QUANTIFYING SEISMIC DESIGN CRITERIA FOR CONCRETE BUILDINGS

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
THE MIDDLE EAST TECHNICAL UNIVERSITY

BY

AHMET TÜKEN

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF
DOCTOR OF PHILOSOPHY
IN
THE DEPARTMENT OF CIVIL ENGINEERING

MAY 2004

Approval of the Graduate School of Natural and Applied Sciences

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ABSTRACT

QUANTIFYING SEISMIC DESIGN CRITERIA FOR CONCRETE BUILDINGS

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May 2004, 242 pages

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 $\mu_{\Delta}=5$ 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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Mayıs 2004, 242 sayfa

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

ACKNOWLEDGEMENTS

I express my sincere appreciation to my supervisor Prof. Dr. Ergin Atımtay for his helpful guidance and insight throughout the research. Thanks also to the other faculty member, Prof. Dr. Engin Keyder, for his suggestions and comments. I also would like to convey my thanks to Assoc. Prof. Dr. Ali Ihsan Unay for his valuable help. The assistance of my laboratory friends, especially Ali Faik Ulusoy who wrote the executable Borland Delphi programs, is also gratefully acknowledged. I am also indebted to my dear friend, Mustafa Kaya, for his support during my studies. To my wife, Dilek, I offer sincere thanks for her unshakable faith in me and her willingness to endure with me the vicissitudes of my endeavors. To my children, Hüseyin Ekrem and Enes Ferid, I also thank them for understanding my frequent absence from home.

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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.

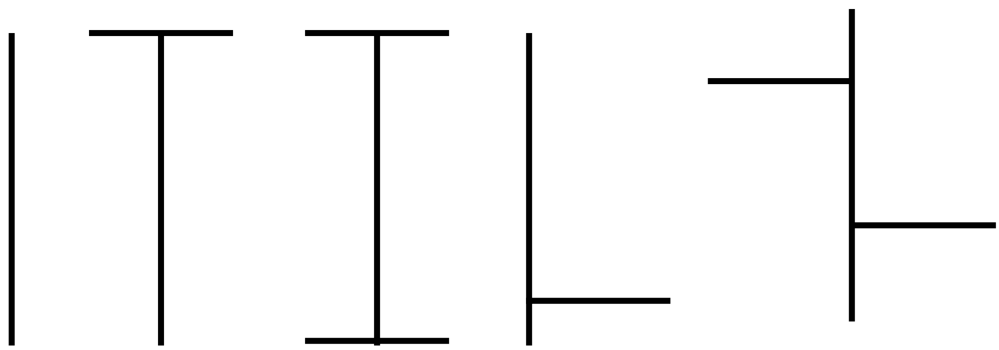


Figure1.1 Some Typical Shapes of Shear Walls

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.

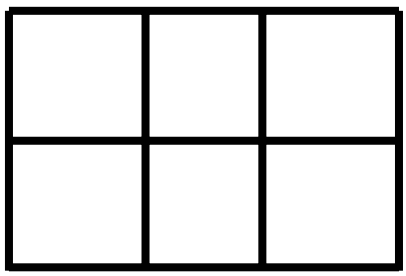


Figure1.2 A Shear Wall System without Openings

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.

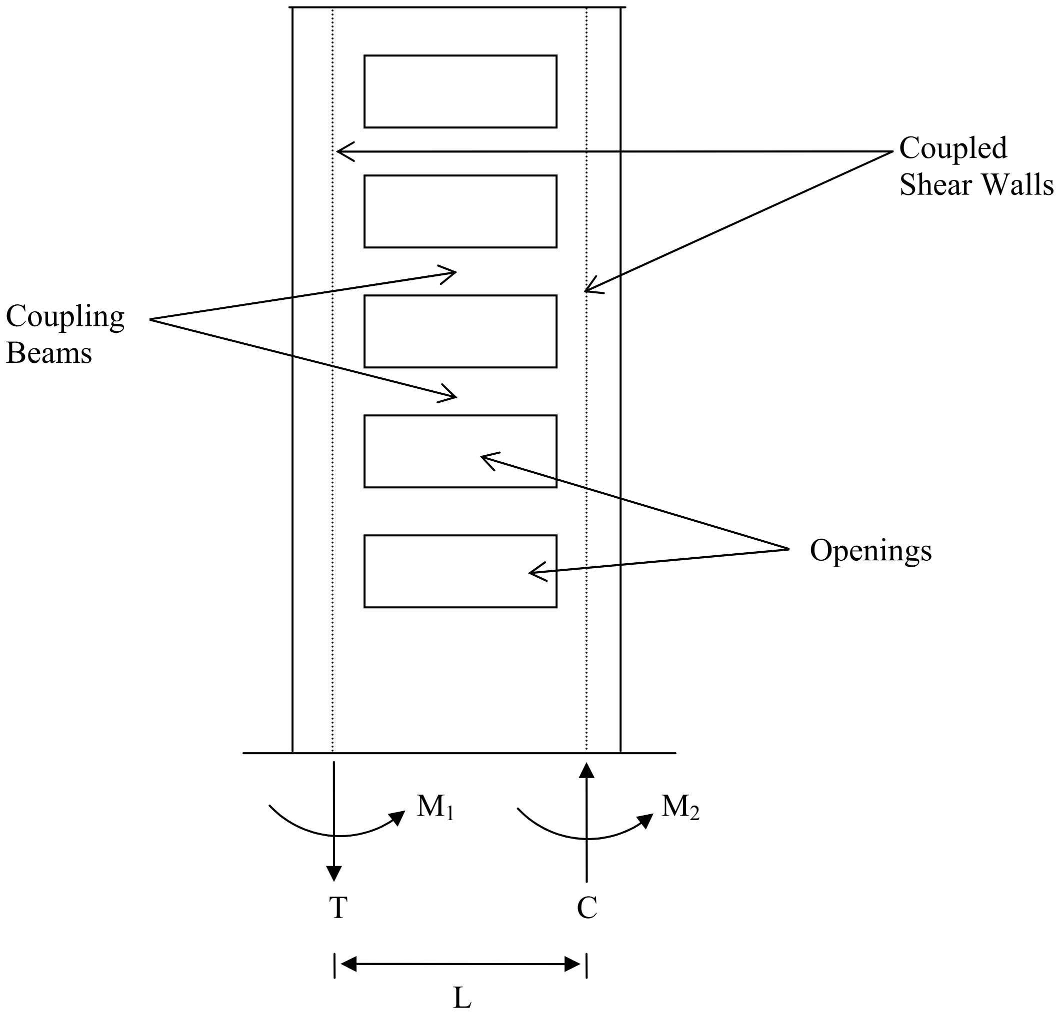


Figure1.3 A Shear Wall System with Openings (Coupled Shear Walls)

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.

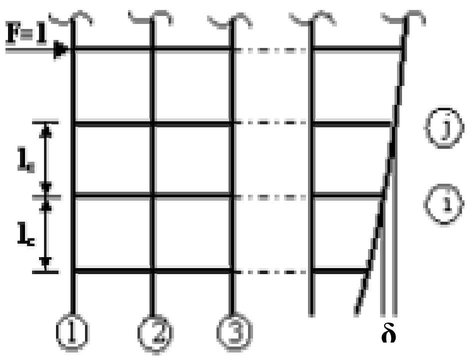


Figure 2.1 Framed Structure Subject to Lateral Load of F=1 and Relative Sway [2]

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

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

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.

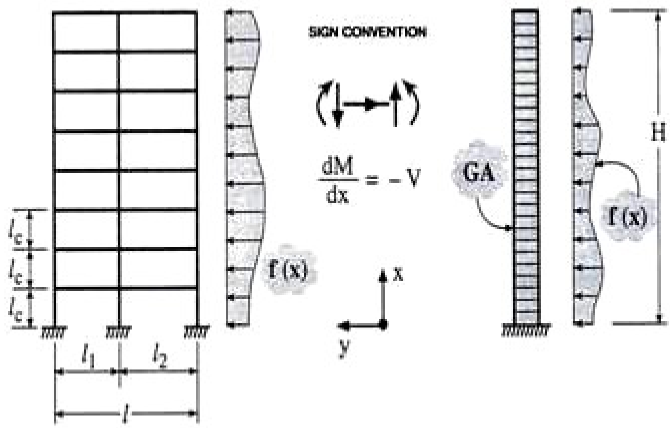


Figure 2.2 Continuous Shear Beam Model of Framed Structure [2]

The solution of the differential equation is as follows:

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

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].

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

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.

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

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.

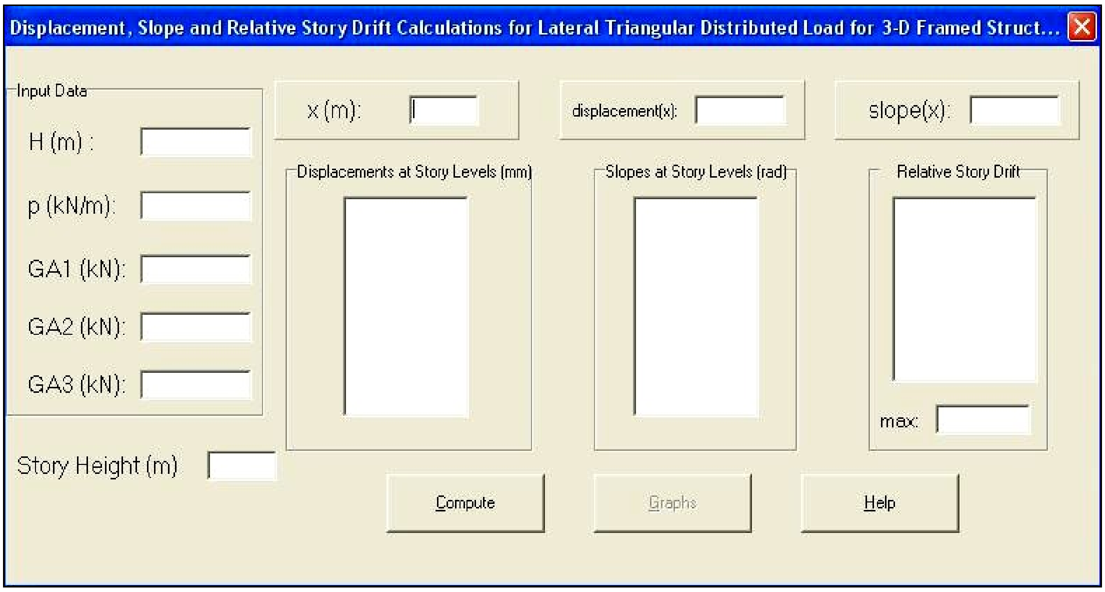


Figure 2.3 Executable "Borland Delphi" Program to Calculate Lateral Sway, Slope and Relative Story Drift for Framed Structures

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].

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

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.

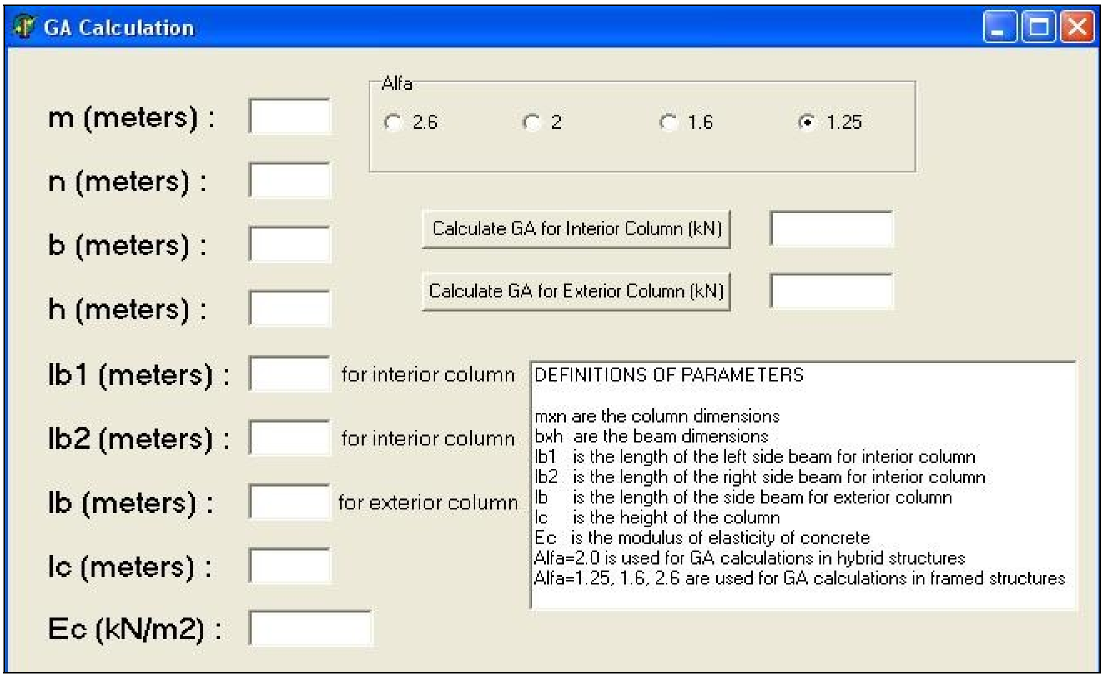


Figure 2.4 Executable "Borland Delphi" Program to Calculate GA

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

$$I_b = \alpha . \left[\frac{1}{12} bh^3 \right] \tag{2.10}$$

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.

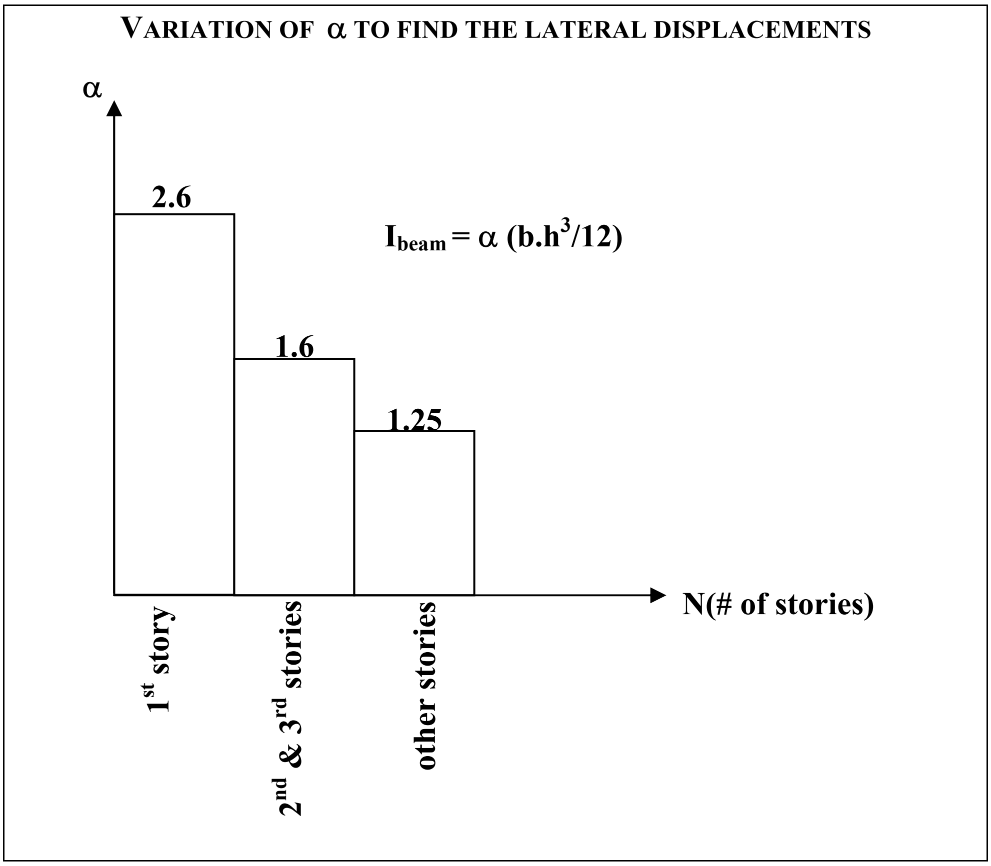


Figure 2.5 Expressing the Stiffness of Flanged Beam as a Multiple (α) of the Stiffness of Rectangular Beam

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.

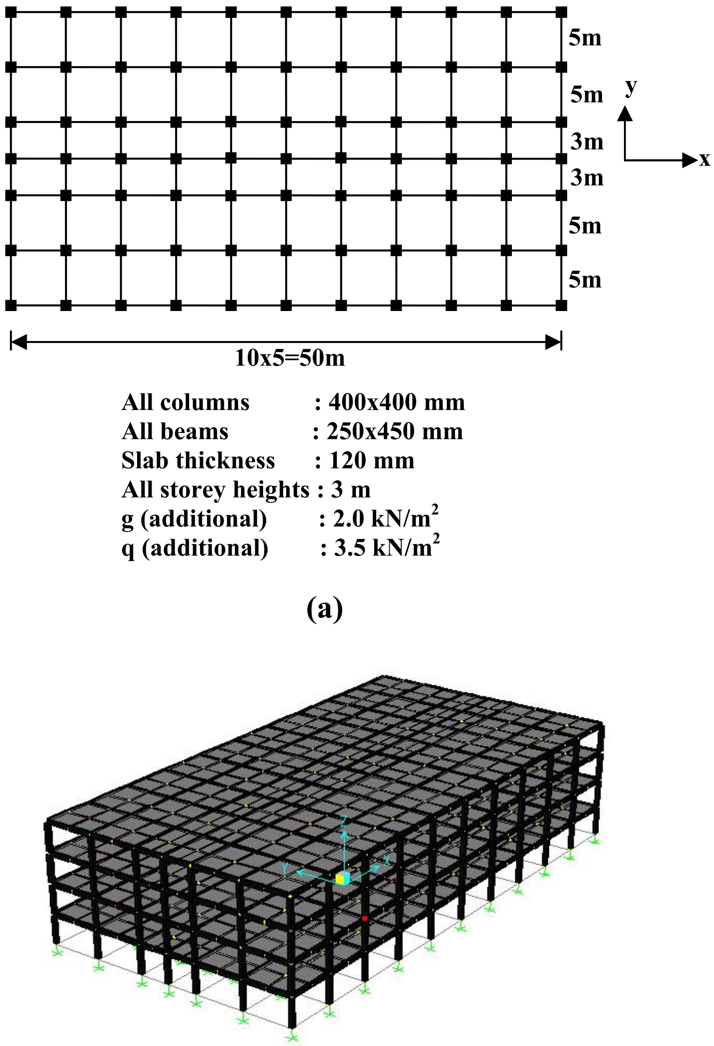


Figure 2.6 Framed Structure Used to Test the Validity of the Analytical Method: **(a)** Typical floor plan, **(b)** 3-D view of a sample 4-storey framed structure

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.

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

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

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

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

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

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

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

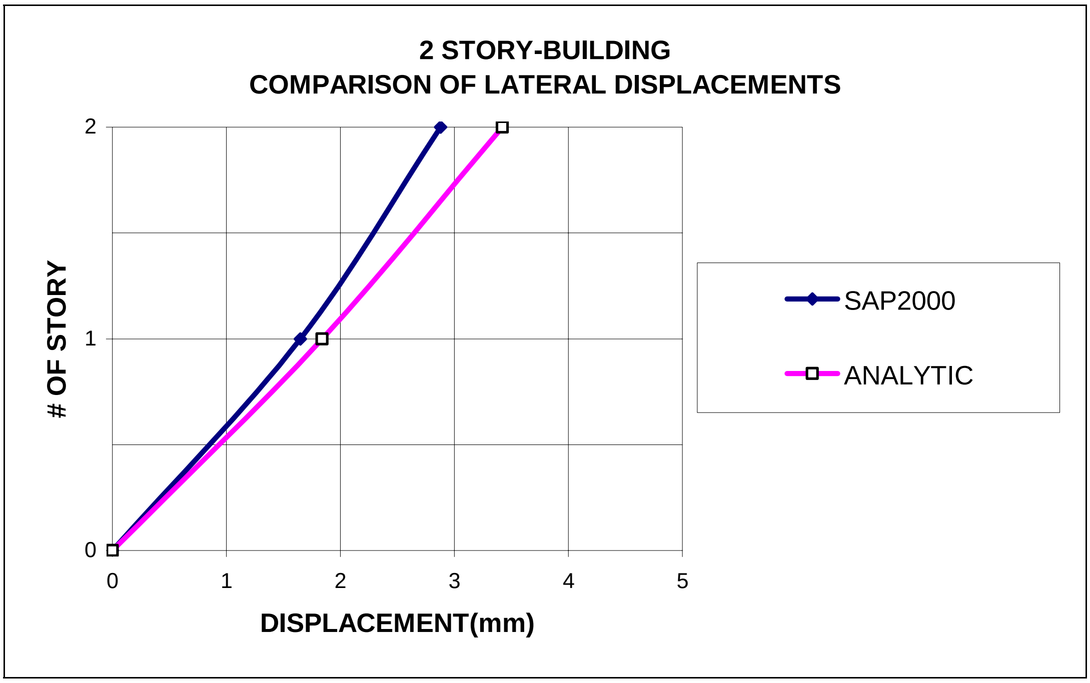


Figure 2.7 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 2 Story-Framed Structure)

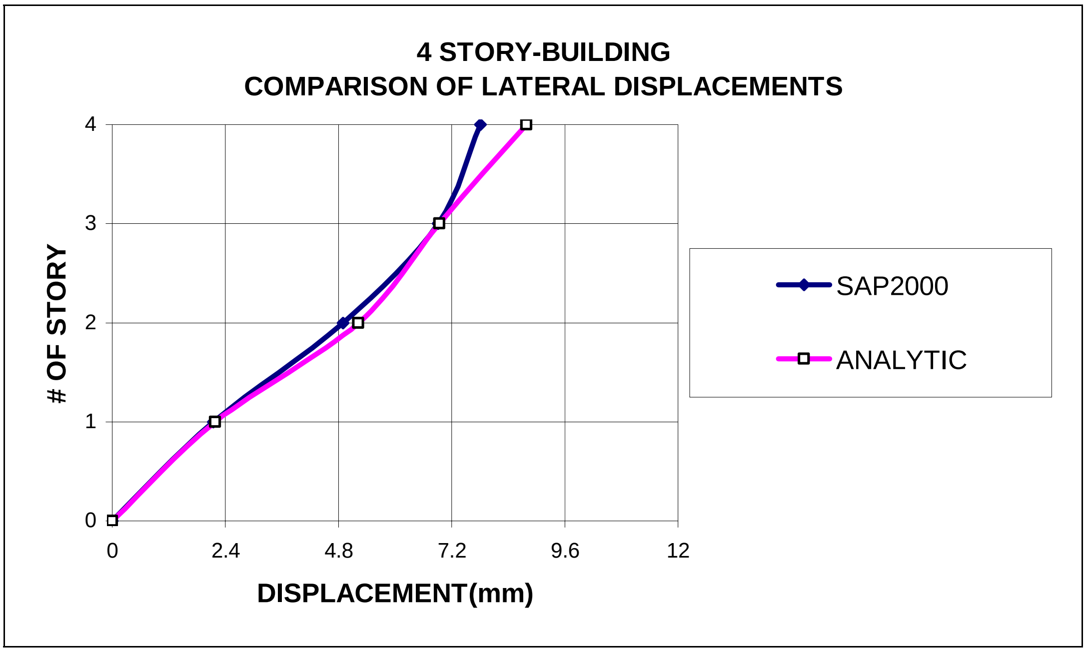


Figure 2.8 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 4 Story-Framed Structure)

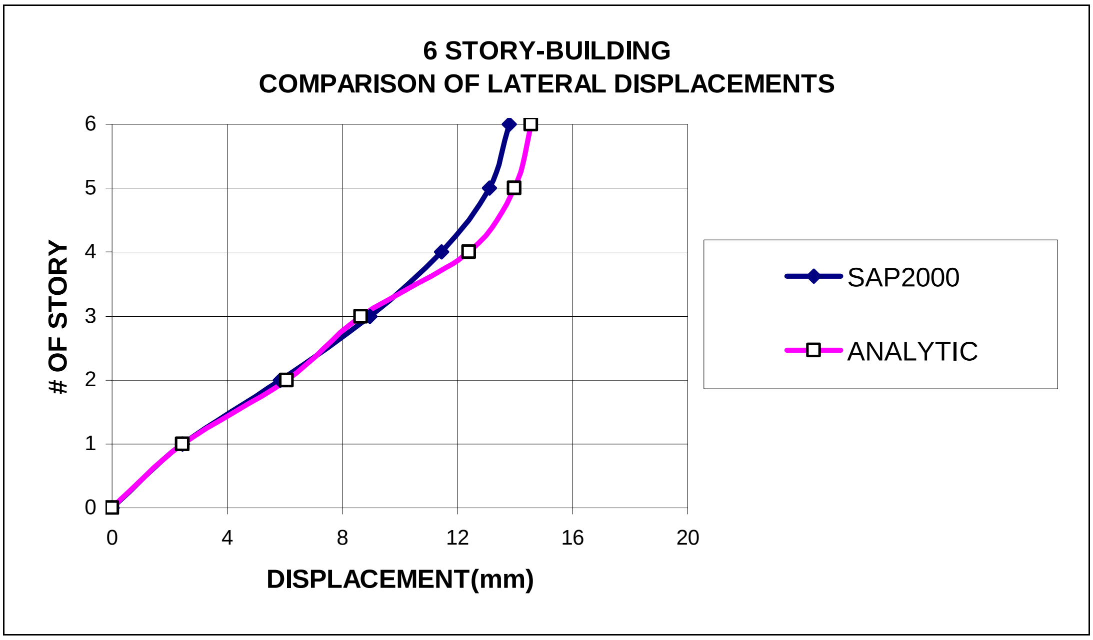


Figure 2.9 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 6 Story-Framed Structure)

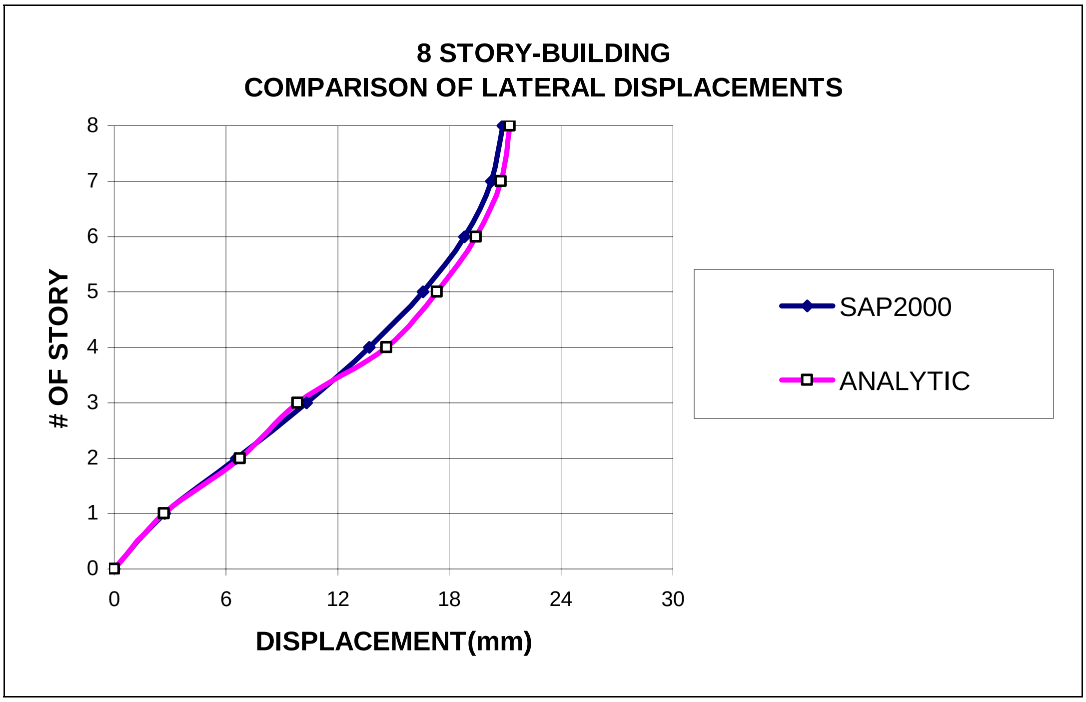


Figure 2.10 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 8 Story-Framed Structure)

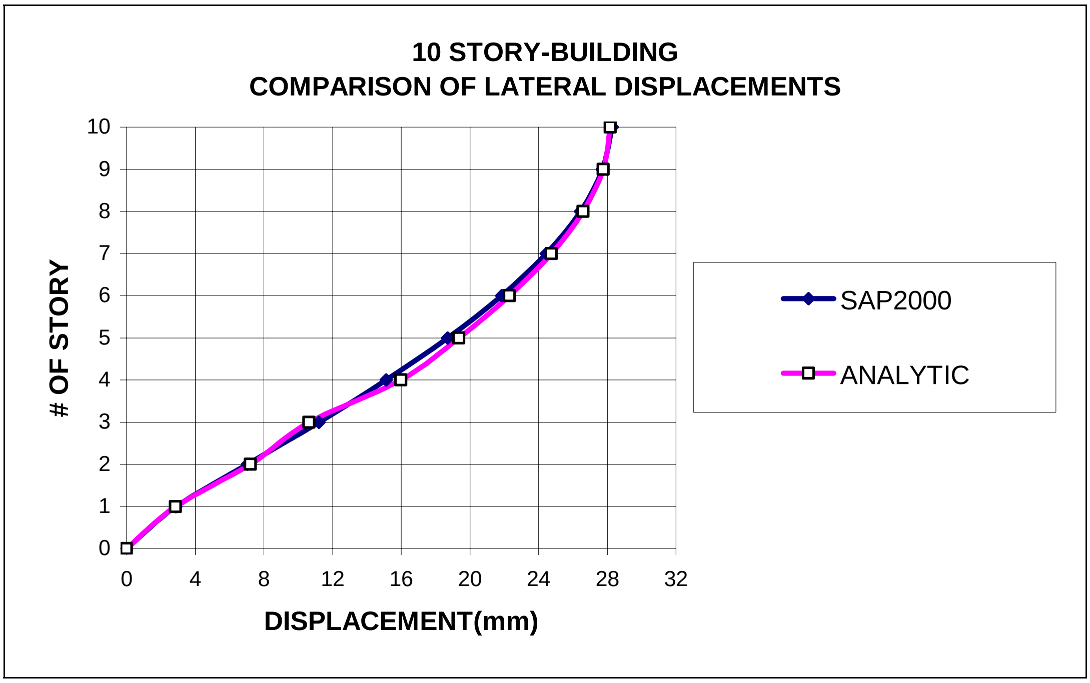


Figure 2.11 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 10 Story-Framed Structure)

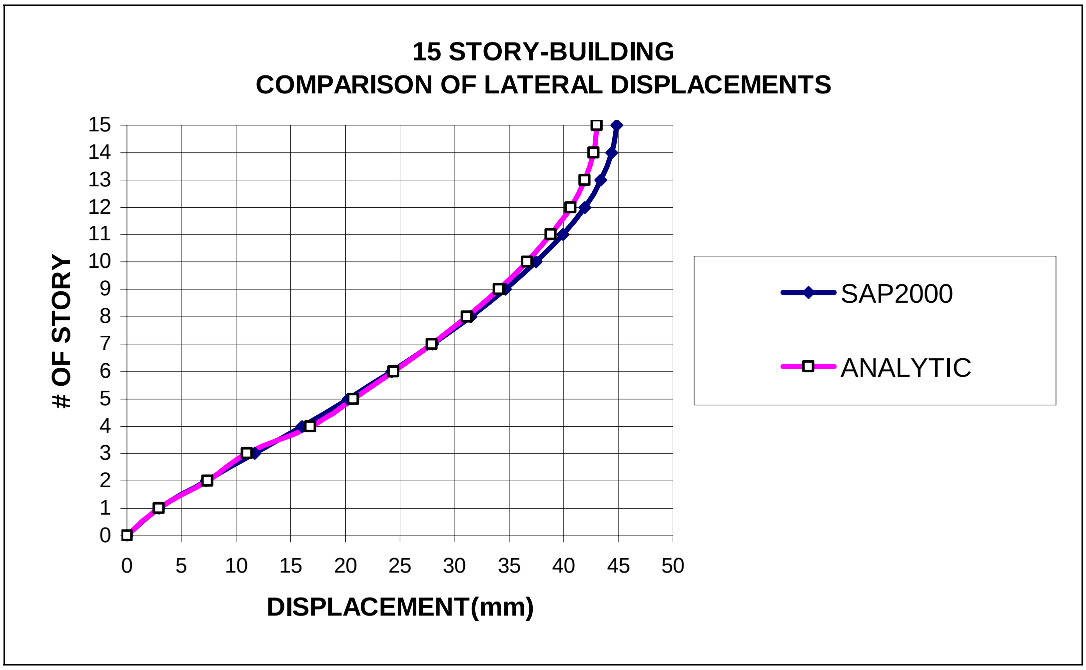


Figure 2.12 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 15 Story-Framed Structure)

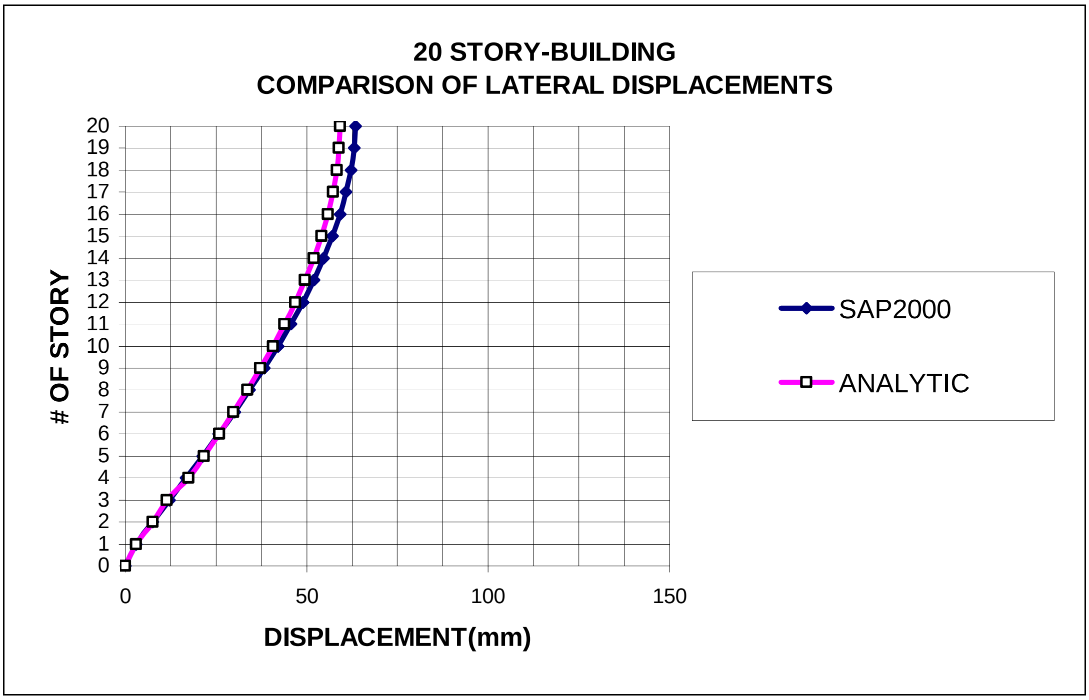


Figure 2.13 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 20 Story-Framed Structure)

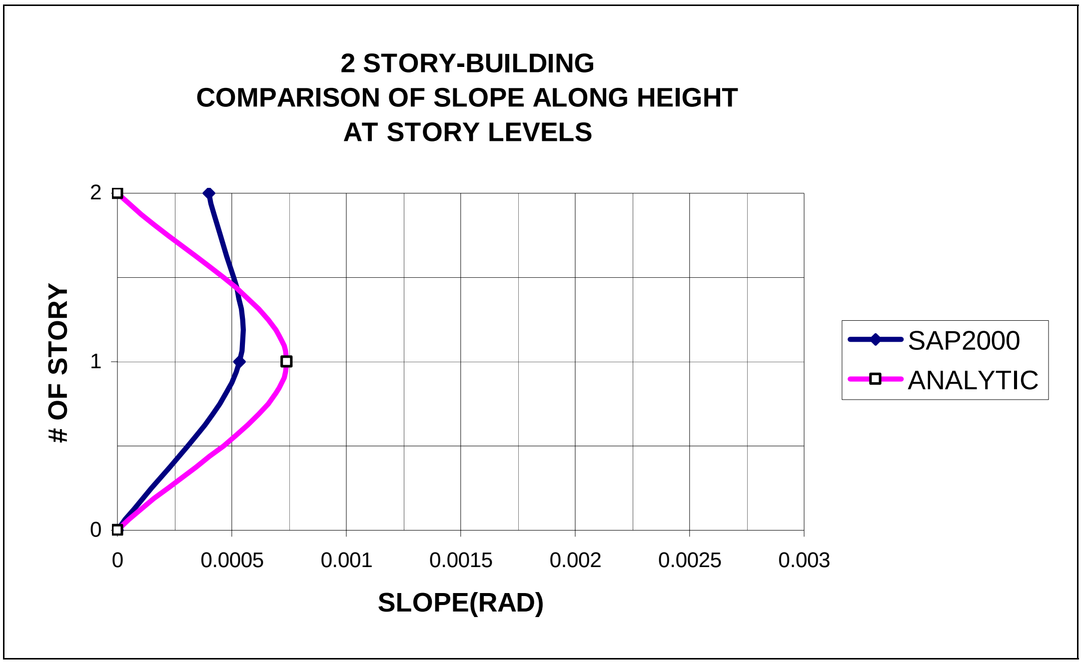


Figure 2.14 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 2 Story-Framed Structure)

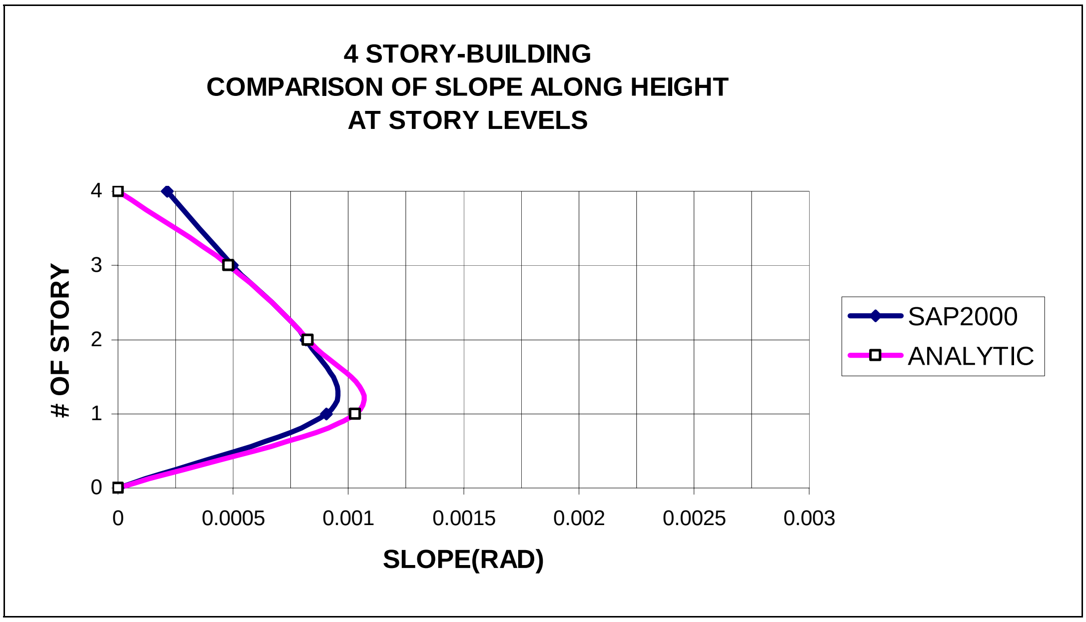


Figure 2.15 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 4 Story-Framed Structure)

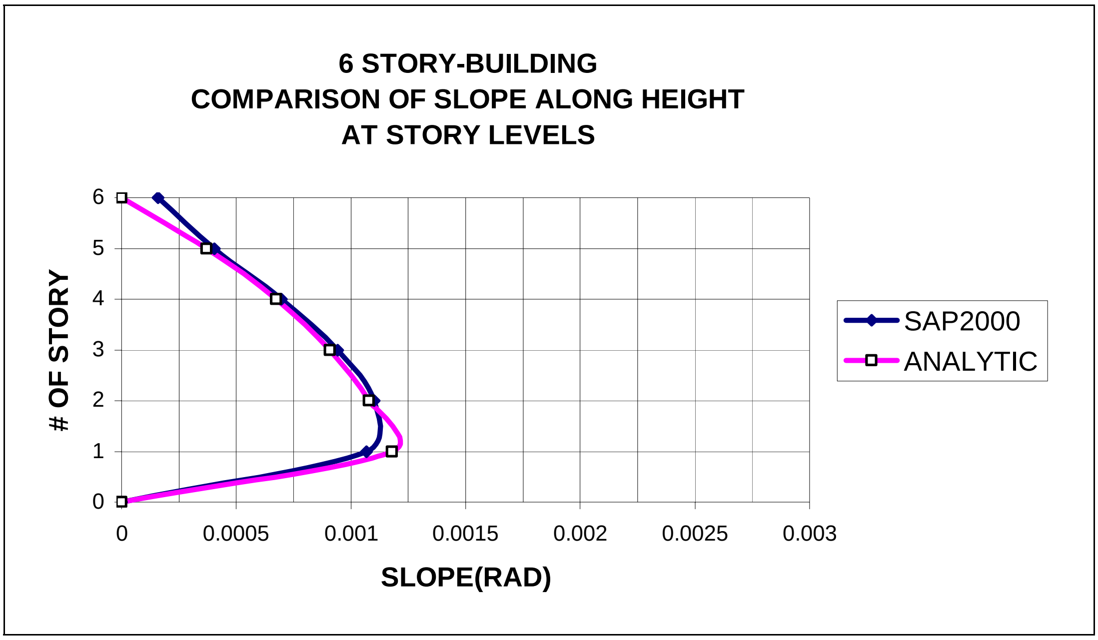


Figure 2.16 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 6 Story-Framed Structure)

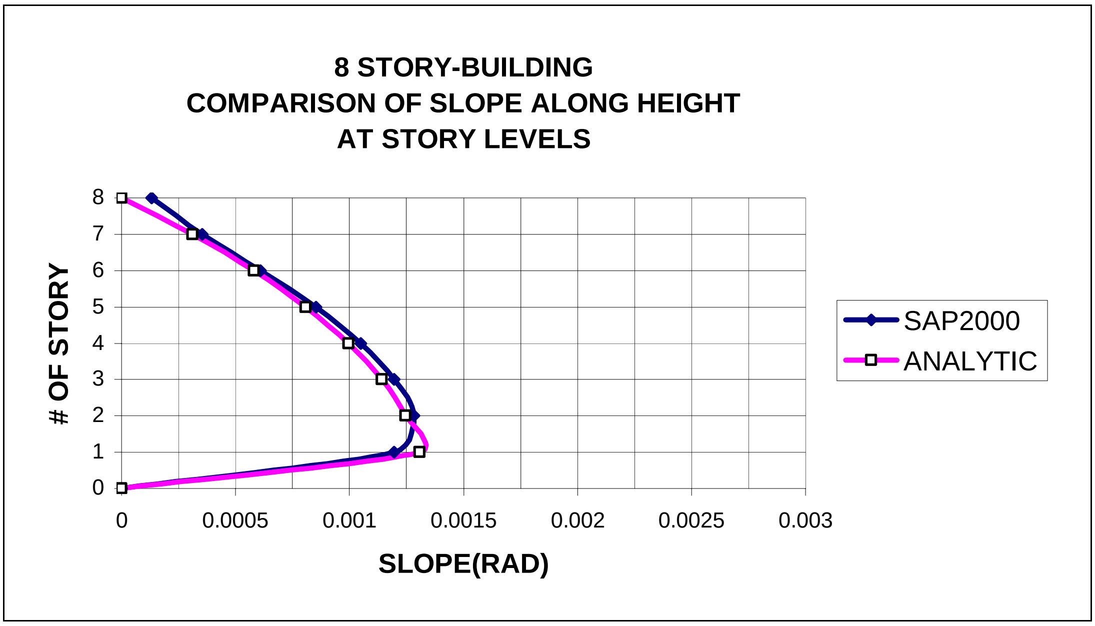


Figure 2.17 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 8 Story-Framed Structure)

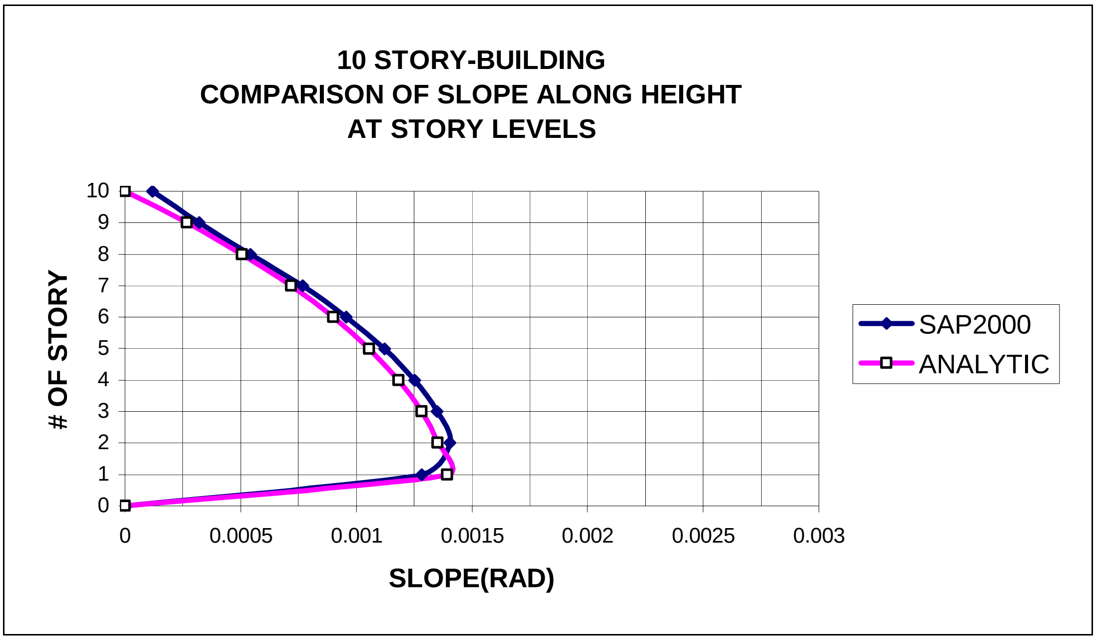


Figure 2.18 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 10 Story-Framed Structure)

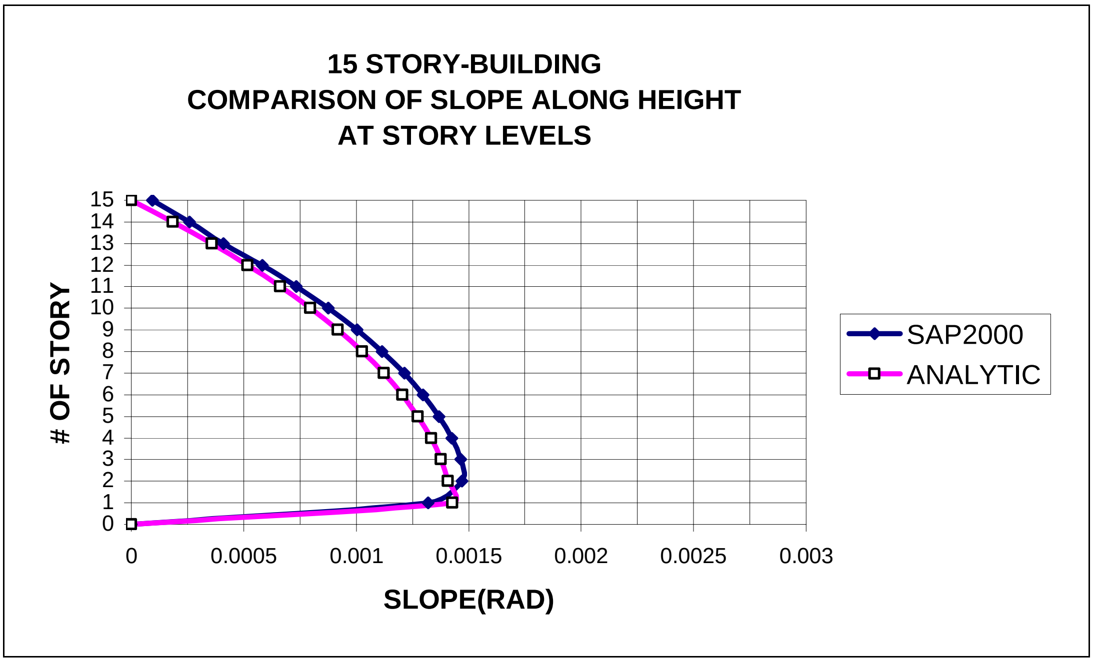


Figure 2.19 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 15 Story-Framed Structure)

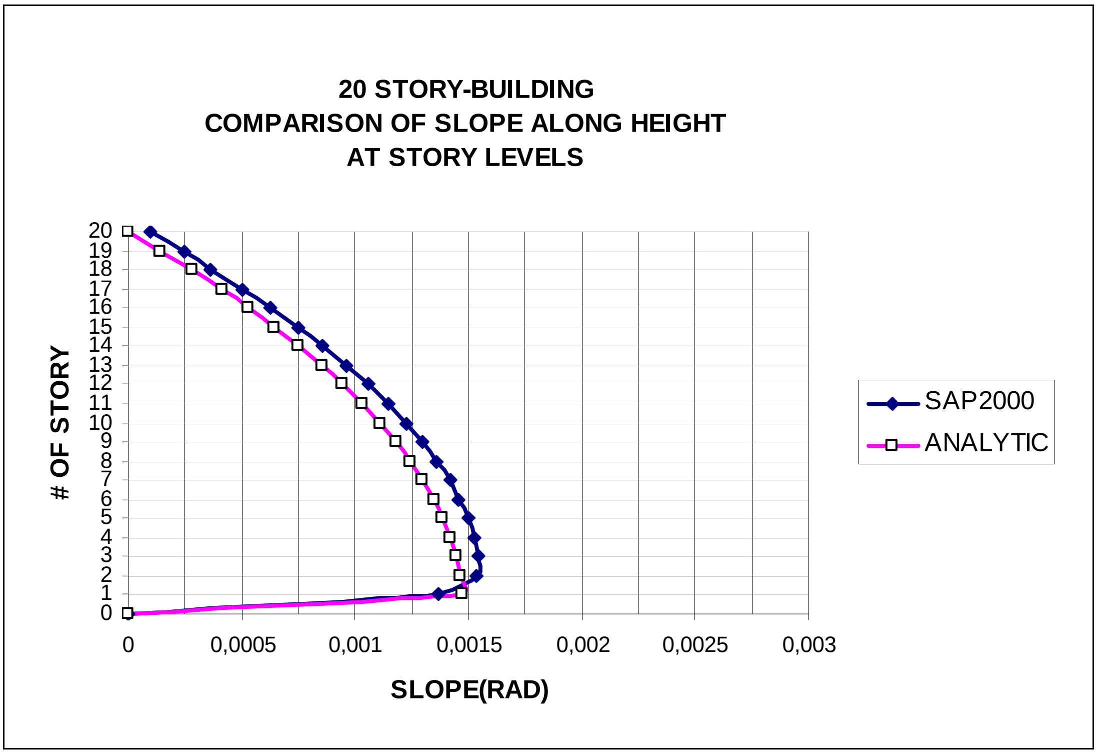


Figure 2.20 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 20 Story-Framed Structure)

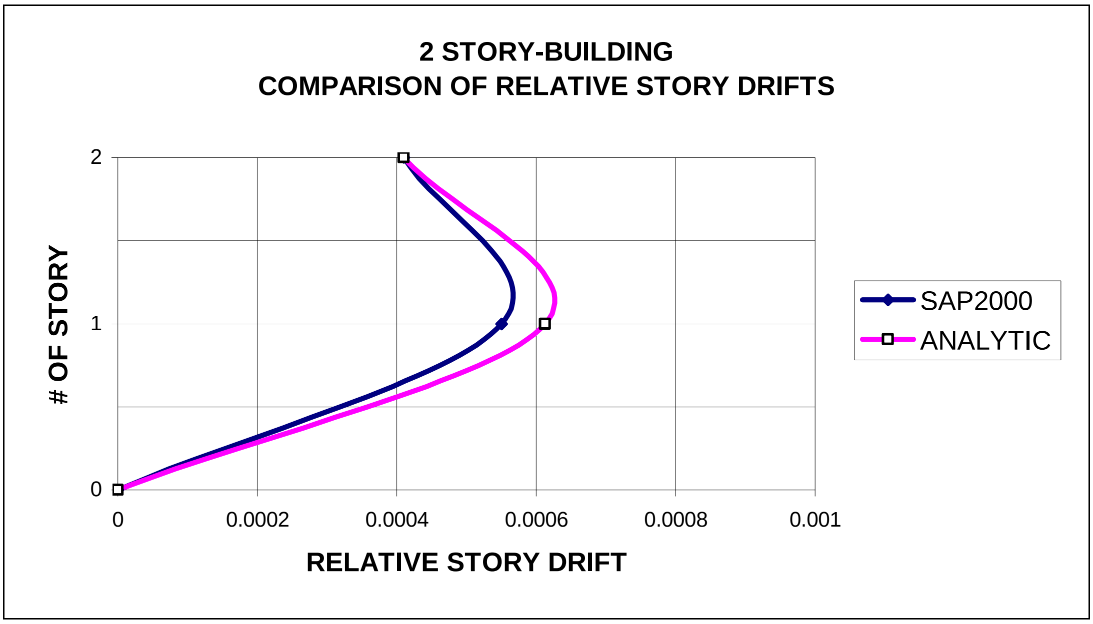


Figure 2.21 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 2 Story-Framed Structure)

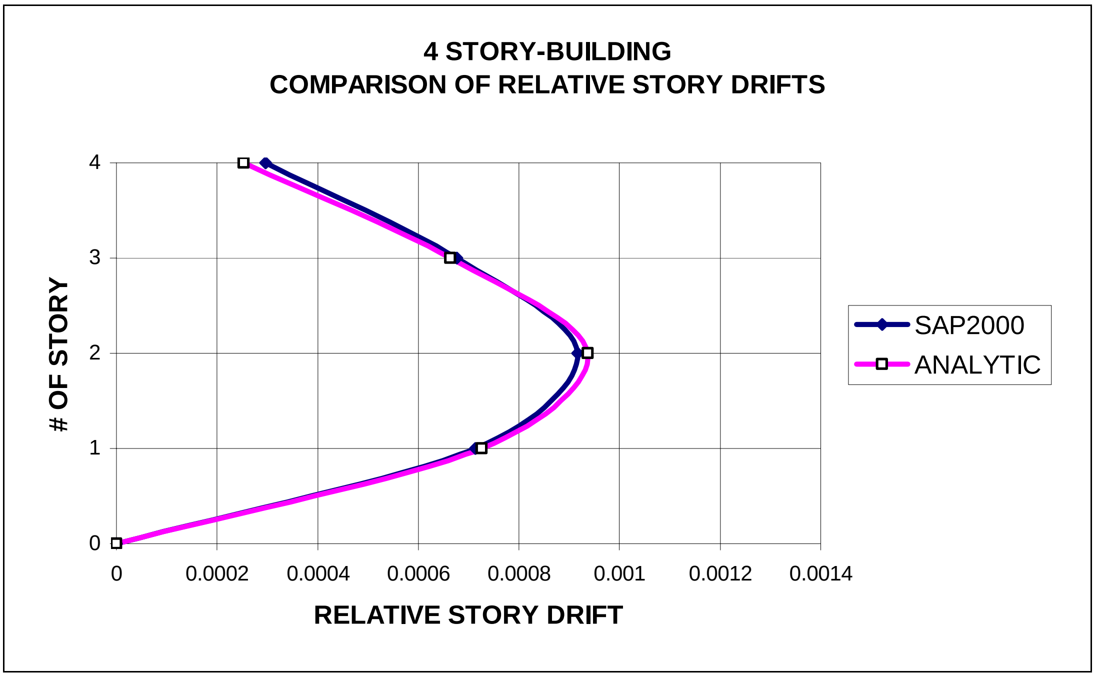


Figure 2.22 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 4 Story-Framed Structure)

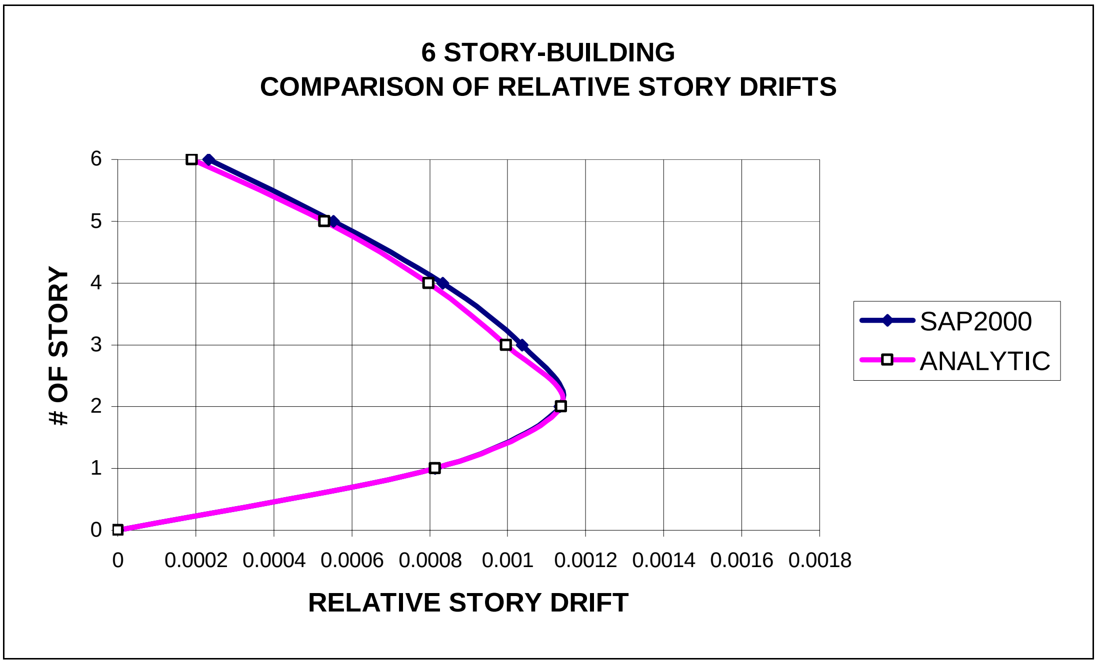


Figure 2.23 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 6 Story-Framed Structure)

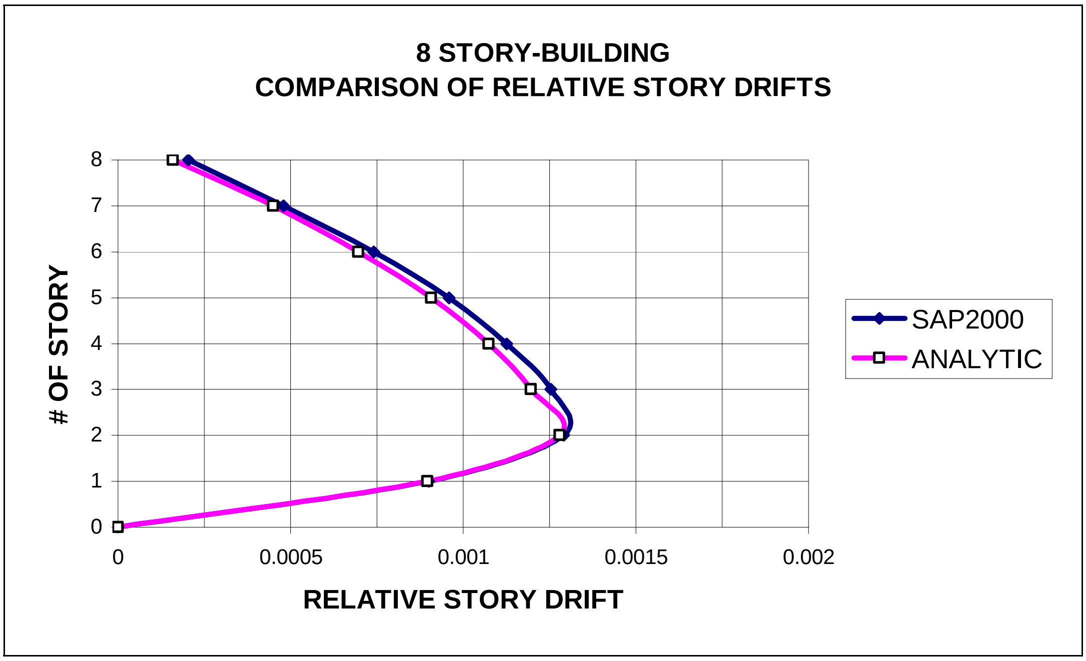


Figure 2.24 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 8 Story-Framed Structure)

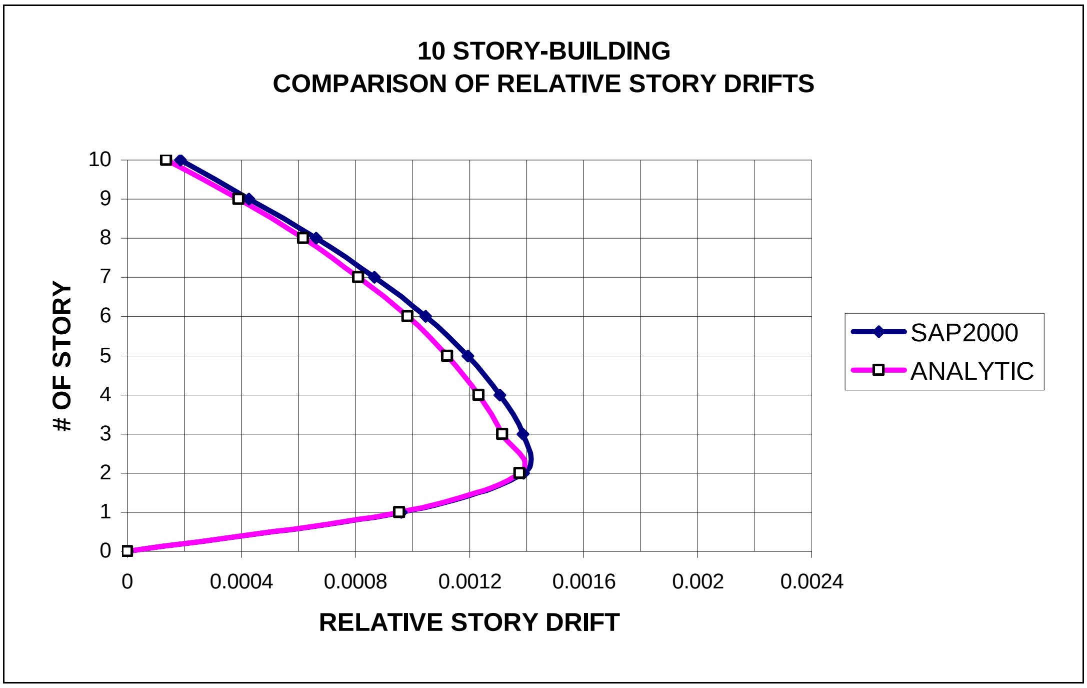


Figure 2.25 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 10 Story-Framed Structure)

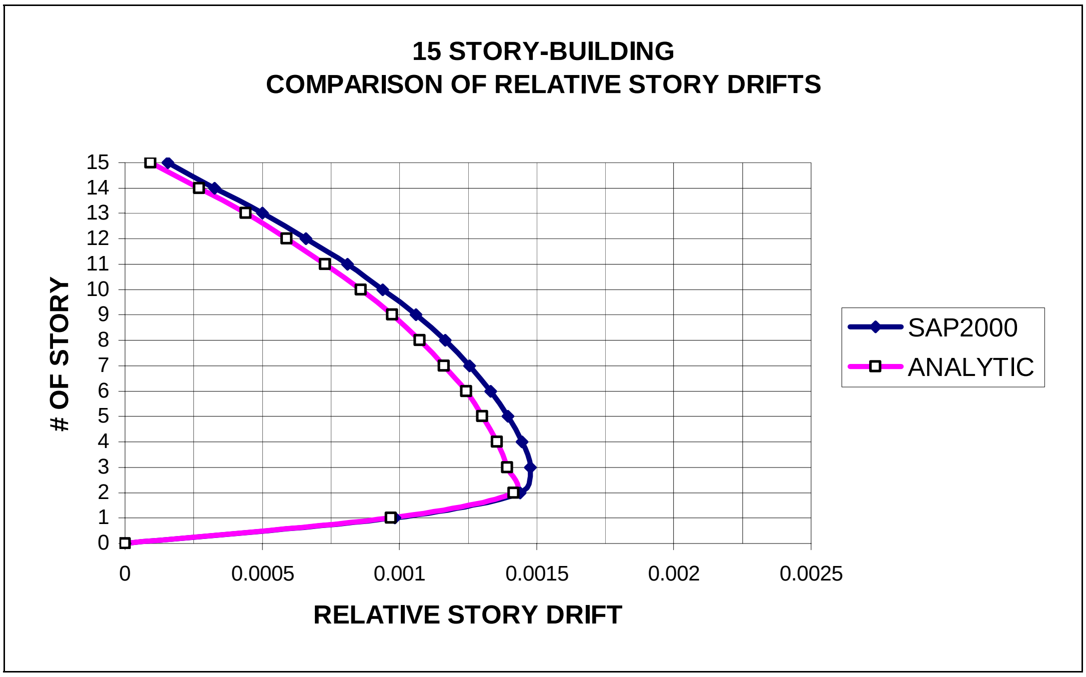


Figure 2.26 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 15 Story-Framed Structure)

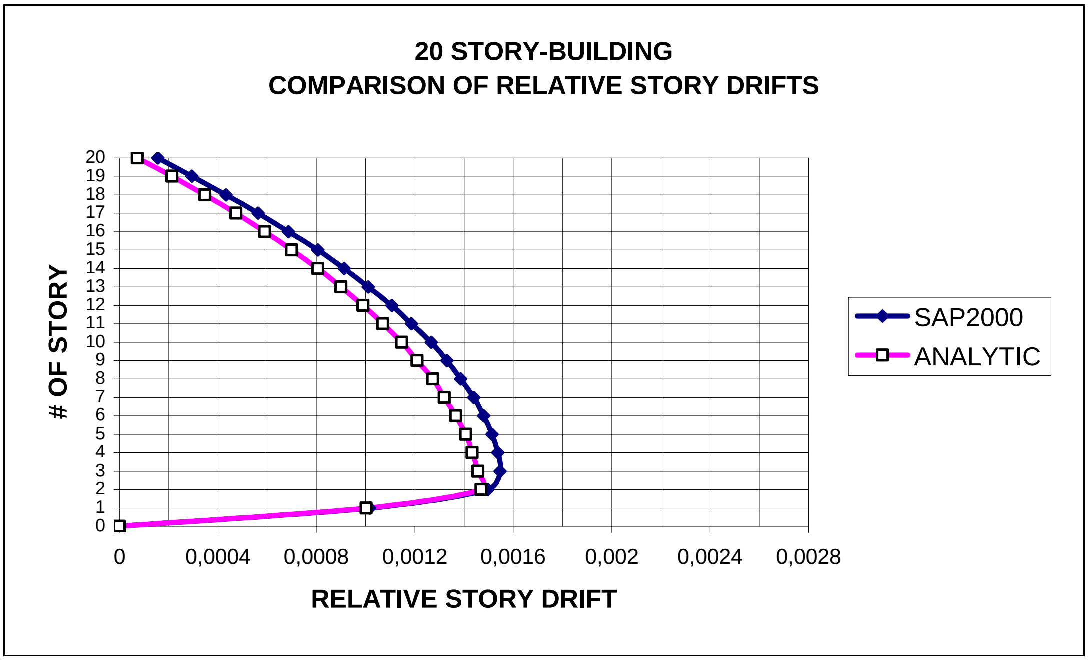


Figure 2.27 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 20 Story-Framed Structure)

CHAPTER 3

PROCEDURE FOR ANALYTICAL METHOD OF SEISMIC ANALYSIS OF MIXED STRUCTURES

3.1 ANALYTICAL MODEL OF MIXED STRUCTURE (FRAME + SHEAR WALL)

A shear wall subjected to a lateral load of $f(x)_p$ is shown in Figure3.1 (a). The lateral displacement of the shear wall under this $f(x)_p$ loading will be $y(x)_p$. As noticed that the shear wall displays as a flexural beam. The flexural rigidity of the shear wall can be defined as EI, where E is the modulus of elasticity and I is the moment of inertia.

A frame subjected to a lateral load of $f(x)_ç$ is shown in Figure3.1 (b). The lateral displacement of the frame under this $f(x)_ç$ loading will be $y(x)_ç$. As noticed that the frame displays as a shear beam. The shear rigidity of the frame per unit height can be defined as GA, where G is the shear modulus and A is the area.

A shear wall-frame interaction subjected to a lateral load of $f(x) = f(x)_p + f(x)_ç$ is shown in Figure3.1 (c). The lateral displacement of the shear wall-frame interaction under this f(x) loading will be y(x).

The relationships between moment, shear wall and loading function are given in Figure3.1 (d) according to the defined axis.

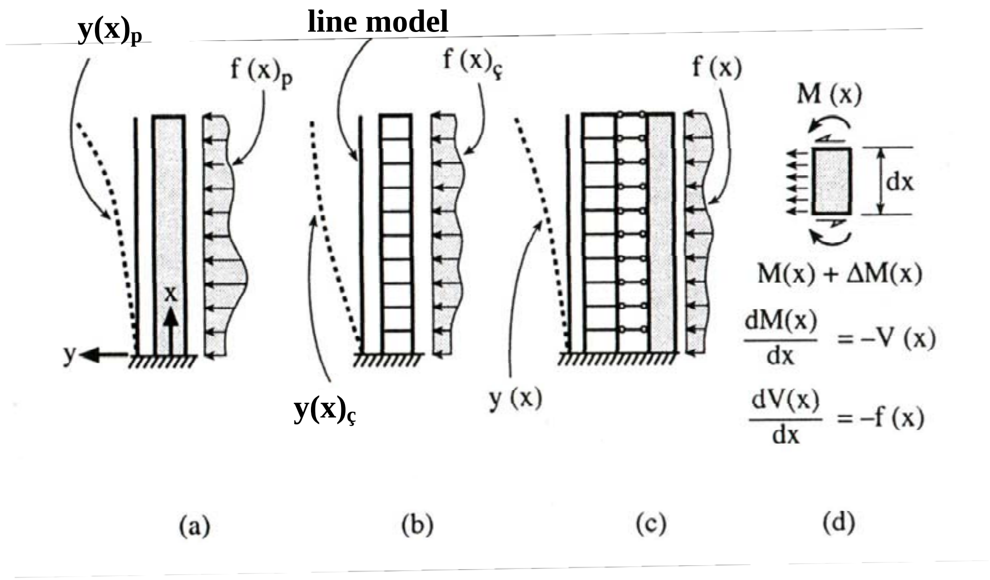


Figure 3.1 Mathematical Model of Shear Wall-Frame Interaction [2]

3.2 GENERAL SOLUTION OF MIXED STRUCTURES

The general differential equation of the flexural beam shown in Figure3.1 (a) can be written as in Eqn.3.1 [68, 70].

$$K.\,(y)^{IV} = f(x) \tag{3.1}$$

where

K= Σ EI = Total stiffness of all shear walls within the story

f(x)= distributed lateral force

The general differential equation of the shear beam shown in Figure3.1 (b) can also be written as in Eqn.3.2.

$$GA\left(y'' + \frac{K.y'' - M(x)}{K_0}\right) = -f(x) \tag{3.2}$$

where

$K.y''$ - M(x) = Moment taken by the frame

M(x) = Moment caused by the external loads at height x

K. y'' = Moment taken by the shear walls

$[K.y^{''} - M(x)] / K_0$= Unit rotation due to axial deformations of frame columns (This rotation forces the frame to display more)

The differential equation of flexural-shear beam (i.e. shear wall – frame) shown in Figure3.1 (c) can be easily written as in Eqn.3.3 & Eqn.3.4.

$$K.y^{IV} - GA\left(y^{II} + \frac{K.y^{II} - M(x)}{K_0} \right) = f(x) \qquad (3.3)$$

$$K.y^{IV} - GA.v^2.y^{II} + \frac{GA.M(x)}{K_0} - f(x) = 0 \qquad (3.4)$$

where

$v^2 = 1 + K / K_0$

The following assumptions were made while writing Eqn.3.3 & Eqn.3.4.

i. Floor slabs were assumed rigid in their own plane. In other words, all vertical elements (shear walls and columns) within the story have the same lateral displacement (i.e. property of diaphragm effect)

ii. The shear rigidity of the frames was assumed to be calculated by considering only slabs, columns and beams on the floor levels and then this calculated shear rigidity for any story was distributed equally within that story height, therefore a continuous medium was obtained. As the number of story increases this process becomes more reliable.

iii. The effects of axial forces at floor beams and vertical structural elements were neglected since this effect is small enough to be neglected in reality.

iv. The unit deformations caused by the shear strength of shear walls were neglected.

