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Nonlinear seismic response evaluation of tunnel form building structures

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Abstract

The occurrences of ($M_w = 7.4$) Kocaeli and ($M_w = 7.1$) Duzce earthquakes in Turkey in 1999 once again demonstrate the nondamaged and high performance conditions of RC shear wall dominant structures commonly built by using the tunnel form technique. This study presents their seismic performance evaluation based on the nonlinear pushover analyses of two case studies. The contribution of transverse walls and slab-wall interaction during the 3D action, the effects of 3D and 2D modeling on the capacity–demand relation, as well as diaphragm flexibility, torsion and damping effects were investigated. An effort was spent to develop a shell element having closing–opening and rotating crack capabilities. This study shows that the applied methodology has a considerable significance for predicting the actual capacity, failure mechanism, and evaluation of the seismic response of tunnel form buildings.

Keywords: Finite elements; Nonlinear analysis; Shear wall; Pushover analysis; Torsion; Capacity spectrum method

1. Introduction

Tunnel form buildings, having a shear wall dominant structural system, are commonly built in countries exposed to substantial seismic risk such as: Japan, Italy, Chile and Turkey. In spite of the abundance of such structures, limited research has been directed to their analysis, design and safety criteria.

A tunnel form system is composed of vertical and horizontal panels set at right angles. The construction details and typical implementation of this structural system are shown in Fig. 1. Tunnel form buildings diverge from the other conventional reinforced concrete (RC) structures due to the lack of beams and columns. These structures utilize all wall elements as primary load carrying members. Walls and slabs, having almost the same thickness, are cast in a single operation. This reduces not only the number of cold-formed joints, but also the assembly time. The simultaneous casting of walls, slabs and cross-walls results in monolithic structures, which provide high seismic performance by retarding the plastic hinge formations at the most critical locations, such as slab-wall connections and around openings.

In spite of their experienced well behavior under earthquake excitations, the current seismic provisions and codes constitute inadequate guidelines for their detailed analysis and design. On the other hand, to obtain more realistic and economical designs as well as conducting more reliable analyses of these buildings, the factors constituting more pronounced impacts on their actual seismic behavior should be considered. The 3D behavior, the diaphragm flexibility, the slab-wall interaction and the material nonlinearity are the most influencing factors in addition to the stress concentration and shear flow around the openings, the amount and location of steel reinforcement, and torsional disturbances. Our effort was spent to illuminate the importance of

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Fig. 1. Tunnel form construction and its formwork system.

these factors by performing nonlinear 3D pushover analyses on selected 5- and 2-story building structures and comparing the obtained results with that of commonly used 2D analyses. To accurately predict the nonlinear seismic response with sufficient accuracy, due care has been given to create detailed and efficient models 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 the nonlinear behavior on the member and structure levels while guaranteeing the numerical stability. In this study, two types of analyses have been performed using the finite element models. Eigenvalue analyses were conducted at the first step to determine fundamental periods and mode shapes of the models needed later to convert the obtained load deflection curves into the acceleration displacement response spectrum format (ADRS). Pushover analyses were then applied using the predefined lateral loading shapes with increasing severity. During the pushover analyses, the geometric nonlinearity was disregarded due to the formation of relatively small deformations.

The analytical modeling, assumptions and approach 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 methodology for the 3D pushover analysis of shear wall dominant buildings based on a specifically developed isoparametric shell element. This new element has a potential to reflect the actual material nonlinearity by minimizing inherent modeling limitations. Other seismic objectives, including torsional disturbance, capacity–demand relation, reinforcement detailing, damping effect, and floor flexibility were also studied.

2. Analytical model development

By way of evaluating the 3D and 2D nonlinear seismic response of tunnel form structures, 5- and 2-story reinforced concrete residential buildings were implemented as representative case studies. A detailed description of their plans and sections is presented in Fig. 2. Their structural systems are composed of solely shear walls and slabs having the 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 the slabs. In addition to their resistance to lateral loads, these distributed walls in the plan were also designed to carry gravitational loads. The shear walls were modeled as sitting on the fixed base supports. The slabs and shear walls were simulated by using finite elements having both flexural and membrane capabilities. Instead of accepting the in-plane floor stiffness to be rigid (rigid-diaphragm assumption), in-plane floor flexibility and slab-wall interaction were taken into account thereby requiring special shell element capabilities. 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. This issue is taken up again with details in the next section.

To reduce the computational time as well as the capacity associated with the 3D modeling of incorporating shell elements, a mixture of finite elements of different order was used. Higher order finite elements were utilized at the critical locations where stress concentrations or stress gradients were expected to be high. For the 5-story building, first 2-story shear walls were modeled with the finer mesh. The reinforcements were modeled as discrete or embedded according to the criticality of their locations. The minimum amount of steel percentage taken in the analyses for shear walls and slabs was 0.4%

