*Fifth International Congress on Advances in Civil Engineering, 25-27 September 2002*
*Istanbul Technical University, Istanbul, Turkey*

# Performance Based Seismic Evaluation of Shear-wall Dominant Building Structures

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## Abstract

The occurrences of 1999 $M_w$ 7.4 Kocaeli and $M_w$ 7.1 Düzce earthquakes in Turkey once again demonstrate the behavioral importance of RC shear-wall dominant structures that commonly built by using tunnel form techniques. Almost non-damaged condition of these buildings in the aftermath of two destructive urban earthquakes drew our attention to focus on their performances under earthquake excitations. In this study, seismic behaviors of those structures are investigated in details by performing nonlinear 3D pushover analyses. Additionally, the importance of 3D action in the structure's behavior because of slab-wall interaction, the efficiency of transverse walls, the effect of 2D and 3D modeling on the evaluation of capacity and demand relations, as well as damping effects are discussed. The importance of 3D behavior is demonstrated through comparison with 2D solutions. For that purpose 2-story and 5-story structures are modeled separately. In the analysis part, material nonlinearity including rotating crack capability is taken into account and reinforcement is modeled as smeared layer of reinforcement and main discrete reinforcement around the openings. In both structural cases, performances of the models are determined by Capacity Spectrum Method (CSM). This study shows that 3D effects have great significance to predict the actual capacity, failure mechanisms, and to evaluate the seismic performance. 3D nonlinear analysis provide higher and at the same time more accurate capacities for tunnel form buildings which leads more economic designs as well as reliable retrofitting and strengthening issues for the damaged structures. The transverse walls provide extra resistance and significantly increase the predicted load capacity as a result of T/C (tension/compression) coupling between vertical walls. In general, tunnel form building structures show well performance under earthquake excitation with close to square and symmetric architectural plans as studied here.

***KEY WORDS***: Shear-wall; tunnel form building; pushover analysis; torsion; seismic performance; finite element modeling; capacity spectrum method

# Introduction

Tunnel form buildings, having a shear-wall dominant structural system, are commonly built in countries under substantial seismic risk like: Chile, Japan, Italy and Turkey. In spite of the abundance of such structures in some parts of the world, limited research has been directed to the analysis, design and safety criteria of this special building type.

Tunnel form building, are composed of vertical and horizontal panels set at right angles. The typical illustration for this special structural system is given in Figure 1. There are no beams or columns and these structures generally utilize all wall elements as primary load carrying members. The walls and slabs having almost same thickness are cast in a single operation. Like the wall forms and table forms, this reduces not only the number of joints, but also the assembly time. Consequently, the casting of walls and slabs can be completed in one day. The simultaneous casting of walls, slabs and cross walls results in monolithic structures, which provide high seismic performance and therefore, they meet seismic codes requirements of many countries located in regions under high earthquake risk. In addition to their considerable resistance, the speed and ease of building up make them preferable as the multi-unit construction of public and residential buildings.

Figure 1. Tunnel form construction technique and its special formwork system

Besides their experienced well behavior under lateral forces, the current seismic provisions constitute inadequate guidelines for their detailed analysis. Although respond of tunnel form buildings to earthquake excitations are different than that of conventional RC frame type shear wall structures because of their discrete lateral load transferring system, most of the time they are accepted as ordinary RC buildings having standard shear walls. This general trend, for the most part stemming from the lack of and/or the imperfect reliability of the specific supporting guidance in current codes, affect all analytical methods and procedures applied for the design of these structures. On the other hand, 3D behavior, floor flexibilities, slab-wall interaction, material nonlinearity including cracking, stress focusing around openings, the amount and location of steel reinforcement, torsional disturbance are all major contributors, that should be considered to investigate the actual behavior of those structures. Our effort was spent to illuminate the importance of those aforementioned factors by performing 3D pushover analyses on selected 2 and 5 story buildings and comparing the obtained results with that of commonly used 2D analyses.

To accurately predict the nonlinear seismic performance of these structures with sufficient accuracy, due care has been given to create detailed and efficient models of the structures, taking into account all necessary geometric and strength characteristics of shear walls, slabs and slab-wall connections. Toward minimizing the computational requirements and the volume of input and output data to be handled, an effort was made to select powerful three and two dimensional models that can provide, with appropriate selection of parameters, acceptable representation of nonlinear behavior on member and structure levels, while guaranteeing numerical stability. The two dimensional modeling was intentionally selected to show the behavioral differences between simplified models and three dimensional detailed analyses.

In this study, two types of analyses have been performed using the structural models. Eigenvalue analyses are conducted to determine the elastic periods and the mode shapes of the buildings needed to convert obtained load deflection curves to acceleration displacement response spectrum format (ADRS). Pushover analyses are then performed using the calculated lateral load shapes with increasing severity. The analyses are progressed until all predefined collapse limits are exceeded. In static analyses permanent loads are first applied and iteration to equilibrium is performed. This is followed by applying incrementally increased lateral loads. Although, analyses are both inelastic, geometric nonlinearity is disregarded due to existing of small deformations.

The analytical modeling, assumptions and approaches besides the results of the analyses complementing this work are summarized in the remaining sections of this paper. With all this available information, this study provides a general methodology for the 3D pushover analysis of shear wall dominant buildings based on specifically developed finite elements characterizing the actual material nonlinearity with associated limitations and uncertainties. Other seismic objectives like torsional disturbances, reinforcement detailing, damping effects and floor flexibility associated with those structures are also investigated.

These available results, as well as additional studies on 3D pushover analyses, will undoubtedly be used in the near future to make any necessary changes and updates to progressively modify and improve the proposed finite element modeling and reduce the inherent uncertainties.

## Analytical Model Development

By way of evaluating the 3D and 2D nonlinear capacity of tunnel form structures, 5-story and 2-story buildings are selected as representative case studies. The subject buildings are typical reinforced concrete residential buildings. A detailed description of the plan and sections of these buildings are given in Figure 2. Their structural systems consist of solely shear-walls and slabs having same thickness as usual applications. All of the intended lateral strength and stiffness of the building reside in the interior shear walls with the contribution of slabs. In addition to their resistance to lateral loads, these distributed walls in the plan are also designed to carry gravity loads.

For the analytical studies, 3D and 2D nonlinear models were constructed. These models consider door openings and include both discrete and embedded reinforcements. The shear-wall bases are modeled as fixed at the foundation levels. The slabs are modeled by using finite elements having both flexural and membrane capabilities, similar to those used in the wall modeling. Instead of accepting in-plane floor stiffness to be rigid, in-plane floor flexibility and slab-wall interaction are taken into account thereby requiring a shell element capability. The performance of a good 3D nonlinear finite element analysis of shear wall dominant structures requires a basic element with representative membrane and flexural characteristics. In that connection, a new nonlinear shell element was developed using an isoparametric serendipity interpolation scheme with 5 d.o.f. per node. This form of element description was selected in order to have a variable order of displacement definition possible along each of the element edges. Additionally, in order to consider the actual cover dimensions for the defined discrete reinforcement, those edge nodes which serve as the end nodes for that reinforcement must be capable of taking a place at the proper location along an edge. This placement location must not be restricted to points having a certain distance from the corner in order to avoid the development of a singularity condition. The element developed for this study allows such an arbitrary placement. This issue is taken up again with details in the next section.

To reduce the computational time as well as capacity associated with 3D modeling of incorporating shell elements, a mixture of finite elements of different order are used in the floor modeling. The number of finite element is increased above the openings. Higher order finite elements are also used at the critical sections where stress-concentrations or stress gradients are expected to be high. For the nonlinear analysis of 5-story case study, first two story walls are modeled with finer finite element mesh. The minimum amount of steel percentage taken in the analyses was 0.4% of the section area in accordance with the specification (ACI 318-89). In the nonlinear analyses part, because of the relatively small deflections occur in such tubular structures, any geometric nonlinearity is disregarded. For that reason, only material nonlinearity is considered. The material properties of the steel and concrete, used in the nonlinear analysis are summarized in Table 1. The importance of concrete cracking is handled as smeared cracks that have the rotating capability as well as closing and reopening potentials.

