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SEISMIC DESIGN PARAMETERS FOR SHEAR-WALL DOMINANT BUILDING STRUCTURES

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Artículo VII-11

RESUMEN

Edificios compuestos principalmente por muros de corte son el tipo de edificios de gran altura que prevalecen, particularmente en regiones con alto riesgo sísmico. Para identificar los parámetros más básicos de diseño, análisis de empuje dinámico y estático en el rango inelástico se realizaron como parte de una metodología de diseño basada en el desempeño. Ese intento facilita la investigación de su impactos muro contra muro, interacción muro-viga y efectos de apertura en muros en el comportamiento. Un nuevo set de formulas empíricas para la determinación explicita de sus periodos fundamentales son presentados, además de un recomendado factor de reducción de respuesta y detalle de armadura alrededor de las aperturas en muros.

ABSTRACT

Shear-wall dominant buildings are the prevailing multi-story RC buildings type particularly in the regions prone to high seismic risk. To identify their most essential design parameters, dynamic and inelastic static pushover analyses were conducted on the backbone of performance based design methodology. That attempt facilitates the investigation on impacts of wall-to-wall and wall-to-slab interactions and effects of wall-openings on the performance. A new set of empirical formulas for explicit determination of their fundamental periods are presented in addition to a recommended response reduction factor and reinforcement detailing around wall-openings.

INTRODUCTION

Shear-wall dominant buildings or so called tunnel form buildings throughout the paper (i.e., box systems or half tunnel form system) have been increasingly constructed as the multi-storey reinforced concrete (RC) buildings in lieu of conventional frame type (with or without shear-wall) RC buildings. Their experienced well performance during many recent earthquakes stimulated their preference particularly in the regions exposed to high seismic risk. In addition to their considerable earthquake resistance, the speed and ease of erection promote their use as the multi-unit construction of residential buildings. To this end, tunnel form buildings are commonly built in earthquake risky countries such as Turkey, Japan, Italy, Chile and also in many other countries such as Venezuela and Korea. Nonetheless, despite the abundance of such structures, limited research has been directed to clarify their design parameters.

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Tunnel form buildings are constructed using tunnel shaped (or half tunnel shaped) formwork system composed of vertical and horizontal panels set at right angles supported with struts and probs. Its typical implementation and constructive details are demonstrated in Figure 1. These buildings diverge from other conventional RC structures due to the absence of beams and columns in their structural system. These buildings utilize all shear-wall elements as primary lateral load resisting and vertical load carrying members. Although, use of load carrying pre-cast members is avoided, non-structural pre-cast elements such as stairs and outside facade panels are commonly used to speed up the construction. In tunnel form buildings, continuity of the shear-walls throughout the height of structure is recommended to avoid local stress concentrations and to minimize torsional effects. During construction, walls and slabs, having almost same thickness (12-20 cm), 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 that are not generally the case for any other conventional frame type RC buildings. With this feature, they gain enhanced seismic performance by retarding plastic hinge formations at the most critical locations, such as slab-wall connections and around wall-openings.