Letting w= K.y in Eqn.4.4, the differential equation of lateral displacement, factored by K multiplier, can be obtained as expressed in Eqn.3.5.

$$s^2.w^{IV} - w^{II} + \frac{v^2 - 1}{v^2}.M(x) - s^2.f(x) = 0 \qquad (3.5)$$

where

$s^2 = K / (v^2.GA)$

K= K (shear walls) + Σ K (columns)

In shear wall-frame structures, since the stiffness of columns is negligibly small when compared to that of shear walls, Σ K (columns) term can be neglected.

Bending moment expression can be easily found by differentiating the equation of w= K.y twice. Therefore, the degree of general differential equation of bending is reduced to two as expressed in Eqn.3.6.

$$s^2.M''(x) - M(x) + \frac{v^2 - 1}{v^2}.M(x) - s^2.f(x) = 0 \tag{3.6}$$

The solution of this differential equation can be expressed as the summation of two separate solutions, one is particular solution and the other one is complementary solution.

For the complementary solution;

$$s^2.M''(x) - M(x) = 0 \tag{3.7}$$

$$M(x)_{complementary} = A_1.\cosh\Phi + A_2.\sinh\Phi \tag{3.8}$$

where

$\Phi = x / s$

For the particular solution;

$$(s^2.D^2 - 1).M(x)_{particular} = s^2.f(x) - \frac{v^2 - 1}{v^2}.M(x) \tag{3.9}$$

where

$D^2 = d^2 / dx^2$

$$M(x)_{particular} = \frac{1}{s^2.D^2 - 1}.\left(s^2.f(x) - \frac{v^2 - 1}{v^2}.M(x) \right) \tag{3.10}$$

$$M(x)_{particular} = -(1 + s^2.D^2).\left(s^2.f(x) - \frac{v^2 - 1}{v^2}.M(x) \right) \tag{3.11}$$

Now, the solution of the differential equation given in Eqn.4.6 can be expressed as in Eqn.3.12 & Eqn.3.13.

$$M(x) = M(x)_{complementary} + M(x)_{particular} \tag{3.12}$$

$$M(x) = A_1.\cosh\varphi + A_2.\sinh\varphi - (1 + s^2.D^2).\left(s^2.f(x) - \frac{v^2 - 1}{v^2}.M(x) \right) \tag{3.13}$$

To find A_1 and A_2, two boundary conditions must be applied as follows;

i. The shear force at the base level must be equal to the total lateral load as expressed in Eqn.3.14.

$$M^{1}(0) = -\int_{0}^{H} f(x)dx \qquad (3.14)$$

ii. Moment at the top of the structure must be zero as expressed in Eqn.3.15.

$$M(H) = 0 \qquad (3.15)$$

Recalling w= K.y, Eqn.3.13 must be integrated twice and two more boundary conditions must be applied in order to find the equation of lateral displacement , w, as expressed in Eqn.3.16

$$w = \int_{0}^{x}\int M(x)dx^{2} + A_{3}.x + A_{4} \qquad (3.16)$$

To find A_3 and A_4, two additional boundary conditions can be as follows;

i. Displacement must be zero at the base level of the structure, i.e. w (0) = 0

ii. Slope must be zero at the base level of the structure due to the fixed base assumption, i.e. $w^{'}(0) = 0$

The shear force distribution along the height of the shear wall can be found by differentiating the moment equation with respect to x, as expressed in Eqn.3.17

$$V_{p} = -\frac{dM(x)}{dx} \qquad (3.17)$$

The shear force carried by the frame, $V(x)_ç$, can be found by subtracting the shear force taken by the shear wall, $V(x)_p$, from the total shear force, V(x) as expressed in Eqn.3.18

$$V(x)_{ç} = V(x) - V(x)_{p} \qquad (3.18)$$

In the design of columns of the frame, the maximum shear force carried by the columns must be known. In order to find where the maximum shear force is along the height of the structure, the derivative of $V(x)_ç$ with respect to x must be taken and solved for x as expressed in Eqn.3.19.

$$\frac{dV(x)_{ç}}{dx} = 0 \qquad (3.19)$$

In the most exterior columns of the frame, the axial column forces caused by the lateral forces can be found by dividing the moment taken by the frame, $M(x)_ç$, to the spacing between columns, b, as expressed in Eqn.3.20.

$$N(x) = \frac{M(x) - M(x)_p}{b} = \frac{M(x)_ç}{b} \tag{3.20}$$

where

M(x) = Total moment at height x due to external loads

N(x) = Axial force at height x due to overturning moment of the frame

b = Spacing between the most exterior columns

3.3 SOLUTION OF MIXED STRUCTURES FOR TRIANGULAR LATERAL LOAD

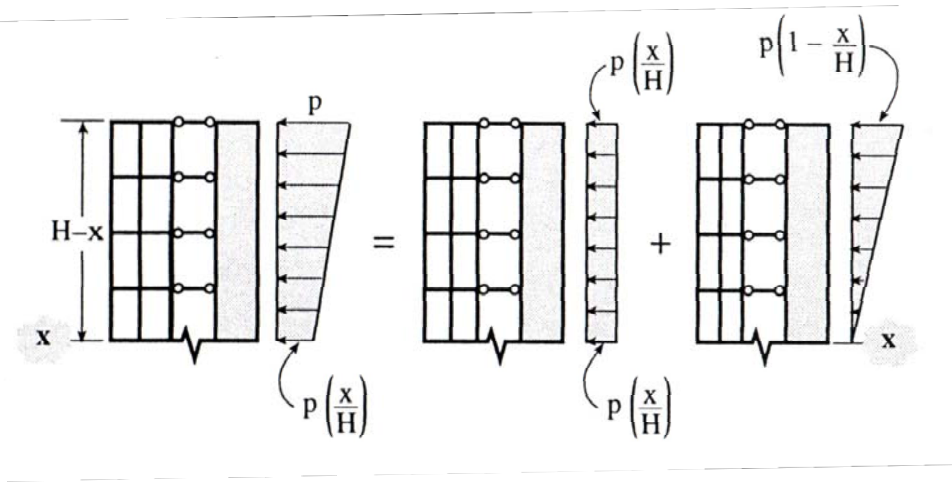


Figure 3.2 Moment at Height x of the Structure for Triangular Distributed Lateral Load [2]

For triangular distributed lateral load of f (x) = p. (x / H) = p. (k), Eqn.3.13 will take the following form.

$$M(x) = A_1.\cosh\varphi + A_2.\sinh\varphi + \frac{v^2 - 1}{v^2}.M(x) - \frac{s^2}{v^2}.p.(k) \tag{3.21}$$

The calculation of M(x) in Eqn.3.21 is shown in Figure 3.2 and can be written as in Eqn.3.22 & Eqn.3.23.

$$M(x) = p.(\frac{x}{H}).\frac{(H-x)^2}{2} + p.(1-\frac{x}{H}).\frac{(H-x)^2}{3} \tag{3.22}$$

$$M(x) = p.\frac{(H-x)^2}{H}.\left(\frac{x}{2} + \frac{(H-x)}{3}\right) \tag{3.23}$$

In order to find A_1 and A_2 in Eqn.3.21, we apply the following two boundary conditions;

i. M (H) = 0

ii. $M^1(0) = -\frac{1}{2}.p.H$

Applying the above two boundary conditions in order, we obtain A_1 and A_2 as expressed in Eqn.3.24 and Eqn.3.25, respectively.

$$A_1 = \frac{p.s^2}{v^2.\cosh\lambda}.\left(1 + (\frac{\lambda}{2} - \frac{1}{\lambda}).\sinh\lambda\right) \tag{3.24}$$

$$A_2 = -\frac{p.s^2}{v^2}.(\frac{\lambda}{2} - \frac{1}{\lambda}) \tag{3.25}$$

The lateral displacement expression can be obtained by integrating Eqn.4.19 twice as expressed in Eqn.3.26 and Eqn.3.27.

$$w(x) = \int_0^x \int M(x).dx^2 \tag{3.26}$$

$$\begin{aligned} w(x) = {} & A_1.s^2.\cosh\varphi + A_2.s^2.\sinh\varphi + \\ & \frac{v^2-1}{v^2}\iint M(x).dx^2 - \frac{s^2.p}{v^2.H}.\frac{x^3}{6} + A_3.x + A_4 \end{aligned} \tag{3.27}$$

To obtain A_3 and A_4, the following two boundary conditions must be applied.

i. w (0) = 0

ii. w'(0) = 0

Applying the above two boundary conditions in order, we obtain A_3 and A_4 as expressed in Eqn.3.28 and Eqn.3.29, respectively.

$$A_3 = -A_2.s \tag{3.28}$$

$$A_4 = -A_1.s^2 \tag{3.29}$$

Finally the equation of lateral displacement factored by K multiplier, w(x), can be written as expressed in Eqn.3.30 or equivalently in Eqn.3.31.

$$K.y(x) = A_1.s^2.\cosh\varphi + A_2.s^2.\sinh\varphi + \frac{v^2-1}{v^2}.p.H^4.$$
$$\left((\frac{k^3}{12} - \frac{k^4}{12} + \frac{k^5}{40}) + \frac{k^2}{3}.(\frac{1}{2} - \frac{k}{2} + \frac{k^2}{4} - \frac{k^3}{20}) \right) - \frac{s^2.p}{v^2.H}.\frac{x^3}{6} + A_3.x + A_4 \tag{3.30}$$

$$K.y(x) = A_1 s^2 \cosh\varphi + A_2 s^2 \sinh\varphi + (1 - \frac{1}{v^2})pH^4(\frac{k^2}{6} - \frac{k^3}{12} + \frac{k^5}{120})$$
$$- \frac{s^2.p.k}{6.v^2}.x^2 + A_3.x + A_4 \tag{3.31}$$

Equation of slope along height of the building, factored by K multiplier, can be written as expressed in Eqn.3.32 too.

$$K.y^{I}(x) = A_1.s.\sinh\varphi + A_2.s.\cosh\varphi + (1 - \frac{1}{v^2})pH^4.$$
$$(\frac{x}{3.H^2} - \frac{x^2}{4.H^3} + \frac{x^4}{24.H^5}) - \frac{s^2.p}{2.v^2.H}.x^2 + A_3 \tag{3.32}$$

Equation of curvature along height of the building, factored by K multiplier, can also be written as expressed in Eqn.3.33.

$$K.y^{II}(x) = A_1.\cosh\varphi + A_2.\sinh\varphi + (1 - \frac{1}{v^2})pH^4.$$
$$(\frac{1}{3.H^2} - \frac{x}{2.H^3} + \frac{x^3}{6.H^5}) - \frac{s^2.p}{v^2.H}.x \tag{3.33}$$

Moment equation can easily be obtained by multiplying the curvature with EI as given in Eqn.3.34 and Eqn.3.35.

$$M(x) = -EI \,.\, y^{II}(x) \tag{3.34}$$

$$M(x) = -\frac{EI}{K}[A_1.\cosh\varphi + A_2.\sinh\varphi + (1 - \frac{1}{v^2})pH^4.$$
$$(\frac{1}{3.H^2} - \frac{x}{2.H^3} + \frac{x^3}{6.H^5}) - \frac{s^2.p}{v^2.H}.x] \tag{3.35}$$

Shear equation can then be readily obtained by differentiating the moment equation with respect to x as expressed in Eqn.3.36 and so Eqn.3.37 is attained.

$$V(x) = -M'(x) = EI \,.\, y^{III}(x) \tag{3.36}$$

$$V(x) = \frac{EI}{K}.\left\{\frac{A_1}{s}\sinh\varphi + \frac{A_2}{s}\cosh\varphi + \left(1-\frac{1}{v^2}\right)pH\left(\frac{x^2}{2H^2}-\frac{1}{2}\right) - \frac{s^2.p}{v^2.H}\right\} \tag{3.37}$$

Force equation (i.e. equation of load coming to shear wall) can then be readily obtained by differentiating the shear equation with respect to x as expressed in Eqn.3.38 and so Eqn.3.39 is attained.

$$P(x) = -V'(x) = -EI \, . \, y^{IV}(x) \tag{3.38}$$

$$P(x) = -\frac{EI}{K}.\left\{\frac{A_1}{s^2}.\cosh\varphi + \frac{A_2}{s^2}.\sinh\varphi + \left(1-\frac{1}{v^2}\right).\frac{p}{H}.x\right\} \tag{3.39}$$

Lateral sway at any height of the building and relative story drifts as well as slope & curvature along height of the building can be easily calculated by using the executable "Borland Delphi" program developed, which is shown in Figure 3.3. The graphs showing the number of story versus displacements & relative story drifts and the number of story versus slope & curvature along height at story levels can also be drawn easily by using the developed "Borland Delphi" program.

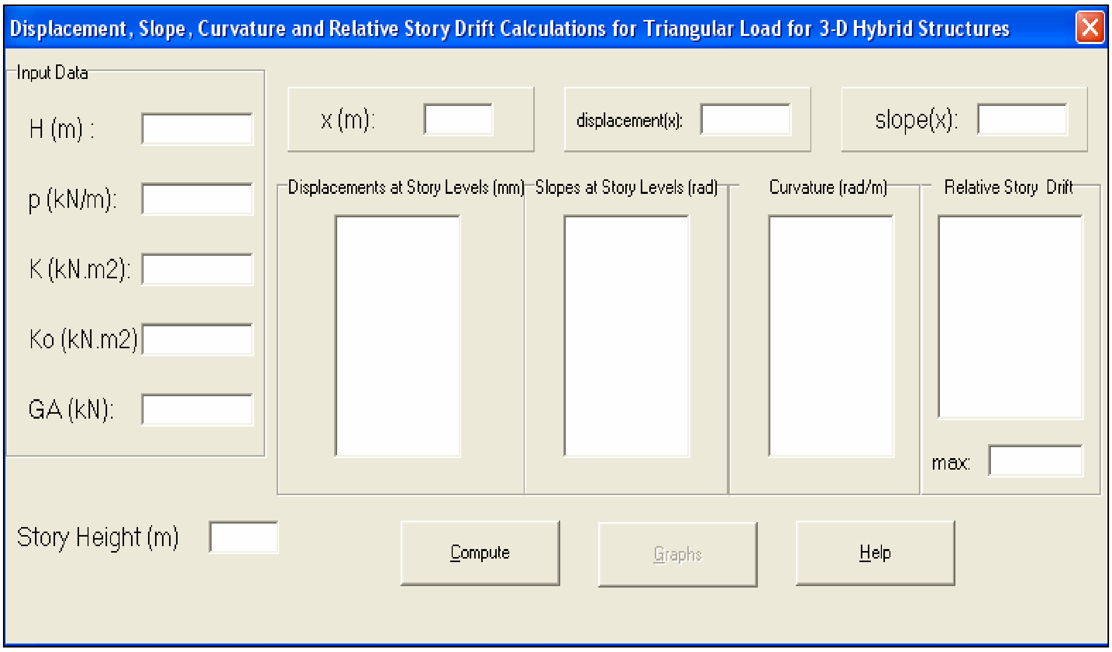


Figure 3.3 Executable "Borland Delphi" Program to Calculate Lateral Sway, Slope, Curvature and Relative Story Drift for Mixed Structures

3.4 ASSESSING THE VALIDITY OF THE ANALYTICAL MODEL (EXAMPLE 1)

Firstly the validity of the analytical model developed was tested on a 3D-mixed structure having only 2 shear walls with l_w=6m and b_w=0.25m (with different number of stories) as shown in Figure3.4. The results that are determined by using SAP2000 and analytical equation were then compared both in tabular and graphical forms.

The parameters used in the analytical expression were calculated as below.

K= K (shear walls) + Σ K (columns)

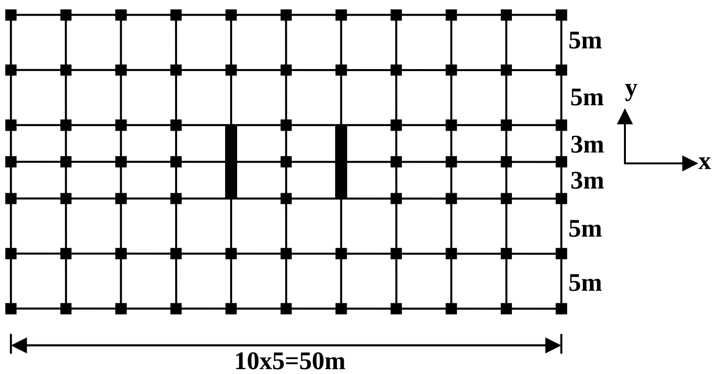
Since Σ K (columns) term can be neglected, then along y-direction

$$K = 28\,500\,000\,\text{kN/m}^2.\left[\frac{1}{12}(0.25)(6)^3\right]x2 = 256\,500\,000\,\text{kN.m}^2$$

$$K_0 = 28\,500\,000\,\text{kN/m}^2.\left[\left((0.4)(0.4)(13)^2\right)x11\right]x2 = 16\,954\,000\,000\,\text{kN.m}^2$$

GA = 22 x 41 793 + 18 x 47 542 + 9 x 5 182 + 22 x 28 169 = 2 441 558 kN

The seismic force (i.e. base shear) is converted to an equivalent distributed lateral static force having an inverted triangular shape. This equivalent lateral static force having an assumed top intensity of p=1 000 kN/m was applied to the structure and analyzed by the computer using SAP2000 and the analytical equation, Eqn.3.31.



All columns : 400x400 mm
All beams : 250x450 mm
Slab thickness : 120 mm
All storey heights : 3 m
g (additional) : 2.0 kN/m^2
q (additional) : 3.5 kN/m^2

(a)

(b)

(EXAMPLE 1)

Figure 3.4 Mixed Structure Used to Test the Validity of the Analytical Method: **(a)** Typical floor plan, **(b)** 3-D view of a sample 4-storey mixed structure

3.5 COMPARISON OF RESULTS (EXAMPLE1)

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 3.1 and Table 3.2, respectively. On the other hand, comparison of lateral displacements is also shown graphically from Figure 3.5 to Figure 3.11 while the comparison of slope along height at story levels is shown from Figure 3.12 to Figure 3.18. Finally, the comparison of relative story drifts is shown graphically from Figure 3.19 to Figure 3.25.

It should be emphasized here that the shear walls was modeled in SAP2000 as shell elements in this study. If one models the shear walls as wide columns, then the results may change a little bit (but not drastic). Therefore, the results may be little different than the ones given in this study. That is to say, the modeling details may yield slightly different results, which are within the acceptable limits.

Table 3.1 Comparisons of Lateral Displacements and Relative Story Drifts as Determined by SAP2000 and Analytical Model for Mixed Structure (Example1)

# of	Displacement	Displacement	Difference	Relative Story Drift	Relative Story Drift	Difference
story	Sap2000(mm)	Analytic(mm)	(%)	(Sap2000)	(Analytic)	(%)
2	0.616	0.40	34.90	**0.000104**	**0.000087**	16.67
1	0.304	0.14	53.61	0.000101	0.000047	53.62
				max=0.000104	**max=0.000087**	**16.67**
4	5.16	4.60	10.93	0.000460	**0.000486**	5.65
3	3.78	3.14	16.98	**0.000513**	0.000480	6.56
2	2.24	1.70	24.15	0.000469	0.000395	15.76
1	0.832	0.51	38.34	0.000277	0.000171	38.34
				max=0.000513	**max=0.000486**	**5.32**
6	17.2	16.06	6.63	0.000987	0.001043	5.67
5	14.24	12.93	9.19	0.001100	0.001088	1.12
4	10.94	9.67	11.62	**0.001157**	**0.001104**	4.58
3	7.47	6.36	14.89	0.001107	0.001021	7.71
2	4.15	3.29	20.65	0.000903	0.000779	13.76
1	1.44	0.96	33.61	0.000480	0.000319	33.61
				max=0.001157	**max=0.001104**	**4.58**
8	38.45	36.05	6.23	0.001540	0.001586	2.96
7	33.83	31.30	7.48	0.001710	0.001680	1.77
6	28.7	26.26	8.50	0.001863	0.001794	3.74
5	23.11	20.88	9.66	**0.001953**	**0.001860**	4.79
4	17.25	15.30	11.31	0.001937	0.001819	6.06
3	11.44	9.84	13.98	0.001757	0.001616	8.04
2	6.17	4.99	19.05	0.001367	0.001190	12.9
1	2.07	1.42	31.25	0.000690	0.000474	31.26
				max=0.001953	**max=0.001860**	**4.79**
10	69.8	65.31	6.43	0.002053	0.002066	0.6
9	63.64	59.11	7.11	0.002270	0.002199	3.13
8	56.83	52.51	7.59	0.002507	0.002392	4.57
7	49.31	45.34	8.05	0.002717	0.002580	5.03
6	41.16	37.60	8.65	0.002867	0.002709	5.49
5	32.56	29.47	9.49	**0.002893**	**0.002730**	5.64
4	23.88	21.28	10.88	0.002770	0.002593	6.38
3	15.57	13.50	13.29	0.002437	0.002245	7.89
2	8.26	6.77	18.07	0.001847	0.001619	12.29
1	2.72	1.91	29.85	0.000907	0.000636	29.85
				max=0.002893	**max=0.002730**	**5.64**

Table 3.1 Comparisons of Lateral Displacements and Relative Story Drifts as Determined by SAP2000 and Analytical Model for Mixed Structure (Example1) (Continued)

# of	Displacement	Displacement	Difference	Relative Story Drift	Relative Story Drift	Difference
story	**Sap2000(mm)**	**Analytic(mm)**	**(%)**	**(Sap2000)**	**(Analytic)**	**(%)**
15	197.48	184.82	6.40	0.003163	0.003104	1.89
14	187.99	175.51	6.63	0.003463	0.003295	4.87
13	177.6	165.63	6.74	0.003827	0.003606	5.77
12	166.12	154.81	6.80	0.004237	0.003978	6.11
11	153.41	142.88	6.86	0.004650	0.004364	6.14
10	139.46	129.79	6.93	0.005030	0.004728	5.99
9	124.37	115.60	7.05	0.005353	0.005039	5.88
8	108.31	100.49	7.22	0.005583	0.005266	5.69
7	91.56	84.69	7.50	**0.005693**	**0.005380**	5.49
6	74.48	68.55	7.96	0.005657	0.005348	5.46
5	57.51	52.50	8.70	0.005430	0.005128	5.55
4	41.22	37.12	9.94	0.004967	0.004670	5.97
3	26.32	23.11	12.19	0.004217	0.003905	7.39
2	13.67	11.39	16.64	0.003090	0.002742	11.27
1	4.4	3.17	27.97	0.001467	0.001056	27.98
				max=0.005693	**max=0.005380**	**5.49**
20	406.4	384.99	5.26	0.004257	0.004265	0.2
19	393.63	372.19	5.44	0.004597	0.004482	2.5
18	379.84	358.75	5.55	0.005020	0.004845	3.48
17	364.78	344.21	5.63	0.005527	0.005298	4.14
16	348.2	328.32	5.70	0.006073	0.005796	4.56
15	329.98	310.93	5.77	0.006627	0.006308	4.8
14	310.1	292.01	5.83	0.007163	0.006809	4.94
13	288.61	271.58	5.90	0.007660	0.007278	4.98
12	265.63	249.74	5.98	0.008110	0.007698	5.08
11	241.3	226.65	6.07	0.008483	0.008053	5.07
10	215.85	202.49	6.18	0.008767	0.008326	5.02
9	189.55	177.51	6.34	0.008950	0.008500	5.03
8	162.7	152.01	6.56	**0.009000**	**0.008552**	4.98
7	135.7	126.36	6.88	0.008893	0.008454	4.94
6	109.02	101.00	7.36	0.008597	0.008171	4.95
5	83.23	76.48	8.10	0.008067	0.007653	5.13
4	59.03	53.53	9.32	0.007237	0.006834	5.57
3	37.32	33.02	11.50	0.006040	0.005623	6.89
2	19.2	16.16	15.85	0.004367	0.003898	10.75
1	6.1	4.46	26.83	0.002033	0.001488	26.84
				max=0.009000	**max=0.008552**	**4.98**

Table 3.2 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model for Mixed Structure (Example1)

# of story	Slope along height at story levels Sap2000(rad)	Slope along height at story levels Analytic(rad)	Difference (%)
2	0.0000953	0.000089	6.4
1	0.0001187	0.000078	34.1
4	0.0004333	0.000482	11.3
3	0.0005000	0.000490	2.0
2	0.0005000	0.000456	8.8
1	0.0004040	0.000311	23.1
6	0.0009333	0.001030	10.4
5	0.0010667	0.001064	0.3
4	0.0011300	0.001107	2.1
3	0.0011467	0.001084	5.5
2	0.0010200	0.000932	8.7
1	0.0007353	0.000590	19.7
8	0.0014533	0.001565	7.7
7	0.0016567	0.001623	2.0
6	0.0017867	0.001739	2.6
5	0.0019167	0.001840	4.0
4	0.0019600	0.001862	5.0
3	0.0018633	0.001749	6.1
2	0.0015800	0.001445	8.5
1	0.0010867	0.000887	18.4
10	0.0019466	0.002039	4.7
9	0.0022000	0.002116	3.8
8	0.0023800	0.002291	3.7
7	0.0026167	0.002492	4.8
6	0.0028033	0.002659	5.2
5	0.0028900	0.002742	5.1
4	0.0028500	0.002692	5.5
3	0.0026267	0.002459	6.4
2	0.0021633	0.001984	8.3
1	0.0014500	0.001195	17.6

Table 3.2 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model for Mixed Structure (Example1) (Continued)

# of story	Slope along height at story levels Sap2000(rad)	Slope along height at story levels Analytic(rad)	Difference (%)
15	0.0030133	0.003067	1.8
14	0.0033667	0.003173	5.7
13	0.0036300	0.003436	5.4
12	0.0040300	0.003786	6.1
11	0.0044433	0.004172	6.1
10	0.0048433	0.004553	6.0
9	0.0052000	0.004895	5.9
8	0.0054767	0.005169	5.6
7	0.0056533	0.005344	5.5
6	0.0056900	0.005391	5.2
5	0.0055633	0.005273	5.2
4	0.0052267	0.004944	5.4
3	0.0046233	0.004345	6.0
2	0.0036833	0.003399	7.7
1	0.0023900	0.001998	16.4
20	0.0040866	0.004224	3.4
19	0.0044833	0.004343	3.1
18	0.0047900	0.004644	3.0
17	0.0052700	0.005061	4.0
16	0.0058000	0.005542	4.4
15	0.0063500	0.006052	4.7
14	0.0068967	0.006563	4.8
13	0.0074167	0.007050	4.9
12	0.0078900	0.007498	5.0
11	0.0083033	0.007888	5.0
10	0.0086367	0.008204	5.0
9	0.0088700	0.008431	4.9
8	0.0089867	0.008548	4.9
7	0.0089633	0.008531	4.8
6	0.0087667	0.008347	4.8
5	0.0083567	0.007956	4.8
4	0.0076833	0.007300	5.0
3	0.0066800	0.006302	5.7
2	0.0052433	0.004858	7.3
1	0.0033467	0.002824	15.6

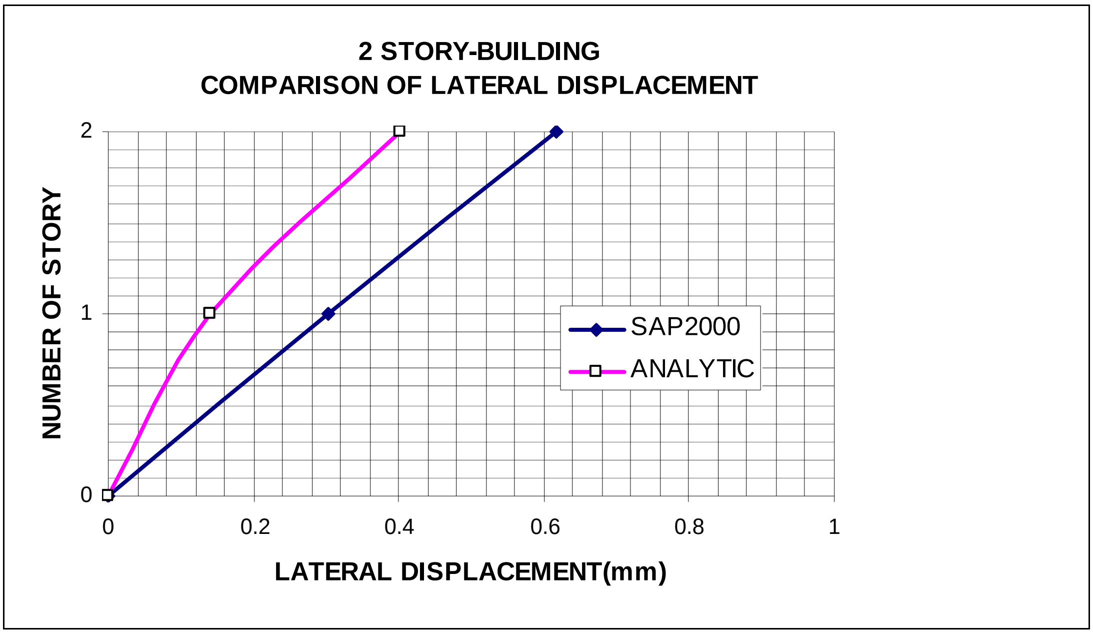


Figure 3.5 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 2 Story-Mixed Structure Example1)

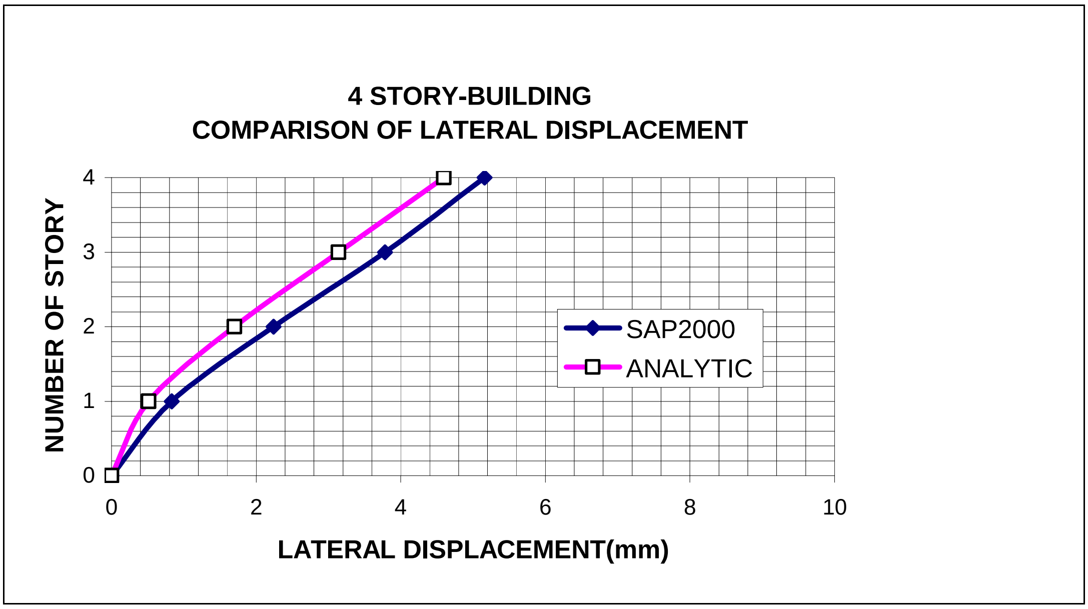


Figure 3.6 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 4 Story-Mixed Structure Example1)

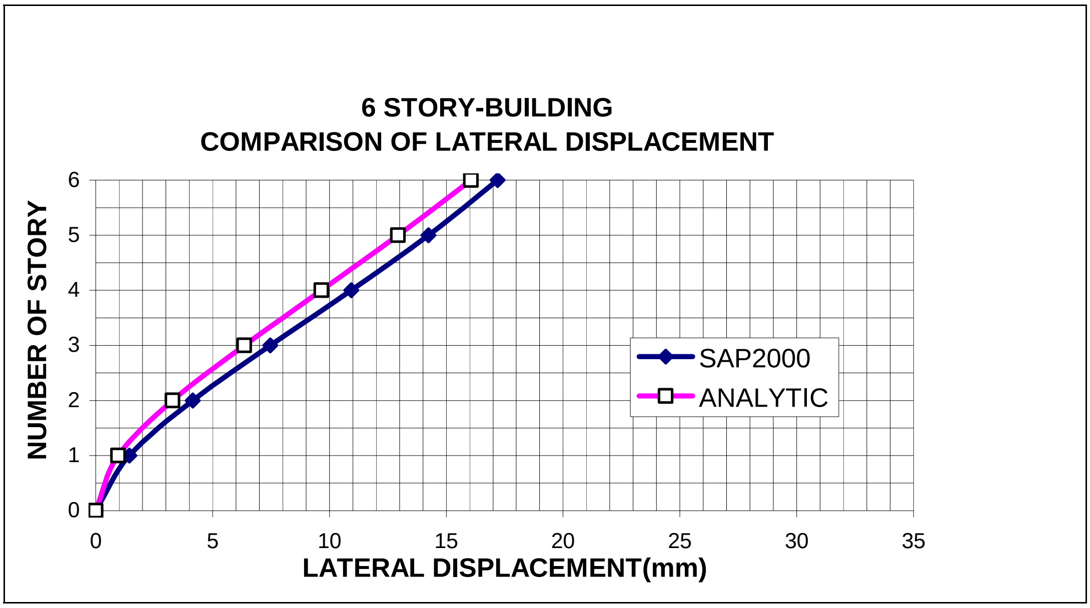


Figure 3.7 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 6 Story-Mixed Structure Example1)

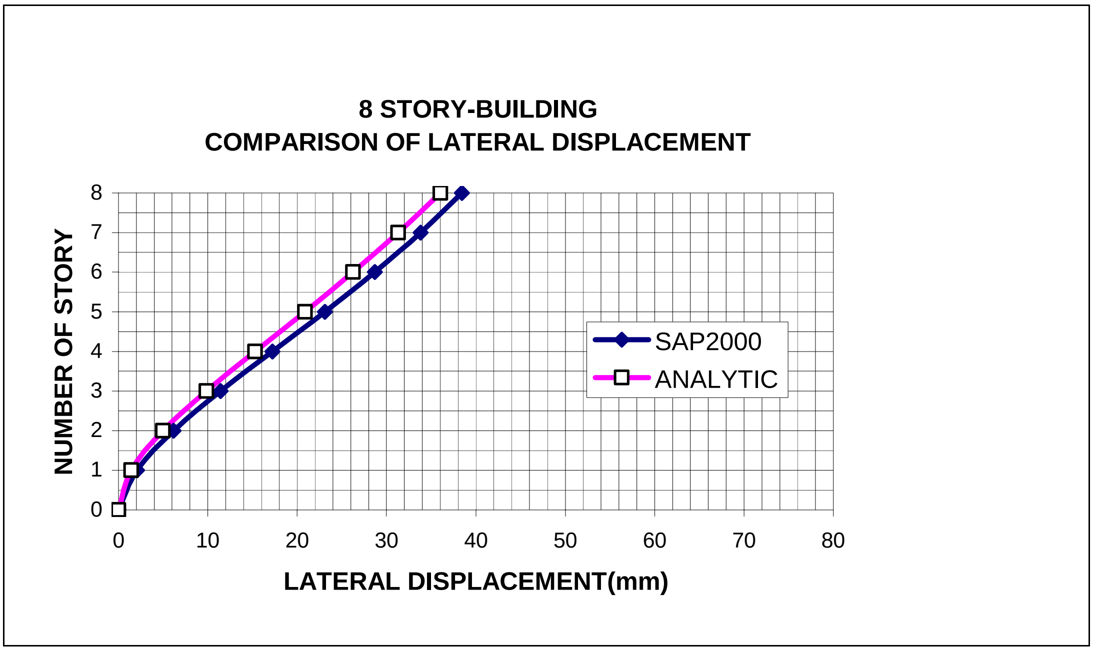


Figure 3.8 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 8 Story-Mixed Structure Example1)

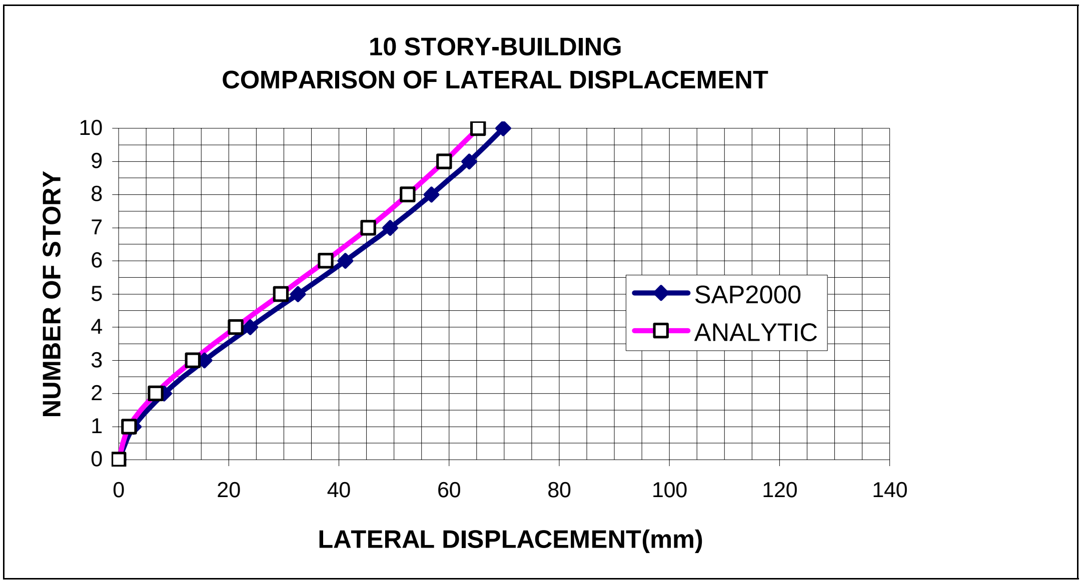


Figure 3.9 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 10 Story-Mixed Structure Example1)

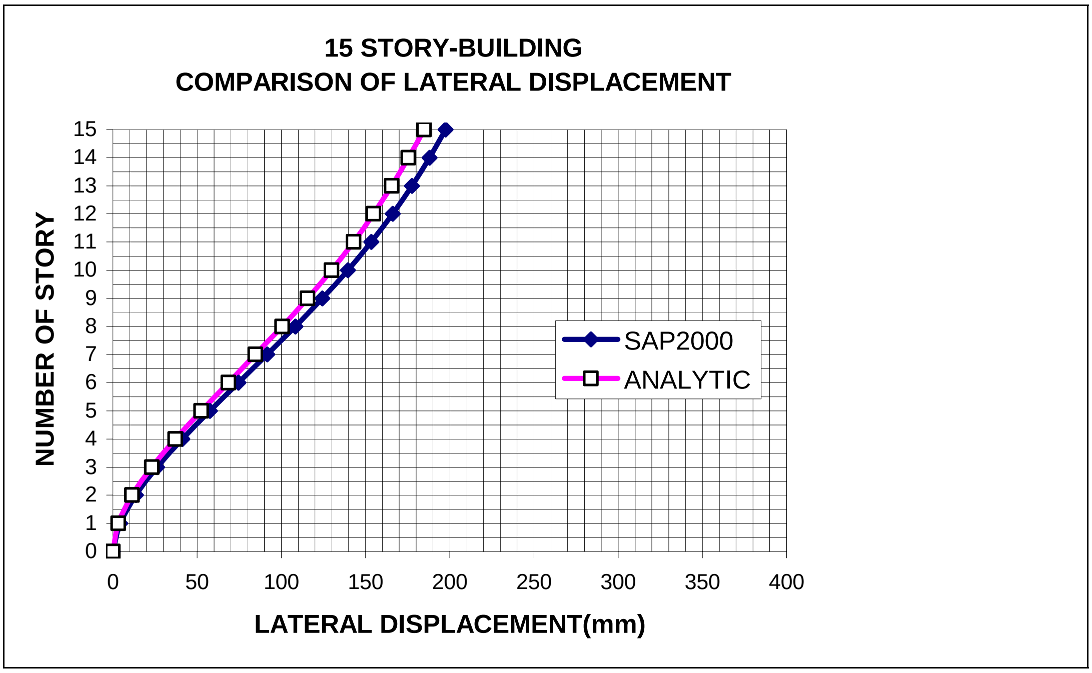


Figure 3.10 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 15 Story-Mixed Structure Example1)

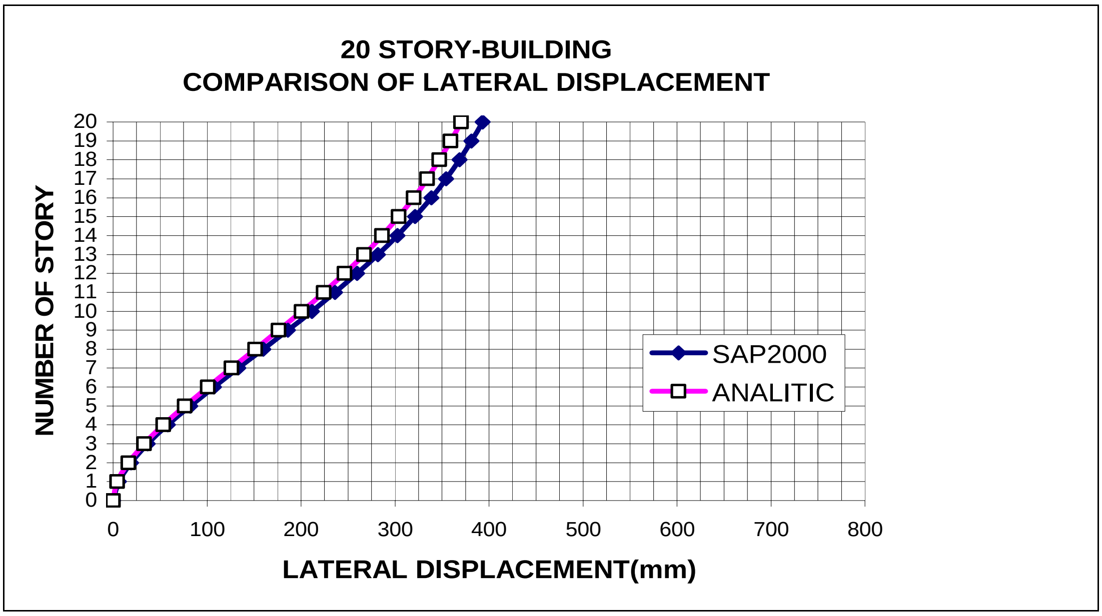


Figure 3.11 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 20 Story-Mixed Structure Example1)

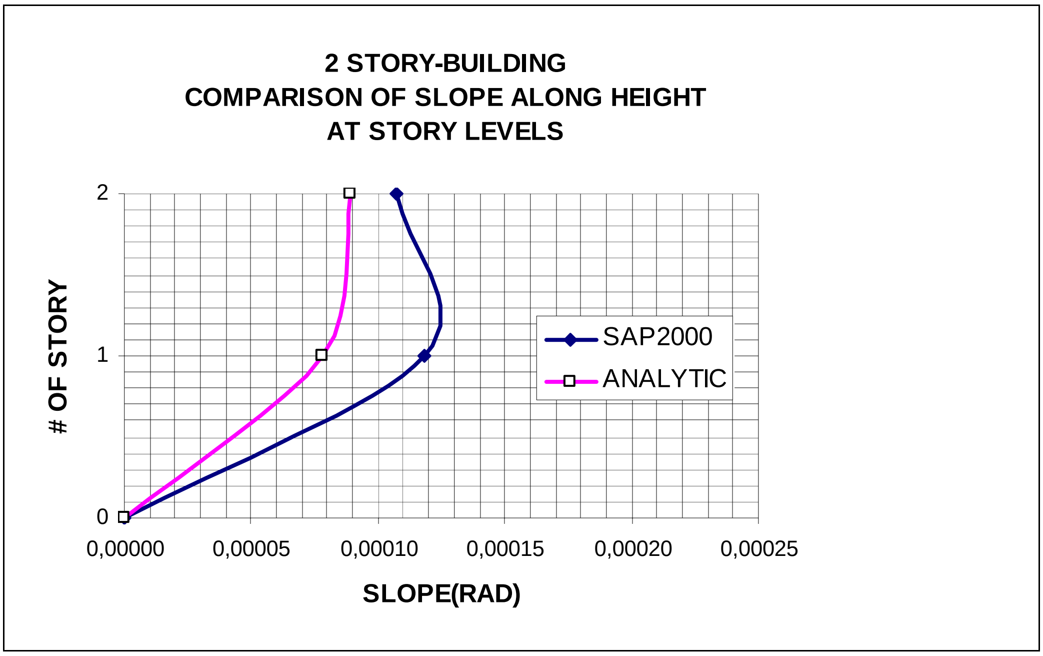


Figure 3.12 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (2 Story-Mixed Structure Example1)

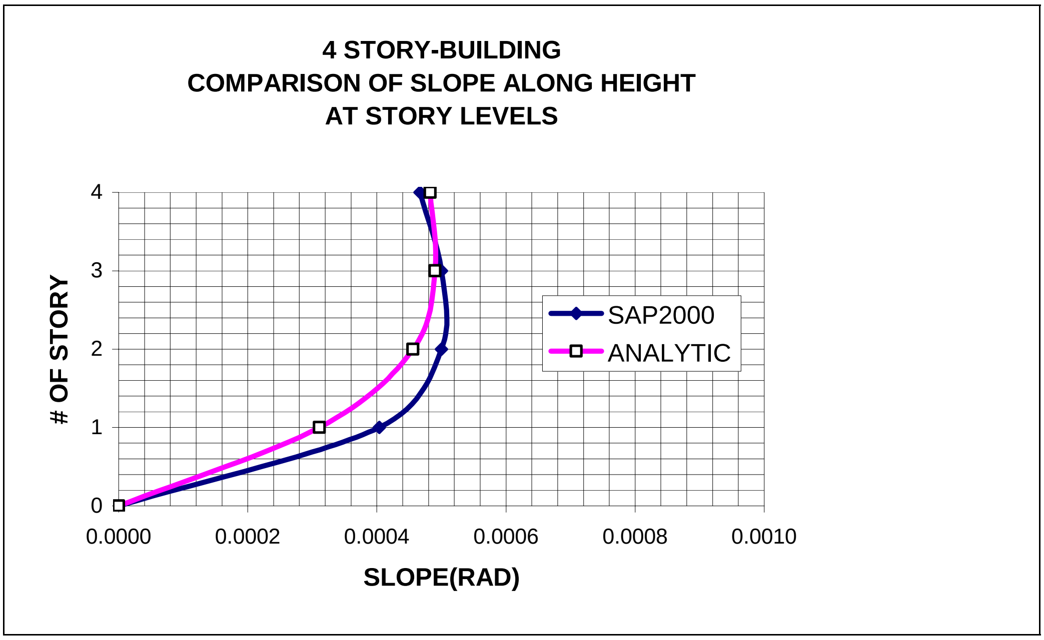


Figure 3.13 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (4 Story-Mixed Structure Example1)

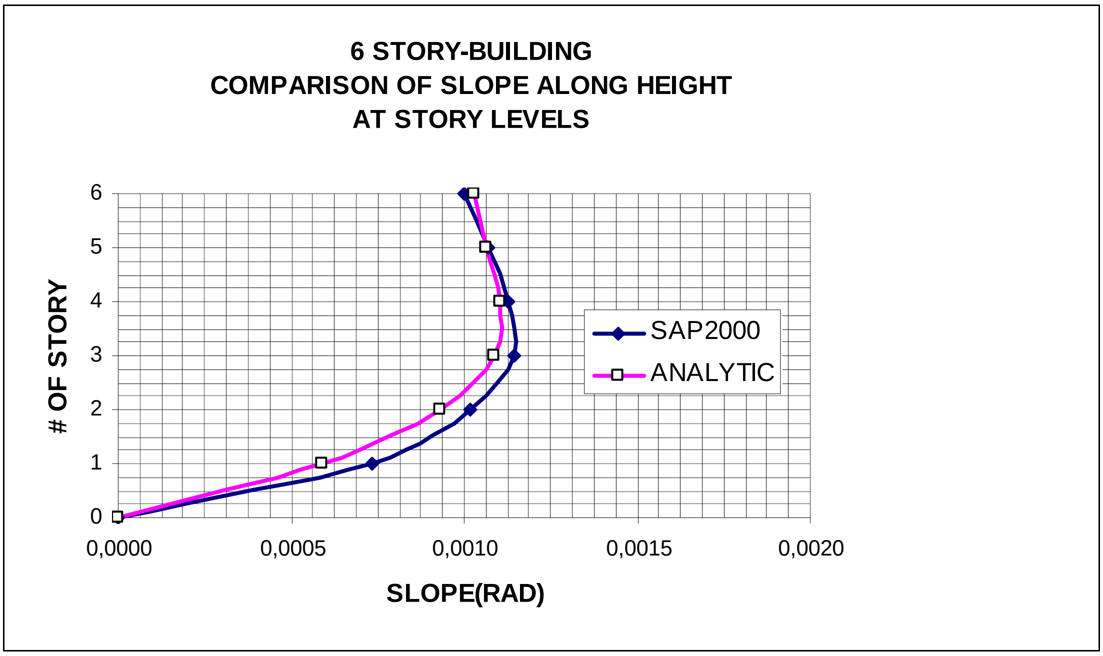


Figure 3.14 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (6 Story-Mixed Structure Example1)

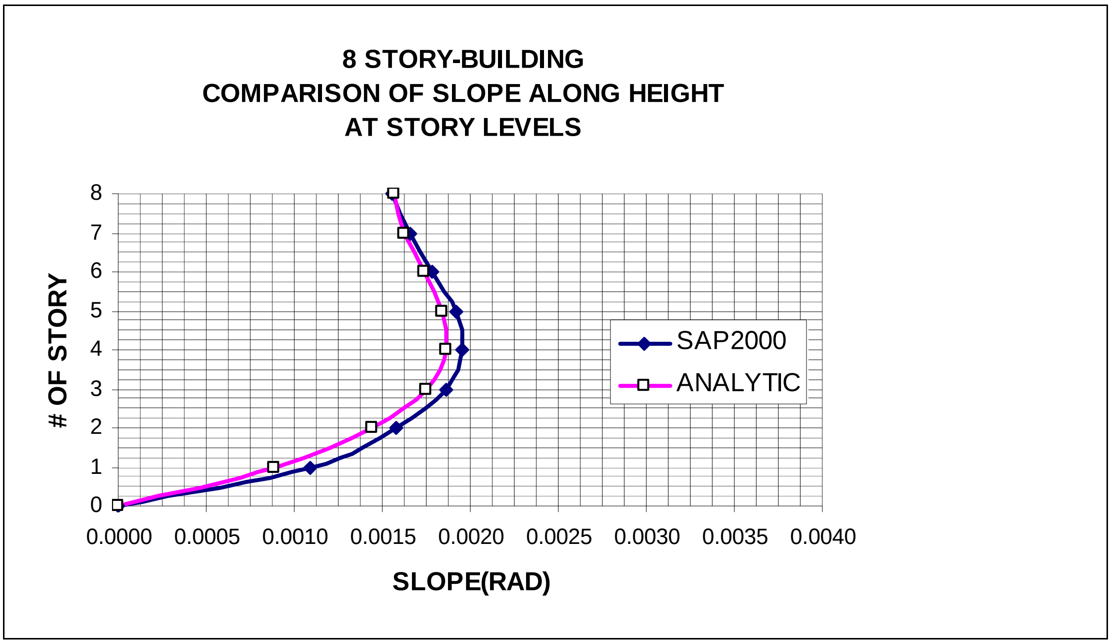


Figure 3.15 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (8 Story-Mixed Structure Example1)

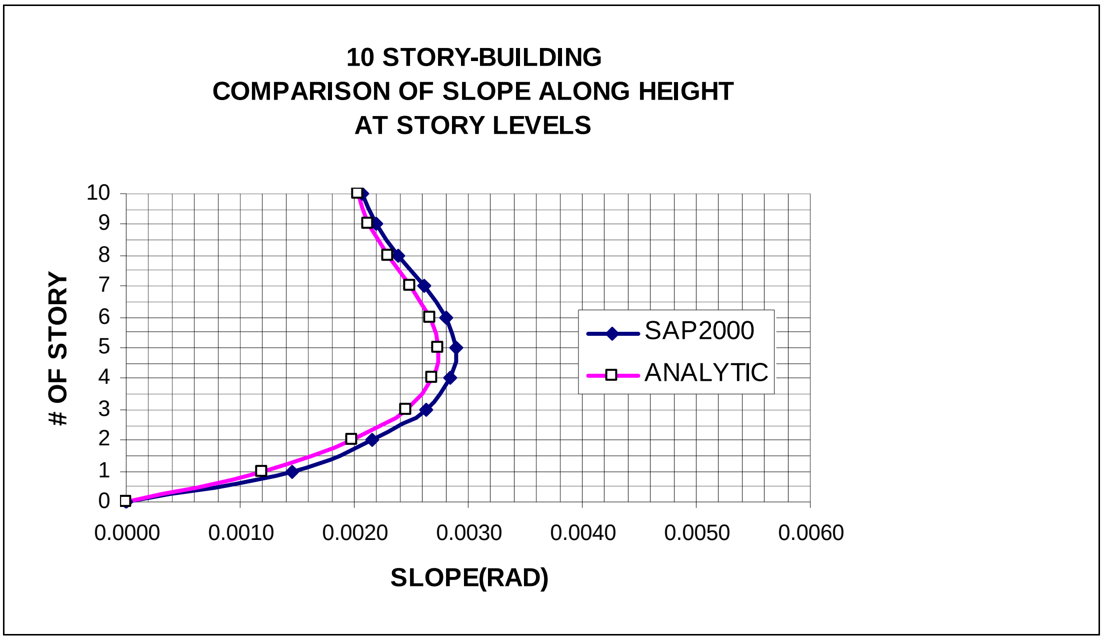


Figure 3.16 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (10 Story-Mixed Structure Example1)

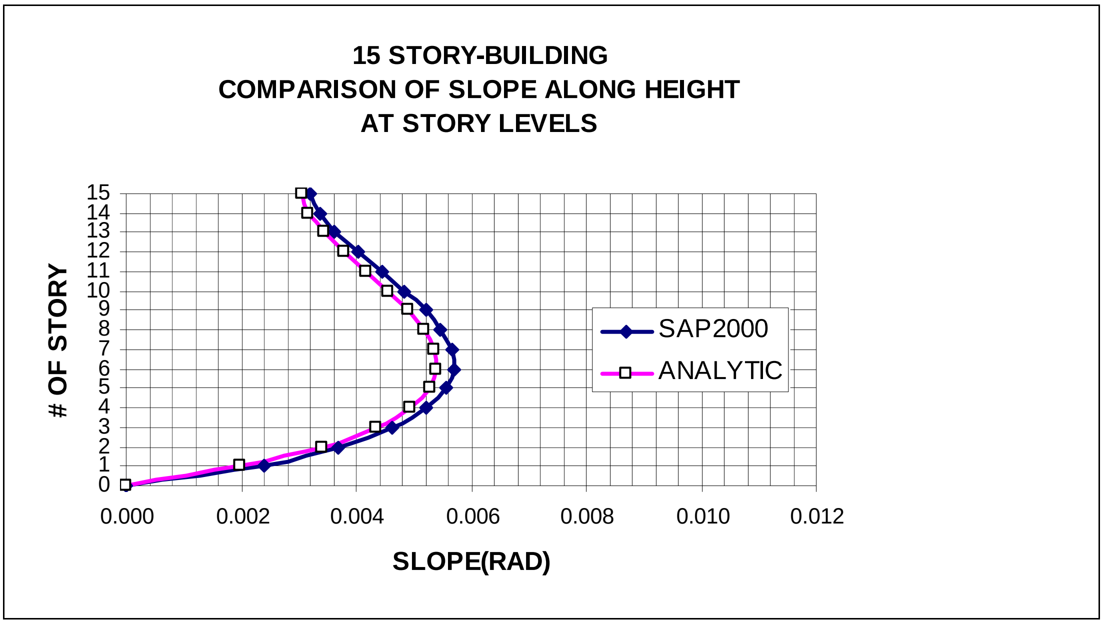


Figure 3.17 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (15 Story-Mixed Structure Example1)

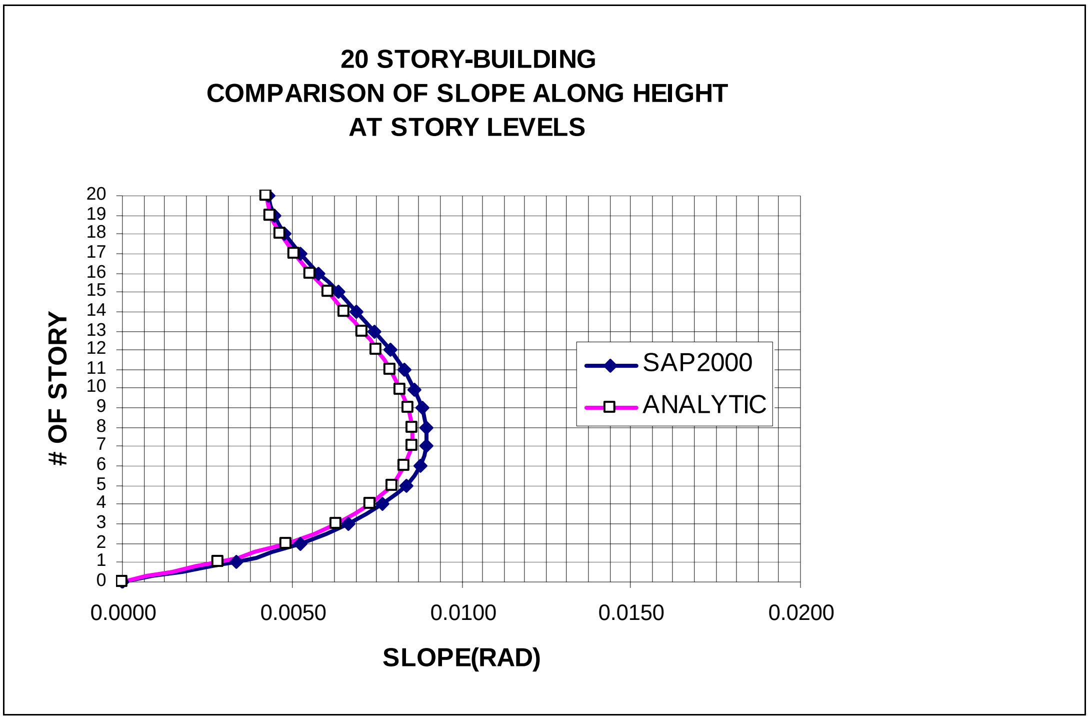


Figure 3.18 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (20 Story-Mixed Structure Example1)

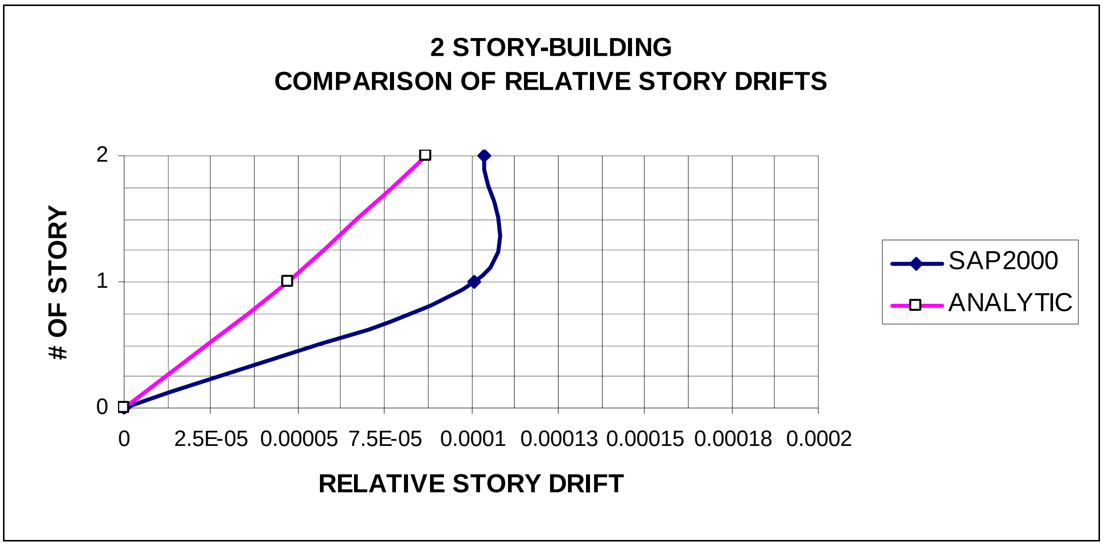


Figure 3.19 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 2 Story-Mixed Structure Example1)

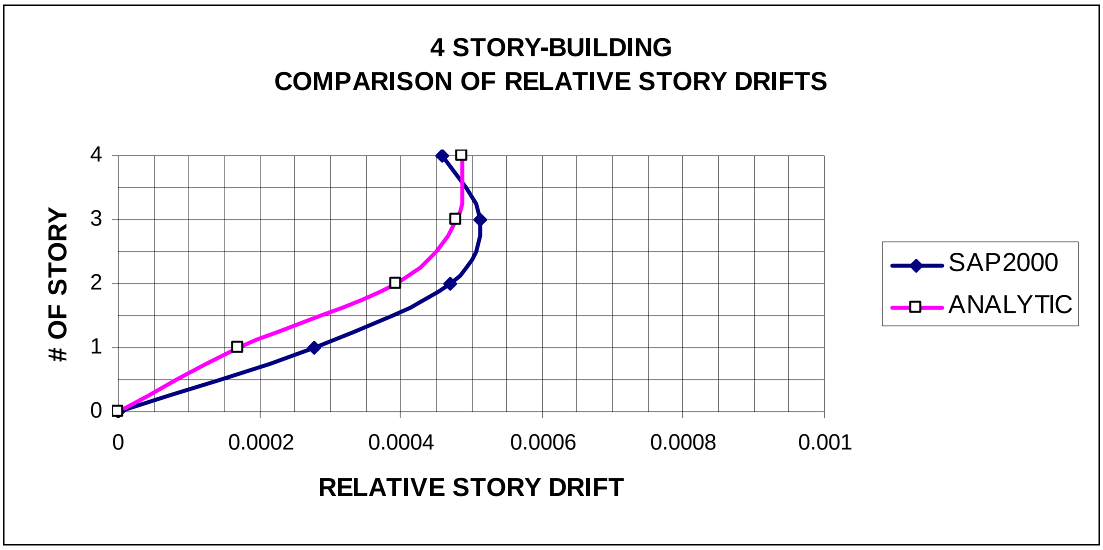


Figure 3.20 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 4 Story-Mixed Structure Example1)

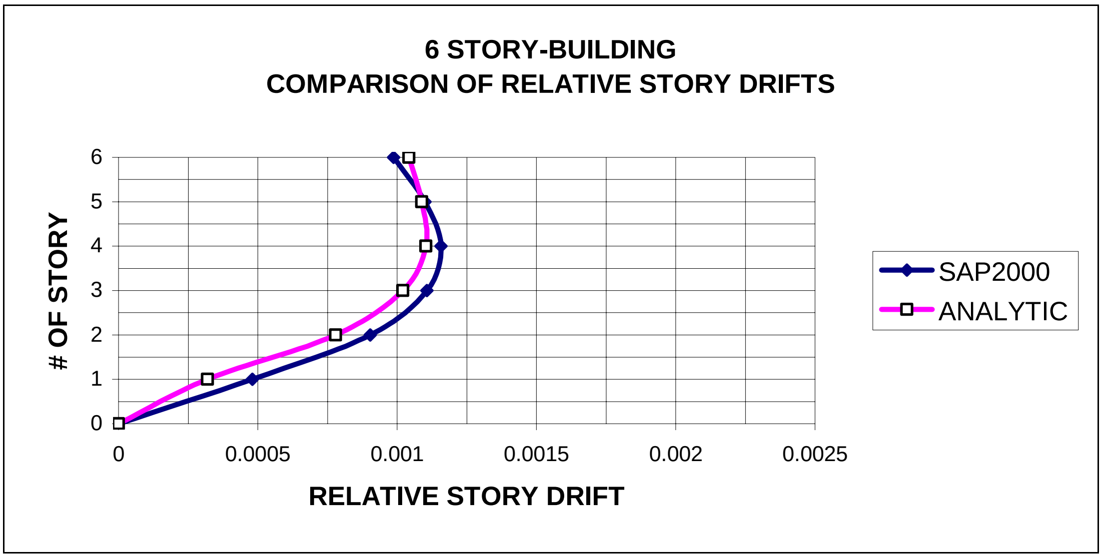


Figure 3.21 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 6 Story-Mixed Structure Example1)

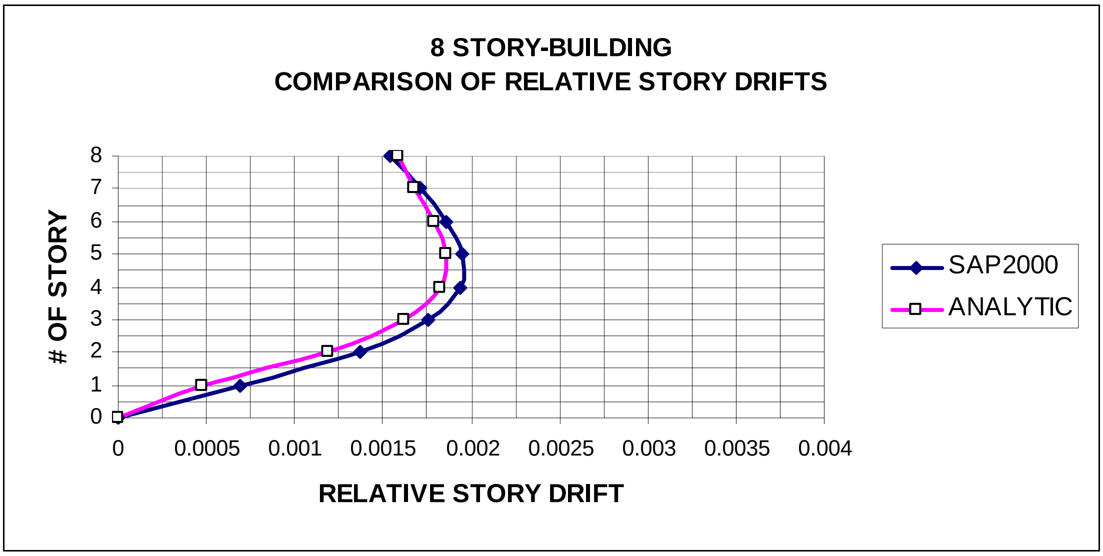


Figure 3.22 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 8 Story-Mixed Structure Example1)

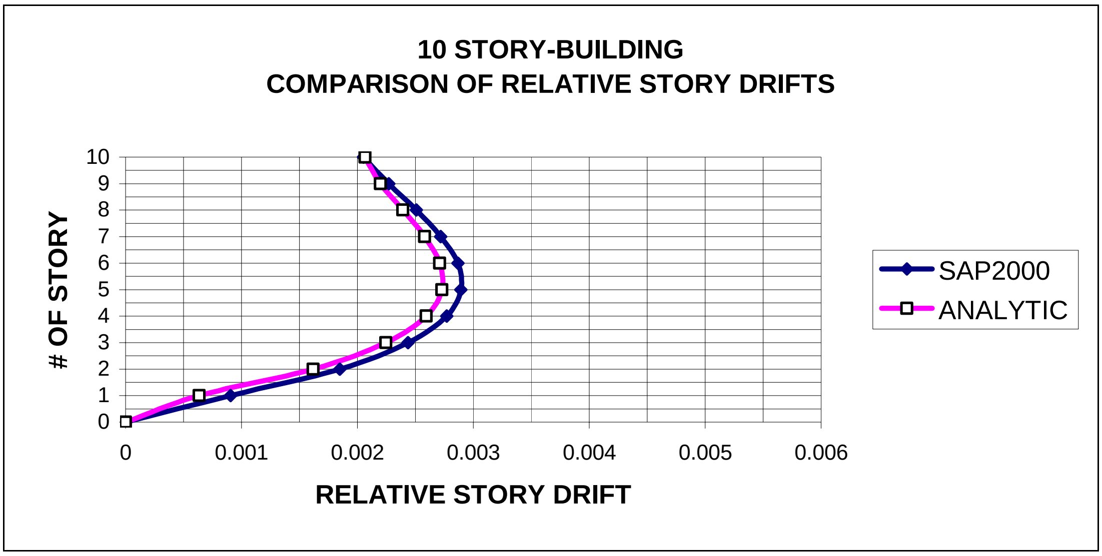


Figure 3.23 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 10 Story-Mixed Structure Example1)

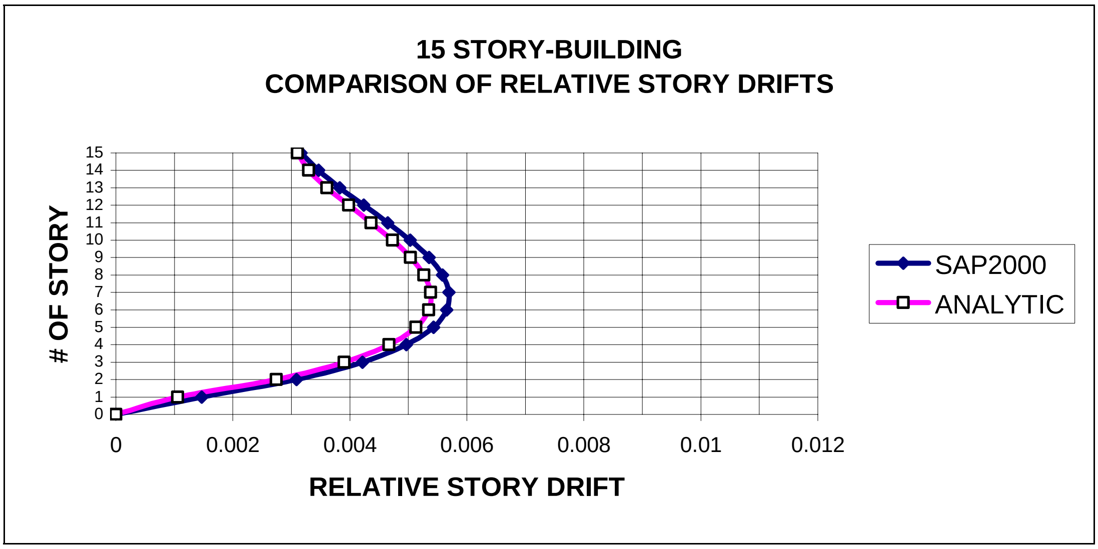


Figure 3.24 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 15 Story-Mixed Structure Example1)

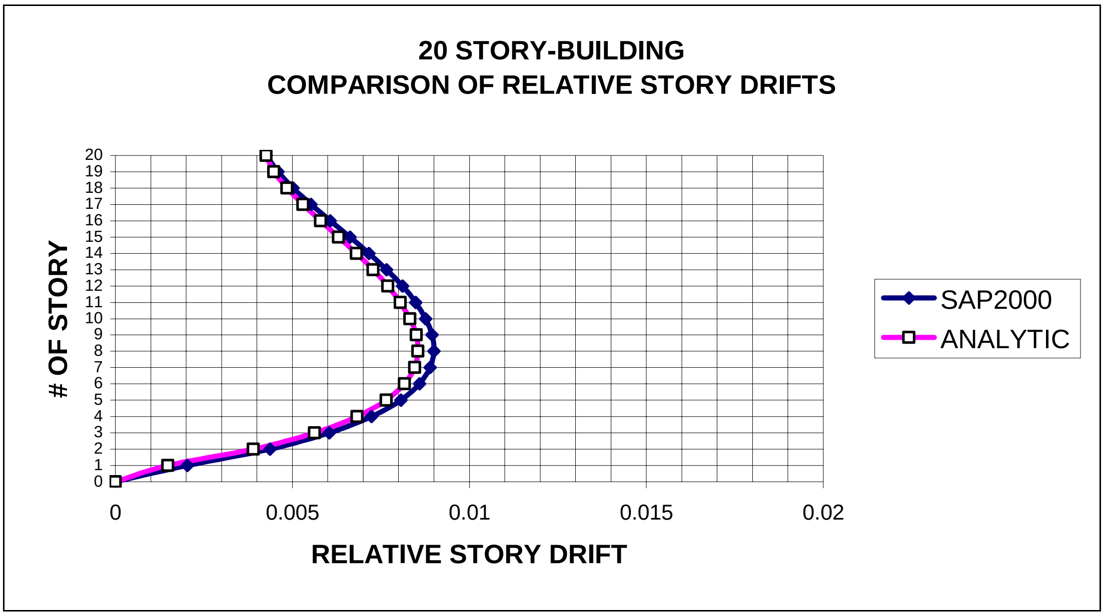


Figure 3.25 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 20 Story-Mixed Structure Example1)

3.6 ASSESSING THE VALIDITY OF THE ANALYTICAL MODEL (EXAMPLE 2)

Secondly the same structure having 4 shear walls, two of which have l_w=6m and b_w=0.25m and the other two have l_w=10m and b_w=0.25m (with different number of stories) as shown in Figure 3.26 was also tested as example 2 to show the validity of analytical model developed.