**Table 1. Material Properties for Concrete and Steel**

| Concrete | Steel | Rod Element |
|---|---|---|
| $E = 2.14 \times 10^6$ t/m$^2$ | $E = 2 \times 10^7$ t/m$^2$ | $E = 2 \times 10^7$ t/m$^2$ |
| $v = 0.2$ | $v = 0.0$ | $v = 0.3$ |
| $f_{tu} / f_{cu} = 0.06823$ | $Q_s$(top) = 0.2% in both direction | $A_s = 0.000226$ m$^2$ (openings) |
| $f_{c28} = 1925$ t/m$^2$ | $Q_s$(bot.) = 0.2% in both direction | $A_s = 0.000452$ m$^2$ (at edges) |
| | fy = 22000 t/m$^2$ | fy = 22000 t/m$^2$ |

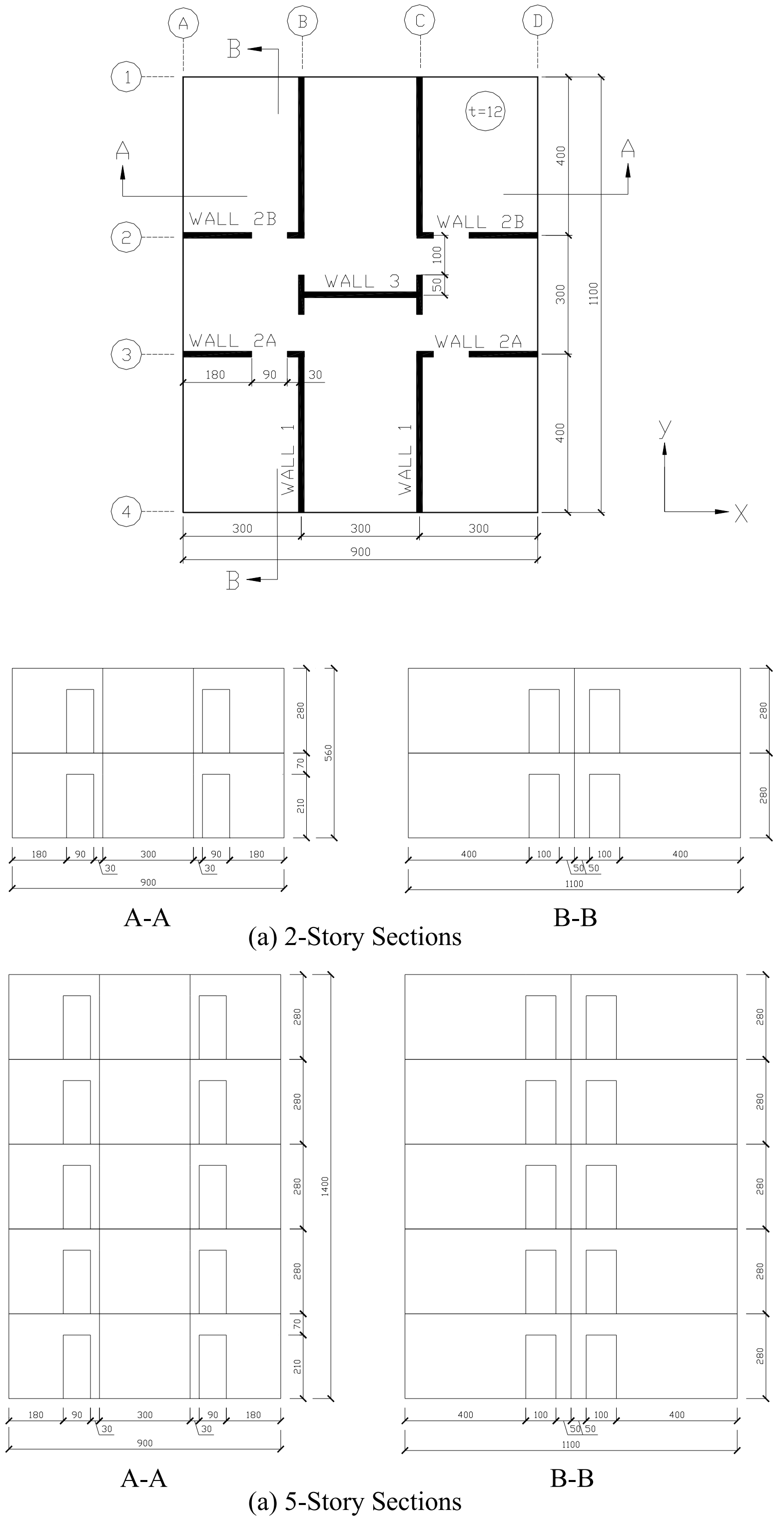



Figure 2. Typical plan view and sections for 2 and 5 story buildings (units are in cm)

3D computer model given in Figure 3 took this form as a result of the intensive mesh and convergence studies involving the finite element modeling.

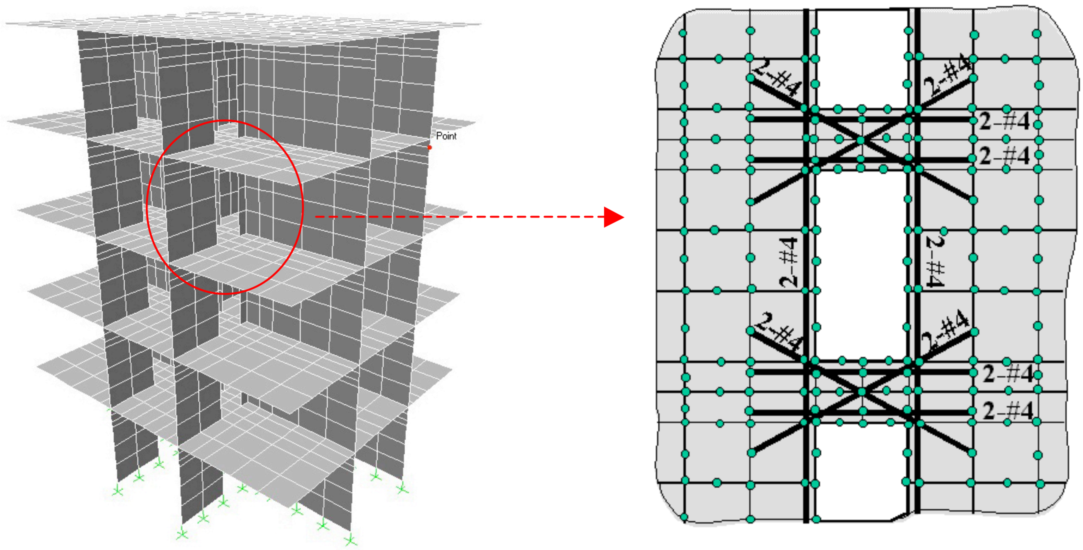



Figure 3. (a) Finite element modeling of shear-walls and slabs, (b) discreet and smeared reinforcement modeling

## 1. Nonlinear Isoparametric Shell Element

The nonlinear isoparametric shell element, 'CBAL' (Balkaya and Schnobrich, 1993) which provides the capability of a variable edge order and arbitrarily placed movable edge nodes (to consider the location and amount of main reinforcement near the edges and around openings as discrete reinforcement) was used for modeling. This element, 'CBAL', was adapted to (POLO-FINITE) and analyses were performed by using this nonlinear finite element analysis program.

Shifting of the edge nodes of the physical element normally causes a node mapping distortion if a standard parent element is used. Because unequally spaced nodes results in an unacceptable distortion, some correction techniques for eliminating that distortion are applied using a special mapping between parent element and the physical element (Figure 4). For this purpose, the standard shape functions and their derivatives normally used for isoparametric elements are modified for movable edge nodes (El-Mezaini and Citipitioglu, 1991). The capability of moving any of the element's edge nodes to any location along an edge allows these edge nodes to be placed in the proper position that, these nodes serve as end nodes for the cover of the main discrete reinforcement. This provides a robust stiffness contribution coming from the main reinforcement.

Besides arbitrarily movable edge nodes, the advantage of a variable edge order in the finite element modeling can be put to good use when the stress gradients are expected to be high. This allows increasing the order of the displacement field in such areas as around openings and in the vicinity of the slab-wall connections. The matching of displacement fields between different order finite elements can be adjusted to retain compatibility along their common edge. Additionally, the use of variable order finite elements can reduce the capacity and computational time required to reach a solution while retaining the level of accuracy deemed desirable. This approach is especially important in the case of a nonlinear analysis of a multistory structure which is being modeled using a three dimensional model.