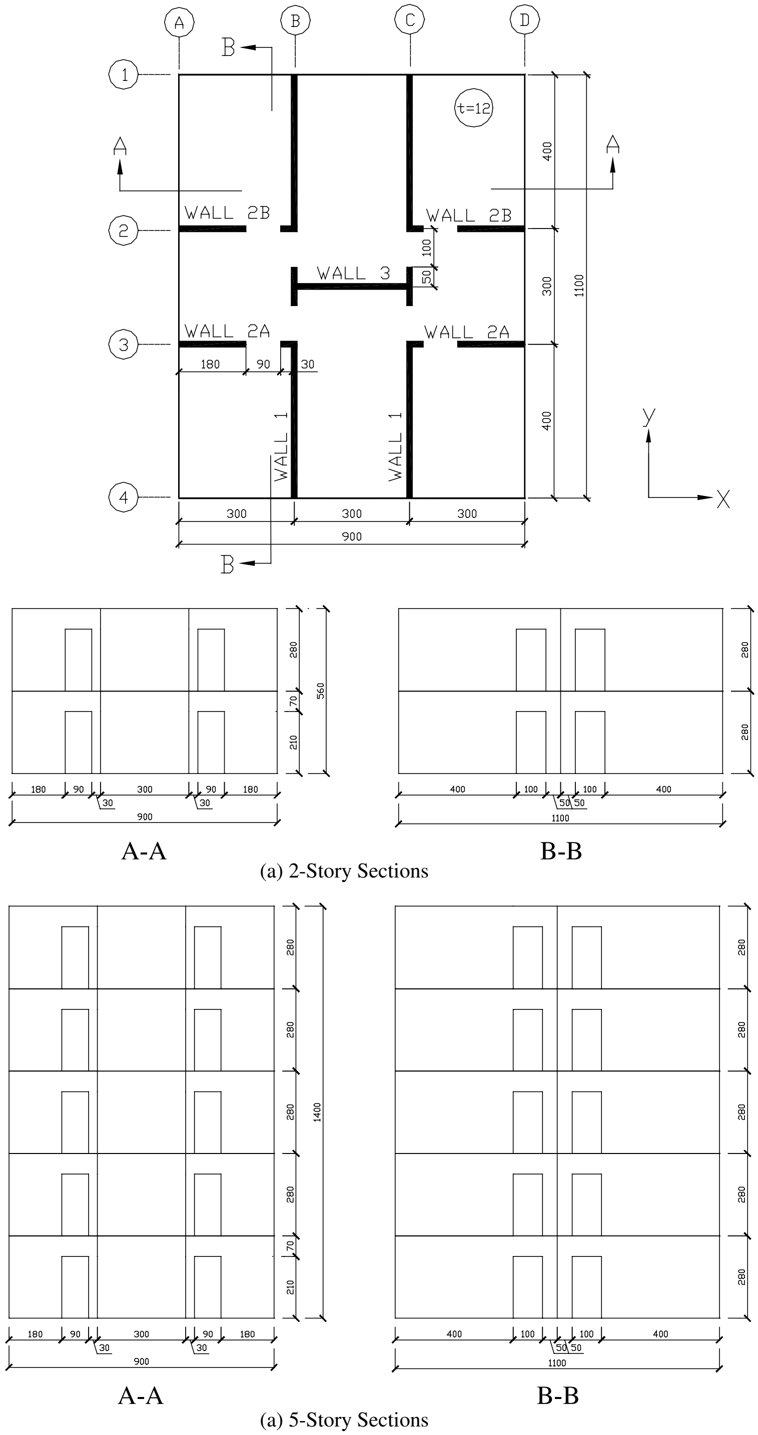


Fig. 2. Typical plan and section views for 2- and 5-story buildings (units are in cm).

of the section area in accordance with the ACI 318-95 specifications [1]. The material properties of the steel and concrete used in the nonlinear analyses are presented in Table 1.

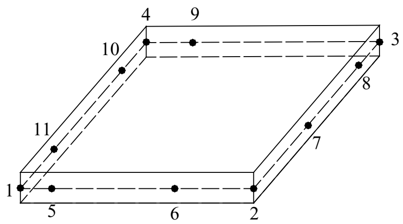
Table 1
Material properties for concrete and steel

Concrete	Steel	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 directions	$A_s = 0.000226$ m^2 (at openings)
$f_{c28} = 1925$ t/m^2	Q_s(bot.) = 0.2% in both directions	$A_s = 0.000452$ m^2 (at edges)
	$f_y = 22\,000$ t/m^2	$f_y = 22\,000$ t/m^2

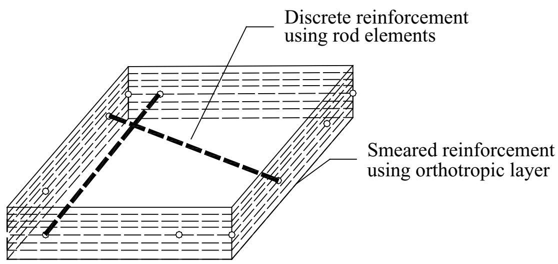
2.1. Nonlinear isoparametric shell element

A nonlinear isoparametric shell element providing 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 the openings as discrete reinforcement) was developed and adapted to POLO-FINITE [2]. The analyses were performed by using this general purpose nonlinear finite element analysis program.

In general, 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 result in an unacceptable distortion, some correction techniques for eliminating this distortion are applied using a special mapping between the parent element and physical element (Fig. 3). For that purpose, the standard shape functions and their derivatives normally used for isoparametric elements were modified for movable edge nodes [3]. 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 a proper position such that, they can 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 [4]. The matching of displacement fields between different order finite elements can be adjusted to retain compatibility along their common edge. To satisfy these conditions during finite element simulations and analyses, a 3D nonlinear isoparametric shell element having variable edge order and arbitrarily placed edge nodes was developed. The typical illustration for this element is given in Fig. 4. One of the improvements resulting from the use of variable order finite element is the reduction in the capacity and computational time required to reach a solution while retaining the level of accuracy deemed desirable. This concept is especially important in the case of a 3D nonlinear analysis of multi-story structures. In this study, the shape of the stress–strain curve, tension stiffening and the cracking having opening and closing capability [5,6] were considered in the context of material nonlinearity.