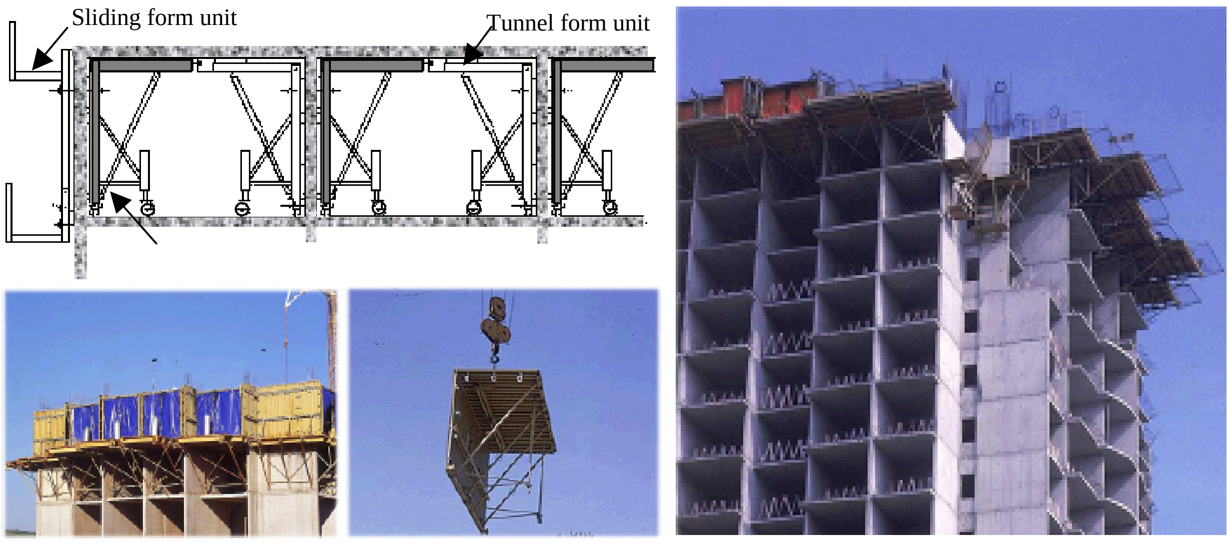


Figure 1. Typical implementation of tunnel form construction system

The seismic performances of such structures have been recently experienced during the 1999 Turkey earthquakes. The two (M_w) 7.4 and 7.2 events struck the most populated and industrialized heart of Turkey, namely Kocaeli and Düzce provinces two months apart from each other in 1999. In the aftermath of this misfortune, neither demolished nor damaged tunnel form buildings located in the vicinity of damage suffering regions were reported in contrast to poor performance and highly damaged conditions of many RC frame type buildings. Well performance of tunnel form system during these earthquakes and lack of guidelines regarding their seismic design and analysis in current building codes and provisions are the motivation for us to illuminate their seismic behavior and characteristic design parameters. Thus, we first investigated the consistency of code-given formulas used for the determination of their pivotal design parameter and it was revealed that, formulas used in the practice and involved in both the Turkish Seismic Code (1998) and the Uniform Building Code (UBC, 1997) for explicit determination of their fundamental periods yield inaccurate results (Balkaya and Kalkan, 2003a). Therefore, a new set of empirical equations were developed based on the 3D finite element analysis of 80 different buildings having variety of plans, heights and wall-configurations. These equations and the values of their predictor parameters are introduced in the first part of this paper. To gain the full picture of seismic behavior of tunnel form buildings capacity spectrum method (ATC-40, 1996) was utilized following the detailed inelastic static pushover analysis. The intent to conduct such an analysis for shear-wall dominant three-dimensional models inevitably entailed the development of a new isoparametric shell element having opening-closing and rotating crack capabilities. Therefore, the factors constituting more pronounced impacts on their seismic behavior were considered more realistically and efficiently without necessitating any simplifications in the finite element models. These emphasized factors are mainly; 3D behavior, diaphragm flexibility, slab-wall interaction and material nonlinearity and additionally, stress concentration and shear flow around the openings, amount and location of reinforcement around wall-openings, damping and torsional effects. This study illuminates their importance based on the results of 3D inelastic static pushover analyses of two representative case studies. The obtained results were further compared with those of 2D analyses. The reliable value of a response modification factor for these buildings was also investigated by referencing displacement-based design methodology, and obtained values were compared next with those dictated in current seismic design codes. With all the presented information herein, this paper summarizes the most significant results of the analytical work conducted for its possible use in current engineering practice.

EMPIRICAL PREDICTION OF FUNDAMENTAL PERIOD

It is customary in practice to obtain the fundamental periods of structures via simple expressions to establish the proper design force level unless modal analysis based on detailed finite element models is conducted. Therefore accurate estimation of fundamental period is inevitably essential for the reliability of a design and consequently for the performance of a structure. However, it has long been realized that comparatively significant errors are prone to occur when the code-given formulas such as those in the UBC (1997), Turkish Seismic Code (1998) are utilized for shear-wall dominant systems. Other predictive formulas stimulated by the buildings codes of South-East Asia yield similar errors as well (Lee et al., 2000). To overcome this deficiency for tunnel form buildings, a consistent set of predictive formulas were proposed based on the finite element analyses of 16 different as-built plans for 5 different building heights (i.e., for storey levels of 2,5,10,12 and 15). The database compiled for that purpose constitutes basic structural properties of 80 different cases and their modal analysis results. The initial analyses showed that, there is a clear distinction in the fundamental periods of the structures depending on their side ratios. Accordingly, complete database was categorized into two sub-data sets considering the side dimension ratio of buildings. If the ratio of long side to short side dimension is less than 1.5, these plans are accepted as square, and those out of this restriction are accepted as rectangular. These separate data sets are shown in Table 1. Nonlinear regressions considering a great deal of alternative formulas taking into account various structural and architectural parameters were performed next on the two distinct data sets separately. Consequently, the fundamental period estimation equations given in Table 2 were recommended to predict the fundamental period of tunnel form buildings over the full range of storey levels (2 to 15) for both square and rectangular plans. The obtained results were also used to compute the associated errors in the estimation. The standard deviation of residuals, σ_T, expressing the random variability of periods is 0.025 and the value of R^2 (i.e., indication of goodness of fit) is equal to 0.95 for the two data sets. The equations given in Table 3 were compared to those given in the UBC (1997) and Turkish Seismic Code (1998), and also compared with dynamic analysis results in Figure 2 for various selective cases. The obtained finite element analysis results are significantly differ from code-referred values, whereas the general good agreement between all these curves gives support to predicted fundamental periods. The investigation of modal properties of 80 cases also reveals that torsional disturbance is a handicap for these buildings unless appropriate side ratio is selected and shear-walls are configured properly. That might be attributed to interaction of closely spaced dynamic modes. Further details about the buildings in the database, finite element modeling and formula derivation were given in elsewhere (Balkaya and Kalkan, 2003a). Experimental verification of these equations is obviously essential for their reliability.