The parameters used in the analytical expression were calculated as below:

K= K (shear walls) + Σ K (columns)

Since Σ K (columns) term can be neglected, then along y-direction

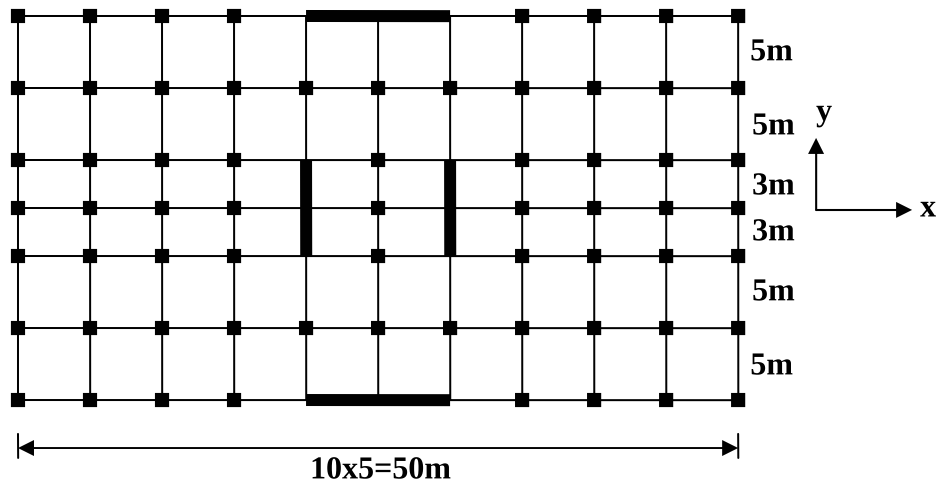
$$K = 28\,500\,000\ \text{kN/m}^2.\left[\frac{1}{12}(0.25)(6)^3 + \frac{1}{12}(10)(0.25)^3\right]x2 = 257\,242\,190\ \text{kN.m}^2$$

$$(K_0)_{columns} = 28\,500\,000\ \text{kN/m}^2.\left[\left((0.4)(0.4)(13)^2\right)x8\right]x2 = 12\,330\,240\,000\ \text{kN.m}^2$$

$$(K_0)_{walls} = 28\,500\,000\ \text{kN/m}^2.\left[(0.25)(10)(13)^2\right]x2 = 24\,082\,500\,000\ \text{kN.m}^2$$

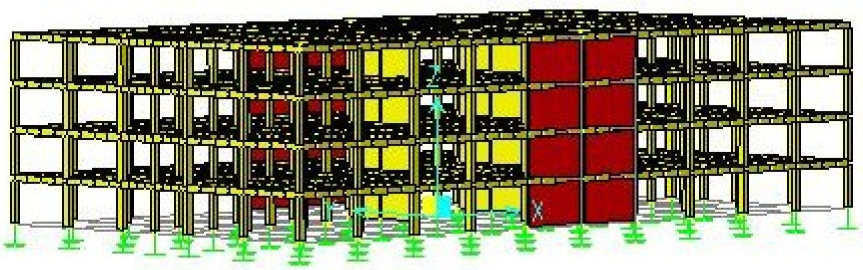
$\sum K_0 = 36\,412\,740\,000 \text{ kN.m}^2$

GA = 22 x 41 793 + 18 x 47 542 + 9 x 5 182 + 16 x 28 169 = 2 272 544 kN



All columns : 400x400 mm
All beams : 250x450 mm
Slab thickness : 120 mm
All storey heights : 3 m
g (additional) : 2.0 kN/m^2
q (additional) : 3.5 kN/m^2

(a)



(b)

(EXAMPLE 2)

Figure 3.26 Mixed Structure Used to Test the Validity of the Analytical Method: **(a)** Typical floor plan, **(b)** 3-D view of a sample 4-storey mixed structure

The seismic force (i.e. base shear) is converted to an equivalent distributed lateral static force having an inverted triangular shape. This equivalent lateral static force having an assumed top intensity of p=1 000 kN/m was applied to the structure and solved by the computer using SAP2000 and the analytical equation, Eqn.3.31.

3.7 COMPARISON OF RESULTS (EXAMPLE2)

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 3.3 and Table 3.4, respectively. On the other hand, comparison of lateral displacements is also shown graphically from Figure 3.27 to Figure 3.33 while the comparison of slope along height at story levels is shown from Figure 3.34 to Figure 3.40. Finally, the comparison of relative story drifts is shown graphically from Figure 3.41 to Figure3.47.

It should be emphasized here that the shear walls was modeled in SAP2000 as shell elements in this study. If one models the shear walls as wide columns, then the results may change a little bit (but not drastic). Therefore, the results may be little different than the ones given in this study. That is to say, the modeling details may yield slightly different results, which are within the acceptable limits.

Table 3.3 Comparisons of Lateral Displacements and Relative Story Drifts as Determined by SAP2000 and Analytical Model for Mixed Structure (Example2)

# of story	Displacement Sap2000(mm)	Displacement Analytic(mm)	Difference (%)	Relative Story Drift (Sap2000)	Relative Story Drift (Analytic)	Difference (%)
2	0.618	0.40	34.78	**0.000106**	**0.000087**	17.86
1	0.299	0.14	52.84	0.000100	0.000047	52.84
				max=0.000106	**max=0.000087**	**17.86**
4	5.14	4.68	8.94	0.000460	**0.000497**	7.97
3	3.76	3.19	15.15	**0.000510**	0.000489	3.92
2	2.23	1.72	22.86	0.000468	0.000401	14.46
1	0.827	0.52	37.12	0.000276	0.000173	37.12
				max=0.000510	**max=0.000497**	**2.61**
6	17.12	16.47	3.79	0.000987	0.001076	9.12
5	14.16	13.24	6.49	0.001093	0.001120	2.43
4	10.88	9.88	9.19	**0.001150**	**0.001132**	1.73
3	7.43	6.49	12.65	0.001103	0.001044	5.13
2	4.12	3.35	18.68	0.000897	0.000794	11.52
1	1.43	0.97	32.16	0.000477	0.000324	32.16
				max=0.001150	**max=0.001132**	**1.74**
8	38.17	36.98	3.11	0.001527	0.001638	7.20
7	33.59	32.07	4.52	0.001693	0.001732	2.36
6	28.51	26.87	5.75	0.001850	0.001845	0.36
5	22.96	21.34	7.05	**0.001940**	**0.001908**	1.71
4	17.14	15.62	8.86	0.001923	0.001862	3.11
3	11.37	10.03	11.78	0.001747	0.001650	5.53
2	6.13	5.08	17.12	0.001357	0.001212	10.81
1	2.06	1.45	29.61	0.000687	0.000482	29.61
				max=0.001940	**max=0.001908**	**1.72**
10	69.11	66.66	3.54	0.002023	0.002116	4.61
9	63.04	60.31	4.33	0.002243	0.002250	0.29
8	56.31	53.56	4.88	0.002473	0.002446	1.07
7	48.89	46.22	5.46	0.002690	0.002636	1.98
6	40.82	38.31	6.14	0.002837	0.002766	2.46
5	32.31	30.02	7.11	**0.002867**	**0.002786**	2.90
4	23.71	21.66	8.64	0.002747	0.002643	3.76
3	15.47	13.73	11.24	0.002420	0.002285	5.50
2	8.21	6.87	16.32	0.001833	0.001646	10.36
1	2.71	1.94	28.41	0.000903	0.000645	28.41
				max=0.002867	**max=0.002786**	**2.91**

Table 3.3 Comparisons of Lateral Displacements and Relative Story Drifts as Determined by SAP2000 and Analytical Model for Mixed Structure (Example2) (Continued)

# of story	Displacement Sap2000(mm)	Displacement Analytic(mm)	Difference (%)	Story drift (Sap2000)	Story drift (Analytic)	Difference (%)
15	193.69	184.05	4.97	0.003053	0.002996	1.85
14	184.53	175.06	5.13	0.003343	0.003192	4.48
13	174.5	165.48	5.16	0.003710	0.003515	5.30
12	163.37	154.94	5.16	0.004123	0.003903	5.33
11	151	143.23	5.14	0.004537	0.004308	5.06
10	137.39	130.31	5.15	0.004923	0.004693	4.67
9	122.62	116.23	5.21	0.005247	0.005024	4.25
8	106.88	101.16	5.35	0.005487	0.005271	3.94
7	90.42	85.35	5.60	**0.005603**	**0.005402**	3.56
6	73.61	69.14	6.07	0.005577	0.005383	3.46
5	56.88	52.99	6.83	0.005360	0.005171	3.54
4	40.8	37.48	8.13	0.004910	0.004714	3.93
3	26.07	23.33	10.51	0.004173	0.003944	5.51
2	13.55	11.50	15.12	0.003063	0.002768	9.68
1	4.36	3.20	26.60	0.001453	0.001066	26.60
				max=0.005603	**max=0.005402**	**3.57**
20	393.27	370.36	5.82	0.003950	0.003723	5.73
19	381.42	359.19	5.82	0.004287	0.003948	7.93
18	368.56	347.35	5.75	0.004713	0.004328	8.13
17	354.42	334.36	5.65	0.005227	0.004805	8.03
16	338.74	319.95	5.54	0.005770	0.005334	7.56
15	321.43	303.94	5.44	0.006330	0.005882	7.10
14	302.44	286.30	5.33	0.006873	0.006422	6.54
13	281.82	267.03	5.24	0.007383	0.006934	6.09
12	259.67	246.23	5.17	0.007837	0.007399	5.57
11	236.16	224.03	5.13	0.008223	0.007800	5.18
10	211.49	200.64	5.13	0.008527	0.008119	4.76
9	185.91	176.28	5.17	0.008720	0.008338	4.35
8	159.75	151.26	5.31	**0.008790**	**0.008432**	4.09
7	133.38	125.97	5.55	0.008703	0.008374	3.79
6	107.27	100.85	5.98	0.008430	0.008124	3.63
5	81.98	76.48	6.70	0.007923	0.007632	3.66
4	58.21	53.58	7.95	0.007123	0.006832	4.07
3	36.84	33.09	10.20	0.005957	0.005631	5.48
2	18.97	16.19	14.65	0.004310	0.003907	9.35
1	6.04	4.47	25.99	0.002013	0.001491	25.99
				max=0.008790	**max=0.008432**	**4.09**

Table 3.4 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model for Mixed Structure (Example2)

# of story	Slope along height at story levels Sap2000(rad)	Slope along height at story levels Analytic(rad)	Difference (%)
2	0.0000740	0.000090	21.2
1	0.0001310	0.000079	40.0
4	0.0004000	0.000493	23.4
3	0.0005200	0.000500	3.8
2	0.0004900	0.000464	5.3
1	0.0003930	0.000315	19.8
6	0.0008800	0.001064	20.9
5	0.0010933	0.001097	0.3
4	0.0011200	0.001138	1.6
3	0.0011367	0.001110	2.3
2	0.0010100	0.000951	5.8
1	0.0007177	0.000601	16.3
8	0.0013733	0.001617	17.8
7	0.0016867	0.001675	0.7
6	0.0017600	0.001791	1.8
5	0.0019033	0.001890	0.7
4	0.0019400	0.001908	1.7
3	0.0018467	0.001788	3.2
2	0.0015667	0.001474	5.9
1	0.0010620	0.000902	15.1
10	0.0018200	0.002088	14.7
9	0.0022333	0.002167	3.0
8	0.0023433	0.002344	0.1
7	0.0025867	0.002547	1.5
6	0.0027667	0.002716	1.8
5	0.0028600	0.002799	2.2
4	0.0028200	0.002746	2.6
3	0.0026000	0.002505	3.6
2	0.0021433	0.002018	5.8
1	0.0014167	0.001214	14.3

Table 3.4 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model for Mixed Structure (Example2) (Continued)

# of story	Slope along height at story levels Sap2000(rad)	Slope along height at story levels Analytic(rad)	Difference (%)
15	0,0027466	0.002958	7,7
14	0,0033500	0.003067	8,4
13	0,0035000	0.003338	4,6
12	0,0039167	0.003702	5,5
11	0,0043300	0.004106	5,2
10	0,0047333	0.004507	4,8
9	0,0050900	0.004870	4,3
8	0,0053733	0.005164	3,9
7	0,0055533	0.005358	3,5
6	0,0056000	0.005421	3,2
5	0,0054833	0.005313	3,1
4	0,0051533	0.004989	3,2
3	0,0045600	0.004388	3,8
2	0,0036467	0.003433	5,9
1	0,0023267	0.002017	13,3
20	0,0035660	0.003680	3,2
19	0,0043100	0.003804	11,7
18	0,0044667	0.004118	7,8
17	0,0049700	0.004555	8,4
16	0,0054967	0.005064	7,9
15	0,0060500	0.005607	7,3
14	0,0066033	0.006155	6,8
13	0,0071300	0.006684	6,2
12	0,0076133	0.007176	5,7
11	0,0080367	0.007611	5,3
10	0,0083800	0.007974	4,8
9	0,0086300	0.008247	4,4
8	0,0087633	0.008408	4,1
7	0,0087567	0.008431	3,7
6	0,0085800	0.008284	3,4
5	0,0081933	0.007923	3,3
4	0,0075467	0.007290	3,4
3	0,0065667	0.006307	4,0
2	0,0051667	0.004868	5,8
1	0,0032567	0.002831	13,1

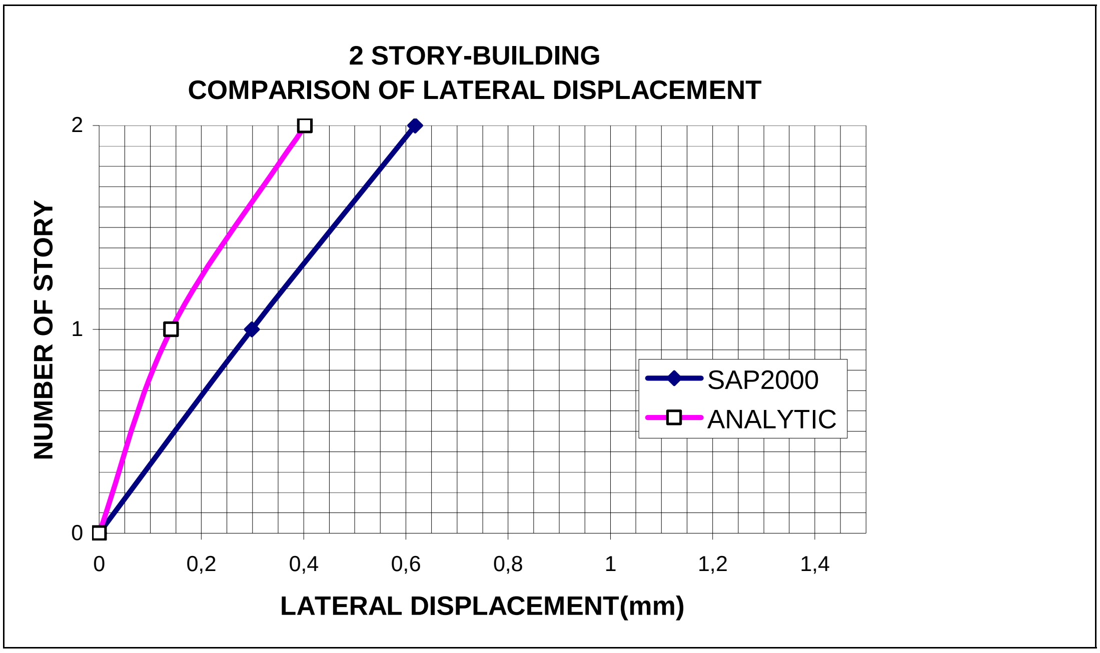


Figure 3.27 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 2 Story-Mixed Structure Example2)

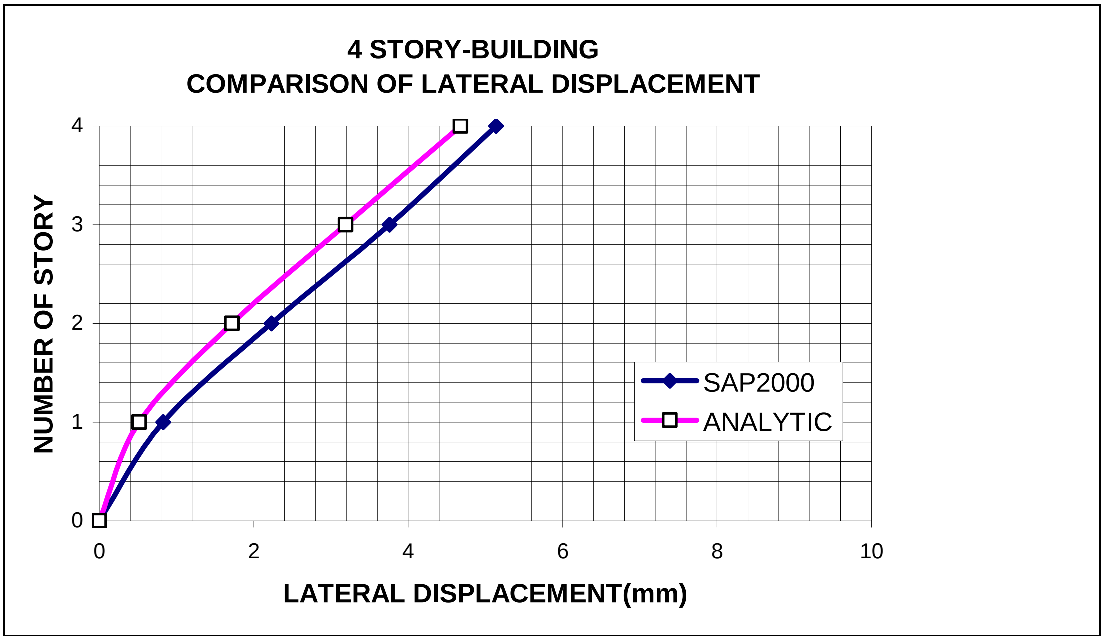


Figure 3.28 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 4 Story-Mixed Structure Example2)

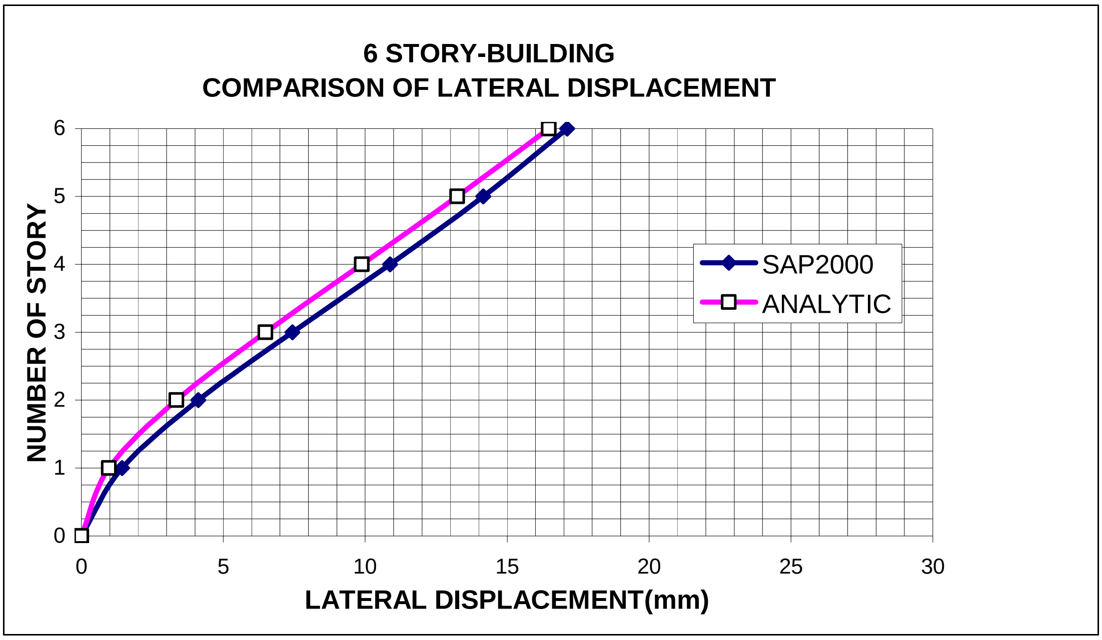


Figure 3.29 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 6 Story-Mixed Structure Example2)

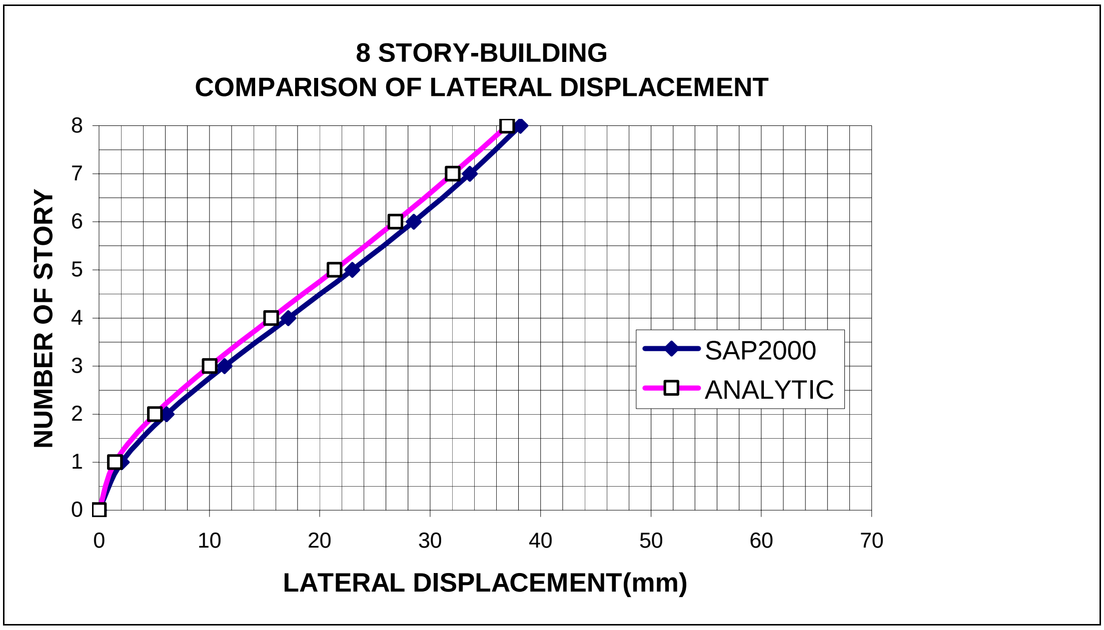


Figure 3.30 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 8 Story-Mixed Structure Example2)

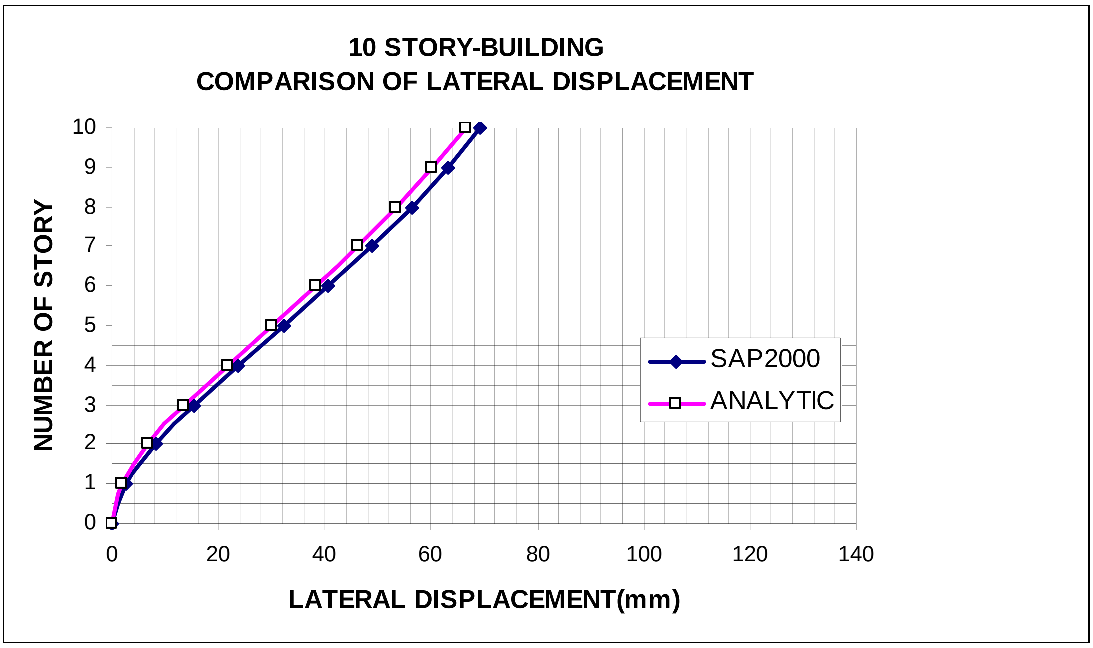


Figure 3.31 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 10 Story-Mixed Structure Example2)

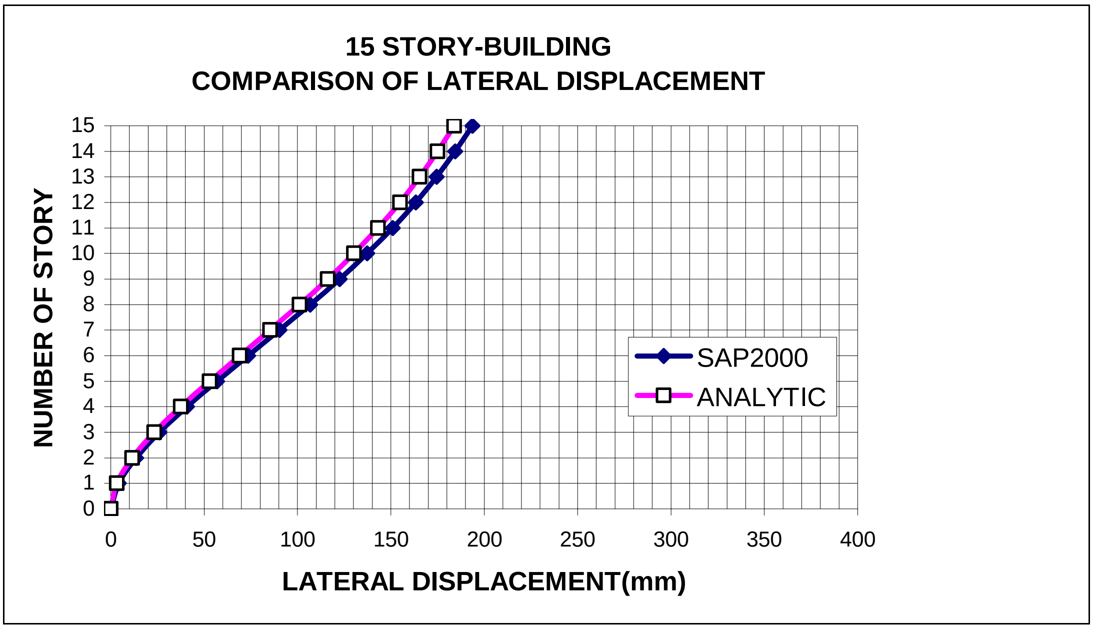


Figure 3.32 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 15 Story-Mixed Structure Example2)

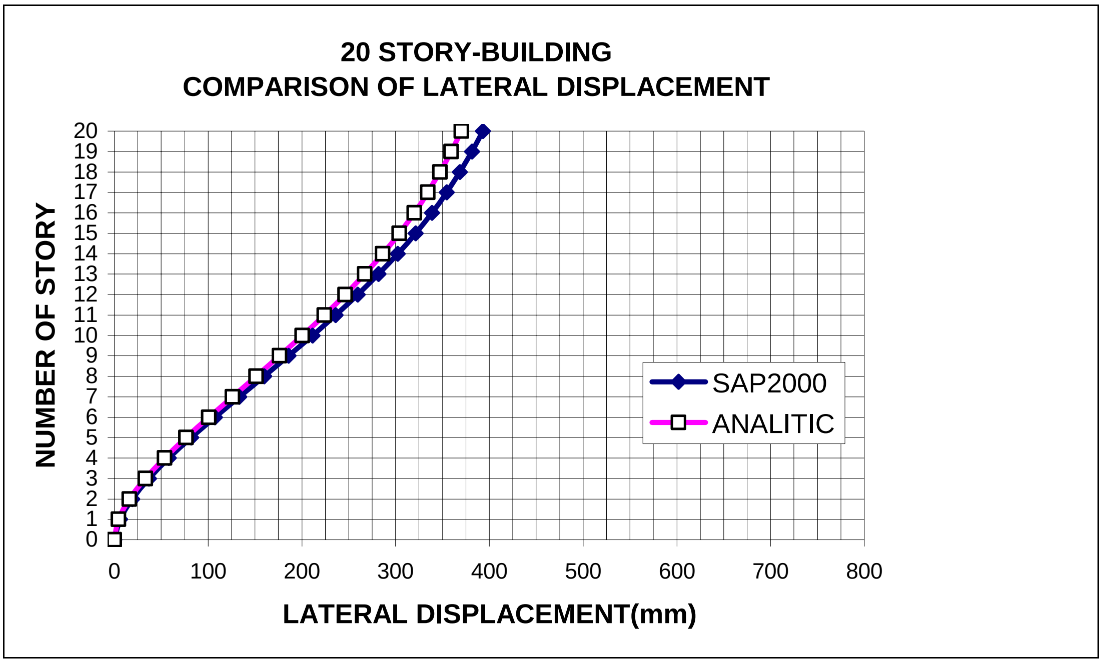


Figure 3.33 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 20 Story-Mixed Structure Example1)

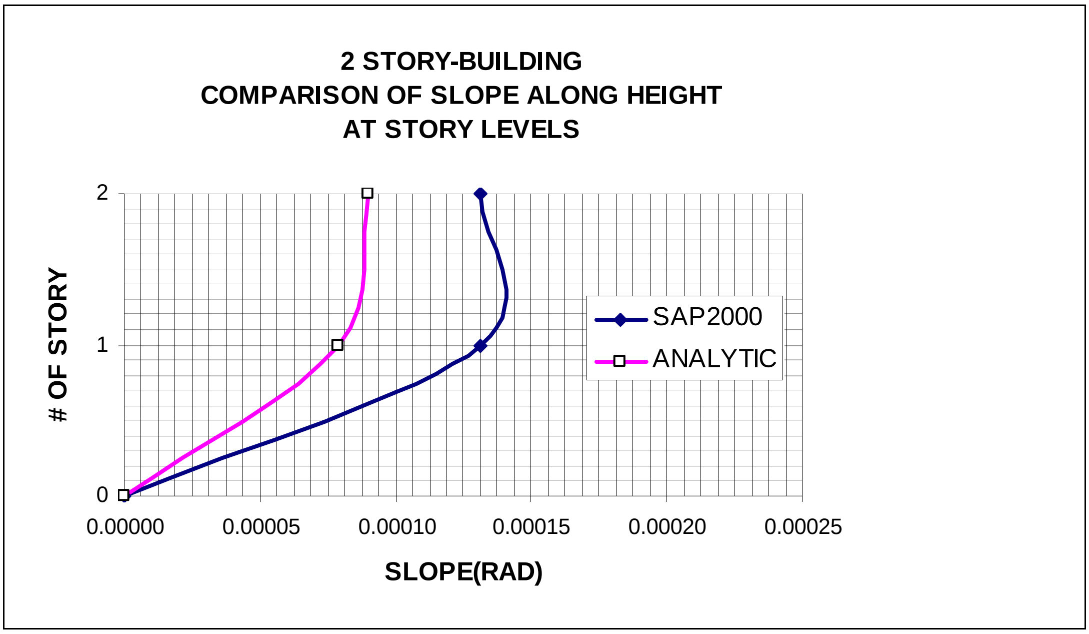


Figure 3.34 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (2 Story-Mixed Structure Example2)

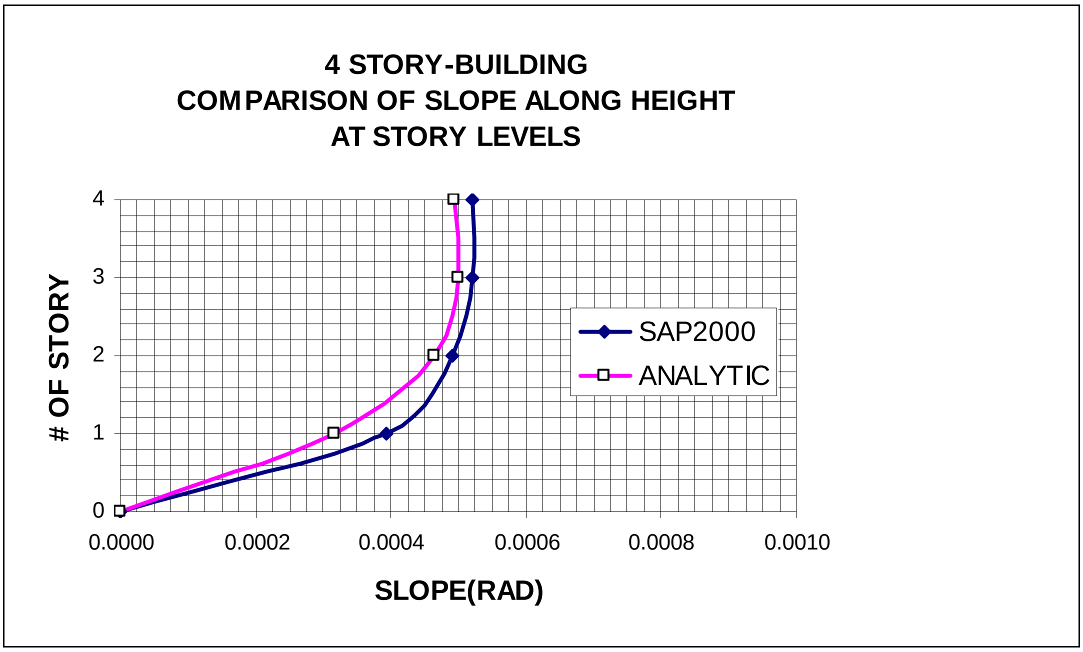


Figure 3.35 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (4 Story-Mixed Structure Example2)

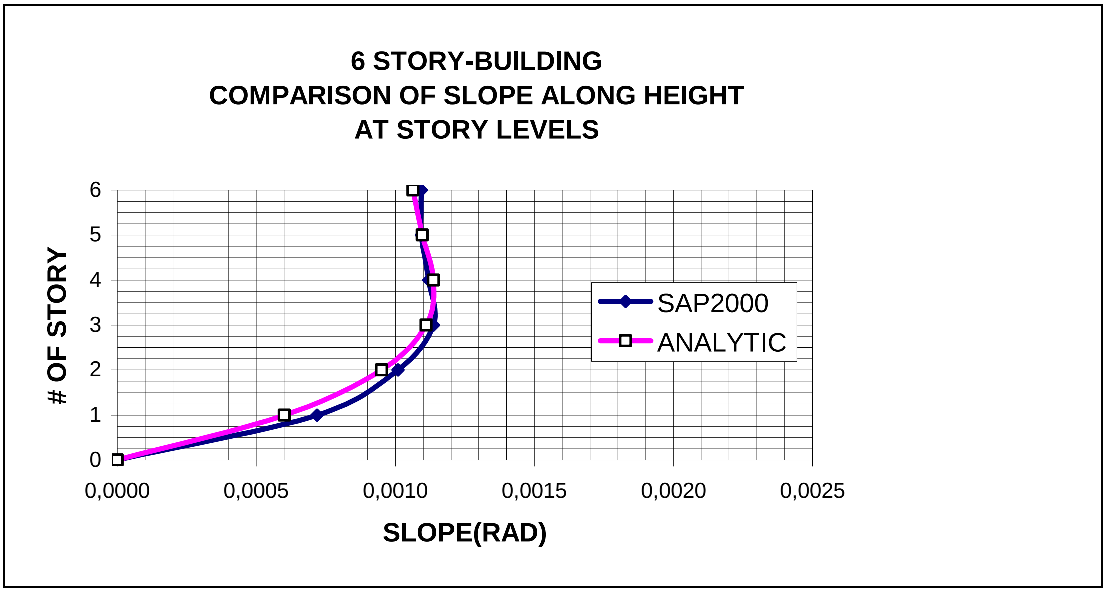


Figure 3.36 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (6 Story-Mixed Structure Example2)

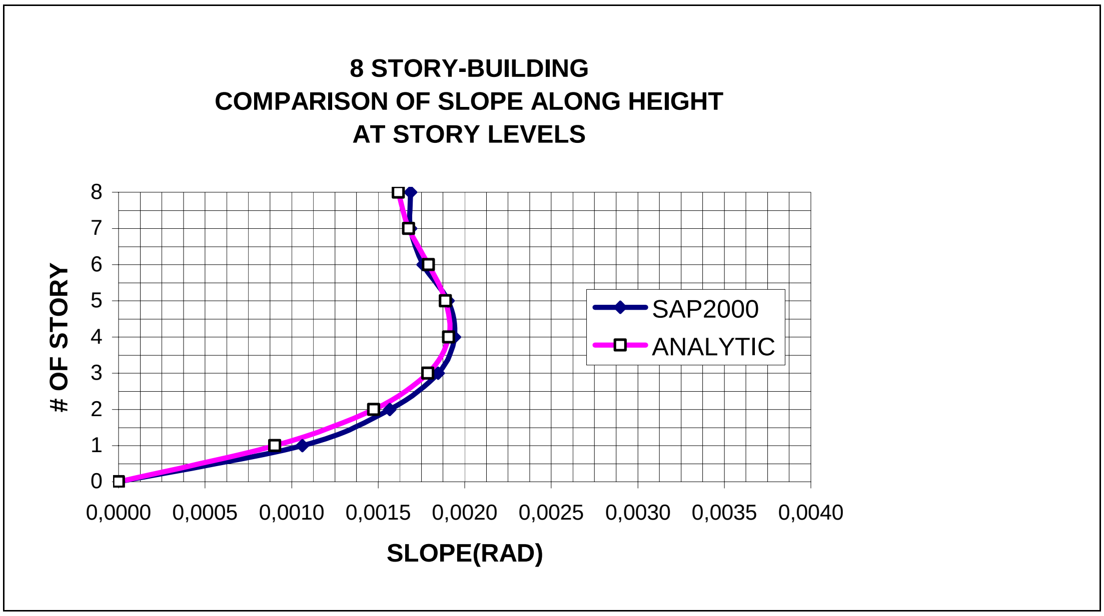


Figure 3.37 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (8 Story-Mixed Structure Example2)

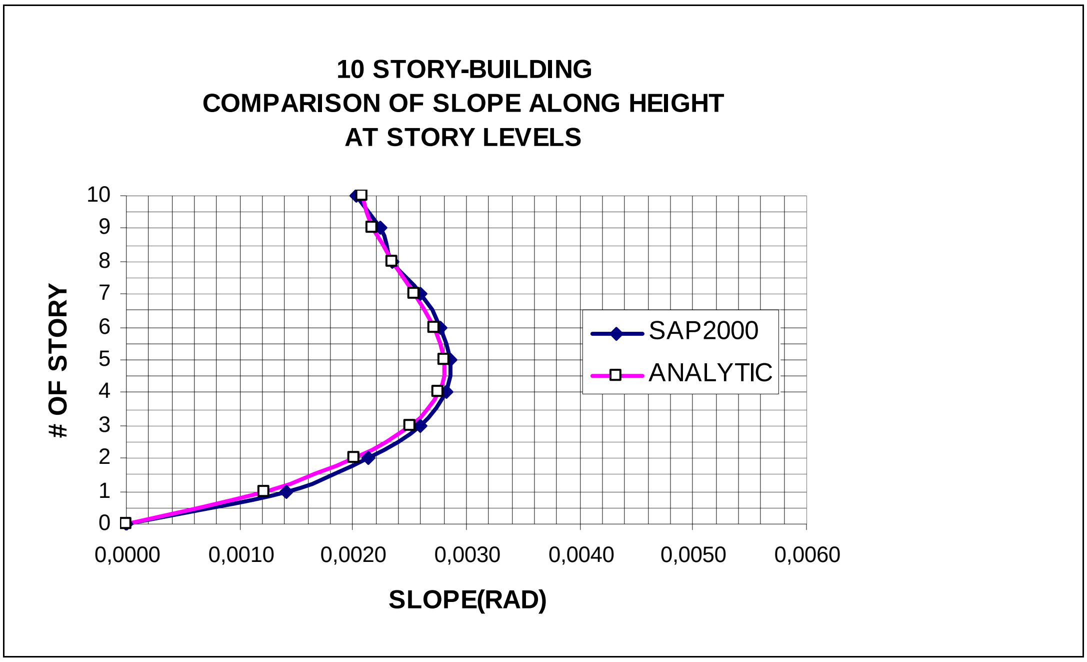


Figure 3.38 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (10 Story-Mixed Structure Example2)

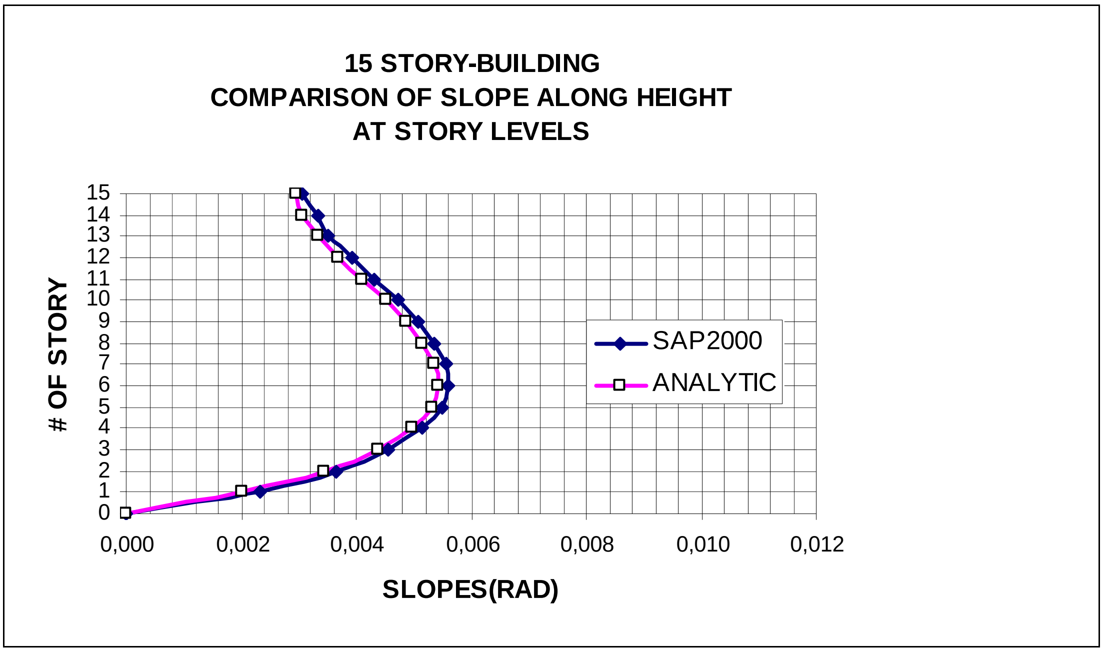


Figure 3.39 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (15 Story-Mixed Structure Example2)

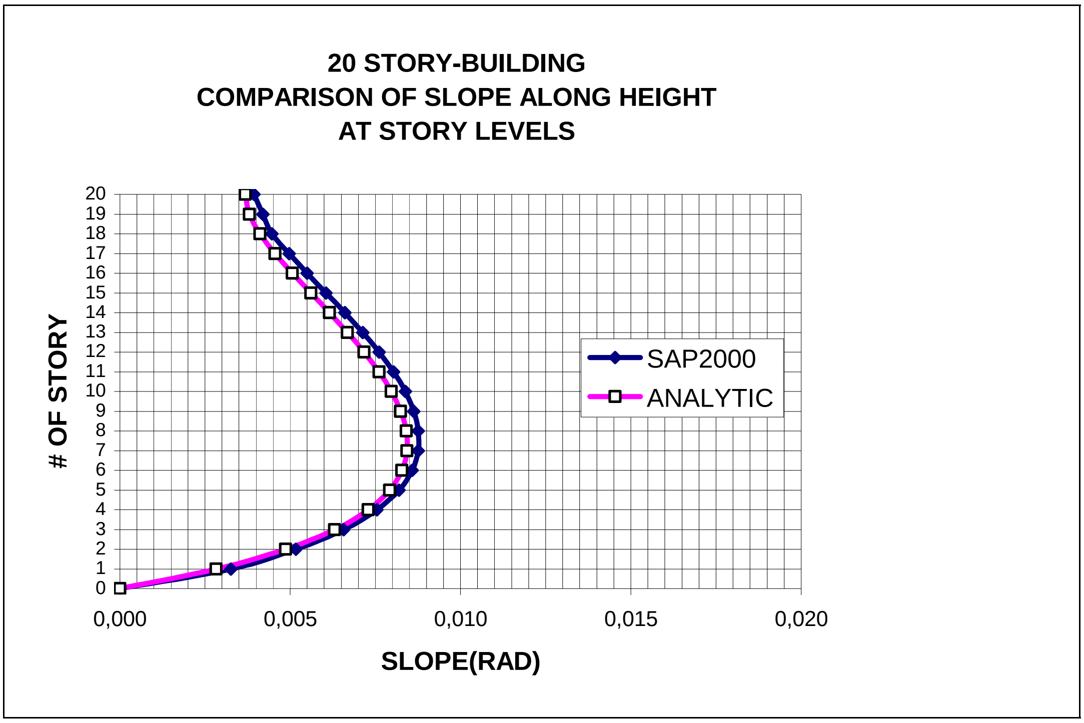


Figure 3.40 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (20 Story-Mixed Structure Example2)

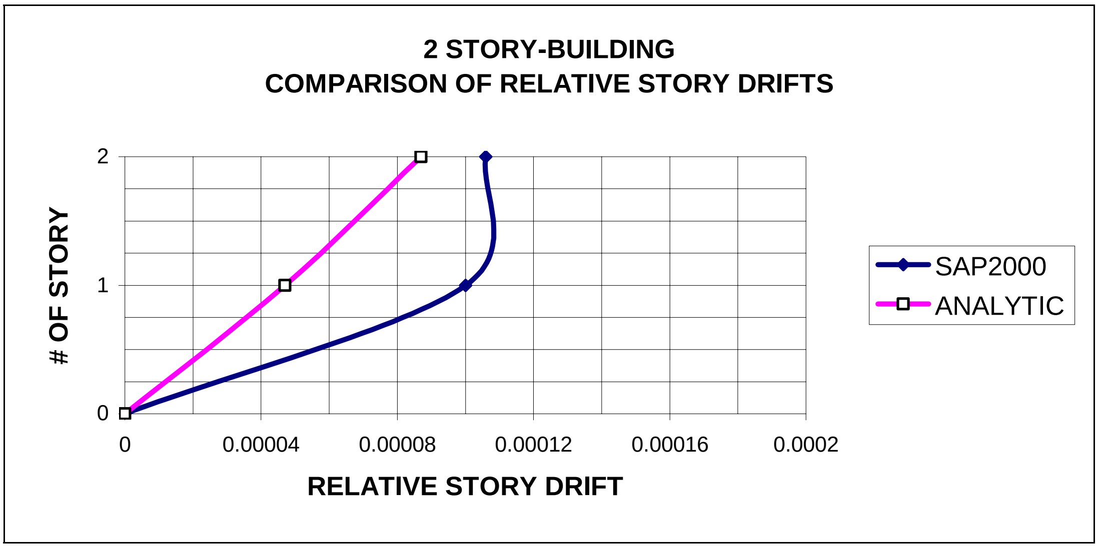


Figure 3.41 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 2 Story-Mixed Structure Example2)

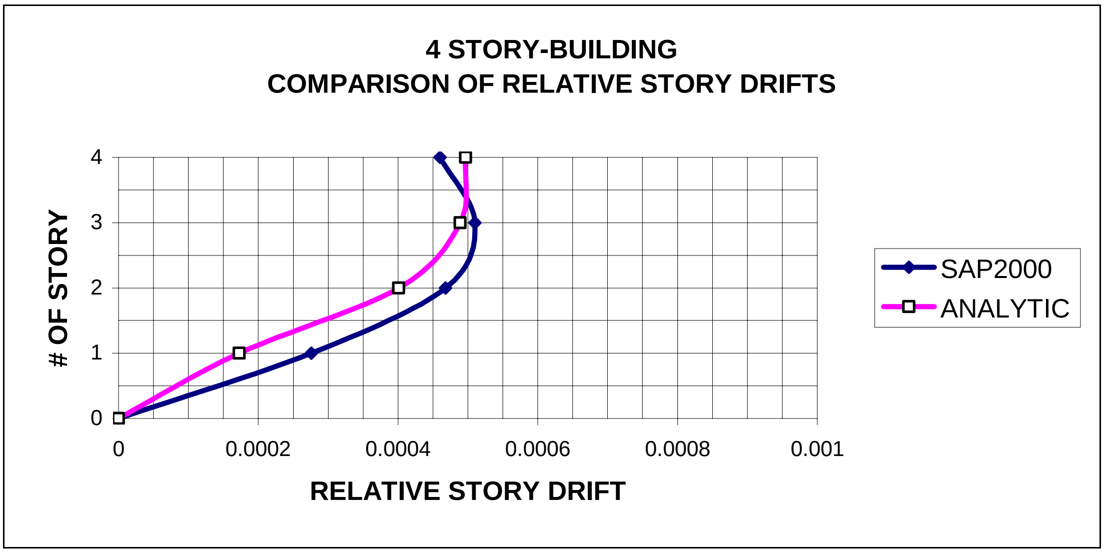


Figure 3.42 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 4 Story-Mixed Structure Example2)

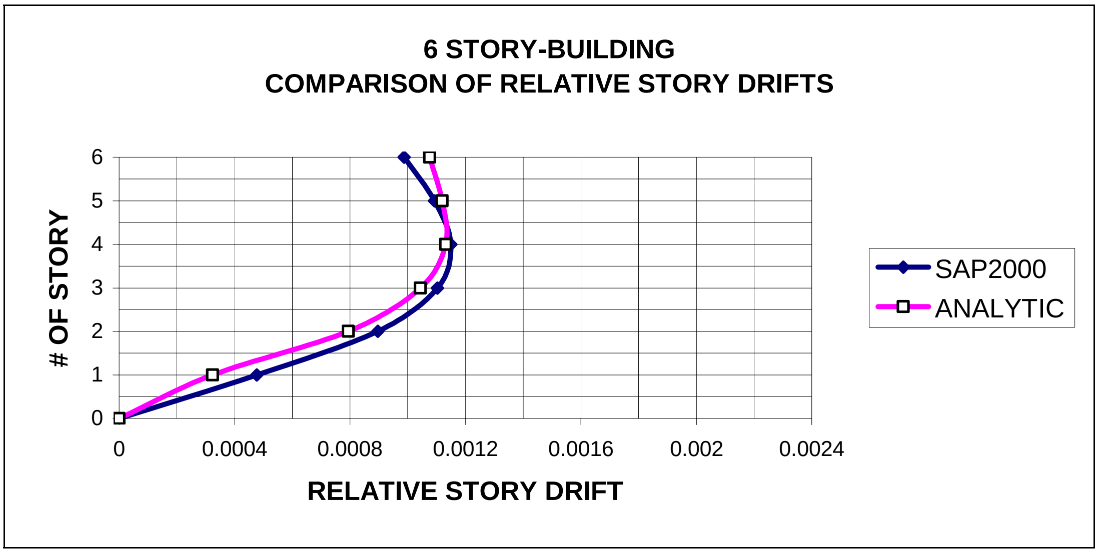


Figure 3.43 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 6 Story-Mixed Structure Example2)

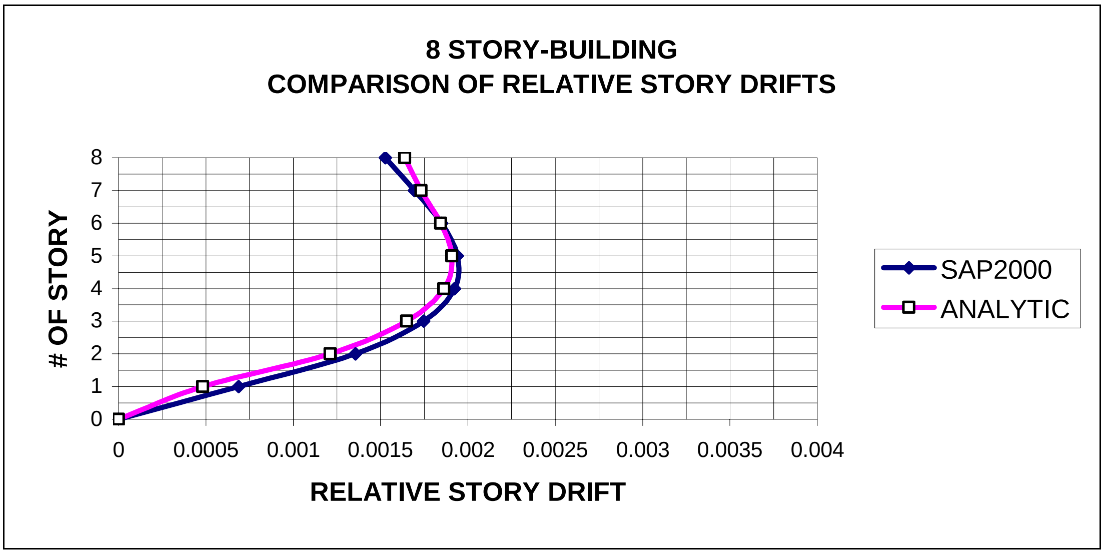


Figure 3.44 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 8 Story-Mixed Structure Example2)

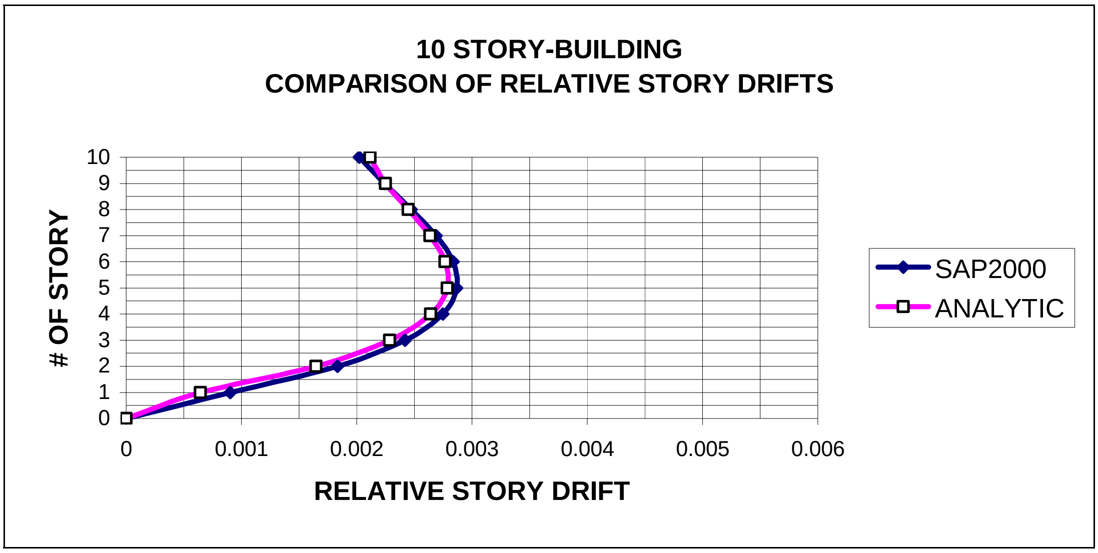


Figure 3.45 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 10 Story-Mixed Structure Example2)

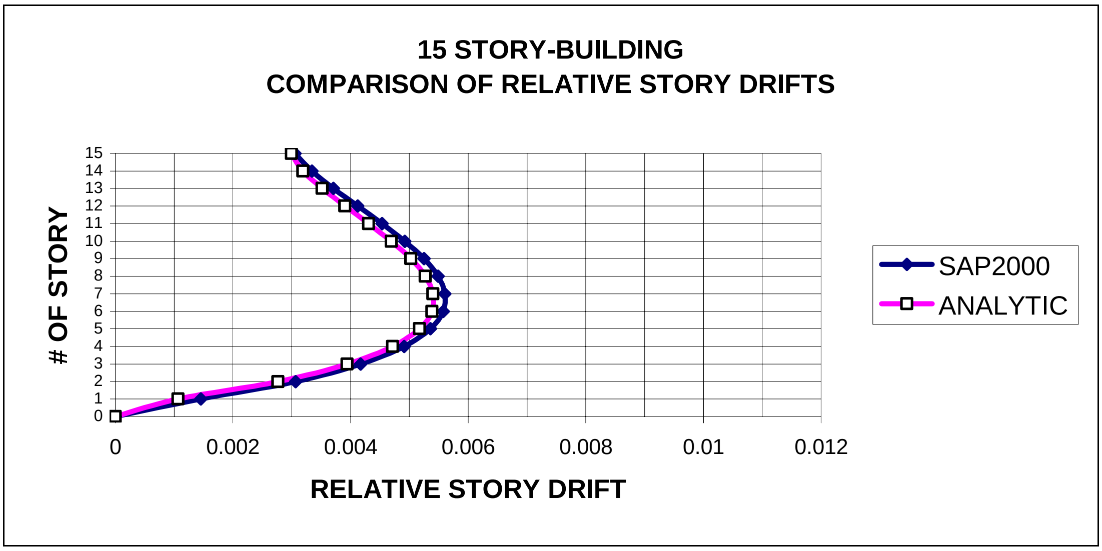


Figure 3.46 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 15 Story-Mixed Structure Example2)

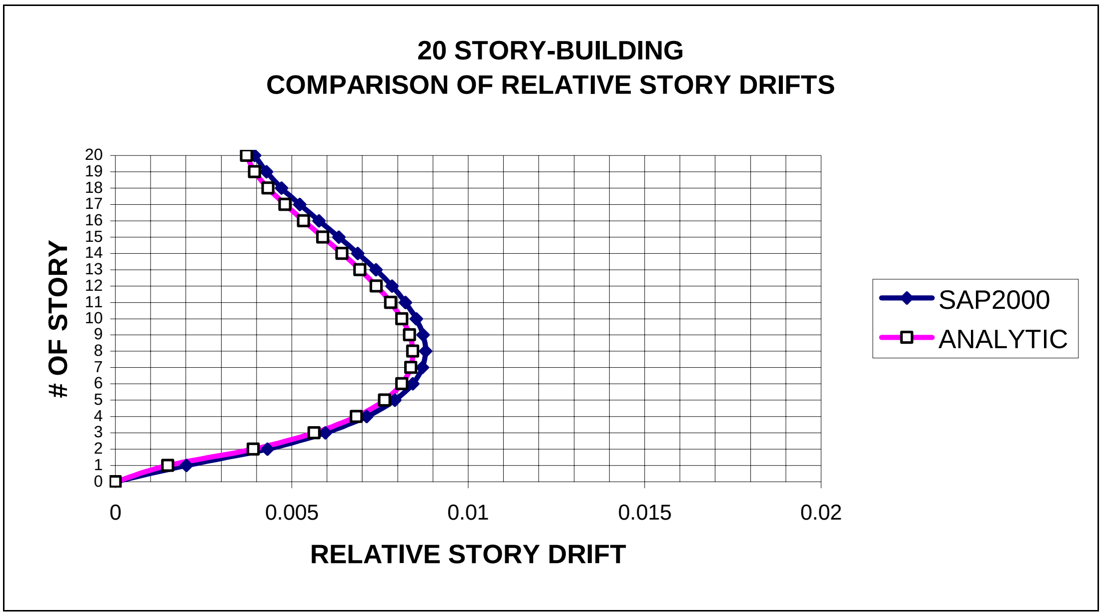


Figure 3.47 Comparisons of Relative Story Drifts as Determined by SAP2000 and Analytical Model (for 20 Story-Mixed Structure Example2)

CHAPTER 4

PROCEDURE FOR ANALYTICAL METHOD OF ANALYSIS OF FRAMED STRUCTURES (FOR CONCENTRATED LATERAL LOAD AT THE TOP)

4.1 ANALYTICAL MODEL OF FRAMED STRUCTURE AS SHEAR BEAM FOR CONCENTRATED LATERAL LOAD AT THE TOP

Figure 4.1 represents the continuous shear beam model of a framed structure, subject to continuous lateral load of f(x).

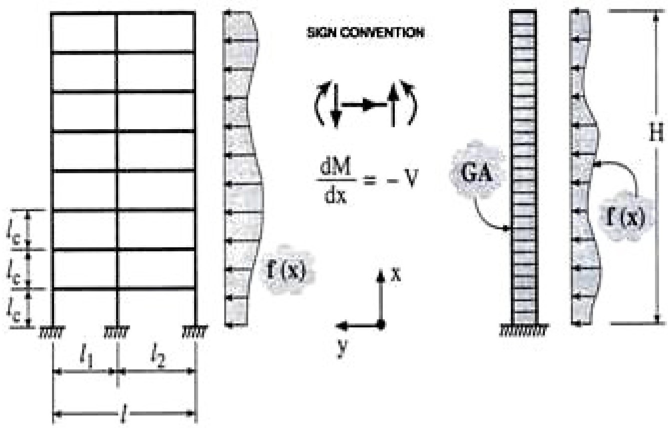


Figure 4.1 Continuous Shear Beam Model of a Framed Structure [2]

Figure 4.2 illustrates the continuous shear beam model of the same framed structure, shown in Figure 4.1, subject to a concentrated lateral load of F at the top of structure.

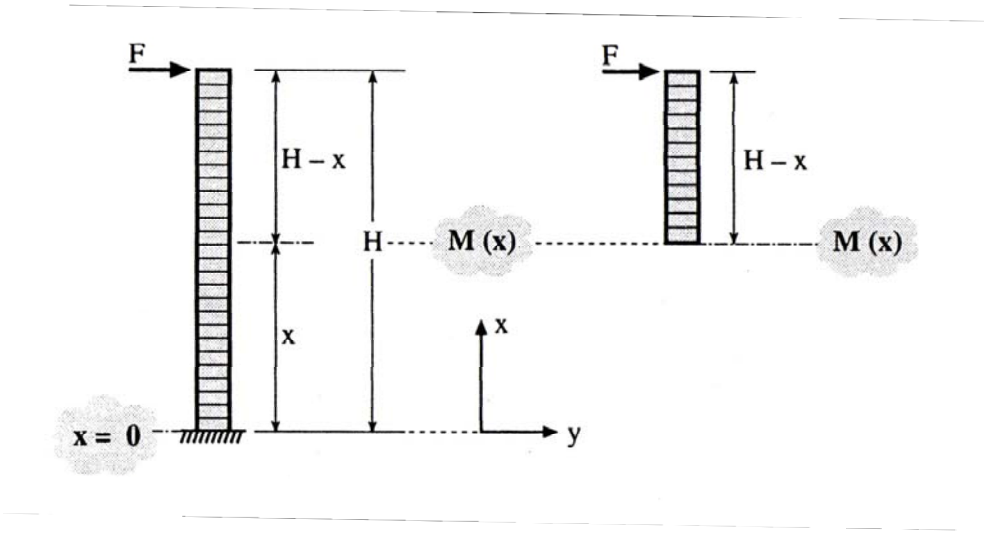


Figure 4.2 Continuous Shear Beam Model Subject to Lateral Load of F at the top [2]

The differential equation of the continuous shear beam model shown in Figure 4.1 can be obtained by differentiating the lateral sway equation, given in Eqn.4.1.

$$y = \frac{1}{GA}\int_0^x V(x).dx \tag{4.1}$$

$$y' = \frac{1}{GA}.V(x) \tag{4.2}$$

$$GA.(y'') = -f(x) \tag{4.3}$$

The solution of this differential equation is given in Eqn.4.4 & Eqn.4.5.

$$y = \frac{1}{GA}\int_0^x V(x).dx = \frac{-[M(x) - M(0)]}{GA} \tag{4.4}$$

$$y = \frac{M(0) - M(x)}{GA} \tag{4.5}$$

where

M (x) = Moment at any height of the shear beam model due to external load

M (0) = Moment at the bottom of the shear beam model (i.e. at x=0)

GA = Shear Rigidity of the framed structure, assumed constant over the height

The solution of the continuous shear beam model of the framed structure subject to a concentrated lateral load of F at the top, shown in Figure 4.2, can be obtained as below.

M (0) = F.(H)

M (x) = F.(H-x) = F.H.(1-k)

where

k= x / H

$$y = \frac{1}{GA}\left[F.H - F.H.(1-k)\right] = \frac{F.H.k}{GA} \qquad (4.6)$$

Therefore, the lateral displacement profile of the shear beam model subject to a concentrated lateral load of F at the top of the structure is expressed as in Eqn.4.7 by neglecting the axial deformations in columns.

$$y = \frac{F.H}{GA}.k = \frac{F}{GA}.x \qquad (4.7)$$

where

GA = equivalent shear stiffness of the continuous shear beam model

k = x / H

Finally, the slope along height of the structure can be expressed as in Eqn.4.8.

$$y' = \frac{F}{GA} \qquad (4.8)$$

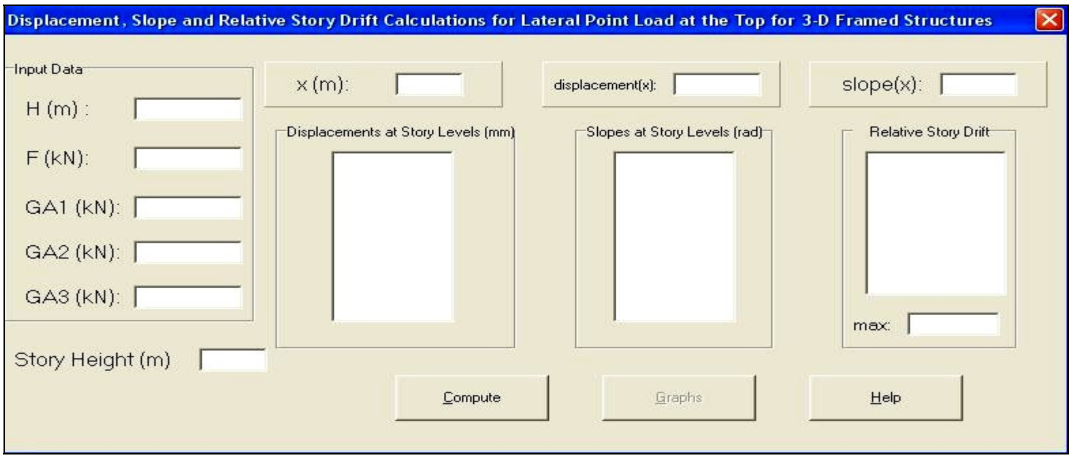
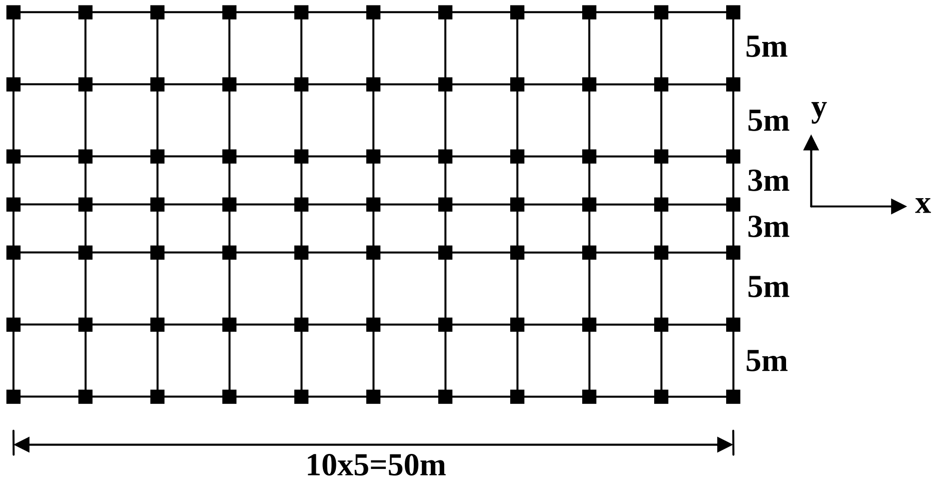


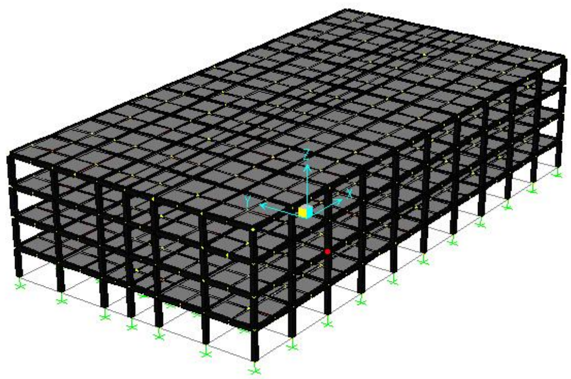
Figure 4.3 Executable "Borland Delphi" Program to Calculate Displacements, Slopes and Relative Story Drifts for Framed Structures

4.2 ASSESSING THE VALIDITY OF THE ANALYTICAL MODEL



All columns : 400x400 mm
All beams : 250x450 mm
Slab thickness : 120 mm
All storey heights : 3 m
g (additional) : 2.0 kN/m^2
q (additional) : 3.5 kN/m^2

(a)



(b)

Figure 4.4 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 validity of the analytical model developed was tested on the same 3D-framed structure (with different number of stories) shown in Figure 4.4 by comparing the results, which are determined by using SAP2000 and analytical method. For simplicity, the concentrated load F at the top of structure was assumed as 1 000 kN.

All analytical results can be obtained easily by using the executable Borland Delphi program developed, which is shown in Figure 4.3.

GA along x-direction can be calculated for the 3-D framed structure shown in Figure 4.3 as follows;

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

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

$$I_{beam} = \alpha\left[\frac{1}{12}(0.25)(0.45)^3\right] = 0.001898437\alpha \quad m^4$$

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

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

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

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.

Relative story drift is calculated by the equation defined in Turkish Earthquake Code (1997) [1] as expressed in Eqn.4.9.

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

$$\frac{\Delta_i}{h_i} = \frac{d_i - d_{i-1}}{h_i} \text{ (Relative Story Drift)} \qquad (4.9)$$

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.4.10 a & b.

$$(\Delta_i)_{max} / h_i \leq 0.0035 \qquad (4.10\text{ a})$$

$$(\Delta_i)_{max} / h_i \leq 0.02 / R \qquad (4.10\text{ b})$$

In the cases where the conditions specified by Eqns.4.10 a & 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.

4.3 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 4.1 and Table 4.2, respectively. On the other hand, comparison of lateral displacements is also shown graphically from Figure 4.4 to Figure 4.10 while the comparison of slope along height at story levels is shown from Figure 4.11 to Figure 4.17. Finally, the comparison of relative story drifts is shown graphically from Figure 4.18 to Figure 4.24.

