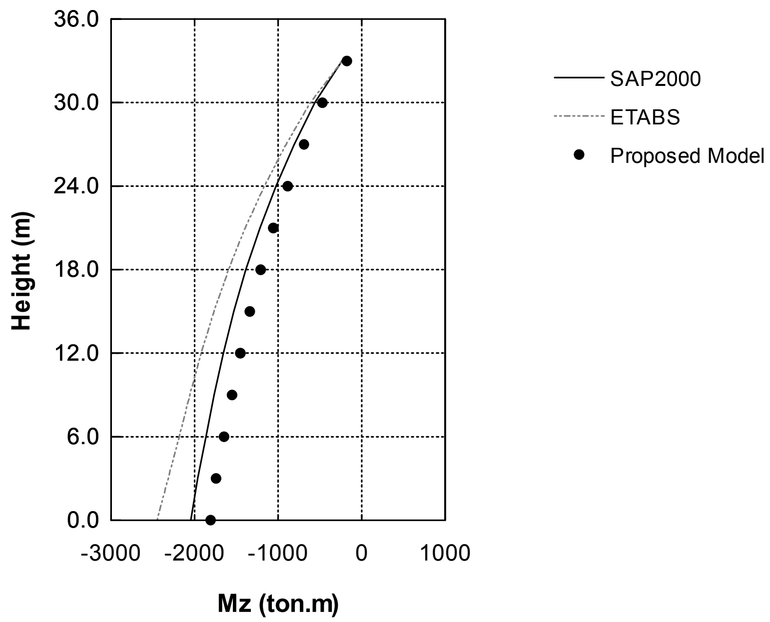


Figure 5.25 Comparison of Resultant Out-of-Plane Moments at Floor Levels of the Shear Wall Assembly (bottom-left) of BS3-12 for Loading Condition 2

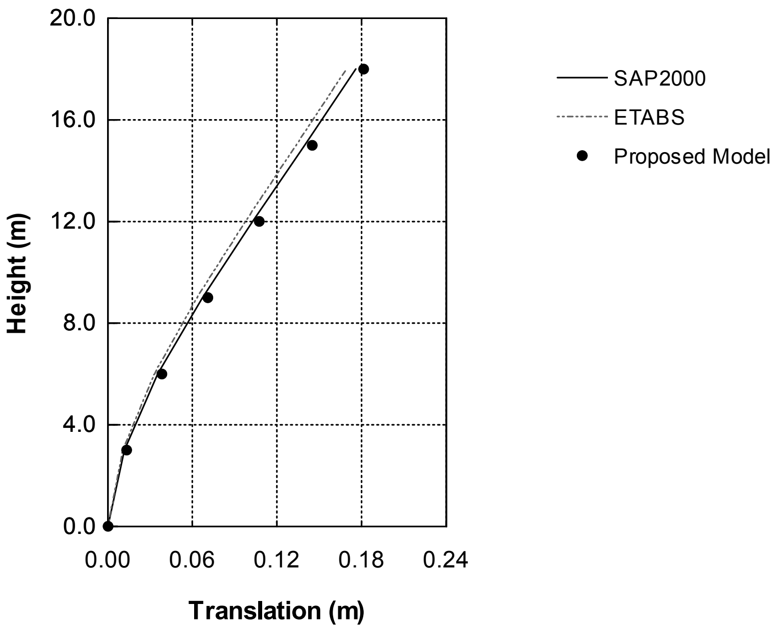


Figure 5.26 Comparison of Floor Displacements of BS4-6 for Loading Condition 1

of BS4-6 building structure is given for the first loading condition, and in Figure 5.27 the floor rotations of the considered building structure are given for the second loading condition.

A comparison of the resultant shear forces in the loading direction at the floor levels of the shear wall assembly for the first loading condition is given in Figure 5.28. In Figure 5.29, the resultant out-of-plane moments at the floor levels of the shear wall assembly for the second loading condition are compared..

In Figure 5.30, a comparison of the floor translations of BS4-15 building structure is given for the first loading condition, while in Figure 5.31, the floor rotations of the considered building structure are given for the second loading condition.

A comparison of the resultant shear forces in the loading direction at the floor levels of the shear wall assembly for the first loading condition is given in Figure 5.32. In Figure 5.33, a comparison of resultant out-of-plane moments at the floor levels of the shear wall assembly for the second loading condition is given.

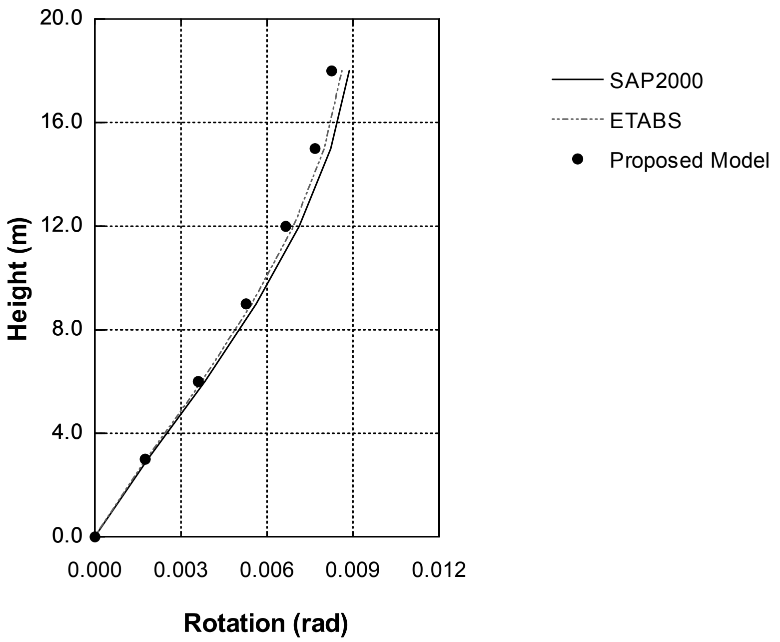


Figure 5.27 Comparison of Floor Rotations of BS4-6 for Loading Condition 2

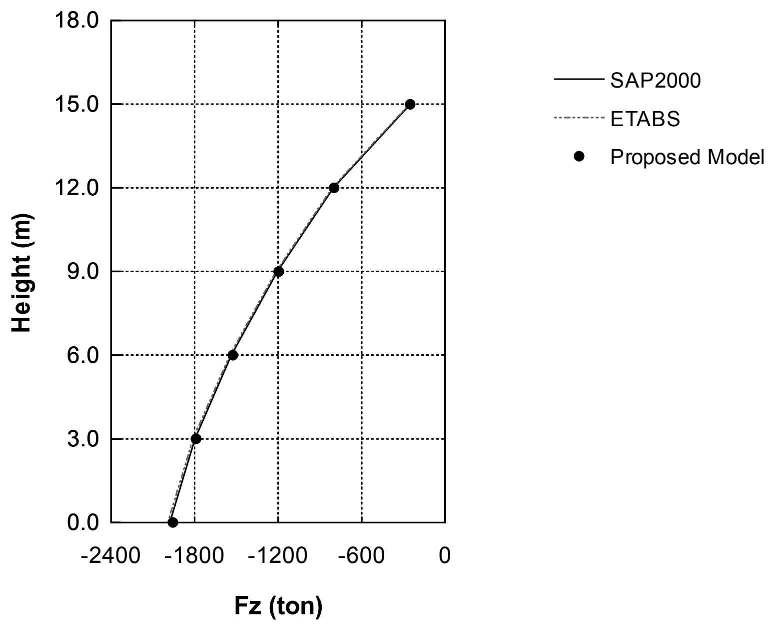


Figure 5.28 Comparison of Resultant Shear Forces at Floor Levels of the Shear Wall Assembly of BS4-6 for Loading Condition 1

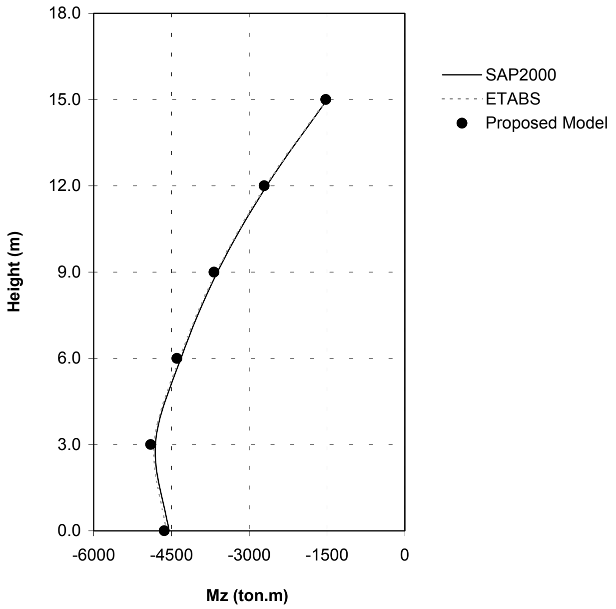


Figure 5.29 Comparison of Resultant Out-of-Plane Moments at Floor Levels of the Shear Wall Assembly of BS4-6 for Loading Condition 2

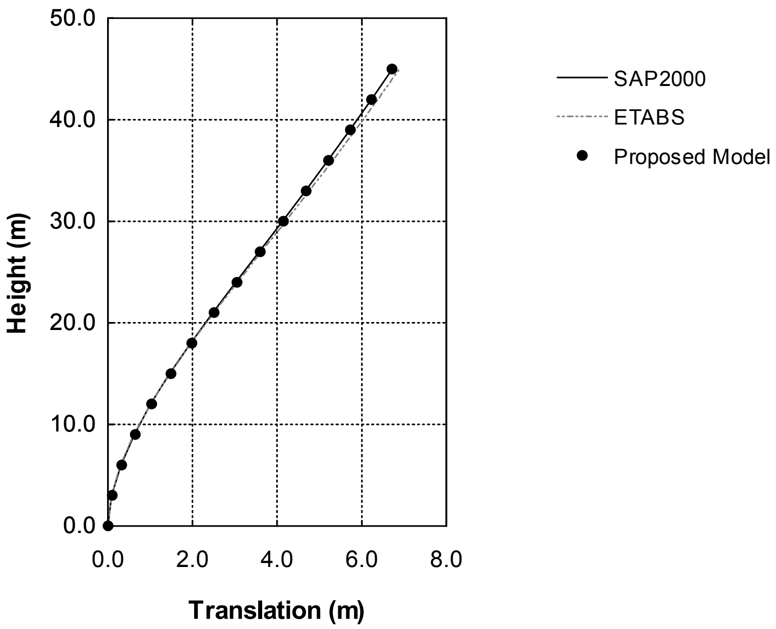


Figure 5.30 Comparison of Floor Displacements of BS4-15 for Loading Condition 1

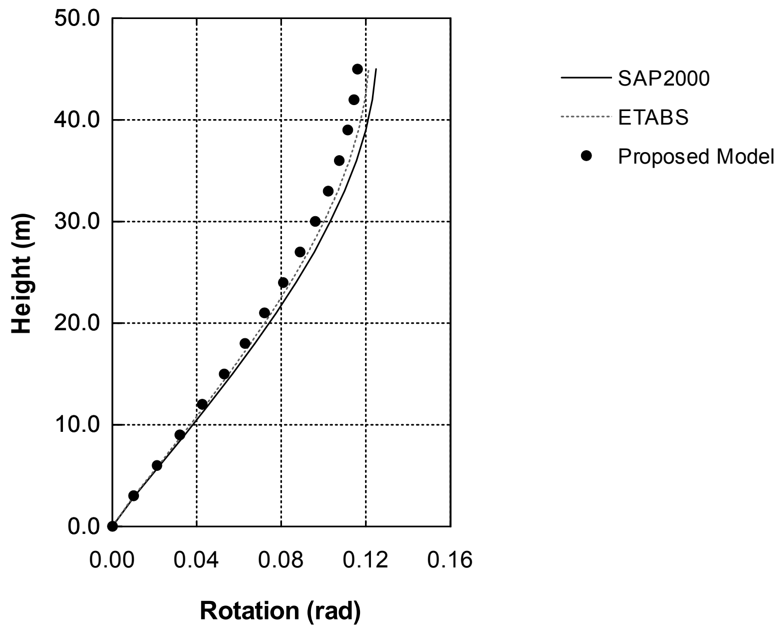


Figure 5.31 Comparison of Floor Rotations of BS4-15 for Loading Condition 2

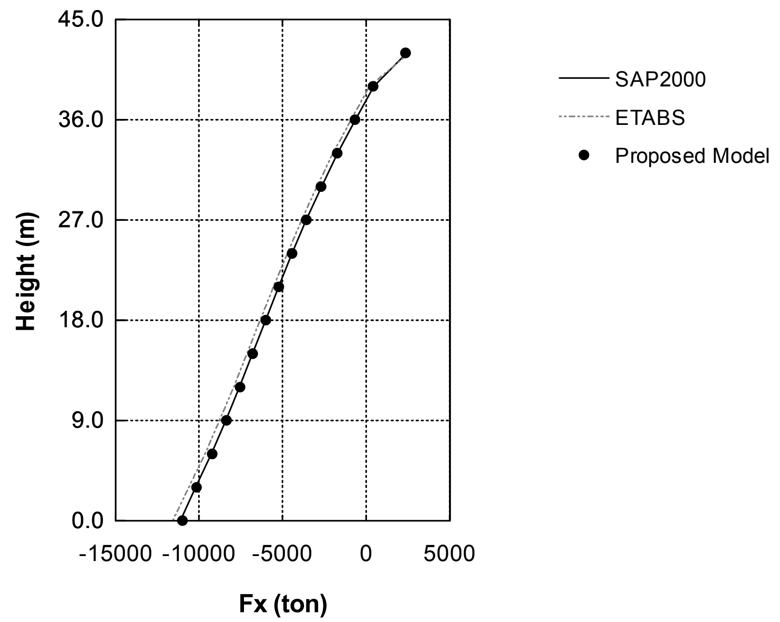


Figure 5.32 Comparison of Resultant Shear Forces at Floor Levels of the Shear Wall Assembly of BS4-15 for Loading Condition 1

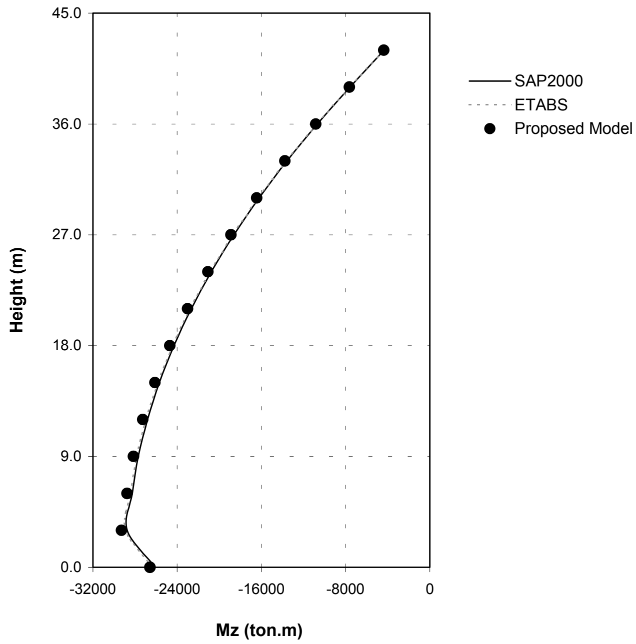


Figure 5.33 Comparison of Resultant Out-of-Plane Moments at Floor Levels of the Shear Wall Assembly of BS4-15 for Loading Condition 2

5.1.1.5 Building Structure Type BS5

Comparisons of the results of the analyses of two BS5 type building structures are presented in this part: BS5-3 and BS5-12. In Table 5.3, a comparison of the floor translations and floor rotations of the BS5-3 type building structure is given for the first and second loading conditions. In Table 5.4, the resultant shear forces and out-of-plane moments at the floor levels of the shear wall assembly obtained by the three methods are tabulated.

In Figure 5.34, the comparison of the floor translations of BS5-12 building structure is given for the first loading condition, and in Figure 5.35 the floor rotations of the considered building structure are given for the second loading condition.

A comparison of the resultant shear forces in the loading direction at the floor levels of the shear wall assembly for the first loading condition is given in Figure 5.36. Figure 5.37 compares the resultant out-of-plane moments at the floor levels of the shear wall assembly for the second loading condition.

In view of the results of the equivalent lateral load analysis of the sample building

Table 5.3 Comparison of Floor Displacements of BS5-3 Obtained by the Two Loading Conditions

Floor Level	SAP2000 S.E.	ETABS W.E.	Proposed Model
	Translation(m)	**Translation(m)**	**Translation(m)**
1	0.00126	0.00110	0.00148
2	0.00331	0.00299	0.00370
3	0.00549	0.00505	000598
	Rotation(rad)	**Rotation(rad)**	**Rotation(rad)**
1	0.00074	0.00068	0.00080
2	0.00174	0.00164	0.00182
3	0.00258	0.00248	0.00264

Table 5.4 Comparison of Resultant Shear Forces and Out-of-Plane Moments at the Shear Wall Assembly in BS5-3 for the Two Loading Conditions

Floor Level	SAP2000 S.E.	ETABS W.E.	Proposed Model
	F_x (t)	**F_x (t)**	**F_x (t)**
0	-582.943	-585.320	-579.35
1	-478.958	-480.530	-477.378
2	-273.877	-275.040	-272.869
	M_z (t.m)	**M_z (t.m)**	**M_z (t.m)**
0	-919.969	-996.755	-832.657
1	-632.252	-666.508	-621.366
2	-113.919	-105.321	-155.068