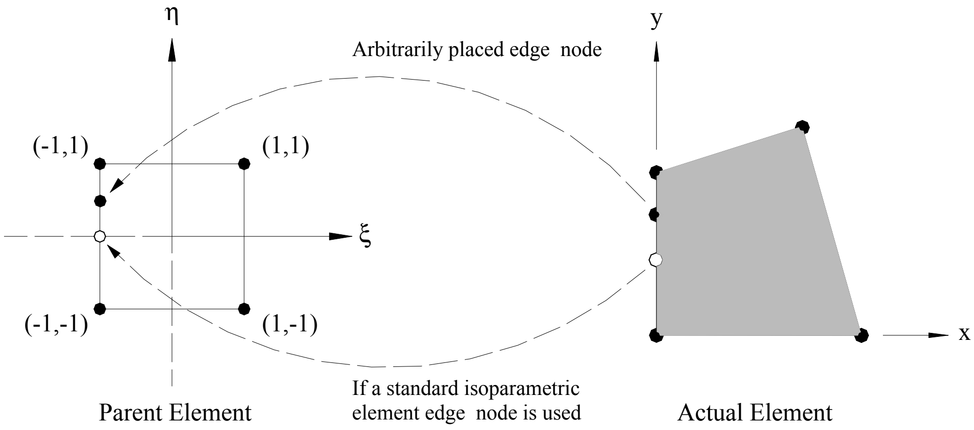
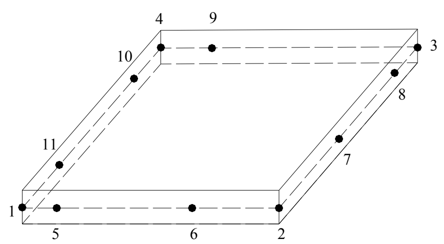


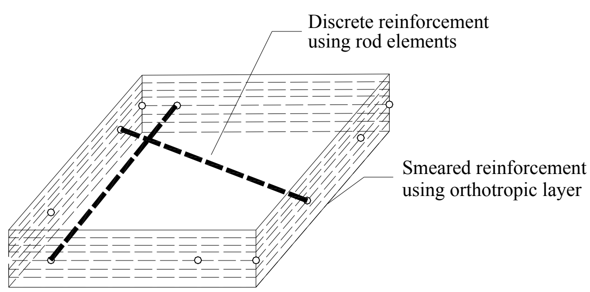

Figure 4. One-to-one mapping between the nodes of parent and actual element

In this study, nonlinearity is considered only in the form of material nonlinearity, the shape of the stress-strain curve, tension stiffening, cracking including the possibility of that crack along with opening and closing capability (Milford and Schnobrich 1985, Gallegos and Schnobrich, 1988) are all taken into account. The effects of out of plane bending of the walls and slab-wall interaction are examined through the use of shell elements. When linear edge orders are used instead of higher order edges, the wall displacement field is not sufficient to transfer the forces from the floor to the wall at the slab-wall connection. In the development of most shell elements, the normal rotation is not really taken into consideration; it is usually taken with a dummy value assigned to avoid numerical problems when only coplanar elements exist at a node. This neglect results in a violation of displacement compatibility between nodal points along the common edge. As a correction, drilling nodes could be introduced. However this correction is complicated and violation is mesh dependent. Thus it is deemed easier to use a refined grid rather than to modify the element definitions.

In the light of all this information, the developed there dimensional nonlinear isoparametric shell element having variable edge order and arbitrarily placed edge nodes is illustrated in Figure 5.




(a) Node numbering




(b) Reinforcement modeling

Figure 5. Nonlinear isoparametric shell element

## 2. Reinforcement Modeling

Finite element modeling of the reinforcement in a reinforced concrete member can be handled in a number of different ways. The steel can be included as discrete steel elements, as individual steel units embedded in a concrete element or as a smeared layer of steel sandwiched within the concrete layers. In the discrete model, reinforcing bars are modeled using special rod elements located between prescribed element edge nodes. In general, these are two nodded elements, which will have compatibility discontinuities with the adjacent concrete. Higher order elements can be used along edges of comparable order concrete elements. If higher order element is desired with the steel placed to pass through the interior of an element, an embedded steel element must be used. Embedded elements use the shape functions for the concrete element and evaluate the integral of $B^tDB$ along the path of the steel reinforcement. Stiffness matrices of these embedded steel elements then involve all concrete element nodes. Smeared reinforcement is the easiest to implement and transfers the effect of the steel directly into concrete elements.

In this study, nonlinear rod elements are used around the openings and near the edges as discrete rebars, which have elasto-plastic stress-strain characteristics. By using the special isoparametric elements, this discrete steel can be included while locating the bars with the proper cover requirements. With a two nodded rod, the stiffness contributions result only to its end nodes. However bond is neglected due to the incompatible nature of the two displacement fields defining the deformations of the steel and concrete. A smeared steel model is used for the general reinforcement. It is treated as an equivalent uniaxial layer of material at the appropriate depth and smeared out over the element as several orthotropic layers. Steel is taken into account in both faces and in both directions considering the minimum steel ratio and cover. The reinforcement modeling used in the analyses is given in Figure 3.

## 3. Crack Modeling

Cracks in concrete can be modeled as a smeared or as a discrete crack model. Gerstle (1981) reported that two approaches could be reliably used in the modeling of cracking in reinforced concrete. First approach is “does not try to predict crack spacing or crack width, the effect of the cracks is ‘smeared’ over the entire element”. Second approach looks “at each single crack ‘under a magnifying glass’; the shear transfer and dilation are expressed quantitatively as properties of a finite element which models the crack” this requires knowledge of the location and extent of each crack (Vecchio and Collins, 1982). In case of structures affected with shear, the possibility of main cracks developing at the base can have a major influence on the response characteristics (Okamura and Maekawa, 1990) and so needs to be modeled in the form of a discrete crack model in order to account the influence of changing crack openings with the changing of displacement geometry.

Within the smeared crack modeling, there are several options. They can be modeled either as a fixed-crack model or as a rotational-crack model. In most of the finite element analysis of reinforced concrete structures, crack directions are assumed as fixed; it means when they take the crack form, they remain open. However, this model leads to crack directions that can be inconsistent with the limit state (Gupta and Akbar, 1984). The change in the crack direction and the consequential change in direction of the maximum stiffness were clearly observed in the experiments of Vecchio and Collins

(1986). Therefore, the need for an algorithm that accounts this rotating crack effect is inevitable (Hu and Schnobrich, 1990).

The rotating crack concept for finite element analysis of reinforced concrete was first introduced by Cope and Rao (1977). In rotating crack models, cracks are assumed to form orthogonal to the direction of either the principal stresses of the principal strains. Depending on which variable is chosen these give a stress-rotating crack model or a strain-rotating crack model. This rotating crack concept has been further extended by Gupta and Akbar (1984) by obtaining the crack tangent stiffness matrix as the sum of the conventional tangent constitutive matrix for cracked concrete, plus a contribution that represent the effects of the possible changes in crack direction. This model has been further modified by Milford and Schnobrich in 1985 by considering the nonlinearity of concrete on compression while including tensile stiffening and shear retention for the cracked condition. In general, rotating crack models represent the actual behavior more accurately (Milford and Schnobrich, 1985). The constitutive matrix used in this study has been derived by Gallegos and Schnobrich (1988).

## Capacity Spectrum Analysis

### 1. Capacity Analysis of 2-Story Case Study

Pushover analysis procedure employing the capacity spectrum method as outlined in ATC-40 (14) was performed on the subject buildings. Per the procedure, the structure was loaded first with vertical gravity loads then pushed with incrementally increased static equivalent earthquake loads until the specified level of roof drift is reached. These code bases calculated loads are applied uniformly to the story levels, as shown in Figure 6 for 3D modeling. The 2-story structure was also modeled as a 2D system considering just the main walls (Section B-B) as presented in Figure 2.

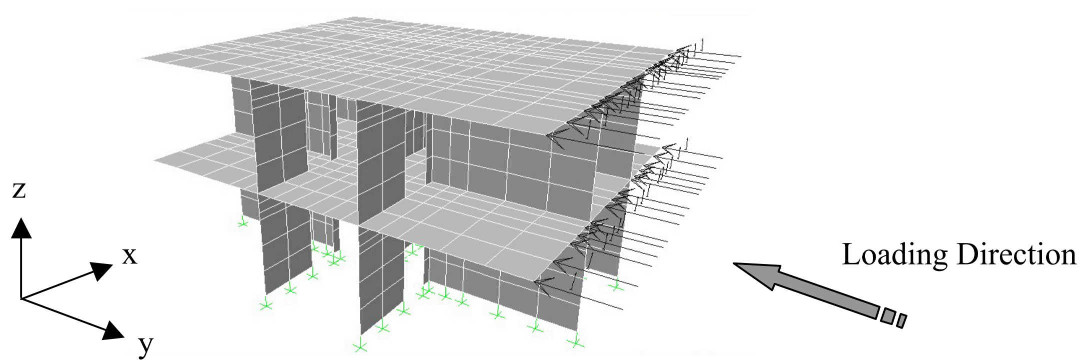



Figure 6. Distribution of uniformly applied lateral loads along the story levels

The shape of the applied lateral load should be selected on the light of anticipated changes in inertia forces as the structure moves from the elastic to the plastic phases. Ideally, this shape should be modified with the changes in inertia forces during the actual earthquake. These changes mainly depend on the characteristics of both the record and the structure. Several trials (15,16) have been made to permit of changes in inertia forces with the level of inelasticity through the use of adaptive load patterns. The underlying approach of this technique is to redistribute the lateral load shape with the extent of inelastic deformations. The load shape is suggested to be redistributed according to global displacement shape, the level of story shear demands or a combination of mode shapes obtained from secant stiffnesses. This redistribution is performed at each time step, which leads to a substantial increase in the computational effort (17). For that reason, variable load distribution option may be appropriate for special and long period structures, despite that, eminence of this technique has not been confirmed yet (18,19). It is also worth mentioning that the NEHRP (FEMA-273) guidelines recommend utilizing fixed load patterns with at least two load profiles. The first shape should be the uniform load distribution and the other is the code profile or the load shape obtained from multi-modal analyses. The code lateral load is allowed if more than 75% of the total mass participates in the predominant mode. Since the results of our previous eigenvalue analysis results satisfying this former condition, lateral loads are introduced according to equivalent earthquake procedure.