Table 4.1 Comparisons 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	1.96	2.23	13.78	0.00037	0.00043	16.2
1	0.86	0.87	1.16	0.00029	0.00029	0
				max=0.00037	**max=0.00043**	**16.2**
4	4.67	5.15	10.28	0.00038	0.00043	13.2
3	3.52	3.34	5.11	0.00044	0.00043	0.1
2	2.19	2.23	1.83	0.00044	0.00043	0.1
1	0.88	0.87	1.14	0.00029	0.00029	0
				max=0.00044	**max=0.00043**	**0.1**
6	7.43	7.73	4.04	0.00039	0.00043	10.3
5	6.26	6.44	2.88	0.00045	0.00043	4.4
4	4.92	5.15	4.67	0.00046	0.00043	6.5
3	3.55	3.34	5.92	0.00045	0.00043	4.4
2	2.19	2.23	1.83	0.00044	0.00043	0.1
1	0.88	0.87	1.14	0.00029	0.00029	0
				max=0.00046	**max=0.00043**	**6.5**
8	10.21	10.31	0.98	0.00039	0.00043	10.3
7	9.04	9.02	0.22	0.00045	0.00043	4.4
6	7.68	7.73	0.65	0.00046	0.00043	6.5
5	6.31	6.44	2.06	0.00046	0.00043	6.5
4	4.93	5.15	4.46	0.00046	0.00043	6.5
3	3.56	3.34	6.18	0.00045	0.00043	4.4
2	2.20	2.23	1.36	0.00044	0.00043	0.1
1	0.88	0.87	1.14	0.00029	0.00029	0
				max=0.00046	**max=0.00043**	**6.5**
10	13.04	12.88	1.23	0.00040	0.00043	7.5
9	11.85	11.59	2.19	0.00045	0.00043	4.4
8	10.49	10.31	1.72	0.00046	0.00043	6.5
7	9.10	9.02	0.88	0.00046	0.00043	6.5
6	7.71	7.73	0.26	0.00046	0.00043	6.5
5	6.32	6.44	1.90	0.00046	0.00043	6.5
4	4.94	5.15	4.25	0.00046	0.00043	6.5
3	3.56	3.34	6.18	0.00045	0.00043	4.4
2	2.20	2.23	1.36	0.00044	0.00043	0.1
1	0.88	0.87	1.14	0.00029	0.00029	0
				max=0.00046	**max=0.00043**	**6.5**

Table 4.1 Comparisons 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	20.34	19.33	4.97	0.00041	0.00043	4.8
14	19.11	18.04	5.60	0.00047	0.00043	8.5
13	17.70	16.75	5.37	0.00047	0.00043	8.5
12	16.28	15.46	5.04	0.00048	0.00043	10.4
11	14.85	14.17	4.58	0.00048	0.00043	10.4
10	13.42	12.88	4.02	0.00047	0.00043	8.5
9	12.00	11.59	3.42	0.00047	0.00043	8.5
8	10.58	10.31	2.55	0.00047	0.00043	8.5
7	9.17	9.02	1.64	0.00047	0.00043	8.5
6	7.76	7.73	0.39	0.00047	0.00043	8.5
5	6.36	6.44	1.26	0.00047	0.00043	8.5
4	4.96	5.15	3.83	0.00046	0.00043	6.5
3	3.58	3.34	6.70	0.00046	0.00043	6.5
2	2.20	2.23	1.36	0.00044	0.00043	0.1
1	0.88	0.87	1.14	0.00029	0.00029	0
				max=0.00048	**max=0.00043**	**10.4**
20	28.00	25.77	7.96	0.00042	0.00043	0.1
19	26.73	24.48	8.42	0.00048	0.00043	10.4
18	25.28	23.19	8.27	0.00049	0.00043	12.2
17	23.80	21.90	7.98	0.00049	0.00043	12.2
16	22.33	20.62	7.66	0.00049	0.00043	12.2
15	20.86	19.33	7.33	0.00049	0.00043	12.2
14	19.39	18.04	6.96	0.00049	0.00043	12.2
13	17.92	16.75	6.53	0.00049	0.00043	12.2
12	16.46	15.46	6.08	0.00049	0.00043	12.2
11	15.00	14.17	5.53	0.00048	0.00043	10.4
10	13.55	12.88	4.94	0.00048	0.00043	10.4
9	12.10	11.59	4.21	0.00048	0.00043	10.4
8	10.66	10.31	3.28	0.00048	0.00043	8.5
7	9.23	9.02	2.28	0.00047	0.00043	8.5
6	7.81	7.73	1.02	0.00047	0.00043	8.5
5	6.39	6.44	0.78	0.00047	0.00043	8.5
4	4.98	5.15	3.41	0.00046	0.00043	6.5
3	3.59	3.34	6.96	0.00046	0.00043	6.5
2	2.21	2.23	0.90	0.00044	0.00043	0.1
1	0.88	0.87	1.14	0.00029	0.00029	0
				max=0.00049	**max=0.00043**	**12.2**

Table 4.2 Comparisons 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	0.0003466	0.00043	24.1
1	0.0003733	0.00043	15.2
4	0.0003533	0.00043	21.7
3	0.0004267	0.00043	0.8
2	0.0004433	0.00043	3.0
1	0.0004033	0.00043	6.6
6	0.0003666	0.00043	17.3
5	0.0004300	0.00043	0
4	0.0004533	0.00043	5.1
3	0.0004533	0.00043	5.1
2	0.0004467	0.00043	3.7
1	0.0004033	0.00043	6.6
8	0.0003666	0.00043	17.3
7	0.0004300	0.00043	0
6	0.0004600	0.00043	6.5
5	0.0004567	0.00043	5.8
4	0.0004600	0.00043	6.5
3	0.0004567	0.00043	5.8
2	0.0004467	0.00043	3.7
1	0.0004033	0.00043	6.6
10	0.0003733	0.00043	15.2
9	0.0004333	0.00043	0.8
8	0.0004633	0.00043	7.2
7	0.0004633	0.00043	7.2
6	0.0004600	0.00043	6.5
5	0.0004633	0.00043	7.2
4	0.0004600	0.00043	6.5
3	0.0004567	0.00043	5.8
2	0.0004500	0.00043	4.4
1	0.0004033	0.00043	6.6

Table 4.2 Comparisons 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	0.0003866	0.00043	11.2
14	0.0004500	0.00043	4.4
13	0.0004733	0.00043	9.1
12	0.0004767	0.00043	9.8
11	0.0004733	0.00043	9.1
10	0.0004767	0.00043	9.8
9	0.0004733	0.00043	9.1
8	0.0004700	0.00043	8.5
7	0.0004700	0.00043	8.5
6	0.0004700	0.00043	8.5
5	0.0004667	0.00043	7.8
4	0.0004633	0.00043	7.2
3	0.0004600	0.00043	6.5
2	0.0004533	0.00043	5.1
1	0.0004033	0.00043	6.6
20	0.0004000	0.00043	7.5
19	0.0004633	0.00043	7.2
18	0.0004900	0.00043	12.2
17	0.0004933	0.00043	12.8
16	0.0004900	0.00043	12.2
15	0.0004900	0.00043	12.2
14	0.0004900	0.00043	12.2
13	0.0004867	0.00043	11.6
12	0.0004867	0.00043	11.6
11	0.0004867	0.00043	11.6
10	0.0004800	0.00043	10.4
9	0.0004833	0.00043	11.0
8	0.0004767	0.00043	9.8
7	0.0004767	0.00043	9.8
6	0.0004733	0.00043	9.1
5	0.0004700	0.00043	8.5
4	0.0004700	0.00043	8.5
3	0.0004600	0.00043	6.5
2	0.0004533	0.00043	5.1
1	0.0004067	0.00043	5.7

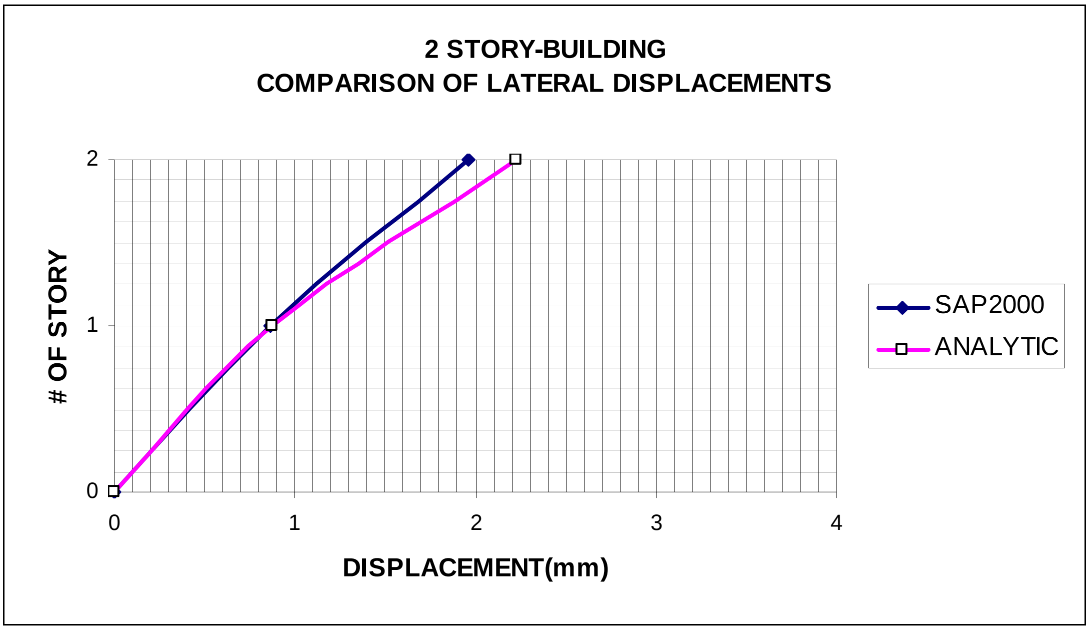


Figure 4.5 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 2 Story-Framed Structure)

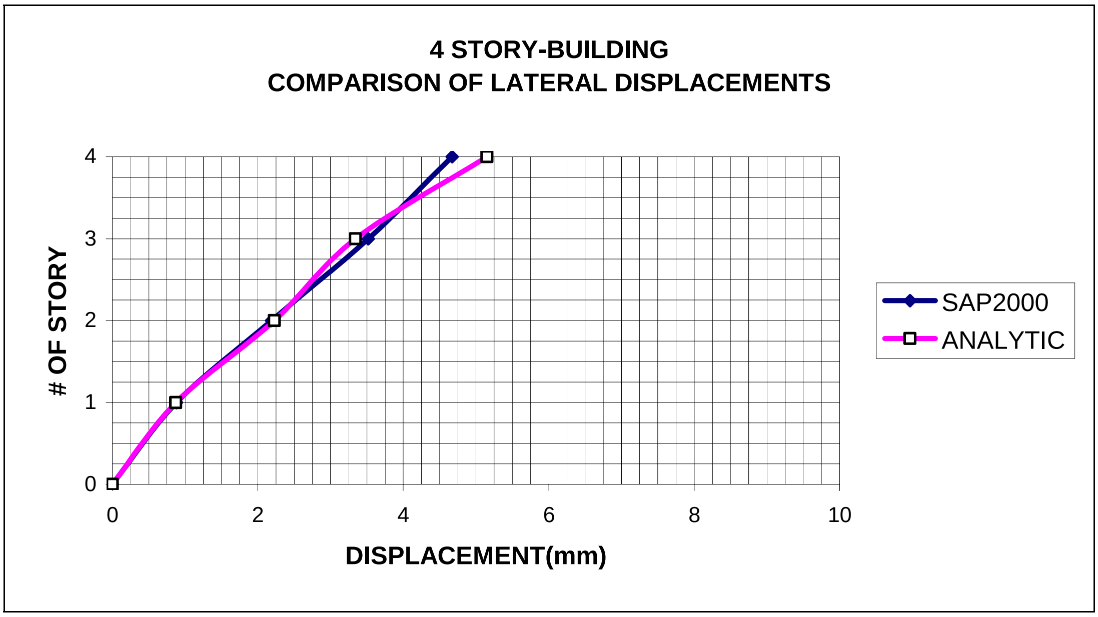


Figure 4.6 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 4 Story-Framed Structure)

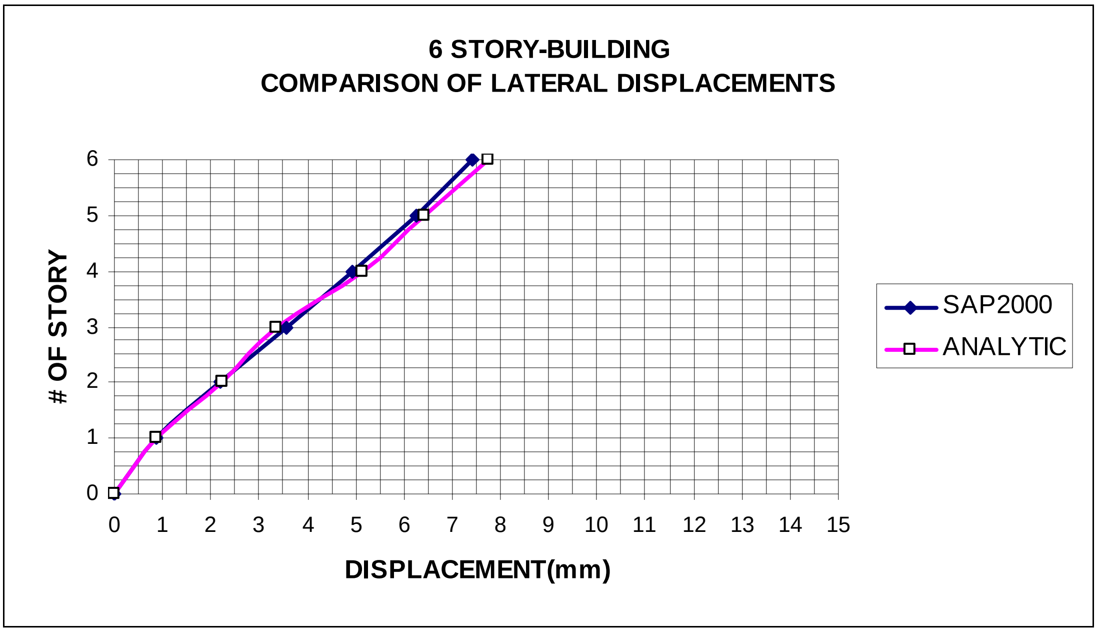


Figure 4.7 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 6 Story-Framed Structure)

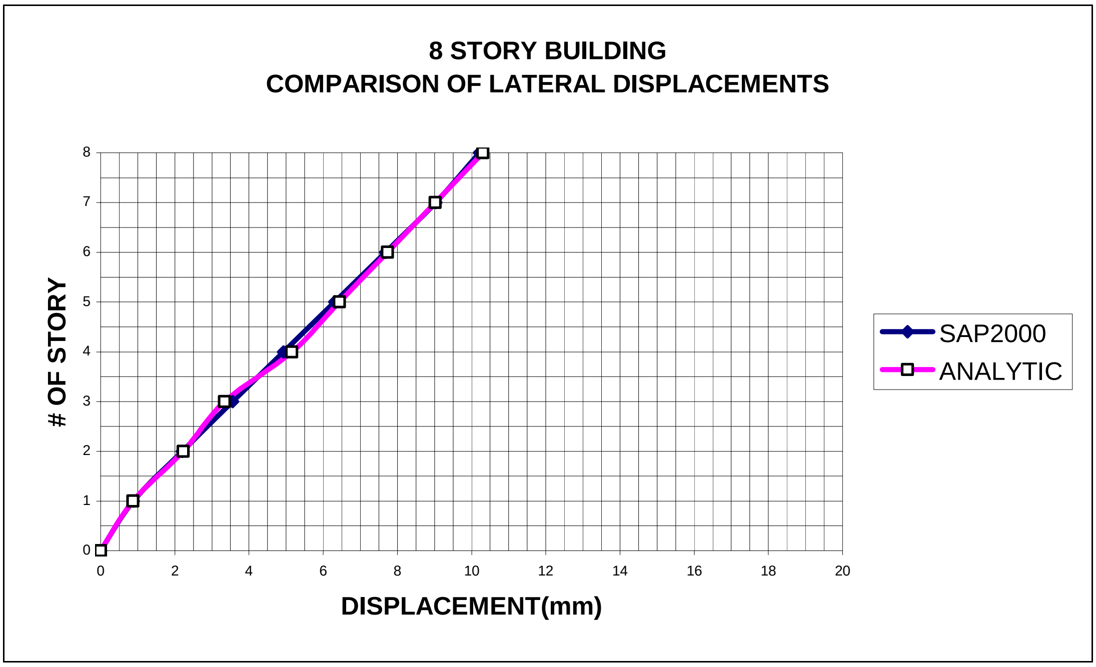


Figure 4.8 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 8 Story-Framed Structure)

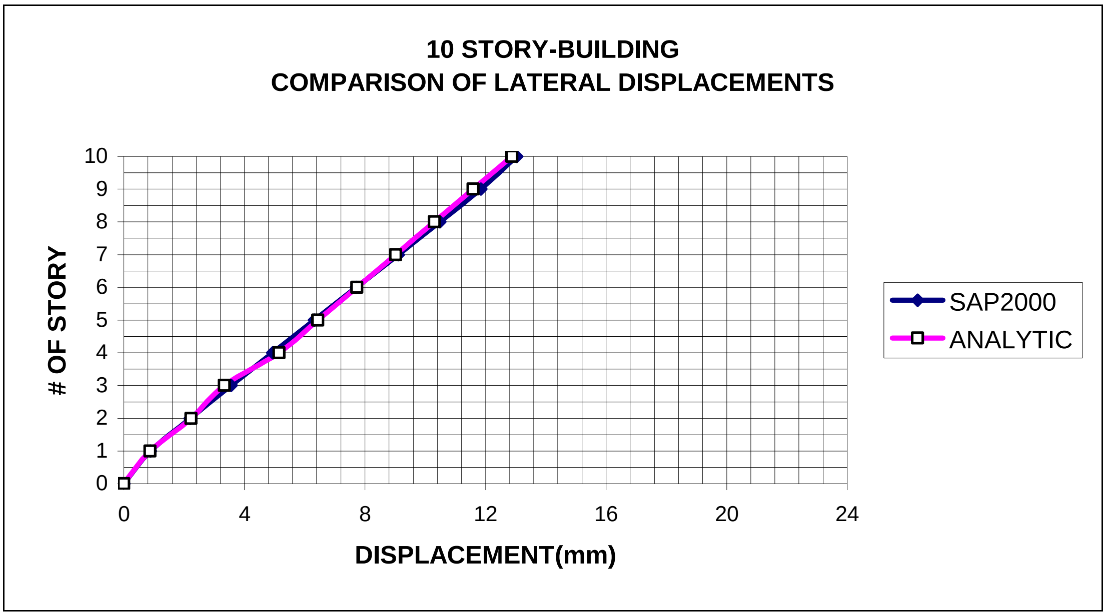


Figure 4.9 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 10 Story-Framed Structure)

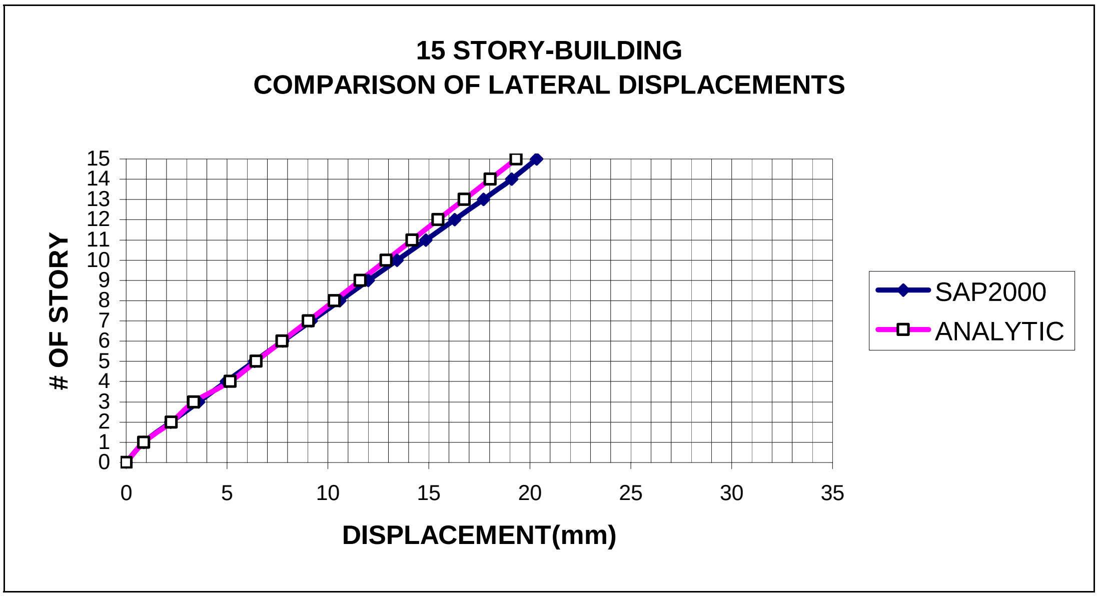


Figure 4.10 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 15 Story-Framed Structure)

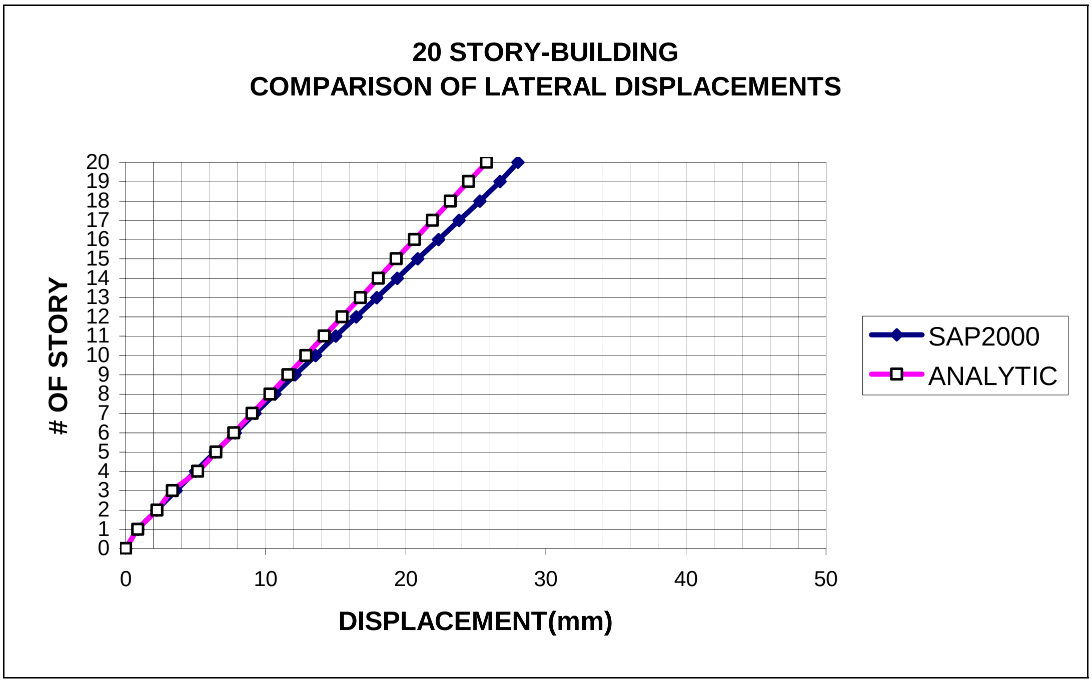


Figure 4.11 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 20 Story-Framed Structure)

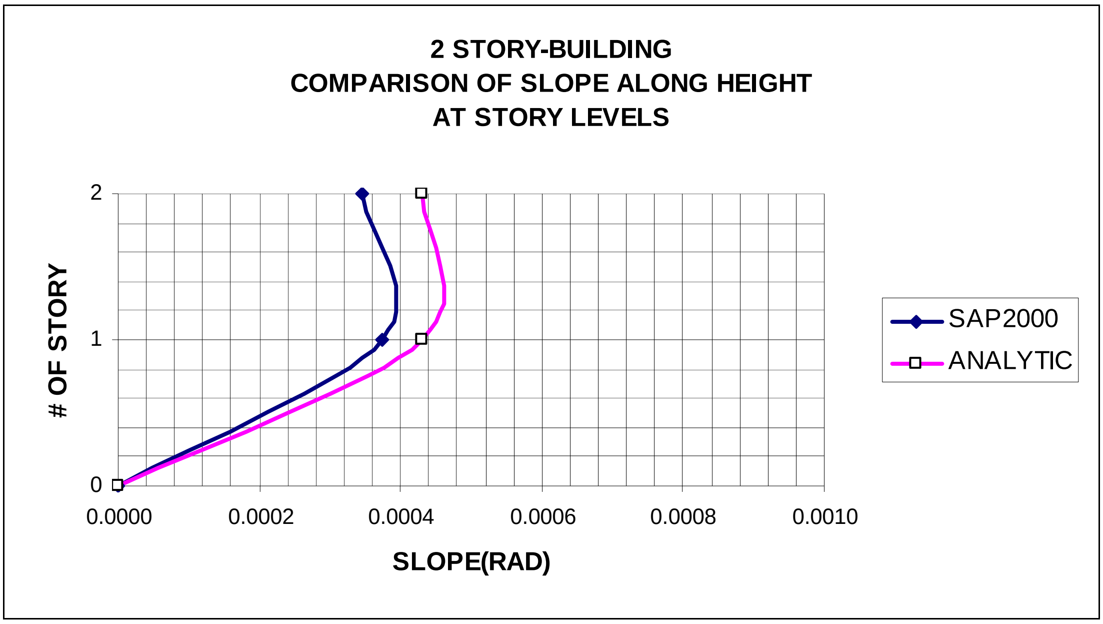


Figure 4.12 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 2 Story-Framed Structure)

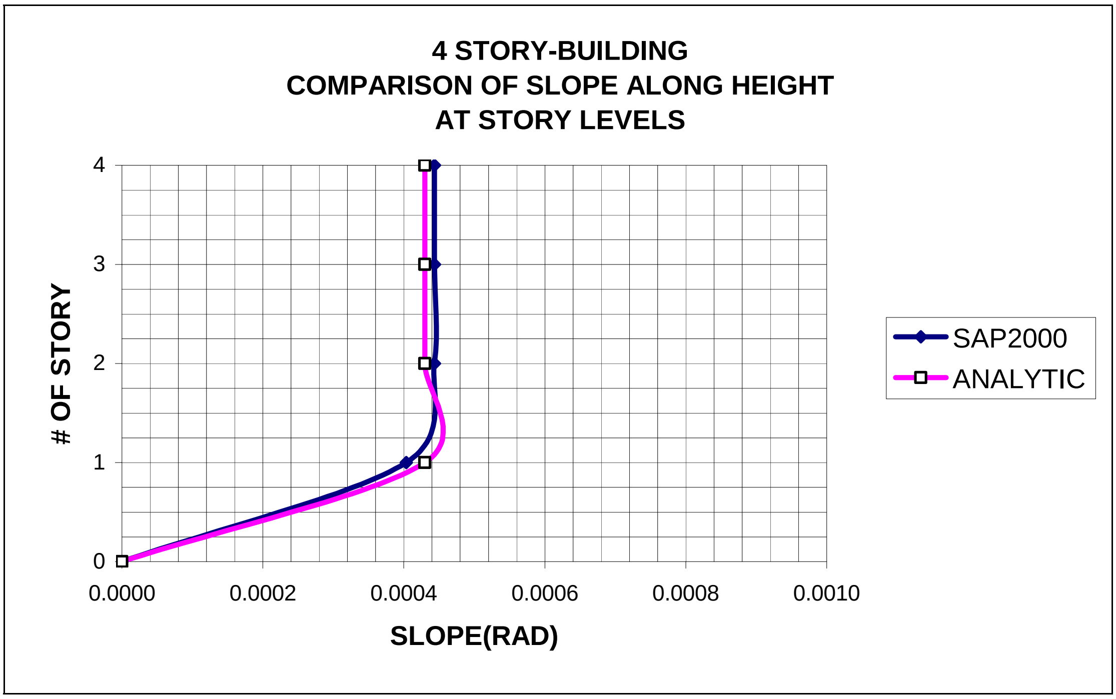


Figure 4.13 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 4 Story-Framed Structure)

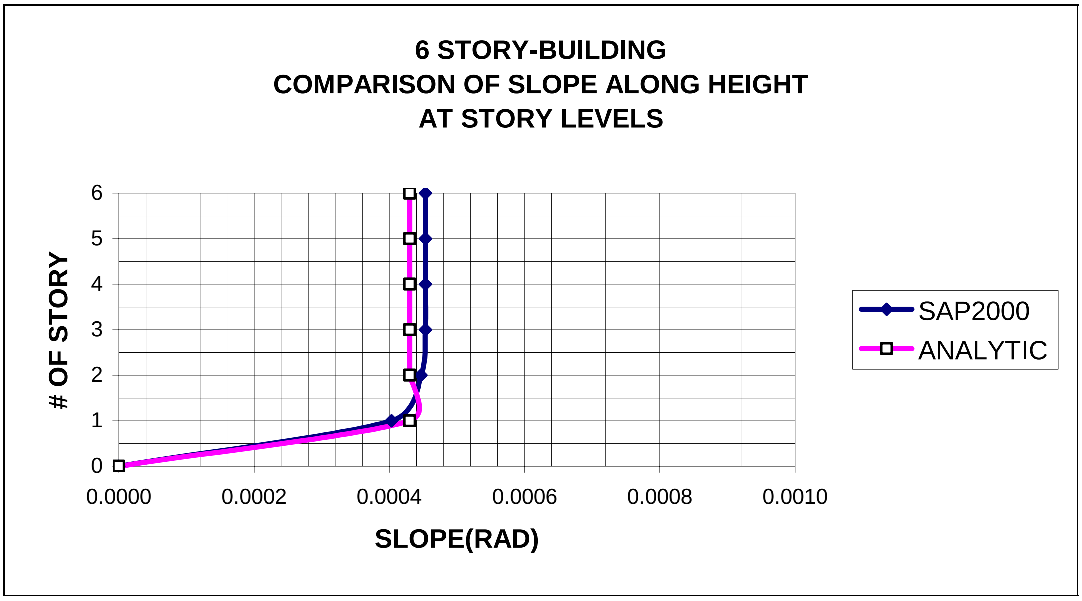


Figure 4.14 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 6 Story-Framed Structure)

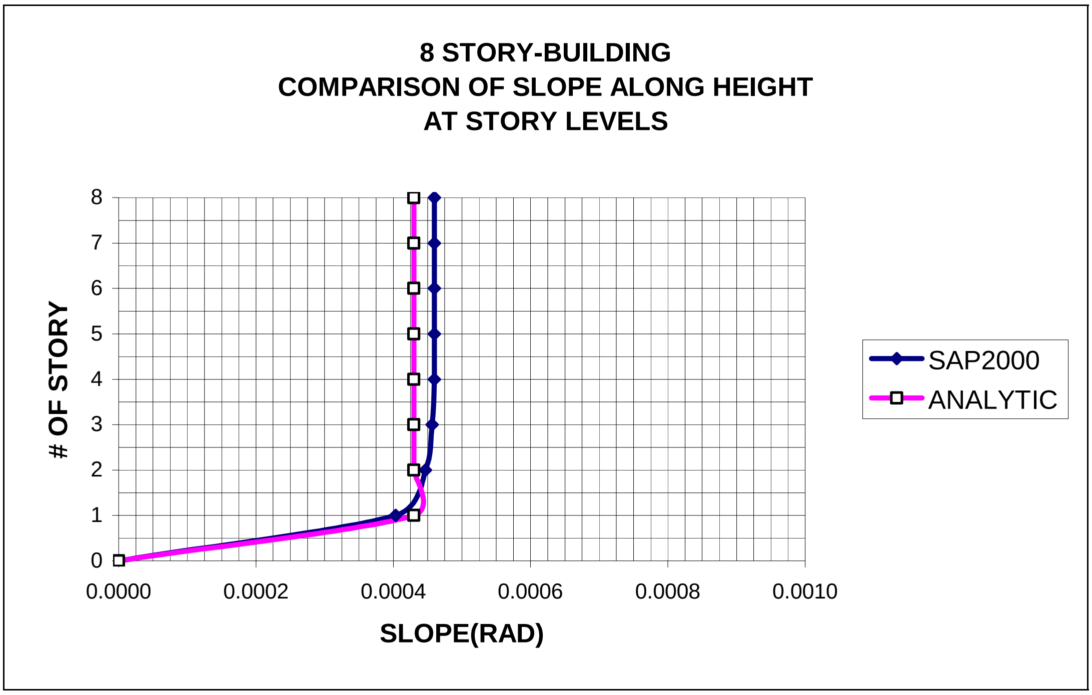


Figure 4.15 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 8 Story-Framed Structure)

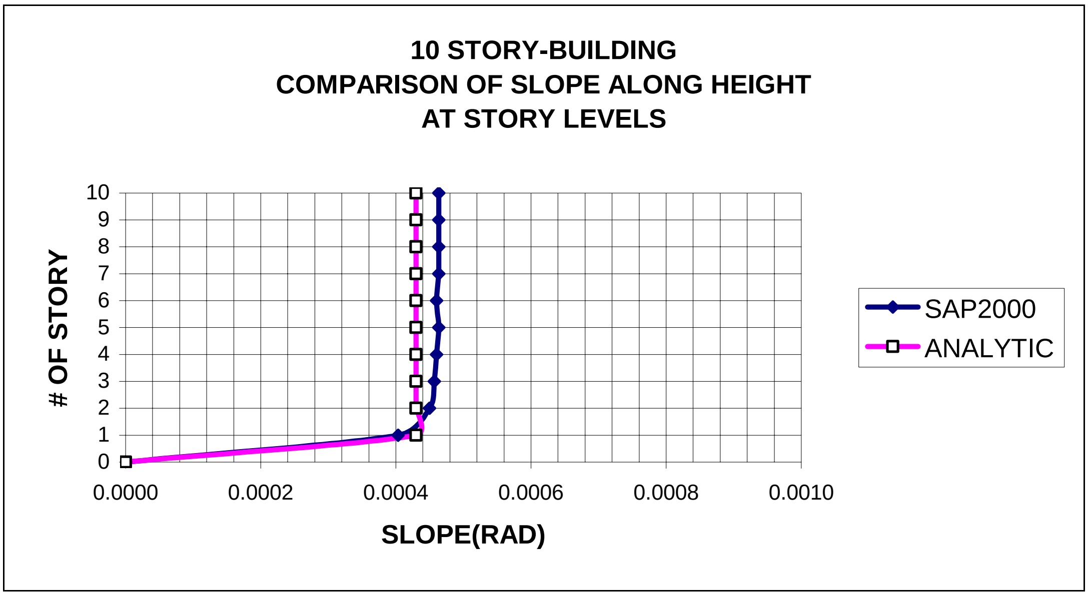


Figure 4.16 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 10 Story-Framed Structure)

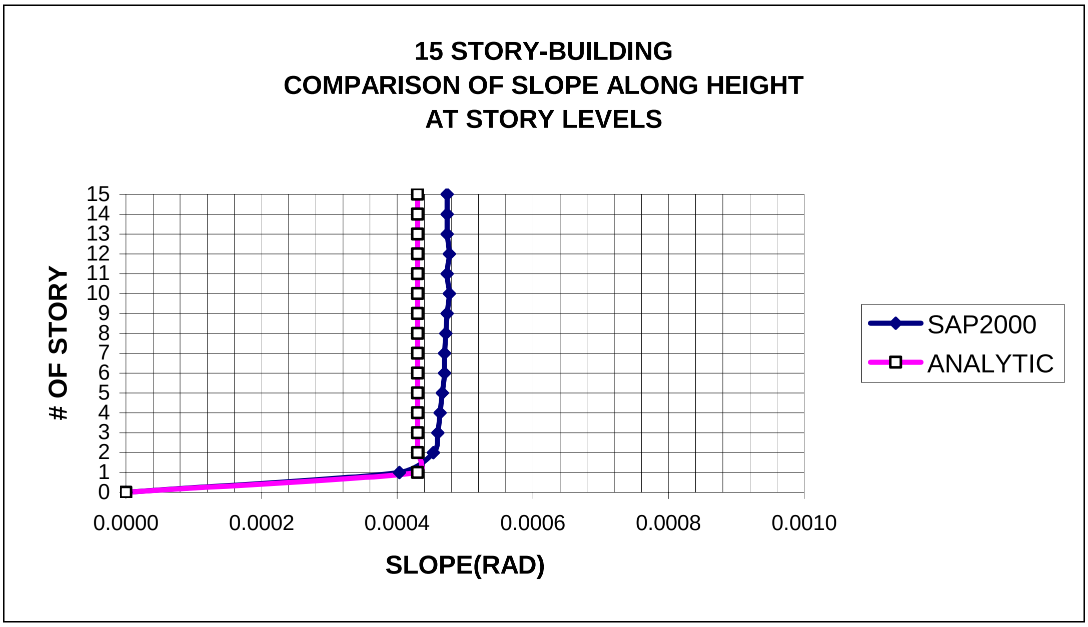


Figure 4.17 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 15 Story-Framed Structure)

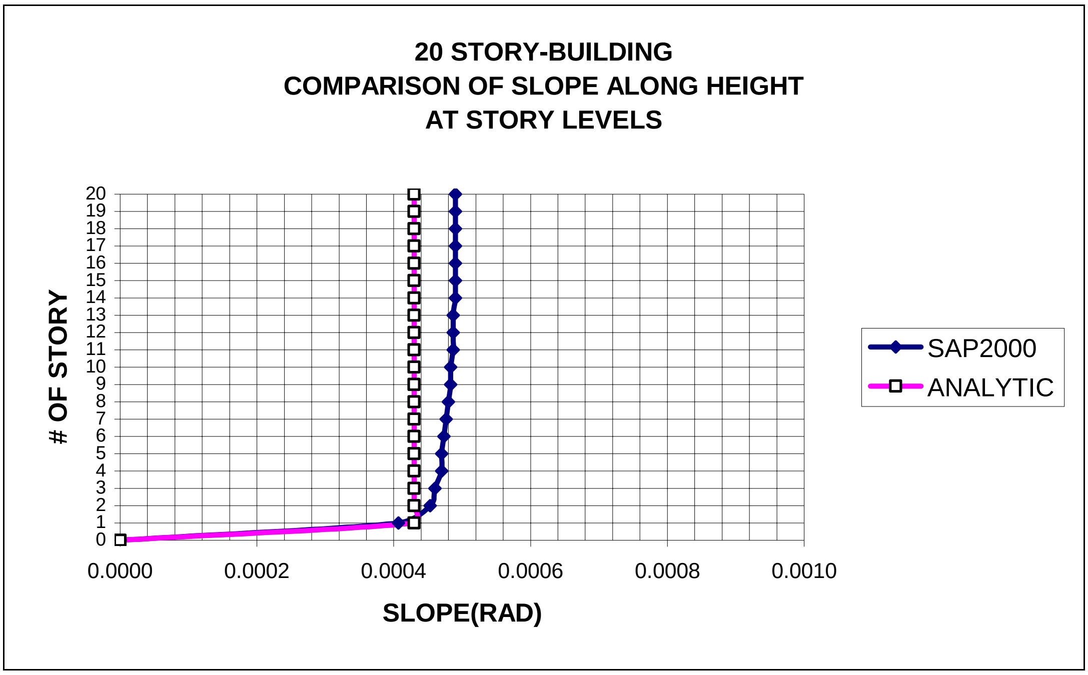


Figure 4.18 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (for 20 Story-Framed Structure)

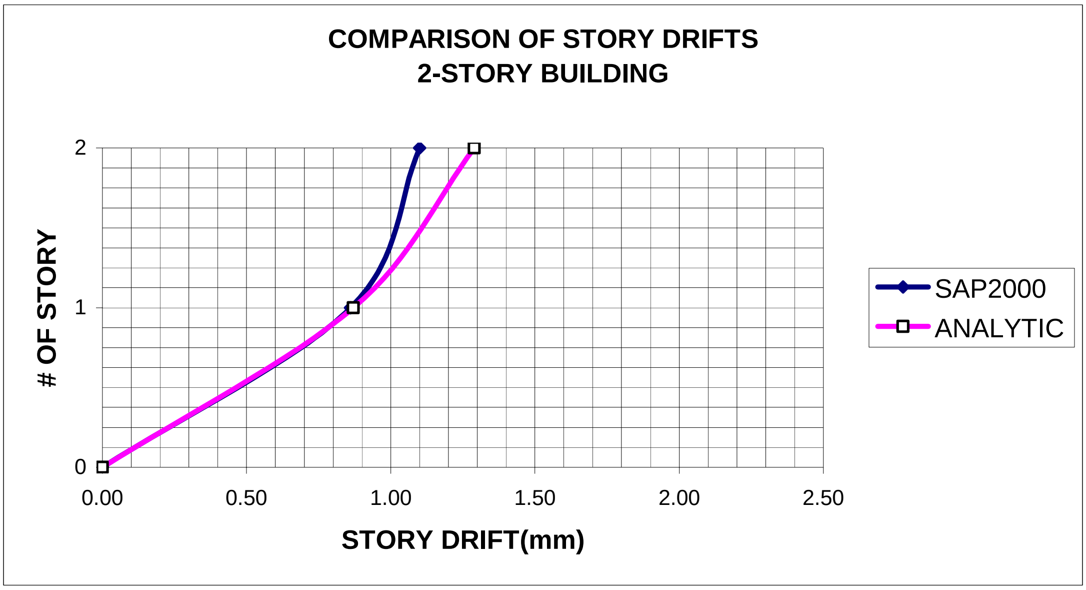


Figure 4.19 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 2 Story-Framed Structure)

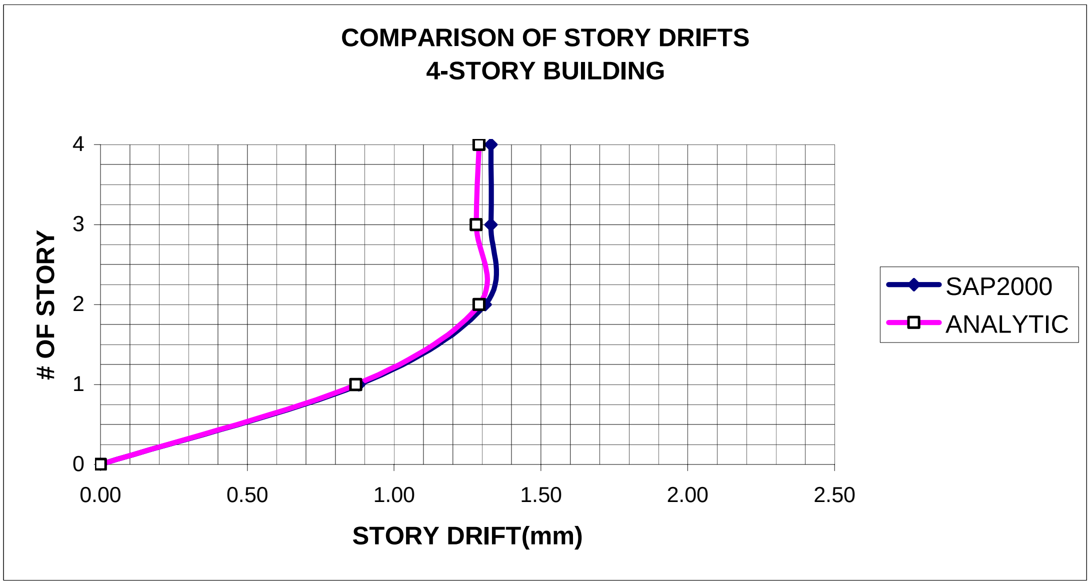


Figure 4.20 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 4 Story-Framed Structure)

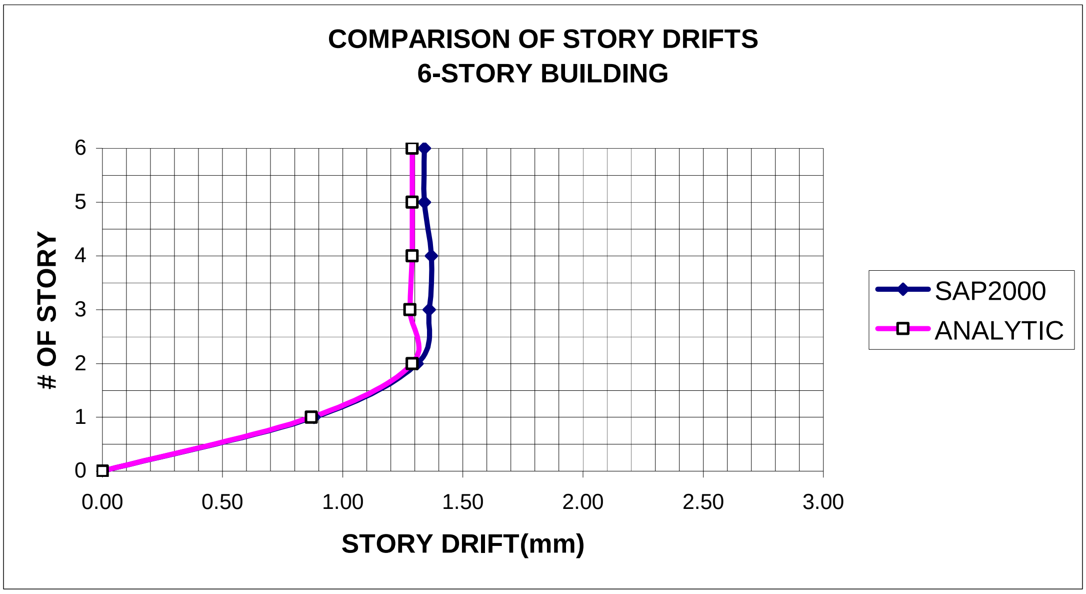


Figure 4.21 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 6 Story-Framed Structure)

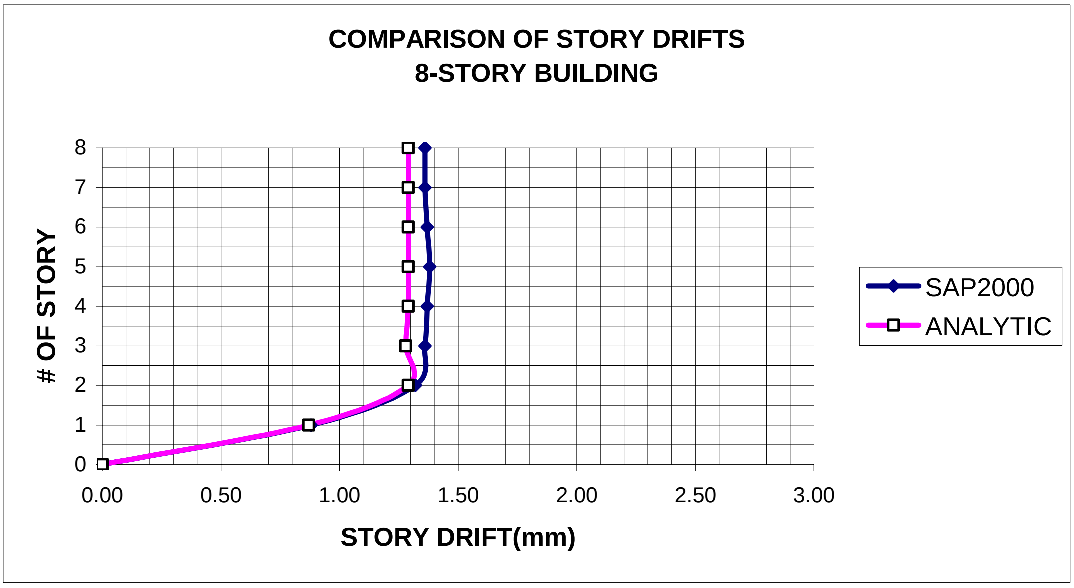


Figure 4.22 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 8 Story-Framed Structure)

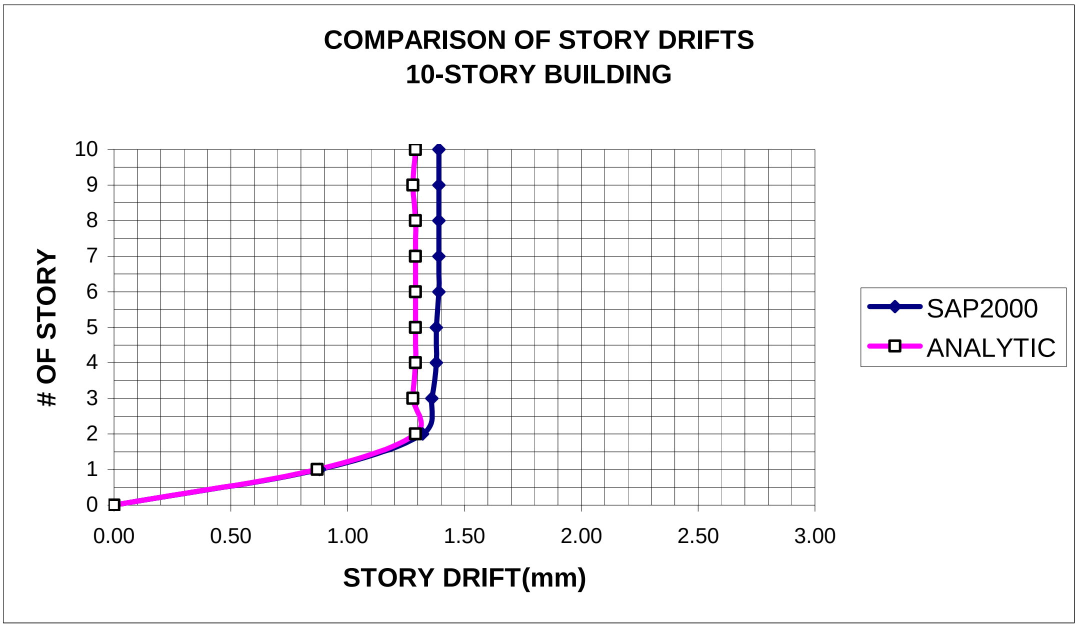


Figure 4.23 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 10 Story-Framed Structure)

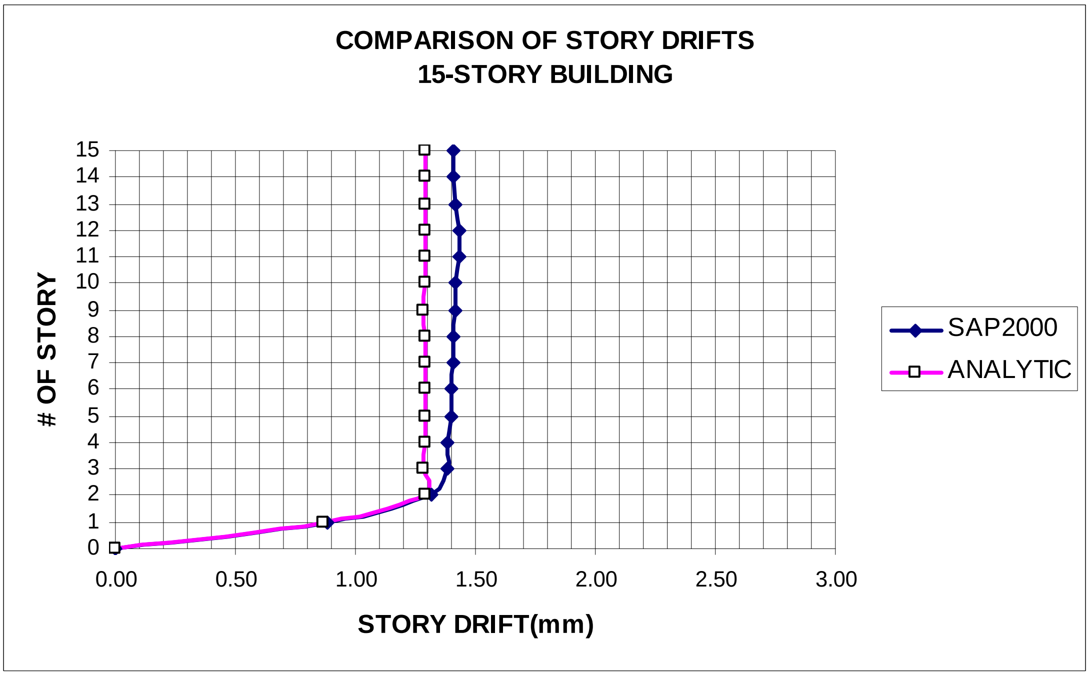


Figure 4.24 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 15 Story-Framed Structure)

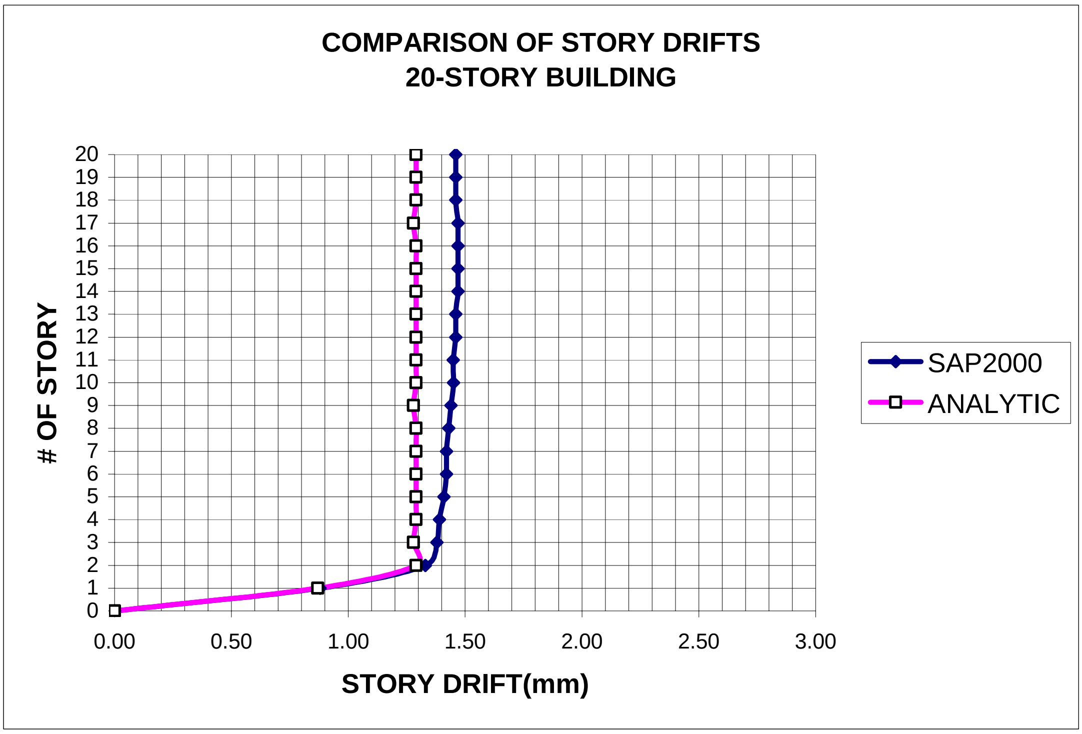


Figure 4.25 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 20 Story-Framed Structure)

CHAPTER 5

PROCEDURE FOR ANALYTICAL METHOD OF ANALYSIS OF MIXED STRUCTURES (FOR CONCENTRATED LATERAL LOAD AT THE TOP)

5.1 SOLUTION OF MIXED STRUCTURES FOR CONCENTRATED LATERAL LOAD AT THE TOP

In case of concentrated lateral load of F at the top of structure, f(x) will be equal to zero and the general differential equation will take the following form.

$$s^2.w^{IV} - w^{II} + \frac{v^2 - 1}{v^2}.M(x) = 0 \tag{5.1}$$

Letting EI.y''= w'' = M(x), Eqn5.1 becomes

$$s^2.M^{II}(x) - M(x) + \frac{v^2 - 1}{v^2}.M(x) = 0 \tag{5.2}$$

The solution of Eqn5.2 is given below.

$$M(x) = A_1.\cosh\varphi + A_2.\sinh\varphi + \frac{v^2 - 1}{v^2}.M(x) \tag{5.3}$$

If we take the first derivative of M(x),

$$M^I(x) = A_1.\frac{1}{s}\sinh\varphi + A_2.\frac{1}{s}\cosh\varphi + \frac{v^2 - 1}{v^2}.M^I(x) \tag{5.4}$$

Moment at any height x of the structure can be written as in Eqn5.5.

$$M(x) = F.(H-x) = F.H.(1-k) \tag{5.5}$$

where

k = x / H

F = Lateral point load at the top of structure

H = Height of building

Taking the derivative of this moment expression yields,

$$\frac{dM(x)}{dx} = -F \tag{5.6}$$

In order to find A_1 and A_2 in Eqn.6.3, following two boundary conditions can be applied.

i. M(H) = 0

ii. M'(H) = -F

Then A_1 and A_2 can be expressed as in Eqn5.7 and Eqn5.8, respectively.

$$A_1 = \frac{F(s)}{v^2}.\tanh\lambda \tag{5.7}$$

$$A_2 = -\frac{F(s)}{v^2} \tag{5.8}$$

Recalling w= K.y, Eqn.5.3 must be integrated twice as expressed in Eqn.5.9 and two more boundary conditions must be applied in order to find the equation of lateral displacement factored by K multiplier, w(x), given in Eqn.5.10.

$$w(x) = \int_0^x \int M(x).dx^2 \tag{5.9}$$

$$K.y(x) = A_1.s^2.\cosh\varphi + A_2.s^2.\sinh\varphi + \frac{v^2-1}{v^2}.F.(H.\frac{x^2}{2} - \frac{x^3}{6}) + A_3.x + A_4 \tag{5.10}$$

where

A_3= -A_2.(s)

A_4= -A_1.(s^2)

Then the slope equation can be written as in Eqn.5.11.

$$K.y^{\text{I}}(x) = A_1.s.\sinh\varphi + A_2.s.\cosh\varphi + \frac{v^2-1}{v^2}.F.(H.x - \frac{x^2}{2}) + A_3 \tag{5.11}$$

Finally the curvature equation can be expressed as in Eqn.5.12.

$$K.y^{\text{II}}(x) = A_1.\cosh\varphi + A_2.\sinh\varphi + \frac{v^2-1}{v^2}.F.(H - x) \tag{5.12}$$

Lateral sway at any height of the building and relative story drifts as well as slope & curvature along height of the building can be easily calculated by using the executable “Borland Delphi” program developed, which is shown in Figure 5.1. The graphs showing the number of story versus displacements & relative story drifts and the number of story versus slope & curvature along height at story levels can also be drawn easily by using the developed “Borland Delphi” program.

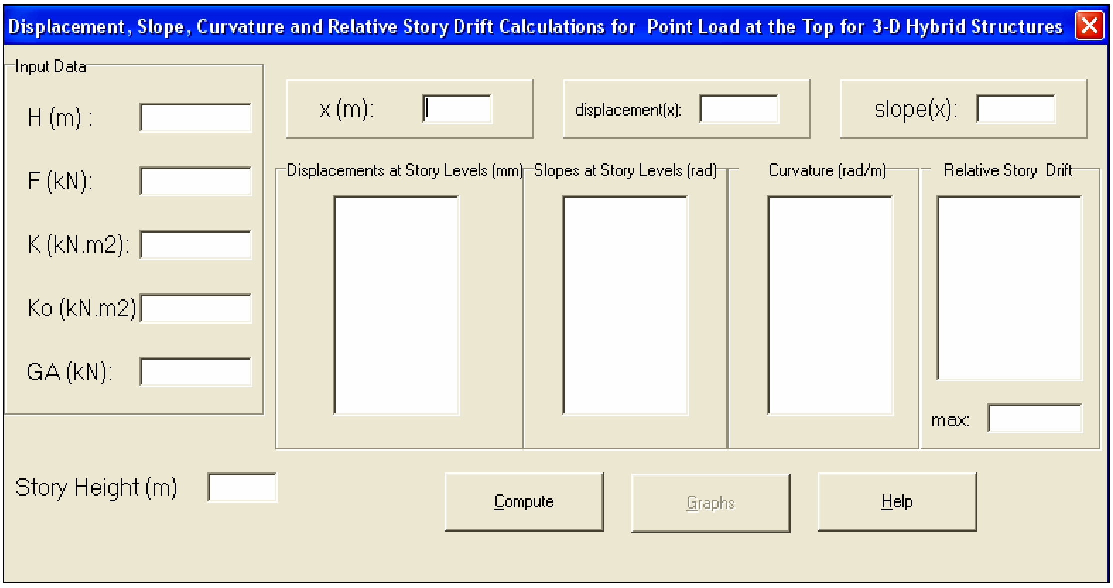


Figure 5.1 Executable “Borland Delphi” Program to Calculate Lateral Sway, Slope, Curvature and Relative Story Drift for Mixed Structures

Moment equation can easily be obtained by multiplying the curvature with EI as given in Eqn.5.13 and Eqn.5.14.

$$M(x) = -EI \,.\, y^{II}(x) \tag{5.13}$$

$$M(x) = -\frac{EI}{K}[A_1.\cosh\varphi + A_2.\sinh\varphi + \frac{v^2-1}{v^2}.F.(H-x)] \tag{5.14}$$

Shear equation can then be readily obtained by differentiating the moment equation with respect to x as expressed in Eqn.5.15 and so Eqn.5.16 is attained.

$$V(x) = -M^{'}(x) = EI \,.\, y^{III}(x) \tag{5.15}$$

$$V(x) = \frac{EI}{K}[\frac{A_1}{s}.\sinh\varphi + \frac{A_2}{s}.\cosh\varphi - \frac{v^2-1}{v^2}.F] \quad (5.16)$$

Force equation (i.e. equation of load coming to shear wall) can then be readily obtained by differentiating the shear equation with respect to x as expressed in Eqn.5.17 and so Eqn.5.18 is attained.

$$P(x) = -V'(x) = -EI \cdot y^{IV}(x) \quad (5.17)$$

$$P(x) = -\frac{EI}{K}\left(\frac{A_1}{s^2}.\cosh\varphi + \frac{A_2}{s^2}.\sinh\varphi\right) \quad (5.18)$$

5.2 ASSESSING THE VALIDITY OF THE ANALYTICAL MODEL (EXAMPLE 1)

Firstly the validity of the analytical model developed was tested on a 3D-mixed structure having only 2 shear walls with l_w=6m and b_w=0.25m (with different number of stories) as shown in Figure 5.2. The results that are determined by using SAP2000 and analytical equation were then compared both in tabular and graphical forms. For simplicity, the concentrated load F at the top of structure was assumes as 1 000 kN.

The parameters used in the analytical expression were calculated as below.

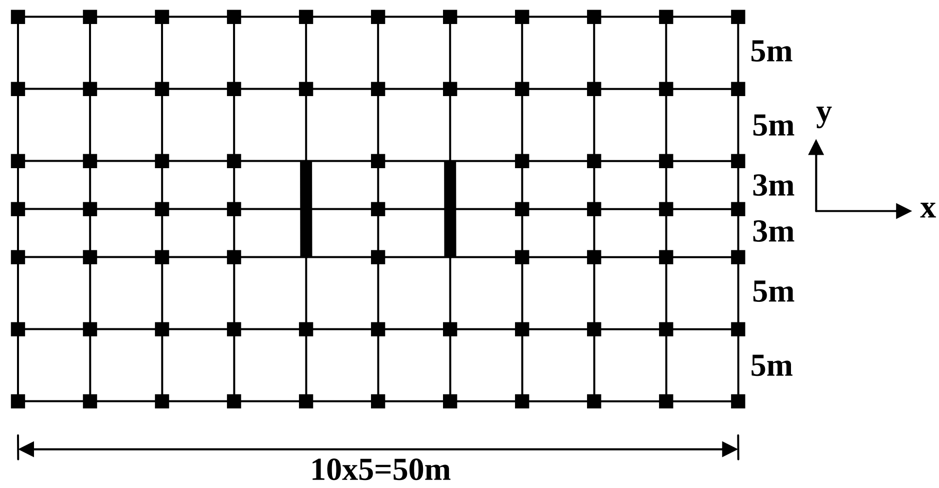
K= K (shear walls) + Σ K (columns)

Since Σ K (columns) term can be neglected, then along y-direction

$$K = 28\,500\,000\ \text{kN/m}^2.\left[\frac{1}{12}(0.25)(6)^3\right]x2 = 256\,500\,000\ \text{kN.m}^2$$

$$K_0 = 28\,500\,000\ \text{kN/m}^2.\left[\left((0.4)(0.4)(13)^2\right)x11\right]x2 = 16\,954\,000\,000\ \text{kN.m}^2$$

GA = 22 x 41 793 + 18 x 47 542 + 9 x 5 182 + 22 x 28 169 = 2 441 558 kN



All columns : 400x400 mm
All beams : 250x450 mm
Slab thickness : 120 mm
All storey heights : 3 m
g (additional) : 2.0 kN/m^2
q (additional) : 3.5 kN/m^2

(a)

(b)

(EXAMPLE 1)

Figure 5.2 Mixed Structure Used to Test the Validity of the Analytical Method: **(a)** Typical floor plan, **(b)** 3-D view of a sample 4-storey mixed structure

5.3 COMPARISON OF RESULTS (EXAMPLE1)

Table 5.1 Comparisons of Lateral Displacements and Relative Story Drifts as Determined by SAP2000 and Analytical Model for Mixed Structure (Example1)

# of	Displacement	Displacement	Difference	Relative Story Drift	Relative Story Drift	Difference
story	Sap2000(mm)	Analytic(mm)	(%)	(Sap2000)	(Analytic)	(%)
2	0.338	0.242	28.4	0.000069	0.000055	20.3
1	0.13	0.076	41.3	0.000043	0.000025	41.3
				max=0.000069	**max=0.000055**	**20.2**
4	1.48	1.376	7.0	0.000167	0.000165	1.2
3	0.98	0.882	10.0	0.000151	0.000146	3.6
2	0.526	0.444	15.5	0.000117	0.000106	9.2
1	0.175	0.126	28.2	0.000058	0.000042	28.2
				max=0.000167	**max=0.000165**	**1.2**
6	3.3	3.166	4.1	0.000247	0.000249	1.0
5	2.56	2.419	5.5	0.000240	0.000238	0.7
4	1.84	1.704	7.4	0.000223	0.000216	3.4
3	1.17	1.057	9.7	0.000187	0.000179	4.0
2	0.61	0.519	14.9	0.000138	0.000125	9.6
1	0.195	0.144	26.2	0.000065	0.000048	26.2
				max=0.000247	**max=0.000249**	**0.8**
8	5.5	5.259	4.4	0.000303	0.000302	0.3
7	4.59	4.351	5.2	0.000303	0.000296	2.3
6	3.68	3.462	5.9	0.000290	0.000284	2.1
5	2.81	2.610	7.1	0.000273	0.000264	3.6
4	1.99	1.819	8.6	0.000247	0.000234	5.3
3	1.25	1.119	10.5	0.000203	0.000191	6.1
2	0.64	0.546	14.7	0.000145	0.000132	9.3
1	0.204	0.150	26.2	0.000068	0.000050	26.2
				max=0.000303	**max=0.000302**	**0.3**
10	7.92	7.521	5.0	0.000340	0.000337	0.9
9	6.9	6.510	5.6	0.000343	0.000333	2.9
8	5.87	5.510	6.1	0.000340	0.000326	4.0
7	4.85	4.531	6.6	0.000330	0.000315	4.6
6	3.86	3.586	7.1	0.000310	0.000298	3,9
5	2.93	2.693	8.1	0.000290	0.000274	5.5
4	2.06	1.870	9.2	0.000257	0.000241	6.0
3	1.29	1.147	11.1	0.000210	0.000196	6.6
2	0.66	0.558	15.5	0.000151	0.000135	10.5
1	0.208	0.153	26.2	0.000069	0.000051	26.2
				max=0.000343	**max=0.000337**	**1.7**

Table 5.1 Comparisons of Lateral Displacements and Relative Story Drifts as Determined by SAP2000 and Analytical Model for Mixed Structure (Example1) (Continued)

# of story	Displacement Sap2000(mm)	Displacement Analytic(mm)	Difference (%)	Relative Story Drift (Sap2000)	Relative Story Drift (Analytic)	Difference (%)
15	14.65	13.840	5.5	0.000400	0.000392	2.0
14	13.45	12.664	5.8	0.000403	0.000391	3.0
13	12.24	11.491	6.1	0.000407	0.000389	4.4
12	11.02	10.324	6.3	0.000407	0.000385	5.3
11	9.8	9.169	6.4	0.000400	0.000380	5.0
10	8.6	8.029	6.6	0.000397	0.000373	5.9
9	7.41	6.909	6.8	0.000387	0.000364	5.9
8	6.25	5.817	6.9	0.000373	0.000352	5.7
7	5.13	4.761	7.2	0.000353	0.000336	4.9
6	4.07	3.753	7.8	0.000337	0.000315	6.3
5	3.06	2.807	8.3	0.000303	0.000288	5.0
4	2.15	1.943	9.6	0.000270	0.000252	6.7
3	1.34	1.187	11.4	0.000220	0.000204	7.4
2	0.68	0.576	15.3	0.000155	0.000139	10.3
1	0.214	0.158	26.2	0.000071	0.000053	26.2
				max=0.000407	**max=0.000392**	**3.7**
20	22.34	21.346	4.5	0.000443	0.000442	0.2
19	21.01	20.018	4.7	0.000450	0.000442	1.8
18	19.66	18.693	4.9	0.000453	0.000441	2.8
17	18.3	17.371	5.1	0.000453	0.000439	3.3
16	16.94	16.055	5.2	0.000453	0.000436	3.9
15	15.58	14.748	5.3	0.000453	0.000432	4.6
14	14.22	13.450	5.4	0.000447	0.000428	4.2
13	12.88	12.166	5.5	0.000443	0.000423	4.7
12	11.55	10.898	5.6	0.000437	0.000416	4.7
11	10.24	9.650	5.8	0.000430	0.000408	5.1
10	8.95	8.426	5.9	0.000420	0.000398	5.1
9	7.69	7.230	6.0	0.000407	0.000387	4.9
8	6.47	6.071	6.2	0.000390	0.000372	4.7
7	5.3	4.955	6.5	0.000370	0.000353	4.5
6	4.19	3.895	7.0	0.000347	0.000330	4.8
5	3.15	2.906	7.8	0.000317	0.000300	5.3
4	2.2	2.006	8.8	0.000277	0.000261	5.6
3	1.37	1.222	10.8	0.000227	0.000210	7.2
2	0.69	0.592	14.3	0.000158	0.000143	9.2
1	0.217	0.162	25.4	0.000072	0.000054	25.4
				max=0.000453	**max=0.000442**	**2.4**

Table 5.2 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model for Mixed Structure (Example1)

# of story	Slope along height at story levels Sap2000(rad)	Slope along height at story levels Analytic(rad)	Difference (%)
2	0.0000673	0.0000601	10.7
1	0.0000637	0.0000454	28.6
4	0.0001600	0.0001678	4.9
3	0.0001650	0.0001585	3.9
2	0.0001353	0.0001298	4.1
1	0.0000927	0.0000786	15.2
6	0.0002400	0.0002508	4.5
5	0.0002500	0.0002455	1.8
4	0.0002300	0.0002291	0.4
3	0.0002073	0.0002001	3.5
2	0.0001640	0.0001554	5.2
1	0.0001063	0.0000909	14.5
8	0.0002933	0.0003034	3.5
7	0.0003100	0.0003005	3.1
6	0.0002967	0.0002913	1.8
5	0.0002833	0.0002752	2.9
4	0.0002600	0.0002504	3.7
3	0.0002253	0.0002147	4.7
2	0.0001760	0.0001646	6.5
1	0.0001123	0.0000953	15.2
10	0.0003266	0.0003375	3.3
9	0.0003500	0.0003358	4.1
8	0.0003400	0.0003306	2.8
7	0.0003367	0.0003214	4.5
6	0.0003200	0.0003074	3.9
5	0.0003033	0.0002873	5.3
4	0.0002700	0.0002594	3.9
3	0.0002360	0.0002211	6.3
2	0.0001820	0.0001687	7.3
1	0.0001150	0.0000973	15.4

Table 5.2 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model for Mixed Structure (Example1) (Continued)

# of story	Slope along height at story levels Sap2000(rad)	Slope along height at story levels Analytic(rad)	Difference (%)
15	0.0003800	0.0003923	3.2
14	0.0004133	0.0003918	5.2
13	0.0004033	0.0003901	3.3
12	0.0004067	0.0003872	4.8
11	0.0004033	0.0003829	5.1
10	0.0004000	0.0003769	5.8
9	0.0003900	0.0003690	5.4
8	0.0003800	0.0003585	5.7
7	0.0003667	0.0003447	6.0
6	0.0003433	0.0003267	4.8
5	0.0003233	0.0003030	6.3
4	0.0002867	0.0002717	5.2
3	0.0002450	0.0002301	6.1
2	0.0001893	0.0001747	7.8
1	0.0001183	0.0001003	15.3
20	0.0004266	0.0004426	3.7
19	0.0004567	0.0004423	3.2
18	0.0004500	0.0004413	1.9
17	0.0004533	0.0004397	3.0
16	0.0004533	0.0004373	3.5
15	0.0004533	0.0004343	4.2
14	0.0004500	0.0004304	4.4
13	0.0004467	0.0004255	4.7
12	0.0004400	0.0004196	4.6
11	0.0004333	0.0004124	4.8
10	0.0004233	0.0004036	4.7
9	0.0004133	0.0003929	4.9
8	0.0004000	0.0003797	5.1
7	0.0003800	0.0003632	4.4
6	0.0003600	0.0003425	4.9
5	0.0003333	0.0003162	5.1
4	0.0002967	0.0002822	4.9
3	0.0002500	0.0002380	4.8
2	0.0001950	0.0001799	7.7
1	0.0001207	0.0001029	14.7

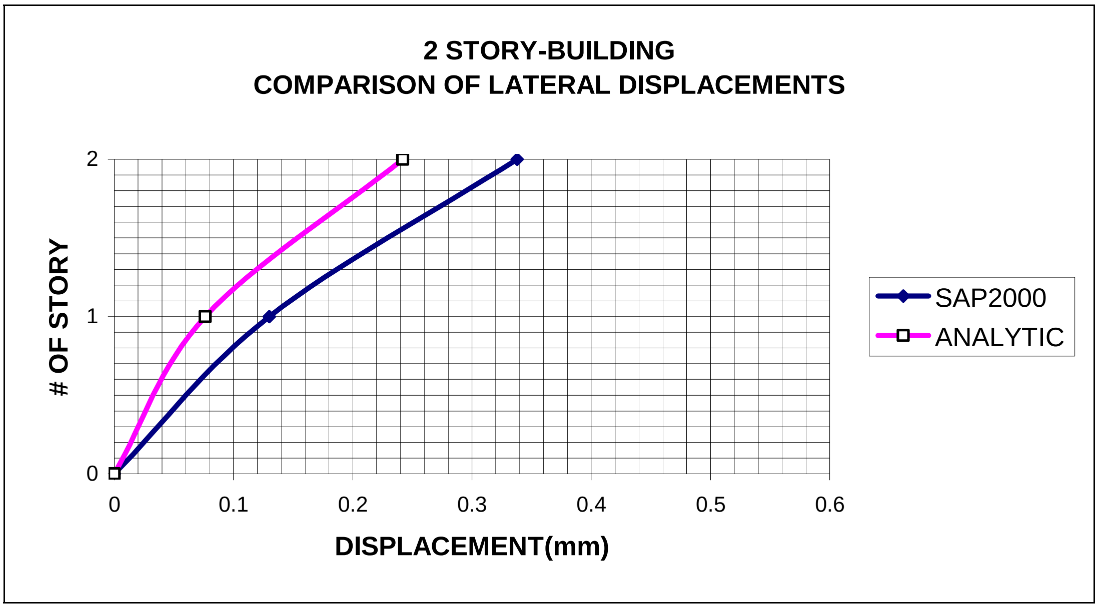


Figure 5.3 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 2 Story-Mixed Structure Example1)