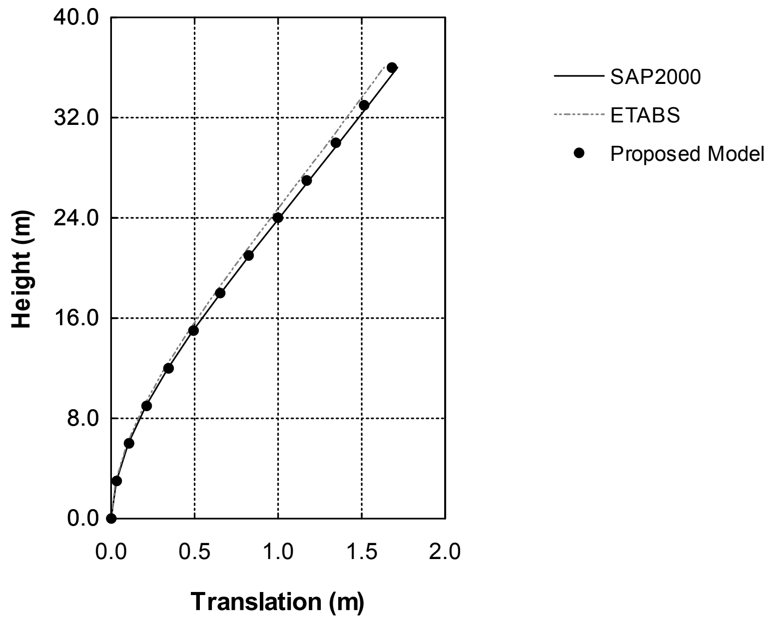


Figure 5.34 Comparison of Floor Displacements of BS5-12 for Loading Condition 1

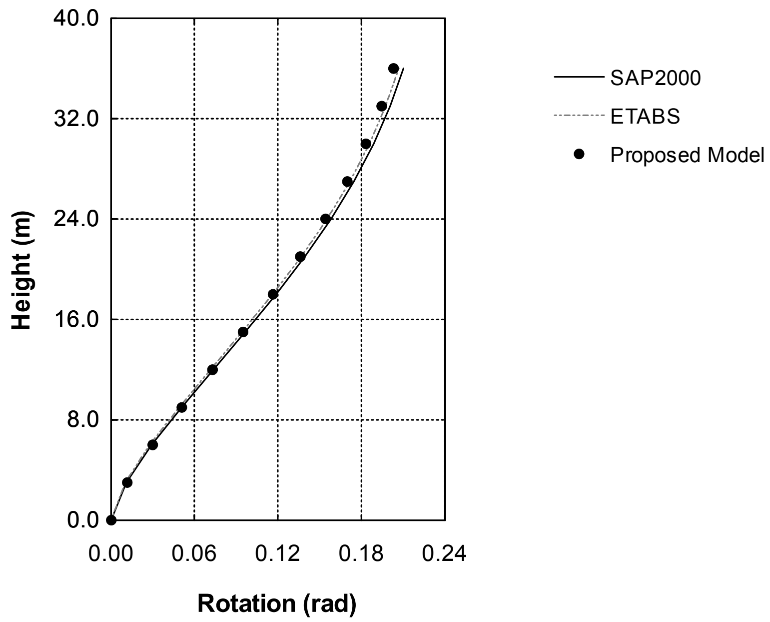


Figure 5.35 Comparison of Floor Rotations of BS5-12 for Loading Condition 2

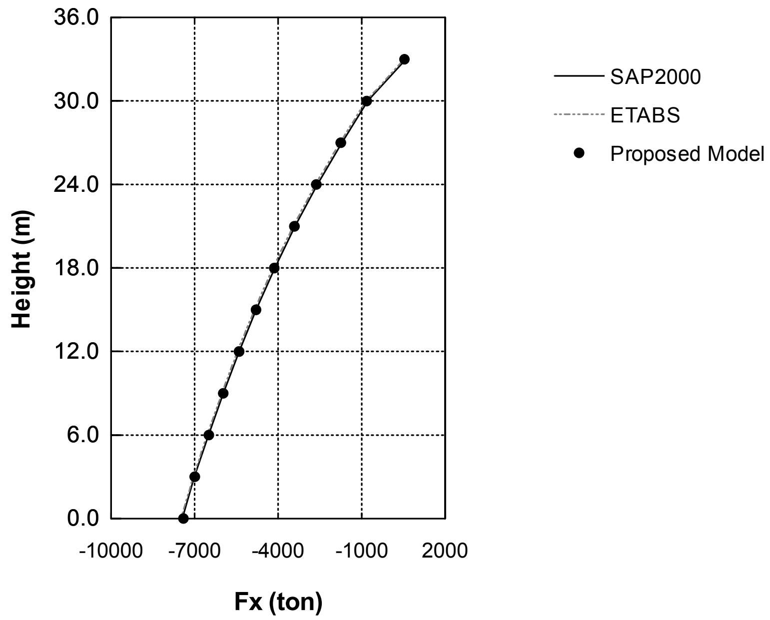


Figure 5.36 Comparison of Resultant Shear Forces at Floor Levels of the Shear Wall Assembly of BS5-12 for Loading Condition 1

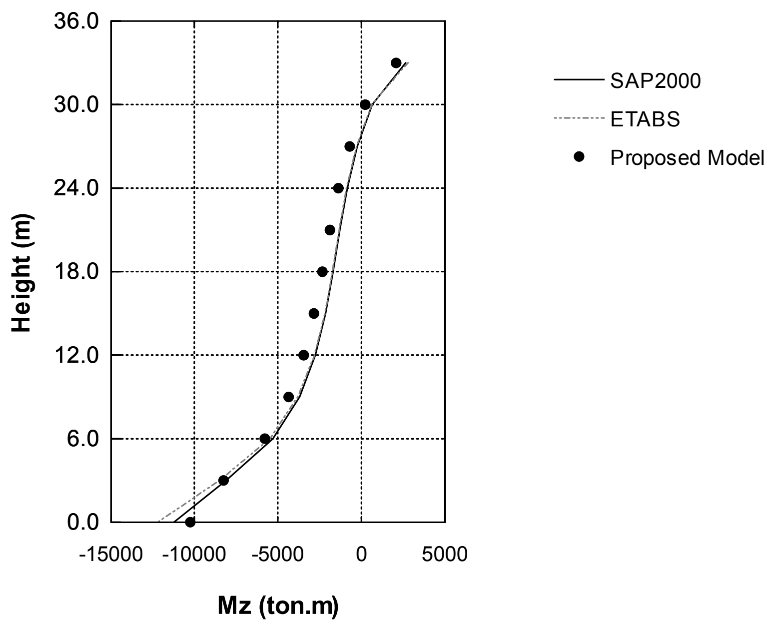


Figure 5.37 Comparison of Resultant Out-of-Plane Moments at Floor Levels of the Shear Wall Assembly of BS5-12 for Loading Condition 2

Table 5.5 Comparison of Relative Difference of Maximum Displacements Obtained by Using Proposed Models and ETABS Wall Elements with the Results Obtained by Using SAP2000 Shell Elements

Building Type	Proposed Models	ETABS
	Rel. Diff. at Max. Translations for Loading Cond.1 (%)	
BS1	5.42	7.84
BS2	3.30	6.98
BS3	5.47	7.61
BS4	9.19	7.50
BS5	8.93	8.05
	Rel. Diff. at Max. Rotations for Loading Cond. 2 (%)	
BS1	3.70	3.40
BS2	2.07	3.45
BS3	4.50	7.06
BS4	7.14	3.03
BS5	3.78	3.80

structures, the relative differences between the three modelling techniques at the locations of maximum displacements and resultant forces are investigated. In Table 5.5, the results of the analyses in which proposed models and ETABS wall elements are used are compared with the results of the analyses in which SAP2000 shell elements are presented. For all types of building structures, the maximum relative differences in translations and rotations obtained from analyses using the proposed models and SAP2000 shell elements are 9.19 % and 7.14 % respectively. These values are 8.05 % and 7.06 % for the results of the analyses in which ETABS wall elements and SAP2000 shell elements are used.

Table 5.6 gives the relative differences at the locations of maximum resultant forces (shear forces and bending moments) for the considered models. For the shear forces obtained by loading condition 1, the maximum relative difference is 1.04%. For the out of plane moment obtained by loading condition 2, the maximum relative difference is 11.57% between the results of the analyses in which shear wall assemblies

Table 5.6 Comparison of Relative Difference of Max. Resultant Forces Obtained by Using Proposed Models and ETABS Wall Elements with the Results Obtained by Using SAP2000 Shell Elements

Building Type	**Proposed Models**	**ETABS**
	Rel. Diff. at Max. Shear Forces (F_x) for Loading Cond.1 (%)	
BS1	0.60	0.57
BS2	0.66	1.00
BS3	0.51	0.59
BS4	1.04	4.02
BS5	0.62	0.47
	Rel. Diff. at Max. Moments (M_z) for Loading Cond. 2 (%)	
BS1	5.28	8.03
BS2	6.83	12.90
BS3	11.57	20.22
BS4	2.07	1.17
BS5	9.49	8.35

are modelled by proposed models and SAP2000 shell elements. These parameters are 4.02% and 20.22% for the results of the analyses in which ETABS wall elements and SAP2000 shell elements are used.

5.1.2 Performance of the Proposed Models in Response Spectrum Analysis

In order to check the validity of the proposed models in dynamic analysis, it is necessary to compare the following parameters, which have been computed in the response spectrum analysis performed in two directions:

1. First three natural vibration periods of the sample buildings
2. Participating mass ratios
3. Displacements of the floors

Table 5.7 Floor Masses and Moments of Inertia of the Sample Type Building Structures

Building Type	**Floor Mass (kg)**	**Moment of Inertia (kg/m^2)**
BS1	$3.67 \cdot 10^3$	$5.00 \cdot 10^3$
BS2	$1.29 \cdot 10^4$	$5.40 \cdot 10^5$
BS3	$5.51 \cdot 10^3$	$1.00 \cdot 10^5$
BS4	$8.72 \cdot 10^3$	$2.50 \cdot 10^5$
BS5	$1.06 \cdot 10^4$	$3.40 \cdot 10^5$

4. Resultant shear forces and bending moments at the floor levels of the shear wall assemblies

5. Base Forces

In the analyses, all structural elements, except the rigid beams, are assumed to have their own masses. The rigid beams at floor levels are assumed to be massless. The mass density of concrete is taken as 255 kg/m^3 in the analyses. The floor masses of the sample buildings are assumed to be concentrated at the centroid of the floors. The values of the floor masses and moments of inertia of the building structures used in dynamic analyses are given in Table 5.7.

The response spectrum function considered in the response spectrum analyses in both directions is given in Figure 5.38. It was used by Özmen [74] previously. A damping ratio of 5 % is taken in the analyses and the CQC (complete quadratic combination) method is used in combining individual modal contributions. In the analyses, the spectrum scale ratio is considered to be 2.943 [74] and structural system behavior ratio, R, is taken as 1.

The results of the response spectrum analysis of only five building structures, chosen from among the whole sample structures, are presented in the following parts (BS1-3, BS2-6, BS3-9, BS4-12 and BS5-15). The analysis results obtained using the proposed models are compared with the results obtained by the SAP2000 shell elements.

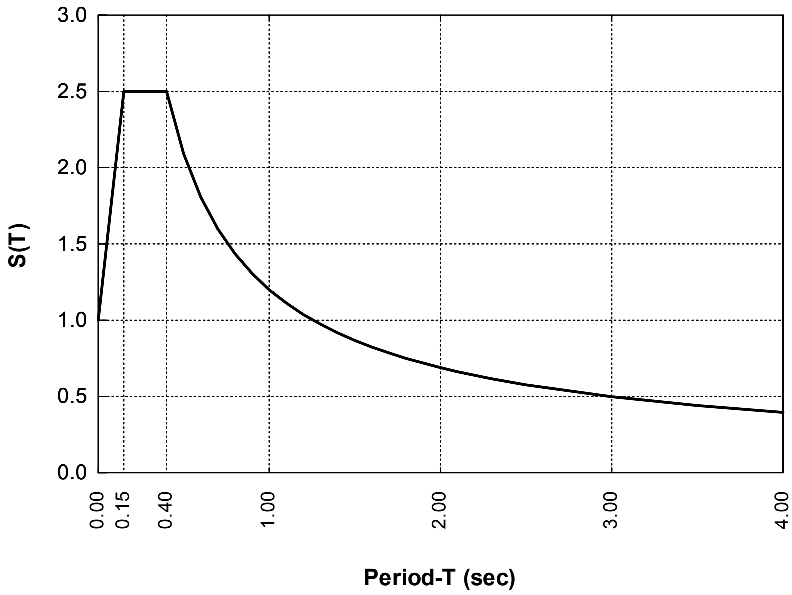


Figure 5.38 Response Spectrum Function Considered in the Analyses

5.1.2.1 Natural Vibration Periods

The evaluation of the natural vibration periods of the building structures is quite important in the dynamic analysis of structures. Moreover, in equivalent lateral load analysis, the first natural period of a building structure should be obtained in order to determine the lateral loads acting at the floor levels.

In the analyses, the first three natural vibration periods of the sample building structures are obtained using three different shear wall modelling methods (SAP2000 shell element, ETABS wall element and the proposed models). The results are given in Tables 5.8 to 5.12. In Table 5.8, the first three natural vibration periods (T_1, T_2 and T_3) of BS1 type building structures (with 3, 6, 9, 12 and 15 storeys) are presented. In Tables 5.9, 5.10, 5.11 and 5.12, the first three natural vibrations periods of BS2, BS3, BS4 and BS5 type building structures are tabulated respectively.

In the analyses using SAP2000 shell elements and the proposed models, the maximum relative differences in the three natural vibration periods of the sample building structures are 2.31% for the first natural vibration periods, 7.06% for the second natural vibration periods and 6.48% for the third natural vibration periods. These parameters

Table 5.8 Comparison of the First Three Natural Periods of BS1 Type Building Structures

Building	Period	SAP2000 S.E. (s)	ETABS W.E. (s)	Proposed Model (s)
BS1-3	T_1	0.307110	0.301200	0.309649
	T_2	0.123912	0.118700	0.126451
	T_3	0.086923	0.082000	0.089053
BS1-6	T_1	0.675322	0.669400	0.678735
	T_2	0.374058	0.362300	0.372019
	T_3	0.202245	0.198400	0.204409
BS1-9	T_1	1.074864	1.068000	1.079450
	T_2	0.705576	0.682400	0.693772
	T_3	0.342493	0.337600	0.345034
BS1-12	T_1	1.499999	1.491100	1.506029
	T_2	1.089213	1.052300	1.065151
	T_3	0.527174	0.519200	0.525052
BS1-15	T_1	1.953667	1.942200	1.961320
	T_2	1.515294	1.464000	1.478281
	T_3	0.743088	0.732600	0.734011

are 7.76%, 5.02% and 24.58% the analyses using SAP2000 shell elements and ETABS wall elements are compared. Especially the first natural periods, which are important in determining the equivalent floor loads, obtained by using the proposed models are closer to the values obtained by using SAP2000 shell elements than those obtained by using ETABS wall elements. The deformed shapes of the building structures are also examined and the corresponding mode shapes obtained by using the three modelling methods are observed to be the same according to the analysis results.

5.1.2.2 Participating Mass Ratios

In the response spectrum analysis, an adequate number of modes should be included in the calculations as it is a measure of accuracy of the analysis. Most building codes, including the Turkish Earthquake Code [33], require that the computations of the responses should include enough modes to capture at least 90 percent of the total building mass. In the analyses, the total number of modes needed to capture 90 percent are determined for each direction. Table 5.13 gives a comparison of the mode participations for the sample buildings in which shear wall assemblies are modelled using SAP2000

Table 5.9 Comparison of the First Three Natural Periods of BS2 Type Building Structures

Building	Period	SAP2000 S.E. (s)	ETABS W.E. (s)	Proposed Model (s)
BS2-3	T_1	0.381536	0.369400	0.383313
	T_2	0.166501	0.158300	0.169762
	T_3	0.114693	0.107500	0.117037
BS2-6	T_1	0.803812	0.786300	0.803481
	T_2	0.474882	0.457900	0.474182
	T_3	0.248584	0.240500	0.249998
BS2-9	T_1	1.246663	1.224200	1.244240
	T_2	0.857680	0.828800	0.849803
	T_3	0.395295	0.385800	0.398995
BS2-12	T_1	1.700775	1.672900	1.696627
	T_2	1.281003	1.239000	1.264968
	T_3	0.591545	0.573500	0.604679
BS2-15	T_1	2.164530	2.131000	2.159201
	T_2	1.734543	1.678900	1.709967
	T_3	0.841665	0.815700	0.850310

Table 5.10 Comparison of the First Three Natural Periods of BS3 Type Building Structures

Building	Period	SAP2000 S.E. (s)	ETABS W.E. (s)	Proposed Model (s)
BS3-3	T_1	0.270471	0.259400	0.269953
	T_2	0.136968	0.130300	0.141726
	T_3	0.070459	0.066600	0.074181
BS3-6	T_1	0.656740	0.605800	0.648689
	T_2	0.402545	0.386900	0.403064
	T_3	0.215717	0.162700	0.214989
BS3-9	T_1	1.083998	1.047300	1.066807
	T_2	0.750576	0.721100	0.744142
	T_3	0.422490	0.396000	0.408510
BS3-12	T_1	1.541342	1.488000	1.505687
	T_2	1.146325	1.100800	1.121010
	T_3	0.669158	0.619600	0.631513
BS3-15	T_1	2.023151	1.950300	1.981266
	T_2	1.576556	1.514100	1.542538
	T_3	0.941287	0.864300	0.880288

Table 5.11 Comparison of the First Three Natural Periods of BS4 Type Building Structures

Building	Period	SAP2000 S.E. (s)	ETABS W.E. (s)	Proposed Model (s)
BS4-3	T_1	0.223960	0.217400	0.220323
	T_2	0.138662	0.131700	0.146073
	T_3	0.105986	0.101300	0.109792
BS4-6	T_1	0.476262	0.462900	0.472876
	T_2	0.401726	0.389300	0.409542
	T_3	0.273845	0.266100	0.270817
BS4-9	T_1	0.799621	0.778000	0.796315
	T_2	0.753173	0.733900	0.759847
	T_3	0.448341	0.43700	0.436242
BS4-12	T_1	1.176619	1.145800	1.171179
	T_2	1.159679	1.132600	1.164334
	T_3	0.616929	0.601800	0.596807
BS4-15	T_1	1.607246	1.571800	1.609430
	T_2	1.594023	1.553700	1.585408
	T_3	0.782197	0.763300	0.754759

Table 5.12 Comparison of the First Three Natural Periods of BS5 Type Building Structures

Building	Period	SAP2000 S.E. (s)	ETABS W.E. (s)	Proposed Model (s)
BS5-3	T_1	0.328583	0.317100	0.333979
	T_2	0.126056	0.120300	0.134953
	T_3	0.109855	0.103800	0.115465
BS5-6	T_1	0.708359	0.691100	0.708212
	T_2	0.326218	0.314800	0.335214
	T_3	0.325690	0.313600	0.330136
BS5-9	T_1	1.114049	1.091700	1.107224
	T_2	0.625761	0.604600	0.625793
	T_3	0.597117	0.578200	0.601417
BS5-12	T_1	1.537066	1.509600	1.523455
	T_2	0.982307	0.949800	0.975672
	T_3	0.914672	0.886500	0.910298
BS5-15	T_1	1.974223	1.941200	1.954243
	T_2	1.380280	1.335200	1.365771
	T_3	1.263933	1.225800	1.248402

Table 5.13 Comparison of Mode Participations in the Response Spectrum Analysis of the Sample Structures

Building Structure	# of Modes (SAP2000 S.E.)	# of Modes (Prop. Models)
BS1-3		
R.S.A. (x-dir)	5	6
R.S.A. (y-dir)	4	4
BS2-6		
R.S.A. (x-dir)	11	11
R.S.A. (y-dir)	10	9
BS3-9		
R.S.A. (x-dir)	9	9
R.S.A. (y-dir)	7	7
BS4-12		
R.S.A. (x-dir)	9	9
R.S.A. (y-dir)	7	7
BS5-15		
R.S.A. (x-dir)	12	12
R.S.A. (y-dir)	9	8

shell elements and the proposed models.

5.1.2.3 Floor Displacements

This section presents the results of the comparisons of floor displacements obtained by using two shear wall modelling techniques (SAP2000 shell elements and proposed models) in response spectrum analyses performed in the x and y directions. Due to the symmetry, the dynamic loading in the x direction causes pure translations on the sample building structures. Dynamic loading in the y direction leads to translations and rotations as the sample building structures are not symmetric in y direction. For this reason, the comparisons of the floor displacements are made for

(a) floor translations in for response spectrum analysis in the x direction and

(b) floor rotations for response spectrum analysis in the y direction.

Table 5.14 gives the comparison of floor displacements of BS1-3 building structure obtained by the response spectrum analyses in two directions. The translations are obtained by performing response spectrum analysis in the x-direction and the rotations

Table 5.14 Comparison of Floor Displacements of BS1-3 Obtained by Response Spectrum Analysis in x and y Directions

Floor	SAP2000 S.E.	Proposed Model
	Translation(m)	**Translation(m)**
1	0.000689	0.000757
2	0.001980	0.002100
3	0.003420	0.003580
	Rotation(rad)	**Rotation(rad)**
1	0.000809	0.000846
2	0.002050	0.002100
3	0.003130	0.003190

are obtained by performing response spectrum analyses in the y-direction.

In Figures 5.39 to 5.46, the comparison of floor displacements obtained by the response spectrum analyses are presented for the sample building structures BS2-6, BS3-9, BS4-12 and BS5-15.

For the five sample building structures considered, the maximum relative difference between the two models was 4.68 % for translations (response spectrum analysis in the x-direction) and 8.73 % for rotations (response spectrum analysis in the y-direction).

5.1.2.4 Resultant Shear Forces and Bending Moments in the Shear Wall Assemblies

The resultant shear forces and bending moments on the shear wall assemblies obtained by the response spectrum analyses in x and y directions are considered in this part. The five sample building structures (BS1-3, BS2-6, BS3-9, BS4-12 and BS5-15) are analyzed using two shear wall modelling techniques (SAP2000 shell elements and the proposed models). The results of these analyses are compared in Tables 5.15 and 5.16 and Figures 5.47 to 5.54.

