QUANTIFYING SEISMIC DESIGN CRITERIA FOR CONCRETE BUILDINGS

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ABSTRACT

QUANTIFYING SEISMIC DESIGN CRITERIA FOR CONCRETE BUILDINGS

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The amount of total and relative sway of a framed or a composite (frame-shear wall) building is of utmost importance in assessing the seismic resistance of the building. Therefore, the design engineer must calculate the sway profile of the building several times during the design process.

However, it is not a simple task to calculate the sway of a three-dimensional structure. Of course, computer programs can do the job, but developing the three-dimensional model becomes necessary, which is obviously tedious and time consuming.

An easy to apply analytical method is developed, which enables the determination of sway profiles of framed and composite buildings subject to seismic loading. Various framed and composite three-dimensional buildings subject to lateral seismic loads are solved by SAP2000 and the proposed analytical method. The sway profiles are compared and found to be in very good agreement. In most cases, the amount of error involved is less than 5 %.

The analytical method is applied to determine sway magnitudes at any desired elevation of the building, the relative sway between two consecutive floors, the slope at any desired point along the height and the curvature distribution of the building from foundation to roof level.

After sway and sway-related properties are known, the requirements of the Turkish Earthquake Code can be evaluated and / or checked.

By using the analytical method, the amount of shear walls necessary to satisfy Turkish Earthquake Code requirements are determined. Thus, a vital design question has been answered, which up till present time, could only be met by rough empirical guidelines.

A mathematical derivation is presented to satisfy the strength requirement of a three-dimensional composite building subject to seismic loading. Thus, the occurrence of shear failure before moment failure in the building is securely avoided.

A design procedure is developed to satisfy the stiffness requirement of composite buildings subject to lateral seismic loading. Some useful tools, such as executable user-friendly programs written by using "Borland Delphi", have been developed to make the analysis and design easy for the engineer.

A method is also developed to satisfy the ductility requirement of composite buildings subject to lateral seismic loading based on a plastic analysis. The commonly accepted sway ductility of μ_Δ=5 has been used and successful seismic energy dissipation is thus obtained.

Keywords: Earthquake, reinforced concrete structures, sway, relative story drift, shear wall, strength, stiffness, ductility, seismic energy, framed structures, composite structures, curvature

ÖZ

BETONARME BİNALARIN DEPREM TASARIMI İÇİN SAYISAL KRİTERLERİN BELİRLENMESİ

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Çerçeveli veya kompozit (çerçeveli-perdeli) bir binanın toplam ve göreceli ötelenmesi, o binanın sismik direncinde en önemli göstergedir. O yüzden, tasarımı yapan mühendis, tasarım işlemi boyunca, binanın yatay ötelenme profilini birkaç kere hesaplamak zorundadır.

Ancak, üç boyutlu bir yapının ötelenmesini hesaplamak çok basit bir işlem değildir. Aslında bunu bilgisayar programları yapabilir fakat bunun için üç boyutlu model geliştirilmesi gerekir ki, bu hem zahmetli hem de zaman alıcıdır.

Sadece sismik yüklemeye maruz çerçeveli ve kompozit binalar için ötelenme profillerinin belirlenmesini sağlayan, uygulanması kolay, analitik bir metot geliştirilmiştir. Yanal sismik yüklere maruz çeşitli çerçeveli ve kompozit üç boyutlu binalar SAP2000 ve önerilen analitik metotla çözülmüştür. Ötelenme profilleri birbiriyle karşılaştırılmış ve bunların çok iyi uyum içinde olduğu bulunmuştur. Bir çok durumda, hata payı % 5'ten az olmuştur.

Analitik metot, istenen herhangi bir yükseklikteki ötelenme miktarını, iki katın birbirine göre göreceli olarak ötelenmesini, yükseklik boyunca herhangi bir

noktadaki eğimi ve temelden çatı seviyesine kadar binanın eğrilik dağılımını belirlemek için kullanılır.

Ötelenme ve ötelenmeye bağlı özellikler bilindikten sonra, Türk Deprem Yönetmeliğinde belirtilen koşullar değerlendirilebilir ve/veya kontrol edilebilir.

Analitik metodu kullanarak, Türk Deprem Yönetmeliğine göre gerekli perde duvar miktarı belirlenebilir. Şimdiye kadar sadece yaklaşık ampirik yöntemlerle çözümlenen hayati bir tasarım problemi de böylece çözülebilir.

Üç boyutlu kompozit bir binanın sahip olması gereken dayanımı karşılamak için bir matematiksel ifade sunulmuştur. Böylece, binadaki kesme kırılması ve moment kırılması güvenli bir şekilde önlenmiştir.

Yanal sismik yüklemeye tabi tutulan kompozit binalar için gerekli rijitliği sağlamak için bir tasarım metodu geliştirilmiştir. Tasarımcının analiz ve tasarım işlemlerini kolaylaştırmak için "Borland Delphi" ile yazılan kullanımı kolay programlar hazırlanmıştır.

Yanal sismik yüklere maruz kompozit binaların sahip olması gereken sünekliği sağlamak için plastik analize dayanan bir metot da geliştirilmiştir. Genel olarak kabul gören $\mu_\Delta = 5$ ötelenme sünekliği kullanılmış ve başarılı bir enerji sönümü elde edilmiştir.

Anahtar Kelimeler: Deprem, betonarme yapılar, ötelenme, göreceli kat ötelenmesi, perde duvar, mukavemet, rijitlik, süneklik, sismik enerji, çerçeveli yapılar, kompozit yapılar, eğrilik

Dedicated to my family

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LIST OF SYMBOLS

A_{ch}	Gross section area of shear wall
A(T)	Spectral acceleration coefficient
A_0	Effective ground acceleration coefficient
A_p	Plan area of building
b_w	Width of shear wall
d_i	Lateral displacement at the ends of any column or wall at i'th storey
E	Modulus of elasticity of concrete
f_c, f_{ck}	Characteristic compressive cylinder strength of concrete
f_{cd}	Design compressive strength of concrete
f_{ctd}	Design tensile strength of concrete
f_{yk}	Characteristic yield strength of longitudinal reinforcement
f_{yd}	Design yield strength of longitudinal reinforcement
F	Concentrated lateral load at the top of structure
GA	Equivalent shear stiffness of building
h_i	Height of i'th storey
H	Height of building
H_w	Total shear wall height
I	Building importance factor
K	Total stiffness of all shear walls along the axis considered
K_0	Flexural rigidity of structure in the horizontal plane
l_i	i^{th} story column length, measured from axis to axis
l_p	Height of plastic hinge region of a shear wall
l_w	Length of shear wall
M_{ot}	Total overturning moment
M_p	Plastic moment

M_d	Design moment
N	Axial force calculated under simultaneous action of vertical & seismic loads
ΔN	Axial force due to earthquake effect
N_0	Axial load capacity of a cross section of column, beam or wall
n	Number of storey
p	Top intensity of uniformly distributed lateral triangular load
R	Structural behavior factor
S(T)	Spectrum coefficient
T	Building natural vibration period
TS	Turkish Standards
UBC	Uniform Building Code
V_t	Total equivalent seismic load (base shear) acting on a building
V_r	Shear strength of a cross section of column, beam or wall
V_p	Shear force due to plastic moment
W	Total weight of building
Δ_i	Storey drift of i'th storey of building
θ_i	Second order effect indicator defined at i'th storey of building
Φ	Stability index
ρ_{sh}	Ratio of horizontal web reinforcement of wall to the gross area of wall web
μ_Φ	Curvature ductility
μ_Δ	Displacement ductility
Φ_u	Ultimate curvature
Φ_y	Yield curvature
ε_y	Yield strain of reinforcing steel
Δ_u	Displacement at the top of the structure at ultimate stage
Δ_y	Displacement at the top of the structure at initiation of yielding at the base of shear wall
δ	Lateral sway between two consecutive stories

CHAPTER 1

INTRODUCTION, LITERATURE SURVEY & OBJECTIVE AND SCOPE OF THE STUDY

1.1 INTRODUCTION

During the last five decades, shear wall-structures have been widely accepted as a rational and economical feature for buildings in highly-populated countries. In the design of shear wall-structures, resistance against both vertical and lateral loads has been assigned to shear walls situated in proper positions. In these structures, shear walls are connected with deep beams and flat slabs to satisfy the requirements of adequate interaction.

The great boom in the construction of shear wall-structures was caused by the high migration of population into cities where there is a necessity to meet the social requirements for quality housing. Shear wall-structures are now widely accepted as a rational and economical part of multi storey constructions. For buildings taller than 10 or 15 stories, the use of shear walls in one form or another becomes necessary for economic reasons. Shear wall-buildings usually employ typical designs so that the construction material can be economically used.

Shear walls situated in proper positions in a building form an efficient and economic resisting system to lateral forces resulting from mainly wind or seismic loading. If the walls are properly designed, they absorb the energy of the earthquake so that little structural damage occurs.

Researches on the subject of buildings that are built by using shear walls are still limited. Therefore, computations done by 1997 Turkish Earthquake Code [1] and the earthquake safety of shear wall-buildings become questionable.

Earthquake engineering has accomplished significant progress during the last half century. At present, there is good understanding of earthquake ground motions and earthquake response of structures. As a result, building codes have undergone a big development. Numerous design methods have been formulated and adopted by building codes to guide the seismic design of buildings.

The primary function of current seismic regulations for building structures is to provide minimum standards for use in design and to maintain public safety during the event of extreme earthquakes likely to occur at the site of the building. These regulations are intended primarily to safeguard against major failures and loss of life, not to limit damage, maintain functions, or provide for easy repair.

Current state of practice for earthquake-resistant design of regular structures considers the effect of an earthquake by means of equivalent static lateral loads acting at floor levels. The proportioning of structural members is based on their strength demand obtained from linear analysis for the combined actions of the equivalent static lateral loads with all other loading conditions.

Lateral displacement (drift), in a reinforced concrete structure, is typically computed under the equivalent static lateral loads and for stiffness based on uncracked member properties. The lateral loads correspond putatively to the demand that the design earthquake imposes on the structure assuming that it responds nonlinearly. The design forces are likely to be less than what the structure will sustain during the design seismic event if nonlinear response occurs, so the use of uncracked stiffness is not realistic. Thus, design drifts are not comparable with actual drifts.

In general, building codes have used strength as the main parameter and have placed the computation of forces as the centerpiece of earthquake-resistant design, relegating drift calculations to the background in the design process. There is no realistic quantification of the magnitude of nonlinear displacement that the structure may experience during the design earthquake, or of the structural or nonstructural damage likely to occur.

The importance of drift control is revealed when it is accepted that the inter-story drift ratio (difference in drift response between two consecutive levels divided by the story height) constitutes an acceptable measure of distortion and damage.

The performance criterion most often referred to in earthquake-resistant design is based on the ductility ratio, defined as the ratio of maximum deformation to that corresponding to yield. But ductility ratio is very difficult to relate to observation. Estimating the displacement ductility of a reinforced concrete member is very difficult because of the uncertainties associated with estimating the yield displacement.

Observations have confirmed that there is a good correlation between damage and the drift ratio: a low drift ratio means tolerable damage. On the other hand, the correlation between damage and structural ductility ratio is not always informative. In a flexible frame, a low ductility ratio may be associated with high damage to the nonstructural elements.

A drift-control procedure is based on imposing a limit to the maximum drift ratio. This limit demands a deformation capacity for the structural elements. If this deformation capacity can be attained by following a set of provisions that prescribe minimum details, then drift estimates alone would be sufficient to anticipate performance of buildings during earthquakes, without the need of information on ductility or on acceleration levels.

The advantages of structural walls in the resistance of lateral forces, particularly in terms of deflection control, are well established. The term “shear wall”, although a misnomer, is still widely used. Apart from shear, walls must also resist overturning moments and gravity loads, just like frames, and shear resistance is not necessarily a critical aspect of design. In seismic design special precautions must be taken to suppress shear failures under any circumstances.

The elastic response of reinforced concrete wall systems under earthquake and wind forces has been studied, particularly in United Kingdom [75, 76, 77]. As expected, in seismic regions more emphasis was placed on the elasto-plastic response of wall systems and on aspects of hysteretic response, ductility and energy dissipation [24, 78]. Subsequently, specific seismic design requirements have been formulated in New Zealand and some of these recommendations have also been adopted in other codes [79, 80, 81]. Therefore, only fundamental issues are briefly referred to here. Emphasis is placed on those features of the inelastic seismic response of wall systems that have emerged from more recent research [24].

Critical aspects of the design depend on the number and the length of walls available in a given building to resist earthquake actions. In apartment buildings numerous walls may be utilized and hence demands on individual walls may be small. Often code-specified minimum amounts of reinforcement will suffice strength requirements with modest ductility demands. Even elastic response may be assured. In office buildings, however, the entire lateral force resistance generally may be assigned to a few walls and these then will require special attention.

In studying various features of inelastic response of structural walls and subsequently in developing a rational procedure for their design, a number of fundamental assumptions are made [24]:

1. In all cases studied, structural walls are assumed to possess adequate foundations that can transmit actions from the superstructure into the ground without allowing the walls to rock. Elastic and inelastic deformations that may occur in the foundation structure or the supporting ground are not considered.

2. The foundation of one of several interacting structural walls does not affect its own stiffness relative to the other walls.

3. Inertia forces at each floor are introduced to structural walls by diaphragm action of the floor system and by adequate connections to the diaphragm. In terms of in-plane forces, floor systems (diaphragms) are assumed to remain elastic at all times.

4. Walls considered are generally deemed to offer resistance independently with respect to the two major axes of the section only. It is to be recognized, however, that under skew earthquake attack, wall sections with flanges are subjected to biaxial bending. Suitable analysis programs to evaluate the strength of articulated wall sections subjected to biaxial bending and axial force, are available. They should be employed whenever parts of articulated wall sections under biaxial seismic attack

may be subjected to significantly larger compression strains than during independent orthogonal actions.

It is established that structures can be designed and constructed so as to satisfy various seismic performance criteria, most importantly that of preventing collapse during an exceptionally large earthquake. For most engineers seismic design is synonymous with the complex analysis of elastic or inelastic dynamic response to random ground excitations. This study, reflecting the views of structural designer, attempts to contrast analysis with design strategies that are suited to overcome difficulties that stem from inevitable uncertainties in the prediction of ground motions. Using reinforced concrete buildings as an example, it postulates the precept that the development of energy dissipating mechanisms in structural systems must not be left to the randomness of ground motions. Rather a deterministic design philosophy is advocated whereby the designer can, within certain limits, choose the seismic response of a structure that is safe, rational, predictable and achievable in construction. The designer may thus enhance desirable and suppress undesirable features of structural behavior. In this the vital role of the quality of the design and detailing of the critical regions of structural systems is emphasized because this alone can assure the very desirable characteristic of seismic response; tolerance with respect to the inevitable crudeness of predicting earthquake imposed displacements [74].

1.2 LITERATURE SURVEY

If one tries to get information about shear-wall structures and the analysis principles or design considerations of structures built by using shear walls, very limited knowledge will be obtained since this type of construction is not used widely in the world except in some countries. Another reason for the lack of available information about this subject is the fact that the countries, which use the shear-wall structures, are not in the critical or dangerous earthquake zones.

In our country, this type of structures is the primary construction technique for mass housing or high-rise building construction. But, project offices use their experiences and their previous knowledge to design structures having shear walls. Then they try to adapt and apply Turkish Code’s requirements to their structural designs in a most suitable way since Turkish Codes do not provide enough information and applicable design rules about shear-wall systems.

Z.Hasgür and N.Gündüz [18] explain the behavior of coupled shear walls and coupling beams in the tall shear wall-systems in detail. The results of the tests done by Paulay [25] and Subedi [28, 29] are also presented.

Moment curvature relationship is given with examples by Park, R. and Paulay, T [40]. The photographs of damaged structures because of earthquake actions present the behavior of individual shear wall and interaction between the walls. The diaphragm effect of slabs between the shear walls is also described.

The books by David J.Dowrick [11], P.Fajfar and H.Krawinkler [15], P.M.Ferguson, J.E.Breen and J.O.Jirsa [16], T.Paulay and M.J.N.Priestley [24] are viewed in order to obtain information about the behavior of shear walls, shear wall-buildings, coupled shear walls, coupling beams and also analysis & design principles for earthquake resistance.