(a) Node numbering



(b) Reinforcement modeling

Fig. 4. Nonlinear isoparametric shell element.

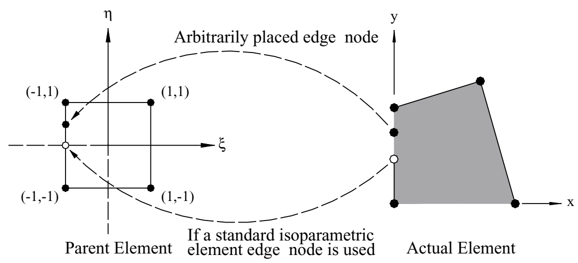


Fig. 3. One-to-one mapping between nodes of parent and actual element.

2.2. 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 considered 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 can be modeled using special rod elements located between prescribed element edge nodes. In general, these are two nodded elements which have compatibility discontinuities with the adjacent concrete unit. Higher order elements can be used along the edges of comparable order concrete elements. If a higher order element is desired with the steel placed to pass through the interior of an element, an embedded steel element should be used. The smeared reinforcement model is the easiest to implement and transfers the effects of the steel directly into the concrete elements. In this study, nonlinear rod elements were used around the openings, and discrete rebars having elasto-plastic stress–strain characteristics were utilized near the edges. By using the developed isoparametric shell element, the discrete steel could be included while locating the rebars with proper concrete cover requirements. With a two noded rod, the stiffness contributed only to its end nodes. For this case, the bond was neglected due to the incompatible nature of the two displacement fields defining the deformations of the steel and concrete. In this study, the smeared steel model was used as the general reinforcement for non-critical locations. It was treated as an equivalent uniaxial layer of the material at the appropriate depth and smeared out over the element as several orthotropic layers.

2.3. Crack modeling

Cracks in concrete can be modeled either as a smeared or a discrete crack model. Gerstle [7] reported that the two approaches could be reliably used in the modeling of cracking in reinforced concrete members. The first approach is ''... does not try to predict the crack spacing or crack width, the effect of the cracks is 'smeared' over the entire element''. The 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 the knowledge of the location and extent of the each crack [8]. In the case of structures affected with the seismic shear, the possibility of main cracks developing at the base can have a major influence on the response characteristics [9] and so needs to be modeled in the form of a discrete crack model in order to account for the influence of changing crack openings with the changing of displacement direction. 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 analyses of reinforced concrete structures, crack directions are assumed to be fixed; this means that when the crack forms, it remains open. However, this model leads to crack directions that can be inconsistent with the limit state [10]. The change in the crack direction and the consequential change in the direction of the maximum stiffness were clearly observed in the experiments of Vecchio and Collins [11]. Therefore, the need for an algorithm that accounts this rotating crack effects is obvious. In rotating crack models, cracks are assumed to form orthogonal to the direction of either the principal stresses or the principal strains. This produces a stress-rotating crack model or a strain-rotating crack model depending on which variable is chosen. This rotating crack concept has been further extended by Gupta and Akbar [10] by obtaining the crack tangent stiffness matrix as the sum of the conventional tangent constitutive matrix for the cracked concrete, plus a contribution that represents the effects of the possible changes in the crack direction. This model has been further modified by Milford and Schnobrich [5] by considering the nonlinearity of concrete on compression while including the tensile stiffening and shear retention for the cracked sections. In general, rotating crack models represent the actual behavior more accurately. The constitutive matrix used in this study has been derived by Gallegos and Schnobrich [6].

3. Capacity spectrum analysis

3.1. Case study-1: 2-story building

The pushover analysis procedure was performed by employing the capacity spectrum method as outlined in ATC-40 [12]. Per this 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 was reached. These code bases calculated lateral loads were applied uniformly to the story levels. The 2-story building was also modeled two-dimensionally considering only the main shear walls (Section B-B in Fig. 2).

During pushover analysis, the shape of the applied lateral load should be selected in 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 an actual earthquake. These changes mainly depend on the characteristics of both the record and the structure. Several trials [13,14] have been made to permit 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 [15]. For that reason, the variable load distribution option may be appropriate for special and long period structures. Despite that, the eminence of this technique has not been confirmed yet [16,17]. It is also worth mentioning that the NEHRP [18] 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 satisfied this former condition, the lateral loads were introduced according to the equivalent earthquake load procedure for both case studies.

Torsion is another exceptionally important criterion appearing in the dynamic mode of tunnel form buildings that should be taken into account during their design stage. Previous analyses show 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, tunnel form buildings behave like thin-wall-tubular structures where torsional rigidity is low [19]. Therefore, torsional moments may increase the crack propagation at the outer free edges of the slab-wall connections. The acceptable approach for considering the effects of torsion for the development of capacity curves is described in ATC-40 [12]. In our study, the appearance of torsion in the first mode of the model structures required modifications in the capacity curves as well. The resulting modified capacity curves for the 2D and 3D analysis of 2-story case study as a result of the applied lateral loading in the *y*-direction are presented in Fig. 5. This figure corresponds to the last loading step of pushover analysis where excessive crack development at the base level of shear walls did not yield any more inelastic deformation. The underlying reason is due to the difficulties in obtaining clear failure mechanisms, especially when these structures are modeled three dimensionally by using shell elements. Since their structural systems constitute only shear walls and slabs as load carrying and transferring members, the behavior of this combination is significantly different than that of conventional beam-column frame type of structures. This causes more complications in locating plastic hinge formations within the shear walls. The deflected shape observed for the 2-story building during the applied lateral loading steps showed that the behavior of structure was dominated by the in-plane and membrane forces and so was rather rigid compared to a more flexible flexural behavior. The base moments and resultant forces were calculated considering couple walls to observe the contribution of the 3D behavior. The static equilibrium was also checked during each loading steps.