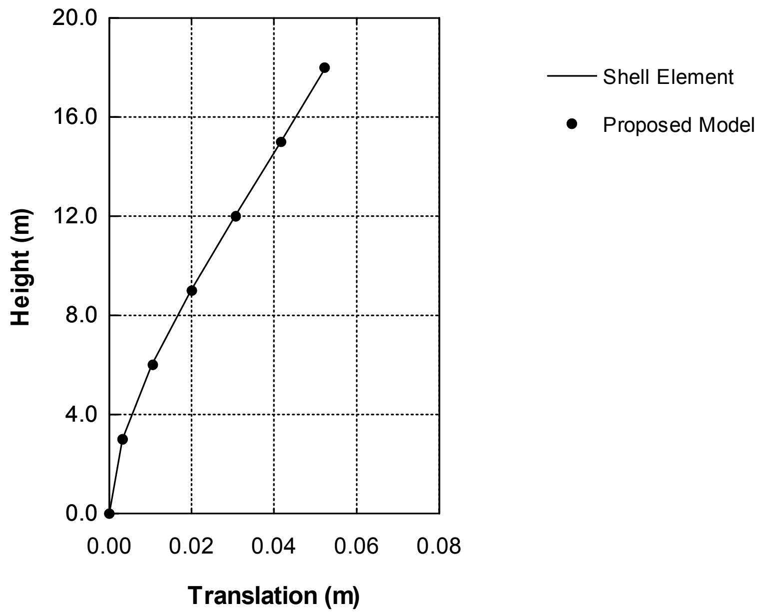


Figure 5.39 Comparison of Floor Translations of BS2-6 Obtained by Response Spectrum Analysis in x-direction

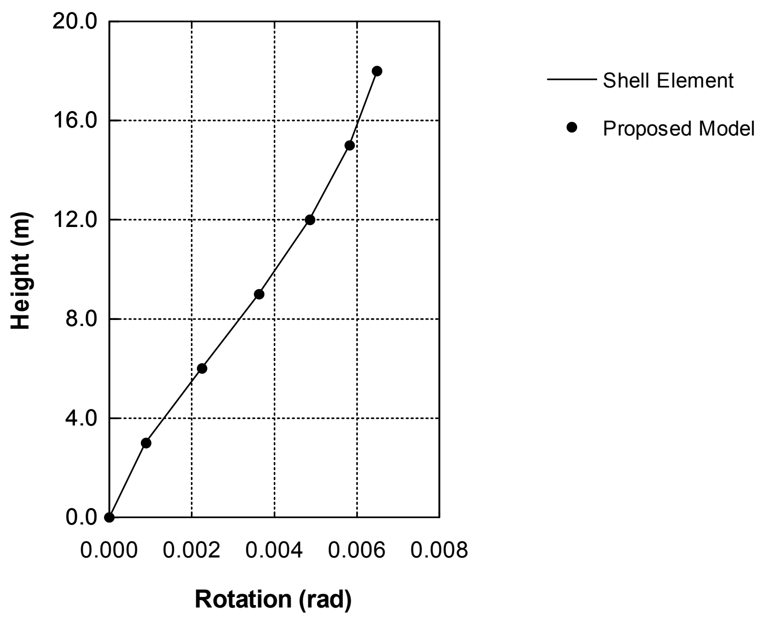


Figure 5.40 Comparison of Floor Rotations of BS2-6 Obtained by Response Spectrum Analysis in y-direction

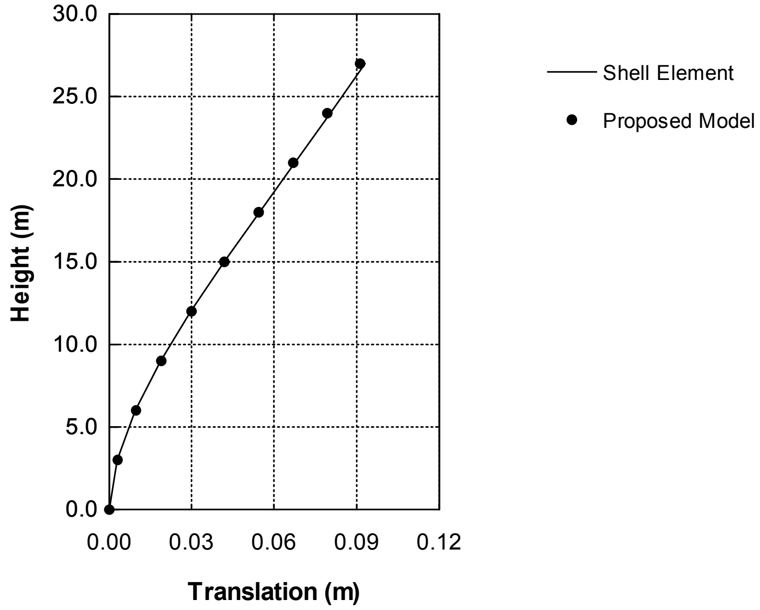


Figure 5.41 Comparison of Floor Translations of BS3-9 Obtained by Response Spectrum Analysis in x-direction

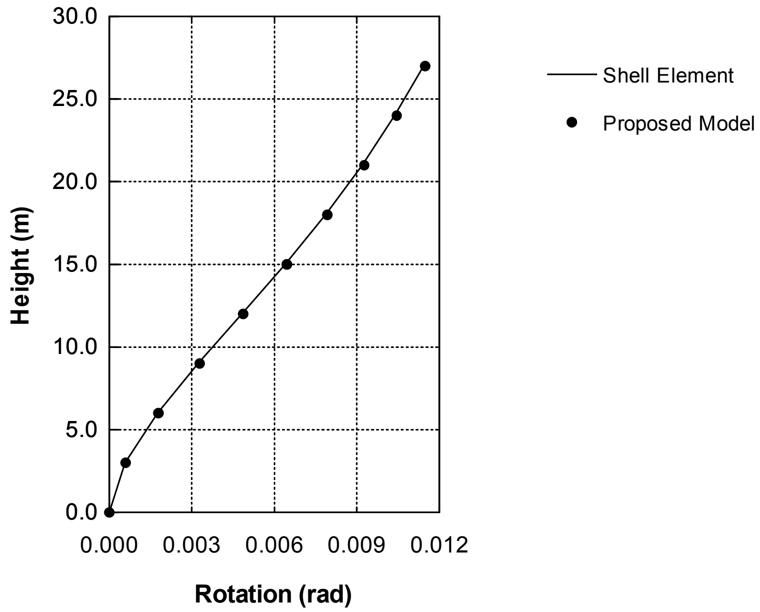


Figure 5.42 Comparison of Floor Rotations of BS3-9 Obtained by Response Spectrum Analysis in y-direction

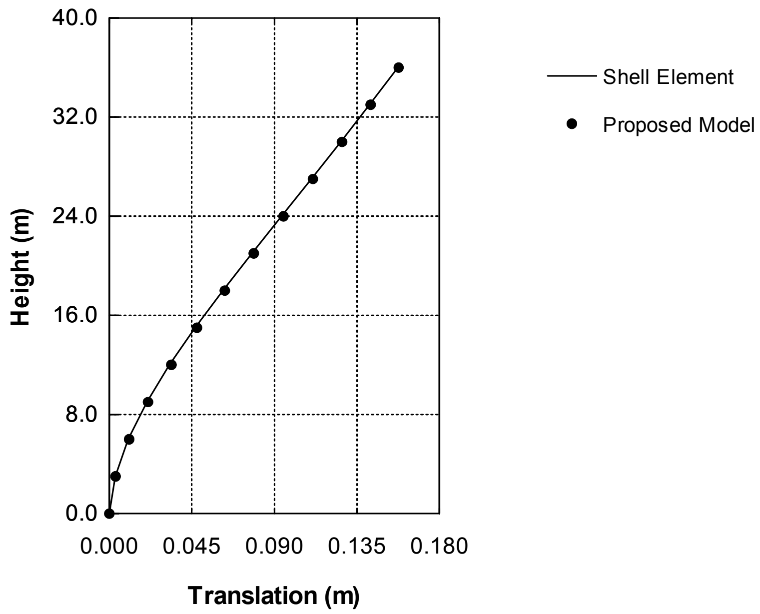


Figure 5.43 Comparison of Floor Translations of BS4-12 Obtained by Response Spectrum Analysis in x-direction

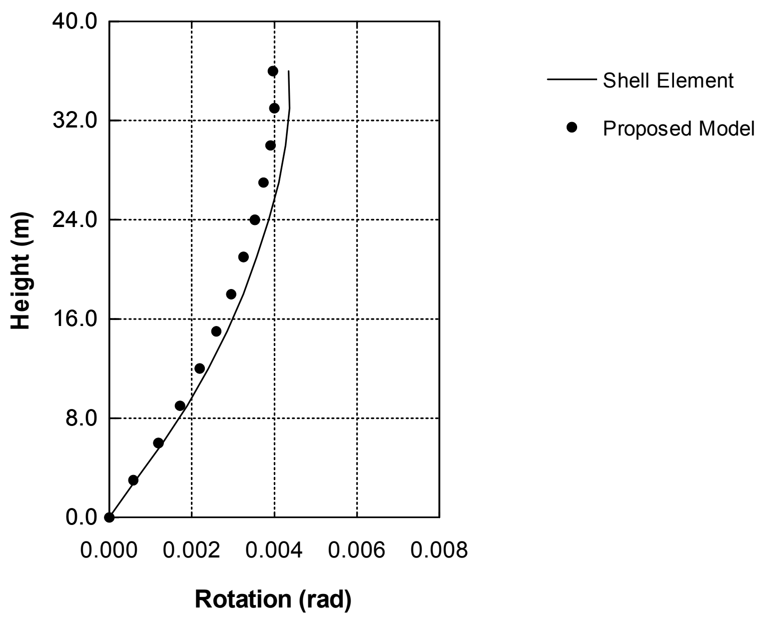


Figure 5.44 Comparison of Floor Rotations of BS4-12 Obtained by Response Spectrum Analysis in y-direction

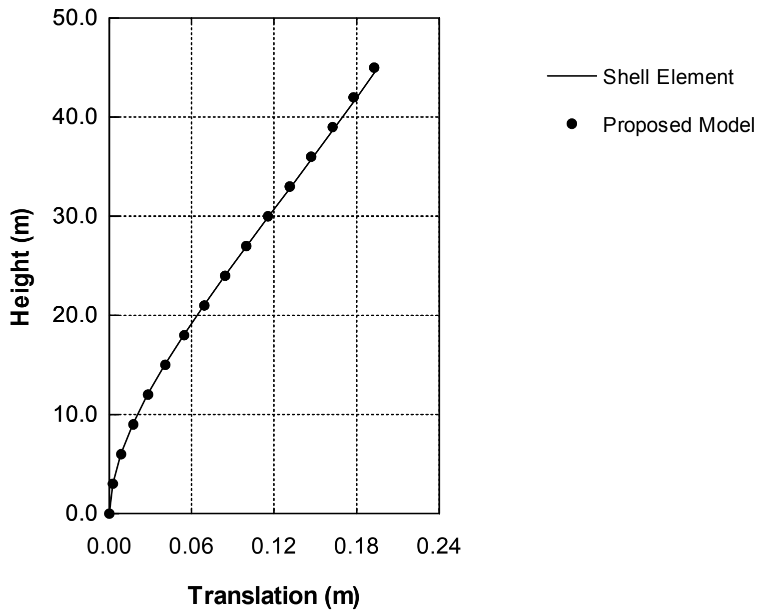


Figure 5.45 Comparison of Floor Translations of BS5-15 Obtained by Response Spectrum Analysis in x-direction

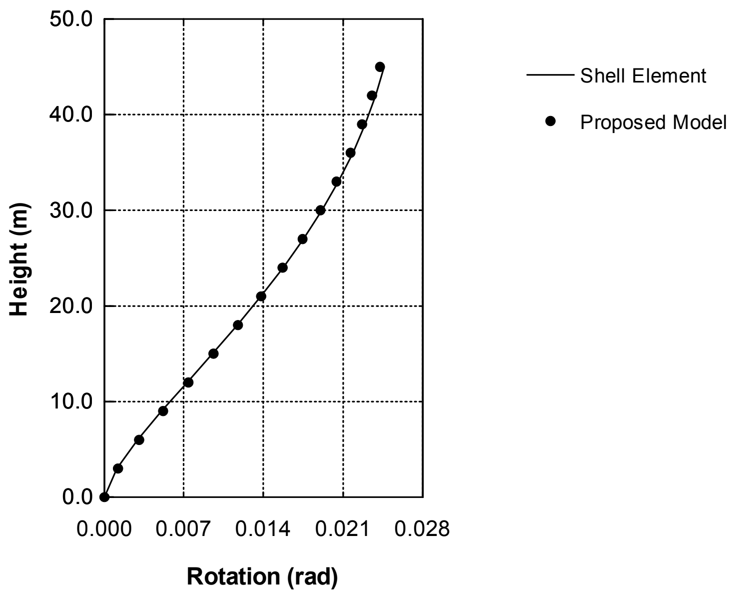


Figure 5.46 Comparison of Floor Rotations of BS5-15 Obtained by Response Spectrum Analysis in y-direction

Table 5.15 Comparison of Resultant Shear Forces and Bending Moments in the Shear Wall Assembly in BS1-3 Obtained by Response Spectrum Analysis in x-direction Loading Conditions

Floor Level	SAP2000 S.E.	Proposed Model
	F_x (t)	**F_x (t)**
0	105.359	107.127
1	90.249	90.053
2	52.875	49.877
	M_y (t.m)	**M_y (t.m)**
0	703.920	708.348
1	398.216	398.028
2	142.259	140.277

In the figures, the resultant shear forces (F_x and F_y) obtained by response spectrum analyses in x and y directions are plotted on the same graph. Similarly, the bending moments (M_y and M_x) obtained by response spectrum analyses of sample structures in both directions are plotted together.

In Table 5.15, resultant shear forces and bending moments obtained by response spectrum analysis in x direction are tabulated for building structure BS1-3. In Table 5.16, the resultant shear forces and bending moments obtained by response spectrum analysis in y direction are compared.

In Figures 5.47, 5.49, 5.51 and 5.52, the resultant shear forces in the shear wall assemblies obtained by the response spectrum analyses of BS2-6, BS3-9, BS4-12 and BS5-15 sample building structures in x and y directions are compared. The corresponding resultant bending moments of the assemblies are given in Figures 5.48, 5.50, 5.52 and 5.54.

According to the results of the response spectrum analyses of the sample building structures in both directions, the maximum relative differences in resultant shear forces

Table 5.16 Comparison of Resultant Shear Forces and Bending Moments in the Shear Wall Assembly in BS1-3 Obtained by Response Spectrum Analysis in y-direction Loading Conditions

Floor Level	SAP2000 S.E.	Proposed Model
	F_y (t)	**F_y (t)**
0	72.504	72.543
1	53.160	52.942
2	21.641	18.672
	M_x (t.m)	**M_x (t.m)**
0	381.292	382.536
1	183.908	183.295
2	48.856	44.755

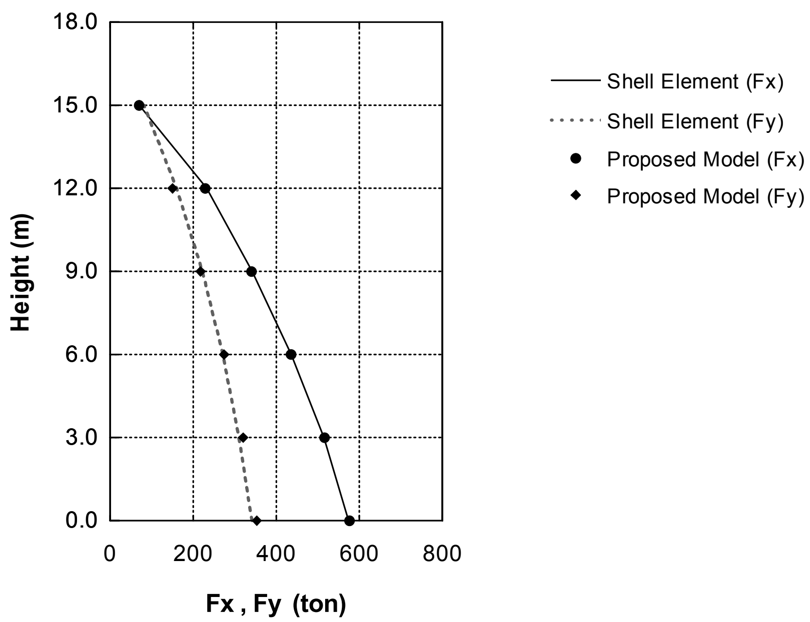


Figure 5.47 Comparison of Resultant Shear Forces on the Shear Wall Assembly in BS2-6 Obtained by Response Spectrum Analyses in x and y-directions

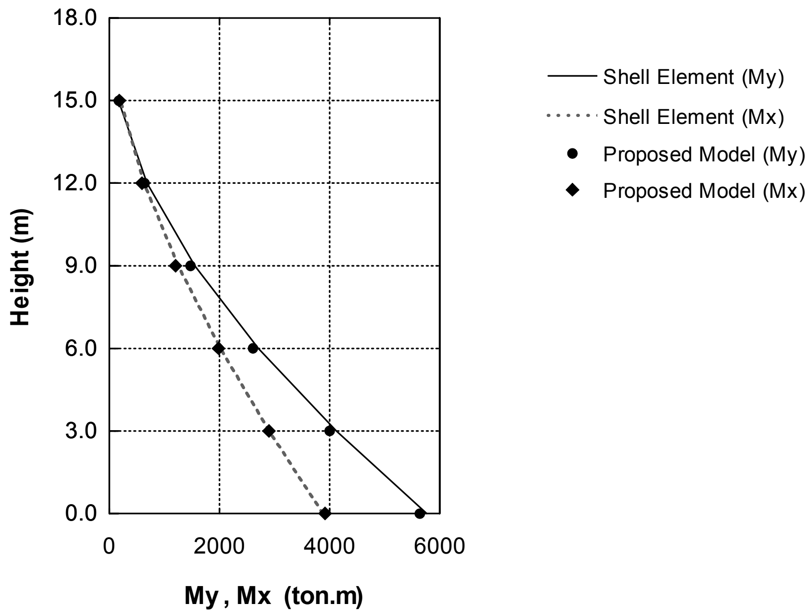


Figure 5.48 Comparison of Resultant Bending Moments in the Shear Wall Assembly in BS2-6 Obtained by Response Spectrum Analyses in x and y-directions

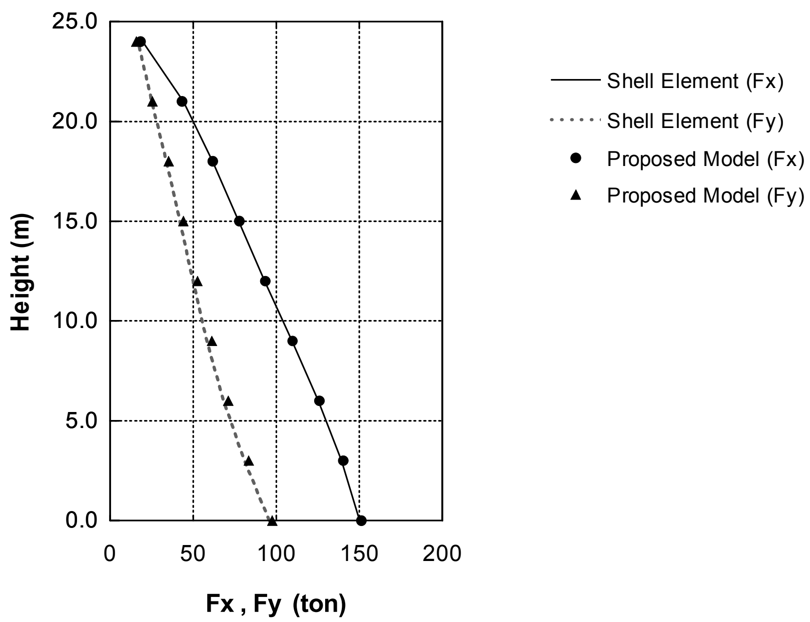


Figure 5.49 Comparison of Resultant Shear Forces on the Shear Wall Assembly (bottom-left) in BS3-9 Obtained by Response Spectrum Analyses in x and y-directions