Ersoy [13] is explaining the design principles for seismic resistant reinforced concrete structures in his paper. Basic principles, such as strength, ductility and stiffness are summarized for seismic design. In the second part of the paper, the author summarizes his opinion about the damage observed in the past earthquakes.

Thomas Paulay [23] explains the strategy in the positioning of walls, the establishment of a hierarchy in the development of strength to ensure that brittle failure will not occur and preferred mode of energy dissipation in a predictable region. His paper is based on the design of ductile reinforced structural walls of earthquake resistance. He is also mentioning about capacity design principles that can be an applicable design method for ductile structures, which may be subjected to large earthquakes. He shows that the capacity design procedure ensures that the chosen means of energy dissipation can be maintained. This approach is given in a rational, deterministic and relatively simple manner.

Tassios, T.P., Moretti, M. and Bezas, A. [33] have an experimental research on the subject of the behavior and ductility of reinforced concrete coupling beams of shear walls. In this research the result of an experimental program on coupling beams subjected to cyclic loading is presented. Ten specimens with five different reinforcement layouts and two different shear ratios had been tested. An attempt is made to classify the performance of the specimens according to the ductility they exhibit.

Subedi, N.K. [28, 29] has two papers published in the Journal of Structural Engineering. The papers are based on the subject of reinforced concrete coupled shear wall structure. First, some analyses are carried on coupling beams. Here, the behavior of coupling beams in the shear mode of failure, known as diagonal splitting, is represented by a mathematical model, and a method for the ultimate strength analysis is presented. The proposed method of analysis for RC coupling beams is used to verify the results of nine beams tested by Thomas Paulay. Second, ultimate strength calculations of reinforced concrete coupled shear walls are presented. Three modes of failure of reinforced concrete coupled shear wall structures, observed in micro-concrete models of 15 story-structures, are described. A method is proposed to predict the mode of failure and the ultimate strength of coupled shear wall structures. The method is based on the evaluation of the strengths of coupling beams and the walls at failure. Two lateral load cases have been considered; a point load at the top and a four-point equivalent triangular distribution. Finally, the proposed analysis and the test results are compared.

In order to get detailed information about coupling beams, the deep beam subject should also be investigated. In this context, the study of Mau and Hsu [21] about the shear strengths of deep beams has been examined. The authors of the paper drive an explicit formula for the shear strength of deep beams using three equilibrium equations from the truss model theory.

Tegos, L.A. and Penelis G.Gr. [34] have an experimental investigation to study the behavior of short columns and coupling beams reinforced with inclined bars under seismic conditions. A simple technique to prevent these elements from falling in premature splitting shear is tested for the first time. According to this technique, the main reinforcements are arranged at an inclination such as to form a rhombic truss. Test results show that inclined arrangement of main reinforcements is one of the most effective ways to improve the seismic resistance of reinforced concrete low slenderness structural elements.

The study of Siao, W.B. [27] includes prediction of the shear capacity of the reinforced concrete wall specimens using formulas established for top-loaded deep beams and corbels and also the comparison of results against experimental values.

Based on his conscientious observations on a multitude of reinforced concrete buildings in the past earthquakes, Fintel concludes one of his articles [52] by saying that "…Safety against collapse has been the major preoccupation of earthquake engineering. In addition to safety, damage control should be our major goal. Judging from the behavior of multistory concrete buildings in earthquakes, it seems that to achieve damage control the ductile shear wall may be the most logical solution. Actually, from observations in earthquakes, it seems that we can no longer afford to build our multistory buildings without shear walls."

Bilyap, S. [10, 62] explains in detail the analytical methods to analyze high-rise reinforced concrete buildings of mixed type (shear wall-frame) subjected to lateral forces.

The book [70] by Murashev, V., Sigalov, E., and Baikov, V. N. provides the fundamental differential equations for flexural beams, shear beams and response of mixed type multistory reinforced concrete structures (shear wall-frame) to lateral forces.

1.2.1 Shear Wall Structures [45]

Shear wall structures comprise a large proportion of commercially constructed buildings. These structures serve as residential and office space occupancy, and range up to thirty stories and beyond. Shear wall buildings may be classified into two broad categories:

1. Shear / Flexural Wall Lateral Resisting Buildings
2. Bearing Wall Buildings

The major difference between the two categories is their lateral resisting design. The first category, a shear/flexural wall building, relies on a primary vertical load carrying system, such as columns & beams while the shear walls function primarily for lateral resistance. The specific intent of the shear and flexural walls is to provide lateral stiffness. Vertical loads are carried by the beam-column system.
The shear walls brace the concrete moment frame against lateral deflections while the frame handles the vertical loads. This structural system is commonly utilized in multistory office structures.

The second category comprises a shear wall system that functions both as a vertical load carrying system and also a lateral resisting system. Vertical loads are transferred to walls and eventually to the foundation. Therefore, the axial force/stress increases on the wall toward the base of the building. In addition to this axial force, the wall is also expected to resist large dynamic loads (due to earthquake or wind) that strike "in-plane" and "out-of-plane" to the wall.

From experience, shear wall buildings have demonstrated an excellent performance during earthquakes. They are stiff structures with high ductility. Generally, shear wall buildings survive earthquakes with minimal damage. This is due to a particular characteristic of shear wall structures, which is their stiff in-plane resistance. The in-plane shear resistance provides bracing against dynamic loads and shortens the period of the structure. In-plane load resistance is the principal strength of shear walls. Providing lateral bracing (against out-of-plane buckling) allows shear walls to accept very high in-plane loads. Shear walls require bracing against out-of-plane loads by either additional shear walls or ductile moment frames.

If the out-of-plane bracing is not provided, the shear wall will fail prematurely. From a practical standpoint, shear walls are usually braced in their perpendicular direction by additional walls to alleviate potential failure. With exception of retaining walls, in a building with a shear wall design, the out-of-plane forces are counteracted by means of either another wall or dual bracing system.

"Shear wall" is the industry-accepted term. However, not all shear walls behave in a shear capacity. Tall slender walls are required to resist flexural stresses at the base. Flexural walls are referred to as "Structural Walls" by some researchers and practitioners, as opposed to "Shear Walls" that are shorter and thicker. The difference is the in-plane capacity being linked to a flexural or shear deformation failure. For simplicity, in this thesis the term "shear walls" will be used throughout.

Since a structural wall absorbs significant bending stresses, its deflected shape may be calculated with flexural bending theory (in the elastic range) and ignoring shear deformation contributions. For a pure shear wall, it is necessary to account for shear deformation contributions. Therefore, the failure modes of these two types of walls are quite different. To analyze linear and nonlinear behavior requires a model that can allow for contribution of shear deformation displacement along with flexural displacement. Both are necessary to properly describe the wall behavior.

In order to develop flexural and shear strength, two significant components of a shear wall are necessary:

1. Web reinforcing: Web steel consists of horizontal and vertical reinforcing at uniform spacing.

2. Boundary reinforcing: Vertical steel with ties located at both ends of the shear wall.

Boundary reinforcing develops large axial tension/compression forces that create an in-plane force-couple system to resist external moments. Boundary steel with horizontal ties (similar to column ties) contributes to confinement of the concrete. Concrete confinement increases the material stress-strain curve to an enhanced capacity (i.e., the concrete is stronger and has greater ductility).

External moments also result in web shear that cause diagonal tension cracks. Web steel is responsible to resist in-plane shear stresses. Diagonal tension stress is a concept familiar from basic concrete courses. The "compression strut theory" identifies concrete as the principle vector to resist compression stresses, while steel provides tension resistance. Nevertheless, shear walls seldom fail due to high compression stress, but rather will crack in tension areas due to insufficient web steel.

In a typical uniform thickness shear wall, confinement at the boundary elements is provided and thus increases flexural capacity. Web steel provides in-plane shear resistance. Cross-sections of this type are commonly used in shear wall buildings of shorter height (i.e., less than five stories) because they provide good shear resistance and ductility, but do not have high flexural capacity under axial loads as in the framed shear wall and T-shape shear wall. Additionally, web buckling is a consideration in slender sections. Framed shear walls are particularly strong in developing moment-capacity because of the high axial forces in the boundary elements. These types of shear walls are used for tall multistory applications where vertical load capacity and lateral resistance are both necessary. In a T-shaped shear wall, the perpendicular (flange) wall increases the web's in-plane moment of inertia. Although the flange is out-of-plane to the web, structural engineers have observed the performance of T-shaped shear walls to demonstrate strong bending resistance. Flanged shear walls do not enhance shear capacity as much as the moment, because the flange does not increase the gross area as it does the moment of inertia. Therefore, T-shaped shear walls have their best application in tall multistory buildings, which require both vertical and lateral load capacity.

1.2.2 Analysis of Shear Wall Structures [47]

Buildings that incorporate concrete shear walls as structural elements to resist both vertical and lateral loads are commonplace. The calculation of stresses and deflections in a simple shear wall requires only rudimentary bending theory. There are several methods of analysis used for numerical analysis. From a designer point of view the most important methods of analysis are;

* The Lamina Method (Continuous Connection Method or Rosman Method)
* The Finite Element Method
* The Equivalent Frame Method (Wide Column Analogy)

The Lamina Method

In the analysis with Lamina Method, the individual coupling beams between the structural walls are replaced with a continuous, uniform, homogeneous medium referred to as a lamina. It assumes that the point of counter-flexure occurs in the mid-span of the coupling beams if the walls deflect equally when subjected to horizontal loads in proportion to their stiffness. The method takes into consideration the contributions made to the shear walls by the bending and shear in connecting the beams. However, it is limited to relatively high walls, with constant floor heights and uniform openings.

On the other hand Finite Element Method and Equivalent Frame Method are the main methods used by design offices due to resource, time and cost restrictions.

Finite Element Method

This method partitions a complex element into smaller components of finite size and number. Theoretically the finite element method can be utilized in any kind of engineering problem regardless of its complexity and heterogeneity. The geometry of these finite elements is simpler than the boundaries of the overall element. Choosing appropriate elements for the specific problem concerned should develop the model of the structure. It is gaining wider use and may be the most appropriate method of analysis for some complex problems.

In the application of finite element method, the coupled shear wall structures can be modeled by using shell elements. Finite elements used to model walls and coupling beams must be compatible with each other. In general, two-dimensional four-node finite elements can be utilized in the modeling of shear walls.

In addition coupling beams can be modeled by using conventional or modified one-dimensional frame elements. However for the compatibility, two-dimensional plane element formulation must include the rotational degrees of freedom at the modal points. In addition to that the rotation at beam/wall joints must be defined as the rotations of the vertical fibers. The mesh of the model should be finer in the wall joints where the stress concentrations and discontinues are present. To avoid parasitic shear problem, elements that are able to curve themselves to take the deformed shape of structure under bending, must be used in the analysis of shear walls [6, 11]. Because of the amount of calculations required, even for simple elements, this method is limited to computer applications. Even so, with large complex elements idealized into small numerous finite elements, computation time can be significant.

Equivalent Frame Method

Also referred as the wide column analogy, it replaces the coupled shear wall components with an idealized frame structure that behaves as identically shear walls. This idealized structure is resolved using matrix techniques.

Theoretically, shear walls are replaced with idealized wide columns that behave as shear wall. Connecting beams and slabs are defined to provide adequate interaction between structural walls. The additional horizontal sections between the frame columns and the connecting beams are stiff-ended rigid arms that rotate but don’t bend. Centerlines of walls coincide with wide columns and those of beams coincide with connecting beams. Centerline of idealized wide columns, connecting beams, and rigid arms form the equivalent frame.

The sectional properties of the columns in the equivalent frame are generally those for the corresponding wall sections, since the structure behaves in a linear fashion. For squat walls (length is greater than two times the height), shear deflection governs whereas for slender walls (height is greater than two times the length), bending deflection governs. Designers must consider shear deformation for walls with small (height / depth) ratio, where reduced moments of inertia may be in order [5, 12]. Shear deflection must also be considered to model the behavior of the beams connecting to shear walls properly, since the connecting beams can undergo relatively large deformations especially for those in the upper portion of the frame [14, 15].

Theoretically, rigid arms should have infinite areas and moments of inertia. Extremely large moments of inertia and very large cross-sectional areas lead to ill-conditioned matrices in many frame programs. However, there is no need to assume end sections that are perfectly rigid if very small inaccuracies in the analysis are acceptable.

Adaptability and flexibility of wide column analogy has made it popular in engineering offices. The equivalent frame method provides a good balance of effectiveness, efficiency and the ease of use. Equivalent frame method is applicable to almost any shear wall configuration. Such limitations as constant floor-to-floor height and constant size of openings are not imposed by this method. The method is capable of handling any type of loading such as uniform loading, triangular loading or joint loading of arbitrary magnitude and locations.

1.2.3 Behavior of Shear Walls [48]

In the design of reinforced concrete multistory buildings, in which lateral load resistance has been assigned to structural walls, the emphasis should be on a rational strategy in the positioning of walls and the establishment of a hierarchy in the development of strength to ensure that in the event of a very large earthquake brittle failure will not occur [23]. The preferred mode of energy dissipation should be flexure in a predictable region. Therefore failures due to diagonal tension or compression, crushing of concrete in compression, sliding along the construction joints, instability of wall elements or reinforcing bars and breakdown of the anchorages should be suppressed. These aims may be achieved with the application of a deterministic design philosophy and they necessitate special detailing and dimensioning of potentially plastic regions of walls.

The usefulness of structural walls in the framing of buildings has long been recognized. When walls are situated in advantageous position in a building, they can form an efficient lateral load-resisting system, while also simultaneously fulfilling other functional requirements. An attempt should be made to inhibit shear failures in reinforced concrete structures subjected to seismic loading. To avoid this undesirable effect of shear, structural walls are used.

1.2.4 Basic Design Philosophy and Requirements [48]

Design principals can not be laid down unless there is a well-defined design philosophy. The generally accepted design philosophy is summarized below [13]:

*Building should suffer no structural damage in minor frequent earthquakes. Normally there should not be non-structural damage either.

*Buildings should suffer none of minor structural damage (i.e. repairable) in occasional moderate earthquakes.

*Buildings should not collapse in rarely occurring major earthquakes. During such earthquakes structures are not expected to remain in the elastic range. Yielding of reinforcing steel will lead to plastic hinges at critical sections.

The general design philosophy will not have much practical use unless design requirements are developed in parallel with this philosophy. The design requirements can be summarized in three groups;

1-Strength requirements

2-Stiffness requirements (or drift control)

3-Ductility requirements

These three requirements will be briefly discussed below.

Strength Requirements

Members in the structure should have adequate strength to carry the design loads safely. Since the designers are well acquainted with this requirement, it will not be discussed in detail. However, it should be pointed out that the designers avoid brittle type of failure, by making a capacity design. If the design shear is computed by placing the ultimate moment capacities at each end of the beam, the designer can make sure that ductile flexural failure will take place prior to shear failure.

Stiffness Requirements

In designing a building for gravity loads, the designer should consider serviceability in addition to ultimate strength. In seismic design, drift limitations imposed might be considered to be some kind of serviceability requirement. However, the drift limitation in seismic design is more important than the serviceability requirement. The limiting drift is usually expressed as the ratio of the relative storey displacement to the storey height (inter-storey drift). Excessive inter-storey drift leads to considerable damage in non-structural elements. In many cases the cost of replacing or repairing of such elements is very high. Excessive inter-storey drift can also lead to very large second order moments (P-Δ effect) that can endanger the safety and stability of the structure. Therefore inter-storey drift control is considered to be one of the most important requirements in seismic design. In the Turkish Earthquake Code [1], the maximum inter-storey drift is limited to the unfavorable one of 0.0035 or 0.02 / R where R is the seismic force reduction factor.

Ductility Requirements

In general it is not economical to design reinforced concrete structures to remain elastic during a major earthquake. It has been demonstrated that structures designed for horizontal loads recommended in the codes can only survive strong earthquakes if they can have the ability to dissipate considerable amount of energy. The energy dissipation is provided mainly by large rotations at plastic hinges. The energy dissipation by inelastic deformations requires the members of the structure and their connections to possess adequate ductility. Ductility is the ability to dissipate a significant amount of energy through inelastic action under large amplitude deformations, without substantial reduction of strength. Adequate ductility can be accomplished by specifying minimum requirements and by proper detailing.