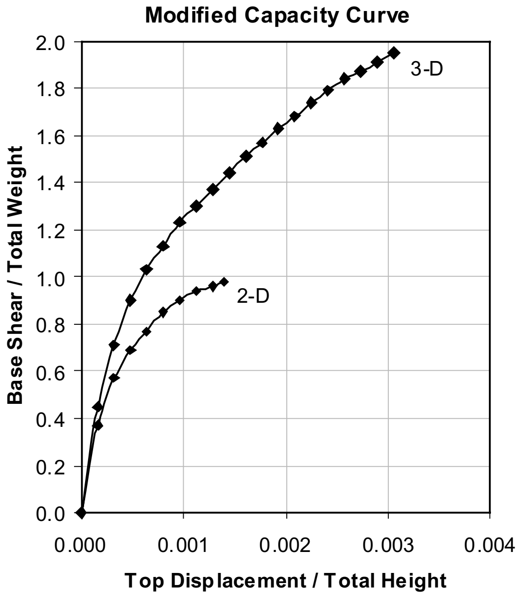


Fig. 5. Modified capacity curves for 3D and 2D models of the 2-story building.

3.2. Case study 2: 5-story building

The same plan and sections that were used for the 2-story case were applied to generate the model for the 5-story building. Similar 2D and 3D simulation and analysis procedures were followed to obtain the capacity curves presented in Fig. 6. In this case study, the global yielding occurred at the location of the shear wall bases and the connection joints around the openings. A combination of a distributed shear wall mechanism and a story mechanism lead to the collapse stage accompanying the inelastic deformation. The overall system behavior was completely controlled by the symmetrically distributed shear walls. The 5-story case study provided enhanced deformation capacity.

4. Performance evaluation with capacity spectrum method

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 earthquake levels, the obtained capacity curves should be plotted on the same format with the selected demand spectrum. This general trend has been followed for performance evaluations in recent years. Herein, the demand curve is represented by earthquake response spectra. In general, a 5% damped response spectrum is used to represent the demand when the structure is responding linearly-elastic (LERS). In

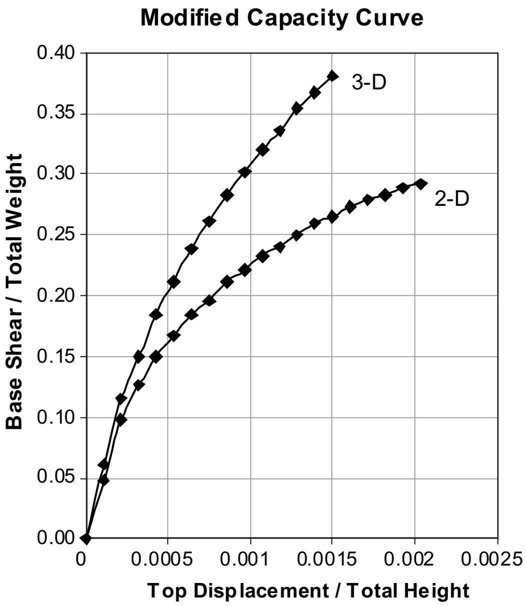


Fig. 6. Modified capacity curves for 3D and 2D models of the 5-story building.

this study, the capacity curves were converted into the ADRS for comparison with demand curves. This procedure required making adjustments on the capacity curve by the modal mass coefficient and the modal participation factor of the first natural mode of the building [12]. The effective vibration periods of the 2- and 5-story buildings obtained from the eigenvalue analysis were 0.073 and 0.23 s, respectively. The 2- and 5-story buildings were pushed to roughly 1.71 and 2.10 cm of displacement at the roof level as a result of the applied 3D analyses. The structural behavior type was selected as *Type A* for both cases as described in ATC-40 [12]. The obtained values of the modal participation factors (PF_{RF}) and the effective mass coefficients (α_m) were 1.30 and 0.89 for the 2-story, and 1.38 and 0.76 for the 5-story models. The seismic demand was determined in accordance with the current Turkish Seismic Code [20]. The corresponding seismic demand and capacity spectra are presented in ADRS format for comparison in Figs. 7 and 8 for the 2- and 5-story buildings, respectively. The 2-story building possesses an energy dissipation capacity at the ultimate stage equivalent to 28.9% viscous damping ($a_y = 1.22$ g, $d_y = 0.17$ cm, $a_p = 2.28$ g, $d_p = 1.32$ cm) for which the reduced demand spectrum intersect with its capacity spectrum at the smaller spectral displacement. The energy dissipation capacity of the 5-story building is less than that of the first one, which corresponds to 24.6% viscous damping ($a_y = 0.31$ g, $d_y = 0.41$ cm, $a_p = 0.51$ g, $d_p = 1.52$ cm). These results verify that the buildings are capable of satisfying the code requirements at the acceleration sensitive region of the design spectra. The capacity and demand intersect at a performance point where the roof displacement to the total height ratio is 0.003 and 0.0015 for the 2- and 5-story buildings, respectively. At this level, the buildings are considered to be satisfying the immediate occupancy performance level [12]. By referring to Fig. 8, the performance point is 1.42 cm (S_d) for the 5-story building. This spectral displacement can be back translated to a roof displacement of 1.95 cm ($\Delta_R = S_d PF \Phi_R$) 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