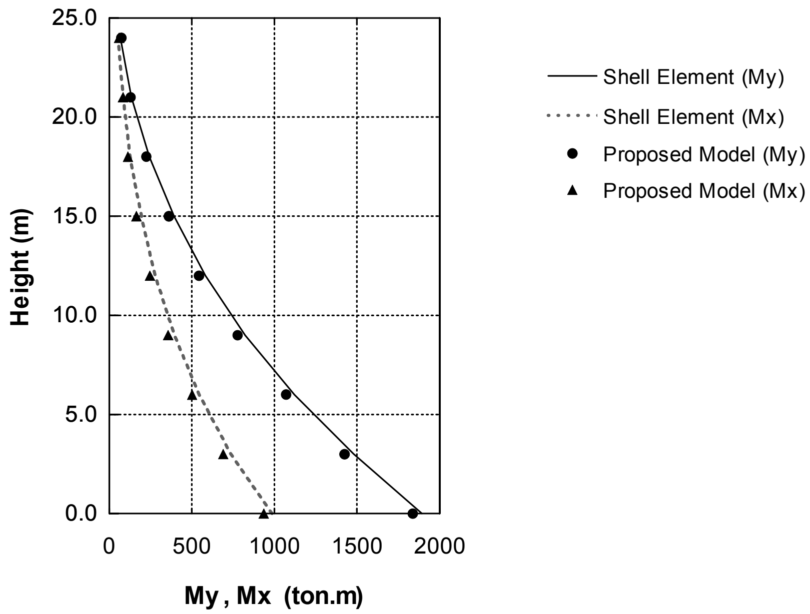


Figure 5.50 Comparison of Resultant Bending Moments in the Shear Wall Assembly (bottom-left) in BS3-9 Obtained by Resp. Spec. Analyses in x and y-directions

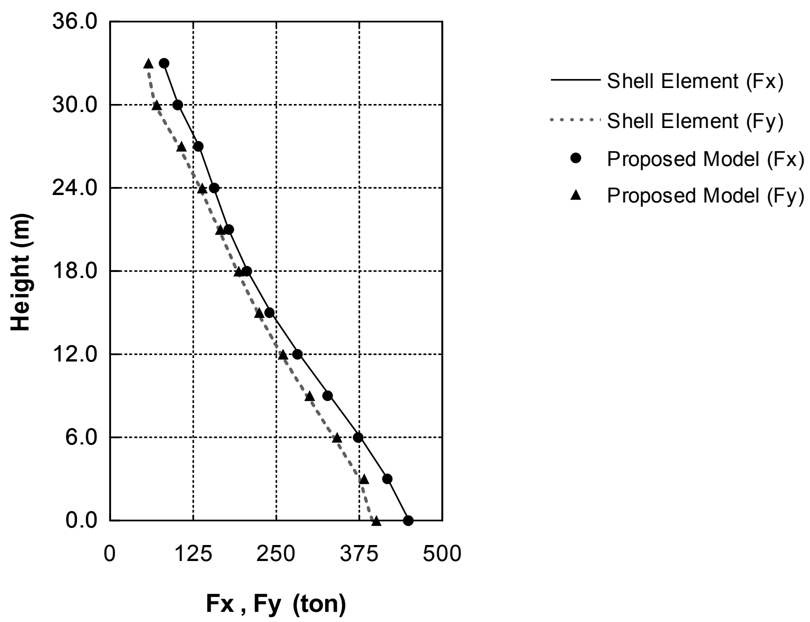


Figure 5.51 Comparison of Resultant Shear Forces on the Shear Wall Assembly in BS4-12 Obtained by Response Spectrum Analyses in x and y-directions

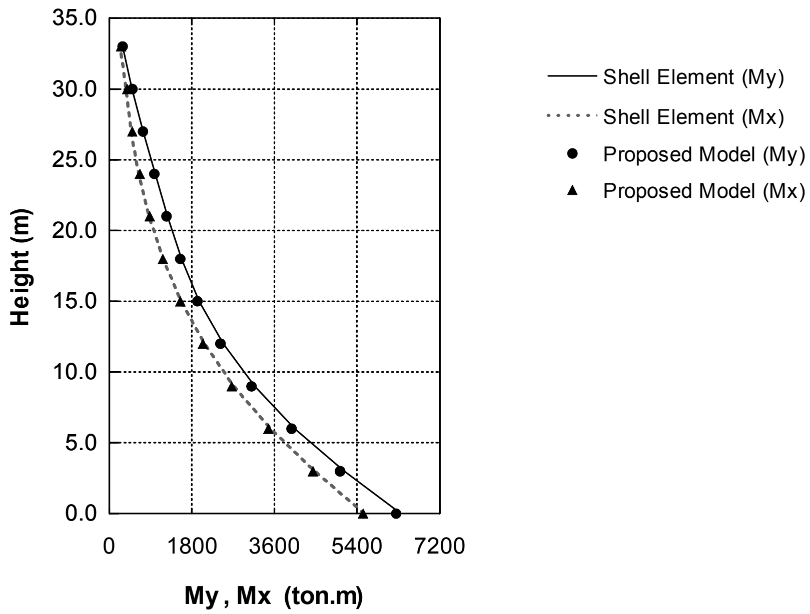


Figure 5.52 Comparison of Resultant Bending Moments in the Shear Wall Assembly in BS4-12 Obtained by Resp. Spec. Analyses in x and y-directions

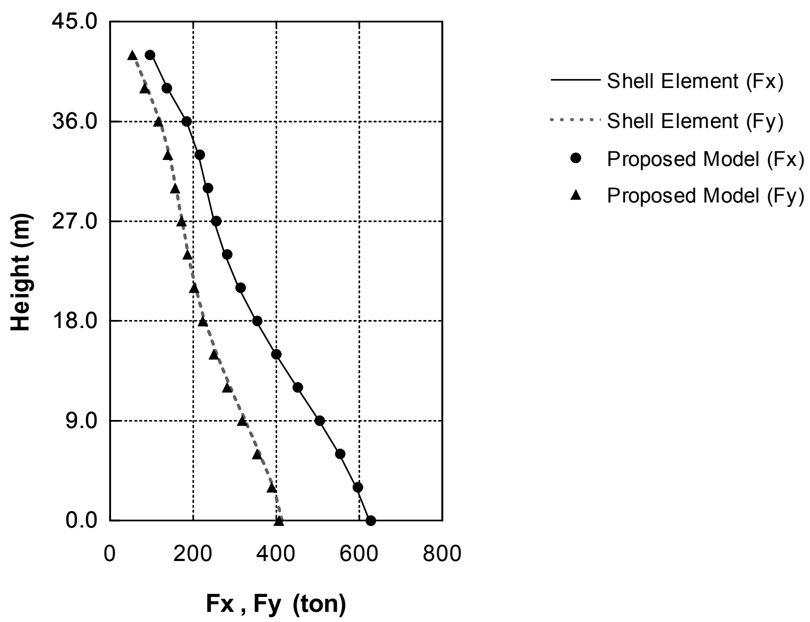


Figure 5.53 Comparison of Resultant Shear Forces on the Shear Wall Assembly in BS5-15 Obtained by Response Spectrum Analyses in x and y-directions

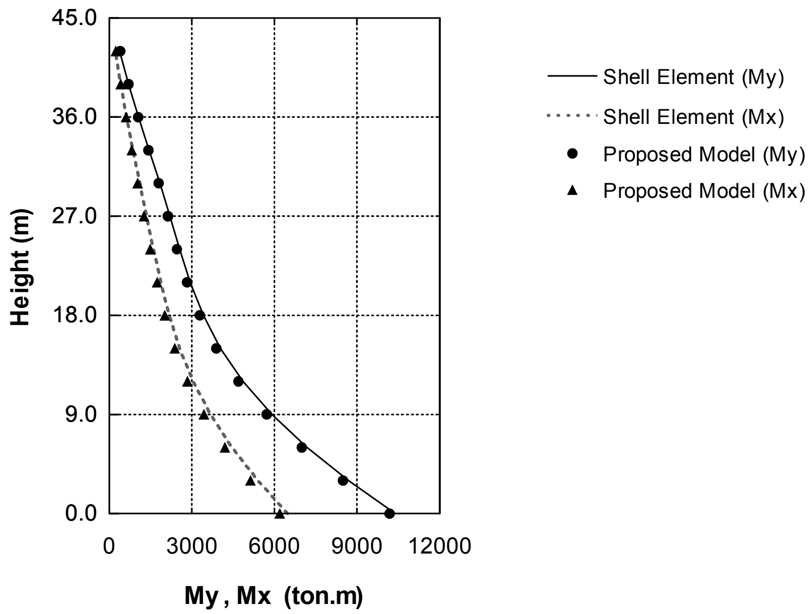


Figure 5.54 Comparison of Resultant Bending Moments in the Shear Wall Assembly in BS5-15 Obtained by Response Spectrum Analyses in x and y-directions

are 1.68 % for F_x and 3.47 % for F_y when the analyses using SAP2000 shell elements and the proposed models are compared. The maximum relative differences in resultant bending moments are 2.86 % for M_x and 5.02 % for M_y.

5.1.2.5 Base Forces

The base forces (base shear and bending moments) are quite important in earthquake analysis and the design of building structures. Especially in dynamic analysis, the total effect of lateral inertia forces acting on the structure can be determined by base forces. In the analyses, total shear forces and bending moments at the base of the sample building structures are obtained in order to check the validity of the proposed models. In Table 5.17, the results obtained by performing response spectrum analysis in x direction are presented. Table 5.18 gives the shear forces and bending moments at the base of the sample building structures obtained by response spectrum analysis in y direction. The maximum relative differences obtained for the base shear forces and bending moments of the five sample building structures are 2.03 % for F_x, 4.07 % for F_y, 2.29 % for M_x and 2.07 % for M_y.

Table 5.17 Comparison of Base Forces on the Sample Building Structures (Response Spectrum Analysis in x-direction)

Building Type	SAP2000 S.E.	Proposed Model
BS1-3		
Base Shear-F_x (t)	108.799	111.007
Base Bending Moment-M_y (t.m)	773.522	789.503
BS2-6		
Base Shear-F_x (t)	622.292	627.855
Base Bending Moment-M_y (t.m)	8223.755	8273.918
BS3-9		
Base Shear-F_x (t)	316.104	320.260
Base Bending Moment-M_y (t.m)	5750.044	5836.639
BS4-12		
Base Shear-F_x (t)	481.150	484.480
Base Bending Moment-M_y (t.m)	10710.871	10711.735
BS5-15		
Base Shear-F_x (t)	652.850	659.963
Base Bending Moment-M_y (t.m)	17318.902	17519.181

5.1.3 Performance of the Proposed Models in Time History Analysis

The performance of the proposed shear wall models in time history analysis are checked against the models in which shell elements of SAP2000 are used for modelling shear wall assemblies. The results of the time history analyses of the three sample building structures (BS2-6, BS4-12 and BS5-15), in which two modelling techniques were used, are presented in this part. The same mass and inertia values used in the response spectrum analyses are considered in the time history analyses.

The acceleration-time record of El Centro Earthquake (Fig. 5.55) is applied directly to the base of the sample building structures. The record of the first 20 seconds of the earthquake, having a stepsize of 0.02 seconds, is used in the analyses. The acceleration values in the original record, in terms of g (gravitational acceleration), are converted to meters per second by multiplying by 9.81. A damping ratio of 5 per cent is used in the analyses.

Table 5.18 Comparison of Base Forces on the Sample Building Structures (Response Spectrum Analysis in y-direction)

Building Type	**SAP2000 S.E.**	**Proposed Model**
BS1-3		
Base Shear-F_y (t)	100.007	100.997
Base Bending Moment-M_x (t.m)	695.144	702.390
BS2-6		
Base Shear-F_y (t)	423.275	440.511
Base Bending Moment-M_x (t.m)	5290.628	5359.226
BS3-9		
Base Shear-F_y (t)	224.476	228.359
Base Bending Moment-M_x (t.m)	3959.739	4050.426
BS4-12		
Base Shear-F_y (t)	463.908	471.694
Base Bending Moment-M_x (t.m)	10439.675	10561.303
BS5-15		
Base Shear-F_y (t)	471.974	473.311
Base Bending Moment-M_x (t.m)	11937.788	11940.819

Time history analyses are performed on the x and y directions of the sample building structures and the following parameters are computed for both models:

1. Maximum base shear
2. Maximum displacement of the top storey
3. Maximum resultant shear at the base of the shear wall assembly.

5.1.3.1 Base Shear

Figures 5.56 to 5.58 give the base shear history graphs obtained by time history analysis in x-direction for the three sample buildings, in which the nonplanar shear wall assemblies are modelled by the proposed models. Maximum base shear forces of the two modelling techniques are compared in Table 5.19 for three sample structures. In this table, the maximum base shear in x direction (F_x) is obtained by time history analysis of the sample building structures in x-direction and the maximum base shear in y direction (F_y) is obtained by time history analysis in y-direction.

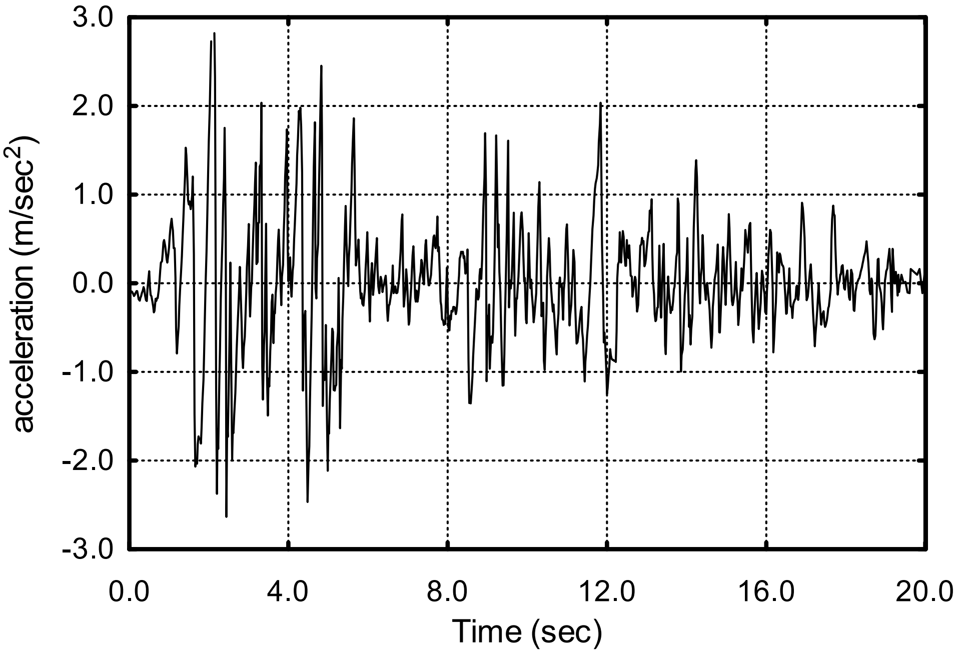


Figure 5.55 El Centro Earthquake Record

Table 5.19 Comparison of Maximum Base Shear Forces on the Sample Building Structures

Building Type	SAP2000 S.E.	Proposed Model
BS2-6		
Max.Base Shear-F_x (t)	806.40	804.00
Max.Base Shear-F_y (t)	539.40	589.60
BS4-12		
Max.Base Shear-F_x (t)	539.00	534.20
Max.Base Shear-F_y (t)	480.40	500.20
BS5-15		
Max.Base Shear-F_x (t)	485.00	516.20
Max.Base Shear-F_y (t)	464.80	453.70

For the three sample building structures, the maximum relative differences in base shear forces between the two modelling techniques are 9.31 % for the base shear in x-direction (F_x) and 6.43 % for the base shear in y-direction (F_y).

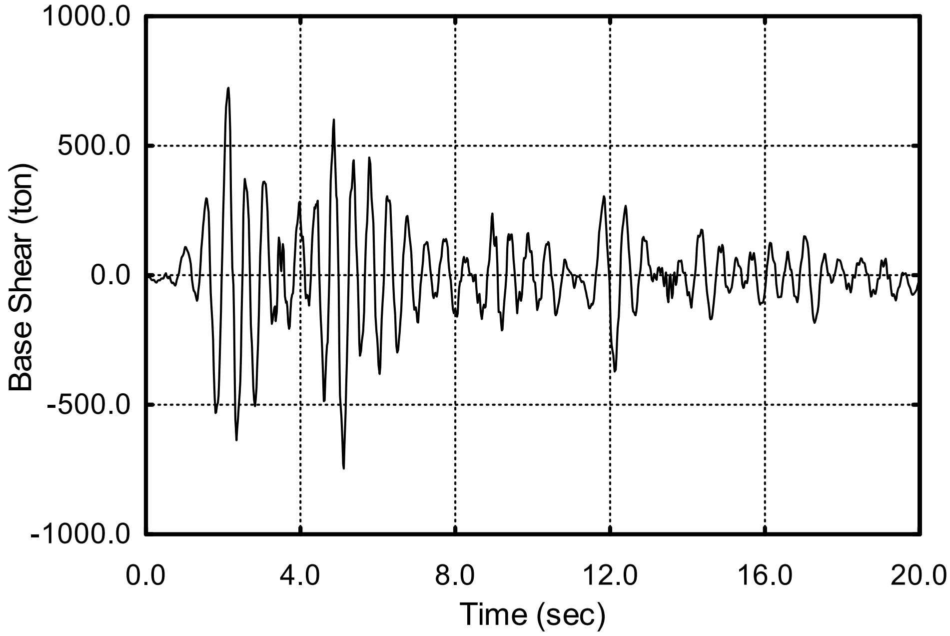


Figure 5.56 Base Shear Force History for Building Structure BS2-6 (Time History Analysis in x-direction)

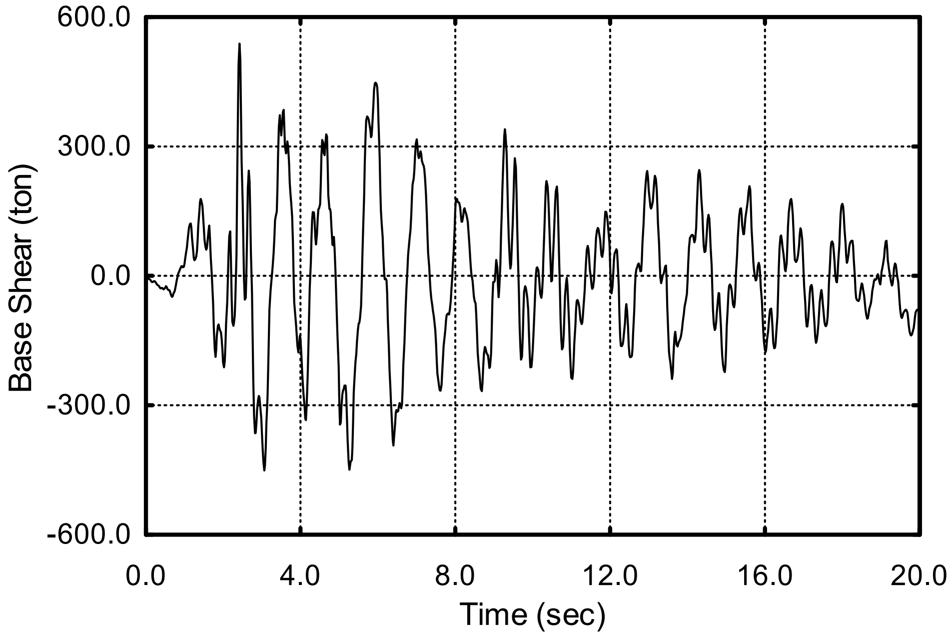


Figure 5.57 Base Shear Force History for Building Structure BS4-12 (Time History Analysis in x-direction)

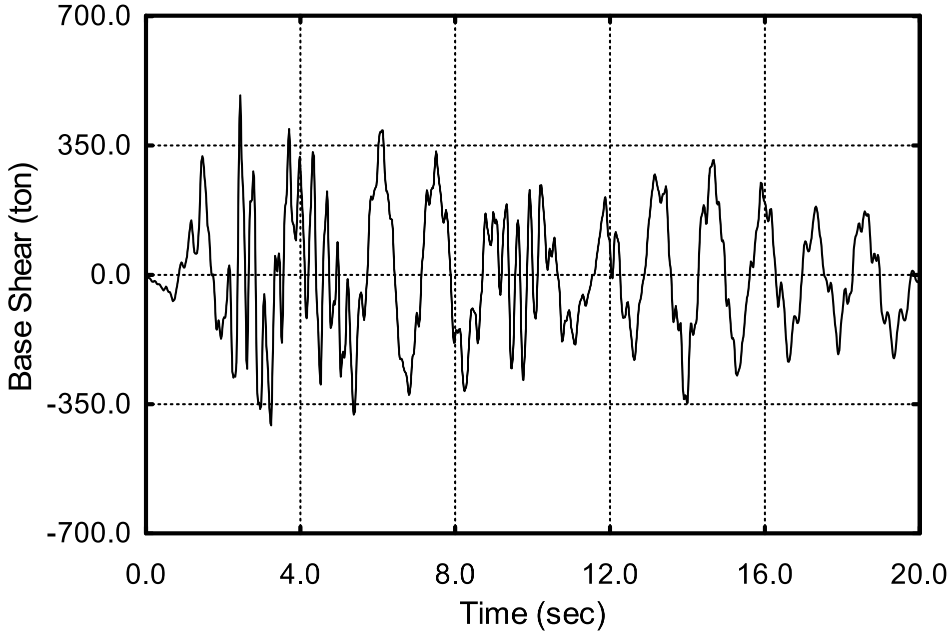


Figure 5.58 Base Shear Force History for Building Structure BS5-15 (Time History Analysis in x-direction)

5.1.3.2 Displacement of the Top Storey

Figures 5.59 to 5.61 give the top storey displacement history graphs obtained by time history analyses in x-direction using the proposed modelling techniques for three sample buildings. Table 5.20 provides a comparison of the two techniques as regards the maximum top storey displacements for the three structures. In the table, u_x is obtained by time history analysis of the sample building structures in x-direction and u_y is obtained by time history analysis in y-direction. The maximum relative difference between the two models is 1.23 % for maximum top storey displacements in x-direction and 2.31 % for maximum top storey displacements in y-direction.