1.2.5 Shear Wall Building Design Considerations [45]

There are many examples of shear wail buildings. From an engineering standpoint, there are many reasons for specifying shear wall resisting systems. From an architectural point of view, a problem arises with placing the shear walls in a strategic location to avoid impacting the view and/or floor plan arrangement of the design. Economical design of shear wall buildings so that the maximum structural efficiency is achieved is of tremendous value to all parties involved. Architectural considerations for the placement of shear walls revolve around efficient use of floor space to satisfy client requirements. A shear wall building requires permanent walls that cannot be moved for future tenant preferences. This is because the wall provides structural resistance and is tied to the floor and ceiling diaphragms. Consequently, for office buildings and retail space, ductile moment frame structures are selected because of the added flexibility provided to the architect designer. Floor plans may be readjusted to accommodate tenant requirements without compromising structural resistance.

1.2.6 Deciding the Location of Shear Walls [48]

Individual walls, generally acting as cantilevers, in a group of structural walls within one building may be subjected to axial, translational and torsional displacements. Depending on its geometric configuration, orientation and location within the plan of the building, a wall will contribute to the resistance of overturning moments, storey shear forces and storey torsion. The position of the structural walls within a building is usually dictated by functional requirements. These may or may not suit the structural planning.

Paulay proposes three aspects in choosing suitable locations for the structural walls [23]:

a. For the best torsional resistance, as many of the walls as possible should be located at the periphery of the building. The walls on each side may be individual cantilevers or they may be coupled to each other.

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

c. In multistory buildings situated in high-seismic-risk areas, a concentration of the total lateral force resistance in only one or two structural walls may introduce very large forces to the foundation of the structure, so that conventional foundations may not be adequate and special enlarged foundations may be required.

1.2.7 Cross-Sectional Shape of the Shear Walls [48]

Individual structural walls of a group may have different sections as shown in Figure1.1 below.

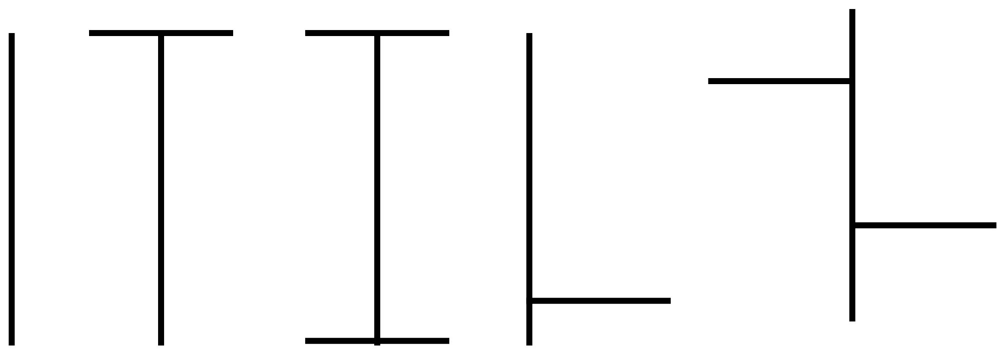

Figure1.1 Some Typical Shapes of Shear Walls

The thickness of such walls is often determined by code requirements for minimum to ensure workability of fresh concrete or to satisfy fire ratings. When the earthquake loading is significant, shear strength and stability requirements, to be examined subsequently in detail, may necessitate an increase in the thickness.

1.2.8 Effect of Shear Wall Geometry [48]

Often walls have openings either in the web or in the flange part of the section. Some judgment is required to assess whether such openings are small, so that they can be neglected in design computations, or large enough to affect either shear or flexural strength. In that case, adequate allowances need to be made in both strength evaluation and the detailing of the reinforcement. It is convenient to examine separately the solid cantilever structural walls and those pierced with openings in some pattern.

i. Shear Walls without Openings

Most cantilever walls, such as shown in Figure1.2, can be treated as ordinary reinforced concrete beam-columns. The lateral load is introduced by means of a series of point loads through the floors acting as diaphragms. Because the floor slab stabilizes the wall against lateral buckling, relatively thin wall sections, may be used. In such walls it is relatively easy to ensure that, when required, a plastic hinge at the base will develop with adequate plastic capacity.

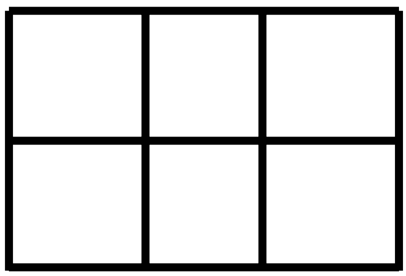

Figure1.2 A Shear Wall System without Openings

ii. Shear Walls with Openings (Coupled Shear Walls)

Frequently, vertical rows of doors or windows may be required within the shear wall for architectural purposes. These shear walls may be considered as full shear walls coupled by connecting beams at each floor level. Coupled shear walls behave as lateral load bearing elements since they are more ductile then solid shear walls while they have favourable characteristics of solid shear walls because of their high strength and rigidity. Plastic deformations on the connecting beams increase the ductility of these structural elements.

When two or more shear walls are interconnected by a system of beams or slabs, it is well known that the total stiffness of the system exceeds the summation of the individual wall stiffness. This is because the connecting beam or slab restrains the individual cantilever action of each wall by forcing the system to work as a composite section.

Planar-coupled shear walls are widely used in apartment buildings and prove to be economical because they divide one apartment unit from another, carry gravity loading and provide stiffness and strength against lateral loading [73].

When arranging openings, it is essential to ensure that a rational structure results, the behavior of which can be predicted by bare inspection. The designer must ensure that the integrity of the structure, in terms of flexural strength, is not in danger by gross reduction of wall area near the extreme fibers of the section. Similarly the shear strength of the wall, in both the horizontal and vertical directions, should remain viable and adequate to ensure that its flexural strength can fully developed.

Extremely efficient structural systems, particularly suited for ductile response with very good energy dissipation characteristics, can be conceived when openings are arranged in a regular pattern. An example is shown in Figure1.3.

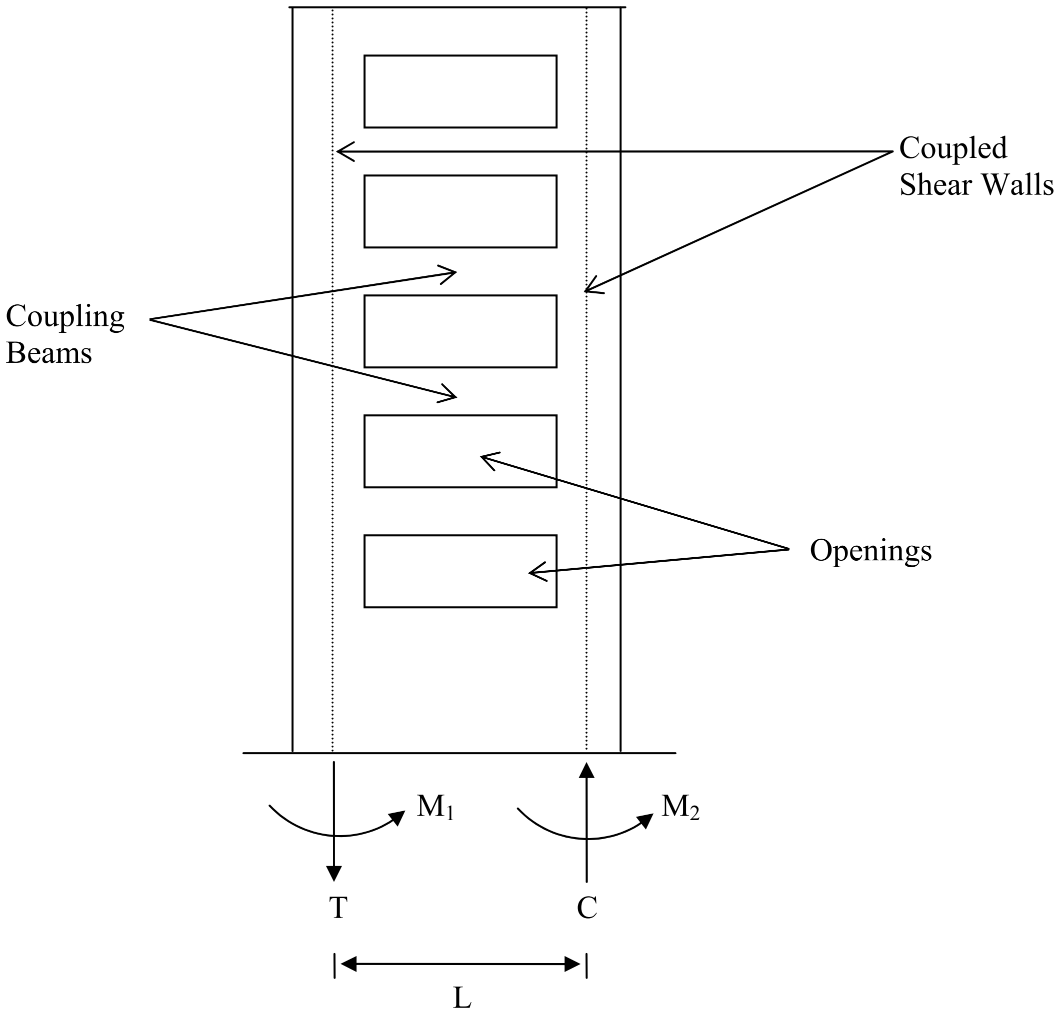

Figure1.3 A Shear Wall System with Openings (Coupled Shear Walls)

It is seen that a number of walls are interconnected or coupled to each other by beams. For this reason they are generally referred to as coupled structural walls. The implication of this terminology is that the connecting beams, which may be relatively short and deep, are substantially weaker than the walls. The walls, which behave predominantly as cantilevers, can then impose sufficient rotations on these connecting beams, when necessary, to make them yield. When suitably detailed, such beams can dissipate energy over the entire height of the structure. These identical walls or two walls of different stiffness may be coupled by a single row of beams. In other cases a series of walls may be interconnected by rows to beams between them.

The coupling beams may be identical at all floors or they may have different depths or widths. In service cores coupled walls may extend above the roof level where elevator machine rooms or space for other services are to be provided. In such cases, walls may be considered to be interconnected by an infinitely rigid diaphragm at the top.

1.2.9 Coupled Shear Wall Buildings [45]

The connecting beams are sized to be lower stiffness (i.e., weaker) than the shear walls. During wind/earthquake loads, the coupling beams will form plastic hinges at their joints with the shear walls. This allows for a ductile response to lateral loads by not allowing the shear walls to deform plastically. Rather, the inelastic damage is confined to the joints of the coupling beams. A specific class of shear wall buildings consists of a combination of shear wall and frame / beam connects as shown in Figure1.3 above. This is called a coupled-wall structure.

Additionally, the coupling effect of the two structural walls results in moment resistance M_t = Total Resisting Moment:

$M_t = M_1 + M_2 + T.L$

where,

M_1 = Moment resistance of shear wall #1.

M_2 = Moment resistance of shear wall #2.

T = Tension force in shear wall #1.

C = Compression force in shear wall #2.

L = Coupling distance/moment arm.

By increasing L, the M_t of the coupled-wall is increased proportionally.

A principle issue is to calculate the necessary strength and detailing of the coupling beams so as to assure proper yield strength. Oversized coupled beams will cause plastic behavior in the structural walls resulting in premature failure. Conversely, under sizing will lead to premature yielding of the joints and also low ductility prior to failure. Therefore, the correct stiffness and detailing of the coupling beam and its joint connection to the shear wall should result in plastic hinging for maximum ductility before failure. This is critical in the overall response of the coupled wall structure.

Coupled walls form plastic hinges at the joint connections with the structural walls. This plastic hinge provides greater wall ductility by allowing energy dissipation in the beam-wall connection. It also creates damage control because the plastic hinge takes the majority of the rotational strain energy.

Ductility plays a vital role in. earthquake response due to the unpredictable nature of seismic excitation. The better design usually should offer maximum ductility and energy dissipation.

Because of their greater stiffness and the dispersal of energy dissipation, coupled structural walls, when suitably detailed, possess optimal seismic properties. The modeling and analysis of these structures has been extensively covered in the relevant literature [24, 75, 77, 78, 82, 83, 84].

1.2.10 Seismic Displacement Compatibility [72]

Recent studies and reviews of established practices in structural seismic design revealed unintentional misuse of fundamental principles (Priestley, 1995 and Paulay, 1997). In certain cases this may seriously affect the expected performance of structures designed for fully or limited ductile response. Mixed structural systems are particularly affected. Typical structures of this type are, for example, those where lateral force resistance is assigned to a set of reinforced concrete cantilever walls with markedly different dimensions and cross sections. Dual systems, in which ductile interacting cantilever structural walls and frames resist lateral forces, belong also to this group. Ductile frames, in which primary elements providing the major fraction of the lateral force resistance and secondary gravity load dominated elements are subjected to similar lateral displacements, are also examples of a mixed structural system.

An important aim in the design for ductile seismic response is to ensure that the probable ductility demand imposed by the design earthquake does not exceed the potential ductility capacity of the structural system. The ductility capacity of the system depends, however on the lateral force resisting element with the minimum displacement ductility capacity. In shear wall dominant structures, significant variations in the element ductility capacities may exist due to the amount of reinforcement and confined regions.

1.3 OBJECTIVE AND SCOPE OF THE STUDY

In the design of reinforced concrete structures, the calculation of lateral sway is very important. However, it is a task that is rather tedious and time consuming. To be able to calculate the sway, a three-dimensional mathematical model becomes necessary. Of course, a computer can do the job. However, every time dimensions and/or placement of structural members change, a new computer model and solution become necessary.

The design engineer needs an analytical tool that can calculate the sway of the structure with ease, particularly at the preliminary design stage. This becomes of utmost importance in deciding on the amount and location of shear walls for seismic design efforts.

In seismic design, employment of shear walls is inevitable. Therefore, an analytical method to accurately access the sway of a composite structure will facilitate the design engineer's efforts to reach an acceptable solution of the earthquake resistant structure.

The analytical method developed will be used to calculate the stability index (Φ), as required by TS-500 in the design of slender columns. The stability index requires the calculation of storey drift (Δ_i) which is tedious work, each time a column is to be designed. The proposed analytical method can do this calculation easily and accurately.

The state-of-the art of seismic design of reinforced concrete structures widely requires and accepts that the employment of shear walls is necessary, but the quantity of the shear walls to be used is still a gray area. The amount of shear walls to be used must fulfill the following three requirements of seismic design.

a. The structure must have adequate strength against the seismic forces

b. The structure must have adequate stiffness against the seismic forces

c. The structure must have adequate ductility to be able to dissipate the seismic energy securely

The analytical tool developed will be used to answer (a) and (b) requirements. In these efforts, it will be assumed that the shear walls, which contain the minimum reinforcement as required by Turkish Earthquake Code, will solely take the total earthquake force. Thus, the amount of shear walls required to meet the strength demand will be determined as a ratio of the floor plan area.

The analytical method developed will be used to determine the amount of shear walls to be used in the structure, such that the structure will possess enough stiffness against sway as required by the Turkish Earthquake Code.

The ratios of shear walls to be used that satisfy strength and stiffness demand will be coordinated to propose a method for the design of shear walls.

An important task expected of the shear walls during the seismic attack is to be able to dissipate the seismic energy. In order to perform this vital task, the total structure must possess enough ductility. The accepted measure in the state-of-the art of seismic design is that when the structure performs the displacement ductility ratio of $\mu_{\Delta} = \Delta_u / \Delta_y = 4\text{-}5$, the structure is accepted as capable of dissipating the seismic energy successfully. But how can it be decided when a structure consisting of many shear walls begins to yield (i.e. the yield sway of the structure, Δ_y)?

The analytical method proposed will be used to assess the yield sway of the structure. In order to do this, the moment distributions along the height of shear walls have to be known. The analytical method proposed, will successfully determine the sway, moment, shear force and distributed load patterns along the height of the shear walls.

In determining the sway ductility ratio, the ultimate sway of the structure has to be known. A plastic analysis will be applied to determine the ultimate sway of the structure.

However, the question has not yet been answered where the shear walls are ductile enough to permit the realization of the sway ductility ratio. Ductility is an elusive concept. The best way the design engineer knows is to quantify ductility on the Thrust-Moment-Curvature relationship. Therefore, relating the sway ductility of the structure to curvature ductility of the shear wall becomes necessary. Sway ductility of the structure will be related to the curvature ductility of the shear wall by using a plastic analysis.