Table 1. Building inventory and their architectural and structural properties

Rectangular Plans *

Plan No	Storey #	Height (m)	Plan Dimensions (m) Long Side	Plan Dimensions (m) Short Side	Shear Wall Area (m^2) Long Side	Shear Wall Area (m^2) Short Side	T (s)
1	2	5.6	29.70	15.70	4.78	17.80	0.048
2	2	5.6	31.04	19.92	3.40	19.92	0.049
3	2	5.6	38.80	17.03	3.98	19.60	0.052
4	2	5.6	12.00	8.00	1.44	2.88	0.042
5	2	5.6	12.00	8.00	3.84	1.92	0.042
6	2	5.6	12.00	8.00	1.44	3.84	0.037
7	2	5.6	12.00	8.00	2.88	2.64	0.043
8	2	5.6	12.00	8.00	2.88	3.36	0.035
9	2	5.6	12.00	8.00	4.80	1.92	0.042
10	2	5.6	35.00	20.00	7.20	12.96	0.040
1	5	14.0	29.70	15.70	4.78	17.80	0.129
2	5	14.0	31.04	19.92	3.40	19.92	0.123
3	5	14.0	38.80	17.03	3.98	19.60	0.143
4	5	14.0	12.00	8.00	1.44	2.88	0.130
5	5	14.0	12.00	8.00	3.84	1.92	0.157
6	5	14.0	12.00	8.00	1.44	3.84	0.110
7	5	14.0	12.00	8.00	2.88	2.64	0.131
8	5	14.0	12.00	8.00	2.88	3.36	0.123
9	5	14.0	12.00	8.00	4.80	1.92	0.158
10	5	14.0	35.00	20.00	7.20	12.96	0.156
1	10	28.0	29.70	15.70	4.78	17.80	0.293
2	10	28.0	31.04	19.92	3.40	19.92	0.276
3	10	28.0	38.80	17.03	3.98	19.60	0.312
4	10	28.0	12.00	8.00	1.44	2.88	0.350
5	10	28.0	12.00	8.00	3.84	1.92	0.425
6	10	28.0	12.00	8.00	1.44	3.84	0.322
7	10	28.0	12.00	8.00	2.88	2.64	0.354
8	10	28.0	12.00	8.00	2.88	3.36	0.330
9	10	28.0	12.00	8.00	4.80	1.92	0.420
10	10	28.0	35.00	20.00	7.20	12.96	0.384
1	12	33.6	29.70	15.70	4.78	17.80	0.368
2	12	33.6	31.04	19.92	3.40	19.92	0.346
3	12	33.6	38.80	17.03	3.98	19.60	0.385
4	12	33.6	12.00	8.00	1.44	2.88	0.494
5	12	33.6	12.00	8.00	3.84	1.92	0.542
6	12	33.6	12.00	8.00	1.44	3.84	0.450
7	12	33.6	12.00	8.00	2.88	2.64	0.495
8	12	33.6	12.00	8.00	2.88	3.36	0.462
9	12	33.6	12.00	8.00	4.80	1.92	0.539
10	12	33.6	35.00	20.00	7.20	12.96	0.484
1	15	42.0	29.70	15.70	4.78	17.80	0.489
2	15	42.0	31.04	19.92	3.40	19.92	0.466
3	15	42.0	38.80	17.03	3.98	19.60	0.498
4	15	42.0	12.00	8.00	1.44	2.88	0.758
5	15	42.0	12.00	8.00	3.84	1.92	0.725
6	15	42.0	12.00	8.00	1.44	3.84	0.690
7	15	42.0	12.00	8.00	2.88	2.64	0.754
8	15	42.0	12.00	8.00	2.88	2.88	0.700
9	15	42.0	12.00	8.00	4.80	1.92	0.719
10	15	42.0	35.00	20.00	7.20	12.96	0.638

Square Plans **

Plan No	Storey #	Height (m)	Plan Dimensions (m) Long Side	Plan Dimensions (m) Short Side	Shear Wall Area (m^2) Long Side	Shear Wall Area (m^2) Short Side	T (s)
11	2	5.6	11.00	9.00	2.64	1.80	0.073
12	2	5.6	31.50	27.15	9.70	13.86	0.047
13	2	5.6	25.50	25.04	10.70	10.88	0.041
14	2	5.6	14.00	12.00	2.88	3.60	0.039
15	2	5.6	27.00	24.00	8.40	13.55	0.045
16	2	5.6	32.00	26.00	9.40	15.00	0.045
11	5	14.0	11.00	9.00	2.64	1.80	0.231
12	5	14.0	31.50	27.15	9.70	13.86	0.157
13	5	14.0	25.50	25.04	10.70	10.88	0.135
14	5	14.0	14.00	12.00	2.88	3.60	0.136
15	5	14.0	27.00	24.00	8.40	13.55	0.166
16	5	14.0	32.00	26.00	9.40	15.00	0.172
11	10	28.0	11.00	9.00	2.64	1.80	0.630
12	10	28.0	31.50	27.15	9.70	13.86	0.422
13	10	28.0	25.50	25.04	10.70	10.88	0.404
14	10	28.0	14.00	12.00	2.88	3.60	0.396
15	10	28.0	27.00	24.00	8.40	13.55	0.486
16	10	28.0	32.00	26.00	9.40	15.00	0.487
11	12	33.6	11.00	9.00	2.64	1.80	0.819
12	12	33.6	31.50	27.15	9.70	13.86	0.551
13	12	33.6	25.50	25.04	10.70	10.88	0.549
14	12	33.6	14.00	12.00	2.88	3.60	0.541
15	12	33.6	27.00	24.00	8.40	13.55	0.647
16	12	33.6	32.00	26.00	9.40	15.00	0.638
11	15	42.0	11.00	9.00	2.64	1.80	0.830
12	15	42.0	31.50	27.15	9.70	13.86	0.769
13	15	42.0	25.50	25.04	10.70	10.88	0.801
14	15	42.0	14.00	12.00	2.88	3.60	0.785
15	15	42.0	27.00	24.00	8.40	13.55	0.918
16	15	42.0	32.00	26.00	9.40	15.00	0.877