5.1.3.3 Resultant Shear Force at the Base of the Shear Wall Assembly

Figures 5.62 to 5.64 give the graphs for the resultant shear force history at the base of the shear walls obtained by time history analysis in x-direction using the proposed modelling techniques for the shear wall assemblies of three sample buildings. Table 5.21 compares the two techniques in terms of the maximum resultant shear forces of the three sample structures. In the table, F_x is obtained by time history analysis of

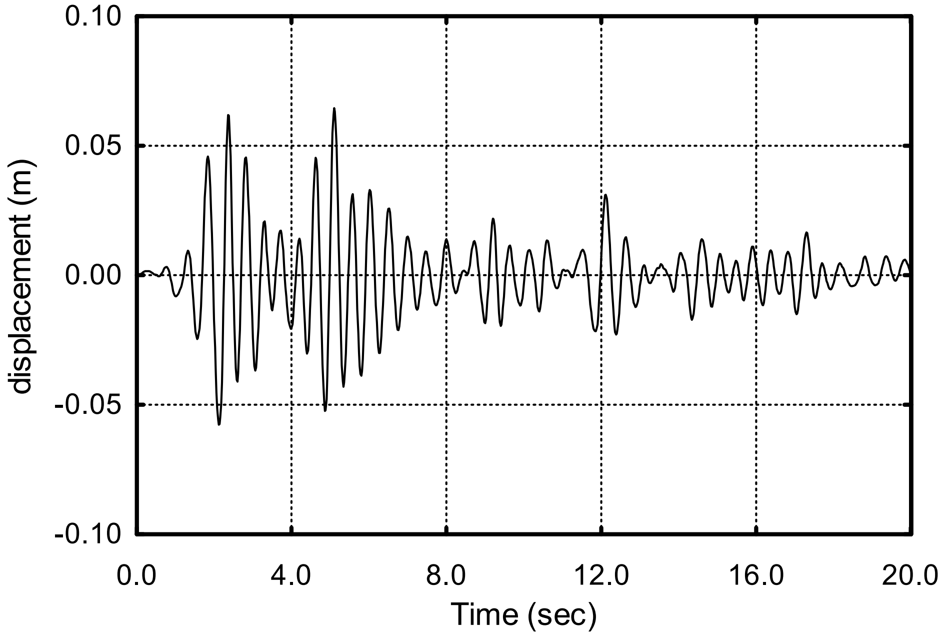


Figure 5.59 Top Storey Displacement History for Building Structure BS2-6 (Time History Analysis in x-direction)

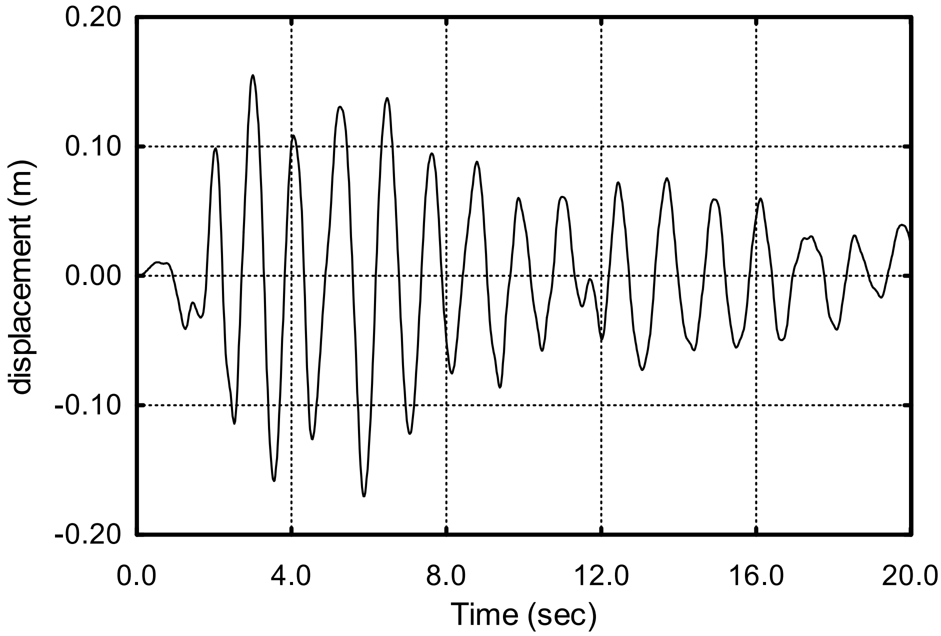


Figure 5.60 Top Storey Displacement History for Building Structure BS4-12 (Time History Analysis in x-direction)

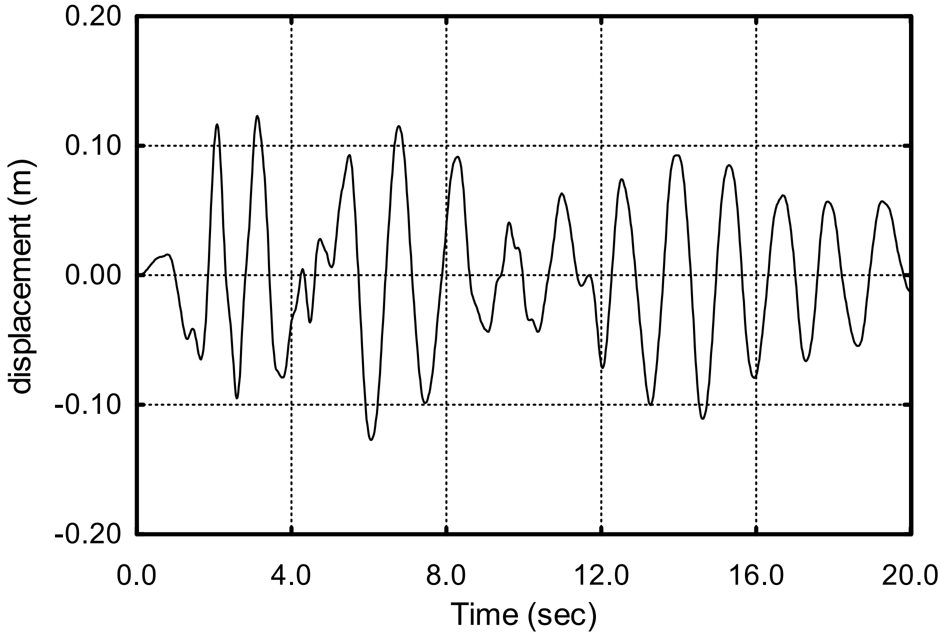


Figure 5.61 Top Storey Displacement History for Building Structure BS5-15 (Time History Analysis in x-direction)

Table 5.20 Comparison of Maximum Top Storey Displacements of Sample Building Structures

Building Type	SAP2000 S.E.	Proposed Model
BS2-6		
Max.Top Storey Disp-u_x (m)	0.06454	0.06480
Max.Top Storey Disp-u_y (m)	0.07219	0.07386
BS4-12		
Max.Top Storey Disp-u_x (m)	0.17030	0.17240
Max.Top Storey Disp-u_y (m)	0.16500	0.16500
BS5-15		
Max.Top Storey Disp-u_x (m)	0.12720	0.12790
Max.Top Storey Disp-u_y (m)	0.17440	0.17100

sample building structures in x-direction and F_y is obtained by time history analysis in y-direction.

The maximum relative differences in the resultant shear forces at the base of the

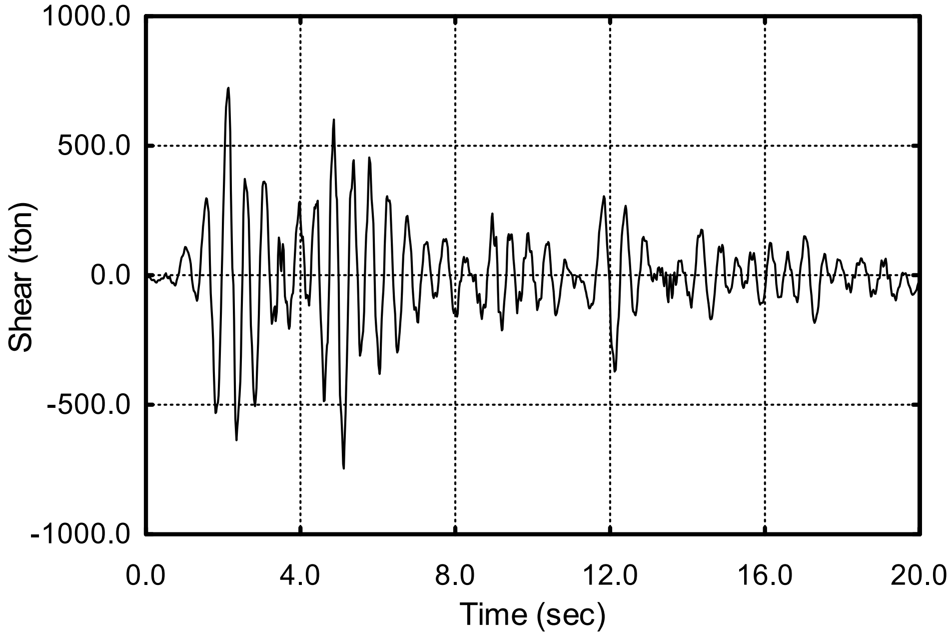


Figure 5.62 Resultant Shear Force History at the base of the Shear Wall Assembly for Building Structure BS2-6 (Time History Analysis in x-direction)

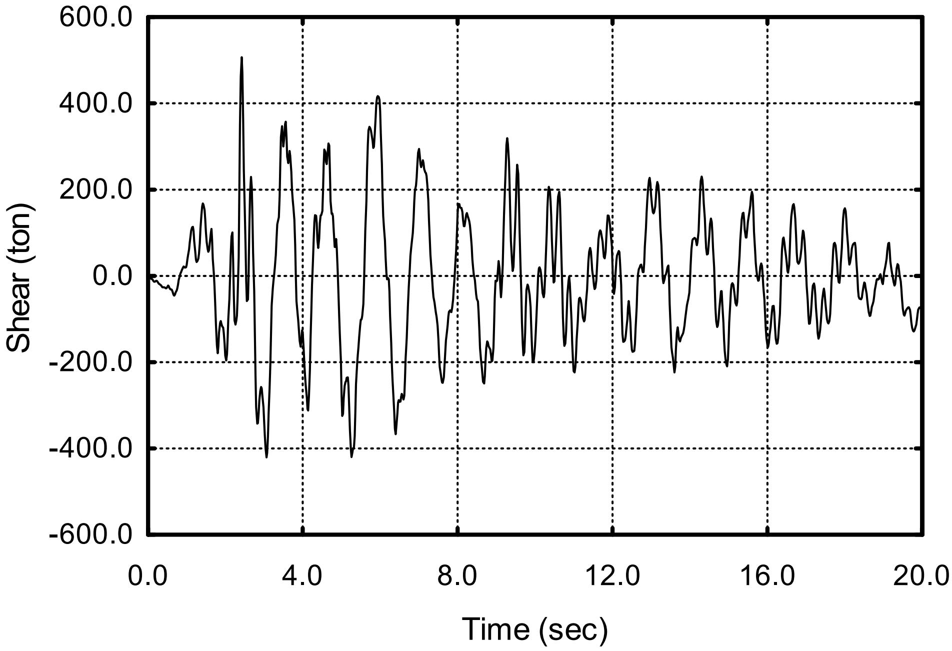


Figure 5.63 Resultant Shear Force History at the base of the Shear Wall Assembly for Building Structure BS4-12 (Time History Analysis in x-direction)

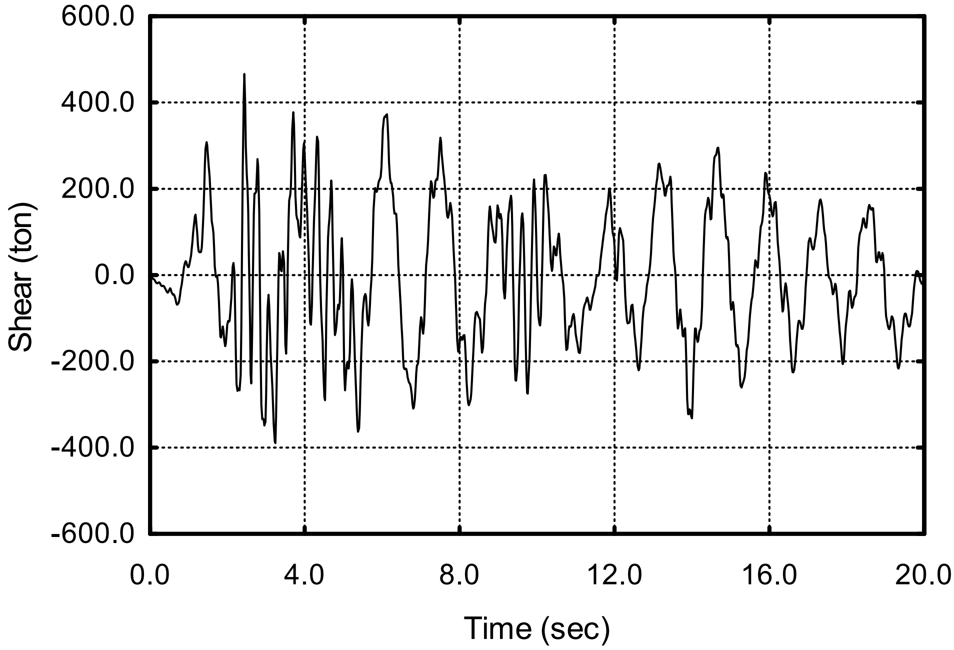


Figure 5.64 Resultant Shear Force History at the base of the Shear Wall Assembly for Building Structure BS5-15 (Time History Analysis in x-direction)

Table 5.21 Comparison of Maximum Resultant Shear in the base of the Shear Wall Assembly in the Sample Building Structures

Building Type	SAP2000 S.E.	Proposed Model
BS2-6		
Max.Resultant Shear Force-F$_x$ (ton)	746.300	739.300
Max.Resultant Shear Force-F$_y$ (ton)	401.300	410.200
BS4-12		
Max.Resultant Shear Force-F$_x$ (ton)	507.400	498.100
Max.Resultant Shear Force-F$_y$ (ton)	403.900	423.200
BS5-15		
Max.Resultant Shear Force-F$_x$ (ton)	467.200	494.400
Max.Resultant Shear Force-F$_y$ (ton)	393.200	391.200

shear wall assemblies obtained for the two models are 5.82 % for the time history analysis in x-direction and 4.78 % for the time history analysis in y-direction.

5.2 Comparison with the Results of Previous Studies

In order to check the validity of the proposed models, five different studies of non-planar shear wall assemblies were considered. These are

1. Variable thickness core assembly analyzed by Kwan [75] and Nadai and Johnson [76].

2. Coupled nonplanar shear wall assembly analyzed by Tso and Biswas [77], Ho and Liu [78] and Kwan [55].

3. Shear wall-frame building structure analyzed by Hoenderkamp [30].

4. Shear wall-frame building structure (DKP1) analyzed by Özmen [79].

5. Shear wall-frame building structure (DKP3) analyzed by Özmen [79].

5.2.1 Variable Thickness Core Assembly

In the studies of Kwan [75] and Nadjai and Johnson [76], a 100 meters high variable thickness closed core wall, which is subjected to torsion, is analyzed. The core consists of two of layers different thickness The thickness of the first part (between 0 and 50 meters) is 1.0 meters and the thickness of the second part (between 50 and 100 meters) is 0.5 meters. The structure is subjected to a torsion of 100 t-m at the top. The cross-section of the core assembly is given in Figure 5.65. The dimensions of the core are taken as 10 meters by 10 meters.

Kwan [75] derived a solid wall element and used it in the analysis of the assembly. In his study, the structure is divided into 20 storeys, each 5.0 meters in height. Each

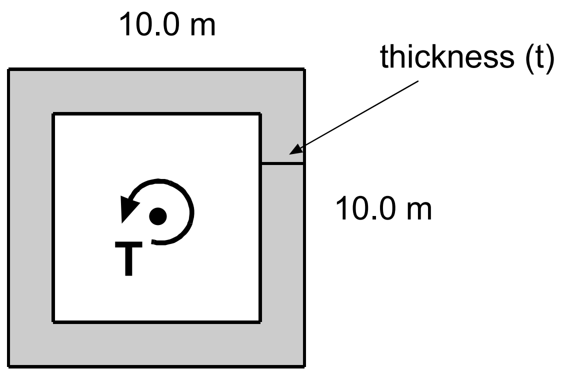


Figure 5.65 Cross-section of the Core

storey is modelled by four solid wall elements interconnected to form a hollow section. The exact theoretical values based on the theory of Bredt–Batho were also given in that study.

Nadjai and Johnson [76] studied the same structure using the discrete force method. Similar to Kwan, they divided the structure into 20 storeys, each of them 5.0 meters high and modelled the shear walls using four discrete solid wall elements.

The proposed model consists of four wide columns located at the middle of each planar shear wall in the assembly and rigid beams at floor levels. Similar to the studies of Kwan and Nadjai and Johnson, the core is divided into 20 storeys each a height of 5.0 meters. The structure is also modelled using SAP2000 shell elements, in which the planar wall modules between two floor levels are divided into 16 elements (4x4).