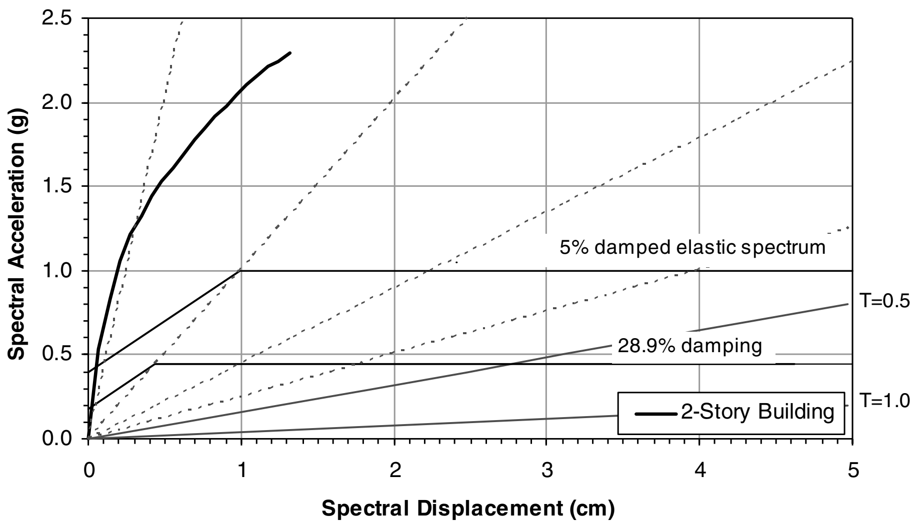


Fig. 7. Application of CSM to the 2-story building on the basis of the Turkish Seismic Code design spectrum (soft soil condition).

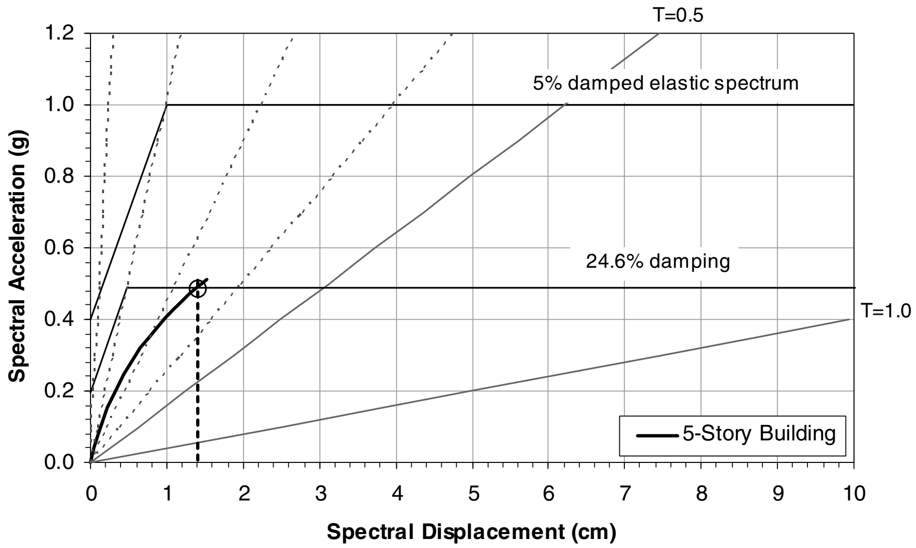


Fig. 8. Application of CSM to the 5-story building on the basis of the Turkish Seismic Code design spectrum (soft soil condition).

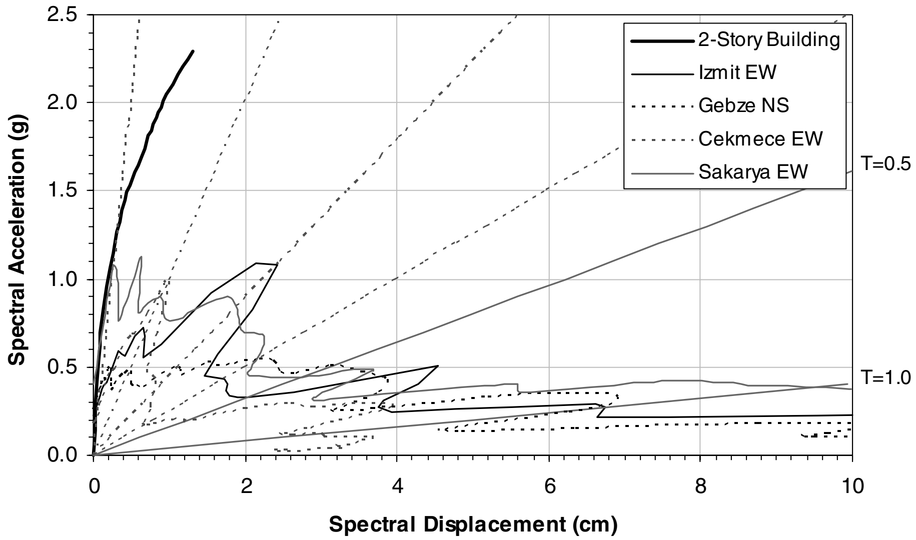


Fig. 9. Application of CSM to the 2-story building using the 1999 Kocaeli earthquakes main shock records.