* Ratio of long side to short side dimension is greater or equal to 1.5
** Ratio of long side to short side dimension is less than 1.5

Table 2. Predictive equations for the fundamental periods of tunnel form buildings

$$T = C\, h^{b1} \beta^{b2} \rho_{as}^{\;b3} \rho_{al}^{\;b4} \rho_{min}^{\;b5}\, j^{\,b6}$$

Plan Type	*C*	*b1*	*b2*	*b3*	*b4*	*b5*	*b6*	σ_T	R^2
Square	0.158	1.400	0.972	0.812	1.165	-0.719	0.130	0.025	0.982
Rectangular	0.001	1.455	0.170	-0.485	-0.195	0.170	-0.094	0.025	0.989

T : Fundamental period (sec)
h : Total building height (m)
β : Ratio of long side to short side dimension
ρ_{as} : Ratio of short side shear wall area to total floor area
ρ_{al} : Ratio of long side shear wall area to total floor area
ρ_{min}: Ratio of minimum shear wall area to total floor area
J : Plan polar moment of inertia (m^4)

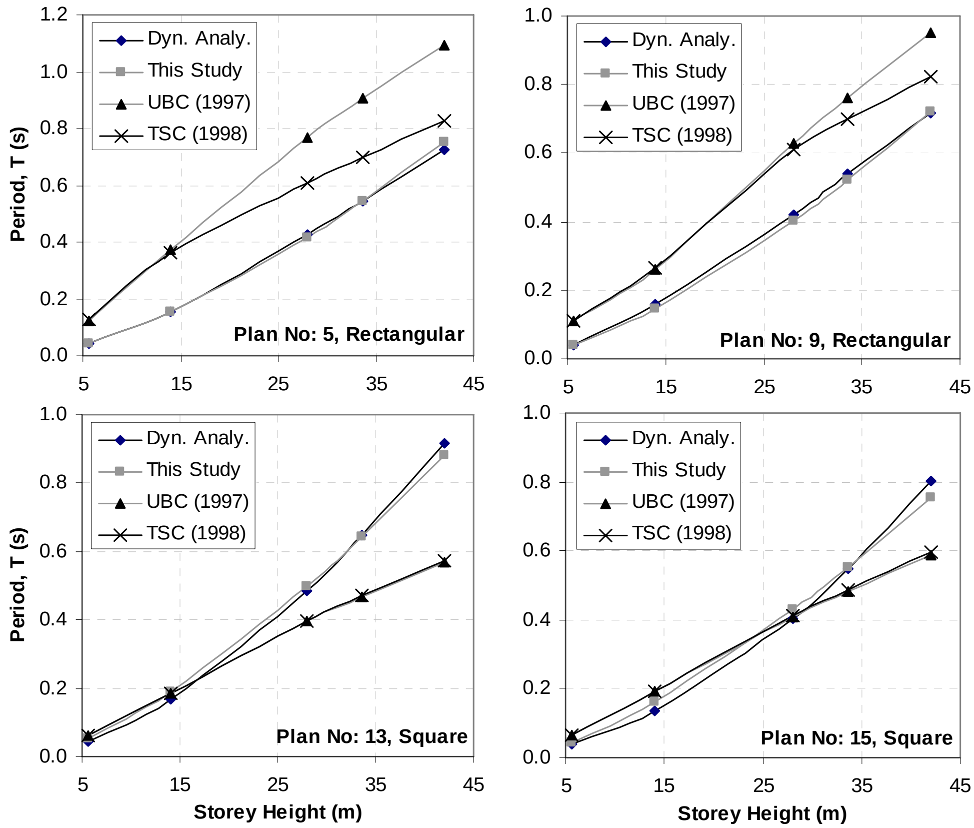
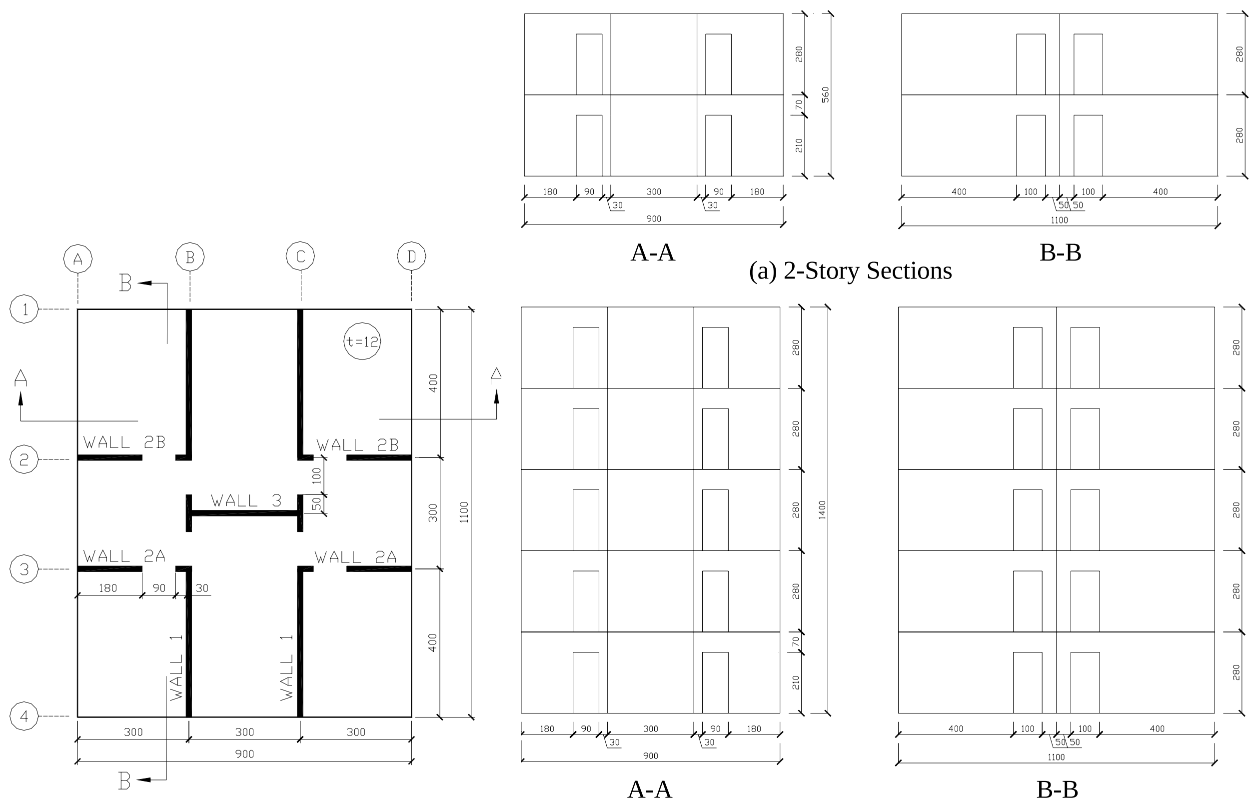