Torsion is another important issue that should be considered during the analysis of tunnel form buildings. Studies show that due to construction limitations of tunnel form technique, distribution of shear walls may result in torsional disturbance in the natural vibration mode (21). The acceptable approach for considering the effects of torsion for the development of capacity curves is given in ATC-40. In our study, torsion appeared in the first mode of the model structures, which required modifications in the capacity curves according to aforementioned approach. The resulting modified capacity curves for 2D and 3D analysis of 2-story case study as a result of loading in y-direction are shown in Figure 7. Deflected shapes and extent of crack development are given in Figures 8 and 9, respectively. These figures correspond to the last loading step of pushover analysis where excessive crack development at the base of shear walls did not yield any more inelastic deformation. The actual reason behind this point is due to the difficulties in obtaining clear failure mechanism, especially when these structures are modeled three dimensionally by using shell elements. Since these structures constitute only shear walls and slabs as load carrying and transferring members, the behavior of this combination is different than that of conventional beam-column frame type, which leads more complications to locate plastic hinges in shear walls.

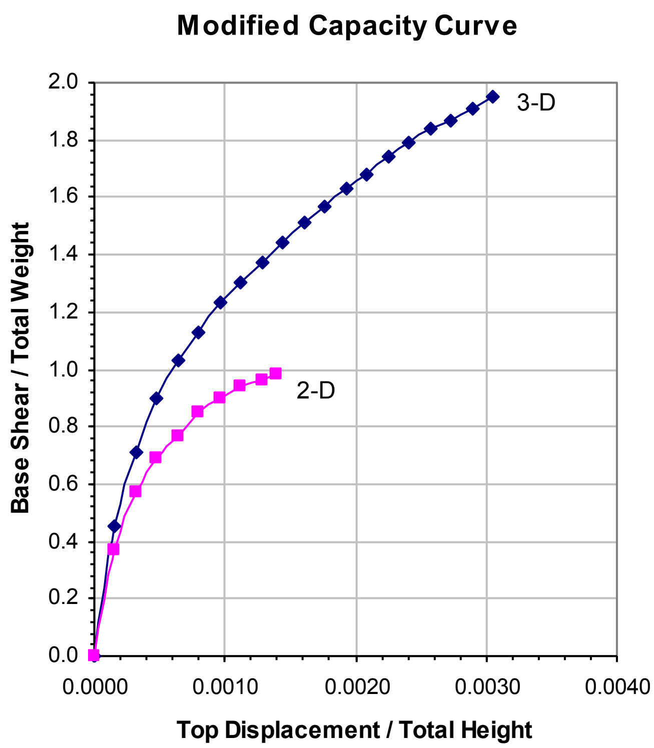
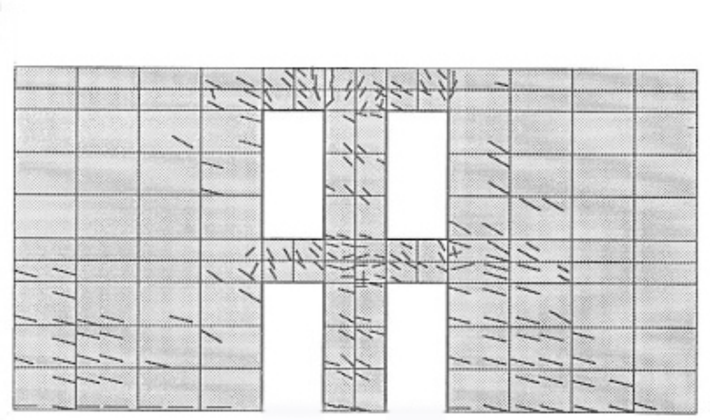
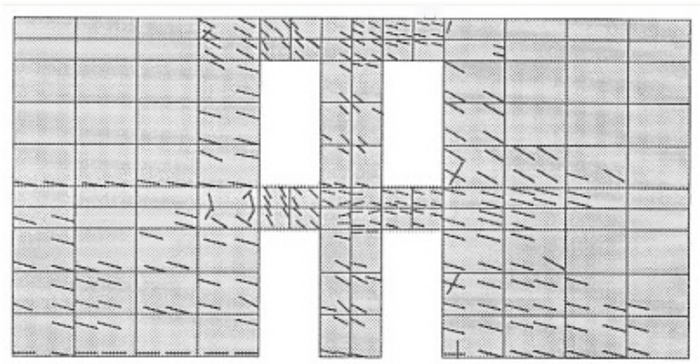


Figure 7. M … y building

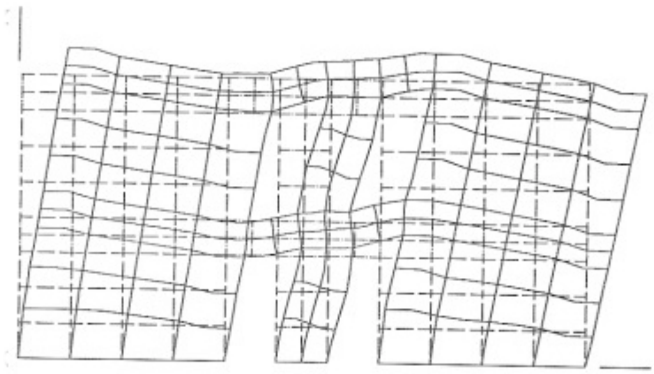


(a) 2D model crack patterns (Section B-B)

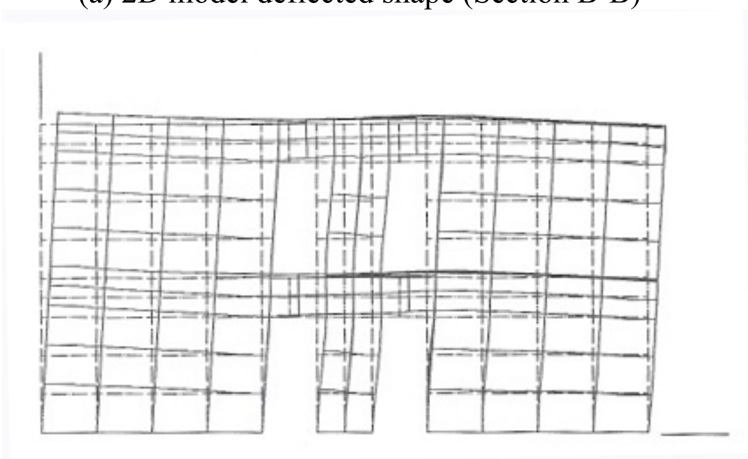


(b) 3D model crack patterns (Section B-B)

Figure 8. Crack patterns for *Wall 1* in 3D and 2D models of 2-story building



(a) 2D model deflected shape (Section B-B)



(b) 3D model deflected shape (Section B-B)

Figure 9. Deflected shapes for *Wall 1*in 2D and 3D models for 2-story building

From the deflected shape observed for 2-story building during applied load steps, the behavior of structure is dominated by in-plane and membrane forces and so is rather rigid compared to a more flexible flexural behavior. The base moments and resultant forces were calculated considering couple walls to observe 3D behavior. Static equilibrium was also checked during the loading steps. For the sake of brevity, only representative results of applied pushover analysis are given in this paper.

### 2. Capacity Analysis of 5-Story Case Study

The same plan and sections that were applied to 2-story model are used to generate the 5-story model. Similar 2D and 3D modeling procedures are also followed for this case study and obtained capacity curves are presented in Figure 10, the global crack patterns are marked on Figure 11. In this system, global yield occurs by the yielding of the shear-wall at the base and the connection around openings. A combination of distributed shear-wall mechanism and a story mechanism lead to the collapse stage accompanied with considerable deformation. Actually, the system behavior is completely controlled by the symmetrically distributed shear-walls. 5-story case study provides enhanced deformation capacity.