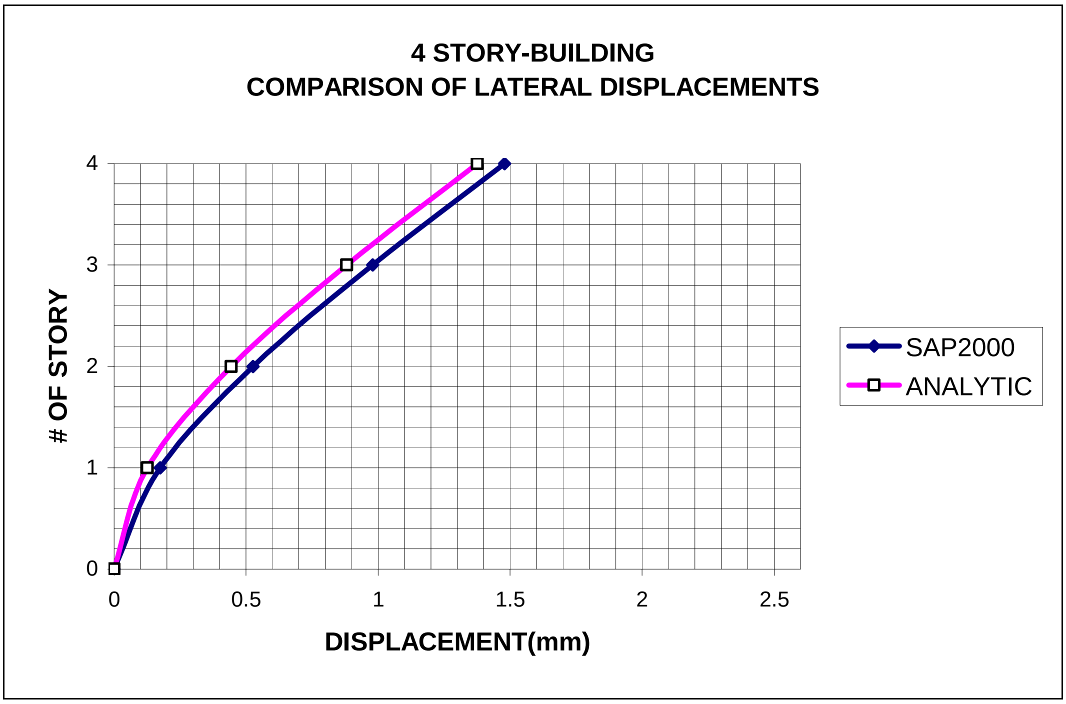


Figure 5.4 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 4 Story-Mixed Structure Example1)

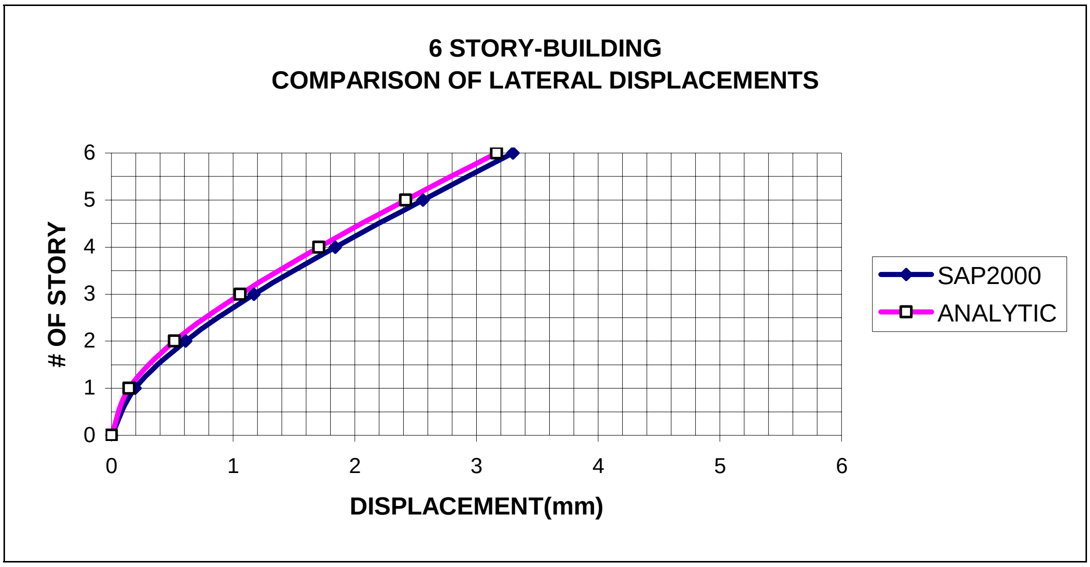


Figure 5.5 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 6 Story-Mixed Structure Example1)

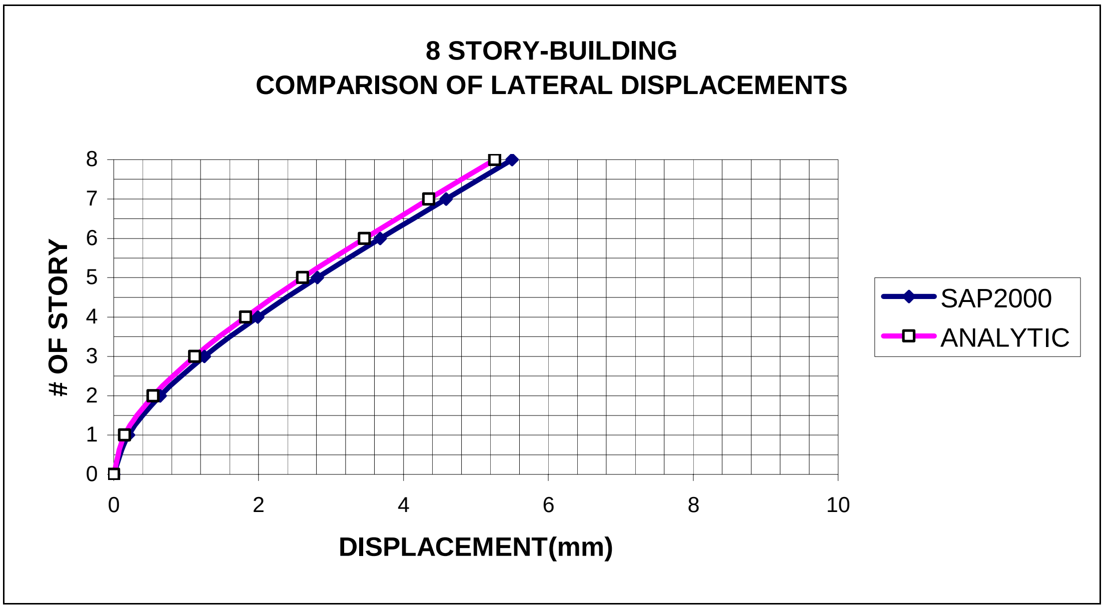


Figure 5.6 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 8 Story-Mixed Structure Example1)

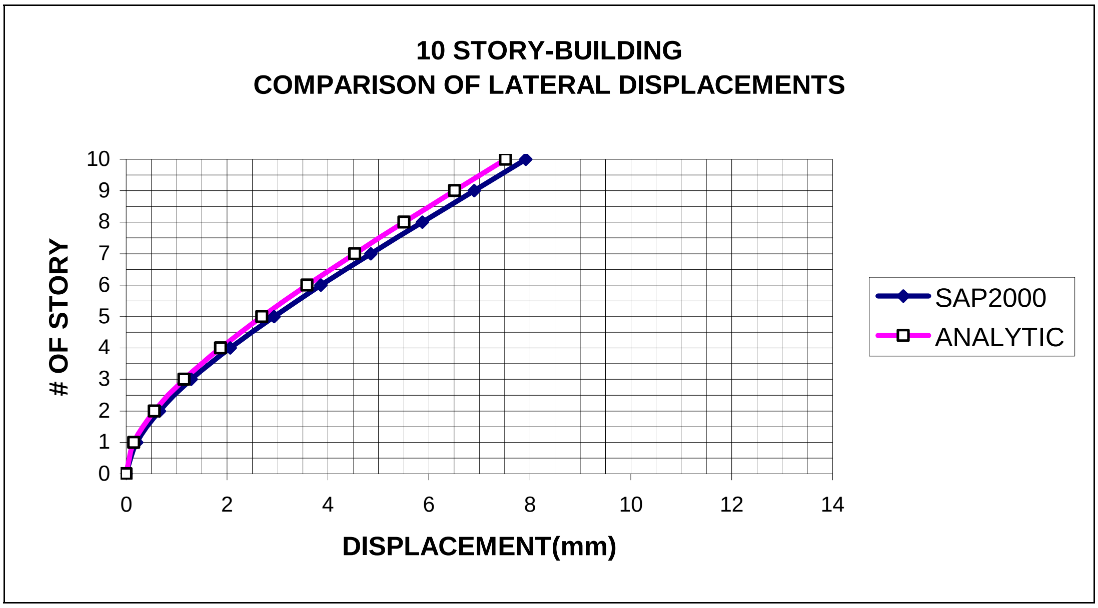


Figure 5.7 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 10 Story-Mixed Structure Example1)

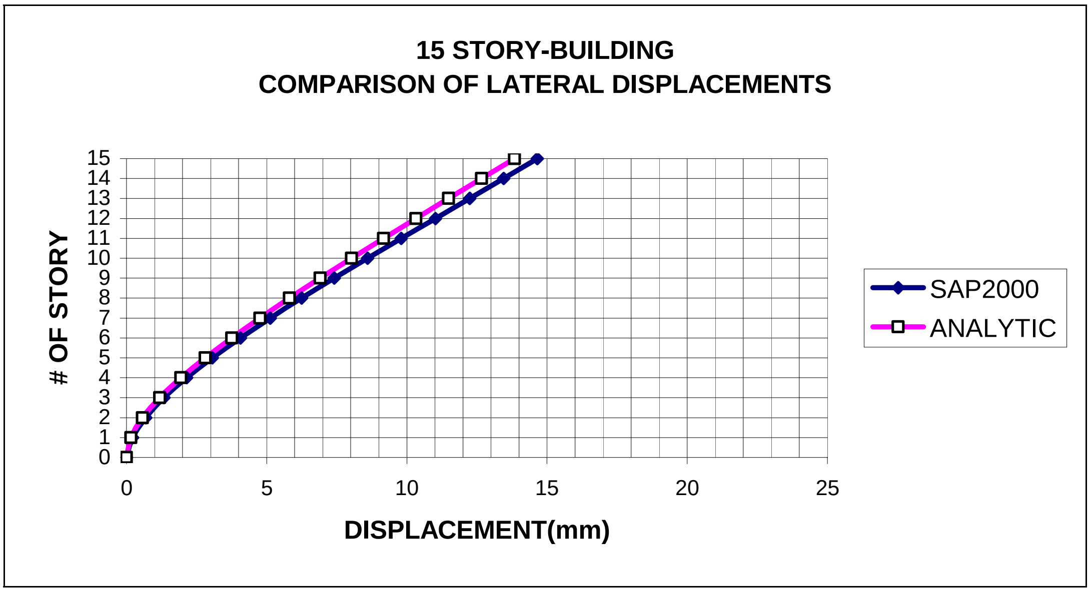


Figure 5.8 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 15 Story-Mixed Structure Example1)

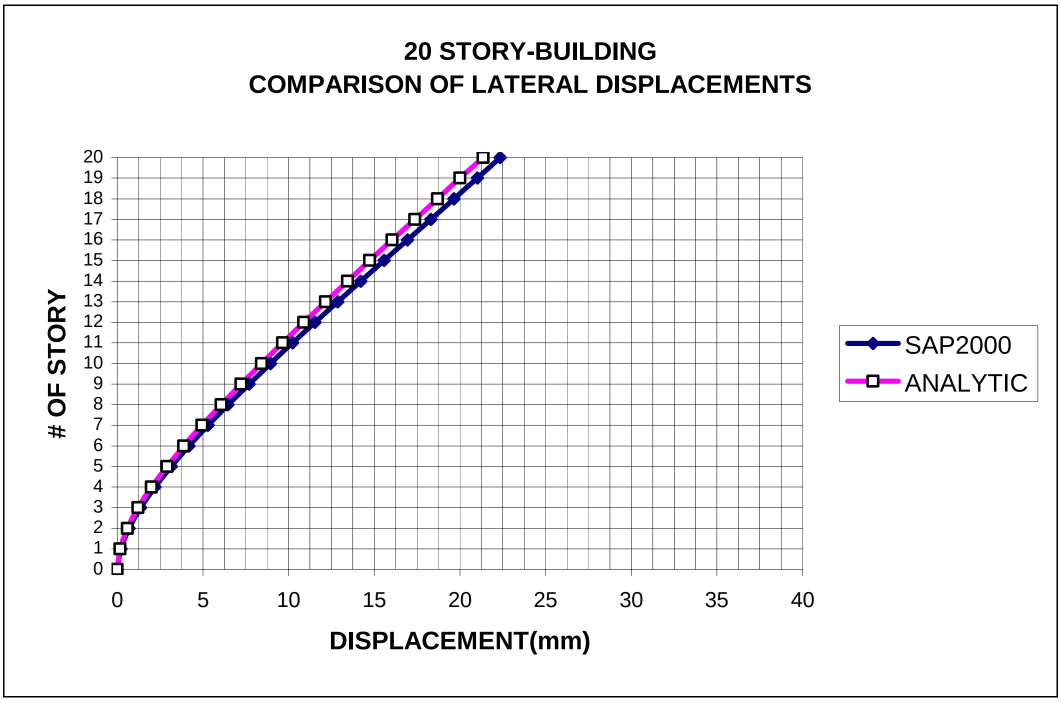


Figure 5.9 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 20 Story-Mixed Structure Example1)

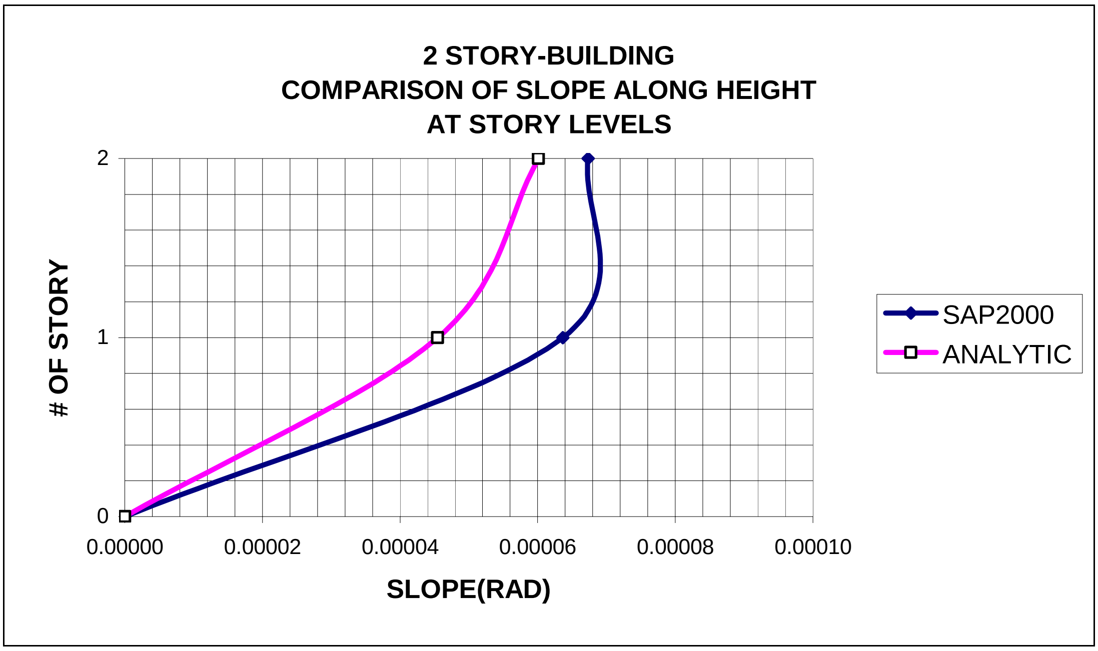


Figure 5.10 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (2 Story-Mixed Structure Example1)

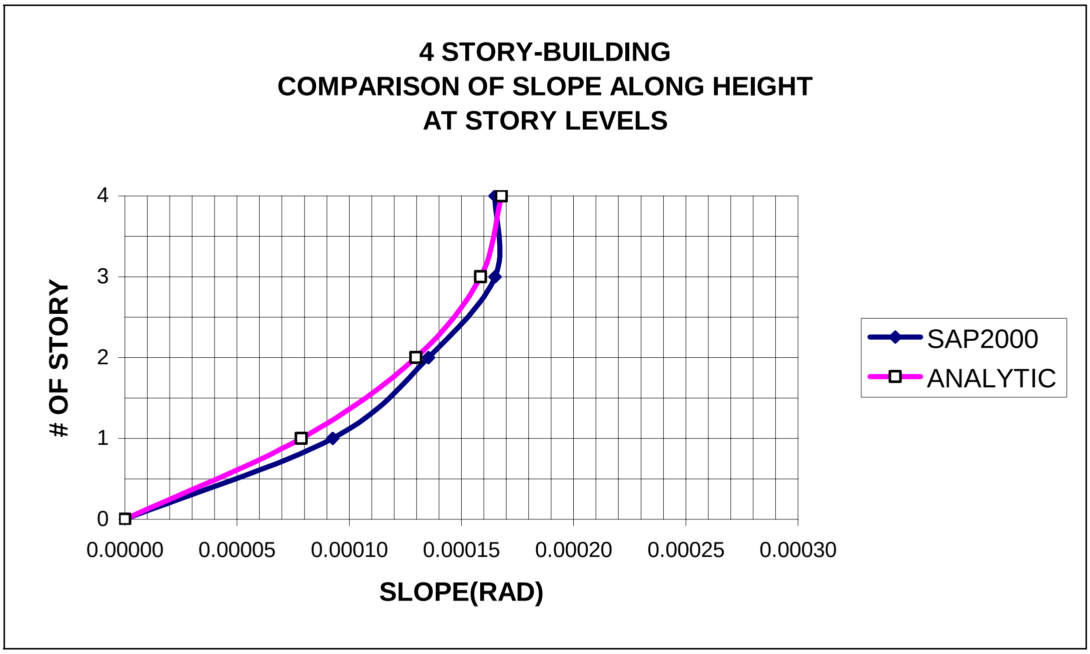


Figure 5.11 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (4 Story-Mixed Structure Example1)

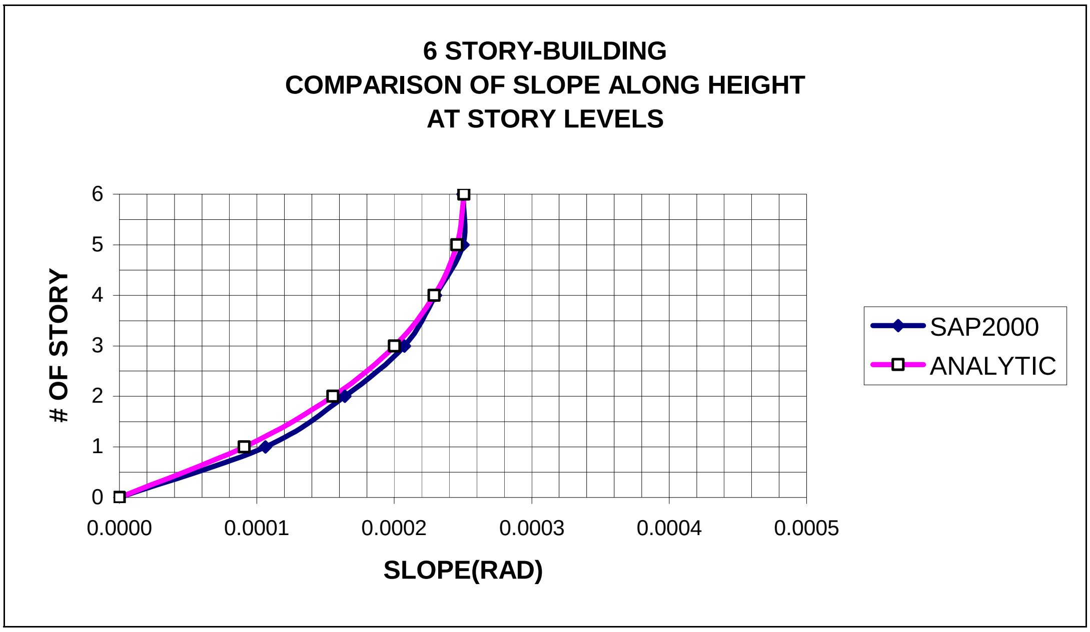


Figure 5.12 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (6 Story-Mixed Structure Example1)

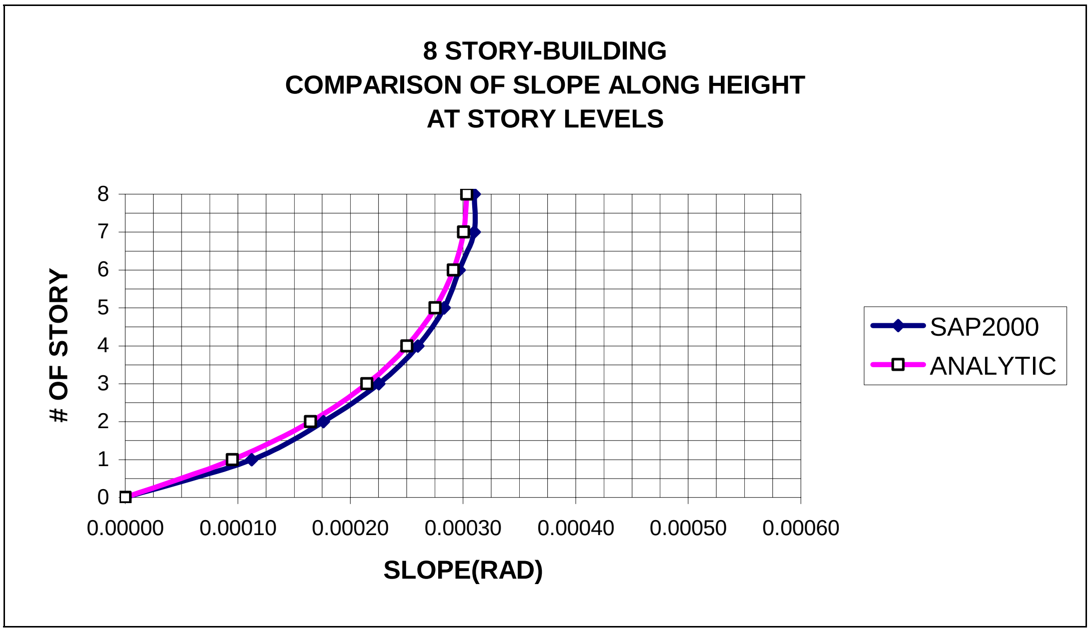


Figure 5.13 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (8 Story-Mixed Structure Example1)

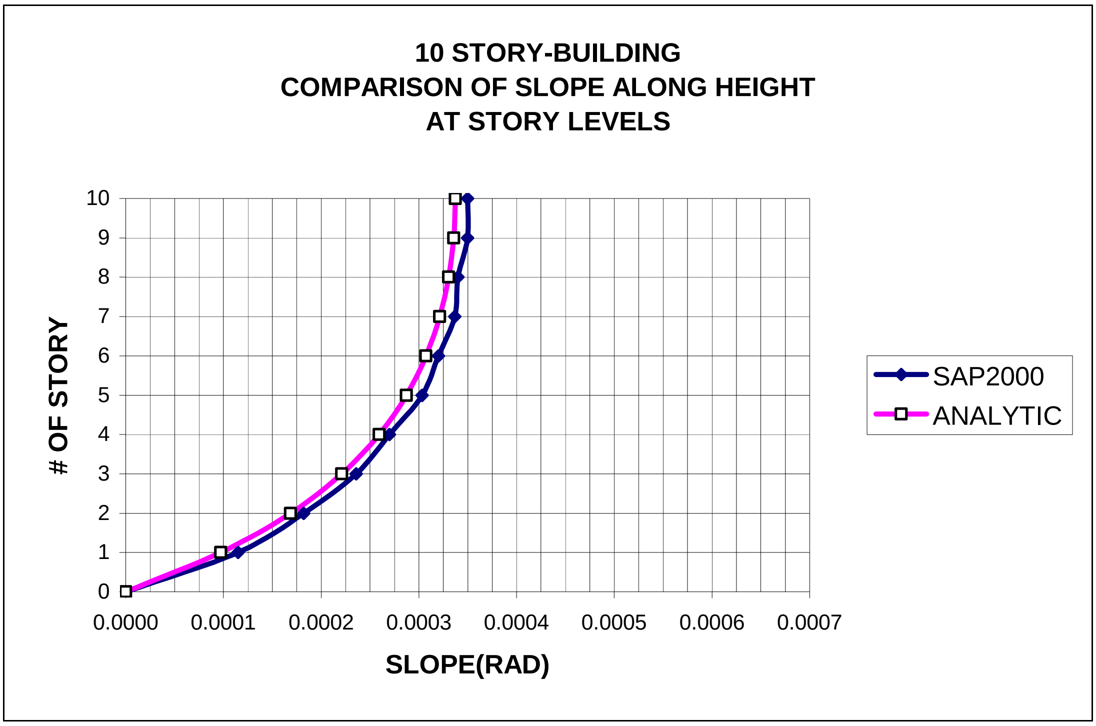


Figure 5.14 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (10 Story-Mixed Structure Example1)

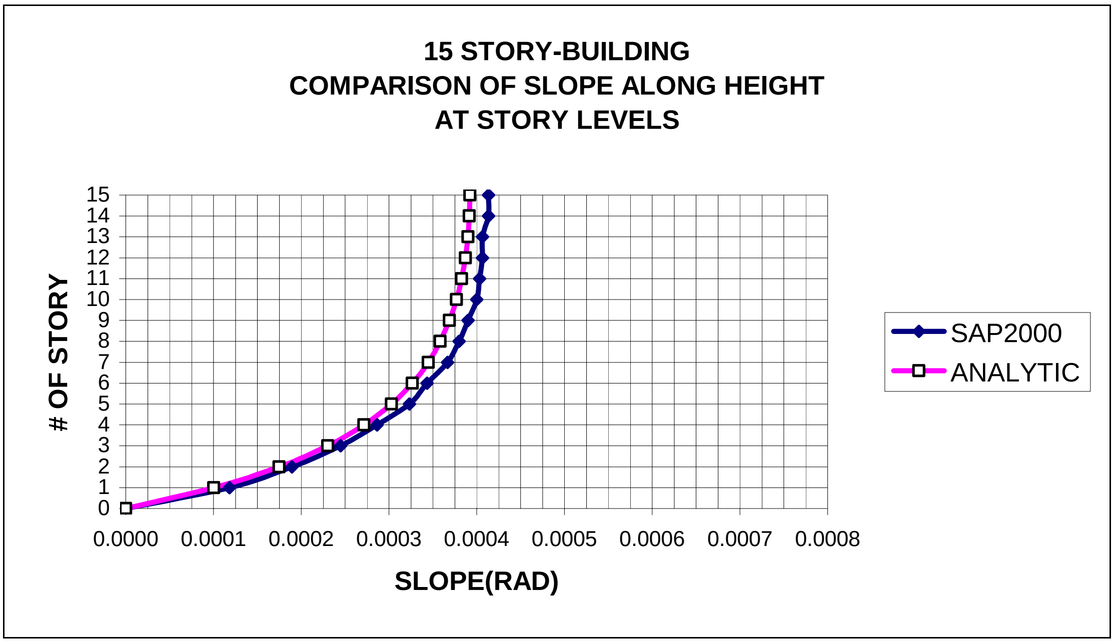


Figure 5.15 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (15 Story-Mixed Structure Example1)

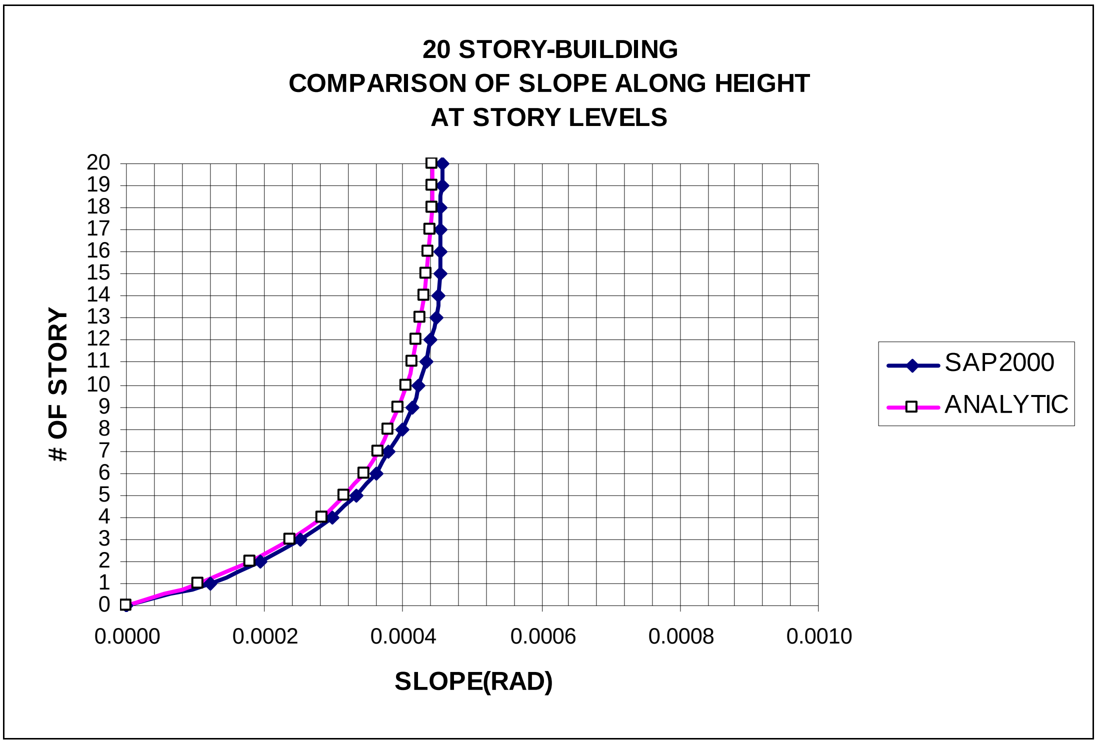


Figure 5.16 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (20 Story-Mixed Structure Example1)

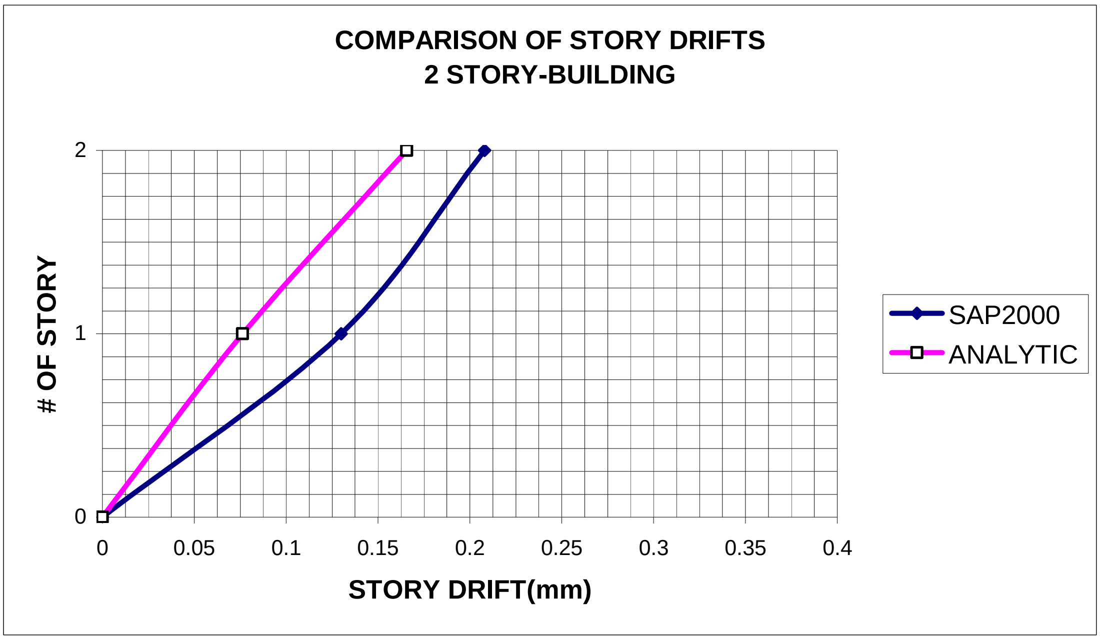


Figure 5.17 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 2 Story-Mixed Structure Example1)

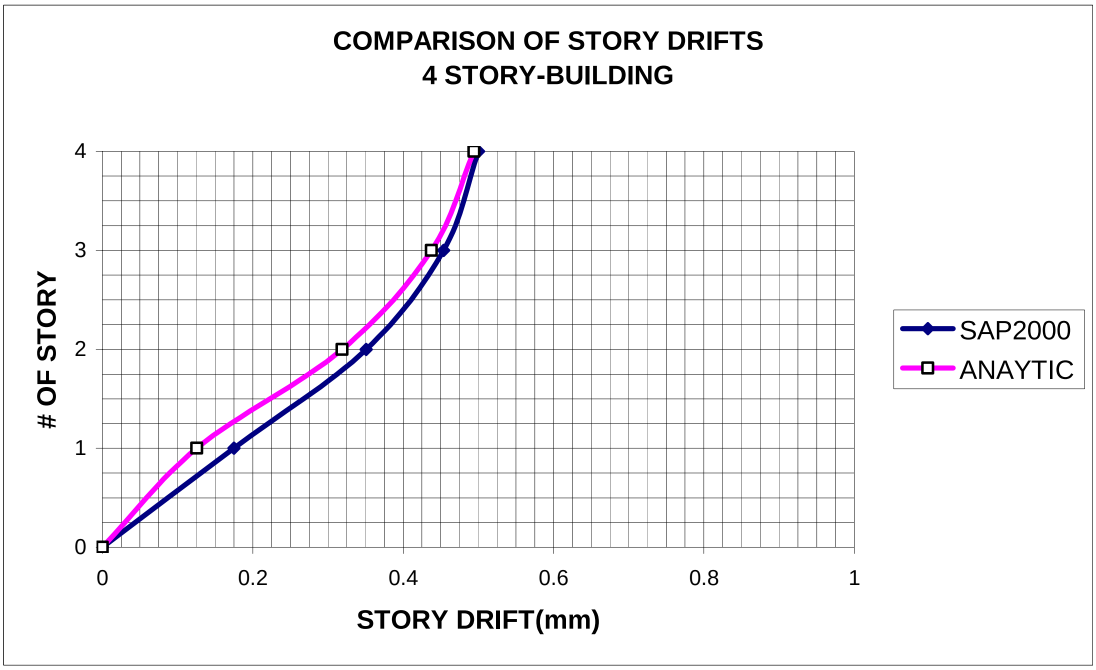


Figure 5.18 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 4 Story-Mixed Structure Example1)

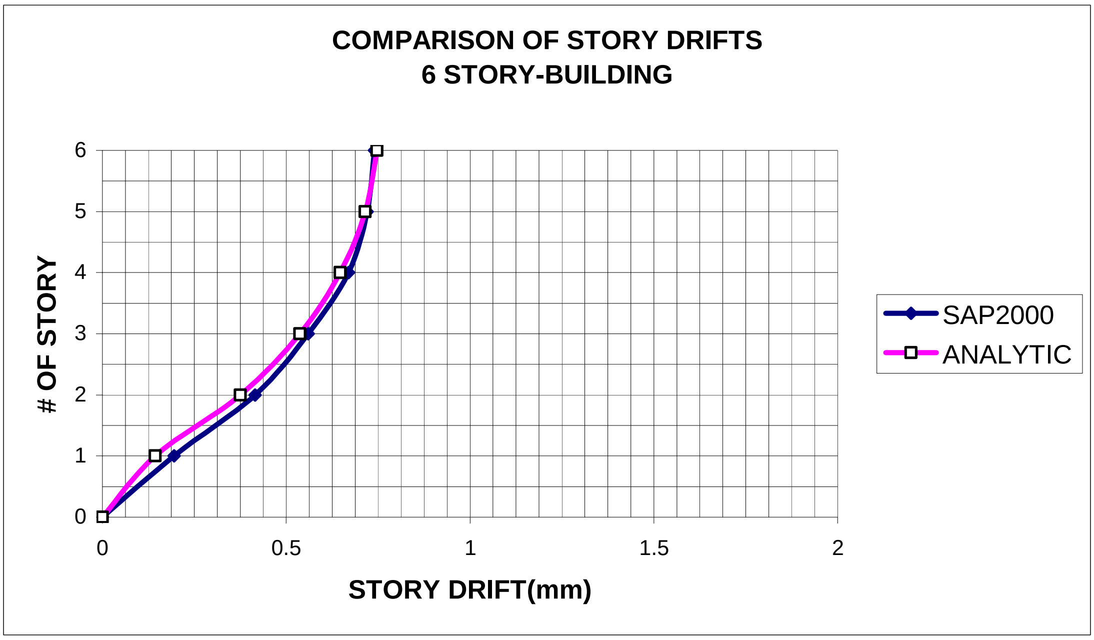


Figure 5.19 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 6 Story-Mixed Structure Example1)

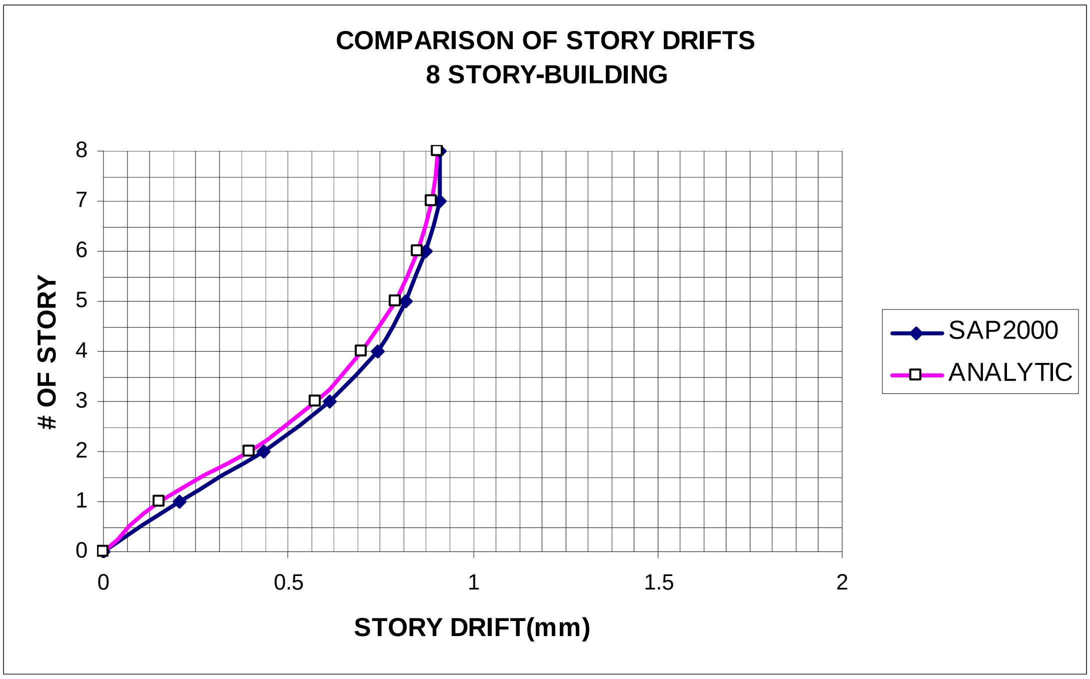


Figure 5.20 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 8 Story-Mixed Structure Example1)

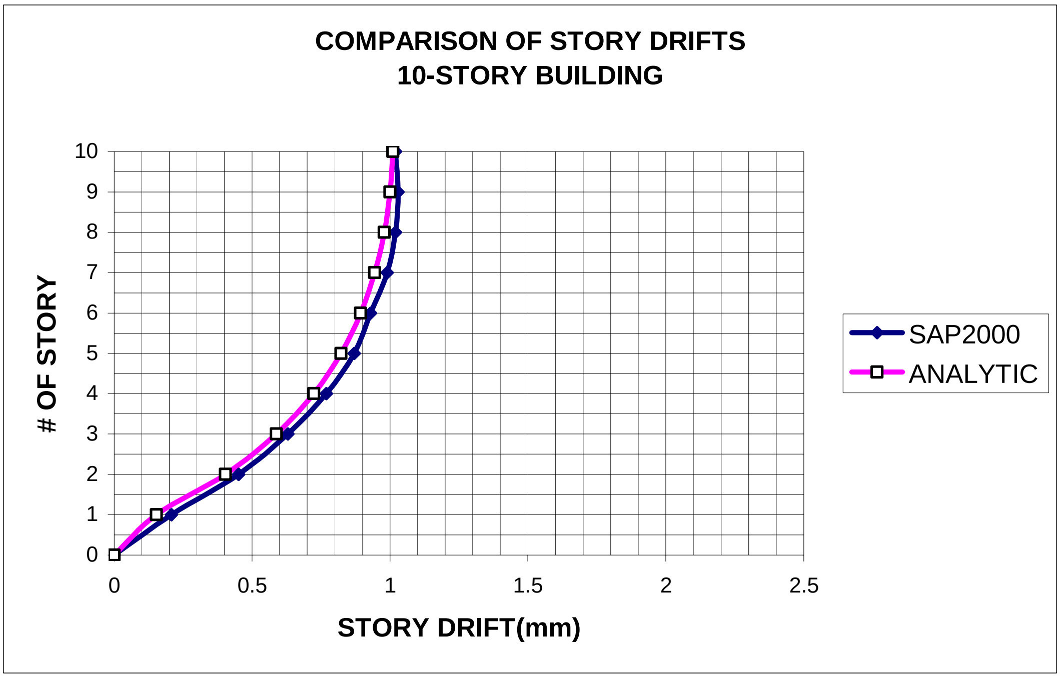


Figure 5.21 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 10 Story-Mixed Structure Example1)

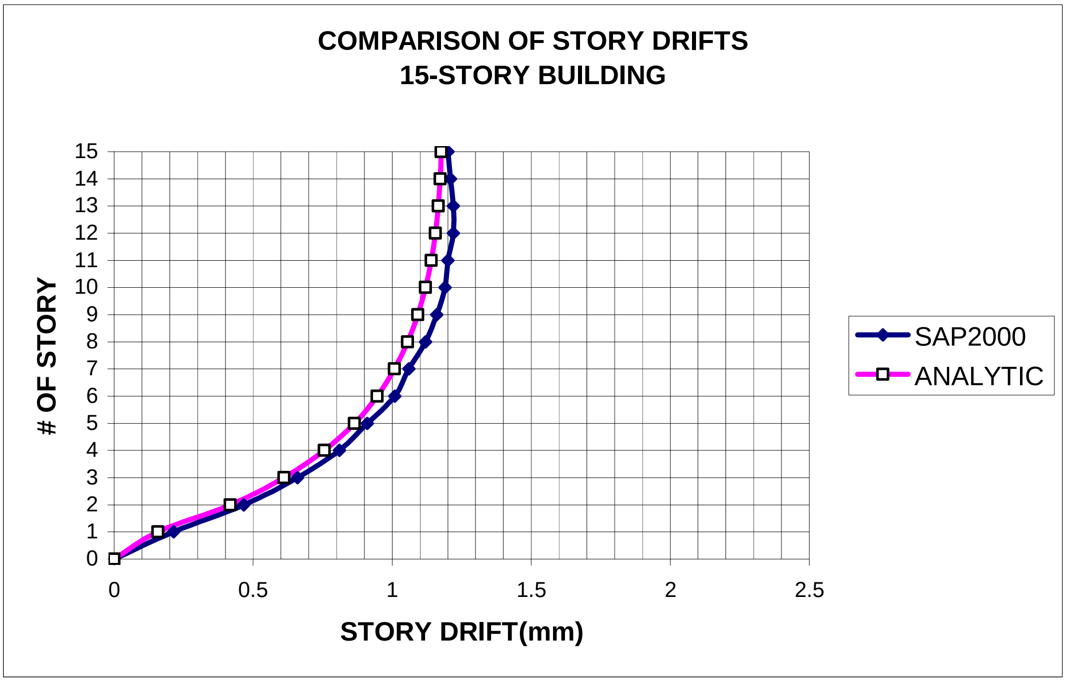


Figure 5.22 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 15 Story-Mixed Structure Example1)

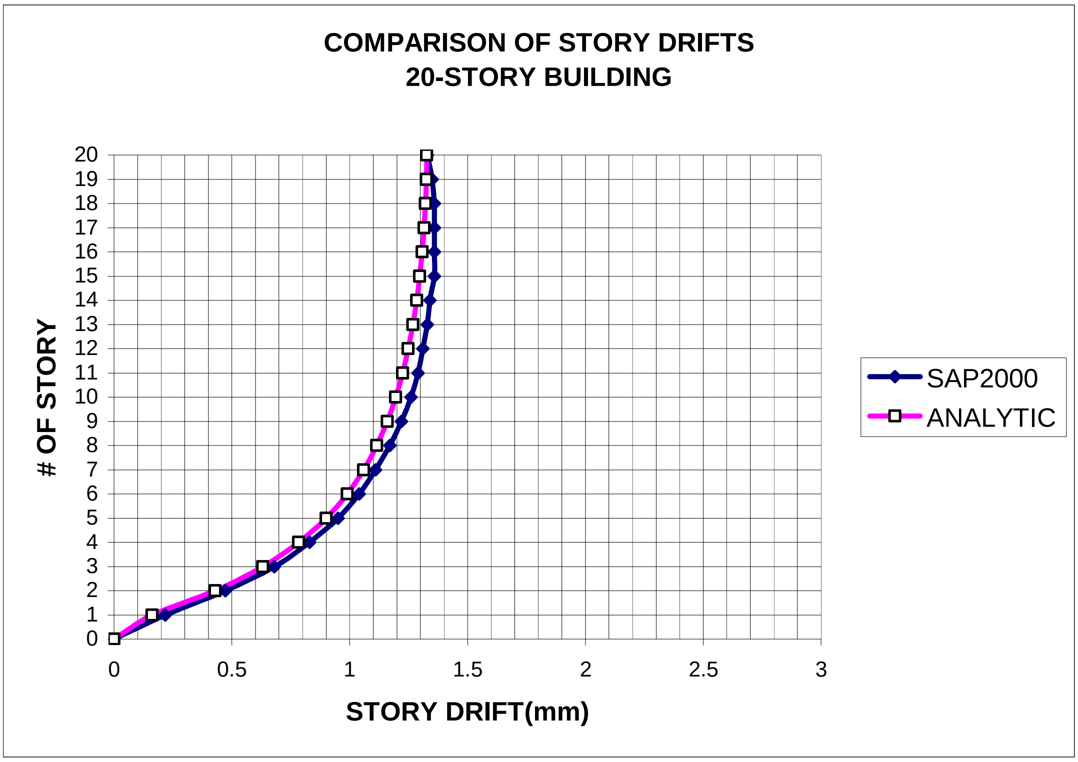


Figure 5.23 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 20 Story-Mixed Structure Example1)

5.4 ASSESSING THE VALIDITY OF THE ANALYTICAL MODEL (EXAMPLE2)

Secondly the same structure having 4 shear walls, two of which have l_w=6m and b_w=0.25m and the other two have l_w=10m and b_w=0.25m (with different number of stories) as shown in Figure 5.24 was also tested to show the validity of analytical model. The concentrated load F at the top of structure was assumes as 1 000 kN.

The parameters used in the analytical expression were calculated as below:

K= K (shear walls) + Σ K (columns)

Since Σ K (columns) term can be neglected, then along y-direction

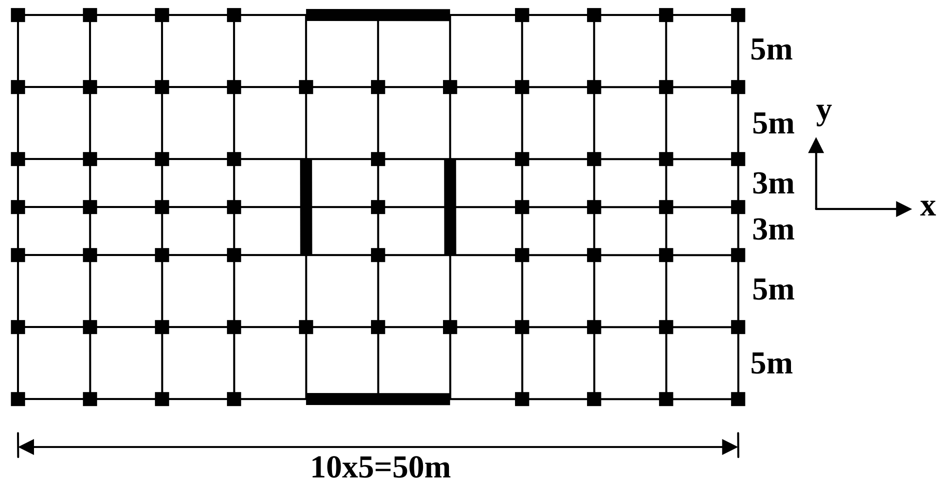
$$K = 28\,500\,000\ \text{kN/m}^2.\left[\frac{1}{12}(0.25)(6)^3 + \frac{1}{12}(10)(0.25)^3\right]x2 = 257\,242\,190\ \text{kN.m}^2$$

$$(K_0)_{columns} = 2\,8\,50\,0\,000\ kN/m^2.\left[\left((0.4)(0.4)(13)^2\right)x8\right]x2 = 12\,330\,240\,000\ kN.m^2$$

$$(K_0)_{walls} = 28\,500\,000\ kN/m^2.\left[(0.25)(10)(13)^2\right]x2 = 24\,082\,500\,000\ kN.m^2$$

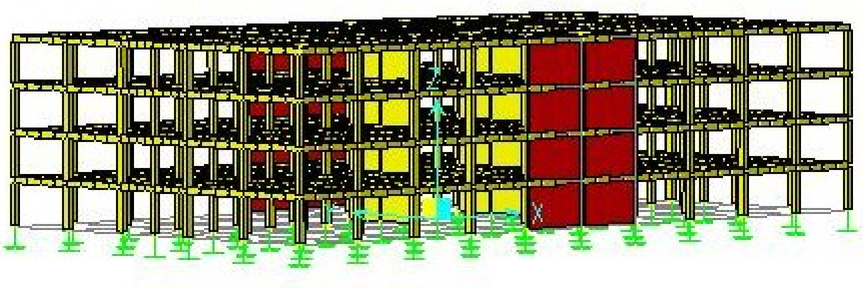
$$\sum K_0 = 36\,412\,740\,000\ kN.m^2$$

GA = 22 x 41 793 + 18 x 47 542 + 9 x 5 182 + 16 x 28 169 = 2 272 544 kN



All columns : 400x400 mm
All beams : 250x450 mm
Slab thickness : 120 mm
All storey heights : 3 m
g (additional) : 2.0 kN/m^2
q (additional) : 3.5 kN/m^2

(a)



(b)

(EXAMPLE 2)

Figure 5.24 Mixed Structure Used to Test the Validity of the Analytical Method: **(a)** Typical floor plan, **(b)** 3-D view of a sample 4-storey mixed structure

5.5 COMPARISON OF RESULTS (EXAMPLE 2)

Table 5.3 Comparisons of Lateral Displacements and Relative Story Drifts as Determined by SAP2000 and Analytical Model for Mixed Structure (Example2)

# of	Displacement	Displacement	Difference	Relative Story Drift	Relative Story Drift	Difference
story	**Sap2000(mm)**	**Analytic(mm)**	**(%)**	**(Sap2000)**	**(Analytic)**	**(%)**
2	0.336	0.243	27.6	0.000069	0.000056	18.8
1	0.129	0.077	40.6	0.000043	0.000026	39.5
				max=0.000069	**max=0.000056**	**18.8**
4	1.47	1.403	4.6	0.000164	0.000168	2.5
3	0.978	0.898	8.1	0.000151	0.000149	1.7
2	0.524	0.452	13.7	0.000116	0.000108	7.0
1	0.175	0.128	27.0	0.000058	0.000043	25.8
				max=0.000164	**max=0.000168**	**2.4**
6	3.28	3.251	0.9	0.000243	0.000256	5.3
5	2.55	2.482	2.7	0.000240	0.000245	2.1
4	1.83	1.747	4.5	0.000220	0.000222	0.7
3	1.17	1.082	7.5	0.000188	0.000184	2.4
2	0.605	0.531	12.3	0.000137	0.000128	6.6
1	0.194	0.147	24.3	0.000065	0.000049	24.6
				max=0.000243	**max=0.000256**	**5.3**
8	5.45	5.399	0.9	0.000297	0.000311	4.7
7	4.56	4.466	2.1	0.000300	0.000305	1.6
6	3.66	3.551	3.0	0.000290	0.000292	0.6
5	2.79	2.675	4.1	0.000273	0.000271	1.0
4	1.97	1.863	5.4	0.000240	0.000240	0.2
3	1.25	1.145	8.4	0.000203	0.000196	3.8
2	0.64	0.558	12.9	0.000146	0.000135	7.5
1	0.203	0.153	24.4	0.000068	0.000051	25
				max=0.000300	**max=0.000311**	**3.7**
10	7.83	7.680	1.9	0.000333	0.000344	3.3
9	6.83	6.647	2.7	0.000340	0.000341	0.2
8	5.81	5.625	3.2	0.000337	0.000334	0.9
7	4.8	4.624	3.7	0.000323	0.000322	0.5
6	3.83	3.659	4.5	0.000310	0.000304	1.8
5	2.9	2.746	5.3	0.000287	0.000280	2.3
4	2.04	1.906	6.6	0.000253	0.000246	2.8
3	1.28	1.168	8.8	0.000208	0.000200	3.8
2	0.656	0.568	13.5	0.000150	0.000137	8.3
1	0.207	0.156	24.7	0.000069	0.000052	24.6
				max=0.000340	**max=0.000344**	**1.2**

Table 5.3 Comparisons of Lateral Displacements and Relative Story Drifts as Determined by SAP2000 and Analytical Model for Mixed Structure (Example2) (Continued)

# of story	Displacement Sap2000(mm)	Displacement Analytic(mm)	Difference (%)	Relative Story Drift (Sap2000)	Relative Story Drift (Analytic)	Difference (%)
15	14.34	13.742	4.2	0.000387	0.000386	0.0
14	13.18	12.583	4.5	0.000397	0.000386	2.8
13	11.99	11.426	4.7	0.000397	0.000384	3.3
12	10.8	10.276	4.9	0.000393	0.000380	3.3
11	9.62	9.134	5.0	0.000393	0.000376	4.4
10	8.44	8.006	5.1	0.000387	0.000370	4.4
9	7.28	6.897	5.3	0.000377	0.000361	4.1
8	6.15	5.813	5.5	0.000367	0.000350	4.5
7	5.05	4.763	5.7	0.000350	0.000335	4.3
6	4.00	3.758	6.1	0.000327	0.000315	3.6
5	3.02	2.813	6.9	0.000300	0.000288	3.9
4	2.12	1.948	8.1	0.000267	0.000252	5.4
3	1.32	1.191	9.8	0.000215	0.000204	5.1
2	0.674	0.578	14.3	0.000154	0.000140	9.4
1	0.211	0.158	24.9	0.000070	0.000053	24.3
				max=0.000397	**max=0.000386**	**2.8**
20	21.54	20.371	5.4	0.000420	0.000414	1.5
19	20.28	19.129	5.7	0.000433	0.000413	4.6
18	18.98	17.889	5.7	0.000433	0.000413	4.8
17	17.68	16.652	5.8	0.000433	0.000411	5.1
16	16.38	15.418	5.9	0.000437	0.000410	6.2
15	15.07	14.190	5.8	0.000430	0.000407	5.3
14	13.78	12.968	5.9	0.000430	0.000404	6.0
13	12.49	11.755	5.9	0.000427	0.000401	6.1
12	11.21	10.553	5.9	0.000420	0.000396	5.8
11	9.95	9.366	5.9	0.000413	0.000390	5.7
10	8.71	8.197	5.9	0.000407	0.000382	6.1
9	7.49	7.051	5.9	0.000393	0.000372	5.4
8	6.31	5.934	6.0	0.000380	0.000360	5.4
7	5.17	4.855	6.1	0.000360	0.000343	4.6
6	4.09	3.826	6.5	0.000337	0.000322	4.4
5	3.08	2.860	7.2	0.000307	0.000294	4.2
4	2.16	1.978	8.4	0.000270	0.000257	4.9
3	1.35	1.208	10.5	0.000222	0.000207	6.5
2	0.684	0.585	14.4	0.000157	0.000142	9.6
1	0.214	0.160	25.1	0.000071	0.000053	25.4
				max=0.000437	**max=0.000414**	**5.3**

Table 5.4 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model for Mixed Structure (Example2)

# of story	Slope along height at story levels Sap2000(rad)	Slope along height at story levels Analytic(rad)	Difference (%)
2	0.0000667	0.0000605	9.3
1	0.0000610	0.0000457	25.1
4	0.0001600	0.0001713	7.1
3	0.0001617	0.0001617	0.0
2	0.0001350	0.0001322	2.0
1	0.0000903	0.0000800	11.5
6	0.0002333	0.0002580	10.6
5	0.0002500	0.0002525	1.0
4	0.0002300	0.0002355	2.4
3	0.0002047	0.0002053	0.3
2	0.0001630	0.0001593	2.3
1	0.0001040	0.0000929	10.7
8	0.0002866	0.0003122	8.9
7	0.0003067	0.0003091	0.8
6	0.0002933	0.0002996	2.1
5	0.0002833	0.0002828	0.2
4	0.0002567	0.0002570	0.1
3	0.0002230	0.0002201	1.3
2	0.0001750	0.0001684	3.8
1	0.0001097	0.0000973	11.3
10	0.0003266	0.0003449	5.6
9	0.0003400	0.0003432	0.9
8	0.0003400	0.0003378	0.6
7	0.0003300	0.0003284	0.5
6	0.0003167	0.0003141	0.8
5	0.0003000	0.0002935	2.2
4	0.0002700	0.0002648	1.9
3	0.0002313	0.0002255	2.5
2	0.0001803	0.0001718	4.7
1	0.0001123	0.0000989	11.9

Table 5.4 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model for Mixed Structure (Example2) (Continued)

# of story	Slope along height at story levels Sap2000(rad)	Slope along height at story levels Analytic(rad)	Difference (%)
15	0.0003800	0.0003866	1.7
14	0.0003933	0.0003862	1.8
13	0.0003967	0.0003847	3.0
12	0.0003967	0.0003822	3.6
11	0.0003967	0.0003785	4.6
10	0.0003867	0.0003732	3.5
9	0.0003833	0.0003660	4.5
8	0.0003733	0.0003563	4.6
7	0.0003567	0.0003433	3.8
6	0.0003400	0.0003260	4.1
5	0.0003133	0.0003029	3.3
4	0.0002833	0.0002721	4.0
3	0.0002420	0.0002307	4.7
2	0.0001863	0.0001753	5.9
1	0.0001153	0.0001006	12.8
20	0.0004133	0.0004138	0.1
19	0.0004300	0.0004136	3.8
18	0.0004333	0.0004130	4.7
17	0.0004333	0.0004120	4.9
16	0.0004333	0.0004105	5.3
15	0.0004367	0.0004085	6.5
14	0.0004300	0.0004059	5.6
13	0.0004300	0.0004025	6.4
12	0.0004233	0.0003983	5.9
11	0.0004167	0.0003930	5.7
10	0.0004100	0.0003862	5.8
9	0.0003967	0.0003775	4.8
8	0.0003867	0.0003665	5.2
7	0.0003700	0.0003522	4.8
6	0.0003500	0.0003336	4.7
5	0.0003233	0.0003092	4.4
4	0.0002900	0.0002771	4.5
3	0.0002470	0.0002345	5.1
2	0.0001897	0.0001778	6.3
1	0.0001170	0.0001019	12.9

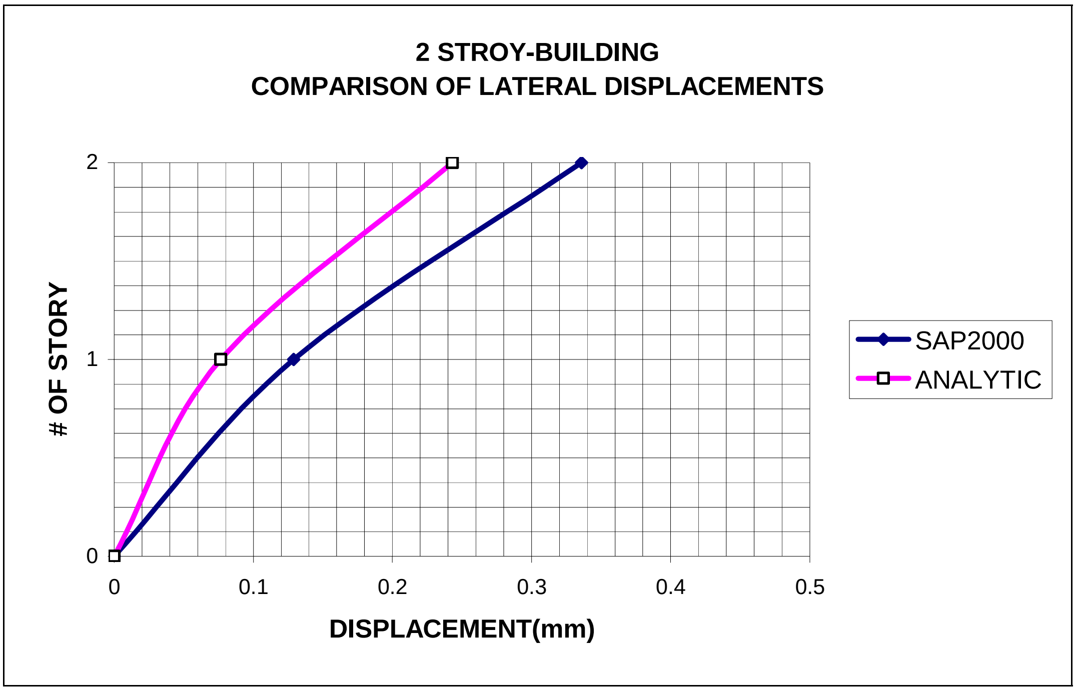


Figure 5.25 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 2 Story-Mixed Structure Example2)

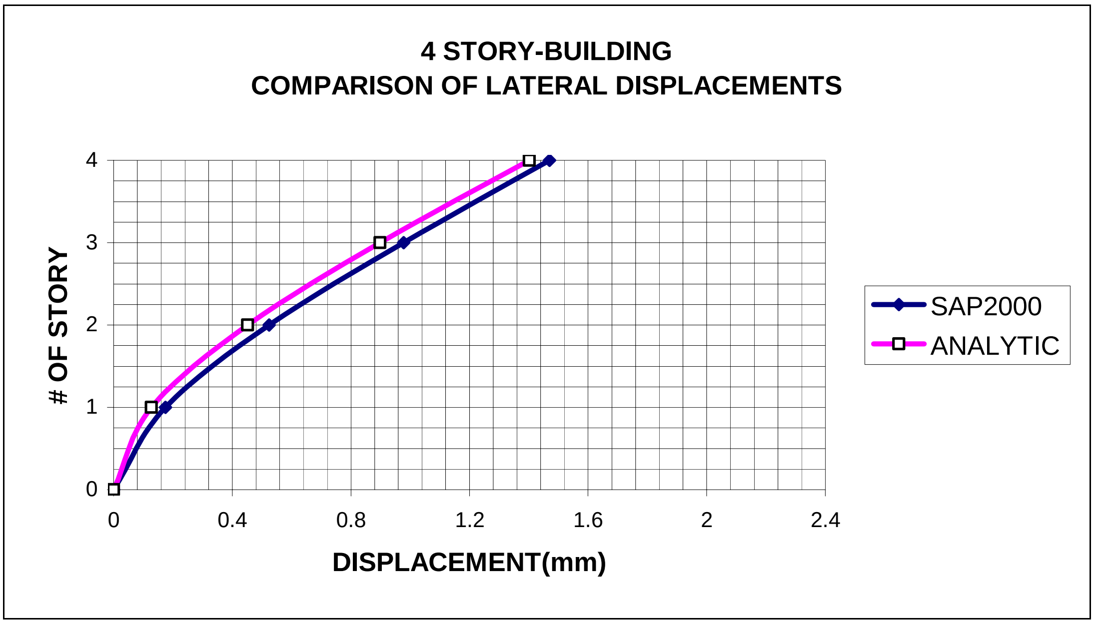


Figure 5.26 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 4 Story-Mixed Structure Example2)

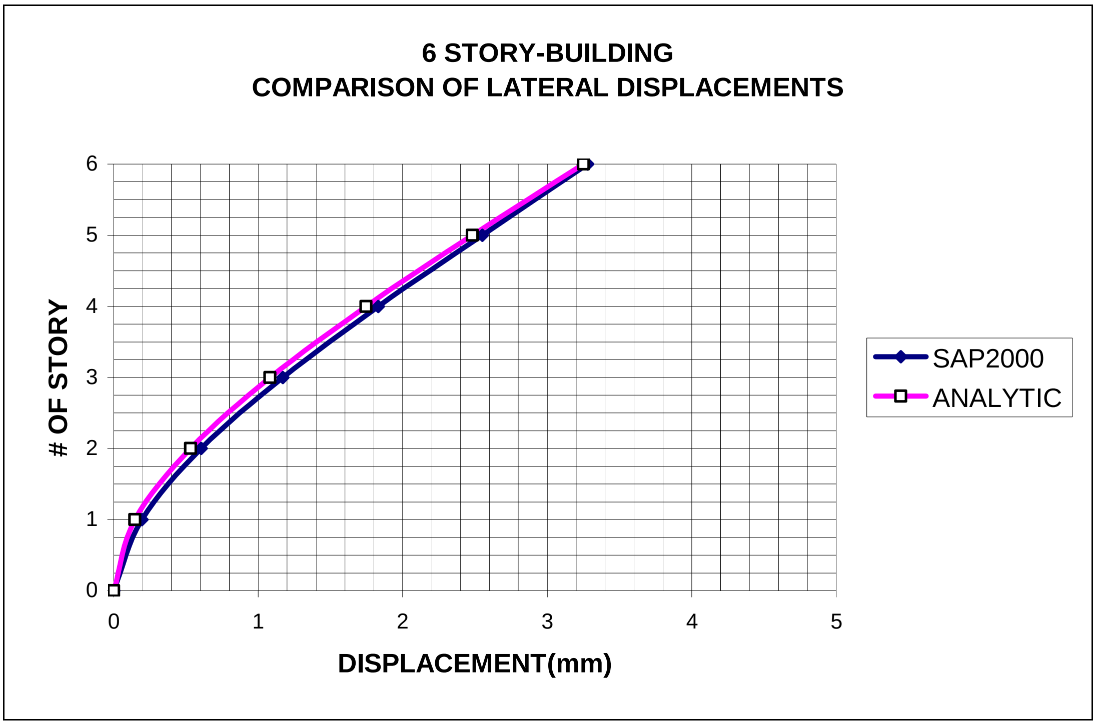


Figure 5.27 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 6 Story-Mixed Structure Example2)

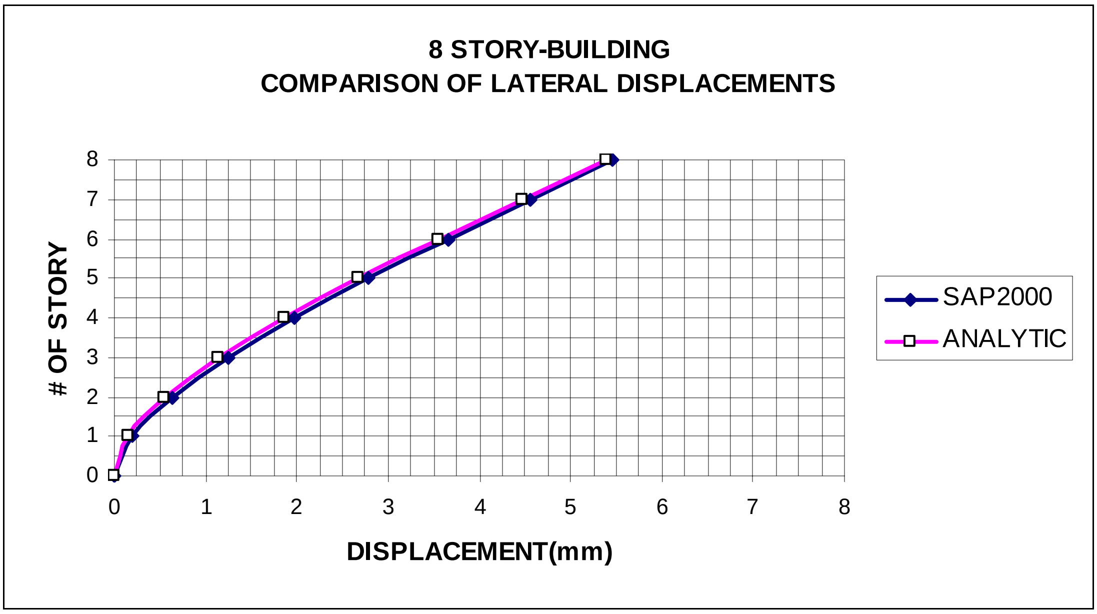


Figure 5.28 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 8 Story-Mixed Structure Example2)

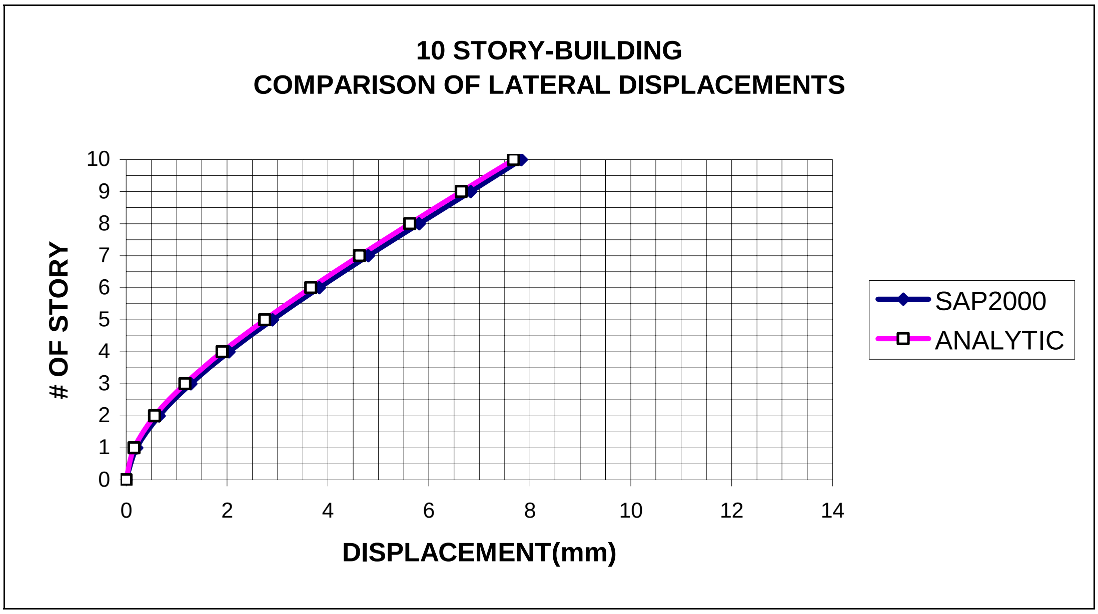


Figure 5.29 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 10 Story-Mixed Structure Example2)

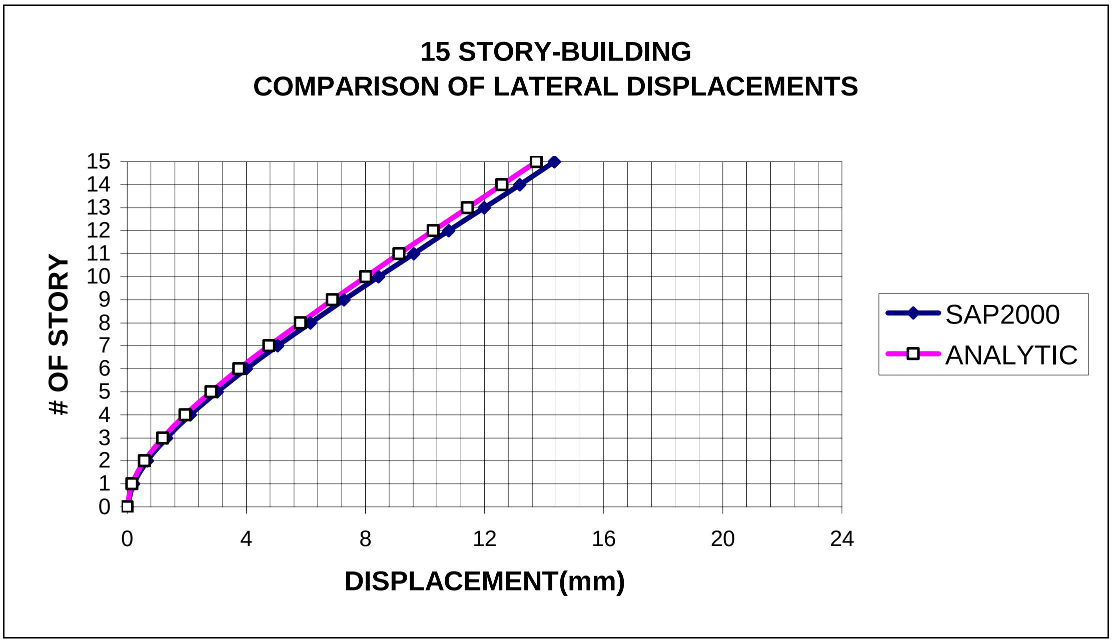


Figure 5.30 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 15 Story-Mixed Structure Example2)