Figure 2. Comparison of fundamental periods for various heights of rectangular and square plans

ANALYTICAL MODEL DEVELOPMENT

By way of evaluating the 3D nonlinear seismic response of tunnel form buildings, 2 and 5-story residential buildings were implemented as the representative cases. A detailed description of their architectural plans and section views are exhibited in Figure 3. Their structural systems are composed of solely shear-walls and slabs having the same thickness (12 cm) as usual applications. It is of interest to note that almost equal slab and wall thickness, generally less than that of used in conventional RC structures having analogous architectural plans, causes a high slab-wall interaction. Therefore, for the modeling of such structures, making a rigid diaphragm (infinitely rigid in their own plane) assumption in order to simplify the analysis and save from the execution time does not reflect their realistic behavior. This may be attributed to fact that tunnel form buildings behave like thin-wall-tubular structures where in-plane rigidity is low. Thus, high stress-concentrations may increase the crack propagation at the edges of slab-wall connections (Balkaya and Kalkan, 2003b). This phenomenon is also discussed by Tena-Colunga and Abrams (1996), and also Fleischman and Farrow (2001). To better reflect the in-plane floor flexibility and the slab-wall interaction, it is more reasonable to model the shear-walls and slabs by using finite elements having both flexural and membrane capabilities. Towards this aim, 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 for representing more realistic reinforcement contribution (Balkaya and Schnobrich, 1993).



(a) 2-Story Sections

(c) Plan View

(b) 5-Storey Sections

Figure 3. Typical architectural plan and section views of the 2 and 5-story buildings (units are in cm)

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. In all models the shear-walls were modeled as sitting on the fixed base supports and soil effects were ignored. The impacts of soil-structure interaction on the dynamic behavior of tunnel form buildings are the subject of our ongoing research. The reinforcements were modeled as discrete or embedded based on the criticality of their locations. The minimum amount of steel percentage taken in the analyses for shear-walls and slabs was 0.4 percent of the section area in accordance with the ACI 318 (1995) specifications. There were also additional longitudinal and diagonal reinforcement used in the modeling in the form of 2 #4 at the inner and outer faces of the edges and 2 #4 around the openings. During all analyses, kinematic nonlinearity was disregarded due to the formation of the relatively small deformations, whereas only material nonlinearity was considered as necessary.