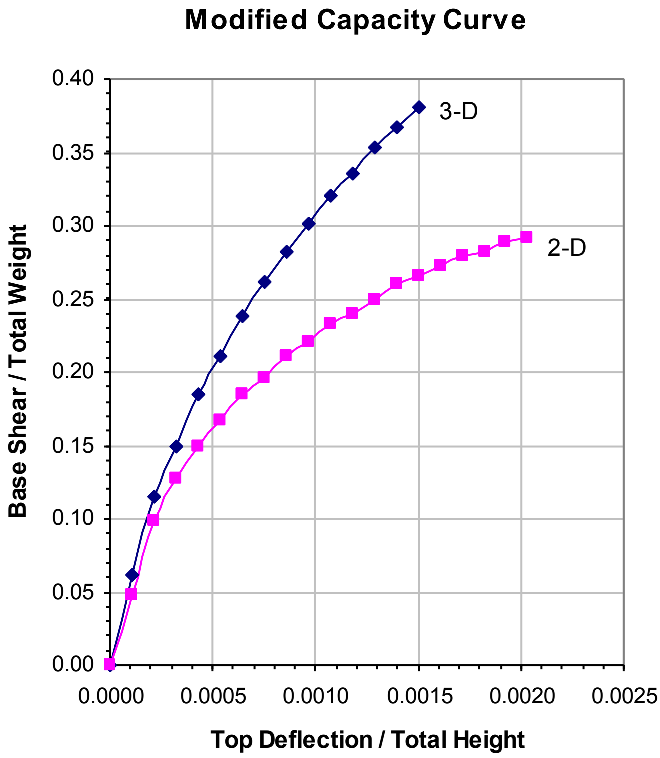


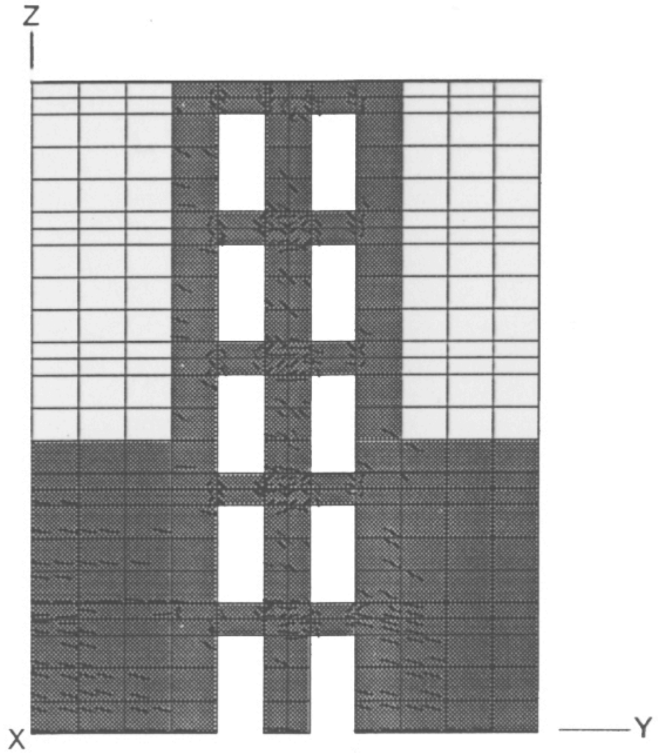

Figure 10. Modified capacity curves for 3D and 2D models of 5-story building

## Performance Evaluation with Capacity Spectrum Method (CSM)

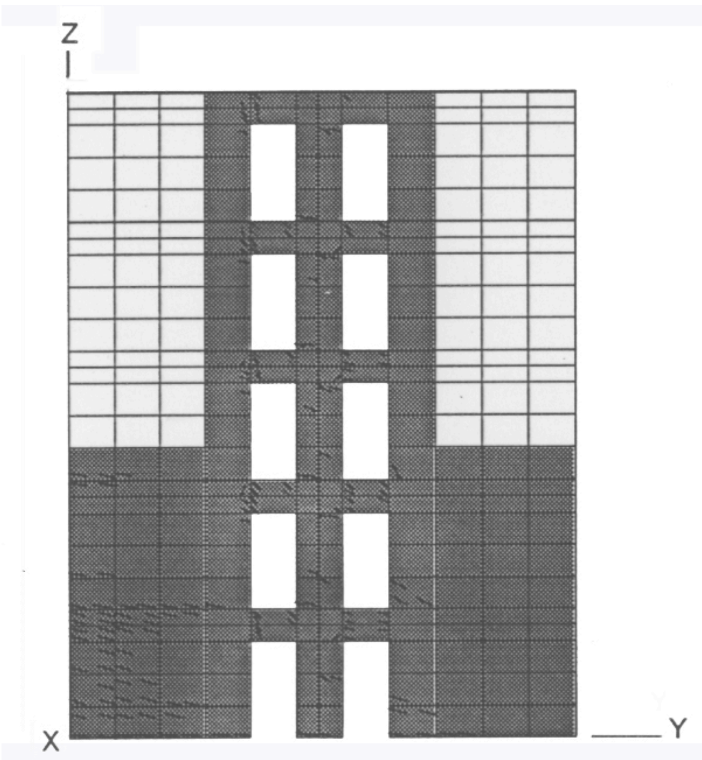
The capacity spectrum is assumed to uniquely define the structural capacity irrespective of the earthquake ground motion input. However, in order to reach a comparable conclusion about the expected demand of structures under design earthquakes levels, the obtained capacity curves should be plotted on the same format with selected demand spectrum. This is a general trend followed for performance evaluations. Hereby, the demand curve is represented by earthquake response spectra. Generally, 5 percent damped response spectrum is used to represent the demand when the structure is responding linearly-elastic (LERS). In this study, the capacity curve is converted to the acceleration displacement response spectrum format (ADRS) by the procedure outlined in ATC-40. This procedure requires making adjustments on the capacity curve by the modal mass coefficient and modal participation factor of the first natural mode of the building. The effective vibration periods of the 2-story and 5-story buildings are obtained from Eigenvalue analysis as 0.0726 and 0.23 seconds, respectively. The 2-story and 5-story buildings pushed to roughly 1.71 and 2.10 cm of displacement at the roof level as a result of applied 3D analysis. Structural behavior type is selected as *Type A* for both cases. The obtained values of modal participation factors ($PF_{RF}$) and effective mass coefficients ($\alpha_m$) are 1.30 and 0.89 for 2-story and 1.38 and 0.76 for 5-story models, respectively. Seismic demand is determined in accordance with the current Turkish Seismic Code. Corresponding seismic demand and capacity spectra of the buildings are shown in Figures 12 and 14 for 2-story and 5-story buildings in the ADRS format.

2-Story building possesses an energy dissipation capacity at the ultimate stage equivalent to 28.9 percent viscous damping ($a_y$=1.22g, $d_y$=0.17cm, $a_p$=2.28g, $d_p$=1.32cm) for which the reduced demand spectrum intersect with its capacity spectrum at smaller spectral displacement. The energy dissipation capacity of the 5-story building is less than that of first one, which has 24.6 percent viscous damping ($a_y$=0.31g, $d_y$=0.41cm, $a_p$=0.51, $d_p$=1.52). These results verify that the case buildings are capable of satisfying the code requirements at the acceleration sensitive region of the code design spectra. The capacity and demand intersects at a performance point where the roof displacement to the total height is 0.0030 and 0.0015 for 2 and 5 story buildings. At this level, the building is considered as satisfying the immediate occupancy performance level described in ATC-40. By referring to Figure 14, the performance point is caught at 1.42cm ($S_d$) for 5-story building, this spectral displacement can be translated back to a roof displacement of 1.95cm ($\Delta_R = S_d \times PF\Phi_R$) and a base shear coefficient of 0.37 ($V/W = \alpha S_a$).

Generally, the design spectra are smooth in shape such as those in building codes; however, response spectra derived from actual earthquake records are irregular and contain spikes at predominant response periods. These spikes tend to fade away at higher damping values. Herein, similar CSM analysis is repeated by using the NS and EW components of 1999 $M_w$ 7.4 Kocaeli earthquake records (5% damping) and given in Figures 13 and 15. 2-Story buildings can easily reach demand spectra however, 5-story building barely exceeds yield in the case of 5% damping response spectra.