The analytical method proposed will be implemented in detail on a design example. Capacity Design procedures will also be applied in order to make sure that shear failure will never occur before a ductile flexural failure.

CHAPTER 2

PROCEDURE FOR ANALYTICAL METHOD OF SEISMIC ANALYSIS OF FRAMED STRUCTURES

2.1 ANALYTICAL MODEL OF FRAMED STRUCTURE AS SHEAR BEAM

Consider a multi-bay multi-storey framed structure subject to a lateral load of F=1 as shown in Figure2.1.

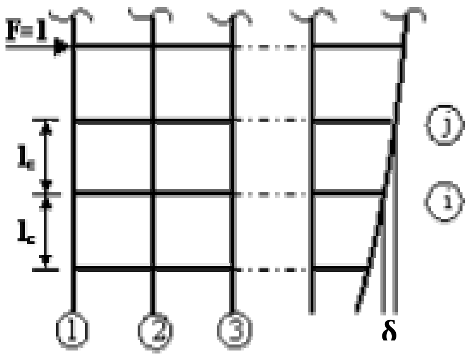

Figure 2.1 Framed Structure Subject to Lateral Load of F=1 and Relative Sway [2]

The relative sway (δ) that occurs between two consecutive stories can be calculated as follows (Baikov, 1974) [53].

$$\delta = \sum_{i=1}^{n} \frac{1}{\dfrac{12EI_c}{l_c^3}\,\dfrac{1}{1+\dfrac{2I_c}{l_c\left(\dfrac{I_{b1}}{l_1}+\dfrac{I_{b2}}{l_2}\right)}}} \tag{2.1}$$

where

n = Number of columns in the storey

I_{b1} = Moment of inertia of beam to the left of the column considered

I_{b2} = Moment of inertia of beam to the right of the column considered

E = Modulus of elasticity of concrete

I_c = Moment of inertia of column

l_c = Length of column

In the case that F=1.0 acts at all floor levels, the total relative storey sway becomes

$$\Delta_i = (\delta).(V_i) \tag{2.2}$$

where

V_i = total shear force at storey level (i) considered.

The total sway of (k) th storey is obtained by summing up all relative storey sways up to the storey (k).

$$y = \sum_{i=1}^{k} \Delta_i = \delta.\sum_{i=1}^{k} V_i \tag{2.3}$$

Considering the framed structure as a continuous shear beam subject to a continuous lateral force along its height, Eqn.2.2 can be expressed as a differential equation (Baikov, 1974) [53].

$$y = \frac{\delta}{l_c}\int_0^x V(x)dx \tag{2.4}$$

$$y = \frac{1}{GA}\int_0^x V(x)dx \tag{2.5}$$

where

GA = equivalent shear stiffness of the continuous shear beam model.

Figure 2.2 represents the continuous shear beam model of a framed structure, subject to continuous lateral load.

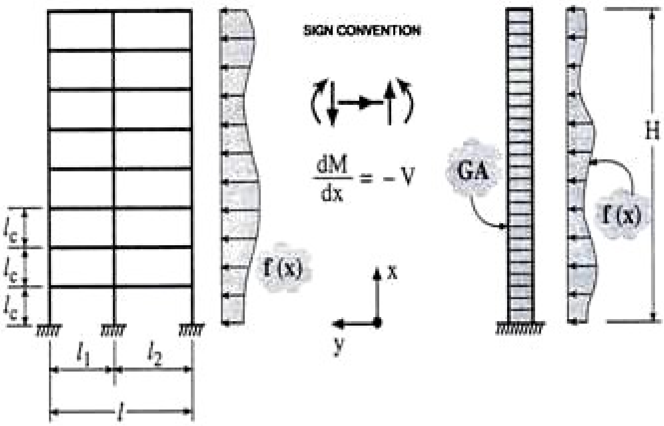

Figure 2.2 Continuous Shear Beam Model of Framed Structure [2]

The solution of the differential equation is as follows:

$$y = \frac{1}{GA}\int_0^x V(x)dx = \frac{-[M(x) - M(0)]}{GA} \quad (2.6)$$

where

M(x) = moment due to the external lateral load at any level (x) of the shear beam.

For a distributed triangular load, which simulates lateral seismic forces, lateral sway at any height of the building can be expressed as below (Atimtay, 2001) [2].

$$y = \frac{pH^2}{2GA}(k - \frac{k^3}{3}) \quad (2.7)$$

where

p = intensity of distributed triangular lateral load at the top of structure

$$k = \frac{x}{H}$$

Slope along height of the building can be expressed as in Eqn.2.8.

$$y^1 = \frac{pH^2}{2GA}(\frac{1}{H} - \frac{x^2}{H^3}) \quad (2.8)$$

Lateral sway at any height of the building and slope along height of the building can be easily calculated by using the executable "Borland Delphi" program developed, which is shown in Figure 2.3.

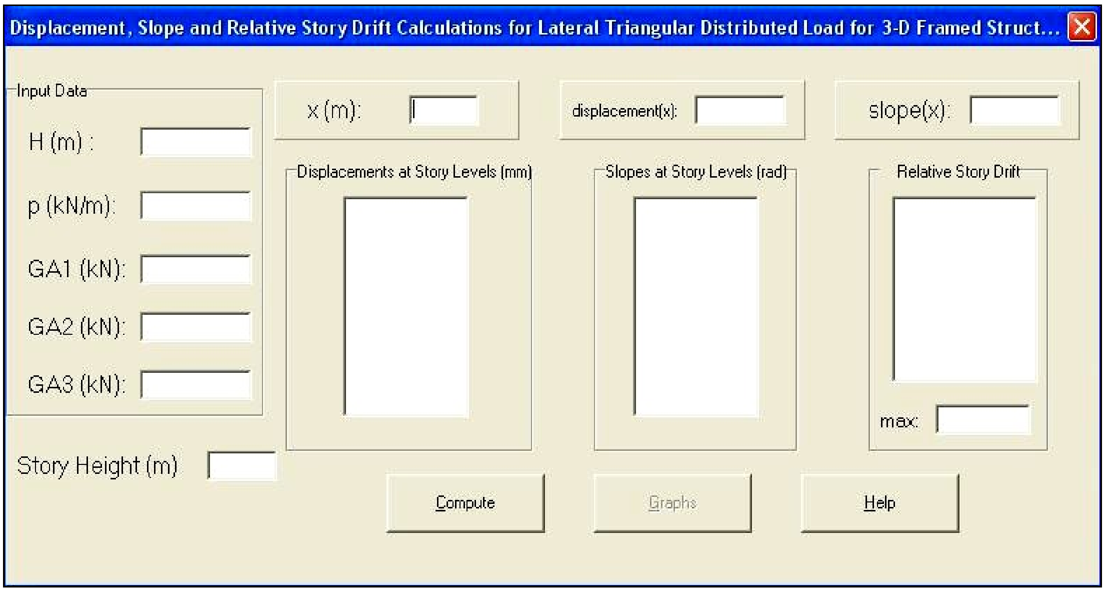

Figure 2.3 Executable "Borland Delphi" Program to Calculate Lateral Sway, Slope and Relative Story Drift for Framed Structures

2.2 EQUIVALENT SHEAR STIFFNESS (GA) OF A 3-D FRAME

The equivalent shear stiffness, GA, for a single column is given as expressed in Equation 2.9 (Baikov, 1974) [53].

$$GA = \frac{12E_c I_c}{l_c^2} . \frac{1}{1 + \dfrac{2I_c}{l_c \left(\dfrac{I_{b1}}{l_1} + \dfrac{I_{b2}}{l_2} \right)}} \tag{2.9}$$

To find the equivalent shear stiffness of a 3-D framed building, Equation 2.9 must be applied to all columns within the story and $\sum GA$ must be evaluated.

For simplicity, a program was written by using “Borland Delphi” to find GA for each column. Then $\sum GA$ can be calculated easily. The executable “Borland Delphi” program, written for α=1.25, α=1.6 and α=2.6 separately, is shown in Figure2.4.

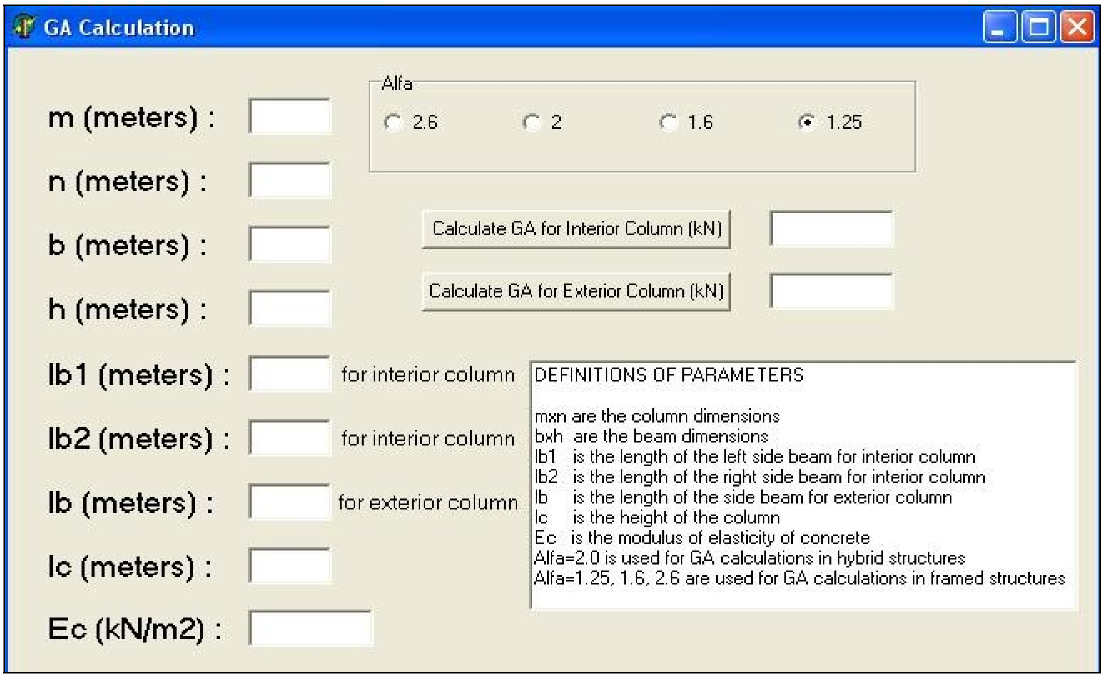

Figure 2.4 Executable “Borland Delphi” Program to Calculate GA

An ambiguity exists in the evaluation of I_{b1} and I_{b2} of the flanged beam. What should the effective flange width be taken?

To determine the effective flange width, a systematic study was done to correlate sways obtained by computer and the developed analytical equation.

The moment of inertia of the flanged beam was expressed as

$$I_b = \alpha . \frac{1}{12} bh^3 \qquad (2.10)$$

where

b = width of the rectangular beam

h = height of the rectangle that can be fit in the flanged beam cross-section

α = coefficient expressing the stiffness of flanged beam as a multiple of the rectangular beam.

The correlation of computer sways with those found analytically, yielded the values of α as shown in Figure 2.5. It is interesting to note that α varies along the building height, which is expressed as a parameter of the number of stories, in lateral displacement calculations. It should be noted that α is taken as 2.6 for the first story only and 1.25 for all the other stories in relative storey drift calculations while α is taken as 1.25 for all stories in slope calculations.

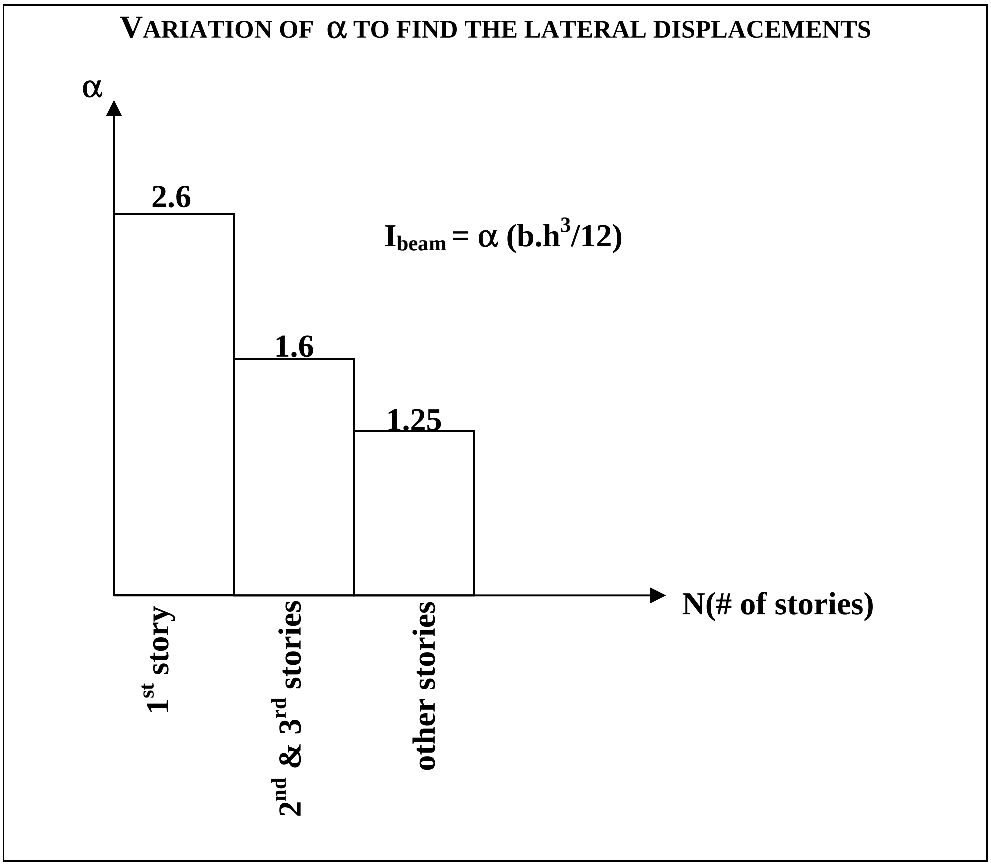

Figure 2.5 Expressing the Stiffness of Flanged Beam as a Multiple (α) of the Stiffness of Rectangular Beam

2.3 ASSESSING THE VALIDITY OF THE ANALYTICAL MODEL

The validity of the analytical model developed was tested on a 3D-framed structure (with different number of stories) shown in Figure 2.6 by comparing the results, which are determined by using SAP2000 and analytical method.

The column axial deformations were neglected in the derivation of the analytical equation. On the other hand, SAP2000 program takes this effect into consideration.

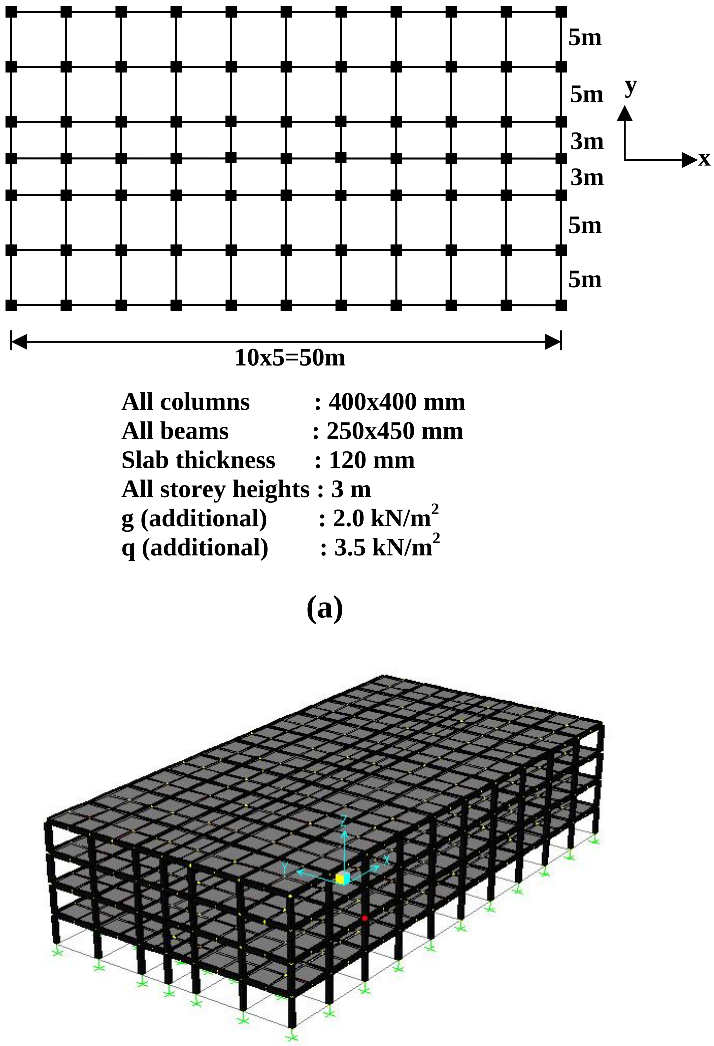

Figure 2.6 Framed Structure Used to Test the Validity of the Analytical Method: **(a)** Typical floor plan, **(b)** 3-D view of a sample 4-storey framed structure

The total lateral force (i.e. base shear) was determined by SAP 2000 using the Response Spectrum Method of dynamic analysis. The seismic force thus obtained is converted to an equivalent distributed static force having an inverted triangular shape. This equivalent lateral static force, tabulated in Table 2.1, was applied to the structure and solved by the computer using SAP 2000 and the analytical equation.