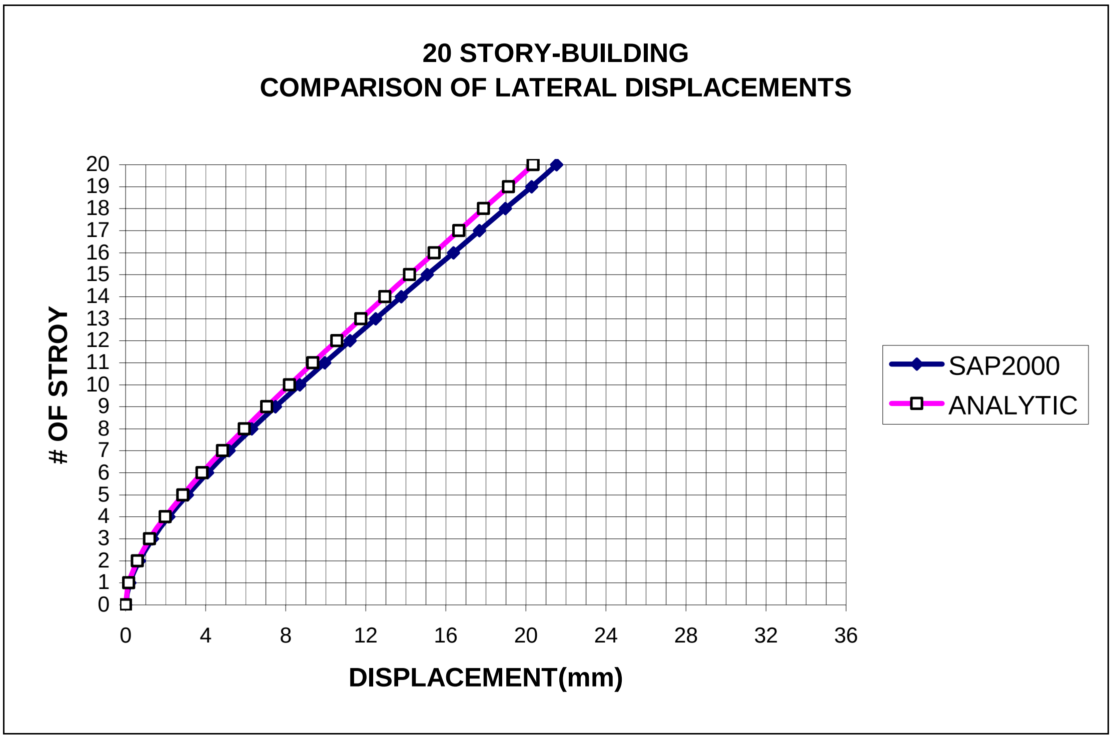


Figure 5.31 Comparisons of Lateral Displacements as Determined by SAP2000 and Analytical Model (for 20 Story-Mixed Structure Example2)

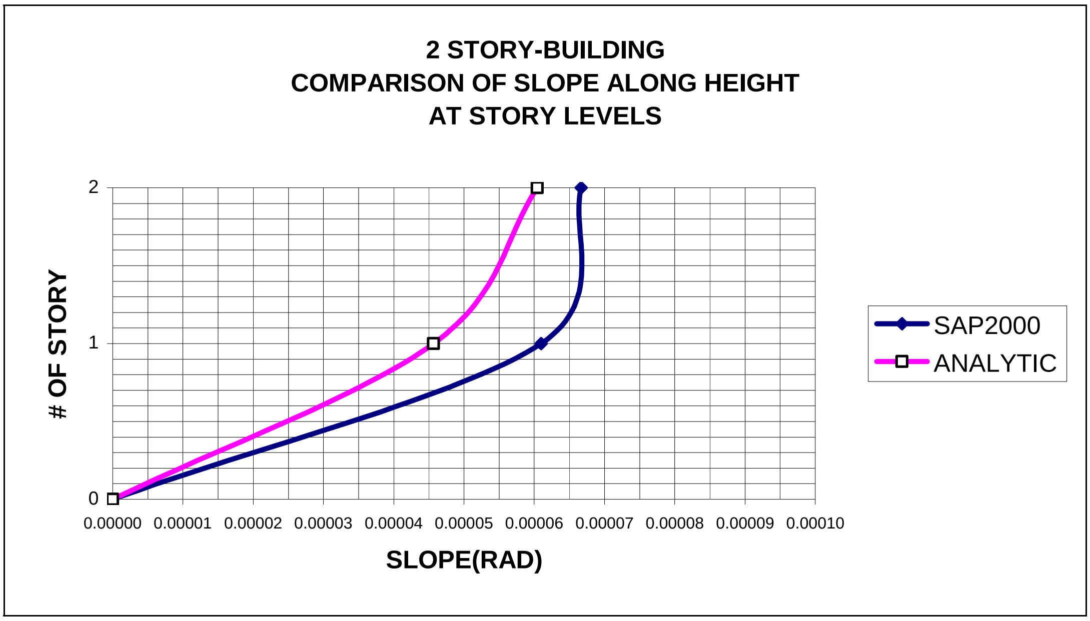


Figure 5.32 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (2 Story-Mixed Structure Example2)

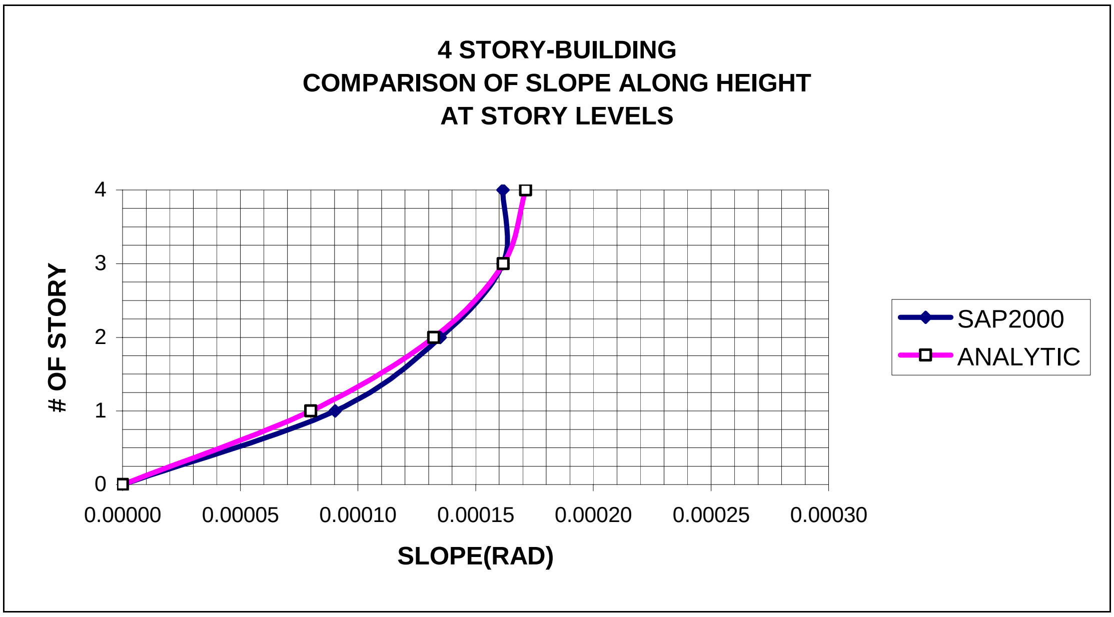


Figure 5.33 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (4 Story-Mixed Structure Example2)

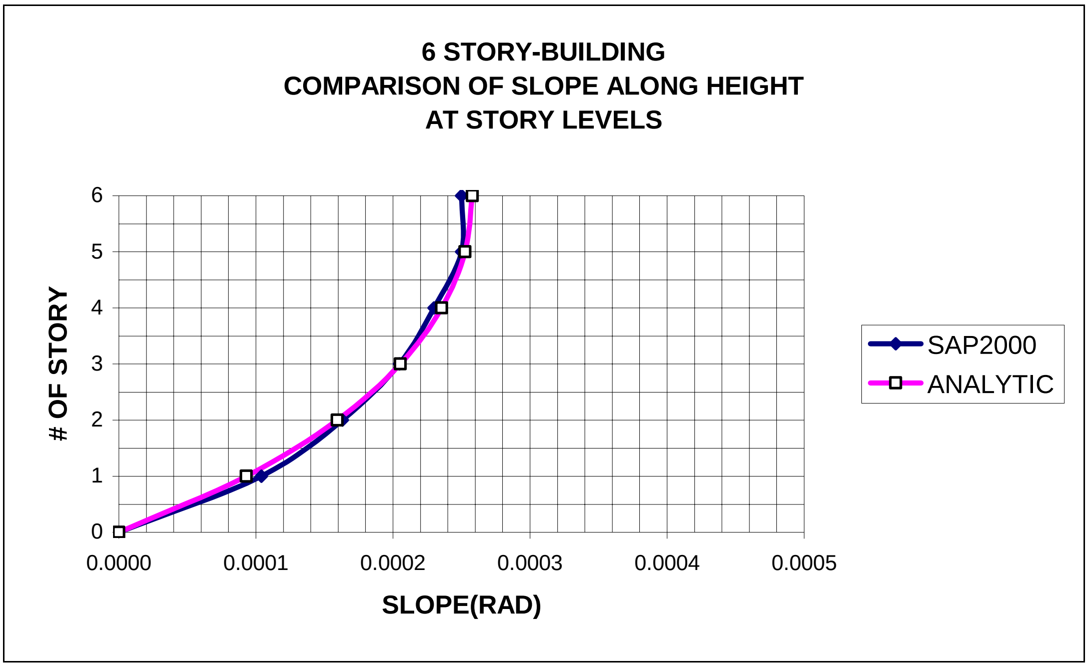


Figure 5.34 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (6 Story-Mixed Structure Example2)

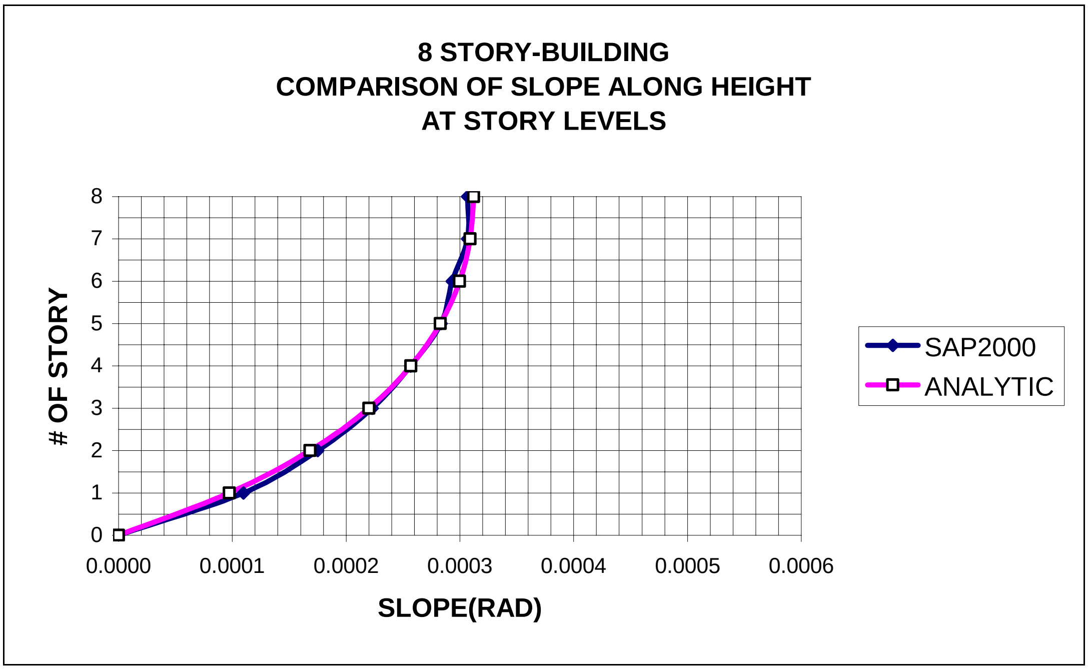


Figure 5.35 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (8 Story-Mixed Structure Example2)

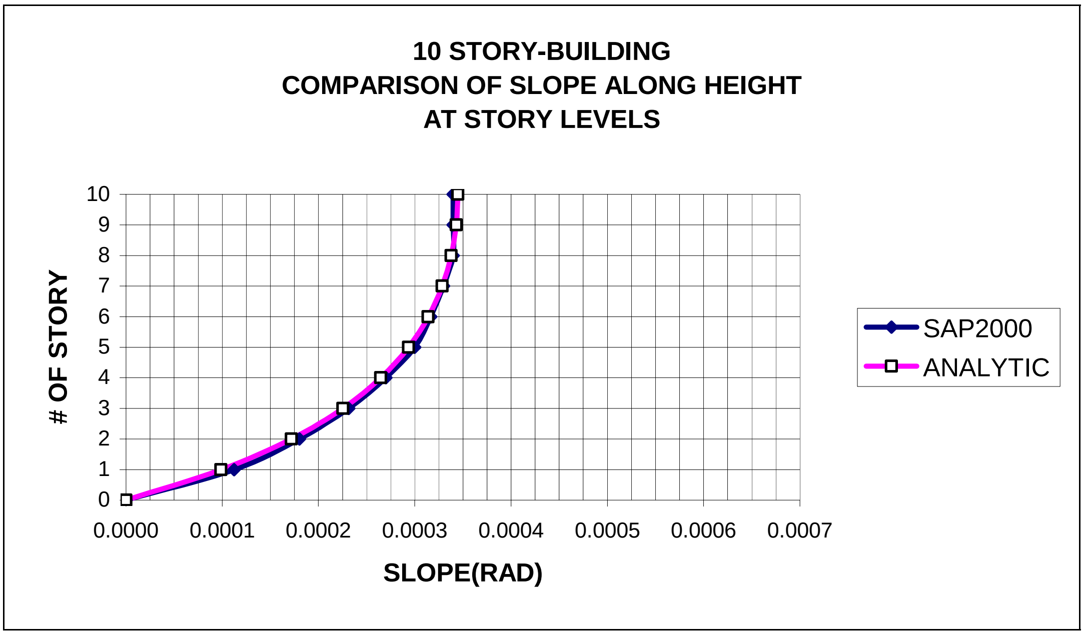


Figure 5.36 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (10 Story-Mixed Structure Example2)

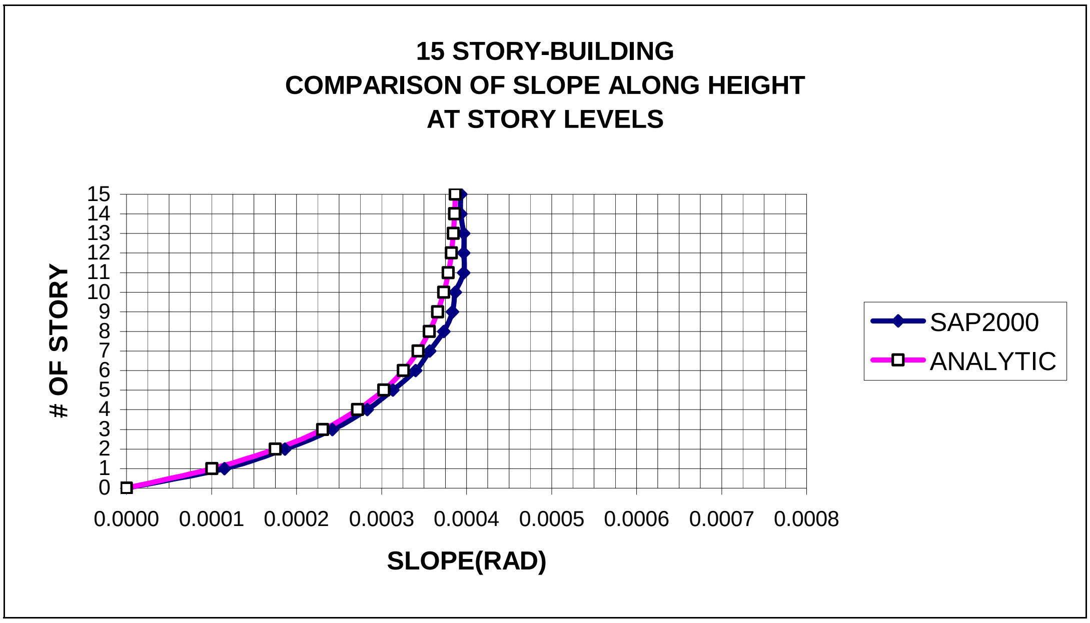


Figure 5.37 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (15 Story-Mixed Structure Example2)

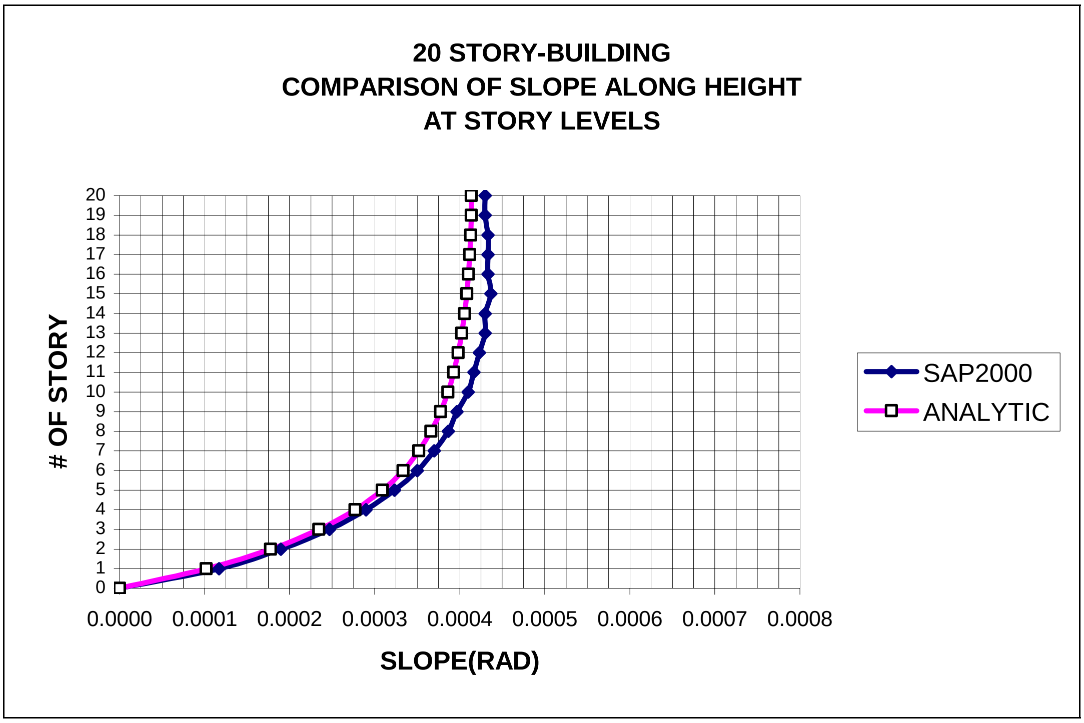


Figure 5.38 Comparisons of Slope along Height at Story Levels as Determined by SAP2000 and Analytical Model (20 Story-Mixed Structure Example2)

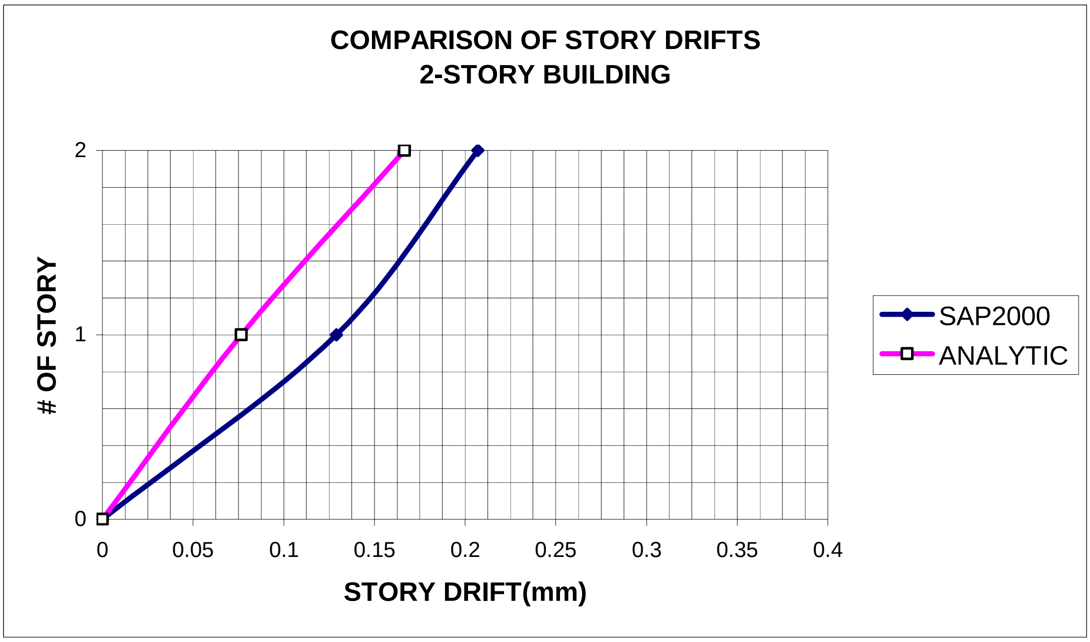


Figure 5.39 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 2 Story-Mixed Structure Example2)

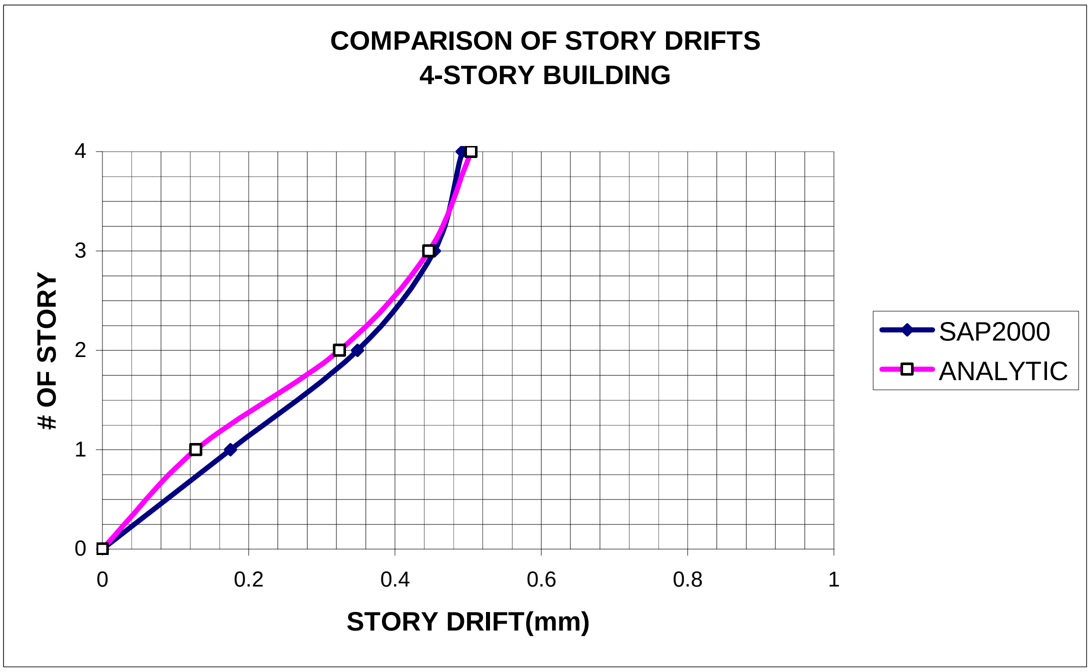


Figure 5.40 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 4 Story-Mixed Structure Example2)

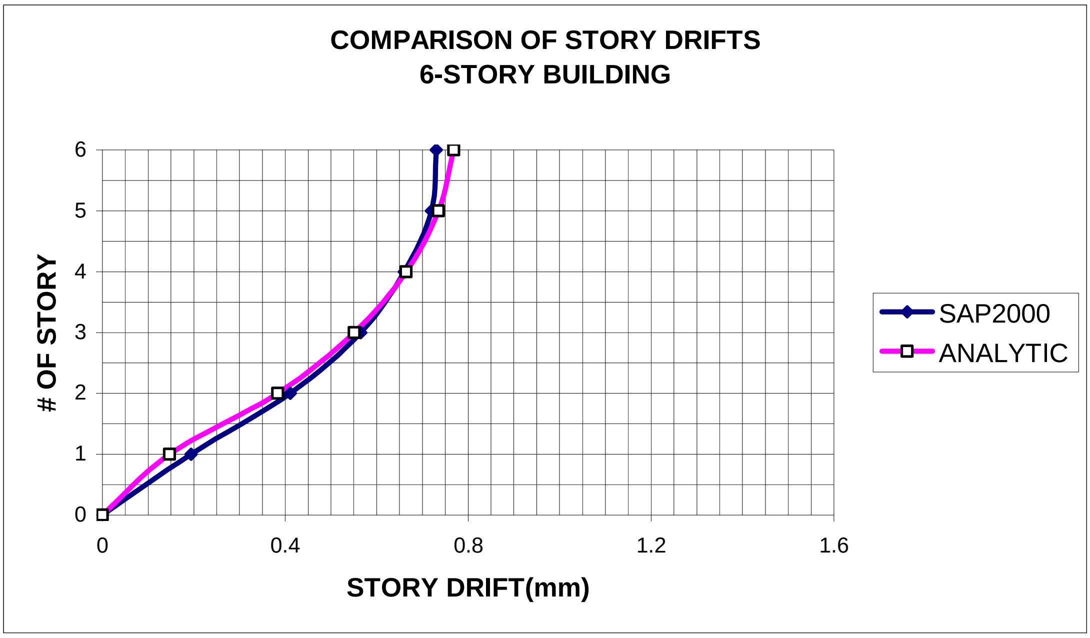


Figure 5.41 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 6 Story-Mixed Structure Example2)

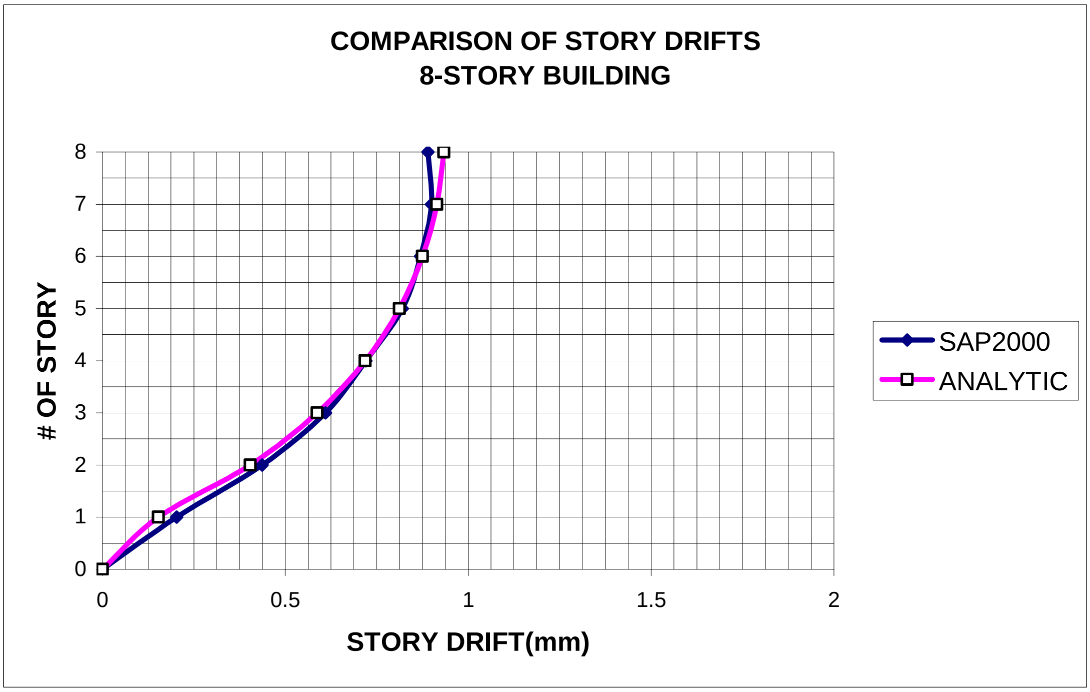


Figure 5.42 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 8 Story-Mixed Structure Example2)

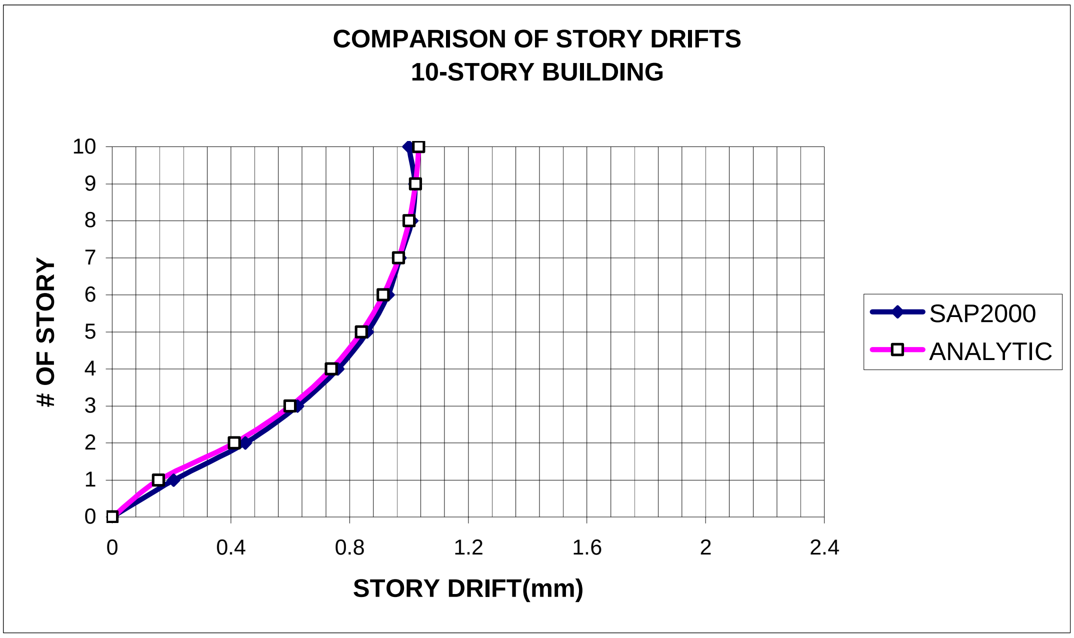


Figure 5.43 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 10 Story-Mixed Structure Example2)

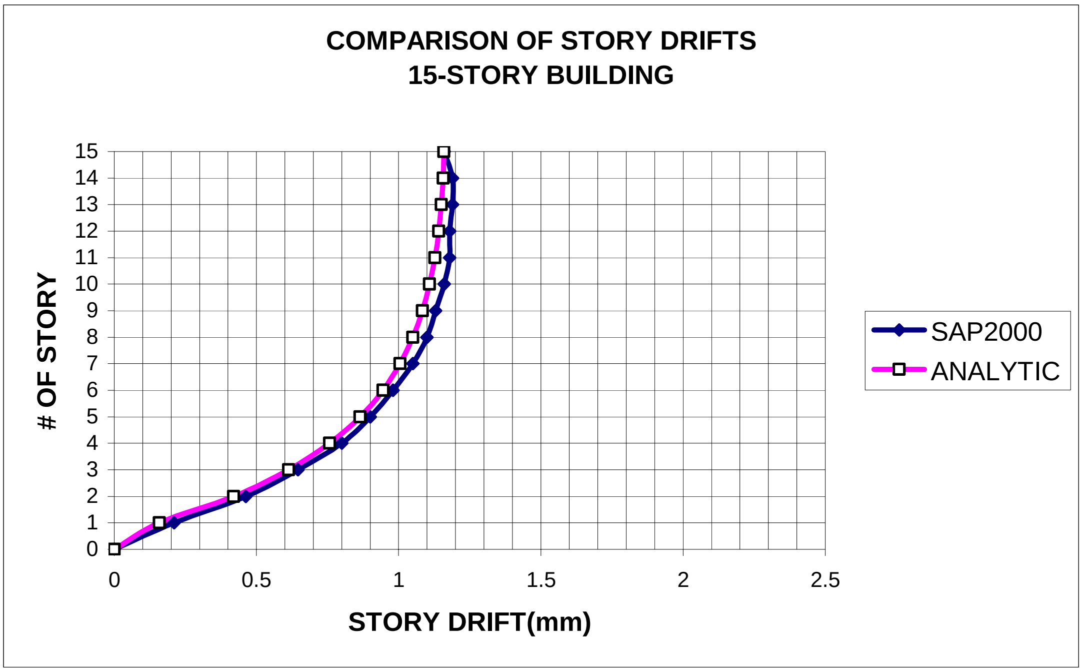


Figure 5.44 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 15 Story-Mixed Structure Example2)

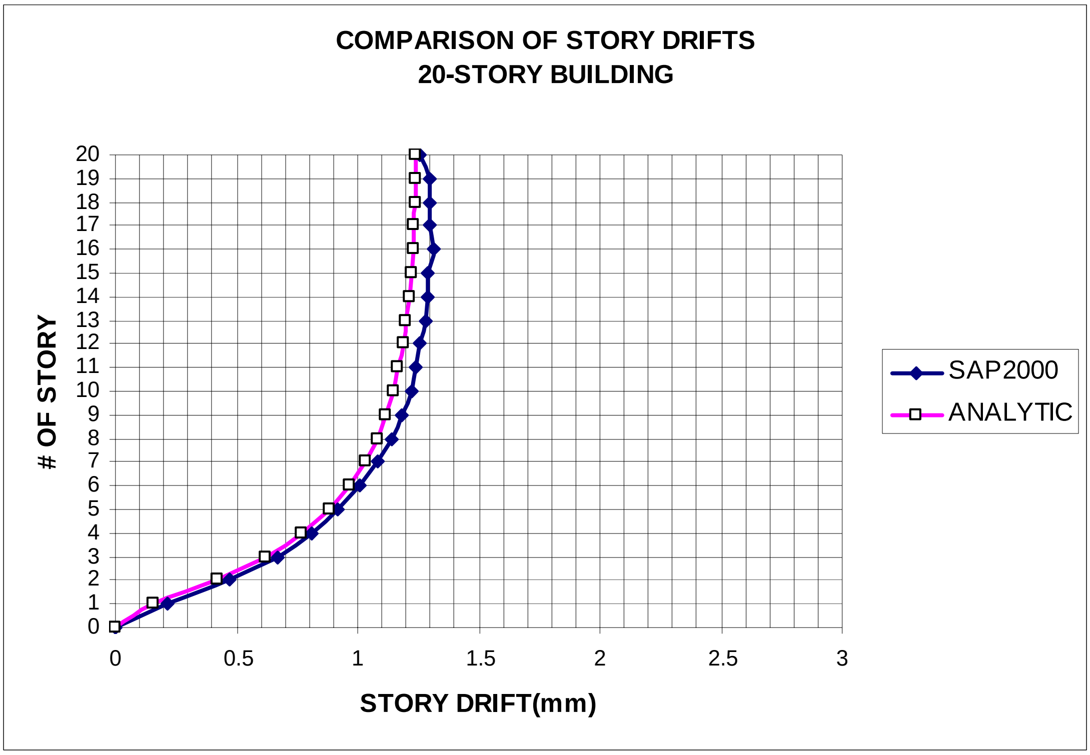


Figure 5.45 Comparisons of Story Drifts as Determined by SAP2000 and Analytical Model (for 20 Story-Mixed Structure Example2)

CHAPTER 6

DEVELOPING SEISMIC DESIGN CRITERIA FOR STRENGTH, STIFFNESS AND DUCTILITY DEMAND OF TALL BUILDINGS

6.1 SATISFYING THE DESIGN CRITERIA FOR STRENGTH DEMAND

6.1.1 General

For ground shaking of large intensity, some repairable damage to contents and the structure may be accepted. Therefore a damage control limit state may be defined, which marks the boundary between economically repairable minor damage and damage that is perhaps not worth repairing. The intensity of ground shaking associated with this limit state should have a low probability of occurrence during the expected life of the building. Here, the most important relevant property is strength developed when the elastic limit is attained or slightly exceeded [74].

6.1.2 Design Strategy for Determining the Necessary Amount of Shear Walls to Meet the Strength Demand ("Dual System" Concept)

What is the correct amount of shear walls to be used, necessary to make a building earthquake resistant? In order to answer this question, the following design strategy will be adopted.

i. The total design base shear must be resisted by shear walls.

ii. Because seismic action occurs in all directions, equal amounts of shear walls must be placed in both orthogonal directions of the structure.

iii. The moment resisting frame elements, which are beams and columns, must independently be able to resist 25 % of the total design base shear.

The Uniform Building Code (UBC, 1999) [61] defines the structure that possesses the above mentioned properties as the "**dual system**".

6.1.3 Determination of the Total Design Base Shear

The total design base shear, V_t, can be determined by using the acceleration response spectrum as defined the same as in both UBC (1999) [61] & Turkish Earthquake Code (1997) [1] and shown in Fig.2.1 & Fig.2.2, respectively.

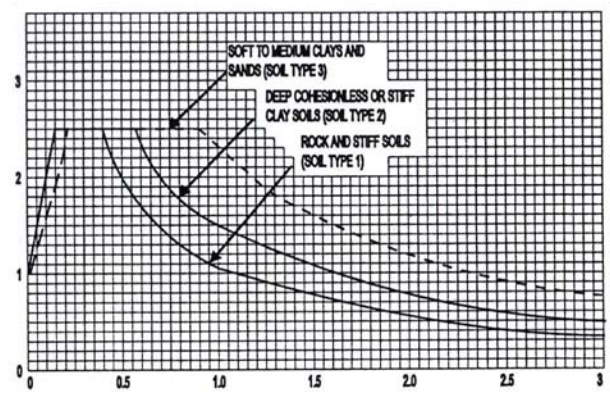


Figure 6.1 Acceleration Response Spectrum (UBC, 1999)

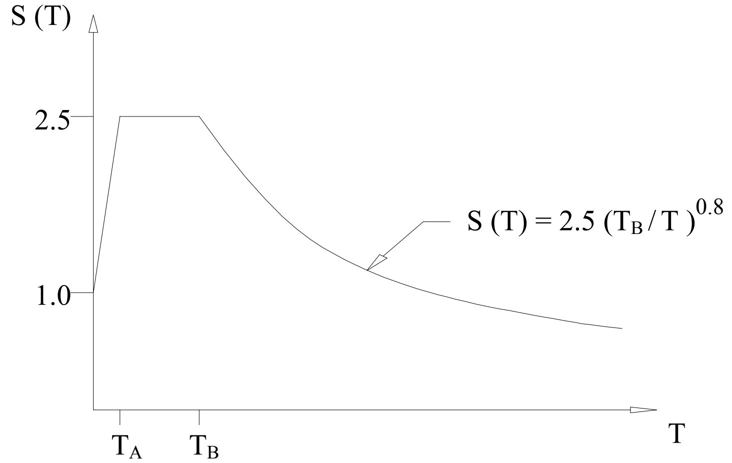


Figure 6.2 Acceleration Response Spectrum (Turkish Earthquake Code, 1997)

A study of the acceleration response spectrum reveals that the effective ground acceleration is magnified by a factor of 2.5, for natural periods of 0.2 – 1.0 seconds that are most typical of building structures commonly employed in practice. Based on this observation, the total design base shear, V_t, can be determined by Eqn.6.1.

$$V_t = S(T).\ (A_0).\ (I).\ (W)\ /\ R \qquad \text{UBC-99 and Turkish Code-97} \qquad (6.1)$$

where

$S(T)$ = spectrum coefficient, which is the ratio of spectral acceleration to effective peak ground acceleration, its maximum value being 2.5

A_0 = effective ground acceleration coefficient

I = building importance factor

W = total weight of the building, as expressed by Eqn.6.2, where A_p is the area of the floor plan

R = seismic force reduction factor (structural behavior factor)

n = number of stories

$$W = \sum_{i=1}^{n} w_i A_{pi} \qquad (6.2)$$

Assuming an average value of w_i (kN/m^2), which is accepted as the same for each story and considering n-stories high, the total design base shear of Eqn.6.1 becomes as expressed in Eqn.6.3.

$$V_t = (2.5)(A_0)(I)(w_i.n.A_p)\ /\ R \quad \text{(kN)} \quad \text{UBC-99 and Turkish Code-97} \qquad (6.3)$$

6.1.4 Determination of Shear Strength of Total Shear Walls

A lower-bound assessment of the shear strength of the total number of shear walls in one orthogonal direction of the building floor plan can be done according to

Uniform Building Code (1999) & Turkish Earthquake Code (1997) as expressed in Eqn.6.4 a & Eqn.6.4 b, respectively.

$$V_r = \varphi \sum A_{ch} (0.166\sqrt{f_c^{'}} + \rho_{sh} f_y) x10^3 \qquad \text{UBC-99} \qquad (6.4\ a)$$

$$V_r = \sum A_{ch} (0.65 f_{ctd} + \rho_{sh} f_{yd}) x10^3 \qquad \text{Turkish Code-97} \qquad (6.4\ b)$$

Considering for C20 / S420 materials, f_c'=20 MPa, f_{ctd}=1 MPa, f_y=420 MPa, f_{yd}=365 MPa and ρ_{sh}=0.0025, Φ=0.7, Eqn.6.5 a & Eqn.6.5 b are obtained, respectively.

$$V_r = 1.25x10^3 \sum A_{ch} \qquad \text{(kN)} \qquad \text{UBC-99} \qquad (6.5\ a)$$

$$V_r = 1.56x10^3 \sum A_{ch} \qquad \text{(kN)} \qquad \text{Turkish Code-97} \qquad (6.5\ b)$$

6.1.5 Determining the Ratio of Total Shear Wall Area to Floor Plan Area

Equating the total design base shear (V_t) to the total shear resistance (V_r) provided by all shear walls in one direction, the ratio of the total area of shear walls to the area of the floor plan can be obtained as expressed in Eqn.6.6 a & Eqn.6.6 b, respectively.

$$\frac{\sum A_{ch}}{A_p} = 0.002.(A_0)(I)(n.w_i)/R \qquad \text{UBC-99} \qquad (6.6\ a)$$

$$\frac{\sum A_{ch}}{A_p} = 0.0016.(A_0)(I)(n.w_i)/R \qquad \text{Proposed Method} \qquad (6.6\ b)$$

Assuming typical values of A_0=0.4, I=1.0, w_i=7 kN/m^2 and R=7, the ratio in Eqn.6.6 a & Eqn.6.6 b can be expressed in a tabular form as a function of the number of stories, as shown in Table 6.1.

It can be easily seen from Table 6.1 that about 1 % of shear wall area will resist the total design base shear for a building that is 15-stories high according to both UBC-99 and proposed method based on the above assumed typical values of A_0, I, w_i and R.

Table 6.1 Ratio of Total Shear Wall Area to Floor Plan Area

Number of Stories (n)	$(\frac{\sum A_{ch}}{A_p})$ (UBC − 99)	$(\frac{\sum A_{ch}}{A_p})$ (Proposed Method)
2	0.0016	0.00128
4	0.0032	0.00256
6	0.0048	0.00384
8	0.0064	0.00512
10	0.008	0.0064
15	0.012	0.0096
20	0.016	0.0128

Note that Table 6.1 was obtained assuming the typical values of A_0=0.4, I=1.0, w_i=7 kN/m^2 and R=7 in Eqn.6.6 a & Eqn.6.6 b. These ratios of total shear wall area to floor plan area change depending on the values of A_0, I, w_i and R used in design.

On the other hand, the amount of total wall area is used as the unfavorable of the following expressions in current practical applications in Turkey.

$\Sigma A_{ch} = 0.02 \times A_p$

$\Sigma A_{ch} = 0.0025 \times \Sigma A_p$

where

A_p = Area of the floor plan

It should be noted that the proposed ratio of total shear wall area to floor plan area is too low to attract attention when compared to the ratio that is currently used in practice in Turkey. While the proposed ratio is almost one tenth of the ratio used in practice for a 4-story building, it is nearly the half of the currently used ratio for a 15-story building. So it is worth to use the proposed ratios in design practices.

6.2 SATISFYING THE DESIGN CRITERIA FOR STIFFNESS DEMAND

6.2.1 General

Relatively frequent earthquakes inducing comparatively minor intensity of ground shaking should not interfere with functionality. This means that no damage to the building and its content needing repair should occur. To achieve this aim, displacements must be limited and resistance provided while the structure remains essentially elastic. The controlling property for this serviceability limit state is stiffness. Because the principles of the analysis of elastic systems are well established, no further attention is given here to this feature [74].

6.2.2 Determination of Stiffness of Total Shear Walls

Using Eqn.6.6 a & Eqn.6.6 b and assuming l_w=3.0m and b_w=0.25m of shear walls, the minimum value of total stiffness of all shear walls (K_{min}) that is calculated by considering the most unfavourable placement of shear walls on the floor plan may be expressed as in Eqn.6.7 a & Eqn.6.7 b, respectively.

K_{min}= 45 000. (A_0). (I). (A_p). ($n.w_i$) / R (kN.m^2) UBC-99 (6.7 a)

K_{min}= 37 500. (A_0). (I). (A_p). ($n.w_i$) / R (kN.m^2) Proposed Method (6.7 b)

6.2.3 Satisfying the Maximum Relative Story Drift Requirement

Using the minimum amount of shear walls obtained from strength requirement of V_r = V_t and the minimum value of total stiffness of all shear walls (K_{min}) expressed as in Eqn.6.7 a & Eqn.6.7 b, stiffness requirement was found to be automatically satisfied up to 9 story-building. After that the minimum stiffness of total shear walls (K_{min}) given in Eqn.6.7 a & Eqn.6.7 b must be increased by a factor of α (the multiplier of K_{min}, which is calculated by assuming l_w=3.0m and b_w=0.25m), which is shown graphically in Figure 6.6.

The value of α was determined by comparing the graph of slope versus number of stories and Code [1] requirement as shown Figure 6.3, Figure 6.4 & Figure 6.5. Slope along height at story levels was 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 was considered as the slope along height at story levels in this study. According to Turkish Earthquake Code [1], 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.

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

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

Since R=7 is used for buildings in which seismic loads are jointly resisted by frames & solid and/or coupled structural walls, the unfavorable condition becomes $0.02 / R = 0.02 / 7 = 0.00286$

The orientation of shear walls to be used in the structure is not important in satisfying that their total stiffness must be greater than $\alpha.K_{min}$. Under all circumferences, total stiffness of all shear walls becomes greater than the required minimum stiffness (K_{min}) and hence the maximum relative story drift requirement is automatically satisfied. For the 3-D buildings tested, it was observed that the maximum relative story drift requirement was always satisfied as long as the minimum amount of shear walls obtained from strength requirement was provided regardless of the orientation of shear walls. In other words, total stiffness of shear walls placed on typical floor plan become always greater than the K_{min} value given in Eqn.6.7 a & Eqn.6.7 b that must be increased by a factor of α defined as in Figure 6.6 for the buildings having more than 9 stories with w_i=7 kN/m^2.

As a summary, K_{min} value given in Eqn.6.7 a & Eqn.6.7 b will be enough to satisfy the stiffness requirement (i.e. drift control) for the buildings having up to 9 stories with w_i=7 kN/m^2. On the other hand, K_{min} value must be increased by a factor of α defined as in Figure 6.6 to satisfy the stiffness requirement (i.e. drift control) for the buildings having more than 9 stories with w_i=7 kN/m^2.

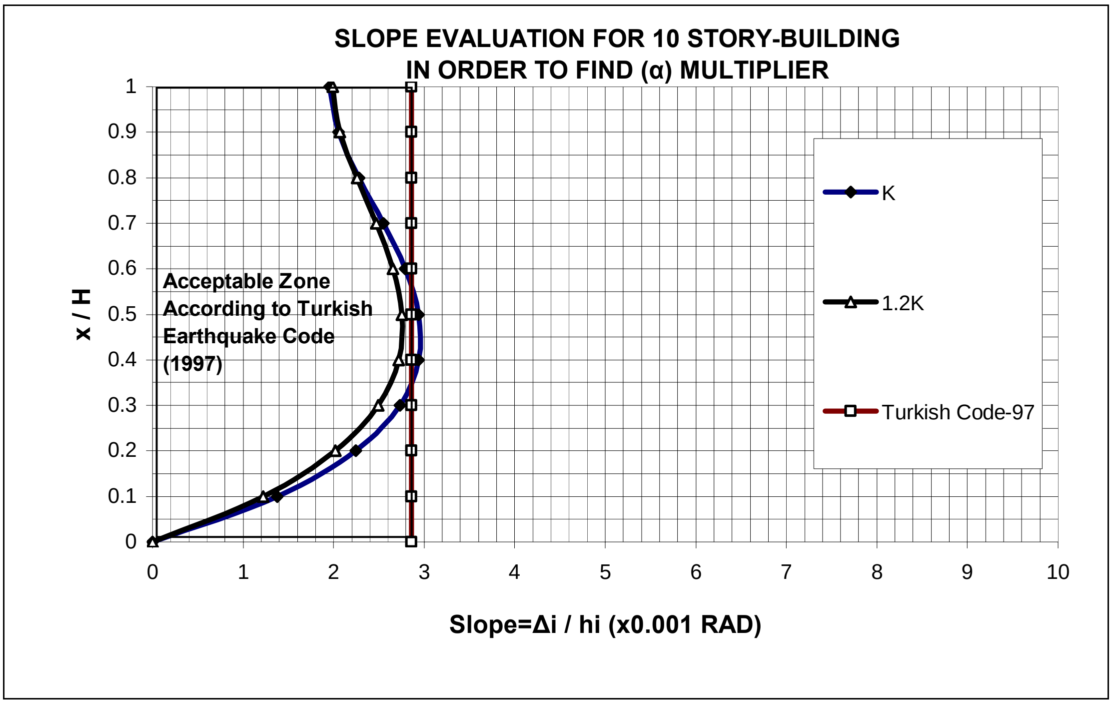


Figure 6.3 Determination of α to Satisfy the Stiffness Requirement for w_i=7 kN/m^2 (for 10 Story-Building)

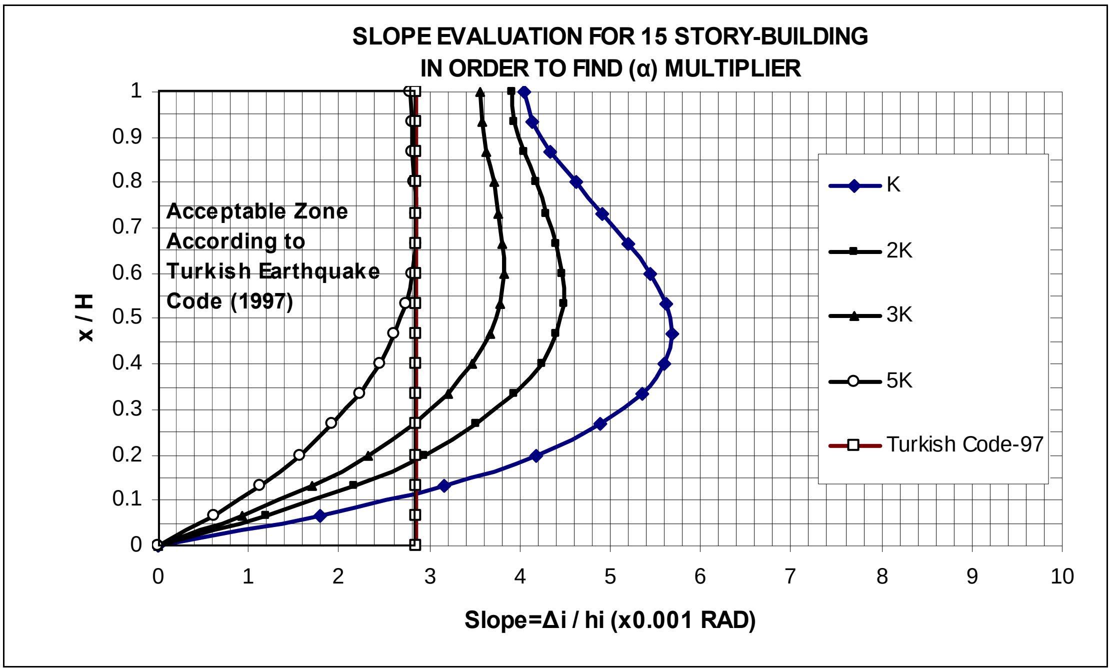


Figure 6.4 Determination of α to Satisfy the Stiffness Requirement for w_i=7 kN/m^2 (for 15 Story-Building)

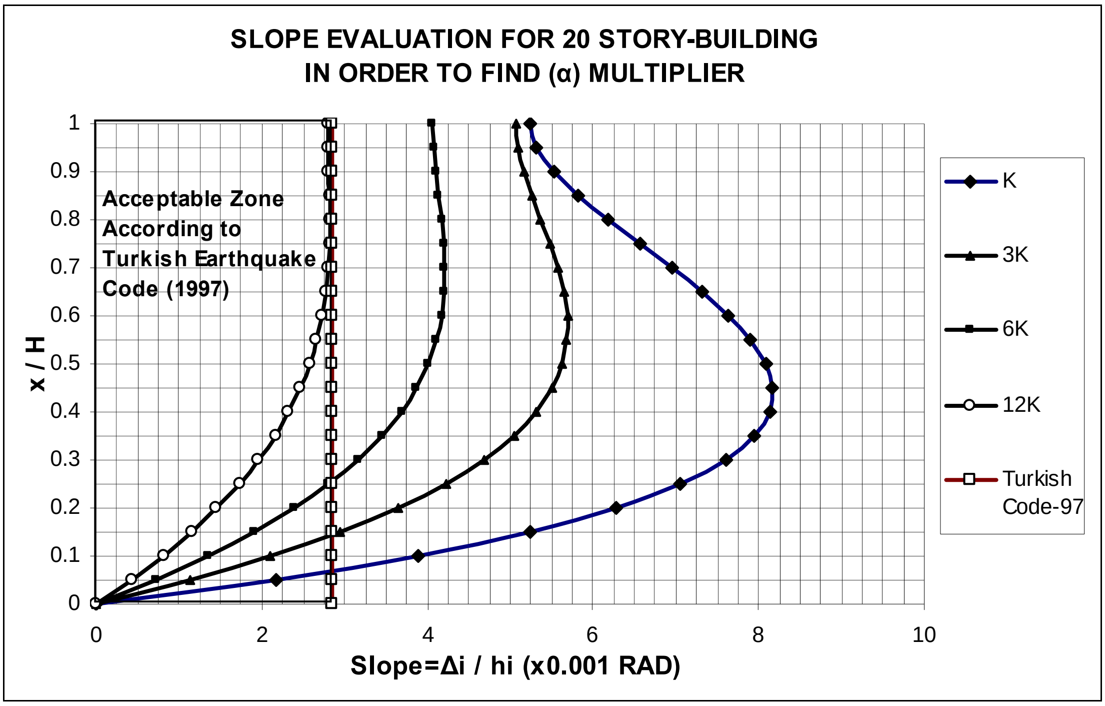


Figure 6.5 Determination of α to Satisfy the Stiffness Requirement for w_i=7 kN/m^2 (for 20 Story-Building)

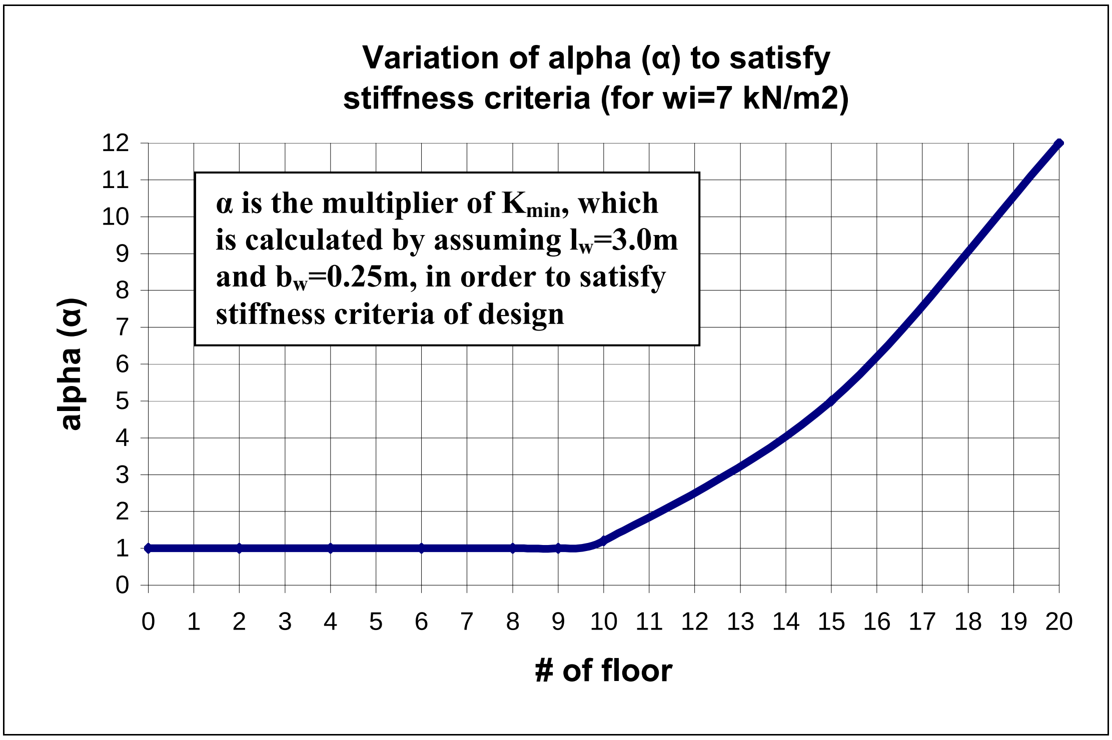


Figure 6.6 Variation of α to Satisfy the Stiffness Requirement for w_i=7 kN/m^2

If w_i was taken as 10 kN/m^2 in the studied example, then the variation of α would be as in Figure 6.7.

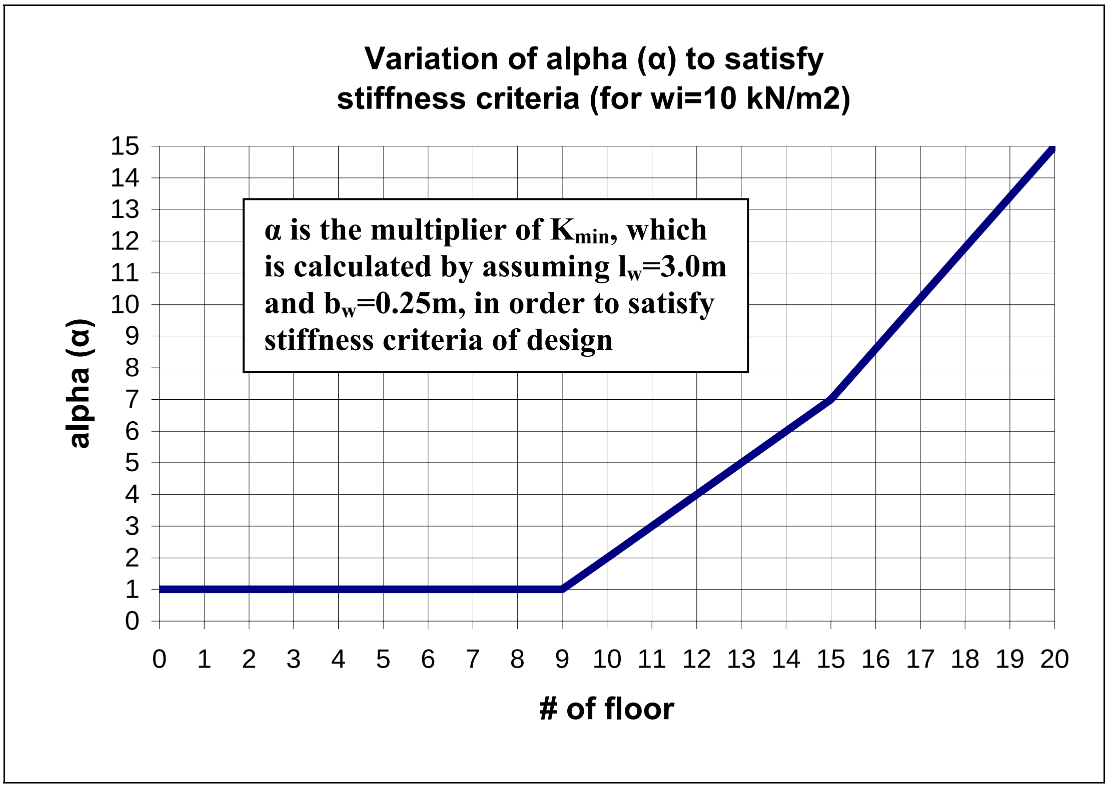


Figure 6.7 Variation of α to Satisfy the Stiffness Requirement for w_i=10 kN/m^2

6.2.4 General Procedure to Find α for Any Particular Case

For any particular case, α can easily be figured out by using the executable "Borland Delphi" program developed, which is shown in Figure 6.8.

First of all, the amount of total shear wall (ΣA_{ch}) is determined by using the following equation.

$$\frac{\sum A_{ch}}{A_p} = 0.0016.(A_0)(I)(n.w_i)/R$$

Secondly assuming b_w=0.25m of shear walls, total length of shear walls to be used in both x and y directions ($\Sigma\ l_w$) is calculated. For the worst situation that may arise in practice, l_w=3.0m of rectangular shear walls are assumed to be placed mostly on the symmetry axis of building so that ($A.d^2$) term will not make any contribution to both K and K_0 values. Then total number of shear walls to be used in both directions can easily be calculated by ($\Sigma\ l_w$) / 3. Hence the total stiffness of all shear walls (K) and the flexural rigidity (K_0), which is due to the most exterior columns only & the shear walls that is not placed on the symmetry axis of building, can be calculated easily as well as the shear rigidity (GA) of all columns.

Finally, all known values of H (height of the building), p (top intensity of uniformly distributed lateral triangular load), K (total stiffness of all shear walls), K_0 (flexural rigidity of building in horizontal plane) and GA (shear rigidity of all columns) are entered into blank spaces provided in the executable "Borland Delphi" program developed, which is shown in Figure 6.8. By looking at the maximum relative story drift value, the value of α that is the multiplier of K can be obtained easily by increasing only the K value until any valid Earthquake Code requirement of relative story drift is satisfied.

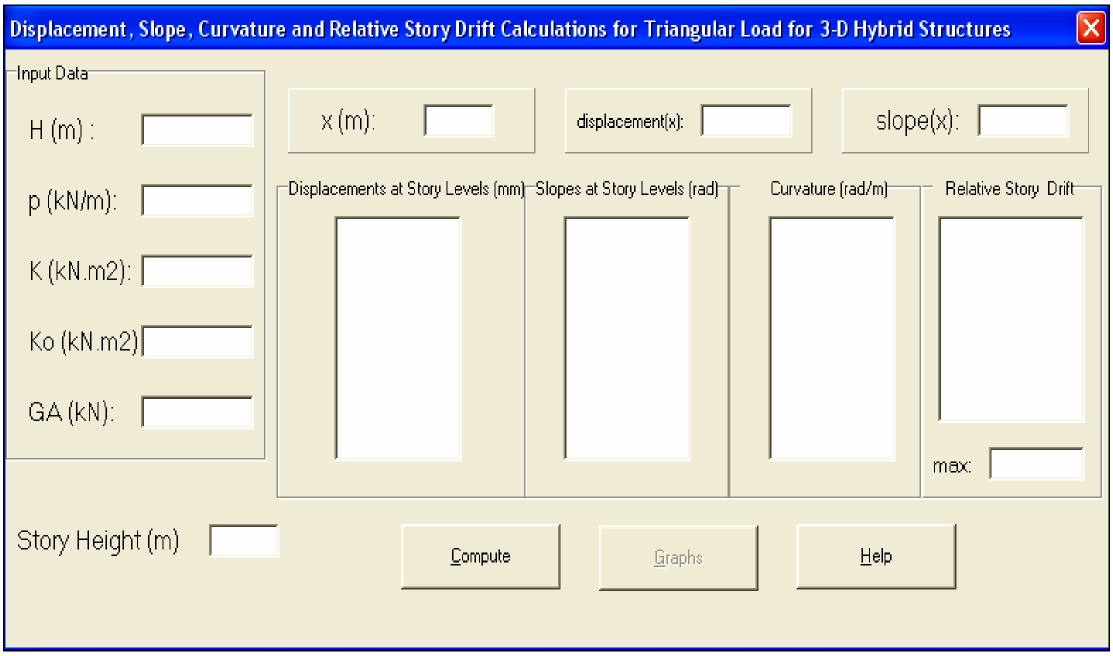


Figure 6.8 Executable "Borland Delphi" Program to Determine α in General

6.2.5 Determination of the Sway Criterion Mentioned in TS 500 (Stability Index Evaluation) [5]

If shear walls or similar members exist in a structural system providing adequate stiffness against horizontal forces, sway can be assumed to be prevented. If the column end moments obtained from a second-order analysis, which is based on linear material behavior assumption, differ at the most by 5 percent from column end moments obtained from a first order analysis, sway can also be assumed to be prevented.

If a second order analysis is not carried out, for cases where the stability index of any story, calculated by considering the whole structural system, does not exceed the limit given below, it can be assumed that adequate stiffness exists in that story and sway is prevented. The stability index is expressed as in Eqn.6.8 in TS 500.

$$\varphi = 1.5\Delta_i \frac{\sum \frac{N_{di}}{l_i}}{V_{fi}} \leq 0.05 \qquad (6.8)$$

where

Φ= Stability index

Δ_i= Drift at i^{th} story

N_{di}= Axial design load

l_i= i^{th} story column length, measured from axis to axis

V_{fi}= Total shear force at i^{th} story

The value to be used in En.6.8 should be calculated using uncracked sections and the most critical of the following load combinations:

F_d= 1.0G + 1.0Q + 1.0E or F_d= 1.0G + 1.3Q + 1.3W

where

G= Permanent load effect (dead load effect)

Q= Live load effect

E= Seismic load effect

W= Wind effect

The required drift (Δ_i) to be used in Eqn.6.8 can be calculated easily by the analytical method presented. Therefore, it becomes very easy to check whether a structure is sway prevented or not without three-dimensional computer modeling.

6.2.6 Limitation of Second Order Effects According to Turkish Earthquake Code [1]

Unless a more refined analysis considering the nonlinear behavior of structural system is performed, second order effects may be taken into account according to the following equation given in Turkish Earthquake Code [1].

$$\theta_i = \frac{(\Delta_i)_{ort} . \sum_{j=1}^{N} w_j}{V_i . h_i} \leq 0.12$$

In the cases where second order effect indicator, θ_i, satisfies the condition given by the above equation for the earthquake direction considered at each storey, second order effects shall be evaluated in accordance with currently enforced specifications of reinforced concrete design. Here $(\Delta_i)_{ort}$ shall be determined as the average value of story drifts calculated for i'th storey columns and structural walls.

In the case where the condition given by the above equation is not satisfied, seismic analysis shall be repeated by sufficiently increasing the stiffness of the structural system.

The required average drift $(\Delta_i)_{ort}$ to be used in the above equation of second order effect indicator can be calculated easily by using the analytical method presented. Therefore, it becomes very easy to check whether the second order effects shall be taken into account or not without three-dimensional computer modeling.

6.3 SATISFYING THE DESIGN CRITERIA FOR DUCTILITY DEMAND

6.3.1 General

Significant damage during very large earthquakes, perhaps beyond repair, must be expected. However, design and construction must ensure that collapse resulting in loss of life will not occur. Therefore the designer must concentrate on structural qualities which will ensure that for the expected duration of an earthquake, relatively large displacements can be accommodated without significant loss of lateral force resistance, and that the integrity of the structure to support gravity loads is maintained. The most important property associated with this survival limit state is ductility, that is, tolerance for large inelastic deformations without significant loss of resistance. The exploitation of this property is a relatively recent feature in the evolution of structural engineering [74].

6.3.2 Satisfying the Ductility Demand

Under the total acting design base shear, it is expected that plastic hinges form at the base of shear walls. During the formation of plastic hinges it is further required that the structure behave ductile.

Ductility is a qualitative term, and it needs to be quantified. To quantify ductility, the commonly accepted measure is the displacement ductility ratio, μ_Δ, as given in Eqn.6.9.

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} \tag{6.9}$$

where

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

In practice, μ_Δ = 4-5 is considered to provide enough ductility. At ultimate stage, plastic hinges form at beam ends, in addition to hinges at the base of shear walls.

However, the above criterion necessitates the calculation of top sway of a reinforced concrete structure, which is rather tedious and uncertain. It is mush easier to calculate the cross-sectional curvature, as commonly expressed by the axial load-moment-curvature relationship (N-M-Φ). Therefore, it will be most convenient if an expression could be developed to relate displacement ductility ratio, μ_Δ, to the curvature ductility ratio, μ_Φ, as shown in Figure6.9.

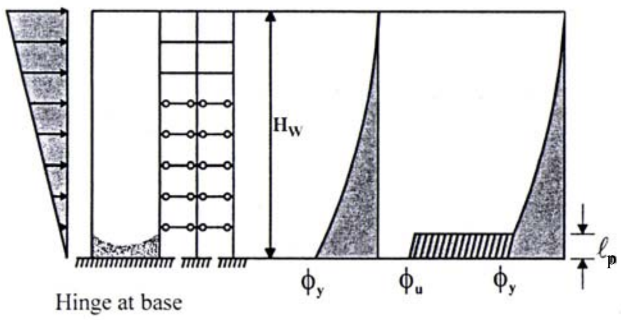


Figure 6.9 Relating the Top Sway of Building to the Cross-Sectional Curvature of Shear Wall [2]

$$\Delta_y = \text{Area}.\bar{y} = (\frac{1}{3}.\varphi_y.H_w).(\frac{3}{4}.H_w) \quad (6.10)$$

$$\Delta_u = \Delta_y + (\varphi_u - \varphi_y).(l_p).(H_w - \frac{l_p}{2}) \quad (6.11)$$

Consider a shear wall - frame structure where a plastic hinge of height l_p= $0.2l_w+0.044H_w$ has formed at the base of the shear wall, as shown in Figure 6.9. The sway at the top of the structure can be calculated at the time of initiation of yielding and concrete crushing as Δ_y and Δ_u, respectively, as expressed in Eqn.6.10 and Eqn.6.11. Consequently, displacement ductility can be readily calculated, as given in Eqn.6.12.

$$\mu_{\Delta} = \frac{\Delta_u}{\Delta_y} = 1 + \frac{4}{H_w^2}.(\frac{\varphi_u}{\varphi_y} - 1).(l_p).(H_w - \frac{l_p}{2}) \quad (6.12)$$

From Eqn.6.12, the curvature ductility ratio, μ_{Φ}, can be readily solved for, as expressed in Eqn.6.13.

$$\mu_{\varphi} = \frac{\varphi_u}{\varphi_y} = \frac{1}{4}.\frac{H_w^2}{l_p}.\frac{(\mu_{\Delta} - 1)}{H_w - 0.5l_p} + 1 \quad (6.13)$$

The displacement ductility ratio, μ_{Δ}, is thus expressed in terms of the more familiar curvature ductility ratio, μ_{Φ}, which only depends on cross-sectional properties of the shear wall and the axial load on it.

6.3.3 Designing the Ductile Reinforced Concrete Structures

A design strategy and its application were outlined for reinforced concrete buildings in which earthquake resistance was provided by ductile frames, by ductile shear walls or by the interactive combined actions of these two systems. The design strategy described in this study evolved from the following fundamental principles [74]:

1) In the content of the state of the art in structural engineering, current predictions of the probable characteristics of large earthquake-generated ground motions are crude. Under these circumstances an aim to achieve a degree of precision in analytical techniques, comparable to those developed for structures to satisfy serviceability and "hypothetical" ultimate limit states, to predict both earthquake-induced actions and deformations within the structure, is not justified.

2) Provided that a reasonable level of resistance to lateral forces, such as prescribed for various seismic regions by relevant national building codes, is chosen, errors arising from crude estimations of the characteristics of ground motions will manifest themselves only in erroneous predictions of earthquake-imposed displacements, that is ductility demands. Thus deformation capacity is the most important structural property in areas of high seismic risk.

3) Types and locations of energy dissipation mechanisms need to be chosen as part of the capacity design procedure, in which a unique hierarchy of strength is

established. All weak and necessarily ductile links must satisfy requirements of the stipulated level of lateral force resistance. The distribution of minimum strengths throughout a ductile structure, both horizontally and vertically, may be based on a simple analysis of elastic systems with subsequent redistribution of design actions, sometimes quite significant, from less to more desirable locations. In such an inelastic system the maximum resistance that may be developed during a major earthquake can be predicted with a relatively high degree of precision. However, ductility demands during an earthquake, being dependent on ground motions, may differ from those anticipated or assumed in building codes.

4) As a general rule, rationally-detailed structures can be made very ductile with relative ease and little if any additional cost. Thereby a considerable reserve in inelastic deformations, that is ductility capacity, can be imparted to structural systems. Detailing of reinforced concrete structures, very often considered a subordinate, depreciated drafting activity with apparent lack in intellectual appeal, deserves at least as much attention as the analytical work used to estimate design actions. Faults in detailing are the first that will be revealed during earthquakes. They are predominant causes of structural distress. The detailing of potential plastic regions is partly an art. It relies on feel for and understanding of the natural disposition of internal forces and often invites innovations. Judiciously-detailed ductile systems will be tolerant with respect to imposed seismic displacements, a valuable feature of structural response, which will compensate for the crudeness in predicting magnitudes of such displacements.

5) Various steps in the description in previous sections of the design procedure were intended to emphasize the designer's determination to simply "tell the structure what to do". It is in this respect that the design strategy is deterministic. It inhibits the activation of mechanisms other than those chosen. The numerous detailed recommendations presented were intended to manifest unambiguously the goodness of detailing. Thereby reinforced concrete buildings can be made very tolerant to a wide range of ductility demands. Hence they can be expected with confidence to perform "as they were told to do".