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. In this study, a similar capacity spectrum method (CSM) was repeated by using the 5% damped response spectra of the NS and EW components of the 1999 ($M_w = 7.4$) Kocaeli earthquake records. The comparisons of the computed responses and capacity curves are presented in Figs. 9 and 10 for the 2- and 5-story buildings, respectively. The construction of 5% damped response curves were based on the worse soil conditions according to the Turkish Earthquake Code [20] site classifications. The 2-story building can easily reach the demand spectra, whereas, the 5-story building barely exceeds these demand curves.

5. Tension–compression coupling and 3D effects

Tension–compression (T/C) coupling, executed by in-plane or membrane forces within the shear walls, is a 3D originated force mechanism build-up in tunnel form buildings due to the combined effects of wall-to-wall (even including walls with openings) and wall-to-slab

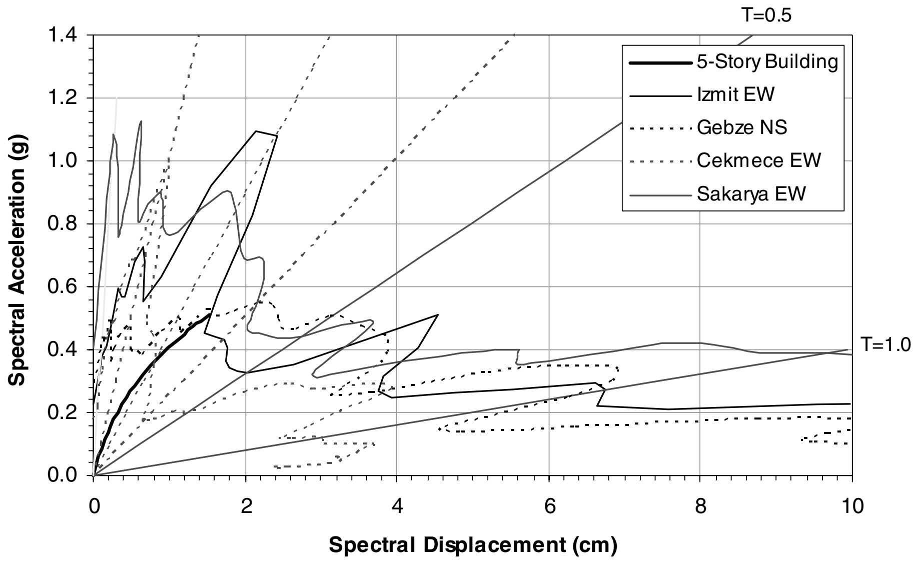


Fig. 10. Application of CSM to the 5-story building using the 1999 Kocaeli earthquake main shock records (5% damped).

interactions. This mechanism exhibits a significant contribution in controlling the capacity and seismic performance of these buildings. The basic development of the T/C coupling within the structural system is illustrated in Fig. 11.

There are other observed attributes which are worthy to mention. First are the effects of openings on the strength and deformation capacity of a shear wall system. These effects are different than those of observed in a frame-wall system resulted in the coupling of connection beams between adjacent shear walls, and these differences are more evident when the 3D behavior is considered. In general no contra-flexure points occur above the openings as they do during the 2D analysis due to the restraint of motion caused by existing transverse walls and slabs having a continuous edge support in three dimensions [21]. In this study, the part of the wall between the vertical openings was deflected more in the 2D models than the 3D models. In the 2D cases, the T/C coupling was weakly accomplished with the transverse shear through the coupling beams, whereas, the transverse walls in the 3D cases stiffened the sections by providing additional paths for the shear transfer. The local moment contribution coming from the main shear walls was not altered significantly between the 2D and 3D cases. This may be attributed to the limitation in the contribution of steel, which is set by the steel area and its yield stress. When the analysis was switched from the 2D to the 3D, the transverse walls provided an extra resistance by substantially increasing the computed lateral load capacity. The total overturning moment capacity of the 2-story building at its failure load level was found to

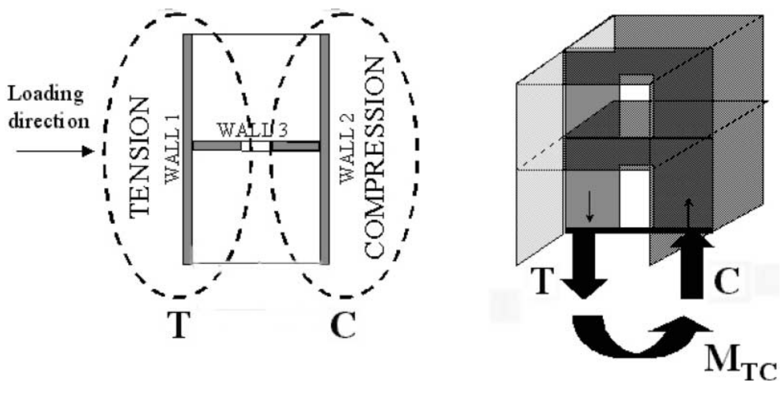


Fig. 11. Slap-wall interaction due to tension and compression (T/C) coupling.

