ARI The Bulletin of the Istanbul Technical University
Communicated by Derin Orhon

VOLUME 53, NUMBER 2

Performance Based Seismic Evaluation of Shear-Wall Dominant Building Structures

Can Balkaya
Department of Civil Engineering, Middle East Technical University, 06531 Ankara, Turkey
Erol Kalkan
Department of Civil and Environmental Engineering,
University of California Davis, Davis, California 95616, USA
(Received 15 August 2003)

The occurrences of the 1999 (M_w 7.4) Kocaeli and (M_w 7.1) Düzce earthquakes in Turkey once again demonstrate the behavioral importance of reinforced concrete (RC) shear-wall dominant structures that commonly built by using tunnel form technique. Reported non-damaged condition of these buildings in the aftermath of these two destructive urban earthquakes drew our attention to focus on performance of shear-wall dominant systems under earthquake forces. For that purpose, seismic behavior of these structures is investigated in details by performing inelastic three and two dimensional pushover analyses on a representative 5-story RC building model. The importance of 3D action in the structure'behavior because of slab-wall interaction, efficiency of transverse walls, effect of 2D and 3D modeling on the evaluation of capacity and demand relations, as well as damping effects are discussed. The 5-story building was modeled in a finite element domain considering material nonlinearity of steel and concrete including rotating and opening/closing crack capabilities. Performance of the model is 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 provides higher and more accurate capacity evaluations.

Keywords: Tunnel form, nonlinearity, shear-wall, pushover analysis, response modification factor, finite elements, capacity spectrum method.

1. 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 many parts of the world, limited research has been directed to establish their analysis, design and safety criteria.

Tunnel form formwork system is composed of vertical and horizontal panels set at right angles. Its typical illustration and application are shown in Fig. 1. There are no beams or columns and these structures generally utilize all shear-wall elements as primary load carrying members. The walls and slabs having almost same thickness are cast in a single operation. This reduces not only the number of joints, but also the assembly time. The simultaneous casting of walls, slabs and cross walls results in monolithic structures, which considerable enhances their seismic performance [1].

Besides their experienced well behavior under lateral forces, the current seismic provisions and design codes present inadequate guidelines re-

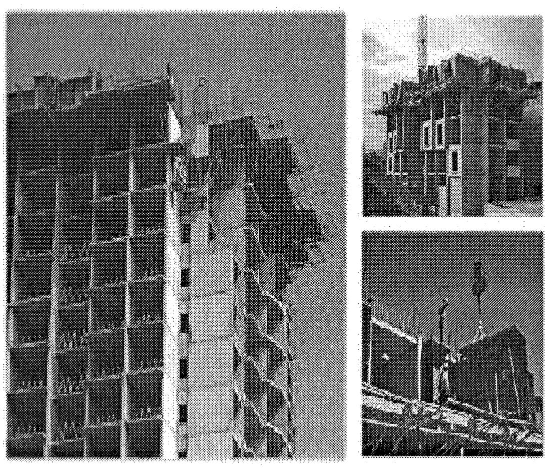

Figure 1. Tunnel form construction technique and its special formwork system

lated to their detailed analysis and design. On the other hand, three-dimensional (3D) behavior, floor flexibilities, slab-wall interaction, material nonlinearity including cracking, stress concentration around openings, the amount and location of reinforcement, torsional disturbance are all major contributors that should be considered to investigate the actual behavior of such structures. Our effort was spent to illuminate the importance of these factors by performing a 3D inelastic static (pushover) analysis on a representative 5-story building, and comparing the obtained results with that of commonly used 2D analysis. To accurately predict the nonlinear seismic performance of the model structure with sufficient accuracy, due care has been given to create detailed and efficient finite element models taking into account all essential geometric and strength characteristics of shear-walls, slabs and slab-wall connections. Toward minimizing the computational requirements and the volume of the input and output data to be handled, an effort was put 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.

In this study, two stages of analyses have been performed using the structural model. First stage constitutes the modal analyses that were needed to determine the fundamental period and the mode shape of the buildings to convert obtained capacity curves to acceleration displacement response spectrum format (ADRS). Inelastic static pushover analyses were next performed until all predefined collapse limits were exceeded. During pushover analyses permanent dead loads were first applied and iteration to equilibrium was performed. This was followed by applying incrementally increased (inverted triangle shaped) lateral loads. Although, analyses were inelastic, geometric nonlinearity was disregarded due to existing of small deformations. The applied displacement-based design methodology and consequently results of capacity evaluations enable us to investigate the reliability of the response modification factors given in design codes and seismic provisions that can be applicable to tunnel form buildings.