The displacement values that were obtained by Kwan, Nadjai and Johnson, Bredt-Batho theory, SAP shell elements and proposed model are compared in Figure 5.66.

As the graph indicates, there is good agreement between the proposed model and the Bredt-Batho theory, considering the floor rotations.

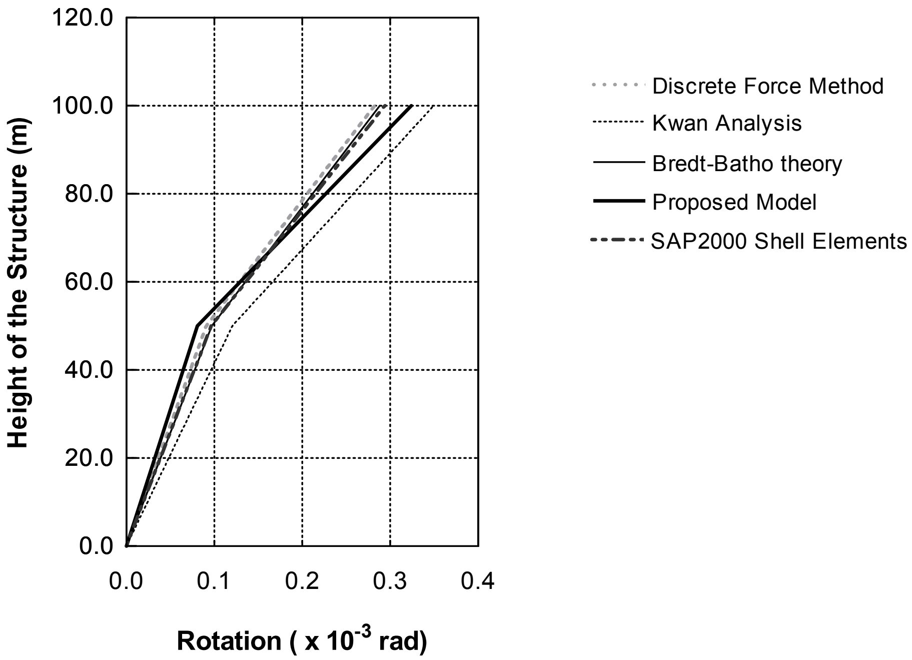


Figure 5.66 Comparison of the Rotations of the Core Assembly

5.2.2 Coupled Nonplanar Shear Wall Assembly

A six-storey plexiglas structure consists of three planar walls interconnected to form a U-channel-shaped shear wall assembly. The central wall has a row of openings at the middle and the assembly is considered to have four planar wall units and a row of coupling beams. The total height of the structure is 48 inches, and it is subjected to a lateral load of 25 pounds at the top. The connecting beams have a depth of 1.5 inches. The other dimensions of the assembly are given in Figure 5.67. The material properties of the model are taken to be E=0.40 x 10^6 psi and G=0.148 x 10^6 psi.

This problem was first analyzed by Tso and Biswas [77]. The theoretical solution and experimental results are given in their study. The theoretical calculations are based on ignoring the axial deformation of the wall, which yields significant errors in the computations. Ho and Liu [78] studied the same structure and analyzed it by using a method which is a combination of the finite strip method and the continuum method.

Kwan [55] also studies the assembly using improved wide column analogy. He used solid wall elements in modelling wall units and frame elements in modelling beams.

In the proposed model, shear wall units are modelled using wide columns and rigid beams at floor levels. In addition, the rigid beam connections are torsionally released. Just like in Kwan, the connecting beams are modelled using frame elements.

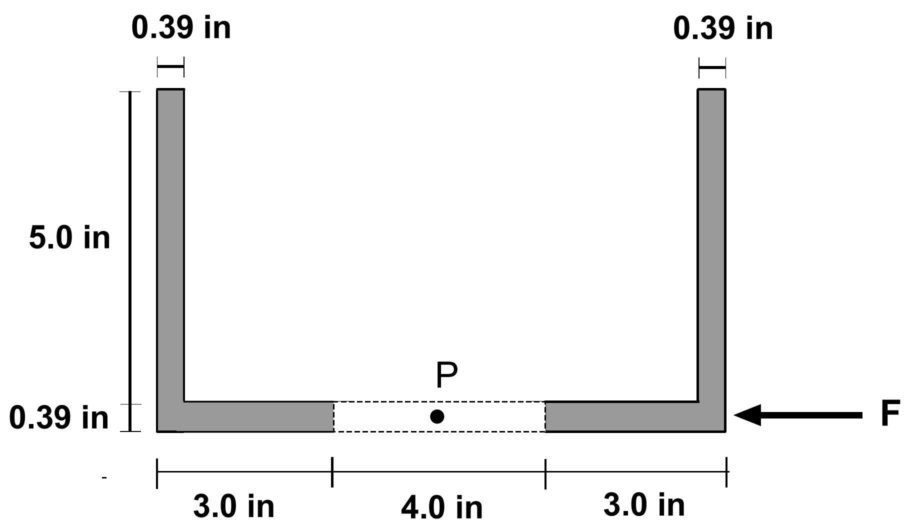


Figure 5.67 Plan of the Open Section Shear Wall Assembly

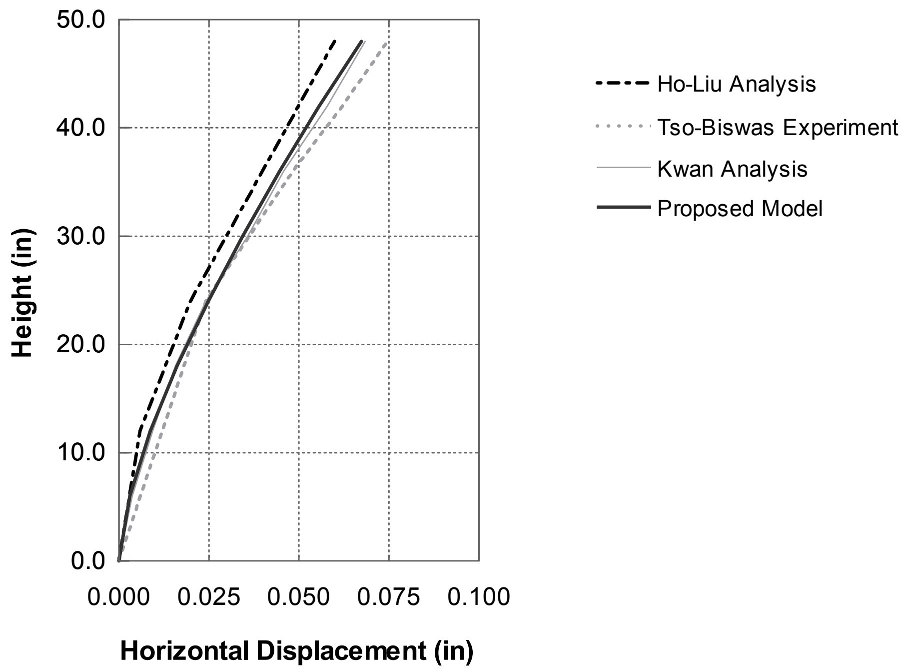


Figure 5.68 Horizontal Displacements of the Open Shear Wall Assembly

In Figures 5.68 and 5.69, the horizontal displacements and rotations of the point P on the floor levels of the assembly are compared respectively. Good agreement is obtained between the proposed model and the experimental results (a relative difference of 9.83 % in top floor displacement and 2.96 % in top floor rotation).

5.2.3 Shear Wall-Frame Building Structure Analyzed by Hoenderkamp

Hoenderkamp [30] studied an asymmetric tall building structure with cores as shown in Figure 5.70. The building structure consists of a core with lintel beams, four single shear walls and five identical rigid frames. It has 16 storeys with a total height of 48 meters. A horizontal load of 40 kN/m acts at the center of the structure in a direction parallel to the y-axis. The modulus of elasticity is taken as 20 x 10^6 kN/m^2. The core has a wall thickness of 0.2 meters and the lintel beams measure 0.2 meters by 0.5 meters. All other dimensions are given in Figure 5.70. Hoenderkamp used a three dimensional analytical method based on the continuum approach in the analysis of the

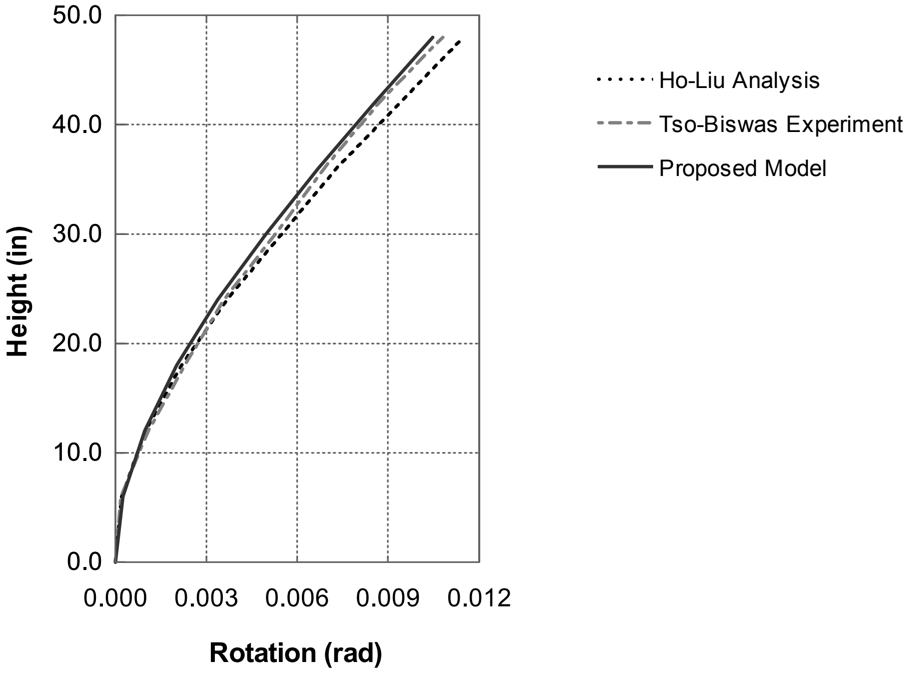


Figure 5.69 Rotations of the Open Shear Wall Assembly

structure. In his study, he compared the results with a computer stiffness matrix analysis where the core with lintel beams are replaced by a line member with seventh degree of freedom to represent warping.

In modelling the building structure according to the proposed model, planar wall units are replaced by wide columns and rigid beams and it is assumed that floors are infinitely rigid. The shear walls of the building are also modelled using the SAP2000 shell elements. In Figures 5.71 and 5.72, the results of the deflections and rotations of the core assembly are compared. The difference at the locations of maximum deformation is 4.71 % between the proposed model and Hoenderkamp's analytical method . This value is 1.8 % between the analysis results of Hoenderkamp's method and those obtained using SAP2000 shell elements. The difference at the locations of maximum rotation is 4.76 % between the proposed model and the analytical method and 0.95 % between the SAP2000 shell elements and the analytical method.

In Figures 5.73 and 5.74, the shear forces and bending moments acting on the core assembly are compared. Especially in the shear forces and bending moments at the first

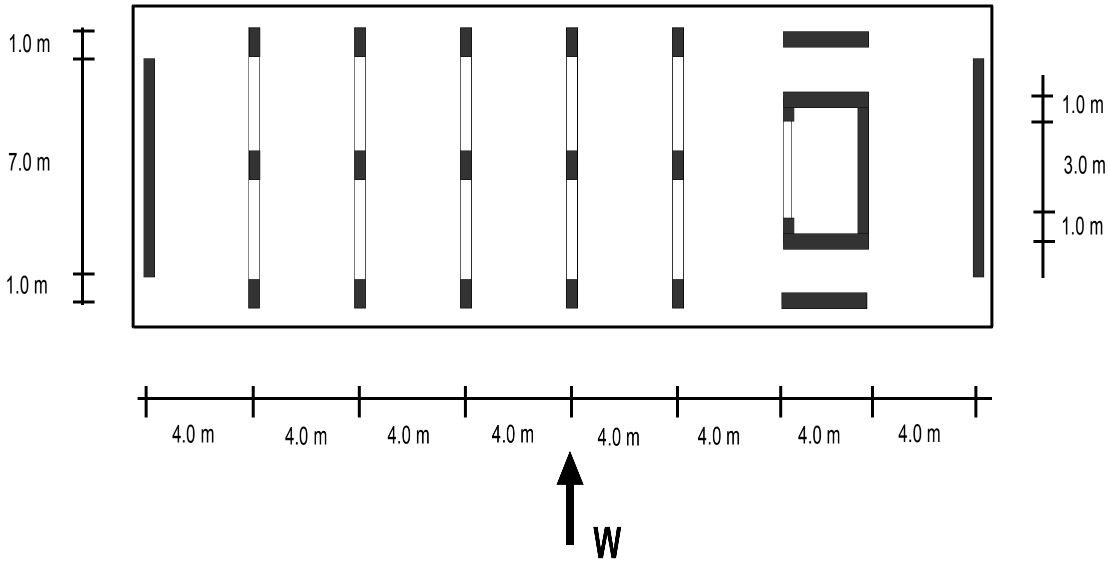


Figure 5.70 Plan View of the Shear Wall-Frame Structure

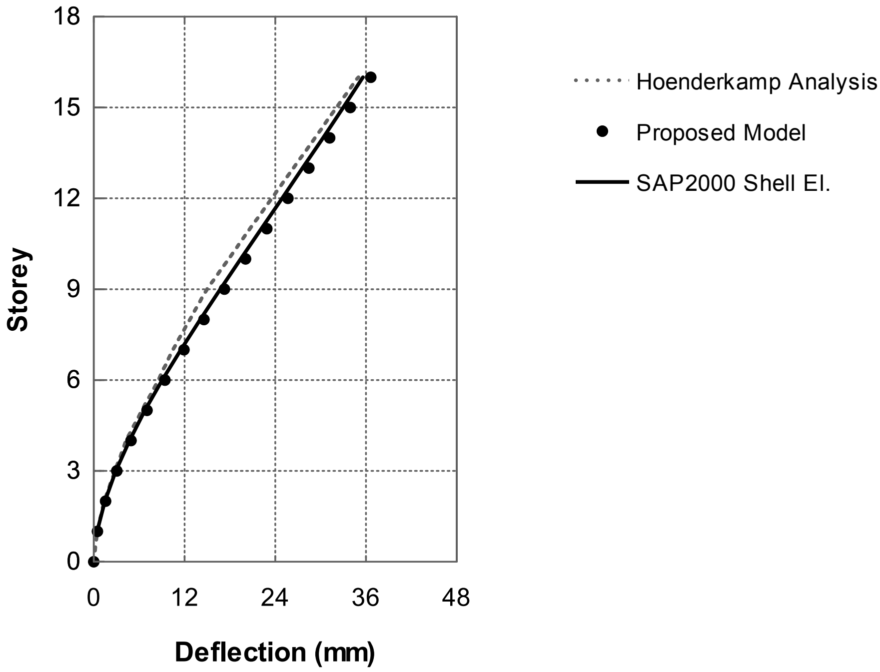


Figure 5.71 Comparison of Storey Deflections

and second floors, there is a significant difference between the analysis results given by Hoenderkamp and the results obtained by using the proposed model and SAP2000. The results of the analysis using the proposed model agree well with the results of the

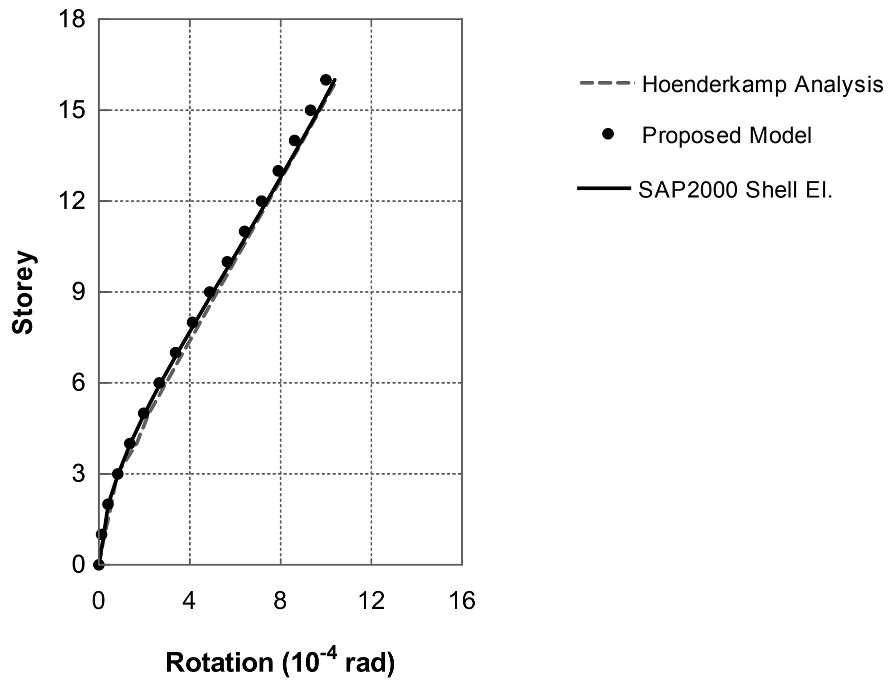


Figure 5.72 Comparison of Storey Rotations

analysis in which shear wall assemblies are modelled using SAP2000 shell elements.

5.2.4 Shear Wall-Frame Building Structure (DKP1) Analyzed by Özmen

Özmen [79] studied a ten-storey reinforced concrete shear wall-frame structure consisting of two nonplanar U shaped shear walls. The height of the first floor is 4.0 meters and the height of each of the other floors is 3.0 meters. The floor plan of the building structure is given in Figure 5.75.