(a) 2D model crack patterns (Section B-B)




(b) 3D model crack patterns (Section B-B)

Figure 11. Crack patterns for *Wall 1* in 3D and 2D models of 5-story building

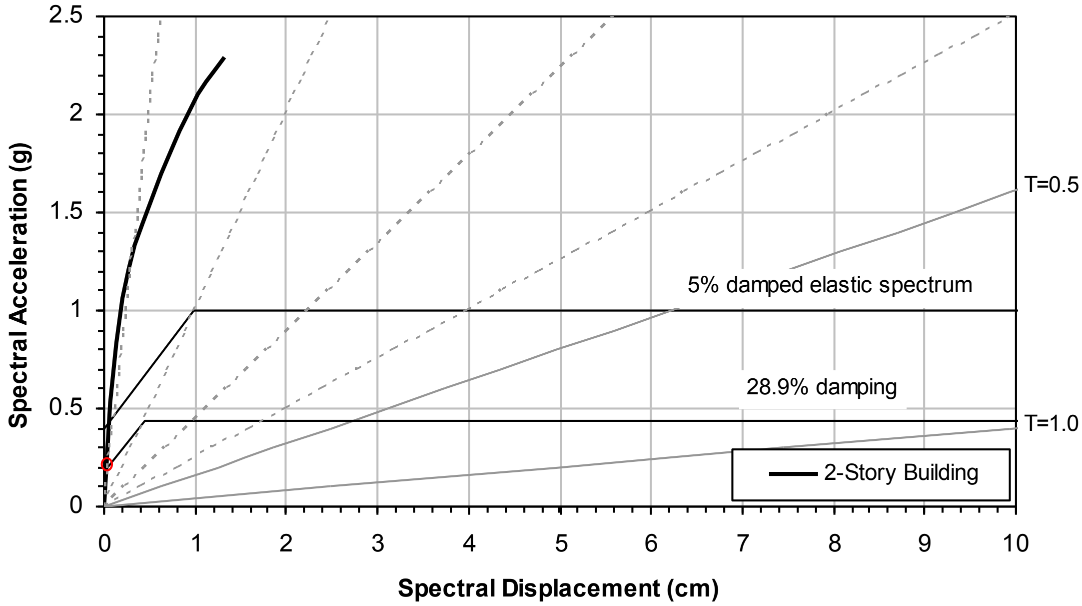



Figure 12. Application of capacity spectrum method to the 2-story building on the basis of Turkish Seismic Code

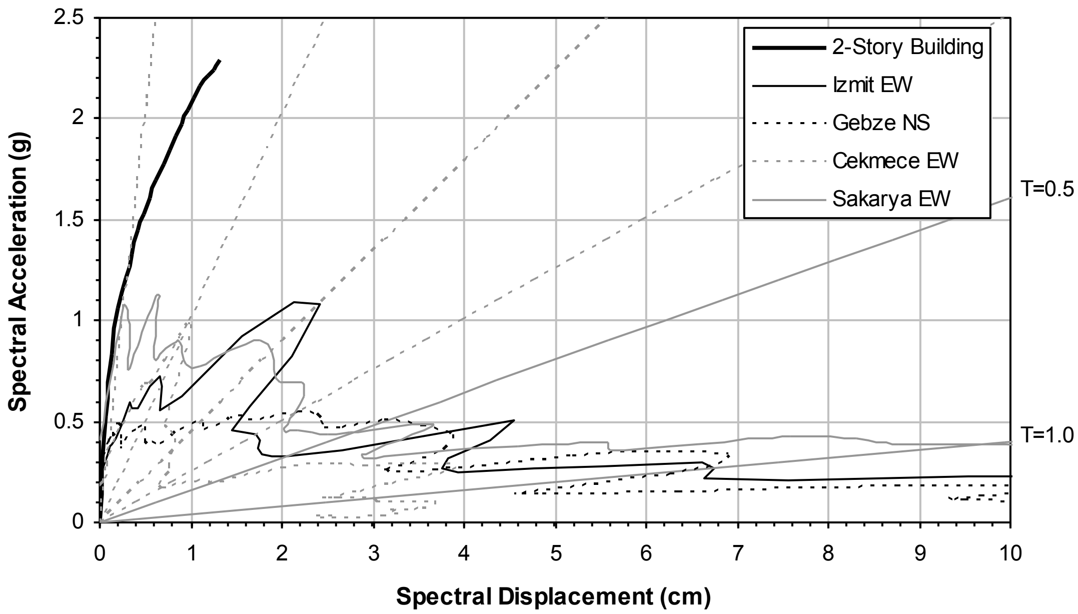



Figure 13. Application of capacity spectrum method to the 2-story building using 1999 Kocaeli earthquakes main shock records

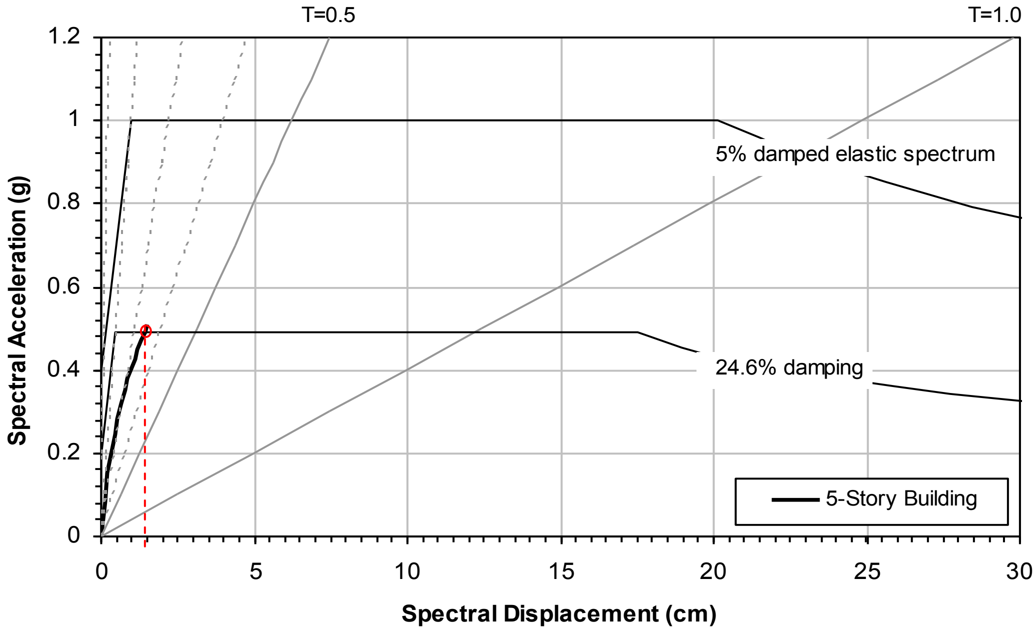



Figure 14. Application of capacity spectrum method to the 5-story building on the basis of Turkish Seismic Code design spectrum (soft soil condition)

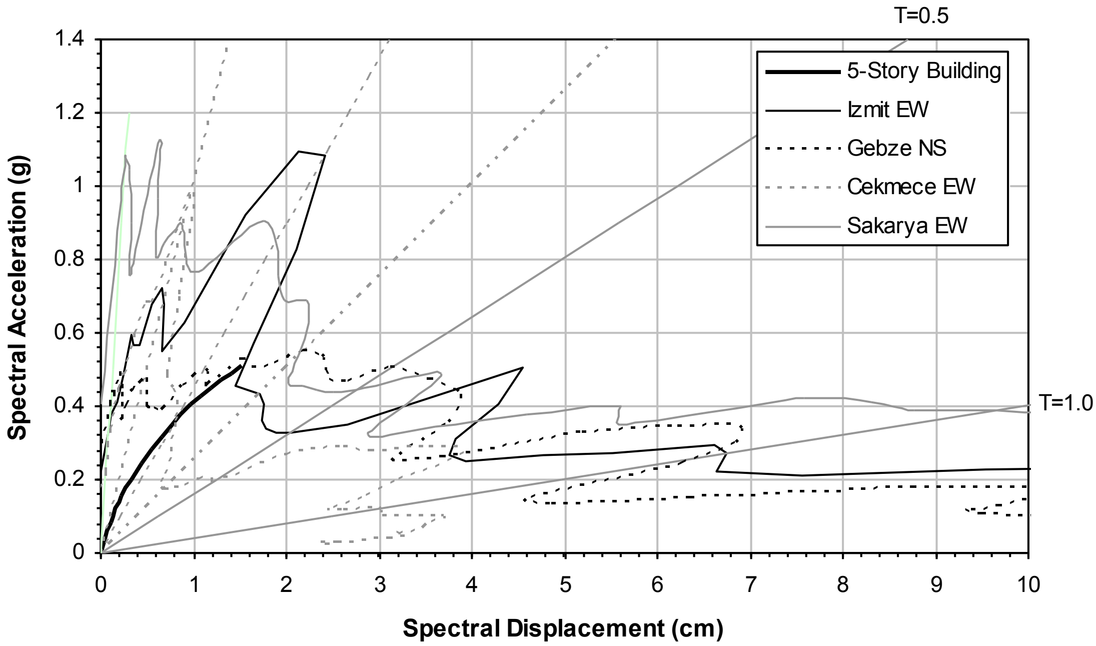



Figure 15. Application of capacity spectrum method to the 5-story building using 1999 Kocaeli earthquake main shock records (5% damped)

## 3D Effects and Tension-Compression (T/C) Coupling

The tension-compression (T/C) coupling, produced by in-plane or membrane forces in the walls, is an important force mechanism originated from wall to wall interaction (including walls with openings). In addition to wall-to-wall, wall-to-slab interaction is another issue develops due to the membrane forces in the slabs. The lateral walls form a system with in-plane walls similar to a typical T-section whose behavior through its 3D effects is like the section above the openings in the walls in the loading direction having a T-section contribution from the floor slabs as shown in Figure 16.