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 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.

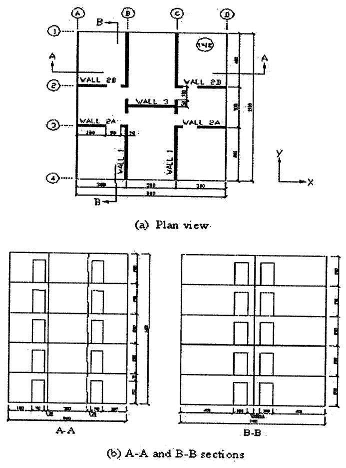

Figure 2. (a) Typical plan view, (b) Sections for 5 story building (units are in cm)

2. Analytical Model Development

By way of evaluating the 3D and 2D nonlinear capacities of tunnel form structures, a 5-story building is selected as a representative case study. This building is in the form of a typical reinforced concrete (RC) residential building. A detailed description of its plan and sections are illustrated in Fig. 2. Its structural system consists of solely shear-walls and slabs having same thickness as usual applications (12-20cm). 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 are constructed. These highly detailed models consider all openings and include both discrete and embedded reinforcements. The shear-wall bases are modeled as fixed at the foundation levels and soil-structure interaction effects are disregarded. The slabs are modeled by using finite elements having both flexural and membrane capabilities

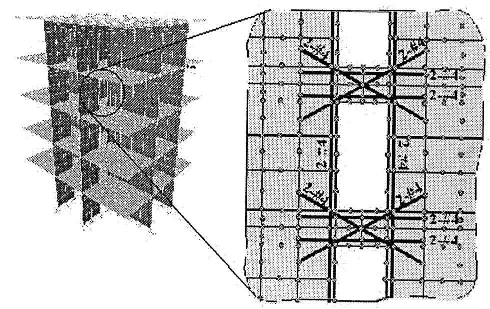

Figure 3. 3D finite element modeling of 5-story building and its discrete reinforcement detailing around the wall-openings

similar to those used in the wall modeling. Instead of accepting in-plane floor stiffness to be rigid (i.e., rigid diaphragm assumption), in-plane floor flexibility and slab-wall interaction are taken into account.

The performance of such detailed 3D nonlinear finite element analyses of shear-wall dominant structures inevitably requires a basic finite element having representative membrane and flexural characteristics. Therefore, 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, 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 used 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. Particularly for the 5-story building case, 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 ACI specification [2]. The 3D finite element model and its detailed reinforcement are shown in Fig. 3, and 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.

2.1. Nonlinear Isoparametric Shell Element

The nonlinear isoparametric shell element, 'CBAL'[3, 1] 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 the openings as discrete reinforcement) was used for the modeling. This element was adapted to POLO-FINITE [4] and analyses were performed by using this nonlinear finite element analysis program.

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 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 the 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. 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 discrete rebars are preferred near the edges, which have elasto-plastic stress-strain characteristics. By using the special isoparametric elements, the discrete steel can be included while locating the rebars with the proper cover requirements. With a two nodded rod, the stiffness con-

Table 1
Material properties of concrete and steel used in analytical models

Concrete	Steel	Steel Rod Element
E=2.1×10^6 t/m^2	E=2×10^7 t/m^2	E=2×10^7 t/m^2
ν=0.2	ν=0.3	ν=0.3
f_{tu}/f_{cu}=0.06823	Q_s(top)=0.2% in both direction	$A_s = 0.000226$ m^2 (at openings)
f_{c28}=1925 t/m^2	Q_s(bot.)=0.2% in both direction	$A_s = 0.000452$ m^2 (at edges)
	f_y=22000 t/m^2	f_y=22000 t/m^2

tributions 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.

2.3. Crack Modeling

Cracks in concrete can be modeled as a smeared or as a discrete crack model. Gerstle [5] 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 [6]. 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 [7] 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 cracks form, they remain open. However, this model leads to crack directions that can be inconsistent with the limit state [8]. 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 [6]. In rotating crack models, cracks are assumed to form orthogonal to the direction of either the principal stresses or 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 [8] 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 [9] 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 [9]. The constitutive matrix used for that purpose in this study has been derived by Gallegos and Schnobrich [10].

3. Capacity Spectrum Analysis for 5-Story Building

Pushover analysis procedure employing the capacity spectrum method as outlined in Ref. 11 was performed on the 5-story building. 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 was reached. These code bases calculated loads were applied as uniformly distributed loads to the story levels.