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 implemented to POLO-FINITE. The rest of the analyses were performed by using this general purpose nonlinear finite element analysis program. 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 they can serve as end nodes for the cover of the main discrete reinforcement. That provides a robust stiffness contribution coming from the main reinforcement (Balkaya and Schnobrich, 1993). 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 critical areas such as around openings and slab-wall connections. The matching of the displacement fields between different order finite elements can be adjusted to retain the compatibility along their common edges. One of the improvements resulting from the use of this element is the reduction in the capacity and computational time required to reach a solution while retaining the level of accuracy deemed desirable. In this study, the shape of the stress-strain curve, tension stiffening and the cracking having opening and closing capability (Milford and Schnobrich, 1985; Gallegos and Schnobrich, 1988) were considered in the context of the material nonlinearity.

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 that present compatibility discontinuities with the adjacent concrete units. 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 preferred. On the other hand, the smeared reinforcement model, that is the easiest to implement, transfers the effects of the steel directly into the concrete element. In this study, nonlinear rod elements were used around the openings, and discrete rebars having elasto-plastic stress-strain characteristics were preferred near the edges. With the help of 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 element, 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. The steel is taken into account in both faces of the slabs and shear-walls, and in both principal directions considering the minimum steel ratio and cover thickness.

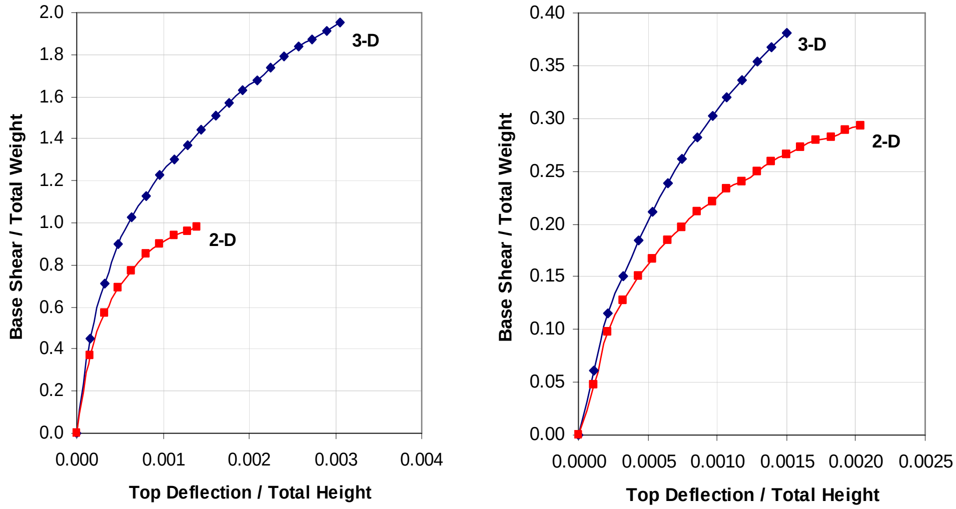
Crack Modeling

Cracks in concrete can be modeled either as a smeared or a discrete crack model. In the smeared crack modeling, there are several options. They can be modeled either as a fixed-crack or as a rotational-crack. In most of the finite element analysis of RC structures, crack directions are assumed to be fixed; this means when the crack forms it remains 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 the 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 effects is inevitable. In fact, the rotating crack models represent the actual behavior more accurately (Milford and Schnobrich, 1985). The constitutive matrix implemented in this study has been derived by Gallegos and Schnobrich (1988). The important concrete cracking behavior was handled through the smeared crack concept that has a rotation as well as closing and opening capabilities. More comprehensive information regarding the capabilities of the isoparametric shell element was given by Balkaya and Kalkan (2003b).

SEISMIC PERFORMANCE EVALUATION VIA CAPACITY SPECTRUM METHOD

In order to evaluate the seismic performance of tunnel form buildings, inelastic static pushover analyses were first performed on 2 and 5-story buildings. Next, capacity spectrum method (ATC-40, 1996) was implemented. During pushover analyses, gravity loads were applied to structures while simultaneously pushed with incrementally increased static equivalent earthquake loads until the specified level of roof drifts were reached. These 3D analyses were performed for 2D models as well. These models were simulated considering only the main shear-walls (Section B-B in Fig. 2). As alluded to earlier, torsion is exceptionally important that appears in the dynamic mode of tunnel form buildings due to the tunnel form construction restrictions, as such part of the outside walls should be opened in order to take the formwork back during construction (Fig. 1). The acceptable approach for considering the effects of torsion in the development of capacity curves is described in ATC-40 (1996). 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 and 5-story case studies as a result of the applied lateral loading in the y-direction (Fig. 2) are presented in Figure 4. These curves correspond to the last loading step of pushover analysis where excessive crack development at the base level of shear-walls did not yield any more permanent deformation. The base moments and resultant forces were calculated considering couple walls to observe the contribution of the 3D behavior. For the 5-story 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. Further details of applied 3D pushover analyses and its implementation under dominant torsional effects were discussed in elsewhere (Balkaya and Kalkan, 2003b).