Table 2.1 Base Shear and Top Intensity of Triangular Lateral Static Load for the Framed Structures Studied

Number of Story	Base Shear in x-direction Vt_x(kN)	Top intensity of triangular lateral static load p (kN/m)
2	2301	767.2
4	2555	425.8
6	2823	313.7
8	3092	257.7
10	3279	218.6
15	3339	148.4
20	3443	114.8

GA along x-direction can be calculated for the 3-D framed structure shown in Figure 2.6 as follows.

$E_c = 28\,500\,000$ kN/m^2

$$I_{column} = \frac{1}{12}(0.4)(0.4)^3 = 0.00213\,\text{m}^4$$

$$I_{beam} = \alpha\ \frac{1}{12}(0.25)(0.45)^3 \ = 0.001898437\alpha\ \text{m}^4$$

$$GA_{ext.column} = \frac{80940\alpha}{\alpha + 3.74078} \quad \text{kN}$$

$$GA_{int.column} = \frac{80940\alpha}{\alpha + 1.87039} \quad kN$$

$$GA_{structure} = 7.[9.GA_{int.column} + 2.GA_{ext.column}] \quad kN$$

For α = 1.25, $GA_{ext.column}$ = 20 284 kN and $GA_{int.column}$= 32 449 kN can be calculated easily by using the executable "Borland Delphi" program written for α = 1.25. Therefore $GA_{structure}$= 2 328 240 kN is obtained.

For α = 1.6, $GA_{ext.column}$ = 24 263 kN and $GA_{int.column}$= 37 348 kN can be calculated easily by using the executable "Borland Delphi" program written for α = 1.6. Therefore $GA_{structure}$= 2 692 640 kN is obtained.

For α = 2.6, $GA_{ext.column}$ = 33 215 kN and $GA_{int.column}$= 47 123 kN can be calculated easily by using the executable "Borland Delphi" program written for α = 2.6. Therefore $GA_{structure}$= 3 433 720 kN is obtained.

As a result, GA (1) = 3 433 720 kN is used for first story, GA (2) = 2 692 640 kN is used for second & third stories and GA (3) = 2 328 240 kN is used for the other stories in displacement calculations. On the other hand, GA (1) =343 372 ton is used for first story only and GA (3) = 2 328 240 kN is used for all the other stories in relative story drift calculations. It should be mentioned that GA (3) = 2 328 240 kN is used for all stories in slope calculations.

All these conclusions for GA calculations in framed structures are tabulated in Table 2.2.

Table 2.2 Variation of α for GA Calculations in Framed Structures

	Displacement Calculations	Relative Story Drift Calculations	Slope Calculations
1st Story	GA (with α=2.6)	GA (with α=2.6)	GA (with α=1.25)
2nd & 3rd Stories	GA (with α=1.6)	GA (with α=1.25)	GA (with α=1.25)
Other Stories	GA (with α=1.25)	GA (with α=1.25)	GA (with α=1.25)

In this study, relative story drift is calculated by the equation defined in Turkish Earthquake Code (1997) [1] as expressed in Eqn.2.11.

$\Delta_i = d_i - d_{i-1}$ (Story Drift)

$$\frac{\Delta_i}{h_i} = \frac{d_i - d_{i-1}}{h_i} \text{ (Relative Story Drift)} \qquad (2.11)$$

The maximum value of storey drifts within a story, $(\Delta_i)_{max}$, calculated for columns and structural walls of the i'th storey of a building for each earthquake direction shall satisfy the unfavorable one of the following conditions given by Eqns.2.12a & b.

$$(\Delta_i)_{max} / h_i \leq 0.0035 \qquad (2.12a)$$

$$(\Delta_i)_{max} / h_i \leq 0.02 / R \qquad (2.12b)$$

In the cases where the conditions specified by Eqns.2.12a & b are not satisfied at any storey, the earthquake analysis shall be repeated by increasing the stiffness of the structural system.

On the other hand, slope along height at story levels is calculated by dividing the story drift between the mid heights of two consecutive stories to the story height. In other words, relative story drift between the mid heights of two consecutive stories is considered as the slope along height at story levels in this study.

2.4 COMPARISON OF RESULTS

The comparison of lateral displacements together with story drifts and the comparison of slope along height at story levels are shown in tabular forms in Table 2.3 and Table 2.4, respectively. On the other hand, comparison of lateral displacements is also shown graphically from Figure 2.7 to Figure 2.13 while the comparison of slope along height at story levels is shown from Figure 2.14 to Figure 2.20. Finally, the comparison of relative story drifts is shown graphically from Figure 2.21 to Figure 2.27.

Table 2.3 Comparison of Lateral Displacements and Relative Story Drifts as Determined by SAP2000 and Analytical Model for Framed Structure

# of	Displacement	Displacement	Difference	Relative Story Drift	Relative Story Drift	Difference
story	**Sap2000(mm)**	**Analytic(mm)**	**(%)**	**(Sap2000)**	**(Analytic)**	**(%)**
2	2.88	3.42	18.75	0.0004100	0.0004119	0
1	1.65	1.84	11.51	**0.0005500**	**0.0006144**	11.51
				max=0.0005500	**max=0.0006144**	**11.51**
4	7.81	8.78	12.42	0.0002967	0.0002515	14.6
3	6.92	6.94	0.28	0.0006767	0.0006630	1.97
2	4.89	5.22	6.75	**0.0009167**	**0.0009373**	2.18
1	2.14	2.18	1.87	0.0007133	0.0007285	1.87
				max=0.0009167	**max=0.0009373**	**2.18**
6	13.81	14.55	5.36	0.0002333	0.0001909	18.57
5	13.11	13.97	6.56	0.0005533	0.0005277	4.21
4	11.45	12.39	8.21	0.0008333	0.0007972	4.4
3	8.95	8.65	3.35	0.0010367	0.0009993	3.85
2	5.84	6.06	3.77	**0.0011333**	**0.0011340**	0.3
1	2.44	2.44	0	0.0008133	0.0008146	0
				max=0.0011333	**max=0.0011340**	**0.3**
8	20.86	21.25	1.87	0.0002033	0.0001591	21.31
7	20.25	20.77	2.57	0.0004800	0.0004497	6.25
6	18.81	19.42	3.24	0.0007400	0.0006987	5.85
5	16.59	17.33	4.46	0.0009600	0.0009062	5.55
4	13.71	14.61	6.56	0.0011267	0.0010723	4.73
3	10.33	9.85	4.65	0.0012533	0.0011968	4.52
2	6.57	6.75	2.74	**0.0012900**	**0.0012798**	0.77
1	2.7	2.69	0.37	0.0009000	0.0008959	0.37
				max=0.0012900	**max=0.0012798**	**0.77**
10	28.28	28.17	0.39	0.0001867	0.0001362	26.78
9	27.72	27.76	0.14	0.0004267	0.0003897	8.59
8	26.44	26.59	0.57	0.0006633	0.0006151	7.03
7	24.45	24.74	1.19	0.0008667	0.0008123	6.53
6	21.85	22.31	2.1	0.0010467	0.0009813	6.05
5	18.71	19.36	3.47	0.0011933	0.0011221	5.86
4	15.13	15.99	5.68	0.0013067	0.0012348	5.61
3	11.21	10.63	5.17	**0.0013867**	0.0013193	5.04
2	7.05	7.21	2.27	0.0013900	**0.0013757**	0.95
1	2.88	2.86	0.69	0.0009600	0.0009519	0.69
				max=0.0013867	**max=0.0013757**	**0.72**

Table 2.3 Comparison of Lateral Displacements and Relative Story Drifts as Determined by SAP2000 and Analytical Model for Framed Structure (Continued)

# of	Displacement	Displacement	Difference	Relative Story Drift	Relative Story Drift	Difference
story	**Sap2000(mm)**	**Analytic(mm)**	**(%)**	**(Sap2000)**	**(Analytic)**	**(%)**
15	44.86	43.02	4.1	0.0001567	0.0000935	40.43
14	44.39	42.74	3.72	0.0003267	0.0002720	17.35
13	43.41	41.93	3.41	0.0005000	0.0004377	12.0
12	41.91	40.61	3.1	0.0006600	0.0005906	10.61
11	39.93	38.84	2.73	0.0008100	0.0007309	9.87
10	37.5	36.65	2.27	0.0009400	0.0008584	8.51
9	34.68	34.07	1.76	0.0010600	0.0009731	8.17
8	31.5	31.15	1.11	0.0011667	0.0010751	8.0
7	28	27.93	0.25	0.0012567	0.0011643	7.43
6	24.23	24.44	0.87	0.0013333	0.0012408	6.75
5	20.23	20.71	2.37	0.0013967	0.0013045	6.68
4	16.04	16.8	4.74	0.0014467	0.0013555	6.22
3	11.7	11.01	5.89	**0.0014767**	0.0013938	5.64
2	7.27	7.39	1.65	0.0014400	**0.0014193**	1.62
1	2.95	2.91	1.35	0.0009833	0.0009710	1.35
				max=0.0014767	**max=0.0014193**	**4.1**
20	63.51	59.17	6.83	0.0001567	0.0000727	53.19
19	63.04	58.95	6.49	0.0002933	0.0002133	27.27
18	62.16	58.31	6.19	0.0004333	0.0003464	20.0
17	60.86	57.27	5.89	0.0005633	0.0004721	15.97
16	59.17	55.85	5.61	0.0006867	0.0005905	14.07
15	57.11	54.08	5.31	0.0008067	0.0007014	13.22
14	54.69	51.98	4.95	0.0009133	0.0008049	11.67
13	51.95	49.56	4.6	0.0010100	0.0009011	10.89
12	48.92	46.86	4.21	0.0011067	0.0009899	10.54
11	45.6	43.89	3.75	0.0011867	0.0010712	9.83
10	42.04	40.68	3.24	0.0012667	0.0011452	9.47
9	38.24	37.24	2.61	0.0013300	0.0012117	9.02
8	34.25	33.61	1.87	0.0013867	0.0012709	8.17
7	30.09	29.79	0.99	0.0014400	0.0013227	8.33
6	25.77	25.83	0.23	0.0014800	0.0013671	7.65
5	21.33	21.73	1.87	0.0015133	0.0014040	7.04
4	16.79	17.51	4.28	0.0015367	0.0014336	6.72
3	12.18	11.43	6.16	**0.0015467**	0.0014558	5.81
2	7.54	7.65	1.46	0.0014967	**0.0014706**	1.78
1	3.05	3.01	1.31	0.0010166	0.0010022	1.31
				max=0.0015467	**max=0.0014706**	**4.96**

Table 2.4 Comparison of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model for Framed Structure

# of story	Slope along height at story levels Sap2000(rad)	Slope along height at story levels Analytic(rad)	Difference (%)
2	0.0003333	0.0000000	-
1	0.0005323	0.0007414	31.1
4	0.0002133	0.0000000	-
3	0.0004967	0.0004801	3.3
2	0.0008167	0.0008230	0.8
1	0.0009057	0.0010287	13.6
6	0.0001600	0.0000000	-
5	0.0004033	0.0003705	8.1
4	0.0006967	0.0006737	3.3
3	0.0009433	0.0009095	3.6
2	0.0011000	0.0010779	2.0
1	0.0010680	0.0011789	10.4
8	0.0001333	0.0000000	-
7	0.0003533	0.0003113	11.9
6	0.0006100	0.0005811	4.7
5	0.0008533	0.0008094	5.1
4	0.0010500	0.0009962	5.1
3	0.0011967	0.0011414	4.6
2	0.0012833	0.0012452	2.9
1	0.0011967	0.0013075	9.2
10	0.0001200	0.0000000	-
9	0.0003200	0.0002676	16.4
8	0.0005433	0.0005071	6.7
7	0.0007700	0.0007184	6.7
6	0.0009567	0.0009015	5.8
5	0.0011233	0.0010564	5.9
4	0.0012533	0.0011832	5.6
3	0.0013500	0.0012818	5.1
2	0.0014033	0.0013522	3.6
1	0.0012833	0.0013945	8.7

Table 2.4 Comparison of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model for Framed Structure (Continued)

# of story	Slope along height at story levels Sap2000(rad)	Slope along height at story levels Analytic(rad)	Difference (%)
15	0.0000933	0.0000000	-
14	0.0002600	0.0001848	28.9
13	0.0004100	0.0003569	12.9
12	0.0005833	0.0005163	11.5
11	0.0007333	0.0006629	9.6
10	0.0008767	0.0007967	9.1
9	0.0010033	0.0009178	8.5
8	0.0011133	0.0010262	7.8
7	0.0012133	0.0011218	7.5
6	0.0012967	0.0012047	7.1
5	0.0013667	0.0012748	6.7
4	0.0014233	0.0013321	6.4
3	0.0014633	0.0013768	5.9
2	0.0014700	0.0014086	4.2
1	0.0013200	0.0014278	8.2
20	0.0000933	0.0000000	-
19	0.0002467	0.0001442	41.5
18	0.0003600	0.0002811	21.9
17	0.0005000	0.0004105	17.9
16	0.0006267	0.0005325	15.0
15	0.0007467	0.0006472	13.4
14	0.0008600	0.0007544	12.3
13	0.0009633	0.0008543	11.3
12	0.0010567	0.0009467	10.4
11	0.0011500	0.0010318	10.3
10	0.0012267	0.0011094	9.6
9	0.0012967	0.0011797	9.0
8	0.0013600	0.0012426	8.7
7	0.0014167	0.0012980	8.4
6	0.0014600	0.0013461	7.8
5	0.0014967	0.0013868	7.4
4	0.0015267	0.0014201	7.0
3	0.0015433	0.0014459	6.3
2	0.0015333	0.0014644	4.5
1	0.0013700	0.0014755	7.7

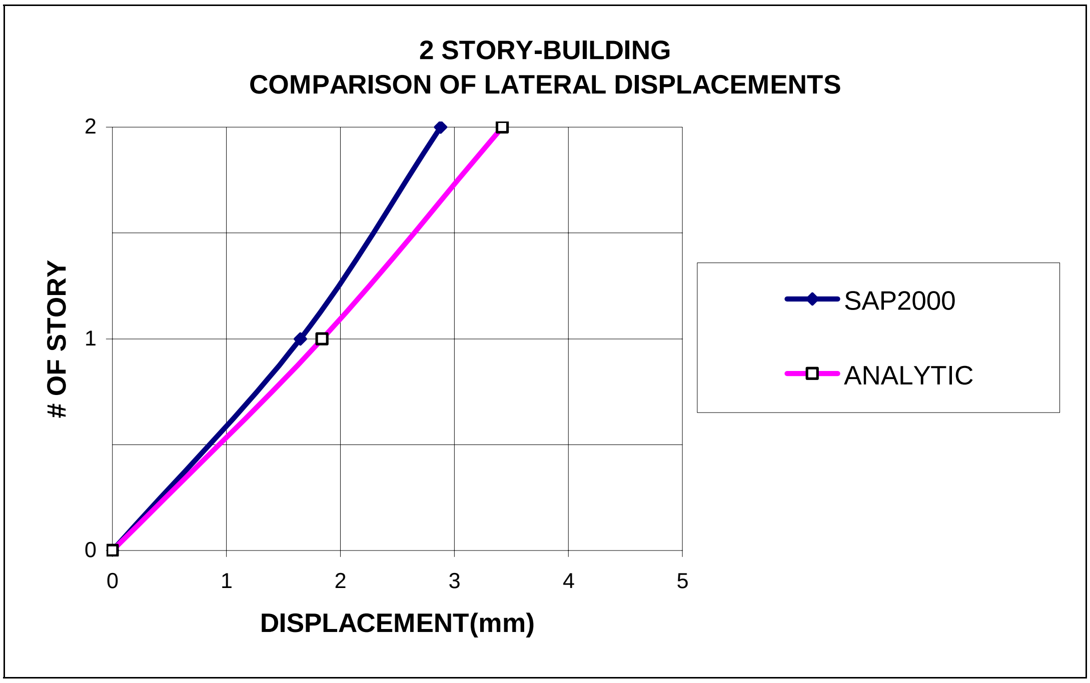

Figure 2.7 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 2 Story-Framed Structure)