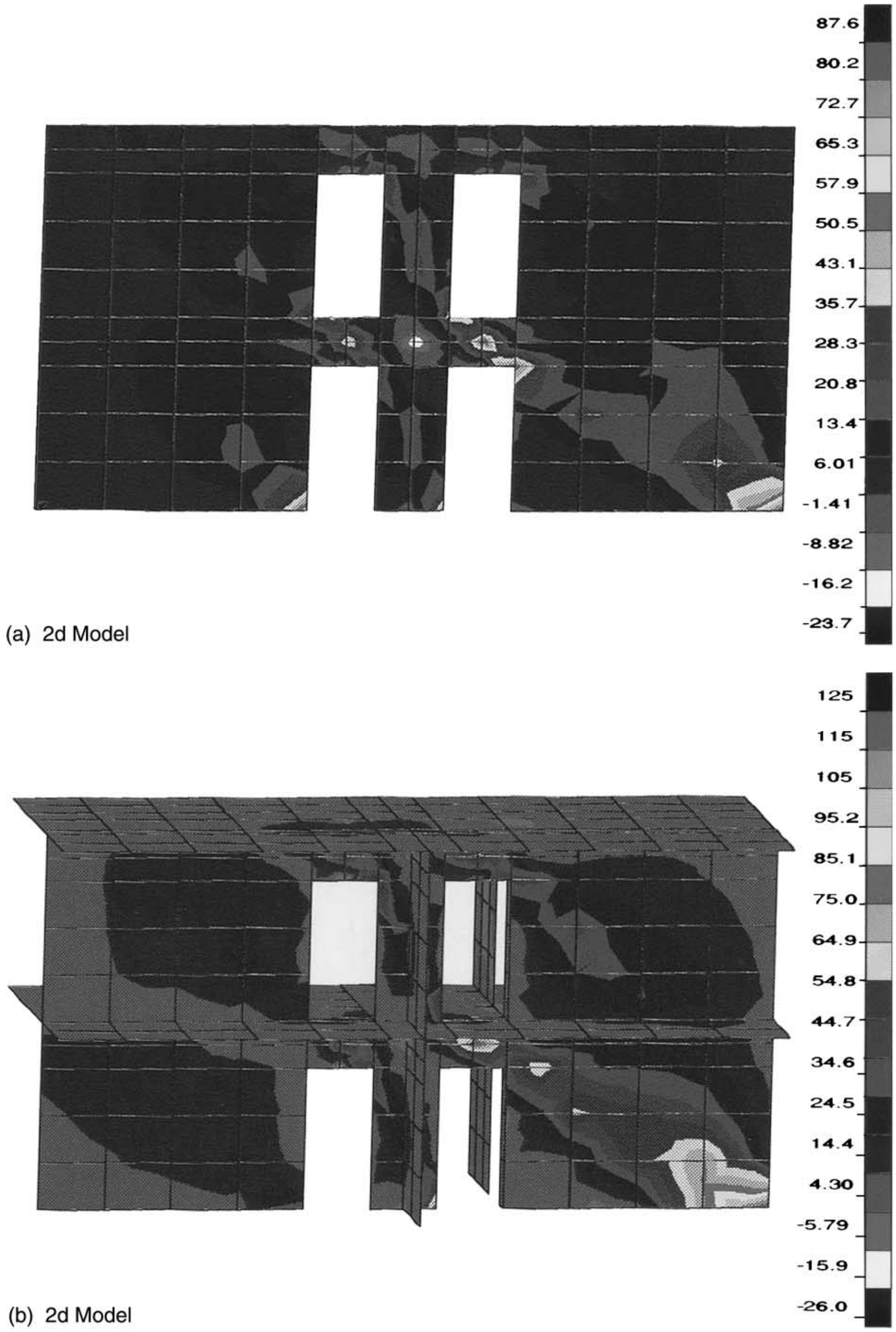


Fig. 12. Shear stress distribution around the openings of the 2-story building.

be 213.0 t.m. in the case of the 2D modeling. When the 3D model was considered at this load level, the moment capacity corresponded to 170.3 t.m., and it gradually increased up to 442 t.m. at the failure load level. This step up can be accredited to increase in the tension and compression forces that were present in the longitudinal walls and their coupling effects with the transverse walls. A similar behavior was observed for the 5-story building as well.

The overall analysis of the case studies shows that the openings introduce a strong disruption of the shear flow between the adjacent shear walls. This observed phenomenon is exhibited in Figs. 12 and 13 for the 2D and 3D models of the 2- and 5-story buildings, respectively.