It should be also pointed out that torsion is an important issue that should be considered during the analysis of tunnel form buildings. Studies on 80 tunnel form building models show that due to construction limitations of tunnel form technique, distribution of shear-walls may result in torsional disturbance in the natural vibration

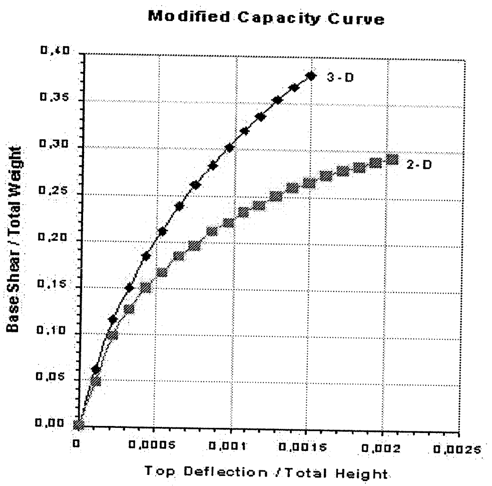

Figure 4. Modified capacity curves for 3D and 2D models of 5-story building

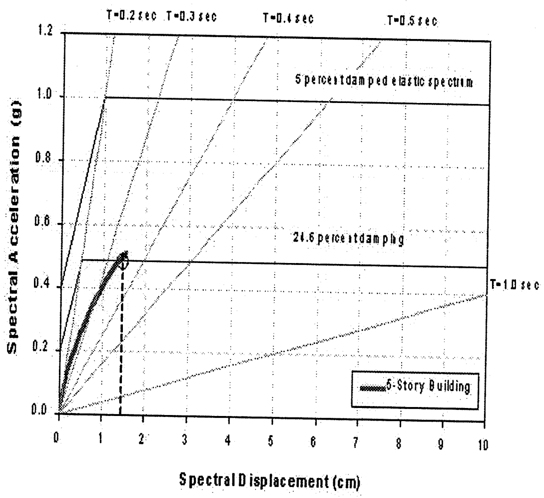

Figure 5. Application of capacity spectrum method to the 5-story building on the basis of Turkish Seismic Code design spectrum (soft soil site condition)

mode [12, 13]. The acceptable approach for considering the effects of torsion for the development of capacity curves is given in Ref. 11. In our study, torsion appeared in the first mode of the 3D model structure as well, which required modifications in the capacity curves according to Ref. 11. The resulting modified capacity curves for 2D and 3D analysis of 5-story case study as a result of loading in the y-direction (Fig. 2) are shown in Fig. 4. These capacity curves correspond to the last loading step of pushover analyses 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 the clear failure mechanisms, especially when shear-wall dominant 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. 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 inelastic deformation. The system behavior was completely controlled by the symmetrically distributed shear-walls.

4. 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 in recent years. Herein, 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 Ref. 11. 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 period of the 5-story building is obtained from modal analysis as

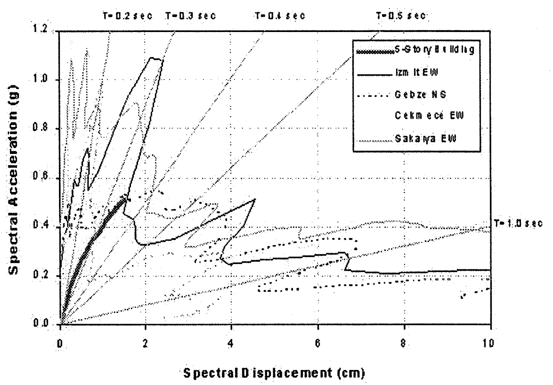

Figure 6. Application of capacity spectrum method to the 5-story building using the 1999 Kocaeli Earthquake main shock records (5% damped)

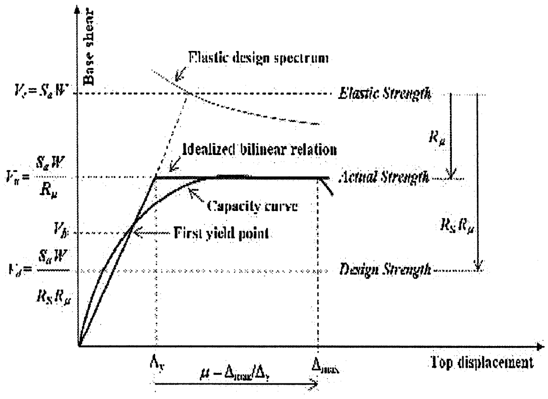

Figure 7. Relationships between the response modification factor (R), ductility factor (R_μ) and overstrength factor (R_S).