6.3.4 Designing the Ductile Shear Walls

Under the light of above mentioned fundamental principles, the following steps should be followed for the design of the ductile shear walls.

i. The seismic demand on each shear wall should be determined by a proper structural analysis.

ii. The minimum amount of reinforcement (ρ_{sh}=0.0025) should be uniformly distributed in the cross section. The end zone details should also be carefully applied, as dictated by the valid seismic code.

iii. The (N-M-Φ) relationship should be developed, as commonly done by an available computer program. The developed (N-M-Φ) relationship will give the yield curvature, Φ_y and the ultimate curvature, Φ_u. Thus the curvature ductility ratio, μ_Φ, can be readily calculated as $\mu_\Phi = \Phi_u / \Phi_y$.

iv. The curvature ductility demand, μ_Φ, corresponding to the displacement ductility demand of $\mu_\Delta = 4 - 5$ can be easily calculated.

v. The supplied μ_Φ as obtained from the (N-M-Φ) relationship must be greater than demanded; otherwise the cross-section must be revised.

6.3.5 Relationship between Curvature Ductility (μ_Φ) and Displacement Ductility (μ_Δ) for Structural Systems Comprised of Walls and Fames

The equation of curvature along the height of mixed structures (i.e. along the height of shear wall) was developed as given in Eqn.6.14.

$$K.y''(x) = A_1.\cosh\varphi + A_2.\sinh\varphi + (1 - \frac{1}{v^2})pH^4. \\ (\frac{1}{3.H^2} - \frac{x}{2.H^3} + \frac{x^3}{6.H^5}) - \frac{s^2.p}{v^2.H}.x \quad (6.14)$$

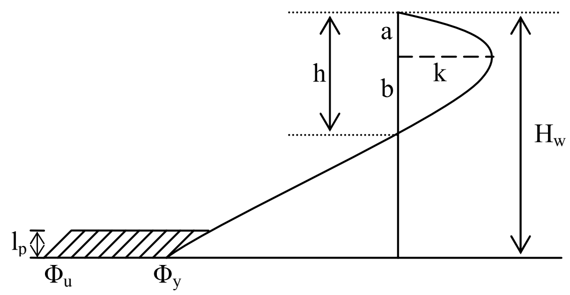


Figure 6.10 Relating the Top Sway of Building to the Cross-Sectional Curvature of Shear Wall for Mixed Systems

The sway at the top of the structure can be calculated at the time of initiation of yielding, Δ_y, and concrete crushing, Δ_u, as expressed in Eqn.6.15 and Eqn.6.16, respectively.

$$\Delta_y = \frac{5(\varphi_y)(H_w - h)(3H_w + h) - k(5a + 4b)^2}{60} \tag{6.15}$$

$$\Delta_u = \Delta_y + (\varphi_u - \varphi_y)(l_p)(H_w - 0.5l_p) \tag{6.16}$$

Consequently, displacement ductility can be readily calculated, as given in Eqn.6.17.

$$\mu_\Delta = \frac{\Delta_u}{\Delta_y} = 1 + \frac{60(\varphi_u - \varphi_y)(l_p)(H_w - 0.5l_p)}{5(\varphi_y)(H_w - h)(3H_w + h) - k(5a + 4b)^2} \tag{6.17}$$

From Eqn.6.17, the curvature ductility ratio, μ_Φ, can be readily solved for, as expressed in Eqn.6.18.

$$\mu_\varphi = \frac{\varphi_u}{\varphi_y} = 1 + \frac{(\mu_\Delta - 1)\left[5(\varphi_y)(H_w - h)(3H_w + h) - k(5a + 4b)^2\right]}{60(l_p)(H_w - 0.5l_p)(\varphi_y)} \tag{6.18}$$

In case of a = b = h / 2, displacement ductility (μ_Δ) and curvature ductility (μ_Φ) can be expressed as in Eqn.6.19 & Eqn.6.20, respectively.

$$\mu_\Delta = 1 + \frac{12(\varphi_u - \varphi_y)(l_p)(H_w - 0.5l_p)}{(\varphi_y)(H_w - h)(3H_w + h) - 4kh^2} \tag{6.19}$$

$$\mu_\varphi = 1 + \frac{(\mu_\Delta - 1)\left[(\varphi_y)(H_w - h)(3H_w + h) - 4kh^2\right]}{12(l_p)(H_w - 0.5l_p)(\varphi_y)} \tag{6.20}$$

The curvature distribution along the height of mixed structures may sometimes be as in Figure 6.11. This generally happens in high rise buildings.

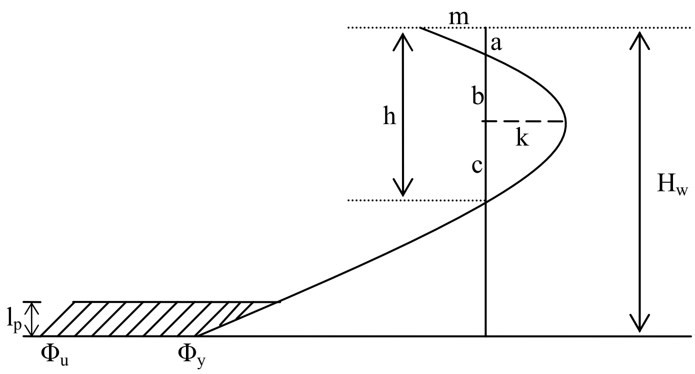


Figure 6.11 Relating the Top Sway of Building to the Cross-Sectional Curvature of Shear Wall (The Most General Case)

In the case given in Figure 6.11, the sway at the top of the structure can be calculated at the time of initiation of yielding, Δ_y, and concrete crushing, Δ_u, as expressed in Eqn.6.21 and Eqn.6.22, respectively.

$$\Delta_y = \frac{5(\varphi_y)(H_w - h)(3H_w + h) - k\left((5b + 4c)^2 + 40a(b + c)\right) + 5a^2m}{60} \tag{6.21}$$

$$\Delta_u = \Delta_y + (\varphi_u - \varphi_y)(l_p)(H_w - 0.5l_p) \quad (6.22)$$

Consequently, displacement ductility can be readily calculated, as given in Eqn.6.23.

$$\mu_\Delta = 1 + \frac{60(\varphi_u - \varphi_y)(l_p)(H_w - 0.5l_p)}{5(\varphi_y)(H_w - h)(3H_w + h) - k\left((5b+4c)^2 + 40a(b+c)\right) + 5a^2m} \quad (6.23)$$

From Eqn.6.23, the curvature ductility ratio, μ_Φ, can be readily solved for, as expressed in Eqn.6.24.

$$\mu_\varphi = 1 + \frac{(\mu_\Delta - 1)\left[5(\varphi_y)(H_w - h)(3H_w + h) - k\left((5b+4c)^2 + 40a(b+c)\right) + 5a^2m\right]}{60(l_p)(H_w - 0.5l_p)(\varphi_y)} \quad (6.24)$$

6.4 DETERMINING THE MOMENT, SHEAR AND LOADING EQUATIONS

The equation of curvature along the height of mixed structures (i.e. along the height of shear wall), multiplied by K multiplier, was developed as given in Eqn.6.25.

$$K.y''(x) = A_1.\cosh\varphi + A_2.\sinh\varphi + (1 - \frac{1}{v^2})pH^4.\left(\frac{1}{3.H^2} - \frac{x}{2.H^3} + \frac{x^3}{6.H^5}\right) - \frac{s^2.p}{v^2.H}.x \quad (6.25)$$

Moment equation can easily be obtained by multiplying the curvature with EI as given in Eqn.6.26 and so Eqn.6.27 is attained.

$$M(x) = -EI \,.\, y''(x) \quad (6.26)$$

$$M(x) = -\frac{EI}{K}[A_1.\cosh\varphi + A_2.\sinh\varphi + (1-\frac{1}{v^2})pH^4.$$
$$(\frac{1}{3.H^2} - \frac{x}{2.H^3} + \frac{x^3}{6.H^5}) - \frac{s^2.p}{v^2.H}.x] \qquad (6.27)$$

Moment distribution profile along the height of shear wall, which is the same as curvature distribution, is shown in Figure 6.12.

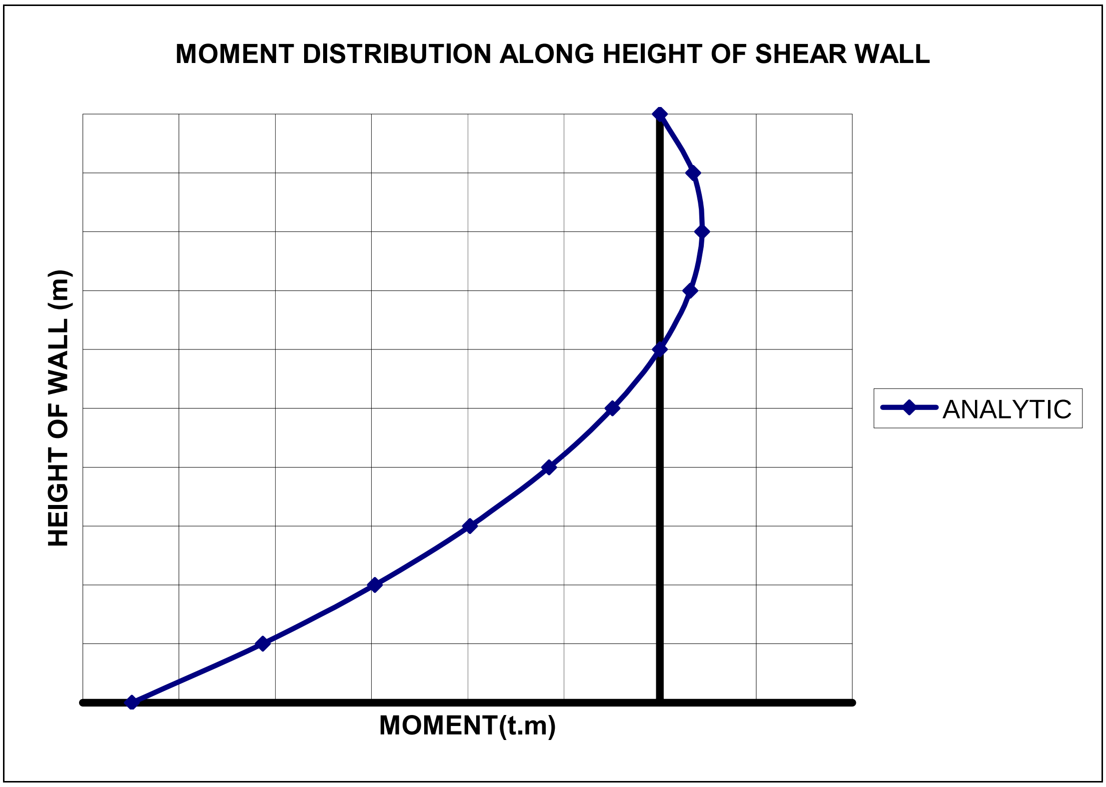


Figure 6.12 Moment Distribution Profile along Height of Shear Wall

Shear equation can then be readily obtained by differentiating the moment equation with respect to x as expressed in Eqn.6.28 and so Eqn.6.29 is attained.

$$V(x) = -M'(x) = EI \,.\, y'''(x) \qquad (6.28)$$

$$V(x) = \frac{EI}{K}.\left\{\frac{A_1}{s}\sinh\varphi + \frac{A_2}{s}\cosh\varphi + \left(1-\frac{1}{v^2}\right)pH\left(\frac{x^2}{2H^2} - \frac{1}{2}\right) - \frac{s^2.p}{v^2.H}\right\} \qquad (6.29)$$

Shear distribution profile along the height of shear wall is shown in Figure6.13.

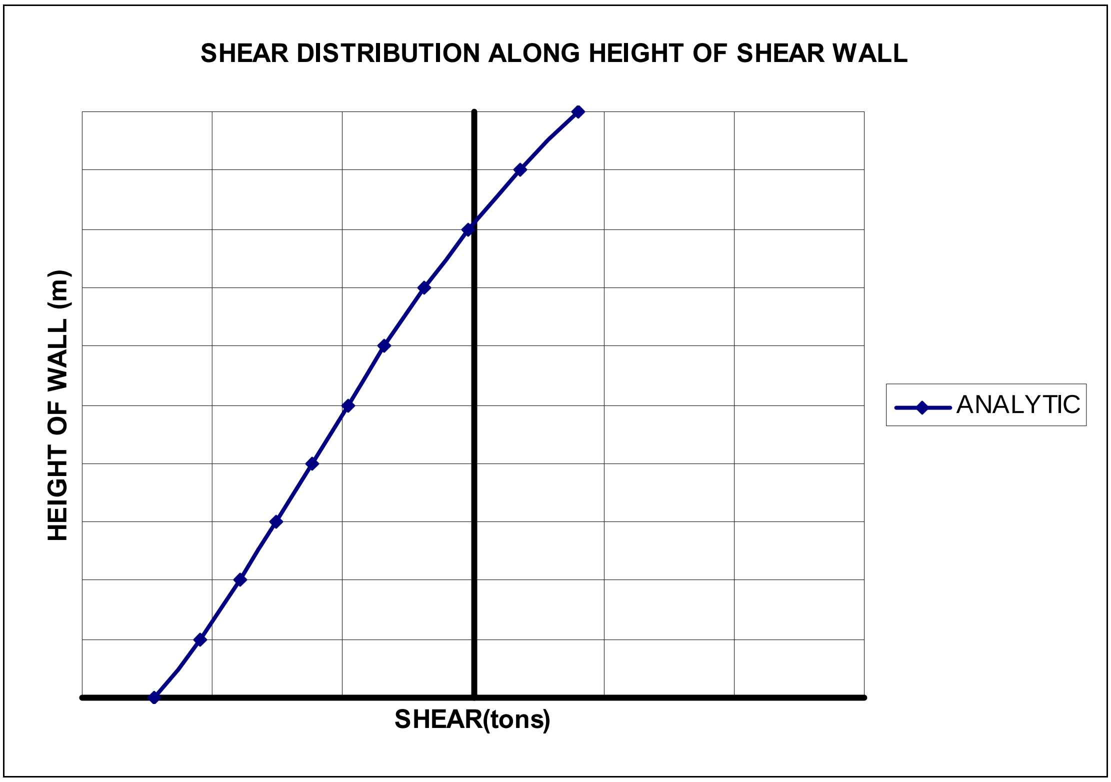


Figure 6.13 Shear Distribution Profile along Height of Shear Wall

Force equation (i.e. equation of load coming to shear wall) can then be readily obtained by differentiating the shear equation with respect to x as expressed in Eqn.6.30 and so Eqn.6.31 is attained.

$$P(x) = -V^{'}(x) = -EI \,.\, y^{IV}(x) \tag{6.30}$$

$$P(x) = -\frac{EI}{K}.\left\{\frac{A_1}{s^2}.\cosh\varphi + \frac{A_2}{s^2}.\sinh\varphi + \left(1-\frac{1}{v^2}\right).\frac{p}{H}.x\right\} \tag{6.31}$$

If the equation of load coming to shear wall obtained in Eqn.6.31 is plotted graphically, the load distribution profile on the shear wall is found to be varying as shown in Figure 6.14 and Figure 6.15.

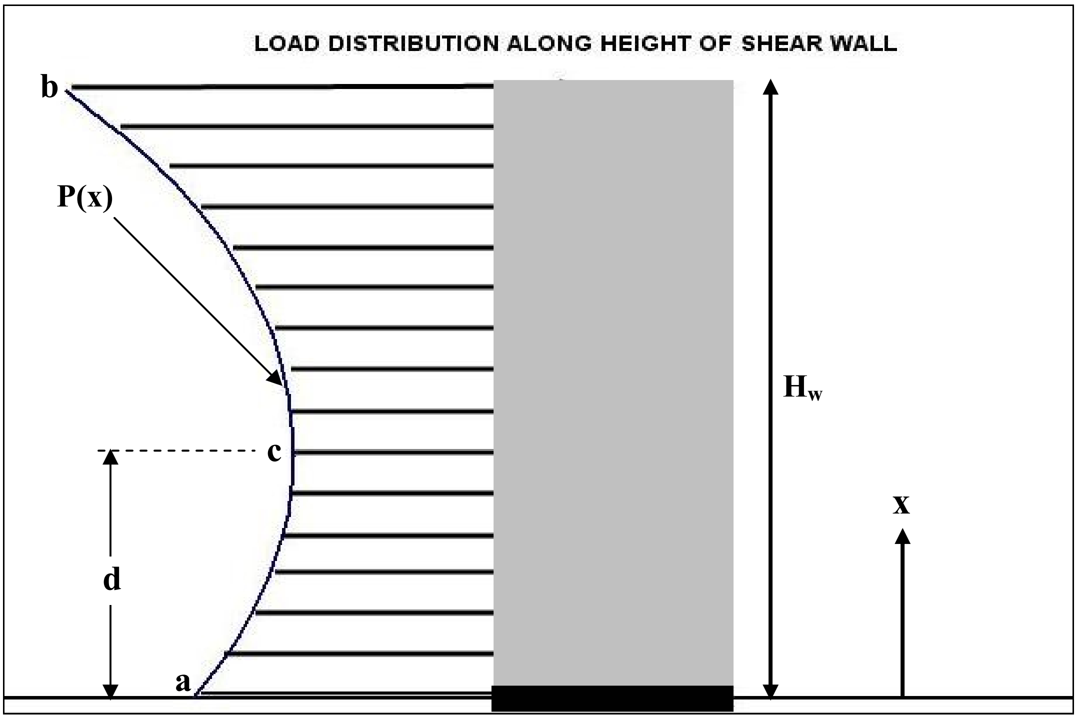


Figure 6.14 Load Distribution Profile (Type 1) along Height of Shear Wall

Letting the bottom intensity of the distributed load of P(x) as "a" and the top intensity as "b", the exact value of moment arm ($\bar{x}$) of the distributed load measured from the bottom of the shear wall shown for the case in Figure 6.14 can be expressed as in Eqn.6.32.

$$\bar{x} = \frac{3}{4}.H_w.\left(\frac{a+b}{2a+b}\right) \tag{6.32}$$

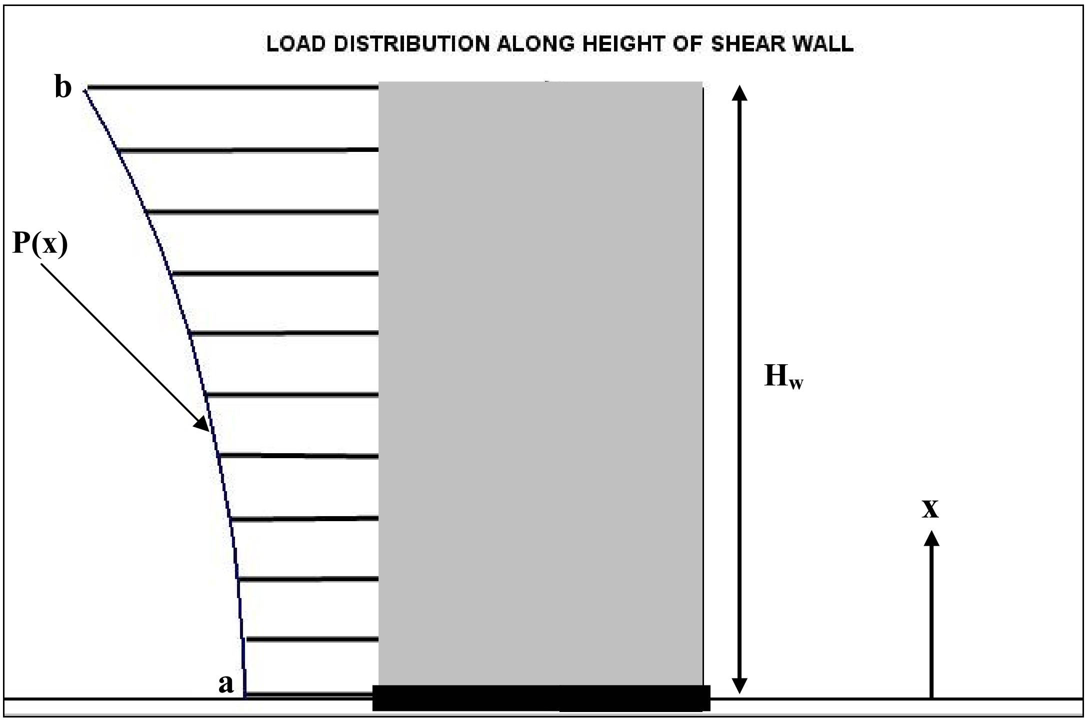


Figure 6.15 Load Distribution Profile (Type 2) along Height of Shear Wall

Letting the bottom intensity of the distributed load of P(x) as "a", the top intensity as "b", the intensity at the vertex of parabola as "c" and the height of vertex from the bottom of the shear wall as "d", the exact value of moment arm ($\bar{x}$) of the distributed load measured from the bottom of the shear wall shown for the case in Figure 6.15 can be expressed as in Eqn.6.33.

$$\bar{x} = \frac{1}{4} \cdot \frac{6cH_w^2 + (a-c)d^2 + (b-c)(H_w - d)(3H_w + d)}{3cH_w + (a-c)d + (b-c)(H_w - d)} \tag{6.33}$$

Moment, shear and loading profiles of shear walls can be obtained easily by using the executable Borland Delphi programs, shown in Figure 6.16 and Figure 6.17, written for both lateral distributed triangular load and lateral concentrated load at the top of structure. After entering the known parameters into blank boxes, the program automatically plots the distribution profiles by clicking the "Graphs" button.

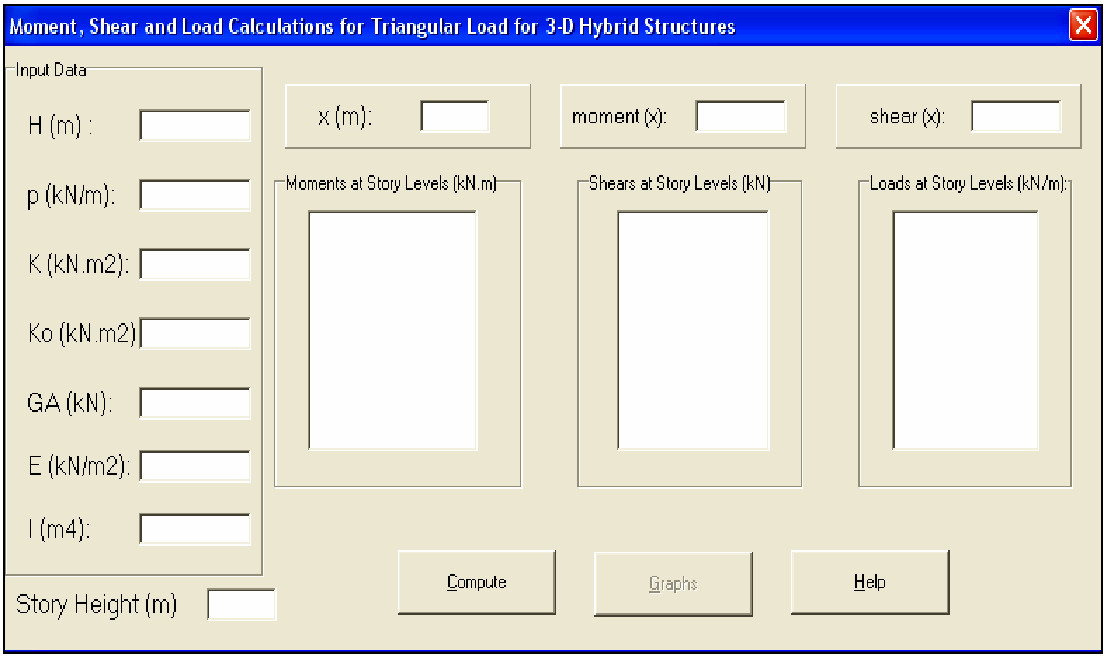


Figure 6.16 Borland Delphi Program Written for Lateral Distributed Triangular Load to Obtain Moment, Shear and Loading Profiles

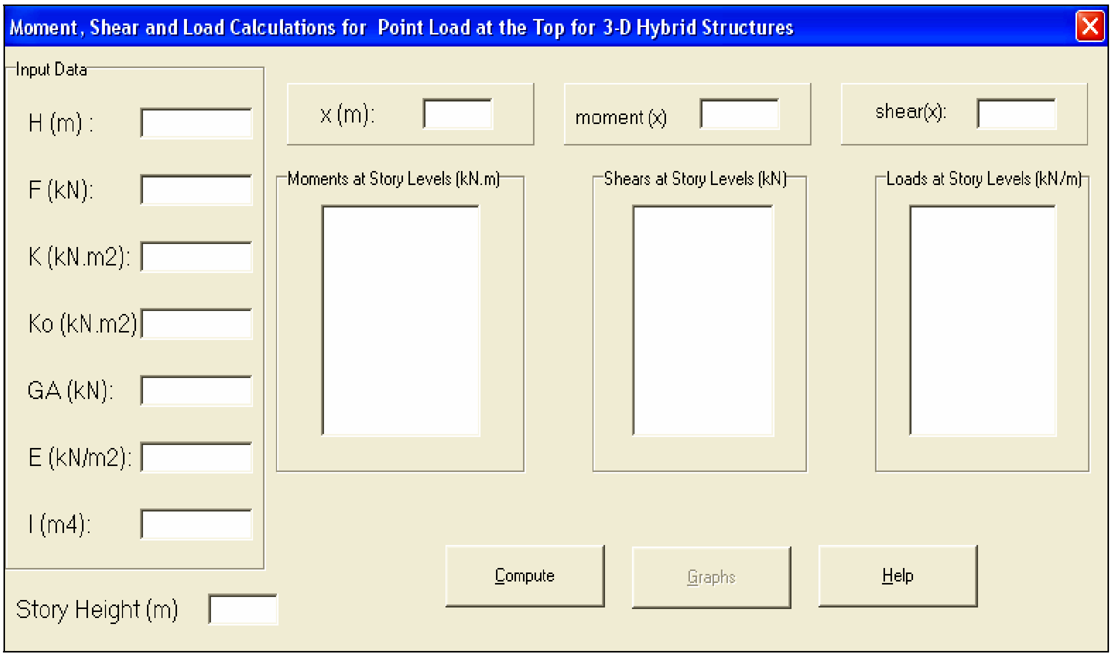


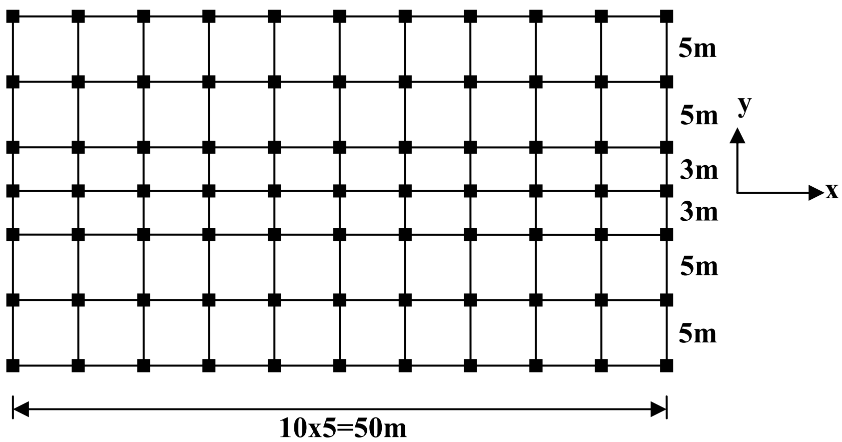
Figure 6.17 Borland Delphi Program Written for Lateral Concentrated Load at the Top of Structure to Obtain Moment, Shear and Loading Profiles

CHAPTER 7

IMPLEMENTATION OF THE PROPOSED METHOD

7.1 DESCRIPTION OF THE IMPLEMENTATION EXAMPLE

The implementation was performed on a 10 story 3-D building having a typical floor plan as shown in Figure 7.1.



All columns : 600x600 mm
All beams : 250x450 mm
Slab thickness : 120 mm
All storey heights : 3 m
g (additional) : 2.0 kN/m^2
q (additional) : 3.5 kN/m^2

Figure 7.1 Typical Floor Plan of the 3-D Building Studied

7.2 SATISFYING THE DESIGN CRITERIA FOR STRENGTH DEMAND

Total design base shear can be calculated by using Eqn.7.1 given in Turkish Earthquake Code (1997).

$$V_t = S(T).\ (A_0).\ (I).\ (W) / R \qquad (7.1)$$

where

S(T) = spectrum coefficient, which is the ratio of spectral acceleration to effective peak ground acceleration, its maximum value being 2.5

A_0 = effective ground acceleration coefficient

I = building importance factor

W = total weight of the building, as expressed by Eqn.7.2, where A_p is the area of the floor plan

R = seismic force reduction factor (structural behavior factor)

n = number of stories

$$W = \sum_{i=1}^{n} w_i A_{pi} \qquad (7.2)$$

Assuming an average value of w_i (kN/m^2), which is accepted as the same for each story and considering n-stories high, the total design base shear of Eqn.7.1 becomes as expressed in Eqn.7.3.

$$V_t = (2.5)(A_0)(I)(w_i.n.A_p) / R \qquad \text{(kN)} \qquad (7.3)$$

A lower-bound assessment of the shear strength of the total number of shear walls in one orthogonal direction of the building floor plan can be done according to Turkish Earthquake Code (1997) as expressed in Eqn.7.4.

$$V_r = \sum A_{ch}(0.65 f_{ctd} + \rho_{sh} f_{yd}) x 10^3 \qquad (7.4)$$

Considering for C20 / S420 materials, f_c'=20 MPa, f_{ctd}=1 MPa, f_y=420 MPa, f_{yd}=365 MPa and ρ_{sh}=0.0025, Eqn.7.5 is obtained.

$$V_r = 1.56 x 10^3 \sum A_{ch} \qquad \text{(kN)} \qquad (7.5)$$

Equating the total design base shear (V_t) to the total shear resistance (V_r) provided by all shear walls in one direction, the ratio of the total area of shear walls to the area of the floor plan can be obtained as expressed in Eqn.7.6.

$$\frac{\sum A_{ch}}{A_p} = 0.0016.(A_0)(I)(n.w_i) / R_w \quad (7.6)$$

Assuming typical values of A_0=0.4, I=1.0, w_i=7 kN/m^2 and R=7, the ratio in Eqn.7.6 can be calculated easily for the 10 story 3-D building studied as expressed in Eqn.7.7.

$$\frac{\sum A_{ch}}{A_p} = 0.0016.(0.4)(1.0)(10x7) / 7 = 0.0064 \quad (7.7)$$

7.3 SATISFYING THE DESIGN CRITERIA FOR STIFFNESS DEMAND

The necessary amount of stiffness of all shear walls (K_{min}) that can be treated as the stiffness of the building to satisfy stiffness criteria can be calculated easily by using Eqn.7.8. This K value was calculated by assuming l_w=3.0m and b_w=0.25m.

$$K_{min} = 37\,500. (A_0). (I). (A_p). (n.w_i) / R \quad (kN.m^2) \quad (7.8)$$

Substituting the known values of A_0=0.4, I=1.0, A_p= 50x26 =1 300 m^2, n=10, w_i=7 kN/m^2 and R=7, the total stiffness of shear walls can be found as below.

K_{min}= 37 500.(0.4).(1.0).(1 300).(10x7) / 7 = 195 000 000 kN.m^2

We have to provide the necessary amount of shear walls obtained from strength requirement so that total stiffness of shear walls must be greater than $\alpha.K_{min}$ where α is 1.2 for 10 story building obtained from Figure 7.2.

Therefore necessary amount of shear walls, whose total stiffness is greater than 1.2K = 1.2(195 000 000 kN.m^2) = 234 000 000 kN.m^2, must be provided so that drift control (i.e. stiffness criteria) is satisfied.

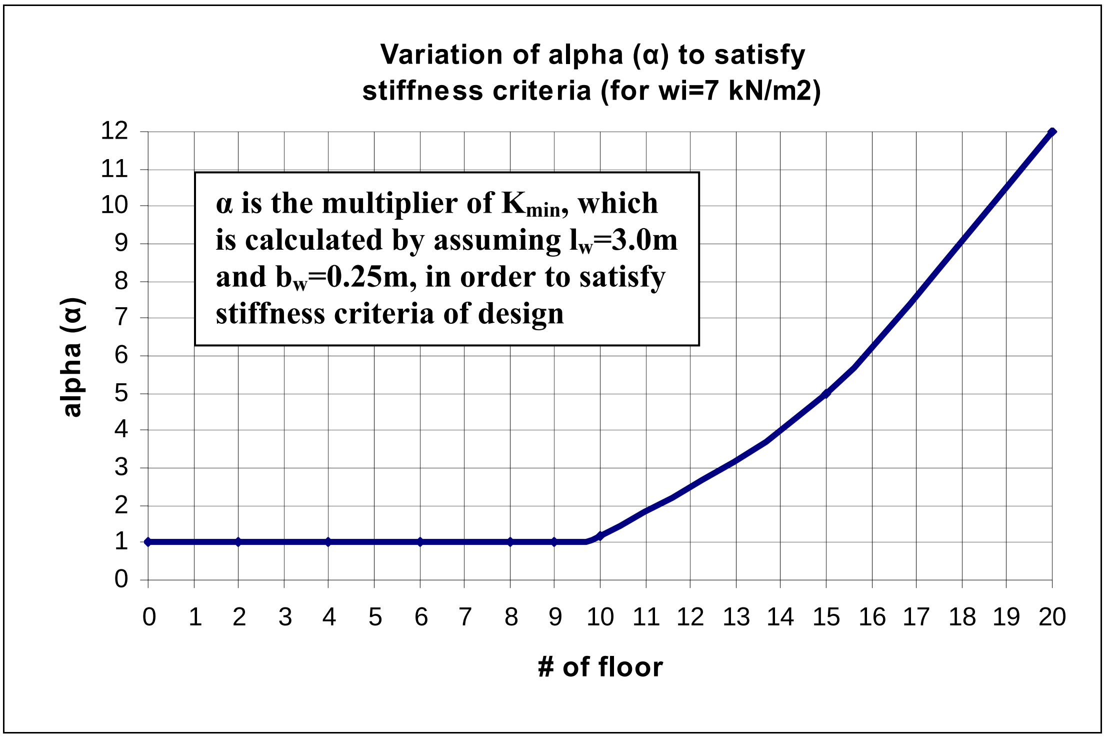


Figure 7.2 Variation of α to Satisfy the Stiffness Requirement for w_i=7 kN/m^2

Recalling the ratio of the total area of shear walls to the area of the floor plan calculated in Eqn.7.7 as expressed below.

$$\frac{\sum A_{ch}}{A_p} = 0.0064$$

where

A_p= 50x26= 1 300 m^2

Then the total area of shear walls to be used can be calculated easily as below.

ΣA_{ch}= 0.0064(1 300)= 8.32 m^2

Using b_w= 0.25m of shear walls, minimum length of shear walls to be used in each directions will be

Σl_w= 8.32 / 0.25= 33.28 m

For the 3-D building studied, shear walls can be placed on the floor plan as shown in Figure 7.3. As it can be seen that the total length of shear walls in x-direction is 40m while the total length of shear walls in y-direction is 38m.

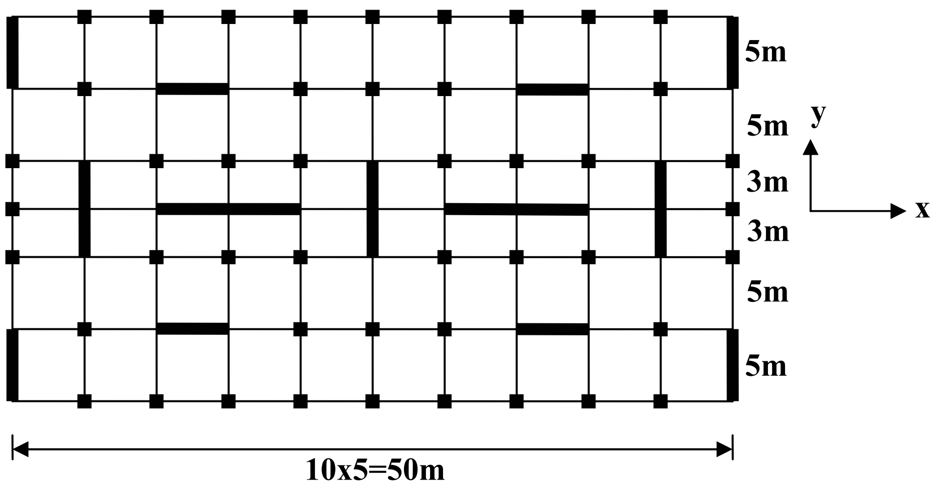


Figure 7.3 Arrangements of Shear Walls for the 3-D Building Studied

The total stiffness of all shear walls along x-direction can be calculated as below.

$$K_x = 2\,850\,000\left[\frac{1}{12}(0.25)(5)^3\,x4 + \frac{1}{12}(0.25)(10)^3\,x2\right]$$
$$= 1\,48\,4\,37\,5\,000\ \text{kN.m}^2 \qquad (\text{in x - direction})$$

The total stiffness of all shear walls along y-direction can be calculated as below.

$$K_y = 2\,850\,000\left[\frac{1}{12}(0.25)(5)^3\,x4 + \frac{1}{12}(0.25)(6)^3\,x3\right]$$
$$= 68\,1\,62\,5\,000\ \text{kN.m}^2 \qquad (\text{in y - direction})$$

As it is seen that both K_x and K_y are greater than 1.2K= 234 000 000 kN.m^2, which is the minimum required value of stiffness in order to satisfy the stiffness criteria (i.e. drift control).

i. Analysis along y-direction

$$K_y = 2\,850\,000\left[\frac{1}{12}(0.25)(5)^3\,x4 + \frac{1}{12}(0.25)(6)^3\,x3\right]$$
$$= 68\,1\,62\,5\,000\ \text{kN.m}^2 \qquad (\text{in y - direction})$$

K_0= 28 500 000.[9(0.6)(0.6)(13)^2]x2 = 31 210 920 000 kN.m^2 (due to the most exterior columns only)

K_0= 28 500 000.[4(0.25)(5)(8)^2 + 4(0.25)(5)(10.5)^2] = 24 830 625 000 kN.m^2 (due to the shear walls)

Therefore the total K_0 will be the summation of both K_0 values.

ΣK_0= 56 041 545 000 kN.m^2

The equivalent Shear Stiffness (GA) of the building along y-direction can easily be calculated by using the executable "Borland Delphi" program written. Then

GA= 10x71 490 + 16x90 090 + 2x106 750 + 18x39 150= 3 074 540 kN

Now, it is time to check whether the maximum relative story drift is less than the allowable limit mentioned in Turkish Earthquake Code (1997).

Relative story drift is calculated by the equation defined in Turkish Earthquake Code (1997) as expressed in Eqn.7.9.

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

$$\frac{\Delta_i}{h_i} = \frac{d_i - d_{i-1}}{h_i} \quad \text{(Relative Story Drift)} \qquad (7.9)$$

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.7.10 a & b.

$$(\Delta_i)_{max} / h_i \leq 0.0035 \qquad (7.10\text{ a})$$

$$(\Delta_i)_{max} / h_i \leq 0.02 / R \qquad (7.10\text{ b})$$

In the cases where the conditions specified by Eqns.7.10 a & b are not satisfied at any storey, the earthquake analysis shall be repeated by increasing the stiffness of the structural system.

Since R was taken as 7 in the example studied, the unfavorable one of the above mentioned conditions is 0.02 / R= 0.02 / 7= 0.00286

Hence the maximum story drift should not exceed 0.00286 in order to satisfy the stiffness requirement.

Analytical method can be used to find the maximum relative story drift. No need to use a computer program for this purpose. This is an advantageous feature of the analytical method proposed.

The equation of lateral displacement factored by K multiplier can be written as expressed in Eqn.7.11.

$$K.y(x) = A_1 s^2 \cosh\varphi + A_2 s^2 \sinh\varphi + (1 - \frac{1}{v^2}) pH^4 (\frac{k^2}{6} - \frac{k^3}{12} + \frac{k^5}{120}) - \frac{s^2.p.k}{6.v^2}.x^2 + A_3.x + A_4 \quad (7.11)$$

where

$$A_1 = \frac{p.s^2}{v^2.\cosh\lambda}.\left(1 + (\frac{\lambda}{2} - \frac{1}{\lambda}).\sinh\lambda\right)$$

$$A_2 = -\frac{p.s^2}{v^2}.(\frac{\lambda}{2} - \frac{1}{\lambda})$$

$$A_3 = -A_2.s$$

$$A_4 = -A_1.s^2$$

K=681 625 000 kN.m^2, K_0=56 041 545 000 kN.m^2, H=30m, p=866.7 kN/m, GA=3 074 540 kN

v^2= 1 + K / K_0 = 1 + 68 162 500 / 5 604 154 500 = 1.012163

s^2= K / (v^2.GA) = 68 162 500 / (1.012163 x 307 454) = 219.0357

Φ= x / s = x / 14.7998

k= x / H = x / 30

λ= H / s = 30 / 14.7998 = 2.027

Substituting the known values, we get A_1, A_2, A_3 and A_4 as follows.

A_1= 14 281, A_2= - 9 757, A_3= 144 397 and A_4= - 3 127 983

Since everything in Eqn.7.11 is known, we can easily calculate the lateral displacements at each story level as well as relative story drifts. Therefore we can check whether the maximum relative story drift is less than the allowable limit mentioned in Turkish Earthquake Code (1997) [1], which is 0.00286 in the studied example.

The lateral displacements and the relative story drifts can be calculated easily by using the executable "Borland Delphi" program developed, which is shown in Figure 7.4. The analytical results along y-direction, calculated by using the executable "Borland Delphi" program, are tabulated in Table 7.1.

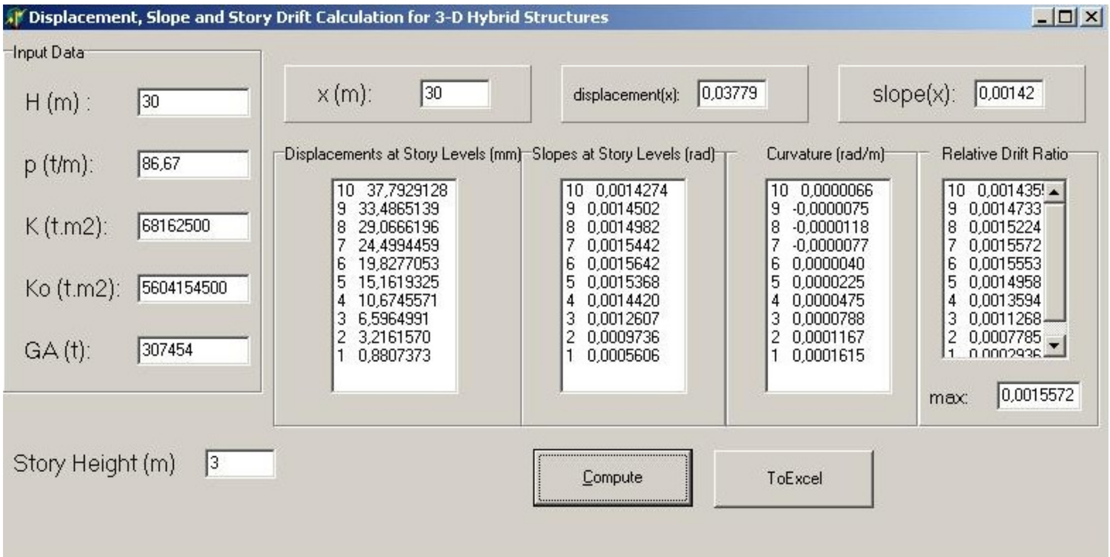


Figure 7.4 Executable "Borland Delphi" Program to Calculate Lateral Sway, Slope, Curvature and Relative Story Drift for Mixed Structures

As it is seen from Table 7.1, maximum relative story drift is 0.001560, which is less than the allowable value of 0.00286 mentioned in Turkish Earthquake Code [1]. So stiffness requirement is satisfied.

Table 7.1 Lateral Displacements and Relative Story Drifts along y-direction (Analytic Results)

# of	Displacement	Story Drift	Relative Story Drift
story	Analytic(mm)	Analytic(mm)	(Analytic)
10	37,79	4,30	0,001433
9	33,49	4,42	0,001473
8	29,07	4,57	0,001523
7	24,50	**4,68**	**0,001560**
6	19,82	4,66	0,001553
5	15,16	4,49	0,001497
4	10,67	4,07	0,001357
3	6,60	3,38	0,001127
2	3,22	2,34	0,000780
1	0,88	0,88	0,000293
		max=4,68	**max=0,001560**

If the analysis was done by using SAP2000, the lateral displacements and the relative story drifts would become as tabulated in Table 7.2.

Table 7.2 Lateral Displacements and Relative Story Drifts along y-direction (SAP2000 Results)

# of	Displacement	Story Drift	Relative Story Drift
story	SAP2000(mm)	SAP2000(mm)	(SAP2000)
10	36,10	3,76	0,001253
9	32,34	3,99	0,001330
8	28,35	4,22	0,001407
7	24,13	4,38	0,001460
6	19,75	**4,44**	**0,001480**
5	15,31	4,33	0,001443
4	10,98	4,00	0,001333
3	6,98	3,39	0,001130
2	3,59	2,47	0,000823
1	1,12	1,12	0,000373
		max=4,44	**max=0,001480**

If the analytical results are compared with SAP2000 results along y-direction, there is only around 5 % difference in top displacements and in the maximum relative story drifts, which are reasonably small and within acceptable limits.

ii. Analysis along x-direction

$$\mathrm{K_x} = 2\,850\,000\left[\frac{1}{12}(0.25)(5)^3\,\mathrm{x}4 + \frac{1}{12}(0.25)(10)^3 x2\right]$$
$$= 1\,48\,4\,37\,5\,000\ \mathrm{kN.m^2} \qquad (\text{ in x - direction })$$

K_0 = 28 500 000.[3(0.6)(0.6)(25)^2]x2 = 38 475 000 000 kN.m^2 (due to the most exterior columns only)

K_0 = 28 500 000.[4(0.25)(5)(25)^2 + 2(0.25)(6)(20)^2 + 4(0.25)(5)(12.5)^2 + 2(0.25)(10)(10)^2] = 159 778 125 000 kN.m^2 (due to the shear walls)

Therefore the total K_0 will be the summation of both K_0 values.

ΣK_0 = 198 253 125 000 kN.m^2

The equivalent Shear Stiffness (GA) of the building along x-direction can easily be calculated by using the executable "Borland Delphi" program written. Then

GA= 40x71 490 + 6x39 150= 3 094 500 kN

Analytical method can be used to find the maximum relative story drift. No need to use a computer program for analysis. This is an advantageous feature of the analytical method proposed.

Recalling the equation of lateral displacement factored by K multiplier, as expressed in Eqn.7.12.

$$K.y(x) = A_1 s^2 \cosh\phi + A_2 s^2 \sinh\phi + (1-\frac{1}{v^2})pH^4(\frac{k^2}{6}-\frac{k^3}{12}+\frac{k^5}{120}) - \frac{s^2.p.k}{6.v^2}.x^2 + A_3.x + A_4 \qquad (7.12)$$

where

$$A_1 = \frac{p.s^2}{v^2.\cosh\lambda}.\left(1+(\frac{\lambda}{2}-\frac{1}{\lambda}).\sinh\lambda\right)$$

$$A_2 = -\frac{p.s^2}{v^2}.(\frac{\lambda}{2} - \frac{1}{\lambda})$$

$$A_3 = -A_2.s$$

$$A_4 = -A_1.s^2$$

K=1 484 375 000 kN.m^2, K_0=198 253 125 000 kN.m^2, H=30m, p=866.7 kN/m, GA=3 094 500 kN

v^2= 1 + K / K_0 = 1 + 148 437 500 / 19 825 312 500 = 1.007487

s^2= K / (v^2.GA) = 148 437 500 / (1.007487 x 309 450) = 476.1169

Φ= x / s = x / 21.8201

k= x / H = x / 30

λ= H / s = 30 / 21.8201= 1.375

Substituting the known values, we get A_1, A_2, A_3 and A_4 as follows.

A_1= 18 032, A_2= 1 634, A_3= - 35 657 and A_4= - 8 585 181

Since everything in Eqn.6.34 is known, the lateral displacements at each story level as well as relative story drifts can be calculated. Therefore we can check whether the maximum relative story drift is less than the allowable limit mentioned in Turkish Earthquake Code (1997), which is 0.00286 in the studied example.

The lateral displacements and the relative story drifts can be calculated easily by using the executable "Borland Delphi" program developed, which is shown in Figure 7.5. The analytical results along x-direction calculated by using the executable "Borland Delphi" program are tabulated in Table 7.3.

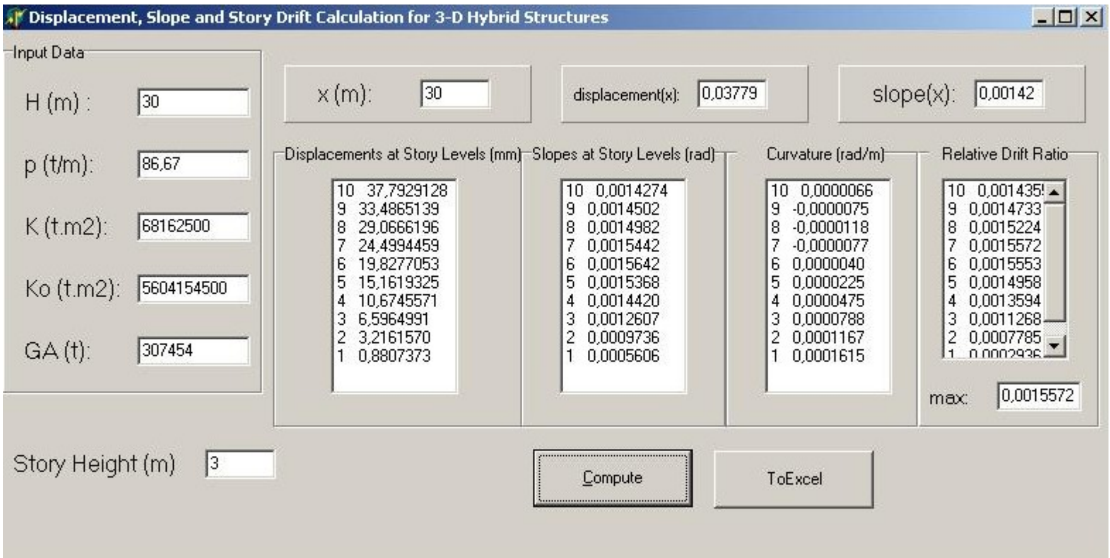


Figure 7.5 Executable "Borland Delphi" Program to Calculate Lateral Sway, Slope, Curvature and Relative Story Drift for Mixed Structures

As it is seen from Table 7.3, maximum relative story drift is 0.001070 that is less than the allowable value of 0.00286 mentioned in Turkish Earthquake Code [1]. So stiffness requirement is satisfied.

Table 7.3 Lateral Displacements and Relative Story Drifts along x-direction (Analytic Results)

# of story	Displacement Analytic(mm)	Story Drift Analytic(mm)	Relative Story Drift (Analytic)
10	25,26	3,14	0,001047
9	22,12	3,18	0,001060
8	18,94	**3,21**	**0,001070**
7	15,73	3,19	0,001063
6	12,54	3,10	0,001033
5	9,44	2,90	0,000967
4	6,54	2,56	0,000853
3	3,98	2,07	0,000690
2	1,91	1,40	0,000467
1	0,51	0,51	0,000170
		max=3,21	**max=0,001070**

If the analysis was done by using SAP2000, the lateral displacements and the relative story drifts would become as tabulated in Table 7.4.

Table 7.4 Lateral Displacements and Relative Story Drifts along x-direction (SAP2000 Results)

# of	**Displacement**	**Story Drift**	**Relative Story Drift**
story	**SAP2000(mm)**	**SAP2000(mm)**	**(SAP2000)**
10	21,32	2,18	0,000727
9	19,14	2,34	0,000780
8	16,80	2,47	0,000823
7	14,33	2,57	0,000857
6	11,76	**2,59**	**0,000863**
5	9,17	2,53	0,000843
4	6,64	2,35	0,000783
3	4,29	2,01	0,000670
2	2,28	1,52	0,000507
1	0,76	0,76	0,000253
		max=2,59	**max=0,000863**

If the analytical results are compared with SAP2000 results along x-direction, there is 18 % difference in top displacements and 24 % difference in maximum relative story drifts, which are within the acceptable limits.

7.4 SATISFYING THE DESIGN CRITERIA FOR DUCTILITY DEMAND

Total overturning moment (M_{ot}) coming to the building in both x and y directions can be calculated easily as expressed in Eqn7.13.

$$M_{ot} = V_t.(2H / 3) \tag{7.13}$$

where

V_t= total design base shear

H= height of the building

Total design base shear can be calculated by using the following equation given in Turkish Earthquake Code (1997).

$V_t = S(T). (A_0). (I). (w_i.n.A_p) / R$

$V_t = (2.5)(0.4)(1.0)(7x10x1300) / 7 = 13\ 000$ kN

Substituting V_t = 13 000 kN and H = 30m into Eqn.7.13, total overturning moment coming to the building in both x and y directions will be

$M_{ot} = V_t.(2H / 3) = 13\ 000.(2x30 / 3) = 260\ 000$ kN.m

Each shear wall will take some portion of this moment with respect to its moment of inertia multiplied by the curvature value at the bottom of structure.

The curvature diagram can be obtained easily by using the executable "Borland Delphi" program shown in Figure 7.6.

The curvature distribution along the height of the structure, obtained from the analytical expression given in Eqn.7.14, for x direction is shown in Figure 7.7.

The curvature distribution along the height of the structure, obtained from the analytical expression given in Eqn.7.14, for y direction is shown in Figure 7.8.

$$K.y^{II}(x) = A_1.\cosh\varphi + A_2.\sinh\varphi + (1 - \frac{1}{v^2})pH^4.\left(\frac{1}{3.H^2} - \frac{x}{2.H^3} + \frac{x^3}{6.H^5}\right) - \frac{s^2.p}{v^2.H}.x \qquad (7.14)$$

If the curvature value at the bottom of structure (i.e. at x = 0) is multiplied by EI of any shear wall, the moment coming to that particular shear wall can be found easily. Here, E is the modulus of elasticity of concrete and I is the moment of inertia of that particular shear wall.

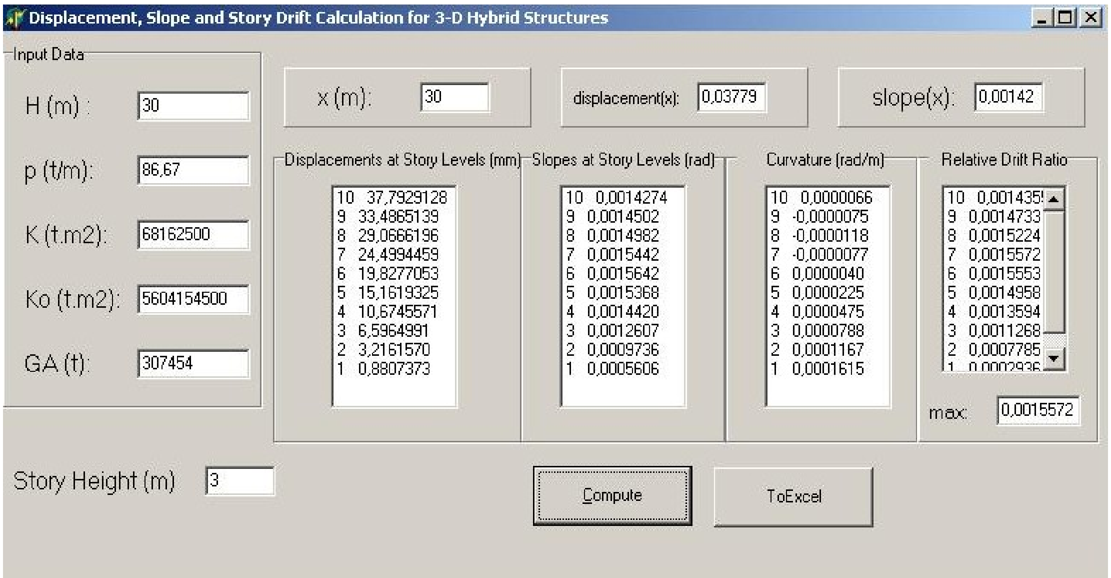


Figure 7.6 Executable "Borland Delphi" Program to Plot the Graph of Curvature Distribution along Height of Structure

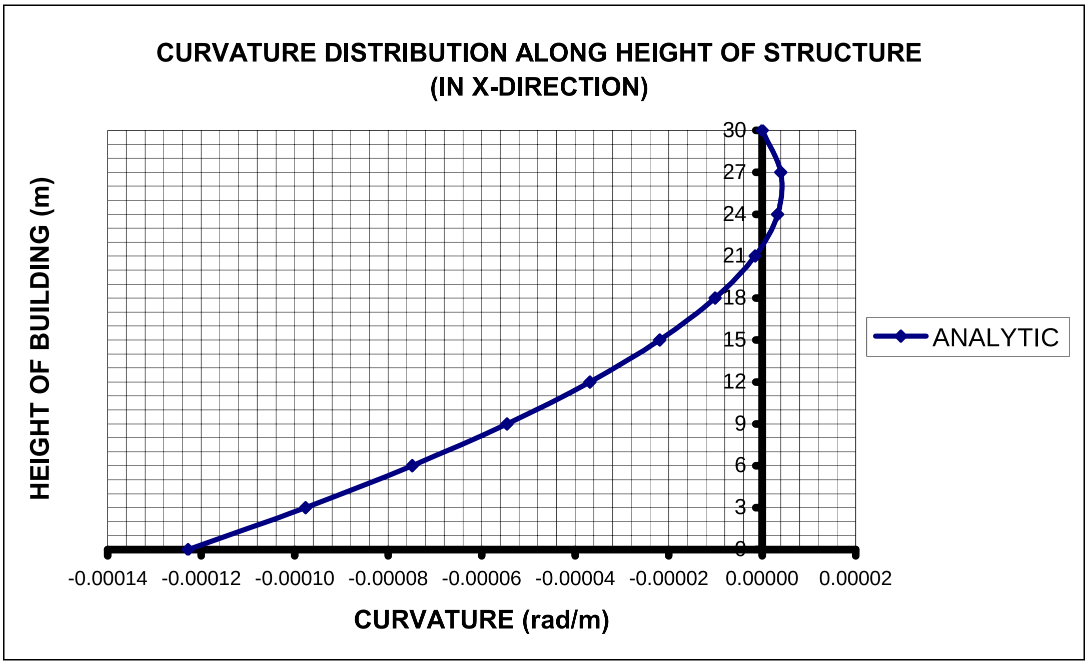


Figure 7.7 Curvature Distributions along Height of Structure in x-direction

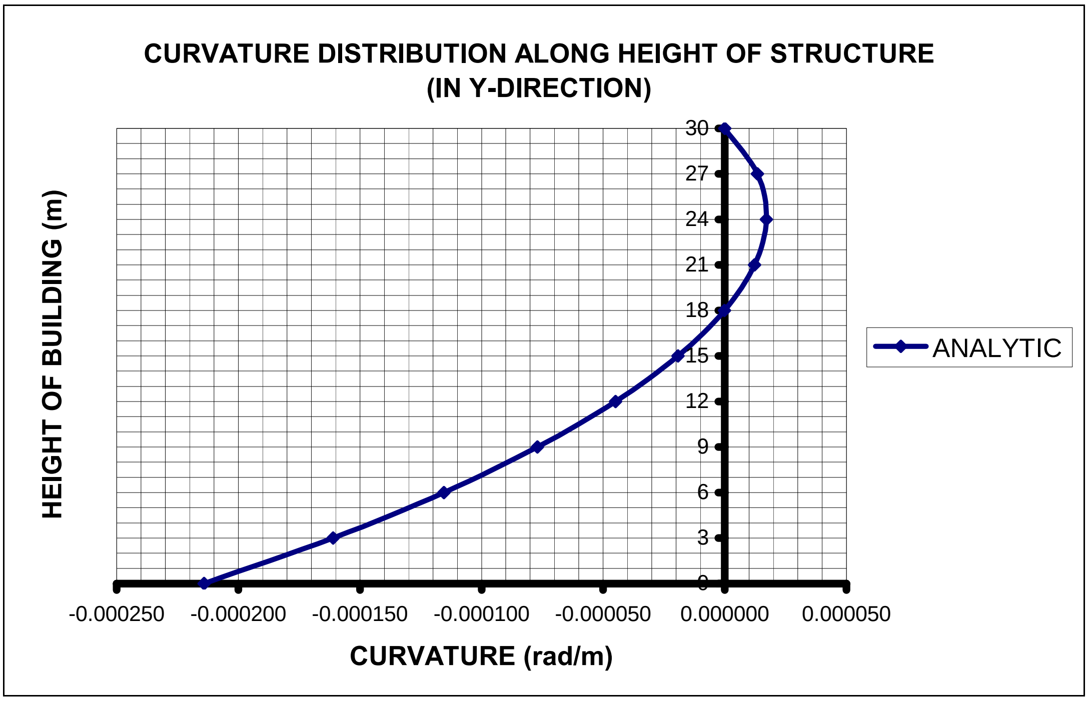


Figure 7.8 Curvature Distributions along Height of Structure in y-direction

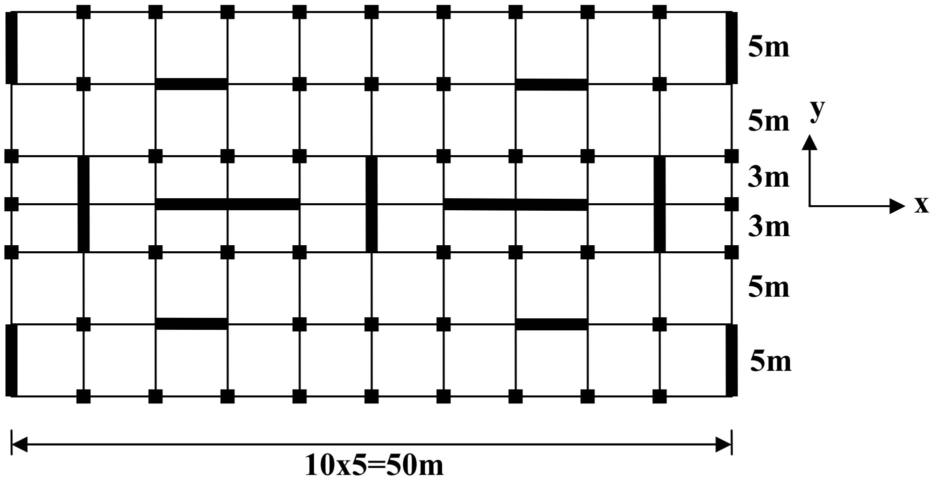


Figure 7.9 Arrangements of Shear Walls for the 3-D Building Studied

i. Analysis along y-direction

The moment of inertia of l_w=5m of shear walls in y-direction is

$$I = \frac{1}{12}(0.25)(5)^3 = 2.6\,m^4$$

The moment of inertia of l_w=6m of shear walls in y-direction is

$$I = \frac{1}{12}(0.25)(6)^3 = 4.5\,m^4$$

Since the curvature value at the bottom of structure (i.e. at x = 0) in y direction is 0.000214 that is obtained from Figure 7.8, then the maximum moment taken by each shear wall in y-direction will be as follows.

M (l_w=5m of shear wall) = EI.y''(x)

= (28 500 000 kN/m^2).(2.6 m^4).(0.000214 rad/m)

= 15 860 kN.m (Analytic Result)

ETABS gives 15 810 kN.m moment for the same shear wall (l_w=5m of shear wall), which is almost the same as obtained by analytical method.

M (l_w=6m of shear wall) = EI.y''(x)

= (28 500 000 kN/m^2).(4.5 m^4).(0.000214 rad/m)

= 27 450 kN.m (Analytic Result)

ETABS gives 25 680 kN.m moment for the same shear wall (l_w=6m of shear wall), which is very close to the analytical result.

The total moment taken by all shear walls in y-direction will then be

Σ M = 4 x 15 860 + 3 x 27 450 = 145 790 kN.m (Analytic Result)

ETABS gives 140 280 kN.m moment taken by all shear walls in y-direction, which is almost the same as obtained by analytical method.

Therefore the unbalanced moment that will be taken by the coupled axial forces of shear walls and columns can be calculated easily as below.

$M_{unbalanced} = M_{ot} - \Sigma M$ = 260 000 kN.m –145 790 kN.m

= 114 210 kN.m (Analytic Result)

ETABS gives 119 720 kN.m unbalanced moment that will be taken by the coupled axial forces of shear walls and columns in y-direction, which is almost the same as obtained by analytic method.

The axial force due to this unbalanced moment for each shear wall can be calculated by the following equation.

$\Delta N = \pm M.c / I$

where

M = Unbalanced Moment

c = Distance from the center of gravity of shear wall to that of the structure

I = Total $A.d^2$ terms due to shear walls and columns

First of all, I is calculated in y-direction by assuming A=1 for l_w=5m of shear wall, hence A=0.288 for columns.

$$I = 4 \text{ x } (1)(10.5)^2 + 4 \text{ x } (1)(8)^2 + 18 \text{ x } (0.288)(13)^2 + 10 \text{ x } (0.288)(8)^2$$
$$+ 16 \text{ x } (0.288)(3)^2 = 1\ 799 \text{ m}^4$$

Then the axial force of l_w=5m of shear wall in y-direction due to unbalanced moment can be calculated easily as below.

$\Delta N = \pm M.c / I = \pm 114\ 210.(10.5) / 1\ 799 = \pm 670$ kN (Analytic Result)

The axial force of l_w=6m of shear wall in y-direction due to unbalanced moment will be zero since c = 0 for that shear wall in y-direction.

ii. Analysis along x-direction

The moment of inertia of l_w=5m of shear walls in x-direction is

$$I = \frac{1}{12}(0.25)(5)^3 = 2.6 \text{ m}^4$$

The moment of inertia of l_w=10m of shear walls in x-direction is

$$I = \frac{1}{12}(0.25)(10)^3 = 20.8 \text{ m}^4$$

Since the curvature value at the bottom of structure (i.e. at x = 0) in x direction is 0.0001228 that is obtained from Figure 7.7, then the maximum moment taken by each shear wall in x-direction will be as follows.

M (l_w=5m of shear wall) = EI.y''(x)

= (28 500 000 kN/m^2).(2.6 m^4).(0.0001228 rad/m)

= 9 100 kN.m (Analytic Result)

ETABS gives 10 330 kN.m moment for the same shear wall (l_w=5m of shear wall), which is very close to the analytical result.

M (l_w=10m of shear wall) = EI.y''(x)

= (28 500 000 kN/m^2).(20.8 m^4).(0.0001228 rad/m)

= 72 790 kN.m (Analytic Result)

ETABS gives 59 030 kN.m moment for the same shear wall (l_w=10m of shear wall), which is reasonably close to the analytical result.

The total moment taken by all shear walls in x-direction will then be

Σ M = 4 x 9 100 + 2 x 72 190 = 180 780 kN.m (Analytic Result)

ETABS gives 159 380 kN.m moment taken by all shear walls in x-direction, which is reasonably close to the analytical result.

Therefore the unbalanced moment that will be taken by the coupled axial forces of shear walls and columns can be calculated easily as below.

$M_{unbalanced} = M_{ot} - \Sigma M$ = 260 000 kN.m – 180 780 kN.m

= 79 220 kN.m (Analytic Result)

ETABS gives 100 620 kN.m unbalanced moment that will be taken by the coupled axial forces of shear walls and columns in x-direction, which is reasonably close to the analytical result.

The axial force due to this unbalanced moment for each shear wall can be calculated by the following equation.

$\Delta N = \pm M.c / I$

where

M = Unbalanced Moment

c = Distance from the center of gravity of shear wall to that of the structure

I = Total $A.d^2$ terms due to shear walls and columns

First of all, I is calculated in x-direction by assuming A=1 for l_w=5m of shear wall, hence A=0.288 for columns.

$I = 4x(1)(25)^2 + 2x(1.2)(20)^2 + 4x(1)(12.5)^2 + 2x(2)(10)^2 + 6x(0.288)(25)^2$

$+ 8x(0.288)(20)^2 + 8x(0.288)(15)^2 + 8x(0.288)(10)^2 = 7\ 322\ m^4$

Then the axial force of l_w=5m of shear wall in x-direction due to unbalanced moment can be calculated easily as below.

$\Delta N = \pm M.c / I = \pm 79\ 220.(12.5) / 7\ 322 = \pm 140$ kN (Analytic Result)

The axial force of l_w=10m of shear wall in x-direction due to unbalanced moment will be

$\Delta N = \pm M.c / I = \pm 79\ 220.(10) / 7\ 322 = \pm 110$ kN (Analytic Result)

i. Designing ductile shear wall in y-direction

In this implementation example, the l_w=6m of shear wall in y-direction will be designed in order to show that how shear walls can de designed for ductility demand.

The design moment coming to this shear wall, M_d, was found as 27 450 kN.m by using the analytic method.

The design axial load coming to this shear wall will be due to only the vertical loads (i.e. dead loads, live loads and additional dead loads) since the axial load due to unbalanced earthquake moment coming to this shear wall is zero because of the distance from the center of gravity of shear wall to that of the structure being zero.

The design axial load coming to this shear wall due to the vertical loads can be easily calculated by considering the tributary area of shear wall as below.

Due to slab: (11x5 m^2)(8.5 kN/m^2)(10 stories) = 4 680 kN

Due to beams: (22 m)(25 kN/m^3)(0.25x0.45 m^2)(10 stories) = 620 kN

Due to own weight of shear wall: (0.25x6 m^2)(25 kN/m^3)(30 m) = 1 130 kN

Then the total design axial force, N_d, is found as 6 430 kN by simple hand calculation considering the tributary area of shear wall. ETABS gives 6 270 kN of axial force for this shear wall, which is very close to the value found by hand calculation.

Knowing the design axial load, the moment curvature diagram shown in Figure 7.11 can be easily obtained by using RESPONSE 2000 program. The reinforcement provided is twice the minimum values given in Turkish Earthquake Code, being

0.005 for web reinforcement and 0.004 for additional end zone reinforcement. The RESPONSE 2000 input values are also shown in Figure 7.10.

The axial load ratio can also be calculated as below.

$$\frac{N}{N_0} = \frac{643}{25x600(0.85x0.13 + 0.005x3.65)} = 0.33$$

The maximum resisting moment, M_r, of the shear wall is obtained as 29 150 kN.m, which is greater than the design moment, M_d, 27 450 kN.m. Hence the strength requirement of design is also satisfied.

On the other hand, the yield curvature (Φ_y) and the ultimate curvature (Φ_u) values are obtained as 0.6 rad/km and 6.783 rad/km, respectively. Hence the available curvature ductility, μ_Φ, can be calculated easily as below.