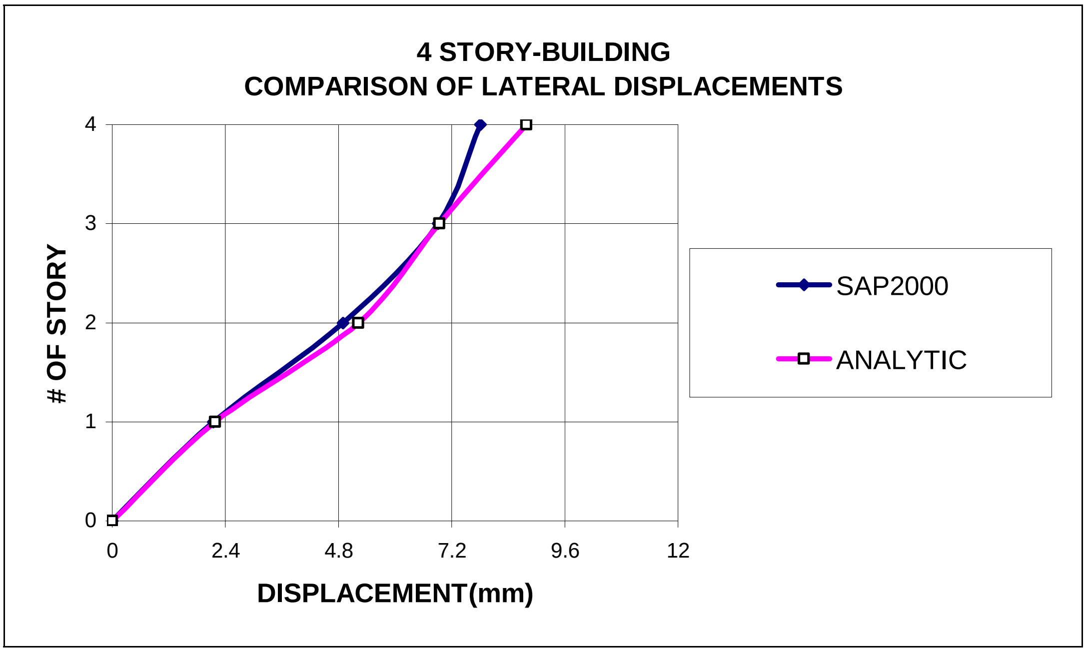

Figure 2.8 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 4 Story-Framed Structure)

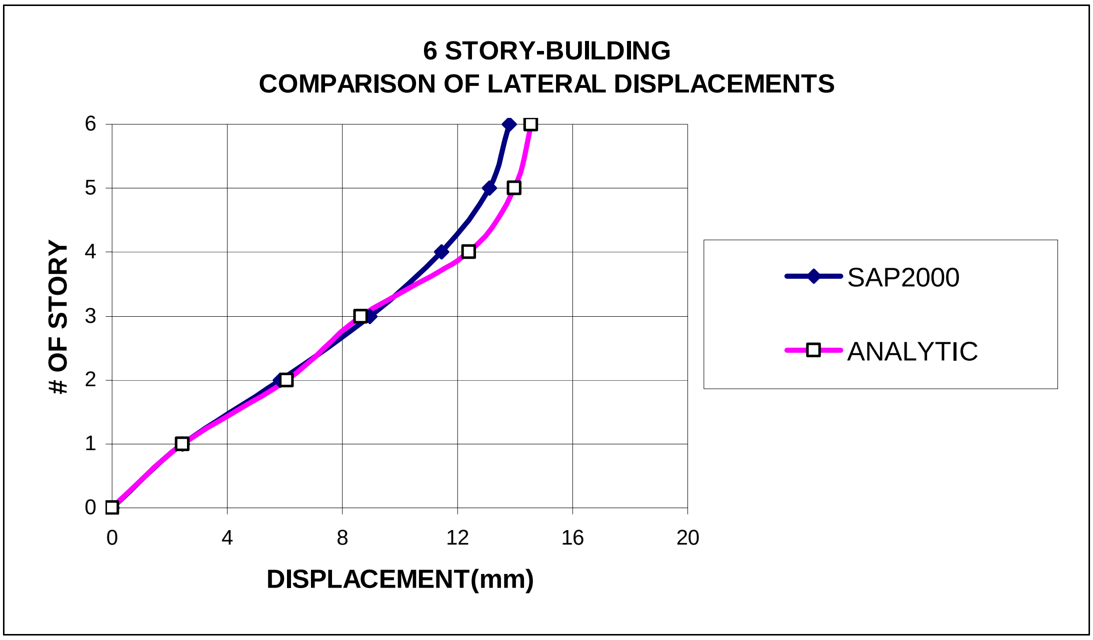

Figure 2.9 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 6 Story-Framed Structure)

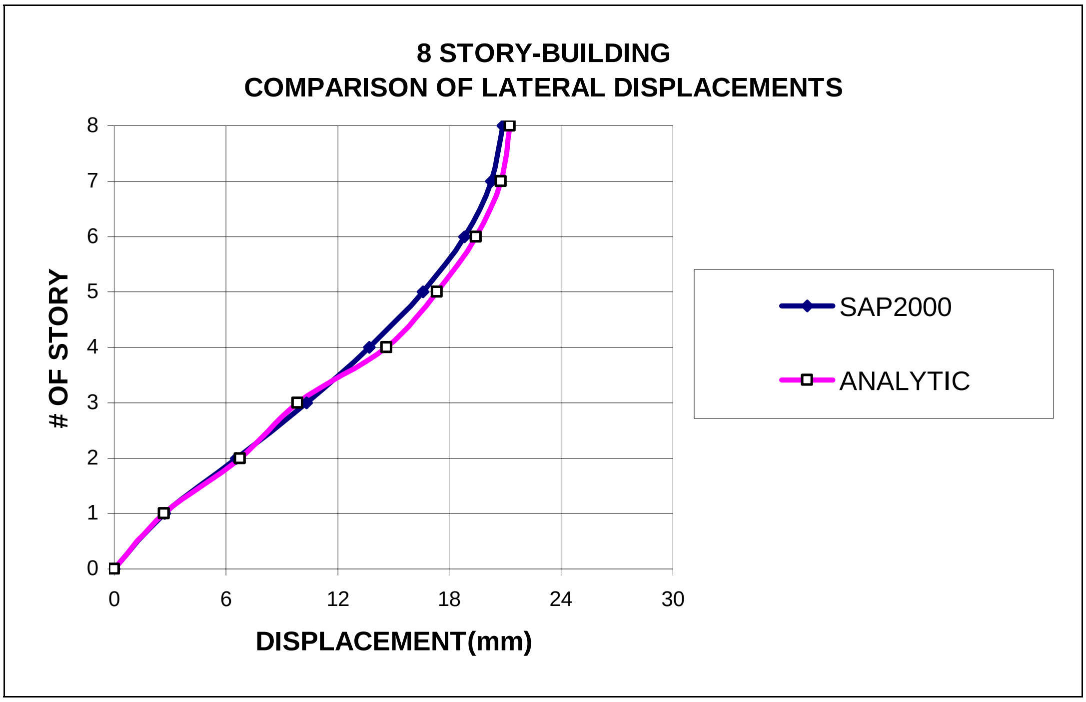

Figure 2.10 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 8 Story-Framed Structure)

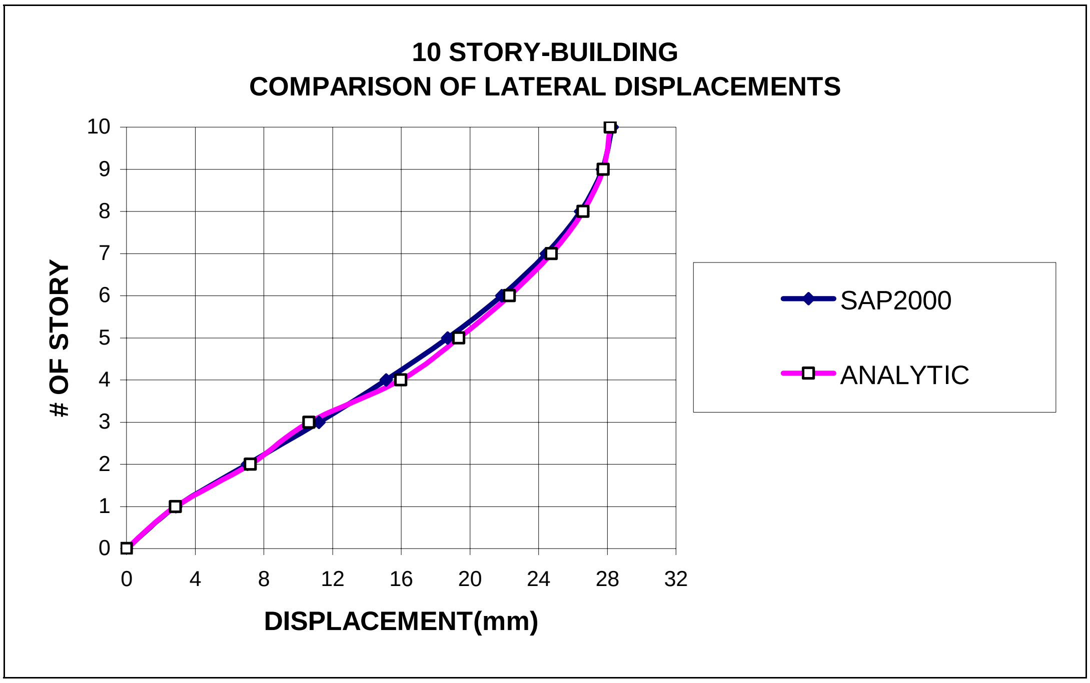

Figure 2.11 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 10 Story-Framed Structure)

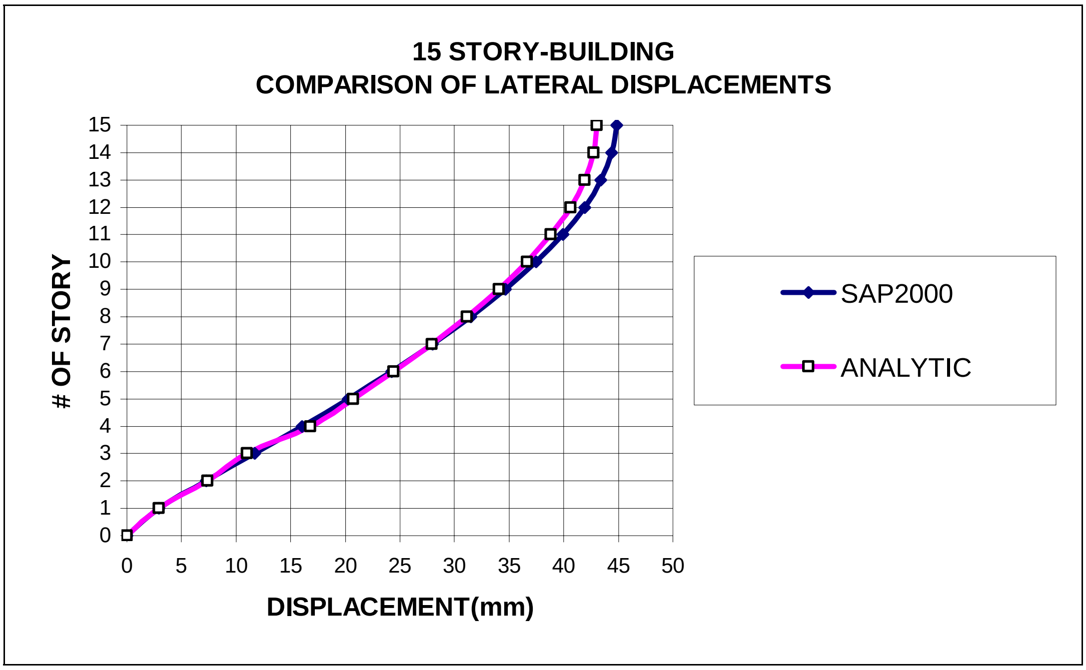

Figure 2.12 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 15 Story-Framed Structure)

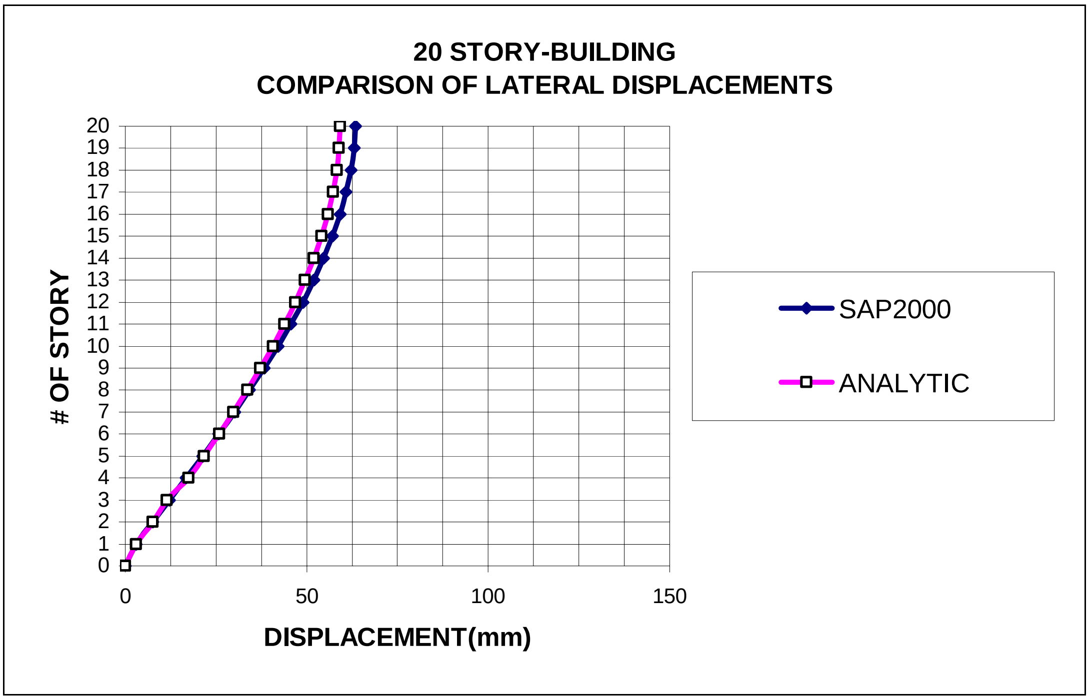

Figure 2.13 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 20 Story-Framed Structure)