All the beams of the building have dimensions of 0.25 x 0.50 meters and the thickness of the shear walls is given as 0.25 meters. The dimensions of the columns of the building are given in Table 5.22. The weight of the top floor, normal floors and the first floor are 165 tons, 238 tons and 291 tons respectively. The modulus of elasticity of concrete is taken as 2.5 x 10^6 t/m^2. The building structure was analyzed using the BİLSAR-YAPI 2000 computer program and the first two natural periods in the x and y directions (T_x and T_y) were computed. The shear distribution on the first floor of the building structure obtained by applying axisymmetric lateral loading in the y-direction

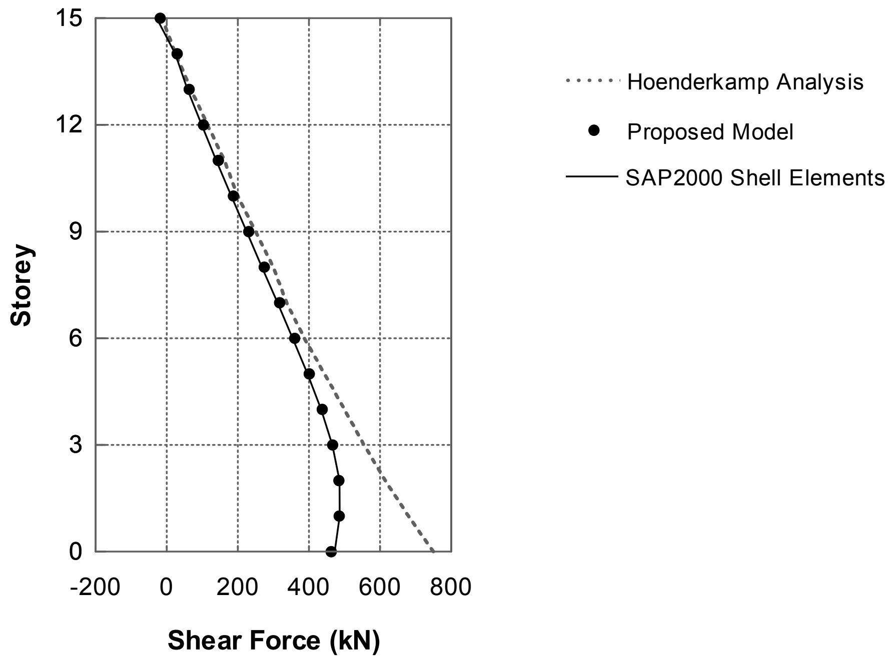


Figure 5.73 Comparison of Shear Forces on the Core

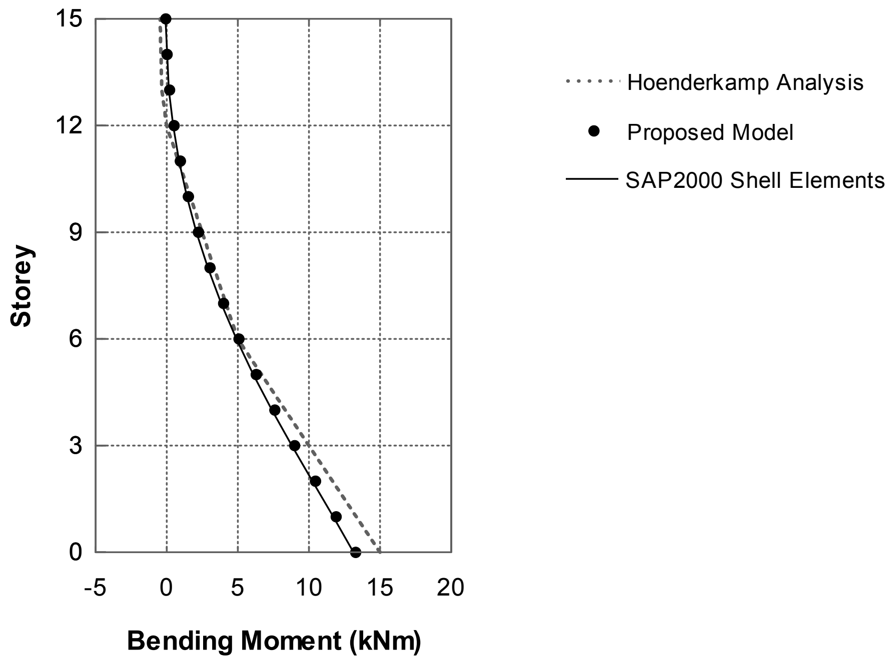


Figure 5.74 Comparison of Bending Moments on the Core

Table 5.22 Dimensions of the Columns of DKP1 Building Structure

Floor	S1	S2	S3
10-9	30 x 30	30 x 30	30 x 30
8-7	30 x 30	30 x 40	40 x 40
6-5	30 x 40	30 x 45	40 x 40
4-3	30 x 50	30 x 55	40 x 50
2-1	30 x 60	30 x 70	40 x 60

is also given in that study.

The proposed model, which consists of wide columns and rigid beams, is used to model the U-shaped shear wall assemblies of the building structure. The natural peri-

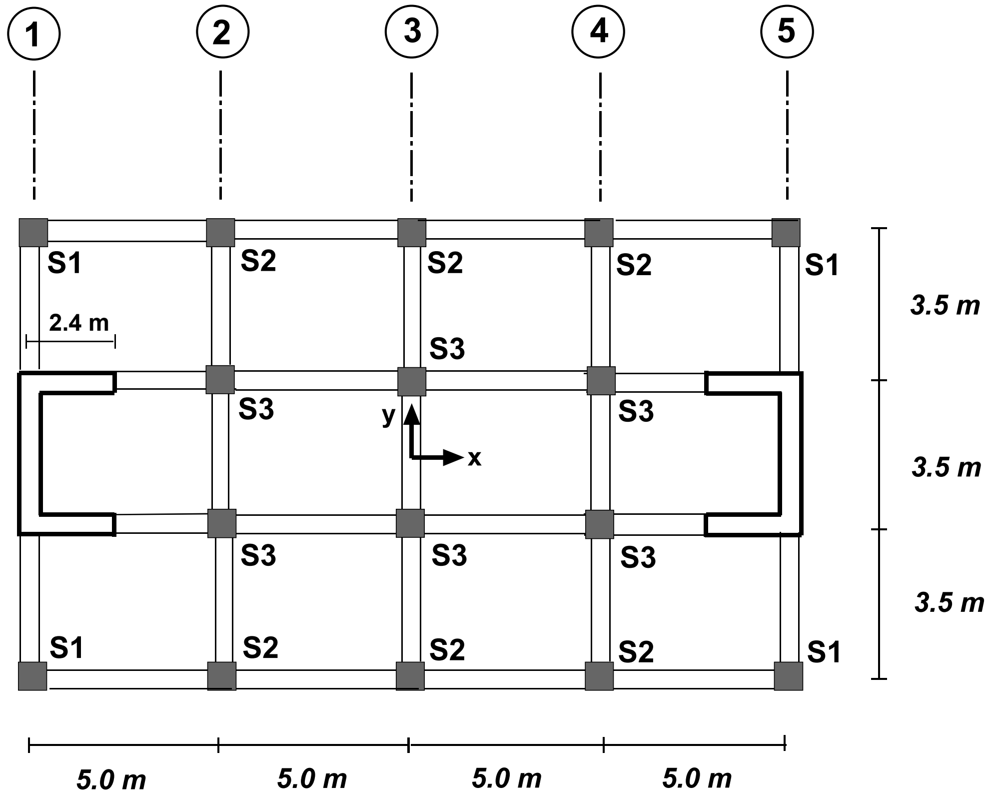


Figure 5.75 Floor Plan of the Building Structure DKP1

Table 5.23 Comparison of the First Two Natural Vibration Periods of DKP1

Period	Özmen (BILSAR-YAPI2000) (s)	Proposed Model (s)
T_x	0.8600	0.8758
T_y	0.7700	0.7566

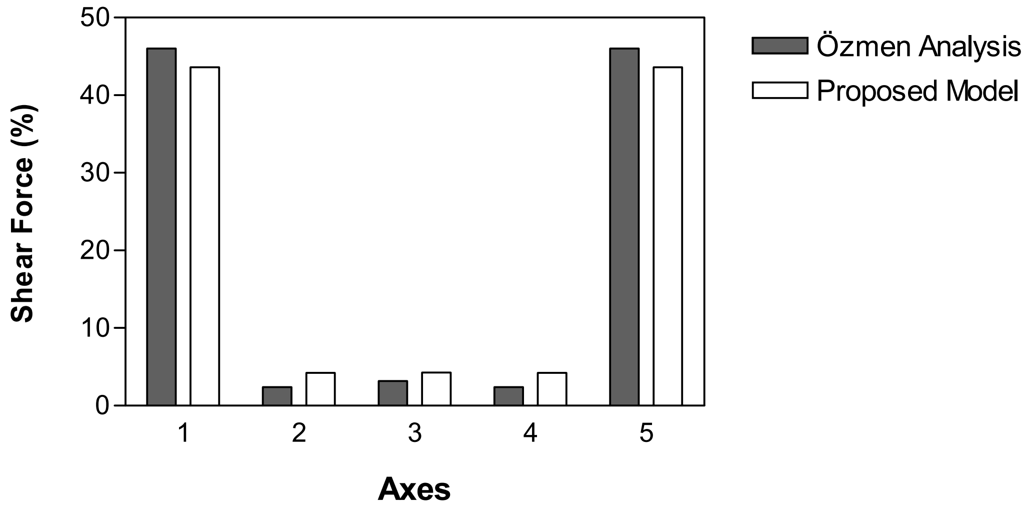


Figure 5.76 Comparison of the Distribution of the Shear Force at the First Floor of Building Structure DKP1

ods of the structure are compared in Table 5.23. The maximum difference in natural periods is less than 2.0%. A comparison of the shear force distribution at the axes of the first floor is given in Figure 5.76. In the view of the results, it can be stated that there is good agreement between two studies.

5.2.5 Shear Wall-Frame Building Structure (DKP3) Analyzed by Özmen

In the Özmen's study [79], an asymmetric shear wall-frame structure is analyzed (Figure 5.77). Two L shaped shear walls are located at the left side of the structure and a planar shear wall is located at the right side of the structure. Similar to the building

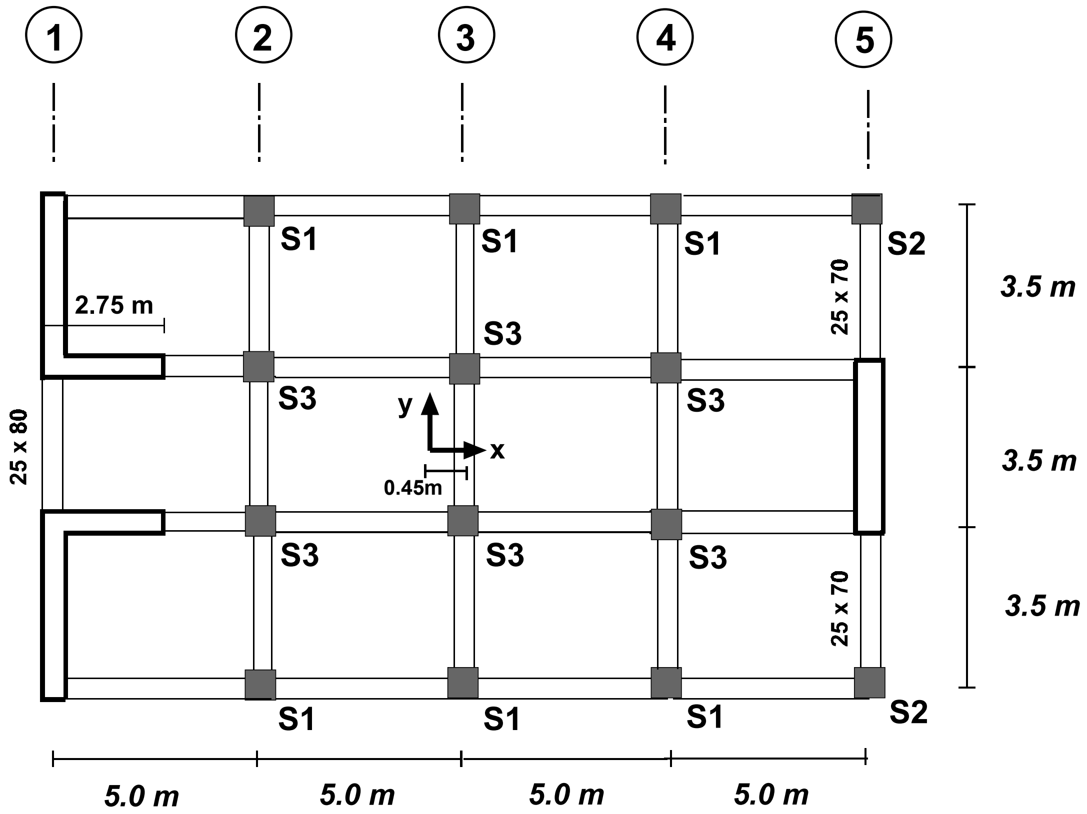


Figure 5.77 Floor Plan of Building Structure DKP3

structure presented above, the building has ten storeys with a total height of 31 meters. The thickness of the shear walls is taken as 0.25 meters and the column dimensions are given in Table 5.24. All beam dimensions are 0.25 x 0.50 meters except the beams at axis-1 (0.5 x 0.80 meters) and at axis-5 (0.25 x 0.70 meters). Equivalent lateral load analysis is performed on the building structure in the y direction and the floor loads are applied at the points located 0.45 m left of the geometric centers of the floors. The BİLSAR-YAPI 2000 computer program is used in the analysis.

The proposed model for open sections is used for modelling the nonplanar shear wall assemblies of the building structure. The planar wall at axis-5 is modelled using the conventional wide column method. A comparison of the shear force distribution at the first floor obtained by two modelling techniques is given in Figure 5.78. As seen from the figure, similar distributions of shear forces are obtained.

Table 5.24 Dimensions of the Columns of Building Structure DKP3

Floor	S1	S2	S3
10-9	30 x 30	30 x 30	30 x 30
8-7	30 x 40	30 x 40	40 x 40
6-5	30 x 40	30 x 50	40 x 40
4-3	30 x 45	30 x 60	40 x 50
2-1	30 x 50	30 x 70	40 x 60

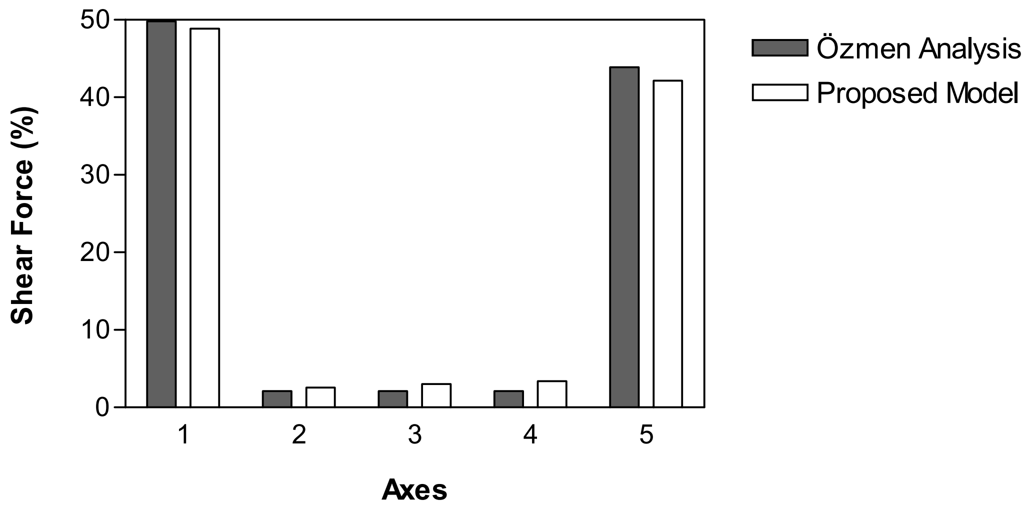


Figure 5.78 Comparison of the Distribution of the Shear Force at the First Floor of Building Structure DKP3

CHAPTER 6

CONCLUSION

In the lateral load analysis of shear wall-frame structures, a fundamental goal is to model the shear wall assemblies using a realistic and feasible method. Modelling and analysis of symmetric building structures having planar shear walls in two dimensions may be a practical solution. However, especially for asymmetric shear wall-frame building structures, which have nonplanar shear wall assemblies, modelling and analysis studies should be in three dimensions.

Three dimensional analysis of shear wall-frame structures may not be feasible for the cases in which the shear wall assemblies are modelled using a large number of plane stress elements. A reasonable method should be used in modelling such building structures.

In this study, three dimensional modelling of nonplanar shear wall assemblies of shear wall-frame structures is investigated. Modelling techniques and methods that are used in analysis of such building structures are presented in Chapter 2. In Chapter 3, common methods for two and three dimensional modelling of nonplanar shear wall assemblies are reviewed. The proposed nonplanar shear wall models for (a) open sections and (b) closed sections are presented in Chapter 4. The results of the verification studies on the proposed models are given in Chapter 5.

The proposed shear wall models are based on the conventional wide column analogy and they can be used effectively in the analysis of multistorey building structures where rigid diaphragm floor assumption is valid. The model for open section shear wall assemblies consists of wide columns located at the middle of the planar shear wall units between two floor levels. These wide columns have the same stiffness properties as the planar shear wall units. Rigid beams are located at the floor levels of the building structure and the connections between the perpendicular rigid beams are released against torsion, which prevents the transfer of torsional moments. As a result, the additional torsional stiffness of the rigid beams is removed from the structural system.

Similar to the open section model, the model proposed for closed section shear wall assemblies consists of wide columns and rigid beams. In addition to releasing torsional moments at the rigid beam connections, the torsional stiffness of the wide columns of the model are adjusted in such a way that the torsional stiffness of the closed section and the sum of the torsional stiffness of the wide columns are equivalent.

The performance of the proposed models is checked by

(a) equivalent lateral load analysis,

(b) response spectrum analysis and

(c) time history analysis

of several sample shear wall-frame building structures having different shapes of shear wall assemblies. The results of the analyses, in which the proposed models are used, are compared with the results obtained using SAP shell elements and ETABS wall elements. Good agreement with the models using SAP shell elements was obtained in both static and dynamic analyses.

The validity of the proposed models was also investigated by considering a number of structures studied by several authors. The analysis results obtained by using the proposed models agree well with those in the literature.

In view of the comparisons summarized above, the proposed methods can be used in modelling nonplanar shear wall assemblies of shear wall-frame structures where the rigid diaphragm floor assumption is valid. Displacement and resultant force values obtained by using the proposed models have an average relative difference of 6% with the results obtained using SAP2000 shell elements.

If the time spent on forming a building model is considered, using the proposed models in modelling nonplanar shear wall assemblies is less time consuming when compared with the models in which plane stress elements are used. The total running time of the computer in analyzing the building structures is also an important issue. In

Table 6.1 Comparison of the Two Shear Wall Models in Equivalent Lateral Load Analysis

Building Type	Total Number of El.	Size of Stiff. File (Bytes)	Running Time (s)
BS1-15			
SAP Shell Elements	480 Fr.El., 720 S.El.	7,952,552	19
Proposed Model	615 Fr.El.	363,100	8
BS2-15			
SAP Shell Elements	1500 Fr.El., 1200 S.El.	19,713,500	57
Proposed Model	1725 Fr.El.	2,141,476	15
BS3-15			
SAP Shell Elements	615 Fr.El., 960 S.El.	11,898,588	24
Proposed Model	795 Fr.El.	528,832	8
BS4-15			
SAP Shell Elements	1065 Fr.El., 960 S.El.	11,883,264	43
Proposed Model	1245 Fr.El.	1,170,280	10
BS5-15			
SAP Shell Elements	1245 Fr.El., 1200 S.El.	19,725,600	46
Proposed Model	1470 Fr.El.	1,608,436	12

Tables 6.1, 6.2 and 6.3, the total running time, the size of the stiffness files produced and the total number of structural elements used for modelling the building structures are given for some selected sample building structures, which are analyzed using the equivalent lateral load, response spectrum and time history methods. The computer used in the analyses is an IBM Thinkpad with a RAM of 64 MB and a Pentium II-500 MHz CPU. Especially in response spectrum and time history analysis, where the total number of calculations is relatively large, the building models for which the proposed models are used have a significant advantage as regards total running time and file size.

Table 6.2 Comparison of the Two Shear Wall Models in Response Spectrum Analysis

Building Type	**Total Number of El.**	**Size of Stiff. File (Bytes)**	**Running Time (s)**
BS1-3			
SAP Shell Elements	96 Fr.El., 144 S.El.	630,844	7
Proposed Model	123 Fr.El.	52,086	4
BS2-6			
SAP Shell Elements	600 Fr.El., 480 S.El.	7,970,056	50
Proposed Model	690 Fr.El.	767,368	13
BS3-9			
SAP Shell Elements	369 Fr.El., 576 S.El.	3,610,888	30
Proposed Model	477 Fr.El.	302,536	9
BS4-12			
SAP Shell Elements	852 Fr.El., 768 S.El.	7,943,000	52
Proposed Model	996 Fr.El.	916,552	16
BS5-15			
SAP Shell Elements	1245 Fr.El., 1200 S.El.	19,725,600	143
Proposed Model	1470 Fr.El.	1,608,436	25

In the analysis of building structures having a large number of elements and degrees of freedom, the improvement is more significant. In order to illustrate this situation, a 60 storey BS2 type building structure is considered and the shear wall assemblies of the structure are modelled using shell elements and the proposed wide column model separately. The three methods of analysis (equivalent lateral load, response spectrum and time history methods) are used for comparison. 6000 frame elements and 4800 shell elements are used in the mathematical model of the building structure where the shear walls are modelled using SAP2000 shell elements (4x4 elements for each planar shear wall module) and the size of the stiffness file obtained is 168.049 MB.