$\mu_\Phi = \Phi_u / \Phi_y = 6.783 / 0.6 = 11.3$ (available)

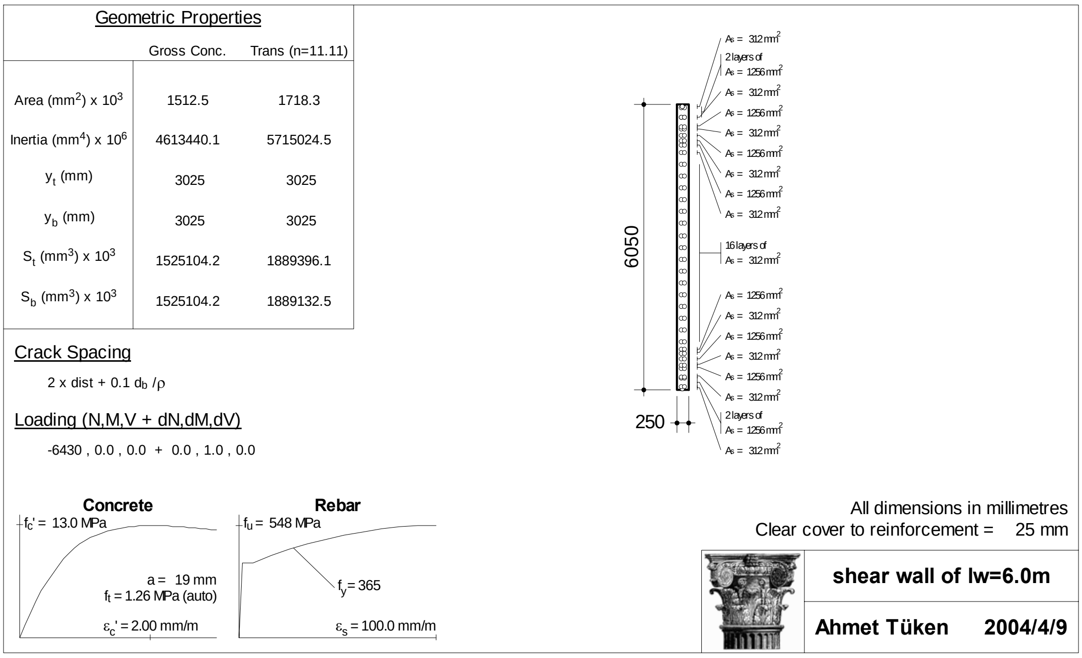


Figure 7.10 RESPONSE 2000 Input Values for the l_w=6m of Shear Wall Designed in y-direction

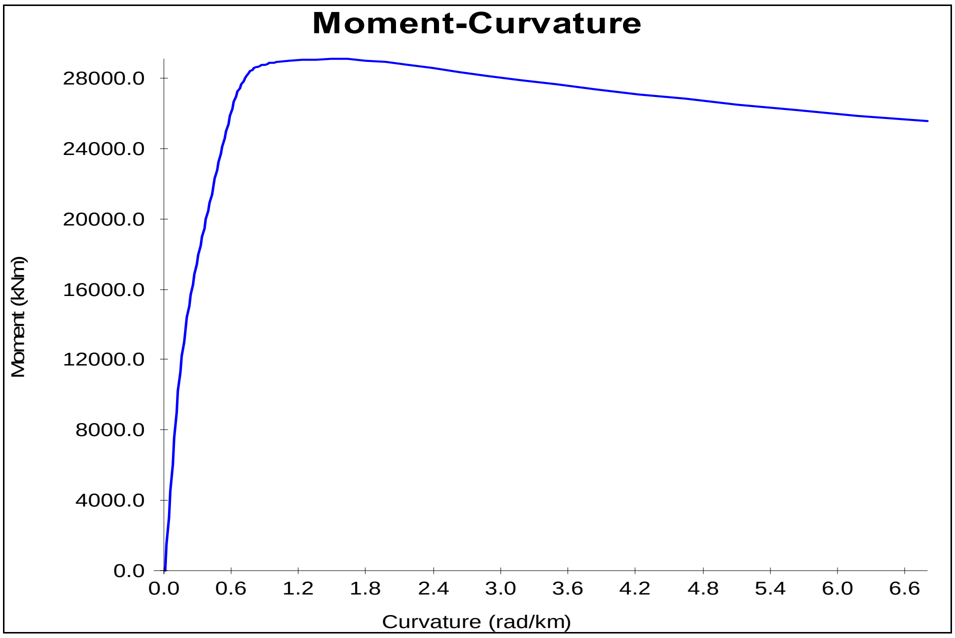


Figure 7.11 Moment-Curvature Diagram for the l_w=6m of Shear Wall Designed in y-direction

The equation relating the curvature ductility to displacement ductility derived for the case shown in Figure 7.12 is given in Eqn.7.15.

$$\mu_\varphi = \frac{\varphi_u}{\varphi_y} = 1 + \frac{(\mu_\Delta - 1)\left[5(\varphi_y)(H_w - h)(3H_w + h) - k(5a + 4b)^2\right]}{60(l_p)(H_w - 0.5l_p)(\varphi_y)} \qquad (7.15)$$

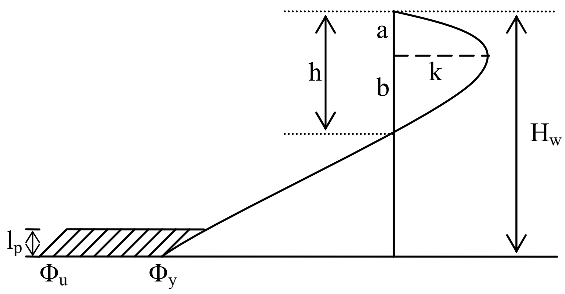


Figure 7.12 Relating Top Sway of Building to Cross-Sectional Curvature of Wall

In case of a = b = h / 2, displacement ductility (μ_Δ) and curvature ductility (μ_Φ) can be related as in Eqn.7.16.

$$\mu_\varphi = 1 + \frac{(\mu_\Delta - 1)\left[(\varphi_y)(H_w - h)(3H_w + h) - 4kh^2\right]}{12(l_p)(H_w - 0.5l_p)(\varphi_y)} \tag{7.16}$$

Substituting the known values of Φ_y=0.0006 rad/m, μ_Δ=5, H_w=30m, h=12m, k=0.0000171 rad/m and l_p= $0.2l_w+0.044H_w$=0.2(6m)+0.044(30m)=2.52m, required curvature ductility can be calculated easily as below.

$$\mu_\varphi = 1 + \frac{(5-1)\left[(0.0006)(30-12)(3x30+12) - 4x0.0000171x12^2\right]}{12(2.52)(30-0.5x2.52)(0.0006)}$$

$$\mu_\varphi = 9.4 \text{ (Required)}$$

Since $(\mu_\Phi)_{available}$ = 11.3 > $(\mu_\Phi)_{required}$ = 9.4, the ductility requirement of design is also satisfied.

Finally, it should be noted that the yield curvature to be used in Eqn.7.15 and Eqn.7.16 can also be found easily by the following expression given in the paper by Pauley, T [72] for rectangular shear walls.

$\Phi_y = (\lambda.\varepsilon_y) / l_w$

where

λ= Constant quantifying the influence of Φ_y / Φ_y' and the depth of neutral axis, $k.l_w$, at the onset of yielding, which can be taken as 2 for design purposes

ε_y= Yield strain of reinforcing steel, which is 0.00183 for S420

l_w= Length of shear wall

Φ_y= Reference yield curvature, relevant to the idealized bilinear section response

Φ_y'= Yield curvature at the stage where nonlinearity begins at the onset of yielding of the bars at the extreme tension fiber

Φ_y = (2 x 0.00183) / 6 = 0.00061 rad/m = 0.61 rad/km

It is seen that the value of 0.61 rad/km found by the expression proposed by Pauley is almost the same as the value of 0.6 rad/km read from Moment-Curvature diagram obtained by RESPONSE 2000.

ii. Designing ductile shear wall in x-direction

In this implementation example, the l_w=10m of shear wall in x-direction will be designed in order to show that how shear walls can de designed for ductility demand.

The design moment coming to this shear wall, M_d, was found as 72 790 kN.m by using the analytic method.

The design axial load coming to this shear wall will be due to both vertical loads (i.e. dead loads, live loads and additional dead loads) and unbalanced earthquake moment coming to this shear wall.

The design axial load coming to this shear wall due to vertical loads can be easily calculated by considering the tributary area of shear wall as below.

Due to slab: (15x3 m^2)(8.5 kN/m^2)(10 stories) = 3 830 kN

Due to beams: (14 m)(25 kN/m^3)(0.25x0.45 m^2)(10 stories) = 390 kN

Due to own weight of shear wall: (0.25x10 m^2)(25 kN/m^3)(30 m) = 1 880 kN

Then the design axial force, N_d, due to vertical loads is found as 6 100 kN by simple hand calculation considering the tributary area of shear wall. ETABS gives 7 130 kN of axial force for this shear wall, which is reasonably close to the value found by hand calculation.

Total design axial force is calculated by subtracting the axial force due to unbalanced earthquake moment (110 kN) from the axial force due to vertical loads (610 tons). Hence,

$(N_d)_{total}$ = 6 100 – 110 = 5 990 kN (Analytic Result)

Knowing the design axial load, the moment curvature diagram shown in Figure 7.14 can be easily obtained by using RESPONSE 2000 program. The reinforcement provided is twice the minimum values given in Turkish Earthquake Code, being 0.005 for web reinforcement and 0.004 for additional end zone reinforcement. The RESPONSE 2000 input values are also shown in Figure 7.13.

The axial load ratio can also be calculated as below.

$$\frac{N}{N_0} = \frac{599}{25x1000(0.85x0.13 + 0.005x3.65)} = 0.19$$

The maximum resisting moment, M_r, of the shear wall is obtained as 72 890 kN.m, which is greater than the design moment, M_d, 72 790 kN.m. Hence the strength requirement of design is also satisfied.

On the other hand, the yield curvature (Φ_y) and the ultimate curvature (Φ_u) values are obtained as 0.4 rad/km and 5.588 rad/km, respectively. Hence the available curvature ductility, μ_Φ, can be calculated easily as below.

$\mu_\Phi = \Phi_u / \Phi_y = 5.588 / 0.4 = 13.97$ (available)

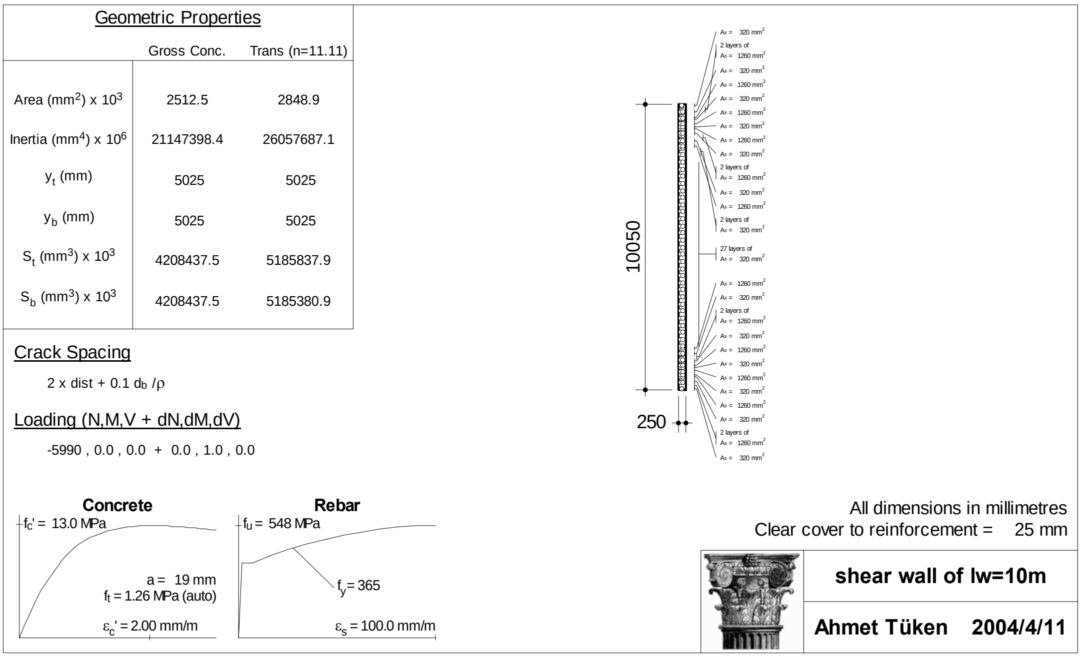


Figure 7.13 RESPONSE 2000 Input Values for the l_w=10m of Shear Wall Designed in x-direction

The equation relating the curvature ductility to displacement ductility derived for the case shown in Figure 7.15 is given in Eqn.7.17.

$$\mu_\varphi = \frac{\varphi_u}{\varphi_y} = 1 + \frac{(\mu_\Delta - 1)\left[5(\varphi_y)(H_w - h)(3H_w + h) - k(5a + 4b)^2\right]}{60(l_p)(H_w - 0.5l_p)(\varphi_y)} \tag{7.17}$$

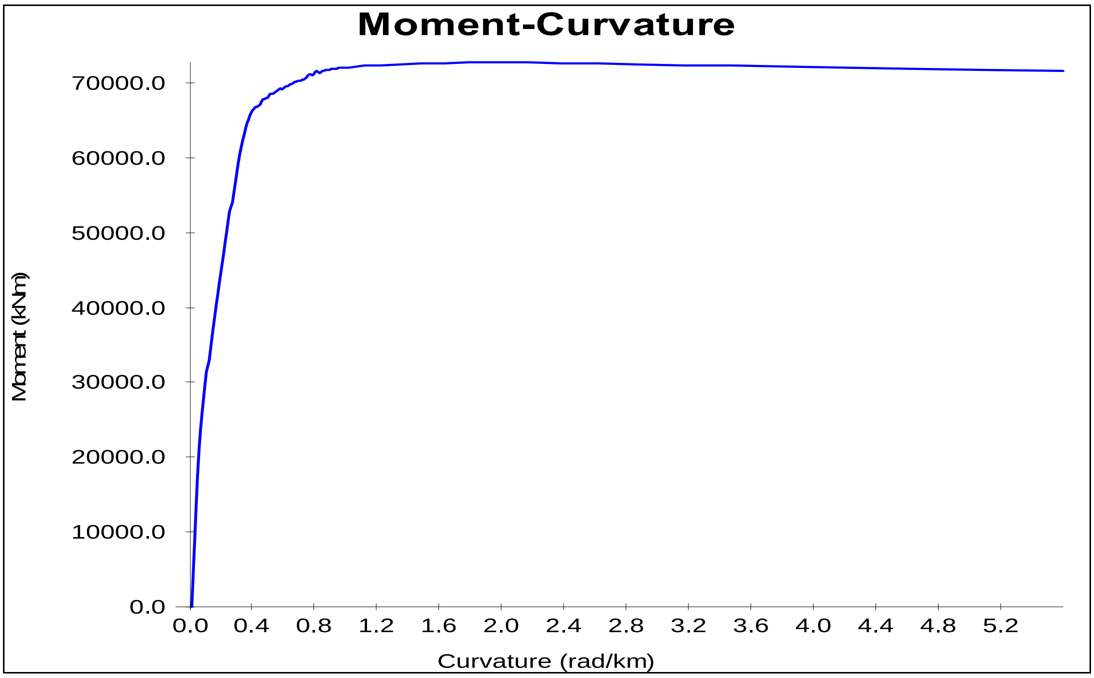


Figure 7.14 Moment-Curvature Diagram for the l_w=10m of Shear Wall Designed in x-direction

In case of a = b = h / 2, displacement ductility (μ_Δ) and curvature ductility (μ_Φ) can be related as in Eqn.7.18.

$$\mu_\varphi = 1 + \frac{(\mu_\Delta - 1)\left[(\varphi_y)(H_w - h)(3H_w + h) - 4kh^2\right]}{12(l_p)(H_w - 0.5l_p)(\varphi_y)} \tag{7.18}$$

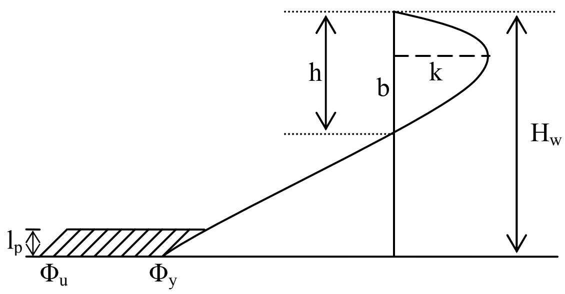


Figure 7.15 Relating Top Sway of Building to Cross-Sectional Curvature of Wall

Substituting the known values of Φ_y=0.0004 rad/m, μ_Δ=5, H_w=30m, h=9m, k=0.000004 rad/m and l_p= $0.2l_w+0.044H_w$=0.2(10m)+0.044(30m)=3.32m, required curvature ductility can be calculated easily as below.

$$\mu_\varphi = 1 + \frac{(5-1)\left[(0.0004)(30-9)(3x30+9) - 4x0.000004x9^2\right]}{12(3.32)(30-0.5x3.32)(0.0004)} = 8.4$$

$$\mu_\varphi = 8.4 \text{ (Required)}$$

Since $(\mu_\Phi)_{available}$ = 13.97 > $(\mu_\Phi)_{required}$ = 8.4, the ductility requirement of design is also satisfied.

Finally, it should be noted that the yield curvature to be used in Eqn.7.17 and Eqn.7.18 can also be found easily by the following expression given in the paper by Pauley, T [72] for rectangular shear walls.

$\Phi_y = (\lambda.\varepsilon_y) / l_w$

where

λ= Constant quantifying the influence of Φ_y / Φ_y' and the depth of neutral axis, $k.l_w$, at the onset of yielding, which can be taken as 2 for design purposes

ε_y= Yield strain of reinforcing steel, which is 0.00183 for S420

l_w= Length of shear wall

Φ_y= Reference yield curvature, relevant to the idealized bilinear section response

Φ_y'= Yield curvature at the stage where nonlinearity begins at the onset of yielding of the bars at the extreme tension fiber

Φ_y = (2 x 0.00183) / 10 = 0.00037 rad/m = 0.37 rad/km

It is seen that the value of 0.37 rad/km found by the expression proposed by Pauley is very close to the value of 0.4 rad/km read from Moment-Curvature diagram obtained by RESPONSE 2000.

7.5 PREVENTING THE OCCURRENCE OF SHEAR FAILURE BY USING THE OVER STRENGTH VALUES OF MATERIALS (CAPACITY DESIGN METHOD)

By using the over strength values for C20 and S420 materials used in the studied example (being $1.25f_{ck}$= 25 MPa for C20 and $1.25f_{yk}$= 525 MPa for S420 respectively), the new moment-curvature diagrams can be obtained easily by using RESPONSE 2000 program as shown in Figure 7.16 & Figure 7.17 for l_w= 6m of shear wall designed in y-direction and Figure 7.18 & Figure 7.19 for l_w= 10m of shear wall designed in x-direction.

Yield moment (i.e. plastic moment) for l_w= 6m of shear wall designed in y-direction and for l_w= 10m of shear wall designed in x-direction can be read from Figure 7.17 as 40 370 kN.m & from Figure 7.19 as 97 320 kN.m, respectively.

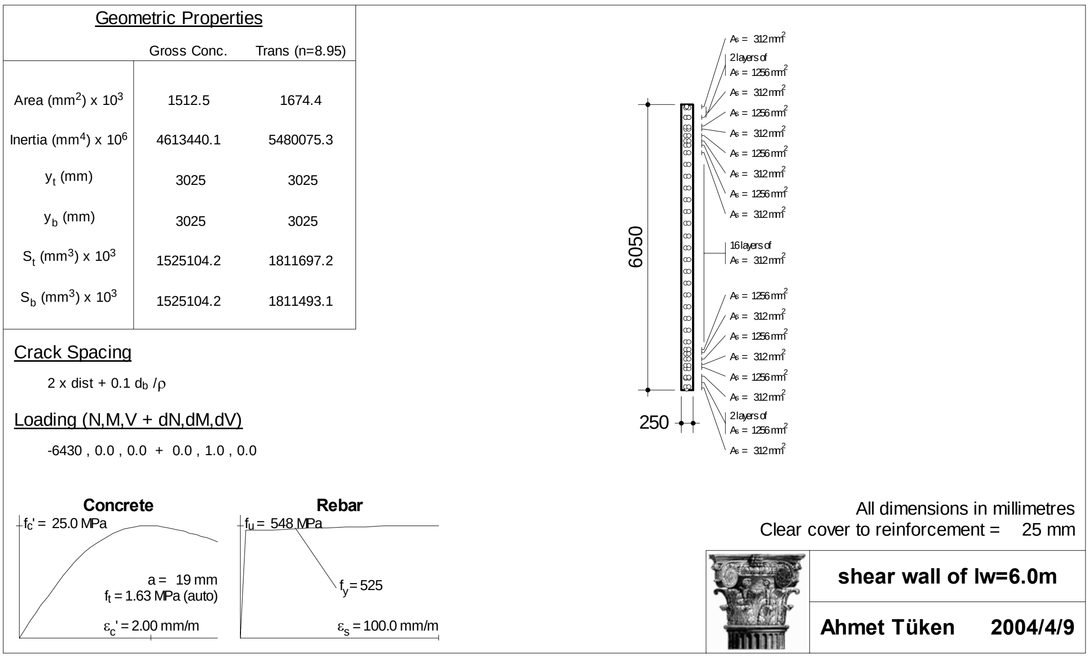


Figure 7.16 RESPONSE 2000 Input Values for the l_w=6m of Shear Wall Designed in y-direction (Over Strength Values of Materials were Used)

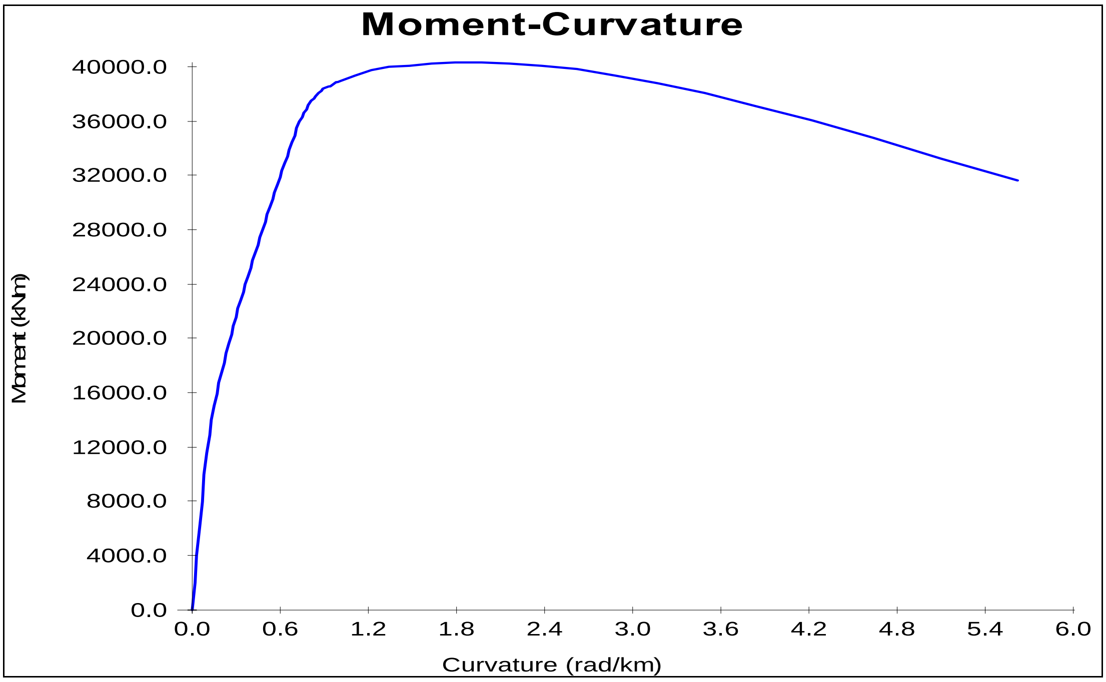


Figure 7.17 Moment-Curvature Diagram for the l_w=6m of Shear Wall Designed in y-direction (Over Strength Values of Materials were Used)

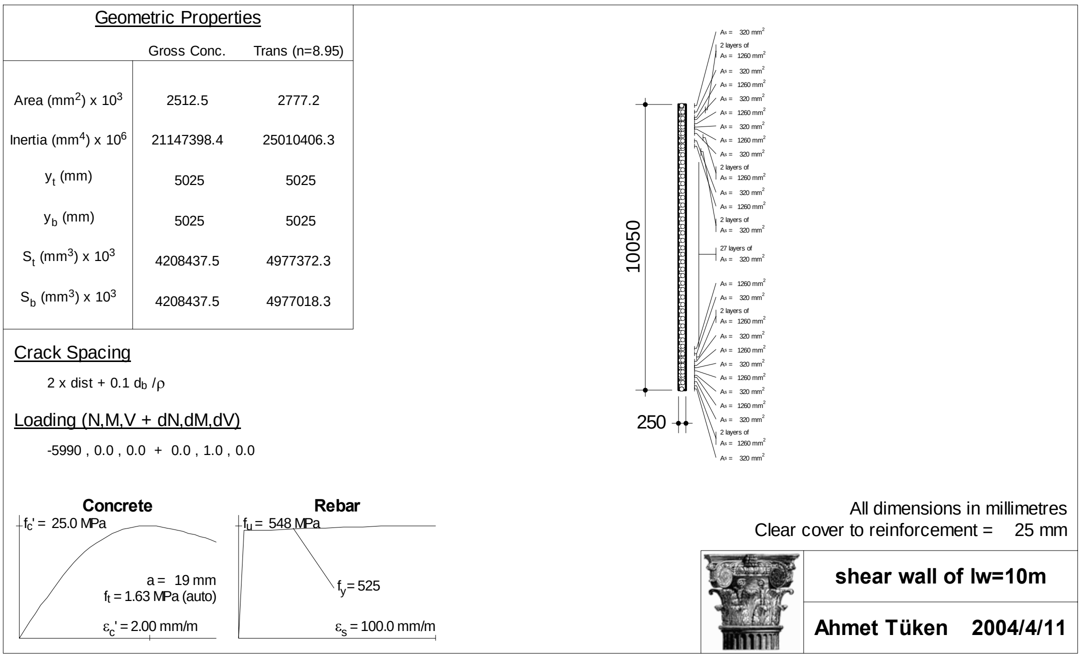


Figure 7.18 RESPONSE 2000 Input Values for the l_w=10m of Shear Wall Designed in x-direction (Over Strength Values of Materials were Used)

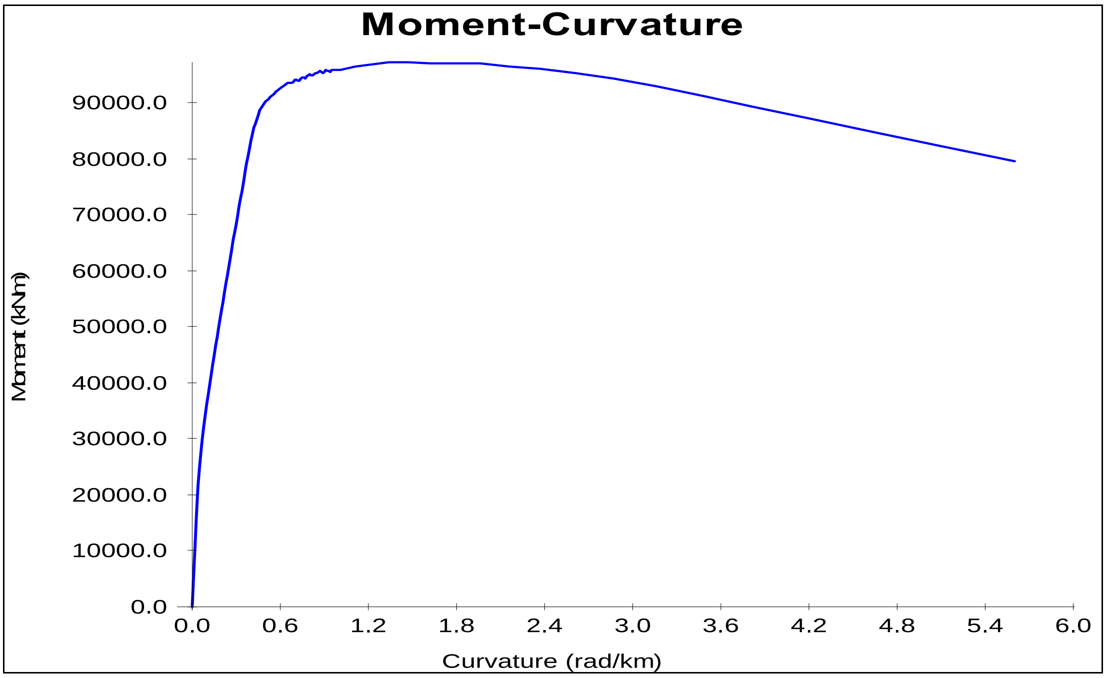


Figure 7.19 Moment-Curvature Diagram for the l_w=10m of Shear Wall Designed in x-direction (Over Strength Values of Materials were Used)

i. Preventing the Occurrence of Shear Failure of l_w= 6m of Shear Wall in y-direction by Using the Over Strength Values of Materials

Moment of inertia of l_w= 6m of shear wall designed in y-direction is

$$I\,(l_w = 6\text{m of shear wall}) = \frac{1}{12}(0.25)(6)^3 = 4.5\ \text{m}^4$$

Total moment of inertia of all shear walls in y-direction is

$$\sum I = (4.5)\text{x}3 + \frac{1}{12}(0.25)(5)^3\,\text{x}4 = 23.9\ \text{m}^4$$

Recalling the total design base shear, V_t,

$V_t = S(T).\ (A_0).\ (I).\ (w_i.n.A_p)\ /\ R$

$V_t = (2.5)(0.4)(1.0)(7\text{x}10\text{x}1300)\ /\ 7 = 13\ 000$ kN

The design shear force taken by l_w= 6m of shear wall in y-direction is

$V_{td} = 13\ 000 \text{ x } (4.5\ /\ 23.9) = 2\ 450$ kN

The value of shear force at the bottom of shear wall obtained from the graph of the analytical expression derived is the same as the design shear force calculated above as shown in Figure 7.20.

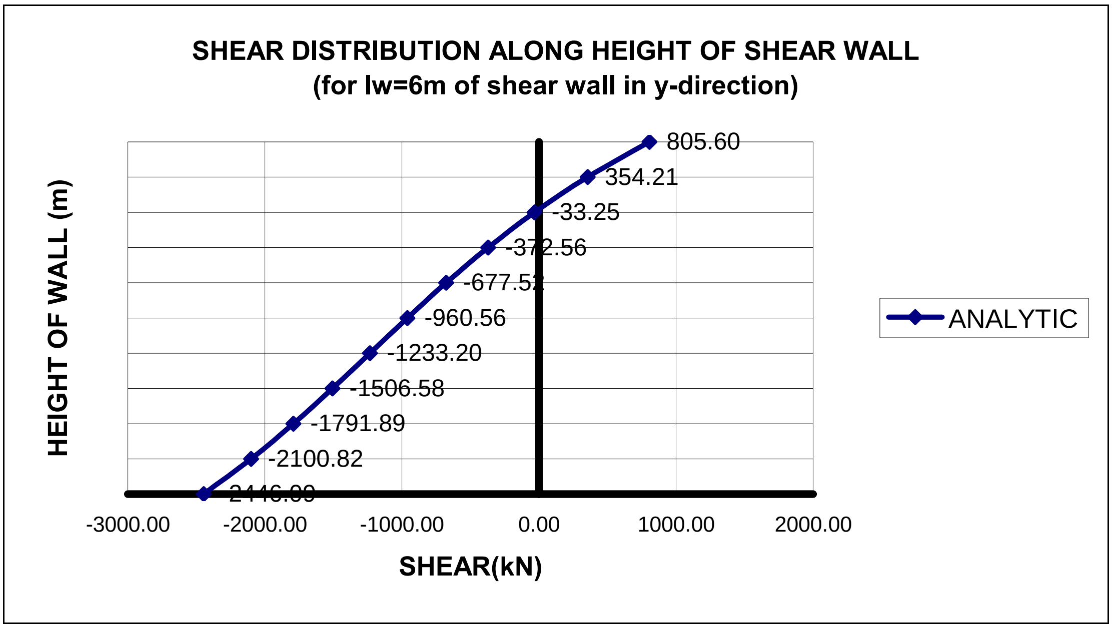


Figure 7.20 Shear Distribution along Height of Shear Wall of l_w=6m

The shear strength of shear wall, V_r, is

$V_r = A_{ch}\,(0.65\ f_{ctd} + \rho_{sh}\ f_{yd}) = 6x0.25\ m^2\ (0.65x1\ 000 + 0.005x365\ 000)$

$V_r = 3\ 710$ kN

It should be noted that $V_{td} < V_r$ requirement of design is satisfied.

On the other hand, the shear force due to yield moment (i.e. plastic moment) obtained by using the over strength values for materials used in design can be calculated as below.

$V_p = M_p / \bar{x} = 40\ 370$ kN.m / 15.924 m = 2 540 kN

where

$\bar{x}$ = Moment arm of the distributed load measured from the bottom of the shear wall shown in Figure 7.21 and given by Eqn.7.19 & Eqn.7.20.

Since V_r = 3 710 kN are greater than V_p = 2 540 kN, shear failure will not take place. This means that brittle failure is prevented by designing ductile shear wall.

In other words, flexural failure will occur before shear failure as reflected by the ratio below.

$$r = \frac{V_r}{V_p} = \frac{3710}{2540} = 1.46$$

It should be noted that reinforcement in the wall is used to satisfy the moment requirement. Therefore, a reduction in the amount of wall reinforcement is not possible.

Letting the bottom intensity of the distributed load of P(x) as "a" and the top intensity as "b", the intensity at the vertex of parabola as "c" and the height of vertex from the bottom of the shear wall as "d", the exact value of moment arm ($\bar{x}$) of the distributed load measured from the bottom of the shear wall shown in Figure 7.21 can be expressed as in Eqn.7.19.

$$\bar{x} = \frac{1}{4}.\frac{6cH_w^2 + (a-c)d^2 + (b-c)(H_w - d)(3H_w + d)}{3cH_w + (a-c)d + (b-c)(H_w - d)} \tag{7.19}$$

Substituting the known values of H_w = 30 m, a = 12.27 t/m, b = 16.31 t/m, c = 9.04 t/m and d = 12 m, the exact value of moment arm $\bar{x}$ can be calculated easily as in Eqn.7.20 for l_w=6 m of shear wall in y-direction.

$$\bar{x} = \frac{1}{4}.\frac{6(9.04)(30)^2 + (12.27-9.04)(12)^2 + (16.31-9.04)(30-12)(3x30+12)}{3(9.04)(30) + (12.27-9.04)(12) + (16.31-9.04)(30-12)}$$

$$\bar{x} = 15.924 \text{ m} \tag{7.20}$$

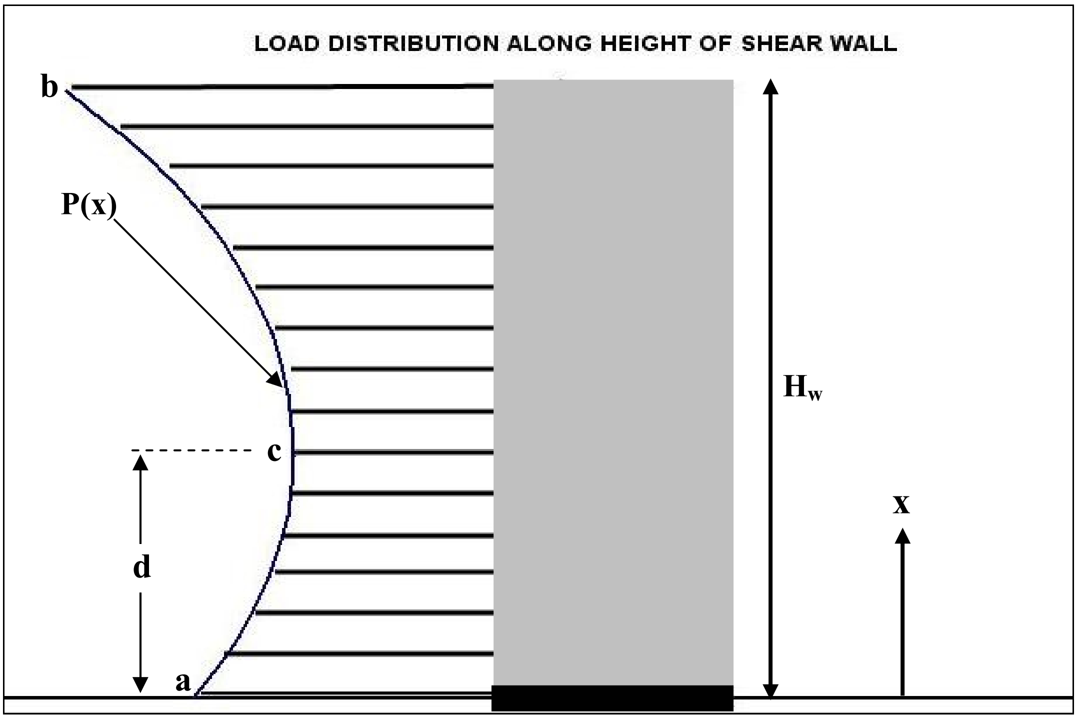


Figure 7.21 Load Distribution Profile along Height of Shear Wall of l_w=6m

ii. Preventing the Occurrence of Shear Failure of l_w= 10m of Shear Wall in x-direction by Using the Over Strength Values of Materials

Moment of inertia of l_w= 10m of shear wall designed in x-direction is

$$I\,(l_w = 10\text{m of shear wall}) = \frac{1}{12}(0.25)(10)^3 = 20.83\ \text{m}^4$$

Total moment of inertia of all shear walls in y-direction is

$$\sum I = (20.83)\text{x}2 + \frac{1}{12}(0.25)(5)^3\,\text{x}4 = 52.07\ \text{m}^4$$

Recalling the total design base shear, V_t,

V_t = S(T). (A_0). (I). (w_i.n.A_p) / R

V_t = (2.5)(0.4)(1.0)(7x10x1300) / 7 = 13 000 kN

The design shear force taken by l_w= 10m of shear wall in x-direction is

V_{td} = 13 000 x (20.83 / 52.07) = 5 200 kN

The value of shear force at the bottom of shear wall obtained from the graph of the analytical expression derived is the same as the design shear force calculated above as shown in Figure 7.22.

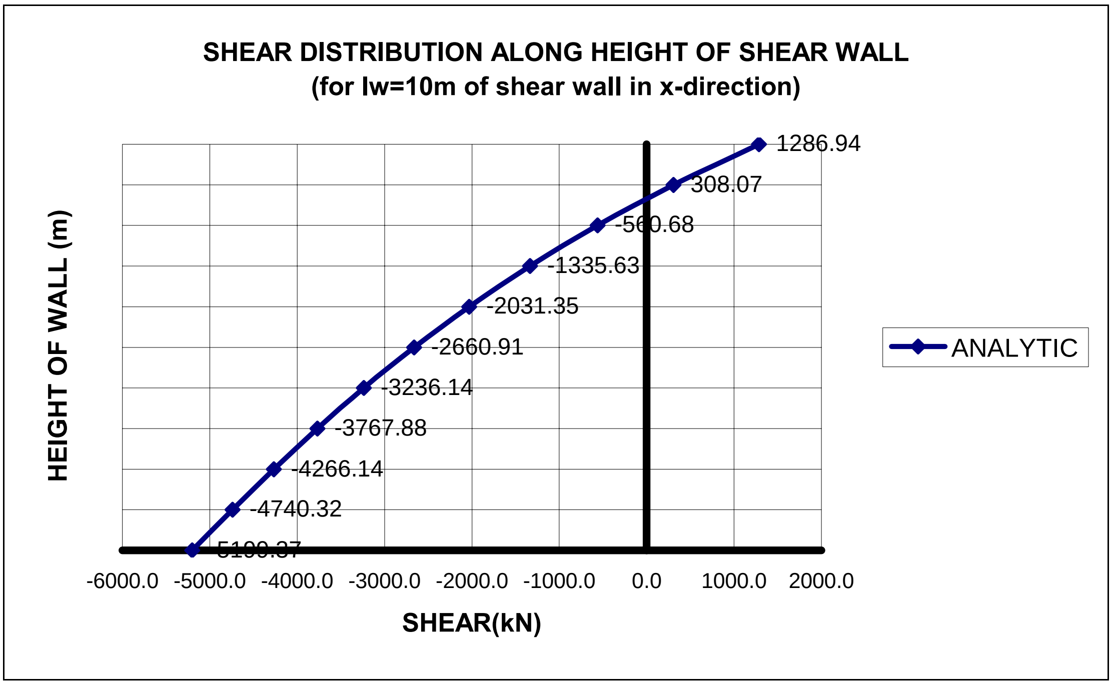


Figure 7.22 Shear Distribution along Height of Shear Wall of l_w=10m

The shear strength of shear wall, V_r, is

$V_r = A_{ch}\,(0.65\,f_{ctd} + \rho_{sh}\,f_{yd}) = 10x0.25\ m^2\,(0.65x1\,000 + 0.005x365\,000)$

$V_r = 6\,180$ kN

It should be noted that $V_{td} < V_r$ requirement of design is satisfied.

On the other hand, the shear force due to yield moment (i.e. plastic moment) obtained by using the over strength values for materials used in design can be calculated as below.

$V_p = M_p / \bar{x} = 97\,320$ kN.m / 17.25 m = 5 640 kN

where

$\bar{x}$ = Moment arm of the distributed load measured from the bottom of the shear wall shown in Figure 7.23 and given by Eqn.7.21 & Eqn.7.22.

Since V_r = 6 180 kN are greater than V_p = 5 640 kN, shear failure will not take place. This means that brittle failure is prevented by designing ductile shear wall.

In other words, flexural failure will occur before shear failure as reflected by the ratio below.

$$r = \frac{V_r}{V_p} = \frac{6180}{5640} = 1.096$$

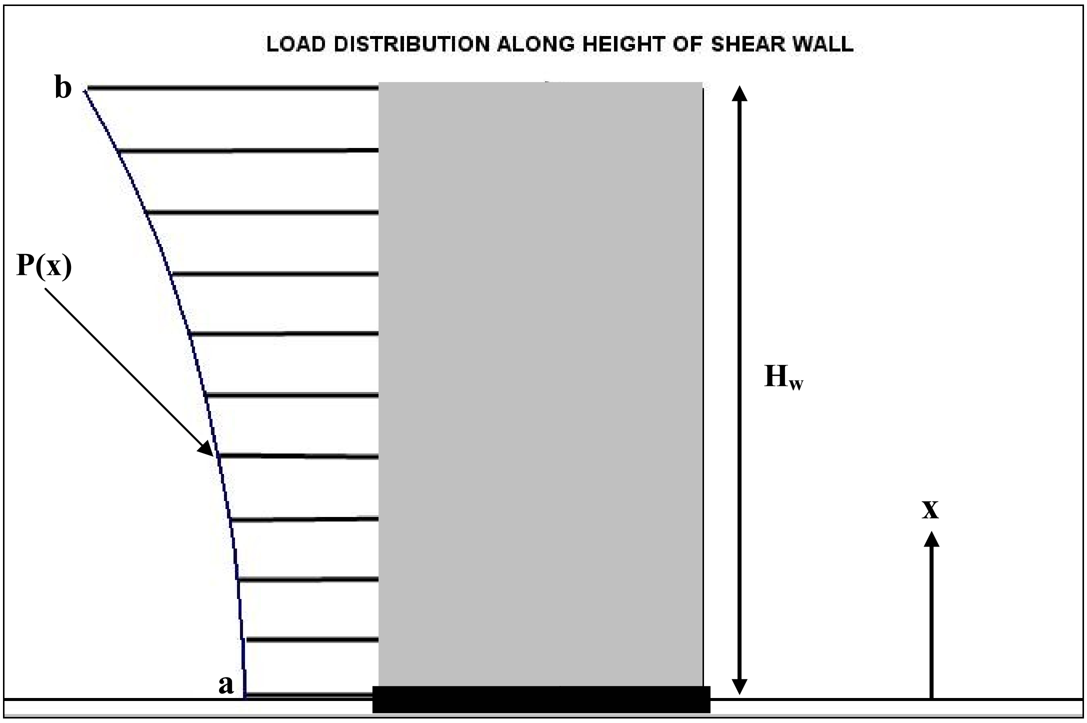


Figure 7.23 Load Distribution Profile along Height of Shear Wall of l_w=10m

Letting the bottom intensity of the distributed load of P(x) as "a" and the top intensity as "b", the exact value of moment arm ($\bar{x}$) of the distributed load measured from the bottom of the shear wall shown in Figure 7.23 can be expressed as in Eqn.7.21.

$$\bar{x} = \frac{3}{4}.H_w.\left(\frac{a+b}{2a+b}\right) \qquad (7.21)$$

Substituting the known values of $H_w = 30$ m, a = 15.14 t/m and b = 34.66 t/m, the exact value of moment arm $\bar{x}$ can be calculated easily as in Eqn.7.22 for l_w=10 m of shear wall in x-direction.

$$\bar{x} = \frac{3}{4}.30.\left(\frac{15.14 + 34.66}{2(15.14) + 34.66}\right) = 17.25\,\text{m} \tag{7.22}$$

CHAPTER 8

CONCLUSIONS & RECOMMENDATIONS

8.1 GENERAL

The reinforced concrete building must be designed to resist earthquake action. This design must incorporate mainly three criteria as adequate lateral stiffness, strength and ductility.

The main tool for satisfying the above mentioned seismic design requirements is the use of shear walls. However, what is the adequate and economical amount of shear walls that will satisfy stiffness, strength and ductility? The design problem of what makes adequate, proper, well placed and well-detailed shear walls was investigated.

The drift of the concrete building was also be investigated by using non-linear computer programs. Developing drift criteria that can be easily used by the design engineer was targeted. The minimum amount of longitudinal steel was established for shear walls having different cross-sectional geometry. The ultimate moment capacity of shear walls of various geometry and reinforcement was investigated too. Simple design equations that can be used by the office design engineer were developed.

Ductility is an indispensable yet an ambiguous concept. The reinforced concrete building must have enough ductility. How can ductility be measured? Curvature ductility is a cross-sectional property and can be readily calculated. On the other hand, the commonly measure of ductility is sway ductility. Design procedures were developed for shear walls, columns and beams in order to relate curvature ductility to sway ductility of the building and also in order to quantify ductility.

Assessment of building failures in recent earthquakes have shown that shear walls are mandatory to make a structure earthquake resistant. Also, the hybrid "shear wall – moment resisting frame" should satisfy the criteria of strength, ductility and stiffness.

The total and relative sway of a "framed" structure or a hybrid "shear wall-moment resisting frame" structure is a very important factor in assessing the seismic performance of a concrete structure. Observations of four major earthquakes in Turkey from 1992 to 1999 have indicated that uncontrolled sway is a significant contributor to collapse due to the occurrence of uncontrolled second order moments.

Being in full appreciation of the importance of P-Δ effects, particularly during a seismic attack, building codes (Turkish Earthquake Code, UBC, ACI and others) require the calculation of seismic drift and impose restrictions on its maximum or relative values.

Calculation of sway of a concrete structure is an effort "easier said than done". Of course, computer programs can do the job. However, to be able to calculate the sway of a three dimensional structure under acting seismic loads, all the geometric properties must be known. However, in the process of design, cross-sectional properties are generally what the design engineer is after. Additionally, the volumetric and time-consuming effort involved in the computer modeling and analysis of a three dimensional structure, considering the floor diaphragm, cannot be overlooked.

An analytical method is presented to calculate the sway of a three dimensional structure subject to any type of lateral load. This will enable the design engineer to evaluate the sway at any vertical level of the building, thus enabling the calculation of relative sway as well as the maximum.

The validity of the proposed analytical method is shown by comparing results with those obtained by computer. The proposed analytical method can be applied to satisfy drift requirements of Turkish Earthquake Code [1], UBC [61] and ACI [19], and illustrate how it can facilitate and improve the seismic design process.

The structure should be designed as a dual system as defined by the Uniform Building Code (UBC). In the dual system, the shear walls are to resist the total design base shear as dictated by a proper structural analysis. Additionally, the moment resisting frame is to resist 25 % of the total base shear independently.

The amount of shear wall is derived by an upper-bound calculation of the total design base shear and a lower-bound calculation of shear strength of shear walls containing minimum steel percentage.

The ductility capacity of a reinforced concrete building system is conveniently quantified by the ratio of the lateral displacement at a suitable level, such as the roof, to the yield displacement at the same level; that is, the displacement ductility factor. Because the transition from elastic to inelastic response is non-linear, acceptable simplifications need to be made particularly with respect to the definition of the displacement at first yield. While such global ductility is indicative of inelastic response of the entire system, the designer must pay even more attention to ductility demands that arise in critical potential plastic regions of the structure. To quantify such demands, ductilities may be expressed in terms of rotations or curvatures or strains and be related to the global ductility.

The dual structure should also possess enough ductility, as expressed by the displacement ductility ratio, $\mu_\Delta = 4 - 5$. Considering the difficulties and ambiguousness of displacement calculations in reinforced concrete structures, a more reliable measure of ductility is employed, as the curvature ductility ratio, μ_Φ. A plastic analysis is performed to relate the displacement ductility ratio to the more readily obtainable curvature ductility ratio.

The shear walls can be designed for a displacement ductility ratio of $\mu_\Delta = 4 - 5$, which in turn necessitates a curvature ductility that can be easily calculated. The cross-sections of the shear walls are to be consecutively designed to provide the curvature ductility demands that are calculated.

This study is directed to the analysis of multi-story buildings with structural frames, walls or their combinations. The stiffness and mass distribution is assumed to be regular over the height of the building. The displacement response of all structural elements, including structural walls, is assumed to be dominated by flexural deformations and influenced by seismic motions in their own plane. Effects of torsional behavior and vertical ground motions are not addressed.

8.2 CONCLUSIONS

An analytical method to calculate the sway of a three-dimensional framed and composite building is developed. Sway profiles of various buildings of different floor plans and different total number of floors are determined. The sway profiles are compared with results of computer solutions done by SAP2000.

- Sway profiles calculated by the proposed analytical method and SAP2000 are in very good agreement, the amount of error being less than 5% in most cases.
- By using the analytical method, the total sway magnitude of framed and composite building can easily and quickly be determined.
- By using the sway magnitudes determined, relative sway values of the building at any floor level along the height of the structure can be calculated. Thus, the requirement of the Turkish Earthquake Code can be met. If relative sway requirements can not be met, changes in the structure as necessary can be made and the design process can be repeated with ease. Without the analytical method proposed, tedious and time-consuming computer modeling efforts should be necessary.
- What is the amount of shear walls necessary in a building subject to the Earthquake Code defined seismic forces? Of course, building of any floor geometry and different number of floors should be considered in answering this vital question.
- A well designed building must possess three fundamental design requirements to be earthquake resistant.

i. Strength requirement must be met: The earthquake force acting on the structure must be successfully put to equilibrium by the resisting elements of the structure.

ii. Stiffness requirement must be met: The earthquake resistant structures must not undergo excessive sway. The relative sway restrictions, as required by the Earthquake Code, must be met.

iii. Ductility requirements must be met: In order to dissipate the seismic energy active in the structure, the sway magnitude corresponding to initial yielding must be grown to five times. This is known as the sway ductility ratio, $\mu_\Delta = \Delta_u / \Delta_y = 5$.

- The amount of shear walls satisfying all the three seismic design requirements is determined. The required amount of shear walls depend on the floor mass (w_i), the number of floors (n), the Earthquake Zone (A_0), the seismic force reduction factor (R) and the importance factor of the building (I).
- The required amount of shear walls, as expressed by the ratio of total shear wall area to total floor plan area, r, vary between 0.00256 for 4-story buildings to 0.0128 for 20-story buildings.

  $$r = \frac{A_w}{A_f}$$

  where

  A_w = total area of shear walls

  A_f = total floor plan area
- It has been shown that the amount of shear walls to satisfy strength requirement can easily met stiffness requirements. Therefore, the total length of shear walls necessary to balance the seismic force can be placed anywhere and in any combination in the floor plan. Any application of shear walls will satisfy stiffness requirement without any difficulty.
- The building must possess ductility to be earthquake resistant. However, ductility is an elusive concept. In order to design for ductility, it must be quantified.
- In the State-of-the-Art of earthquake engineering, a building is considered to be ductile if the sway ductility ratio of $\mu_\Delta = \Delta_u / \Delta_y = 4 – 5$ is used in the design process.

  where

  Δ_u = ultimate sway of the structure

  Δ_y = sway corresponding to initial yielding of the structure

- A method is developed to measure the ductility of the structure to satisfy the sway ductility ratio of $\mu_\Delta = 5$.
- The method developed to measure ductility depends on a plastic analysis and it depends on the tact that the plastic hinge forms at the base of the shear wall.
- It is very difficult to know the sway of the structure corresponding to initial yielding and the sway corresponding to limit state of the building. The developed method relates the sway ductility ratio of the structure to curvature ductility ratio of the shear wall. Thus, an ambiguous concept like the sway ductility of the structure is expressed as the well known curvature ductility of the cross-section. Of course, the curvature ductility is readily measurable.
- The required drift (Δ_i) to be used in the equation of stability index evaluation mentioned in TS 500 [5] can be calculated easily by using the analytical method presented. Therefore, it becomes very easy to check whether a structure is sway prevented or not without three-dimensional computer modeling. The stability index is expressed as below in TS 500.

$$\varphi = 1.5\Delta_i \frac{\sum \frac{N_{di}}{l_i}}{V_{fi}} \leq 0.05$$

where

Φ= Stability index

Δ_i= Drift at ith story

N_{di}= Axial design load

l_i= ith story column length, measured from axis to axis

V_{fi}= Total shear force at ith story

- The effect of gravity loads acting on shear walls should also be considered in design. Neglecting these loads does not necessarily lead to conservative designs.

- The strength of most rectangular reinforced concrete shear walls in high-rise buildings is governed by flexure rather than shear.
- The amount and distribution of vertical reinforcement in high-rise rectangular shear walls has a definite influence on energy absorption characteristics.
- Sufficient stiffness in the structure after the formation of a plastic hinge in the ductile shear wall is required in order to prevent the instability of structure as a whole.
- Since the area under the moment-curvature diagram is a measure of the energy absorbing capacity of reinforced concrete members, the variables affecting the energy absorption of walls are the same as those affecting their moment-curvature characteristics.
- 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.
- Unless a more refined analysis considering the nonlinear behavior of structural system is performed, second order effects may be taken into account according to the following equation given in Turkish Earthquake Code [1].

$$\theta_i = \frac{(\Delta_i)_{ort} . \sum_{j=1}^{N} w_j}{V_i . h_i} \leq 0.12$$

In the cases where second order effect indicator, θ_i, satisfies the condition given by the above equation for the earthquake direction considered at each storey, second order effects shall be evaluated in accordance with currently enforced specifications of reinforced concrete design. Here $(\Delta_i)_{ort}$ shall be determined as the average value of story drifts calculated for i'th storey columns and structural walls.

The required average drift $(\Delta_i)_{ort}$ to be used in the above equation of second order effect indicator can be calculated easily by using the analytical method presented. Therefore, it becomes very easy to check whether the second order effects shall be taken into account or not without three-dimensional computer modeling.

- There are several factors that must be taken into account in the design of ductile shear walls in order to ensure that the ductility, which has been calculated, can be fully realized. These factors are as follows:

i. Sufficient tension steel to ensure a "well-behaved" moment-curvature relationship should be provided

ii. Stability of structure as a whole during formation of plastic hinges in the shear walls should be established

iii. A premature anchorage failure of the tension steel before bending failure should be prevented

iv. A premature shear failure before bending failure should be prevented

v. A premature failure of other structural framing elements should be prevented

vi. Allowance for decrease in concrete strength must be taken into account

vii. Development of full moment capacity of wall at foundation should be established

viii. Construction joint details should be considered carefully

ix. To develop flexural and shear strength, two significant components of a shear wall are necessary; web reinforcing (consisting of horizontal and vertical reinforcing at uniform spacing) and boundary reinforcing (vertical steel with ties located at both ends of the shear wall)

8.3 RECOMMENDATIONS FOR FURTHER RESEARCH

The following topics are recommended for future studies:

- The seismic behavior of structures designed by the proposed method must be validated by experimental studies.
- The application of the proposed method to rehabilitation and strengthening of existing structures must be investigated. How does the proposed amount of shear walls improve the seismic behavior of existing structures that are seismically defective?
- The behavior of structures as designed by the proposed method should be investigated by non-linear computer programs.
- In this thesis, only symmetrical structures have been considered. The possibilities of application of the proposed method to unsymmetrical structures should further be investigated.

REFERENCES

1. “Afet Bölgelerinde Yapılacak Yapılar Hakkında Yönetmelik”, (Specification for Structures to be Built in Disaster Areas), Ministry of Public Works and Settlement, Government of Republic of Turkey, 1997

2. Atımtay, E., “ Çerçeveli ve Perdeli Betonarme Sistemlerin Tasarımı”, (Design of Reinforced Concrete Framed and Shear Wall Structures), Vol. 1 & 2, Revised 2nd Ed., Ankara, 2001

3. Argeşo, A.H.,”A Panel Macro Element for the Analysis of Structural Wall Systems”, MS Thesis, Middle East Technical University, Civil Engineering Department, September 1991

4. Bayulke, N., “Depreme Dayanıklı Betonarme ve Yığma Yapı Tasarımı”, İnşaat Mühendisleri Odası İzmir Şubesi, Yayın No:27, 1998

5. “Betonarme Yapıların Hesap ve Yapım Kuralları, TS 500”, (Building Code Requirements for Reinforced Concrete Structures), Turkish Standards Institute, 2000

6. Can, A.A., “Development of a Computer Program for the Static and Dynamic Analysis of Asymmetric Frame-Shear Wall Structures”, MS Thesis, Middle East Technical University, Civil Engineering Department, June 1991

7. Capuani, D., Merli, M. and Savoia, M., “An Equivalent Continuum Approach for Coupled Shear Wall”, Engineering Structures, Vol.16, No.1, 1994

8. Celep, Z. and Kumbasar, N., “Betonarme Yapılar”, İstanbul Teknik Üniversitesi, İstanbul, 1996

9. Chopra, K. Anil and Rakesh, K. Goel, “Period Formulas for Concrete Shear Wall Buildings”, Journal of Structural Engineering, Vol.124, April 1998

10. Bilyap, Siracettin, “Betonarme Yüksek Yapılarda Burulmasız Perde – Çerçeve Sistemlerinin Yatay Kuvvetlere Göre Yaklaşık Hesap Yöntemleri ve Dinamik Karakteristikleri”, Ege Üniversitesi İnşaat Fak. Yayın No:3, Bornova, 1979

11. Dowrick, David J., “Earthquake Resistant Design for Engineers and Architects”, John Wiley and Sons Inc., Great Britain, 1990

12. Ersoy, U., “Reinforced Concrete”, Middle East Technical University, Ankara, 1994

13. Ersoy, U., “Seismic Resistant Reinforced Concrete Structures - Design Principles”, Journal of Islamic Academy of Sciences 1:20-26, 1998

14. Ersoy, U. and Atımtay E., ”Betonarme (Reinforced Concrete)”, Güven Kitabevi, 1975

15. Fajfar, P. and Krawinkler, H., “Nonlinear Seismic Analysis and Design of Reinforced Concrete Buildings”, Elsevier Science Publishers Ltd., New York, 1992

16. Ferguson, P.M., Breen, J.E. and Jirsa, J.O., “Reinforced Concrete Fundamentals”, John Wiley and Sons Inc., New York, 1988

17. Ghosh, S.K. and Domel, A.W., “Design of Concrete Buildings for Earthquake and Wind Forces”, Portland Cement Association and the International Conference of Building Officials, Illinois, USA, 1992

18. Hasgur, Z. and Gunduz, N., “Betonarme Yüksek Binalar”, İstanbul Teknik Üniversitesi, 1992

19. ACI 318 - Building Code Requirements for Structural Concrete, 2002

20. Kong, F., Teng, S., Singh, A. and Tan, K., “Effects of Length of Tension Reinforcement on the Behavior of Lightweight Concrete Deep Beams”, ACI Structural Journal, V93, No:1, January-February 1996

21. Mau, S.T. and Hsu, T.C., “Formula for the Shear Strength of Deep Beams”, ACI Structural Journal, V86, No:5, September-October 1989

22. Nayef, A., “A Comparison of Methods for the Analysis of Shear Wall- Frame Systems”, MS Thesis, Middle East Technical University, Civil Engineering Department, January 1991

23. Paulay, T., “The Design of Ductile Reinforced Concrete Structural Walls for Earthquake Resistances”, Earthquake Spectra, Vol.2, No:4, 1986

24. Paulay, T. and Priestley, M.J.N., “Seismic Design of Reinforced Concrete and Masonry Buildings”, John Wiley and Sons Inc., 1992

25. Paulay, T., “Coupling Beams of Reinforced Concrete Shear Walls”, Journal of the Structural Division, American Society of Civil Engineers, V.97, No:3, March 1971

26. Tüken, A., Tuna, M. E., Atımtay, E., “Analysis and assessment of seismic drift of concrete framed structures”, SEMC 2004: The Second International Conference on Structural Engineering, Mechanics and Computations, Cape Town, South Africa, 5-7 July 2004 (accepted for publication)

27. Siao, W.B., “Shear Strength of Short Reinforced Concrete Walls, Corbels and Deep Beam”, ACI Structural Journal, V.91, No:2, March-April 1994

28. Subedi, N.K., “Reinforced Concrete Coupled Shear Wall Structures I: Analysis of Coupling Beams”, Journal of Structural Engineering, Vol.117, No:3, March 1991

29. Subedi, N.K., “Reinforced Concrete Coupled Shear Wall Structures II: Ultimate Strength Calculations”, Journal of Structural Engineering, Vol.117, No:3, March 1991

30. Subedi, N.K., “Reinforced Concrete Deep Beams, A Method of Analysis”, Proceedings of Institute of Civil Engineering, ICE 85(2), 1988

31. Bilyap, Siracettin, “Betonarme Yüksek Yapılarda Burulmasız Perde – Çerçeve Sistemlerinin Yatay Kuvvetlere Göre Yaklaşık Hesap Yöntemleri ve Dinamik Karakteristikleri”, Ege Üniversitesi İnşaat Fak. Yayın No:3, İzmir, 1979

32. Tarman, A., “Finite Element Solution of Shear Wall Frames”, MS Thesis, Middle East Technical University, Civil Engineering Department, 1974

33. Tassios, T.P., Moretti, M. and Bezas, A., “On the Behavior and Ductility of Reinforced Concrete Coupling Beams of Shear Walls”, ACI Structural Journal, Vol.93, No:6, November-December 1996

34. Tegos, I.A. and Penelis, G.Gr., “Seismic Resistance of Short Columns and Coupling Beams Reinforced with Inclined Bars”, ACI Structural Journal, Nov.-Dec. 1988

35. Trak, A.Bulent, “Analysis of Open-Section Shear Walls”, MS Thesis, Middle East Technical University, Civil Engineering Department, 1980

36. Tso, W., “Shear Walls Coupled by Cross Wall”, Journal of Structural Engineering, Vol.109, 1983

37. Wallace, M., John, W., "New Methodology for Seismic Design of Reinforced Concrete Shear Walls", Journal of Structural Engineering, Vol.120, No:3, March 1994

38. Han, S.W., Kwon, O.S., Lee, L.H., "Investigation of dynamic P-Delta effect on ductility factor", Structural Engineering and Mechanics, Vol.12, No:3, pp.249-266, 2001

39. Tremblay, R., Leger, P., Tu, J., "Inelastic seismic response of concrete shear walls considering P-Delta effects", Canadian Journal of Civil Engineering, Vol.28, No:4, pp.640-655, 2001

40. Park, R. and Paulay, T., "Reinforced Concrete Structures", John Wiley and Sons Inc., Canada, 1975

41. Kwan, A., "Improved Wide-Column Analogy for Shear / Core Wall Analysis", Journal of Structural Engineering, Vol.119, February 1993

42. Cook, A.D., "Avoidance of Parasitic Shear in Plane Elements", Journal of Structural Division, ASCE, 101(6), 1975

43. Macleod, I.A., "Analysis of Shear Wall Buildings by Frame Element Method", Proceedings Inst. Civil Engineers., Part2, 1, 55, 593, September 1973

44. Wong, Y.C., Coul, A., "Interaction Between Floor Slabs and Shear Walls in Tall Buildings" ACI Pub., SP63, pp.543, 1980

45. Khatri, Dilip, "Nonlinear Analysis of Reinforced Concrete Shear Wall Structures", Ph.D. Thesis, University of Southern California, Faculty of the Graduate School, 1998

46. Lepage, Andres, "A Method for Drift Control in Earthquake-Resistant Design of RC Building Structures", Ph.D. Thesis, University of Illinois at Urbana-Champaign, The Graduate College, 1996

47. Diker, Y., “Seismic Analysis and Design of Tunnel Form Buildings”, MS Thesis, Middle East Technical University, Civil Engineering Department, May 2000

48. Kaya, Y.M., “Capacity Design Considerations of Tunnel Form Buildings”, MS Thesis, Middle East Technical University, Civil Engineering Department, May 2000

49. Demiroğlu, M., “Seismic Capacity Design Investigation of Tunnel Form Buildings”, MS Thesis, Middle East Technical University, Civil Engineering Department, May 2000

50. Ghuneim, N., “Modeling of Tunnel Form Buildings by Equivalent Frame Method”, MS Thesis, Middle East Technical University, Civil Engineering Department, September 1999

51. Tu, Jianhua, “P-Delta Effects on the Inelastic Seismic Response of Reinforced Concrete Shear Wall Buildings”, Ph.D. Thesis, University of Montreal, 2000

52. Fintel, M., “Ductile Shear Walls in Earthquake Resistant Multistory Buildings”, ACI Journal, June 1974, pp. 296-305

53. Baikov, V.N. (Editor), Reinforced Concrete Structures, Mir Publishers, Moscow, 1974

54. ACI Committee 442 Report, “Response of Building to Lateral Forces”

55. Handbook of Concrete Engineering, Edited by Mark Fintel, Van Nostrand, 1974 (Chapter 10: Multistory Structures, pp. 287-365)

56. Fintel, M., “Shear Walls – An Answer for Seismic Resistance “, Concrete International, July 1991, pp. 48-53

57. Mattacchione, A., “Equivalent Frame Method Applied to Concrete Shear Walls”, Concrete International, November 1991, pp. 65-72

58. Celep, Z. ve Kumbasar, N., Betonarme Yapılar, Beta Dağıtım, İstanbul, 1996

59. Chopra, A.K., Dynamics of Structures: Theory and Applications to Earthquake Engineering , Prentice Hall International, 1996

60. Salvadori, M., and Levy, H., Structural Design in Architecture, Prentice Hall, 1978

61. Uniform Building Code (UBC) - International Conference of Building Officials, 1999

62. Bilyap, Siracettin, “Betonarme Yüksek Yapılarda Perde – Çerçeve Sistemlerin Yatay Kuvvetlere Göre Hesabı”, Ege Üniversitesi İnşaat Fak. Yayın No:2, İzmir, 1979

63. “SAP2000” Computer Program, Computers and Structures, Berkeley, USA, 2000

64. Stafford – Smith, B., and Girgis, A., “Simple Analogous Frames for Shear Wall Analysis”, ASCE Journal of Structural Engineering, Vol. 110, No:11, November 1984

65. Qadeer, A., and Stafford – Smith, B., “Bending Stiffness of Slabs Connecting Shear Wall Structures”, ACI Journal, June 1969

66. Coull, A., and Choudhury, J.R., “Analysis of Coupled Shear Walls”, ACI Journal, September 1967

67. “Response of Multistory Concrete Structures to Lateral Forces”, ACI Publication, SP 36, 1973

68. Rosman, R., “An Approximate Method of Analysis of Walls of Multistory Buildings”, Civil Engineering and Public Works Review, Vol. 59, January 1964, pp. 717-734

69. Taranath, B.S., “Analysis of Interconnected Open Section Wall Structures”, ASCE Journal, 1986

70. Murashev, V., Sigalov, E., and Baikov, V. N., “Design of Reinforced Concrete Structures”, Mir Publishers, Moscow, 1976

71. Atımtay, E., and Tuna, M.E., “Designing the concrete dual system”, Structural Engineering, Mechanics and Computation, Vol. 2, 2001, pp.1009-1015

72. Paulay, T., “Seismic Displacement Compatibility in Mixed Structural Systems”, Proceedings of the Uğur Ersoy Symposium on Structural Engineering, July 1999, pp. 275-292

73. Tezcan, S. S., Kaya, Z. D., “Design Charts for Shear Walls with Openings”, Türkiye Deprem Vakfı (Turkish Earthquake Foundation), 2001

74. Paulay, T., “Simplicity and Confidence in Seismic Design”, The Fourth Mallet - Milne Lecture, 1993

75. Irwin, A. W., “Design of Shear Wall Buildings”, Report 102, Construction Industry Research and Information Association, London, 1984

76. Coull, A. and Irwin, A. W., “Analysis and Load Distribution in Multi – Storey Shear Wall Structures”, Structural Engineering, Vol.88, No.8, 1970, pp.301-206

77. Macleod, I.A., “Structural Analysis of Wall Systems”, Structural Engineering, Vol.55, No.11, 1977, pp.487-495

78. Park, R. and Paulay, T., "Reinforced Concrete Structures", John Wiley & Sons, New York, 1975

79. Canadian Standards Association, "Design of Concrete Structures for Buildings", A National Standard of Canada, CAN-A23.3-M84, 1987

80. Comite Euro-International du Beton (CEB), "Seismic Design of Concrete Structures", Ed. P. E. Pinto, Grower Technical Press Ltd, England, 1987

81. Commission of the European Communities, "Structures in Seismic Regions - Eurocode 8", 1988

82. Beck, H., "Contribution to the Analysis of Coupled Shear Walls", ACI Journal, Vol.59, August 1962, pp.1055-1070

83. Rosman, H., "Analysis of Special Concrete Shear Wall Systems", Proceedings of the Institution of Civil Engineers, Supplement (VI) Paper 72665, pp.131-152, 1970

84. Coull, A. and Irwin, A. W., "Design of Connecting Beams in Coupled Shear Wall Structures", ACI Journal, Vol.66, March 1969, pp.205-209

85. "ETABS" Computer Program, Computers and Structures, Berkeley, USA, 2003

86. "Borland Delphi" Computer Program, 2003

87. "Response-2000 Reinforced Concrete Sectional Analysis" Computer Program, 2000

88. Kim, H.S., Lee, D.G., "Analysis of shear wall with openings using super elements", Engineering Structures, Vol.25, No.8, pp.981-991, July 2003

89. Balkaya, C., Kalkan, E., "Estimation of fundamental periods of shear-wall dominant building structures", Earthquake Engineering and Structural Dynamics, Vol.32, No.7, pp.985-998, June 2003

90. Paterson, J., Mitchell, D., "Seismic retrofit of shear walls with headed bars and carbon fiber wrap", Journal of Structural Engineering - ASCE, Vol.129, No.5, pp.606-614, May 2003

91. Aksogan, O., Arslan, H.M., Choo, B.S., "Forced vibration analysis of stiffened coupled shear walls using continuous connection method", Engineering Structures, Vol.25, No.4, pp.499-506, March 2003

92. Lee, S.Y., Haldar, A., "Reliability of frame and shear wall structural systems. II: Static loading", Journal of Structural Engineering – ASCE, Vol.129, No.2, pp.224-232, February 2003

93. Lee, S.Y., Haldar, A., "Reliability of frame and shear wall structural systems. II: Dynamic loading", Journal of Structural Engineering – ASCE, Vol.129, No.2, pp.233-240, February 2003

94. Wen, Z.P., Hu, Y.X., Chau, K.T., "Site effect on vulnerability of high-rise shear wall buildings under near and far field earthquakes", Soil Dynamics and Earthquake Engineering, Vol.22, No.9-12, pp.1175-1182, October - December 2002

95. Brun, M., Reynouard, J.M., Jezequel, L., "A simple shear wall model taking into account stiffness degradation", Engineering Structures, Vol.25, No.1, pp.1-9, Jan 2003

96. Hidalgo, P.A., Jordan, R.M., Martinez, M.P., "An analytical model to predict the inelastic seismic behavior of shear-wall, reinforced concrete structures", Engineering Structures, Vol.24, No.1, pp.85-98, Jan 2002

97. Kuang, J.S., Ng, S.C., “Dynamic coupling of asymmetric shear wall structures: an analytical solution”, International Journal of Solids and Structures, Vol.38, No.48-49, pp.8723-8733, November - December 2001

98. Kwan, A.K.H., He, X.G., “Finite element analysis of effect of concrete confinement on behavior of shear walls”, Computers and Structures, Vol.79, No.19, pp.1799-1810, July 2001

99. Tremblay, R., Leger, P., Tu, J., “Inelastic seismic response of concrete shear walls considering P-delta effects”, Canadian Journal of Civil Engineering, Vol. 28, No.4, pp.640-655, August 2001

100. Li, Q.S., “Stability of tall buildings with shear-wall structures”, Engineering Structures, Vol.23, No.9, pp.1177-1185, September 2001

101. Ishikawa, K., Kawasaki, Y., Tagawa, K., “Effect of shear wall location in rigid frame on earthquake response of roof structure”, Structural Engineering and Mechanics, Vol.11, No.6, pp.605-616, June 2001

102. Kuang, J.S., Ng, S.C., “Coupled lateral-torsion vibration of asymmetric shear-wall structures”, Thin-Walled Structures, Vol.38, No.2, pp.93-104, October 2000

103. Wang, Q.F., Wang, L.Y., Liu, Q.S., “Effect of shear wall height on earthquake response”, Engineering Structures, Vol.23, No.4, pp.376-384, April 2001

104. Lu, X.L., Wu, X.H., “Study on a new shear wall system with shaking table test and finite element analysis”, Earthquake Engineering and Structural Dynamics, Vol.29, No.10, pp.1425-1440, October 2000

105. Lee, L.H., Chang, K.K., Chun, Y.S., “Experimental formula for the fundamental period of RC buildings with shear-wall dominant systems”, Structural Design of Tall Buildings, Vol.9, No.4, pp.295-307, September 2000

106.Hwang, S.J., Fang, W.H., Lee, H.J., et al., "Analytical model for predicting shear strength of squat walls", Journal of Structural Engineering – ASCE, Vol. 127, No.1, pp.43-50, January 2001

107.Mistakidis, E.S., Panagouli, O.K., "Strength evaluation of retrofit shear wall elements with interfaces of fractal geometry", Engineering Structures, Vol.24, No.5, pp.649-659, May 2002

108.Palermo, D., Vecchio, F.J., "Behavior of 3-D Reinforced Concrete Shear Walls", ACI Structural Journal, Vol. 99, No. 1, pp. 81-89, 2002

109.Ile, N., Reynouard, J.M., "Nonlinear Analysis of Reinforced Concrete Shear Wall Under Earthquake Loading", Journal of Earthquake Engineering, Vol.4, No.2, pp.183-213, 2000

110.Massone, L.M., Wallace, J.W., "Load-deformation responses of slender reinforced concrete walls", ACI Structural Journal, Vo.101, No.1, pp.103-113, January-February 2004

111.Lee, T.H., Mosalam, K.M., "Sensitivity of Seismic Demand of a Reinforced Concrete Shear-Wall Building", Ninth International Conference on Application of Statistics and Probability in Civil Engineering (ICASP 9), San Francisco, California, USA, July 6-9, 2003

112.Zhao, Z.Z., Kwan, A.K.H., He, X.G., "Nonlinear finite element analysis of deep reinforced concrete coupling beams", Engineering Structures, Vo.26, No.1, pp.13-25, January 2004

113.Zhao, Q.H., Astaneh-Asl, A., "Cyclic behavior of traditional and innovative composite shear walls", Journal of Structural Engineering – ASCE, Vol.130, No.2, pp.271-284, February 2004

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