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 trend has been followed for performance evaluations in recent years. Herein the demand curve is represented by earthquake response spectra. In general, a 5 percent damped response spectrum is used to represent the demand when the structure is responding linearly elastic. In this study, the capacity curves were converted into the acceleration displacement response spectrum format (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 (ATC-40, 1996). The effective vibration periods of the 2 and 5-story buildings obtained from the modal analyses were 0.073 and 0.230 seconds, 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 reported in ATC-40 (1996). 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, respectively. The seismic demand was determined in accordance with the current Turkish Seismic Code (1998). The corresponding seismic demand and capacity spectra are presented in the ADRS format for comparison in Figures 5 for 2 and 5-story buildings, respectively. The 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 the smaller spectral displacement. The energy dissipation capacity of the 5-story building is less than that of the first one, that corresponds to 24.6 percent viscous damping (a_y=0.31g, d_y=0.41cm, a_p=0.51g, d_p=1.52cm). 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 0.0030 and 0.0015 for the 2 and 5-story buildings, respectively. At this level, the buildings are considered to be satisfying the immediate occupancy (IO) performance level according to ATC-40, 1996. The performance point is 1.42cm (S_d) for the 5-story building. This spectral displacement can be back translated to a roof displacement of 1.95cm ($\Delta_R = S_d \times PF\Phi_R$) and a base shear coefficient of 0.37 ($V/W = \alpha\ S_a$).



(a) 2-Story Building (b) 5-Story Building

Figure 4. Modified capacity curves for 3D and 2D models

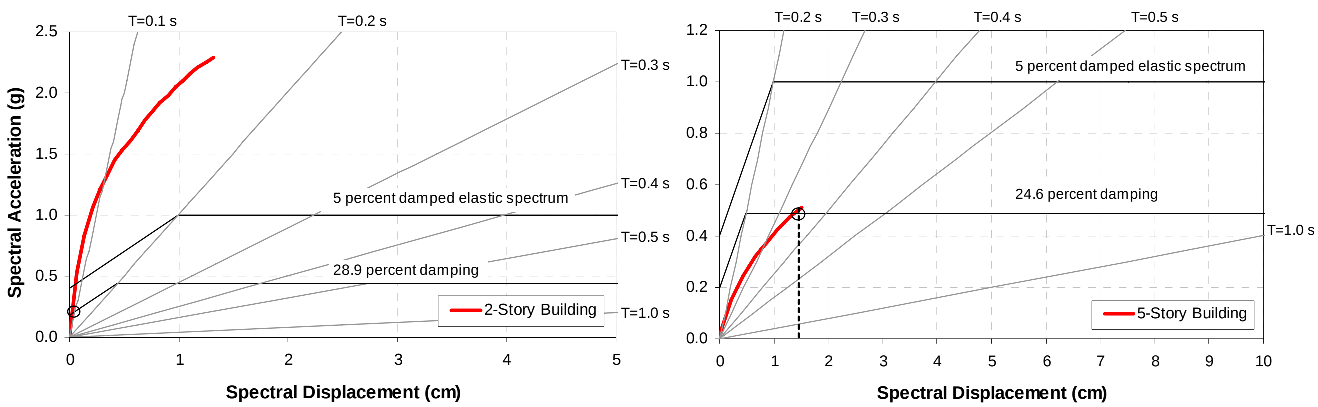


Figure 5. Capacity spectrum method to the 2 and 5 story buildings based on the Turkish Seismic Code (1998) design spectrum (soft soil site condition)

Tension-compression (T/C) coupling, executed by in-plane or membrane forces within the shear-walls, is a 3D originated mechanism building-up in tunnel form buildings due to the combined effects of wall-to-wall (even including walls with openings) and wall-to-slab interactions. In this mechanism, the outer walls, oriented perpendicularly to lateral loading directions, act as flanges when subjected to bending loads and resist overall moment primarily in tension and compression. Whereas, the inner walls, passing from the centroid and oriented to the same direction with the lateral loading, act in bending, and their contribution to overall moment capacity are relatively small. In general, this 3D originated mechanism show a characteristic T-section behavior. Therefore, the resultant force mechanism exhibits a significant contribution in the capacity and seismic performance of these buildings. The basic development of T/C coupling mechanism for tunnel form buildings is presented in Figure 6.