Table 6.3 Comparison of the Two Shear Wall Models in Time History Analysis

Building Type	Total Number of El.	Size of Stiff. File (Bytes)	Running Time (s)
BS2-6			
SAP Shell Elements	600 Fr.El., 480 S.El.	7,970,056	50
Proposed Model	690 Fr.El.	767,368	18
BS4-12			
SAP Shell Elements	852 Fr.El., 768 S.El.	7,943,000	62
Proposed Model	996 Fr.El.	916,552	22
BS5-15			
SAP Shell Elements	1245 Fr.El., 1200 S.El.	19,725,600	143
Proposed Model	1470 Fr.El.	1,608,436	31

For the same building structure, 6900 frame elements are used in the model when the proposed model is used and the size of the stiffness file is reduced to 11.829 MB. The total running times of the three analyses are compared in Table 6.4. According to these results, the data to be stored and the total running time can be reduced significantly when the proposed model is used.

Another advantage of the proposed models is that they can be used in any stiffness based three dimensional frame analysis program. The program should only have (a) rigid diaphragm assignment option and (b) joint release definition. Using a general frame analysis program which models the structures only by frame elements is a more economical way of analyzing shear wall-frame building structures when compared with programs having both frame and plane stress elements.

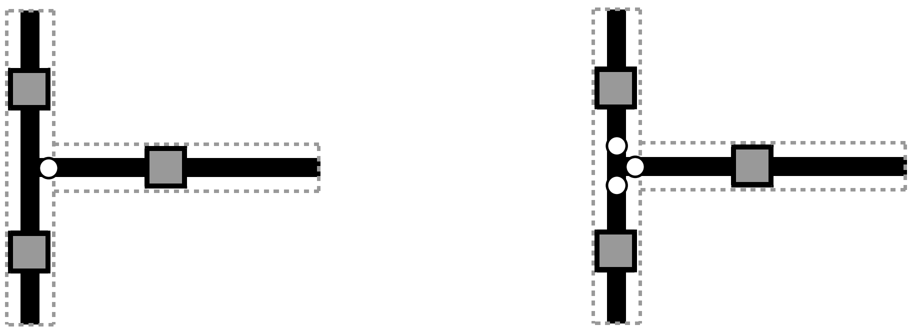
In the modelling of open section shear wall assemblies, there may be more than one alternative to assigning end releases at the rigid beam connections. For example, for a T section shear wall assembly, the two possible ways of assigning end releases are given in Figure 6.1. The analysis results show that using the first alternative gives better results when compared with results obtained with the second alternative. Similar

Table 6.4 Comparison of the Two BS2-60 Building Models for the Three Methods of Analysis

Analysis Type	Running Time (s)
Equivalent Lateral Load Analysis	
SAP Shell Elements	835
Proposed Model	75
Response Spectrum Analysis	
SAP Shell Elements	1236
Proposed Model	142
Time History Analysis	
SAP Shell Elements	1383
Proposed Model	153

to a T section, assigning the end releases in the same way in a W section gives more accurate results.

Another important point concerning the proposed shear wall models is the process of lumping the mass of the wide columns at the joints. Especially for the wide columns



(a) (b)

Figure 6.1 The Two Alternatives for Modelling a T-Section Shear Wall Assembly

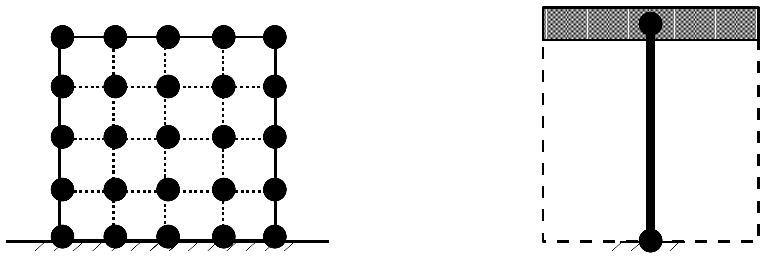


Figure 6.2 Two Shear Wall Models and Mass Lumping in a Planar Shear Wall

at the first storey of the building structures, half mass of the column is lumped at the joint that is located at the base. This situation may cause errors in dynamic analysis of low storey building structurese in which the shear walls are the dominant vertical structural members.

An attempt is made to investigate the difference in the natural vibration periods between the two models. A 3.0 meters high, 3.0 meters wide and 0.25 meters thick planar shear wall is modelled using shell elements and wide column. In Figure 6.2, the two models and corresponding mass distributions are given. The first natural vibration periods are computed as 0.0089 s for the shell model and 0.0111 s for the wide column models. The corresponding relative percentage difference between the two models was 19.82 %, which is a relatively large value.

In order to have more accurate results, the wide column member should be divided into a number of frame elements to obtain a better mass distribution. For example, if the considered wide column is divided into three equal frame elements, the first natural vibration period obtained is 0.0089 s, which is equal to the value obtained using shell elements.

An important factor in the proposed models is the assignment of rigid beam properties. To achieve this task, a series of analyses are performed on the sample building structures and the stiffness properties of the rigid beams are investigated. These investigations led to the conclusion that the cross-sectional area, torsional constant and moments of inertia in both directions of the rigid beams should be chosen such that they are at least 20,000 times greater than those computed for the wide column with the largest dimensions.

REFERENCES

[1] Stafford Smith, B., and Coull A., *Tall Building Structures: Analysis and Design*, John Wiley and Sons, 1991.

[2] Taranath, B. S., *Structural Analysis and Design of Tall Buildings*, McGraw-Hill Company, 1988.

[3] Öztorun, N. K., “Computer Analysis of Multi-Storey Building Structures”, Ph.D. Thesis, Middle East Technical University, 1994.

[4] Wilson, E. L. and Habibullah, A., “SAP2000, Structural Analysis Program”, Computers and Structures Inc., Berkeley, California, USA, 1995.

[5] Wilson, E. L., Dovey, H. H. and Habibullah, A., “ETABS, Three Dimensional Analysis of Building Systems”, Computers and Structures Inc., Berkeley, California, USA.

[6] Boz, C. T., “Modeling of Building Structures for Lateral Load Analysis”, M.S. Thesis, Middle East Technical University, 1987.

[7] Çıtıpıtıoğlu, E. and Boz, C. T., “Çok Katlı Betonarme Binaların Yatay Yükler İçin Modellenmesi”, 3. Yapı Mekaniği Semineri, Orta Doğu Teknik Üniversitesi ve Anadolu Üniversitesi, Eskişehir, 1987.

[8] MacLeod, I. A., *Analytical Modeling of Structural Systems*, Ellis Horwood Limited, 1990.

[9] “Response of Buildings to Lateral Forces”, ACI Committee Report, SP-97, American Concrete Institute, Detroit, 1985: 21-46.

[10] Rutenberg, A. and Eisenberger, M., "Torsion of Tube Structures: Planar Formulation", Computers and Structures, Vol.17, No.2, 1983: 257-260.

[11] Rutenberg, A. and Eisenberger, M., "Simple Planar Modeling of Asymmetric Shear Buildings for Lateral Forces", Computers and Structures, Vol.24, No.6, 1986: 885-891.

[12] Smith, B. S. and Cruvellier, M., "Planar Modeling Techniques for Asymmetric Building Structures", Proc. Instn. Civ. Engrs., Part 2, Vol.89, 1990: 1-14.

[13] Smith, B. S. and Cruvellier, M., "A Planar Model for the Static and Dynamic Analysis of Asymmetric Building Structures", Computers and Structures, Vol.48, No.5, 1993: 951-956.

[14] Wilson, E. L. and Dovey, H. H. "Three Dimensional Analysis of Building Systems - TABS", Report No. EERC 72 – 8, College of Engineering, University of California, Berkeley, California, December 1972.

[15] Wilson, E. L., Dovey, H. H. and Habibullah, A. "Three Dimensional Analysis of Building Systems, TAB80, A Report to U.S. Army Engineering Waterways Experiment Station", Computers / Structures International, June 1980.

[16] Wilson, E. L., Dovey, H. H. and Habibullah, A., "TABS90, Three Dimensional Analysis of Building Systems", Computers and Structures Inc., Berkeley, California, USA, 1994.

[17] Wilson, E. L., "SAP – A General Structural Analysis Program", University of California Structural Engineering Laboratory Report No. UC – SESM 70 –20, 1970.

[18] Wilson, E. L., "SOLID SAP – A Static Analysis Program for Three Dimensional Solid Structures", University of California Structural Engineering Laboratory Report No. UC – SESM 71 – 19, 1972.

[19] Bathe, K. J., Wilson, E. L. and Peterson, F. E., "A Structural Analysis Program for Static and Dynamic Response of Linear Systems – SAP IV", College of Engineering, University of California, Berkeley, California, June 1975.

[20] Wilson, E. L. and Habibullah, A., "SAP90, Structural Analysis Program", Computers and Structures Inc., Berkeley, California, USA, 1992.

[21] Ghali, A.and Neville, A. M., *Structural Analysis – A Unified Classical and Matrix Approach*, New York, Chapman and Hall Ltd, 1978.

[22] Chopra, A. K., *Dynamics of Structures*, Second Edition, Prentice Hall, 2000.

[23] Swaddiwudhipong, S., Balendra T., Quek, S. T. and Lee, S. L., "Computer Program for the Analysis of Asymmetric Frame – Shear Wall Structures", Computers and Structures, Vol.22, No.3, 1986: 343-362.

[24] Thambiratnam, D. P. and Irvine, H. M., "Microcomputer Analysis of Torsionally Coupled Multisorey Buildings – I. Shear Beam Model", Computers and Structures, Vol.32, No.5, 1989: 1175-1182.

[25] Zeng, L. F. and Wiberg N. E., "A Generalized Coordinate Method for the Analysis of 3D Tall Buildings ", Computers and Structures, Vol.33, No.6, 1989: 1365-1377.

[26] Li, C., "TAP – 86 A Three – Dimensional Frame Analysis Program for Tall Buildings", Computers and Structures, Vol.32, No.5, 1989: 953-962.

[27] Syngellakis, S. and Younes, I., "The Transfer Matrix Method Applied to Frame - Shear Wall Systems", Computers and Structures, Vol.41, No.2, 1991: 197-206.

[28] Younes, I. and Syngellakis, S., "Transfer Matrix Analysis of Asymmetric Frame – Shear Wall Systems", Computers and Structures, Vol.43, No.6, 1992: 1057-1065.

[29] Hoenderkamp, J. C. D., "Elastic Analysis of Asymmetric Tall Building Structures", The Structural Design of Tall Buildings, Vol.10, 2001: 245–261.

[30] Hoenderkamp, J. C. D., "Simplified Analysis of Asymmetric High – Rise Structures with Cores", The Structural Design of Tall Buildings, Vol.11, 2002: 93-107.

[31] "STAAD - Structural Analysis and Design Software", Research Engineer International, California, USA.

[32] "ide CAD Structural 5 - Structural Analysis and Design Software", ideCAD Software GmbH, Germany.

[33] *Specifications for the Buildings to be Constructed in Disaster Areas,* Ministry of Public Works and Settlements, 1997, Ankara, Turkey.

[34] *Uniform Building Code*, International Conference of Building Officials, 5360 Workman Mill Rd., Whittier California, 90 610, 1988.

[35] Associate Committee on National Building Code, *National Building Code of Canada*, 1995, National Research Council, Ottawa, 1995.

[36] International Association of Earthquake Engineering, *EUROCODE 8 in Earthquake Resistant Regulations*, A World List, 1996: 42-1 to 42-74.

[37] Celep, Z. and Kumbasar, N., *Deprem Mühendisliğine Giriş*, İstanbul, 2000.

[38] Kıral, E., Özdemir, E. K., Yerli, H. R. and Temel, B., *Yeni Deprem Yönetmeliğinin Analiz Bakımından Uygulamaları*, İnşaat Mühendisleri Odası Adana Şubesi Yayınları, Adana, 2000.

[39] Özmen, G., *1997 Türkiye Deprem Yönetmeliğine Göre Tasarım Uygulamaları*, TDV Yayınları, Şubat 1999.

[40] Clough, R. W. and Penzien, J., *Dynamics of Structures*, Second Edition, McGraw Hill, 1993.

[41] Rosenblueth, E., "A Basis for Aseismic Design", Ph.D. Thesis, University of Illinois, Urbana, III., 1951.

[42] Wilson, E. L., Der Kiureghian, A. and Bayo, E. R., "A Replacement for the SRSS Method in Seismic Analysis", Earthquake Engineering and Structural Dynamics, Vol. 9, 1981: 187–192.

[43] Wilson, E. L., *Three Dimensional Static and Dynamic Analysis of Structures*, Third Edition, Computers and Structures Inc., 2002.

[44] Norio, I., Yang, K. and Shibata, A., "Dynamic Non-linear Analysis of Reinforced Concrete Shear Wall by Finite Element Method with Explicit Analytical Procedure", Earthquake Engineering and Structural Dynamics, Vol.26, 1997: 967-986.

[45] Kongoli, X., Minami, T. and Skai, Y., “Effects of Structural Walls on the Elastic-Plastic Earthquake Responses of Frame-Wall Buildings”, Earthquake Engineering and Structural Dynamics, Vol.28, 1999: 479-500.

[46] Hidalogo, P.A., Jordan, R.M. and Martinez, M.P., “An Analytical Model to Predict the Inelastic seismic Behavior of Shear-Wall, Reinforced Concrete Structures”, Engineering Structures, Vol.24, 2002: 85-98.

[47] Clough, R. W., King, I. P. and Wilson, E. L., “Structural Analysis of Multistorey Buildings”, Journal of Structural Division, ASCE, 90 (19), 1964.

[48] Candy, C. F., "Analysis of Shear Wall-Frames by Computer", New Zealand Engrg., Wellington, New Zealand, 19 (9), 1964: 342-347.

[49] MacLeod, I. A., "Lateral Stiffness Analysis of Shear Walls with Openings", Proc., Symp. of Tall Buildings, London, England, Pergamon Press, 1967: 223-252.

[50] MacLeod, I. A., “General Frame Element for Shear Wall Analysis”, Proc. Instn. Civ. Engrs., Part 2, Vol.61, 1976: 785-790.

[51] MacLeod, I. A., “Structural Analysis of Wall Systems”, The Structural Engineer, November 1977: 487-495.

[52] MacLeod, I. A. and Hosny, H. M., “Frame Analysis of Shear Wall Cores”, Journal of Structural Division, ASCE, 103 (10), 1977.

[53] Smith, B. S. and Girgis, A, “Simple Analogous Frames for Shear Wall Analysis”, Journal of Structural Division, ASCE, 110 (11), 1984.

[54] Koumousis, V. K. and Peppas, G. A., “Stiffness Matrices for Simple Analogous Frames for Shear Wall Analysis”, Computers and Structures, Vol.43, No.4, 1992: 613-633.

[55] Kwan, A. K. H., ”Improved Wide-Column-Frame Analogy for Shear/Core Wall Analysis”, Journal of Structural Division, ASCE, 119 (2), 1993.

[56] Kwan, A. K. H., “Equivalence of Finite Elements and Analogous Frame Modules for Shear / Core Wall Analysis”, Computers and Structures, Vol.57, No.2, 1995: 193-203.

[57] Girijavallabhan, C. V., ”Analysis of Shear Walls with Openings”, Journal of Structural Division, ASCE, 95 (10), 1969.

[58] MacLeod, I. A., ”New Rectangular Finite Element for Shear Wall Analysis”, Journal of Structural Division, ASCE, 95 (3), 1969.

[59] Al–Mahaidi, R. S. and Nilson, A. H., ”Coupled Shear Wall Analysis by Lagrange Multipliers”, Journal of Structural Division, ASCE, 101 (11), 1975.

[60] Cook, R. D., ”Avoidance of Parasitic Shear in Plane Element”, Journal of Structural Division, ASCE, 101 (6), 1975.

[61] Kwan, A. K. H., ”Analysis of Buildings Using Strain-Based Element with Rotational DOF’s”, Journal of Structural Division, ASCE, 118 (5), 1992.

[62] Kwan, A. K. H., ”Mixed Finite Element Method for Analysis of Coupled Shear / Core Walls”, Journal of Structural Division, ASCE, 119 (5), 1993.

[63] Chan, H. C. and Cheung, Y. K., “Analysis of Shear Walls Using Higher Order Elements”, Building and Environment, 14 (3), 1973.

[64] Cheung, Y. K. and Swaddiwudhipong, S., “Analysis of Frame Shear Wall Structures Using Finite Strip Elements”, Proc. Instn. Civ. Engrs., Part 2, Vol.65, 1975: 517-535.

[65] MacLeod, I. A., “Analysis of Shear Wall Buildings by the Frame Method”, Proc. Instn. Civ. Engrs., Part 2, Vol.55, 1973: 593-603.

[66] Lew, I. P. and Narov, F., “Three-Dimensional Equivalent Frame Analysis of Shearwalls”, Concrete International, October 1983.

[67] Ghuneim, N., “Modeling of Tunnel Form Buildings by Equivalent Frame Method”, M.S. Thesis, Middle East Technical University, 1999.

[68] Dikmen, Y., “Seismic Analysis and Design of Tunnel Form Buildings”, M.S. Thesis, Middle East Technical University, 2000.

[69] Smith, B. S. and Girgis, A, “ Deficiencies in the Wide Column Analogy for Shearwall Core Analysis”, Concrete International, April 1986.

[70] Smith, B. S. and Jesien, W., “Two-Column Model for Static Analysis of Mono-Symmetric Thin Wall Beams”, Structural Engineering Report No: 88-3, Department of Civil Engineering and Applied Mechanics, McGill University, May 1988.

[71] Smith, B. S. and Taranath, B. S., “Analysis of Tall Core-Supported Structures Subject to Torsion”, Proc. Instn. Civ., Engrs., 53, September 1972: 173-187.

[72] Felton, L. P. and Nelson, R. B., *Matrix Structural Analysis*, John Wiley and Sons, 1997.

[73] Mukherejee, P. R. and Coull, A., “Free Vibrations of Open–Section Shear Walls”, Earthquake Engineering and Structural Dynamics, Vol. 5, 1977: 81-101.

[74] Özmen, G., Orakdöğen, E. and Darılmaz, K., *Örneklerle SAP2000*, Birsen Yayınevi, 2002.

[75] Kwan, A.K. H., “Reformulation of The Frame Method”, Proc. Instn. Civ., Engrs., 94, Feb. 1992: 103-116.

[76] Nadjai, A. and Johnson, D., “Torsion in Tall Buildings by a Discrete Force Method”, The Structural Design of Tall Buildings, Vol. 7, 1998: 217–231.

[77] Tso, W. K., and Biswas, J. K., "Analysis of Core Wall Structures Subjected to Applied Torque", Building Sciences, 8, 1973: 251-257.

[78] Ho, D. and Liu, C. H., "Analysis of Shear Wall and Shear Core Assembly Subjected to Lateral and Torsional Loading", Proc. Instn. Civ. Engrs, Part 2, 79, 1985: 119-133.

[79] Özmen, G., *Çok Katlı Yapılarda Burulma Düzensizliği*, TDV Yayınları, Ocak 2001.

APPENDIX A

For the proposed closed section model, the modified torsional constants of the wide column are calculated by the following equation:

$$\bar{J}_i = \frac{J_c}{\sum\limits_{k=1}^{n} J_k} \cdot J_i B_i \tag{A.1}$$

In the above equation, $\bar{J}_i$ is the modified torsional constant of the i-th wide column, J_c is the torsional constant of the closed shear wall and J_i is the torsional constant of the i-th wide column. B_i is a constant which depends on the dimensions of the closed section and n is the total number of wide columns in the model.

The torsional stiffness of a rectangular closed section can be obtained by

$$J_c = bt^3 \left[\frac{1}{3} - 0.21 \frac{t}{b} \left(1 - \frac{t^4}{12b^4} \right) \right] \tag{A.2}$$

In this equation, b is the larger and t is the smaller dimensions of the rectangular section. The value of B_i in Eq.A.1 is 1.0 for the wide columns in a square shear wall model. For a rectangular cross-section, the following equation should be used for determining B_i.

$$B_i = \frac{4 J_i a_i^2}{\sum\limits_{k=1}^{n} J_k a_k^2} \tag{A.3}$$

In the above equation, a_i is the distance between the centroid of the i-th wide column and the centroid of the closed shear wall in plan.

VITA

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