0.230 sec. The 5-story building pushed to roughly 2.10 cm of displacement at the roof level during the 3D analysis. Structural behavior type is selected as *Type A* according to Ref. 11. Obtained values of modal participation factors (PF_{RF}) and effective mass coefficients (α_m) are 1.38 and 0.76 for the 5-story model, respectively. Seismic demand is determined in accordance with the current Turkish Seismic Code [14]. Corresponding seismic demand and capacity spectra of the building are shown in Fig. 5 in the ADRS format. The 5-Story building possesses an energy dissipation capacity at the ultimate stage equivalent to 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 studied building is 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 0.0015 for 5 story building. At this level, the building is considered as satisfying the immediate occupancy (IO) performance level as described in Ref. 11. By referring to Fig. 5, the performance point is caught at 1.42cm (S_d) for 5-story building, this spectral displacement can be back translated to a roof displacement of 1.95cm ($\Delta_R = S_d$ x PFΦ_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 presented in Fig. 6. The 5-story building barely exceeds yield in the case of 5% damping response spectra.

5. Evaluation of Response Modification Factor (R) for Tunnel Form Buildings

In many seismic design codes and guidelines, such as UBC [15], NEHRP provisions [16] and Turkish Seismic Code [14], reduction in seismic forces via response modification factor (R) is justified by the unquantified overstrength and ductile response of buildings during design earthquake. However, none of these references address R-factor for shear-wall dominant systems. It is therefore intended to clarify the above, using the results of earlier discussed inelastic static pushover analysis of two cases. In general, the values assigned to R-factor are composed of the following sub-factors; period-dependent ductility factor (R_μ), period-dependent overstrength factor (R_S), and redundancy (R_R) factor (for this study, supplemental damping related factor (R_ξ) was disregarded). In this way, R-factor can simply be expressed as their product [17]:

$$R = (R_S R_\mu) R_R \tag{1}$$

Recent developments in displacement based design methodology [11, 16] enable more quantitative evaluation of these factors. The relations exhibited in Fig. 7. can be established for that purpose with the exception of reflecting the redundancy factor. This third factor, developed as part of the project ATC-34 [18] is proposed to quantify the improved reliability of seismic framing systems that use multiple lines of vertical seismic framing in each principal direction of a building [19]. For our studied cases, it might be appropriate to accept this factor as one. For the evaluation of the other two factors, the seismic design parameters, such as seismic zone, site geology and fundamental period must be clearly identified a priori. Accordingly, the worse scenario (highest seismicity and soft-soil site condition) based on the Turkish Seismic Code [14] was considered for the seismic design of two case studies. As such, the magnitudes of their design base shear were calculated as 0.155W and 0.250W for 2 and 5-story buildings, respectively. The overstrength factor (R_S), which can be determined as the ratio of the maximum lateral strength of a building (V_u) to the design base shear (Vd), envelopes the global effects of story drift limitations, multiple load combinations, strain hardening, participation of nonstructural elements, and other parameters [20]. To quantify this value, Hwang and Shinozuka [21] studied a four-story RC intermediate moment frame building located in seismic zone 2 as per the UBC [15], and they reported an overstrength factor of 2.2. Mwafy and Elnashai [22] performed both inelastic static pushover and time-history collapse analyses on 12 RC frame type buildings designed based on EC8 [23] codes and having various heights and lateral load supporting systems, and they have declared that all studied buildings have overstrength factors over 2. For the two cases investigated herein, the overstrength factors were calculated as 1.76 and 1.96 for 2 and 5-story buildings based on the inelastic pushover analysis results presented in Fig. 5. It is expected that, their actual values may be higher than those estimated due to the contribution of some parameters into the response such as outside pre-cast panel walls. The study conducted on tunnel form buildings seems to advocate this fact by indicating the contributory effects of the non-structural elements into the energy dissipation and lateral stiffness of the structural system [24].

The ductility factor (R_μ) is a measure of the global nonlinear response of the system. Most basically this parameter can be expressed as the ratio of elastic to inelastic strength [22] as illustrated in Fig. 7. Therefore, the resultant ductility factors in our study were found as 2.83 and 2 for 2 and 5-story buildings, respectively. They may yield to response modification factors of 4 and 5 for 2 and 5-story buildings according to Equation (1). The imposed R-factor in current seismic codes for RC frame type structures having shear-wall system that might be accepted as the closest form to tunnel form buildings, is equal to 5.5 in UBC [15] and 4 or 6 (depending on the ductility level) in the Turkish Seismic Code [14] demonstrating the fact that initial values given in these references are in acceptable ranges for tunnel form buildings when compared to obtained response modification factors in this study.