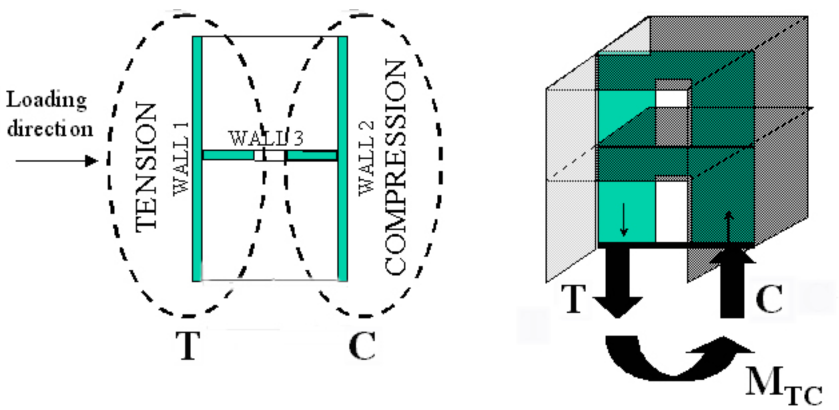



Figure 16. Slap-wall interaction due to tension and compression (T/C) coupling

It is also perceived that, effect of openings on the strength and deformation capacity of shear-wall system is different than that observed with coupling beams in frame-wall system. These differences are more evident while considering the 3D models. Due to the restraint of motion caused by existing transverse walls and slabs having continuous edge support in three dimensions, generally, no contra-flexure points occur above the openings as they do in 2D modeling. The part of the wall between the vertical openings is deflected more in the 2D models than 3D models. In case of 2D modeling, T/C coupling is weaker accomplished with transverse shear through coupling beams. With the 3D modeling, these transverse walls stiffen the section by providing additional paths for shear transfer.

The analysis of the buildings shows that, the openings introduce a strong disruption of shear flow between the walls. Shear flow plots for the 2D and 3D models of 2-story and 5-story buildings are presented in Figures 17 through 20. The general good agreement between all the analysis results gives support that, in spite of the door openings introducing a strong disruption of the shear wall flow, a considerable T/C coupling occurs between these walls. The values of maximum vertical stress at the corner of the openings are increased in the 3D model by 80 percent above the 2D model values. This positive difference is due to an increase in the computed capacity of the 3D model of 2-story case study as it continues to accept more loads than 2D model.

Due to the nature of the stress concentrations around the openings, the use of diagonal shear reinforcement (Figure 3), in addition to edge reinforcement, leads significant contribution for delaying and slowing the crack propagation. However, it is a deficiency in current codes that, they include inadequate guidelines related to reinforcement detailing around the openings of pierced shear-walls in the case of nonexistence of connection beams between these walls.

Torsion is another exceptionally important criterion appearing in the dynamic mode of those structures that should be taken into account for the design. It is to be expected that this phenomenon is the results of tunnel form construction restrictions, since part of the outside walls should be opened in order to take the formwork back after the casting process. For that reason, these buildings may behave like thin-wall-tubular structures where torsional rigidity is low. Torsional moments may cause crack propagation at the outer free edges of slab-wall connections at the floor levels.

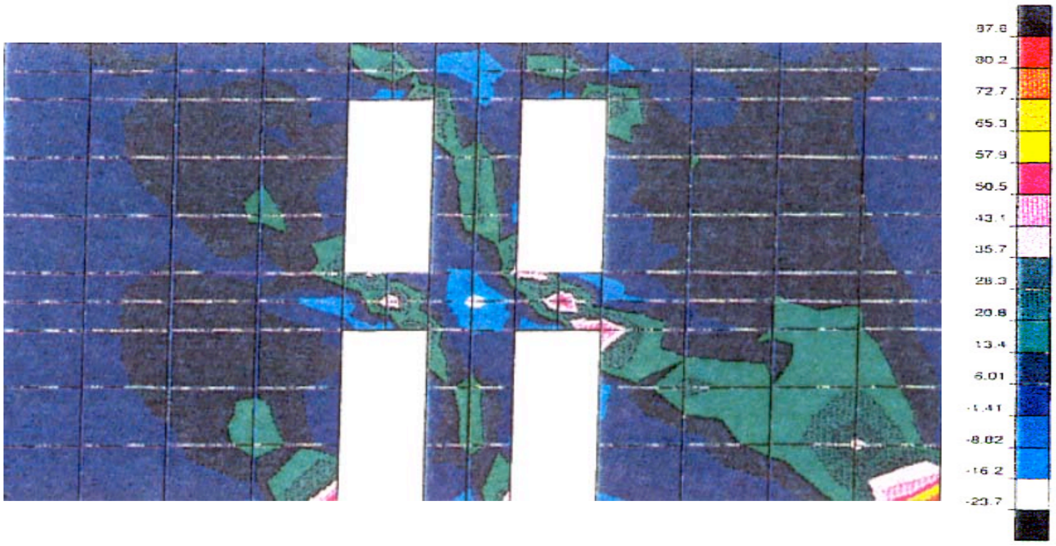



Figure 17. 2-Story 2D model, shear stress distribution around openings on *Wall 1* (Section B-B), (t/m²)

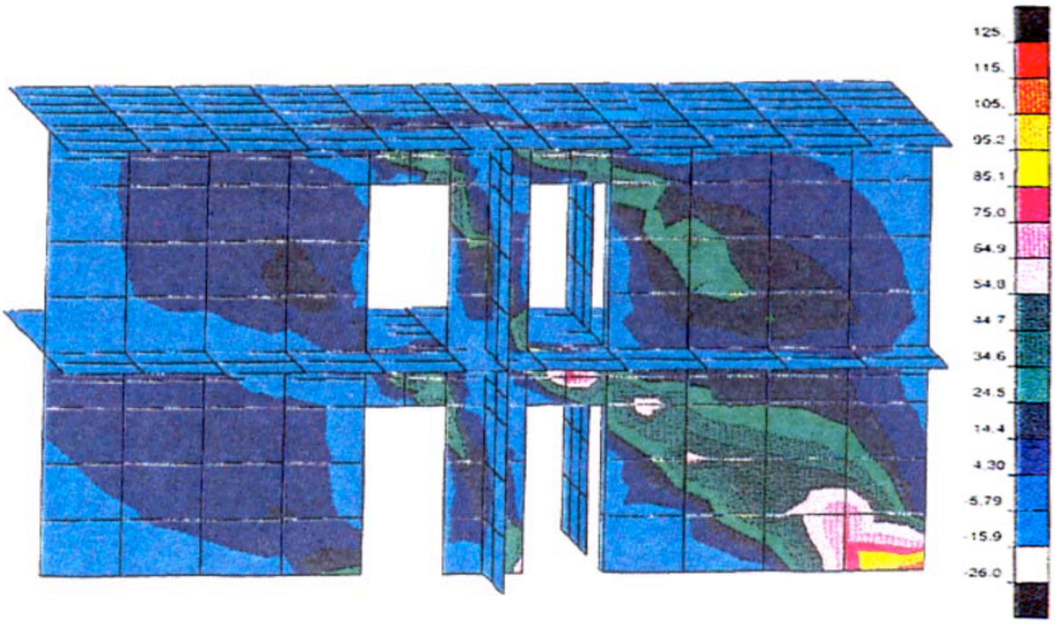



Figure 18. 2-Story 3D model, shear stress distribution around openings, (t/m²)

Considering the analytical simplicity of the linear procedures compared to the sophistication in nonlinear procedures accompanied with a large number of assumptions on modeling, and practicality of the force-based checking compared to the deformation-based acceptance criteria, linear procedures with force-based acceptance appear to be more attractive for routine applications, however, the designer should be aware of these aforementioned observed handicaps when dealing with shear-wall dominant buildings structures.