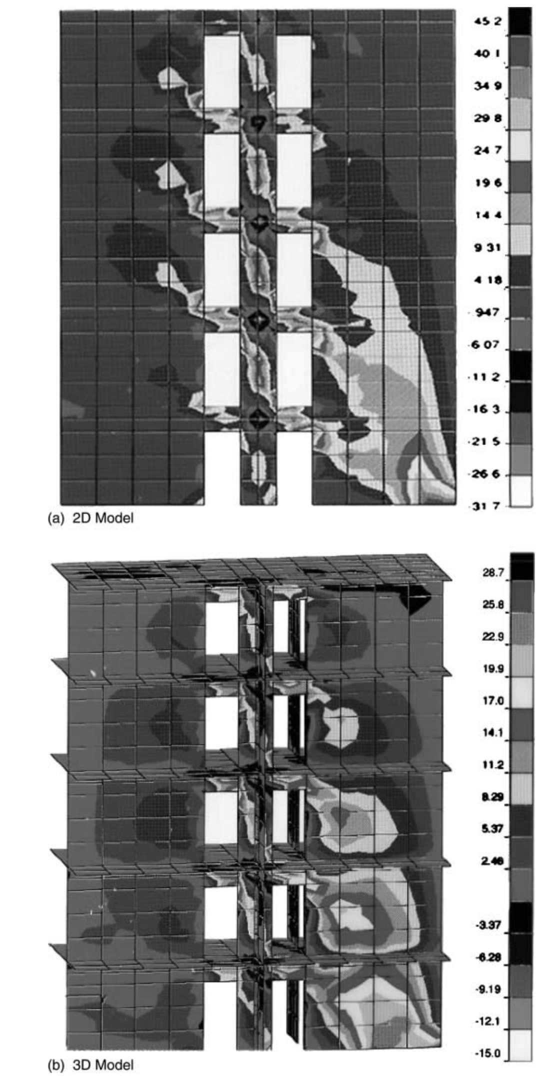


Fig. 13. Shear stress distribution around the openings of the 5-story building.

The general good agreement between all these results gives support that, despite of the door openings introducing a strong disruption of the shear flow, the effects of T/C coupling developed between the shear walls are significant. The values of the maximum vertical stress carrying capacity at the corner of the openings in the 3D model are 80% more than that of the 2D model. This positive difference for the 2-story building is due to the increase in the computed capacity of the 3D model as it continues to accept more loads.

Due to the nature of the stress concentrations around the openings, use of the diagonal shear reinforcement (Fig. 14) in addition to the edge reinforcement, leads to a significant contribution for retarding and slowing down the crack propagation. In spite of this fact, current codes and seismic provisions present inadequate guidelines related to the reinforcement detailing around the openings of pierced shear in the case of nonexistence of connection beams between these walls. The designer should be aware of all these aforementioned observed strong and weak points of tunnel form buildings during their design and analysis stages.

6. Conclusions

The applicability and accuracy of inelastic pushover analysis in predicting the seismic response of tunnel form building structures were investigated in detail. Two buildings having similar plan and sections with different story levels were analyzed by utilizing the 3D and 2D finite element models with the use of the developed isoparametric shell element. This element, having variable edge order and arbitrarily placed edge nodes, promises more accurate and reliable capacity and seismic performance evaluation for shear wall dominant buildings by providing a reasonable simulation of yield locations as well as their crack patterns.

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

The pushover analysis was used as a tool in this study. For structures that vibrate primarily in the fundamental mode like the case studies given herein, pushover analysis will very likely provide good estimates of the global, as well as the local inelastic deformation demands. It will also expose design weaknesses that may remain hidden in an elastic analysis. Such weaknesses include excessive deformation demands, strength irregularities and overloading on critical locations such as openings and connections. If this technique is implemented with caution and good judgment, and with due consideration given to its many limitations, pushover analysis will be a great improvement over presently employed elastic evaluation procedures for the evaluation of tunnel form building structures. Although software limitations and other practical considerations preclude assessment of some complex behaviors (e.g., higher mode effects), this technique still provides insight into structural aspects that control the performance during severe earthquakes. This also 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 the "R" used in current seismic provisions and codes. It should be also noted that the proposed response modification factor in the design codes is

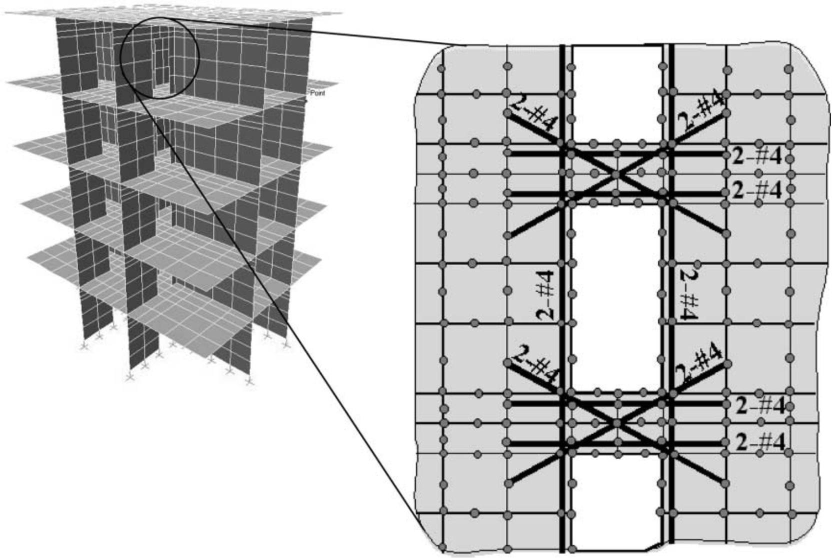


Fig. 14. Diagonal shear reinforcement detailing.

based on a general consensus of engineering judgment and observed structural performance gained from the past earthquakes. The result of this study indicates that inadequate information is available to justify the use of this value for tunnel form building structures.

The analytical approach presented herein has the potential to help in guidance for the nonlinear 3D analysis of shear wall dominant building structures. The technique followed may further be used to highlight potential weak areas hidden in the existing structures to provide more accurate and economical retrofitting solutions. The experience gained from this study may help to handle the discrepancies due to appearing torsional behavior in the dominant mode of these structures, as well.

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

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