It is clear that standardization of response modification factor to be adopted in seismic design codes entails additional nonlinear case studies of tunnel form buildings regarding various plan configurations and seismic zones. The R-factor presented herein based on limited case studies may refrain the designer from blind selections of this factor for the seismic design of these buildings.

6. 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. A 5-story representative building is selected as a case study, and analyzed by utilizing 2D and 3D finite element models with the help of the developed isoparametric shell element. That provided reasonable simulation of yield locations as well as their crack patterns. 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 accurate analysis of tunnel form buildings. 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 indicates that R-factor of 4 to 5, in consistency with the referenced R-factors given in current codes, can be considered for the seismic design of these buildings. It should be also noted that actual response modification factors should be higher than the estimated values herein due to the remedial effects of nonstructural elements.

This paper also makes comparison between conventional 2D solutions and applied 3D analyses of presented case study and illuminates the reasons for their differences. Generally, the total resistance capacity of the three dimensionally analyzed structure is observed to be more than that of two dimensionally modeled structure. 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 controls 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 structures such as tunnel form buildings. The methodology 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 as well.

References

[1] C. Balkaya and E. Kalkan, Computers & Structures, **81**, 153 (2003).

[2] ACI 318, Building code requirements for reinforced concrete and commentary (ACI. Detroit, Michigan, 1989).

[3] C. Balkaya and W. C. Schnobrich , Struct. Eng. and Mech., **1**, 1 (1993).

[4] POLO-FINITE, Structural mechanics system for linear and nonlinear, static and dynamic analysis (Dept. of Civil Eng., UIUC).

[5] K.H. Gerstle, IABSE Colloquium DELF, **33**, 41 (1981)

[6] F.J. Vecchio and M.P. Collins, ACI. Struct. J. Proc., **83**, 219 (1986).

[7] H. Okamura and K. Maekawa, Proc. of SCI-C, Computer Aided Analysis and Design of Reinforced Concrete Structures, 831 (Pineridge Press, 1990)

[8] A.K Gupta and H. Akbar, Journal of Structural Engineering, ASCE., **110**, 1735 (1984)

[9] R.V. Milford and W.C. Schnobrich, Computers & Structures, **20**, 225 (1985)

[10] S. Gallegos-Cezares and W.C. Schnobrich, Struct. Research Series. Report No. 543, UIUC (1988)

[11] ATC-40, Seismic evaluation and retrofit of concrete buildings (Applied Technology Council, Redwood, CA, 1996).

[12] C. Balkaya and E. Kalkan , Earthquake Eng. and Struct. Dyn., **32**, 985 (2003)

[13] C. Balkaya, and E. Kalkan, Engineering Structures (2003), (in review).

[14] Specifications for structures to be built in disaster areas (Ministry of Public Works and Settlement, Ankara, Turkey, 1998).

[15] UBC, Uniform Building Code, International Conference of Building Officials, CA. (1997).

[16] FEMA 273 and 274 (FEMA, Washington, D.C., 1997).

[17] ATC-19, Structural response modification factors (Applied Technology Council, Redwood, CA., 1995).

[18] ATC-34, A critical review of current approaches to earthquake resistant design (Applied Technology Council, Redwood, CA., 1995)

[19] A. Whittaker, G. Hart, and C. Rojahn, Journal of Structural Engineering, ASCE, **125**, 438 (1999).

[20] C.M. Uang, Earthquake Eng. and Struct. Dyn., **23**, 507 (1994).

[21] H. Hwang and M. Shinozuka, Proceedings of 5th U.S. National Conf. on Earthquake Engrg., Earthquake Engineering Research Ins., Oakland, CA, 223 (1994).

[22] A.S. Elnashai and A.M. Mwafy , The Structural Design of Tall Buildings, **11**, 329 (2002).

[23] CEN (Comite European de Normalisation), Design provisions for earthquake resistance of structures, 1-1, 1-2 and 1-3, Eurocode 8, European pre-standard ENV 1998-1-1, 1-2 and 1-3. CEN (Bruxelles, 1994).

[24] M. Celebi, M. Erdik, and O. Yuzugullu, METU-EERI Report No: 77-6 (Middle East Technical University, Ankara, 1977).