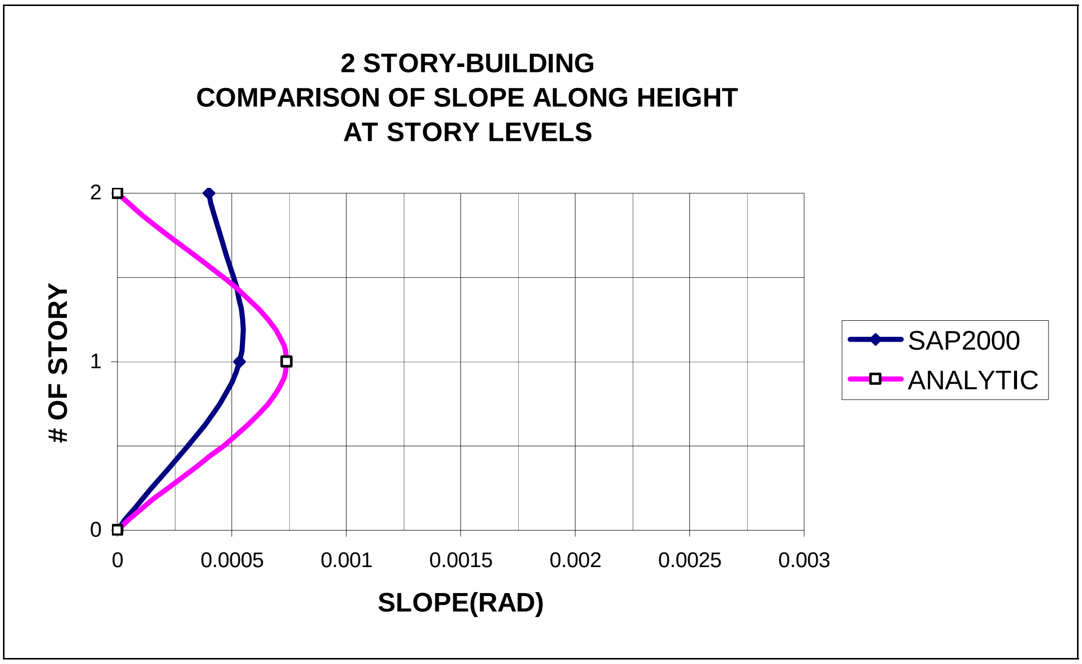

Figure 2.14 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 2 Story-Framed Structure)

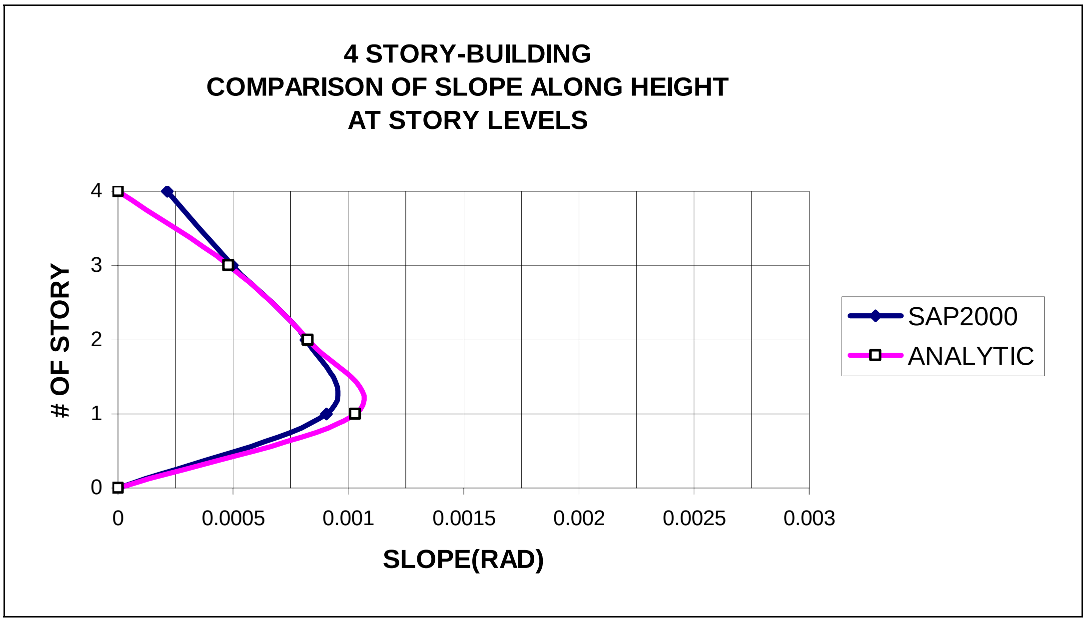

Figure 2.15 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 4 Story-Framed Structure)

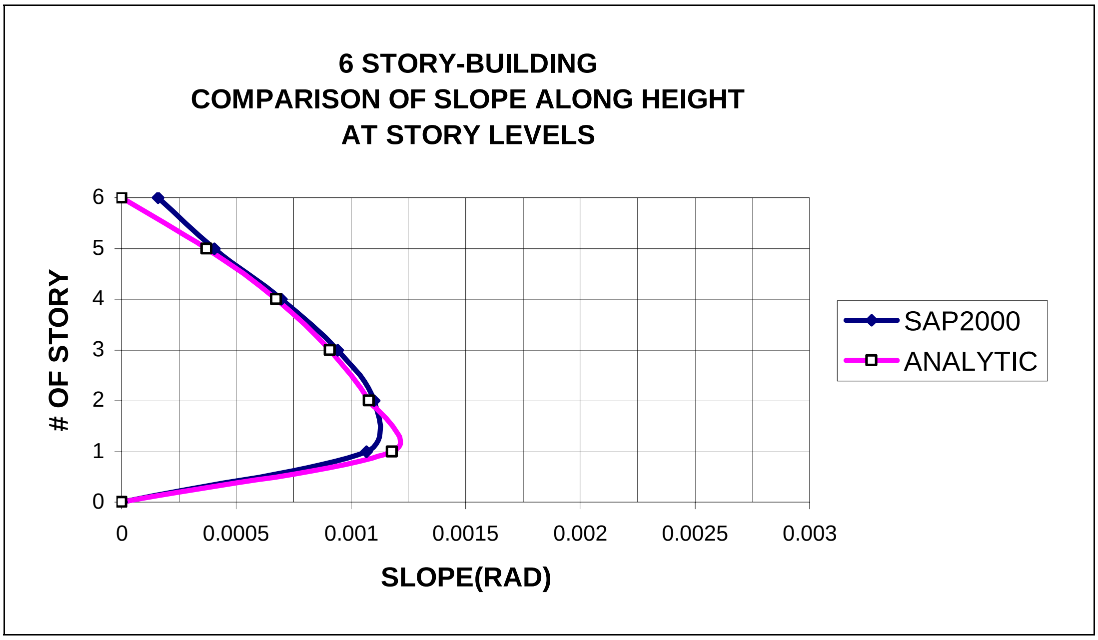

Figure 2.16 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 6 Story-Framed Structure)

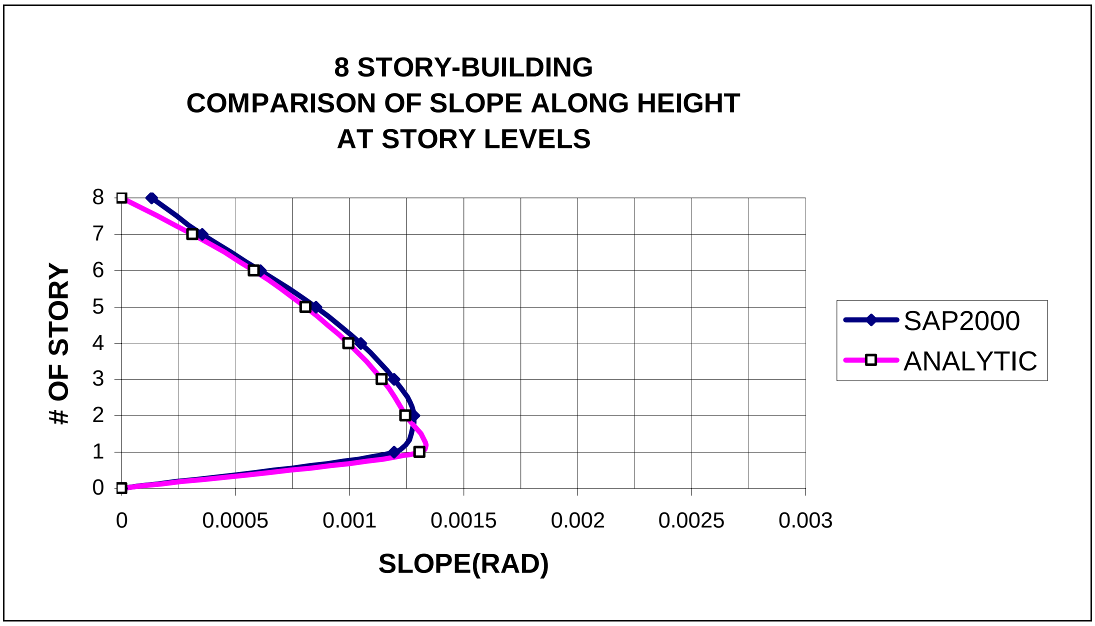

Figure 2.17 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 8 Story-Framed Structure)

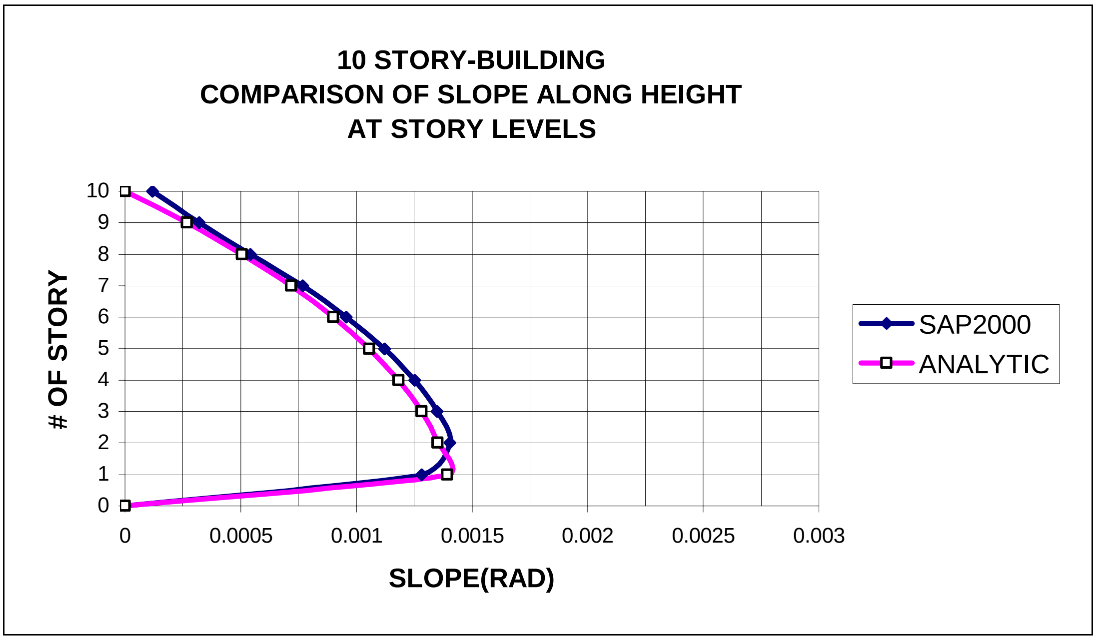

Figure 2.18 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 10 Story-Framed Structure)

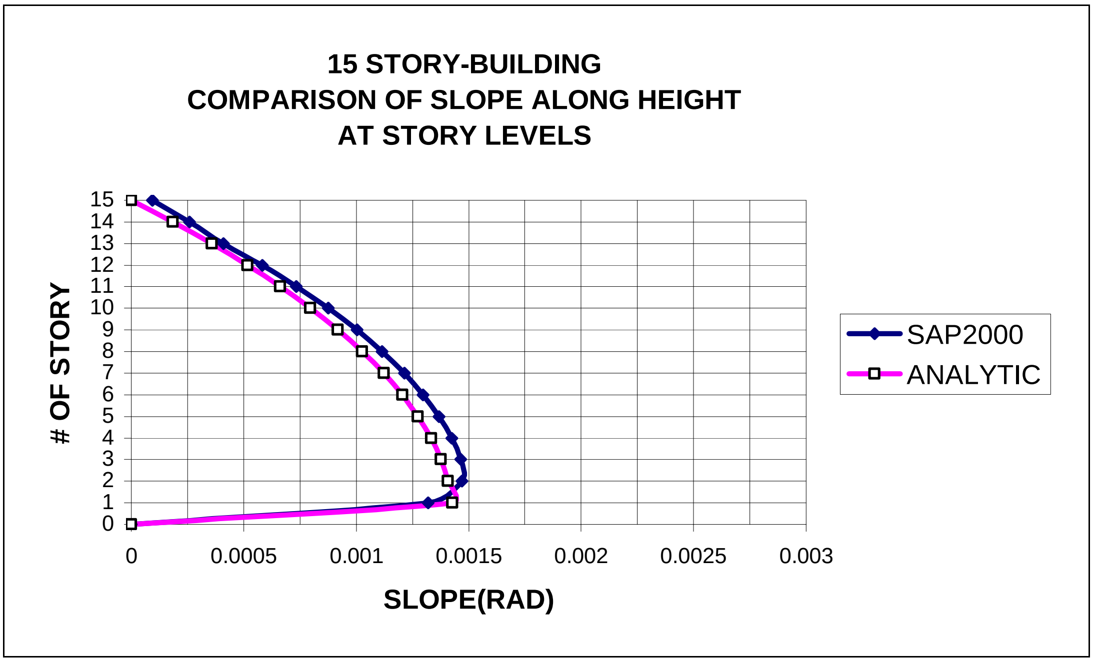

Figure 2.19 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 15 Story-Framed Structure)

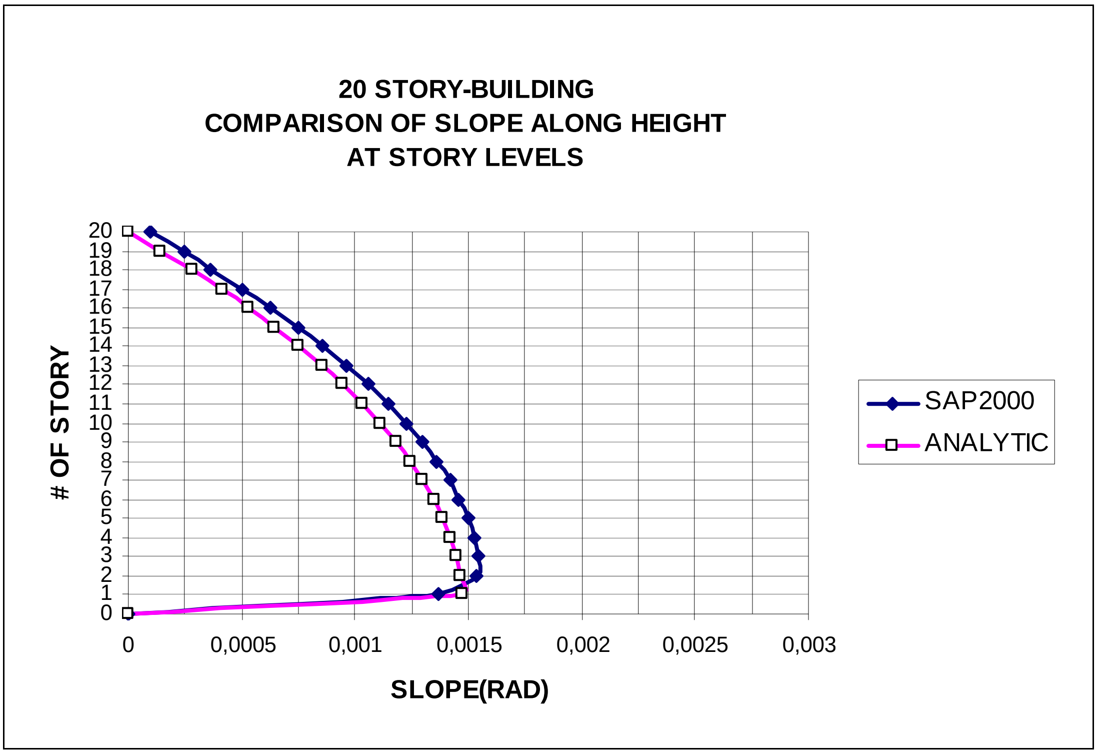

Figure 2.20 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 20 Story-Framed Structure)