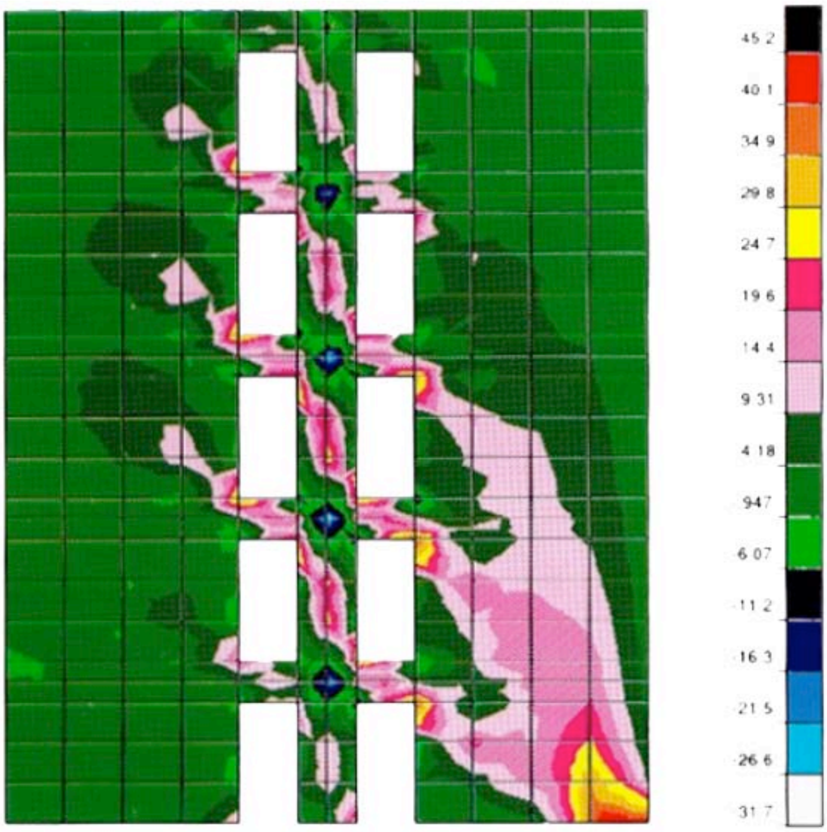



Figure 19. 5-Story, 2D model, shear stress distribution around openings on *Wall 1* (Section B-B), (t/m²)

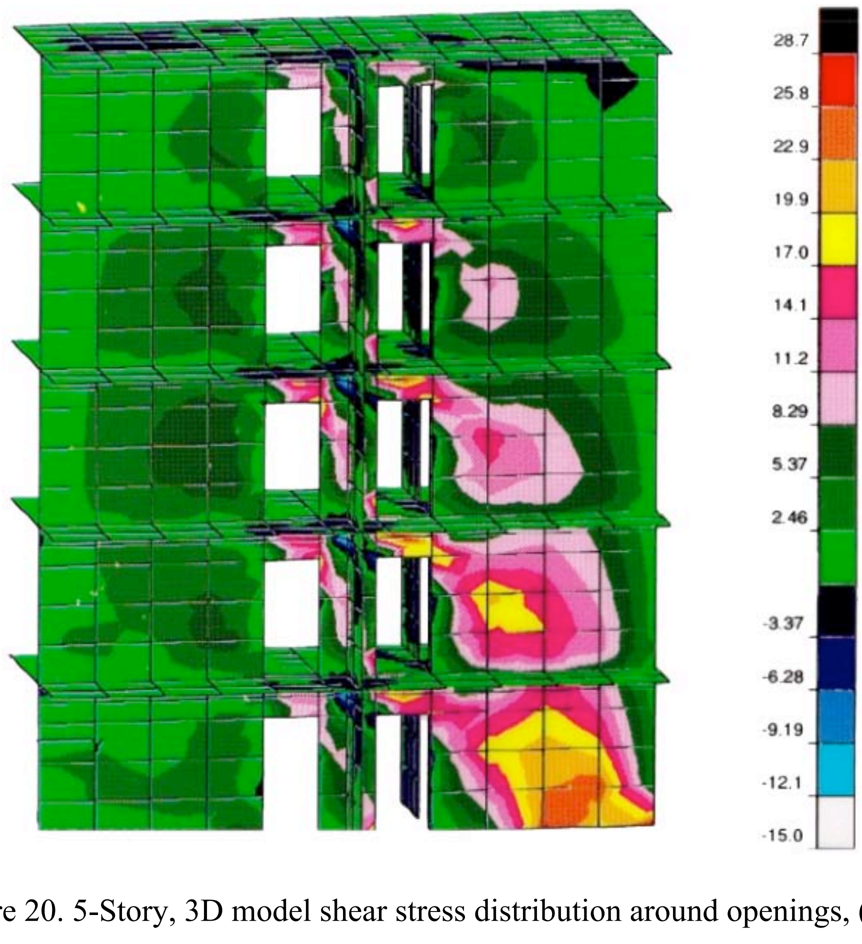



Figure 20. 5-Story, 3D model shear stress distribution around openings, (t/m²)

## Limitations and Uncertainties

There are good reasons for advocating the use of the pushover analysis to well define the seismic behavior of structures by comparing required demand with inelastic capacity, since in many cases it will provide much more relevant information than an elastic static or even dynamic analysis but it would be counterproductive to advocate this method as a general solution technique unless special care is spent for the modeling especially under dominant torsional disturbances.

It must be emphasized that the pushover analysis is approximate in nature and is based on static loading. As such it cannot represent dynamic phenomena with a large degree of accuracy. It may not detect some important deformation modes that may occur in a structure subjected to severe earthquakes, and it may exaggerate others. Limitations are also imposed by the load pattern choices. Whatever load pattern is chosen, it is likely to favor certain deformation modes that are triggered by the load pattern and miss others that are initiated and propagated by the ground. Thus, good judgment needs to be employed in selecting load patterns and in interpreting the results obtained from selected loading cases. Besides that, the pushover analysis procedure is overly simplifying. The procedure is assumed that it is possible to characterize nonlinear 3D behavior by two parameters as base shear and roof displacement. However, it is difficult to capture all possible structural variations in the vertical and plan directions of a structure with these two parameters.

Furthermore, limitations originating from nonlinear analysis program utilized are unavoidably effects the obtained results and findings. For that reason, we have faced with difficulties for obtaining clear failure mechanism when considering three dimensionally modeled shear-walls instead of referring conventional column-beam type modeling. However, the proposed shell element 'CBAL' found as useful tool in that connection to minimize these limitations and herein unpronounced uncertainties.

## Discussion and Conclusions

The applicability and accuracy of inelastic pushover analysis in predicting the seismic response of tunnel form building structures are investigated in details. Two buildings having similar plan and sections with different story levels are analyzed by utilizing 2D and 3D finite element modeling with the help of the proposed isoparametric shell element, which provided reasonable simulation of yielded locations as well as their crack patterns. Based on the large amount of information obtained, which is nonetheless far from comprehensive. It is desirable that more correlative studies using different plan configurations be performed to confirm the approach and conclusions presented in this paper. Such studies would further tune modeling parameters, which then could be used with greater confidence in response prediction analysis of shear wall dominant building structures.

The pushover analysis used as a tool in this study, if implemented with caution and good judgment, and with due consideration given to its many limitations, it will be a great improvement over presently employed elastic evaluation procedures for the design of tunnel form structures. This applies particularly to the seismic evaluation of existing structures whose element behavior cannot be evaluated in the context of presently employed global system response modification factors such as R used in current seismic provisions. It should be also noted that the proposed response modification factor in design codes is based on general consensus of engineering judgment and observed structural performance gained from the past earthquakes. The result of this study shows that inadequate information is available to justify the use of this value especially for tunnel form building type.

This paper also makes comparison between conventional 2D solutions and applied 3D analyses of two case studies and illuminates the reasons for their differences. Generally, the total resistance capacity of the three dimensionally analyzed structures is observed to be more than that of two dimensionally modeled structures.

Although software limitations and other practical considerations preclude assessment of some complex behaviors (e.g. higher mode effects), the nonlinear static pushover procedure will provide insight into structural aspects that control performance during severe earthquakes. For structures that vibrate primarily in the fundamental mode like the case studies given hereby, the pushover analysis will very likely provide good estimates of global, as well as local inelastic, deformation demands. This analysis will also expose design weaknesses that may remain hidden in an elastic analysis. Such weaknesses include excessive deformation demands, strength irregularities, overloads on critical locations such as openings and connections.

The analytical approach presented herein has the potential to help for guidance for the nonlinear 3D analysis of shear-wall dominant buildings. The technique followed in this study may be used to highlight potential weak areas in the structures to perform more accurate and economic strengthening and retrofitting studies. The experience gained from this study can also help to handle the discrepancies due to appearing torsional behavior in the dominant mode of this type of structures.

In this study, it is intended to bring the well performance of tunnel form building structures forward and to highlight their strong and weak points. It is more desirable to have detailed guidelines related to their design and construction conditions in the seismic codes and provisions in the near future.

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