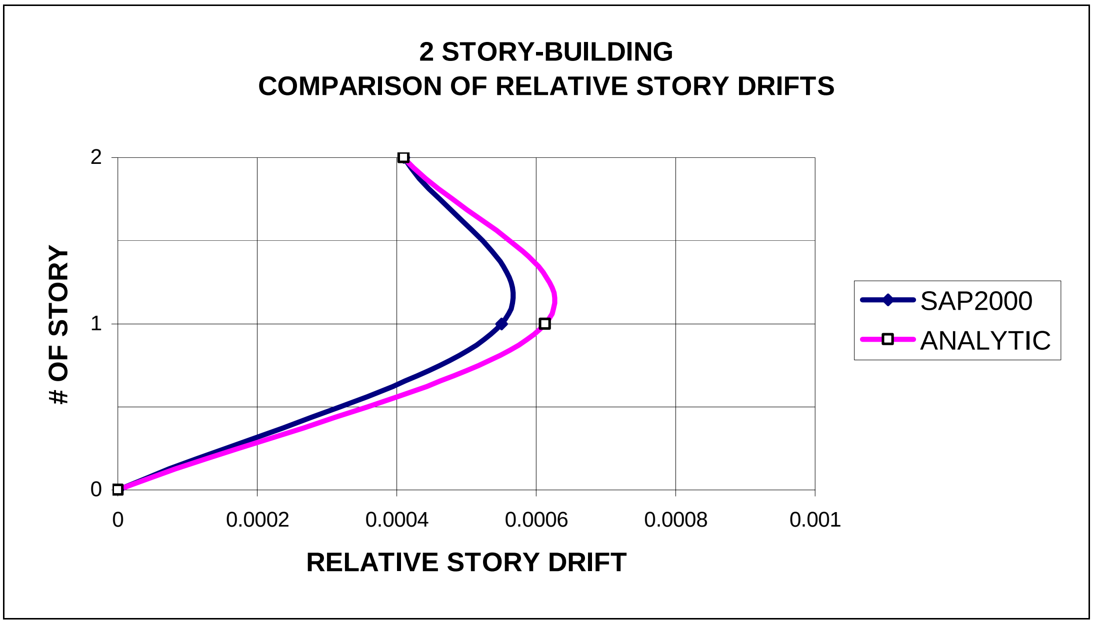

Figure 2.21 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 2 Story-Framed Structure)

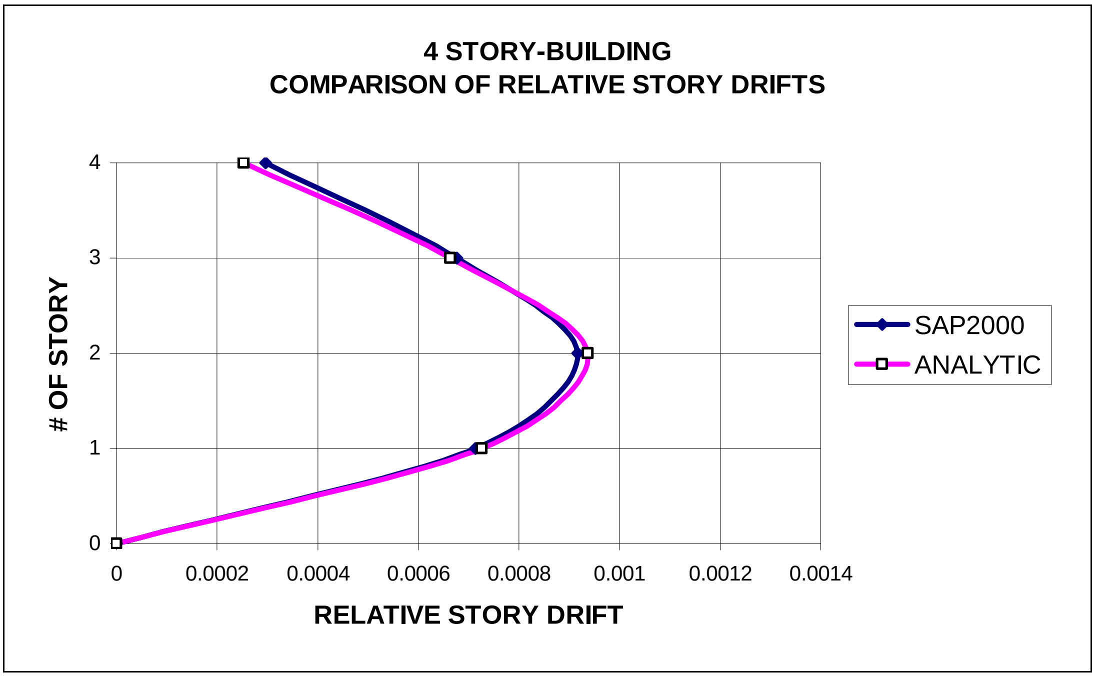

Figure 2.22 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 4 Story-Framed Structure)

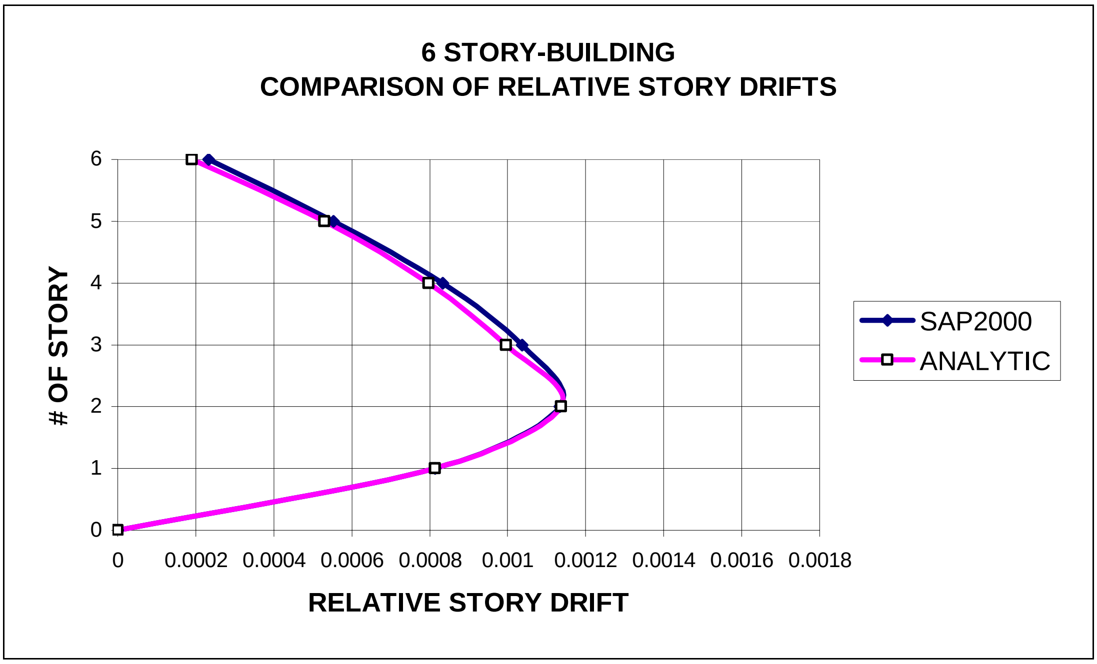

Figure 2.23 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 6 Story-Framed Structure)

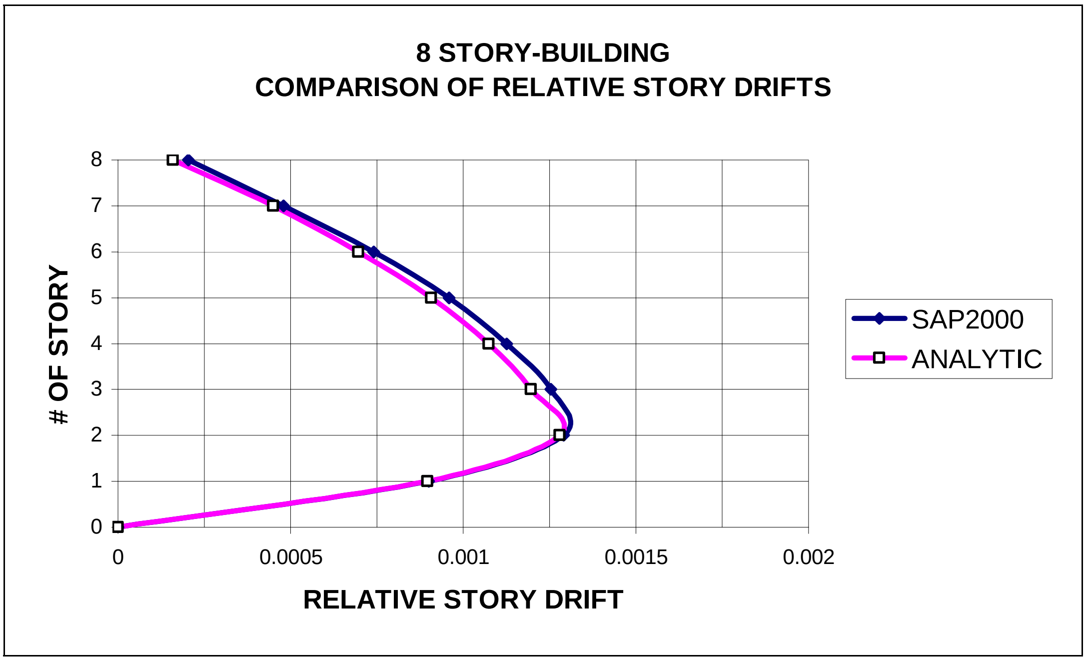

Figure 2.24 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 8 Story-Framed Structure)

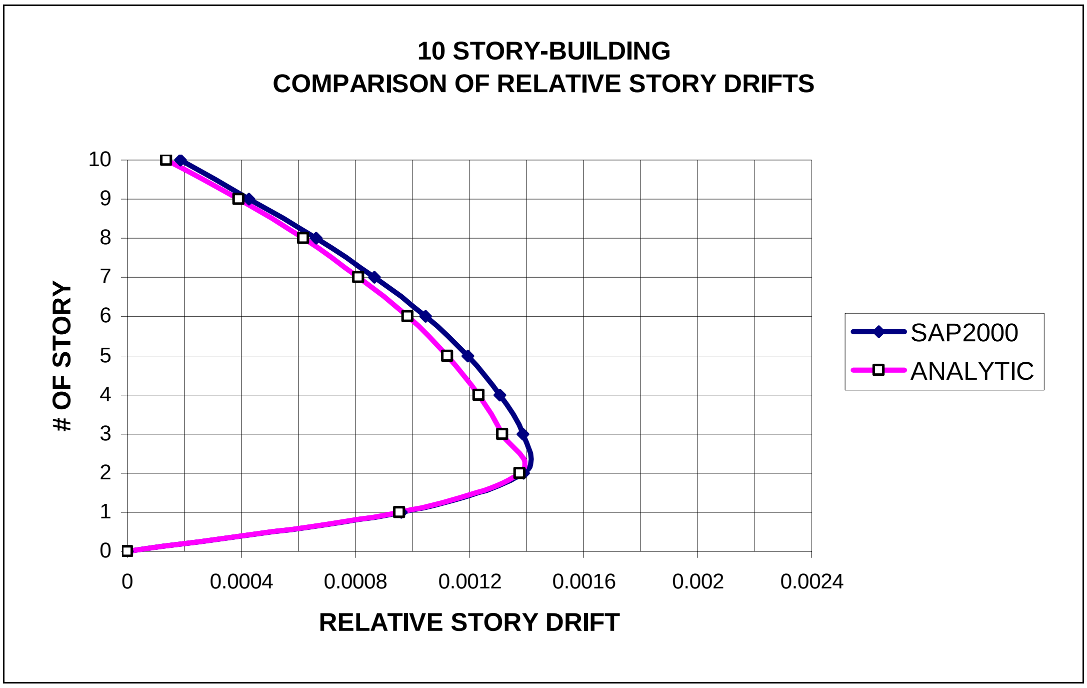

Figure 2.25 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 10 Story-Framed Structure)

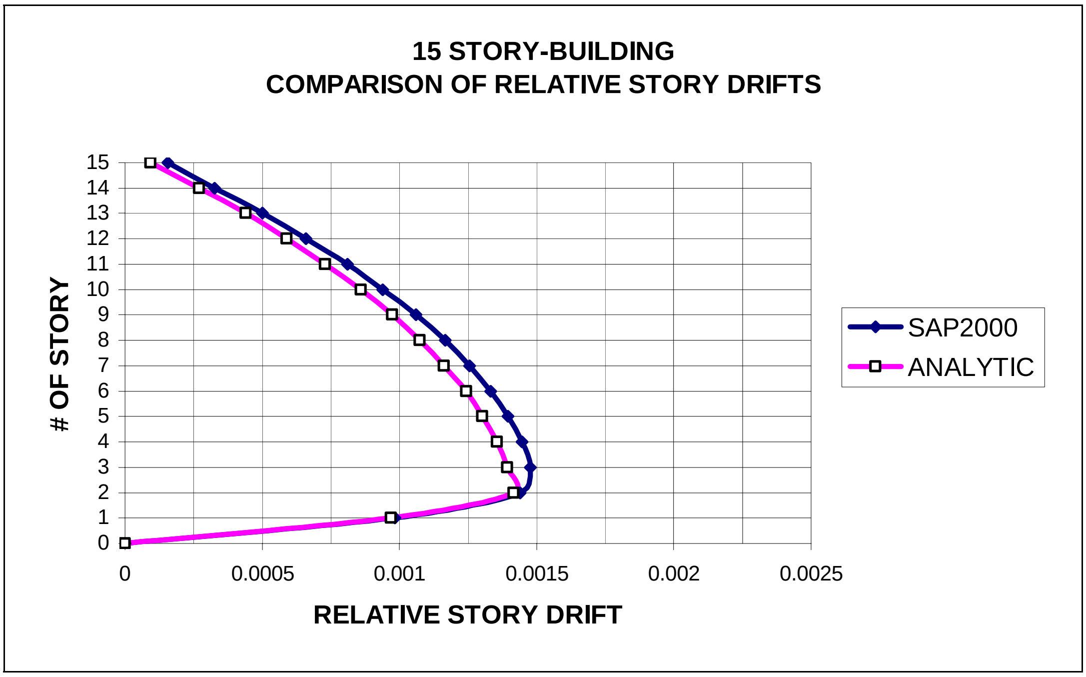

Figure 2.26 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 15 Story-Framed Structure)

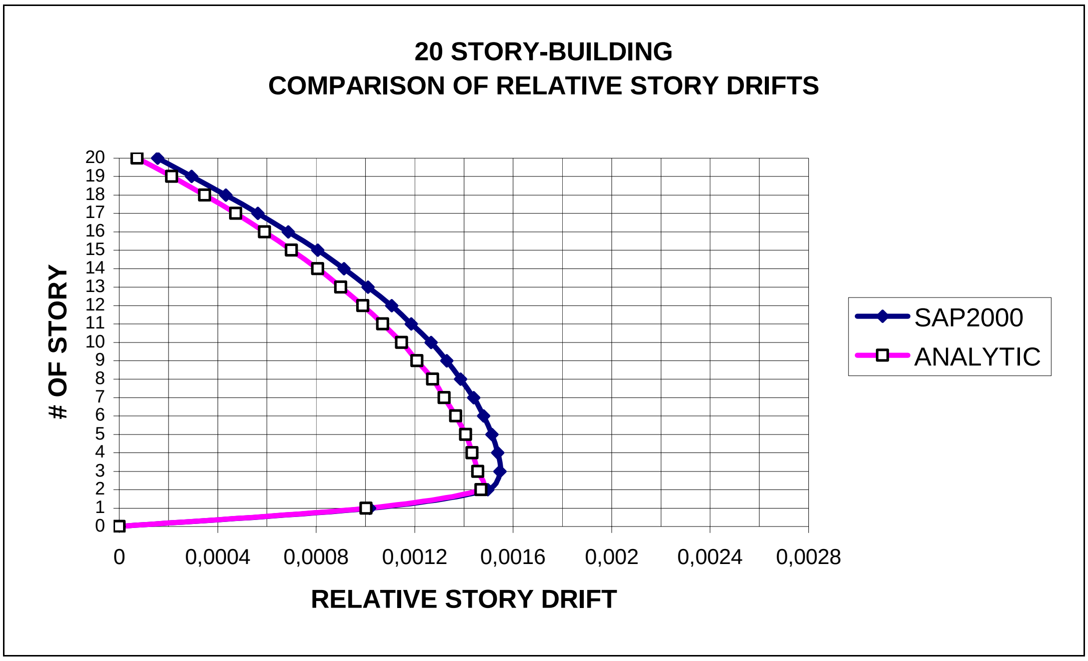

Figure 2.27 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 20 Story-Framed Structure)