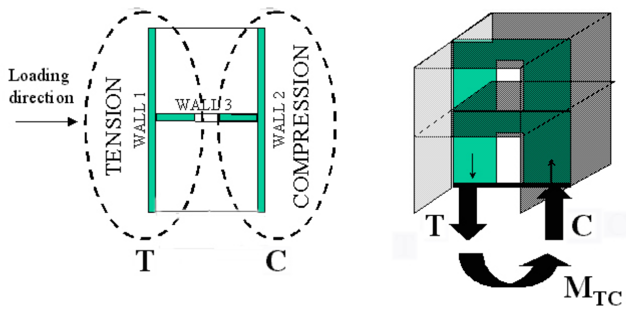
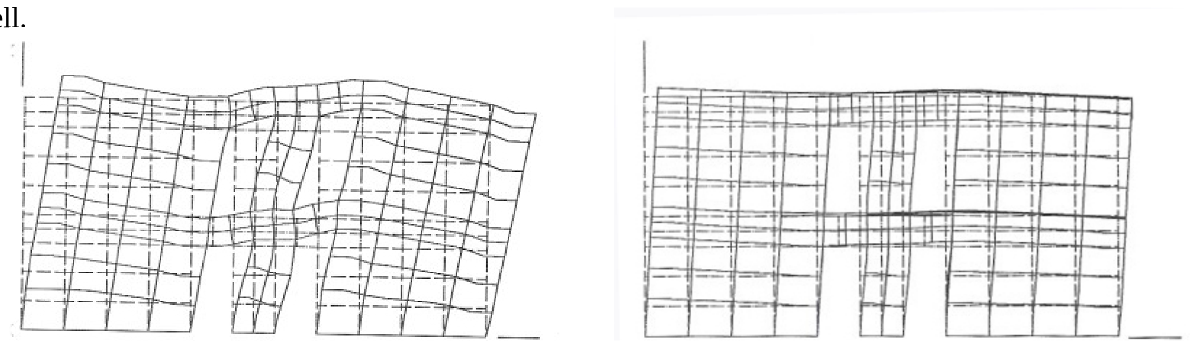


Figure 6. Slap-wall interaction due to tension and compression (T/C) coupling mechanism

The analyses showed that, the part of the walls above the openings were deflected more in the 2D models than 3D models (Fig. 7). During 2D simulations, 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 walls was not altered significantly between the 2D and 3D cases. This may be attributed to the limitation in the contribution of steel that is set by the steel area and its yield stress. When the analysis was switched from the 2D to 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 be 2,130 kN.m (213 ton.m) during the 2D modeling. When the 3D model was considered at this load level, this moment capacity corresponded to 1,703 kN.m (170.3 ton.m) and gradually increased up to 4,420 kN.m (442 ton.m) at its failure load level. This step up was accredited to the 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.



(a) 2D model deflected shape (Section B-B) (b) 3D model deflected shape (Section B-B)

Figure 7. Deflected shapes for *Wall 1*in 2D and 3D models for 2-story building

EFFECTS OF SHEAR-WALL OPENINGS AND THEIR REINFORCEMENT DETAILING

In the tunnel form construction technique, slabs are supported only along their three sides by shear-walls while one side remained unsupported in order to take the formwork back (Fig.1). In the common practice, these three shear-walls contain at least one opening for the functional use and access. The overall analysis of the case studies shows that the openings introduce a strong disruption of the shear flow between the adjacent shear-walls. The effects of openings on the strength and deformation capacity of the shear-wall systems are generally different than those observed in conventional frame-wall systems due to the coupling effects of beams connecting the adjacent shear-walls. These differences are more evident when the 3D behavior is taken into account. In general, no contra flexure points occur above the openings as they do in the 2D coupled wall cases due to the restraint of motion caused by the existing transverse walls and slabs having a continuous edge support in the three dimensions. As inffered from Fig. 7 the part of the wall between the openings was deflected more in the 2D models than 3D models. Due to the nature of stress concentrations around the openings, use of the diagonal shear reinforcement 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 for the reinforcement detailing around the openings of pierced shear-walls in the case of nonexistence of connection beams between these walls. A reasonable estimate for the lower bound of the shear strength of low-rise walls with minimum web reinforcement was presented by Wood in 1990. Additionally, the UBC (1997) indicates that in addition to the minimum reinforcement in the walls, not less than 2 #5 bars shall be provided around the openings. The placement of diagonal shear reinforcement with an angle of 45 degrees at the sections above the openings is recommended in the Turkish Seismic Code (1998). Due to the formation of high stress concentration around the openings, use of the shear reinforcement as stirrups in the pierced part in addition to the edge reinforcement provides a significant confinement to the concrete covering the main longitudinal bars, and prevents the buckling of the bars and the premature shear failure. If diagonal bars are not provided, additional shear reinforcement shall be used to resist the diagonal tension (Balkaya and Kalkan, 2003c). The minimum amount of reinforcement and its detailing shown in Figure 8a is recommended for pierced shear-walls in the case of existence of shallow parts above the openings (using 2 #4 as top and bottom bars, and 2 #4 at each vertical edge). Paulay and Binney (1974) suggested the use of the diagonal reinforcement in deep coupling beams because of the relatively large shears that develop and the likelihood of shear failures under reversed cyclic loadings. Since the deep connections between shear-walls in the tunnel form buildings behave in a similar fashion, the reinforcement details given in Figure 8b and 8c can be suggested when the wall part above the openings of the pierced walls is deeper. The degrees of coupling between the wall parts considering the stiffness of the adjacent slabs and transverse walls in 3D should be the bases for the reliable reinforcement detailing around the wall-openings of tunnel form buildings.

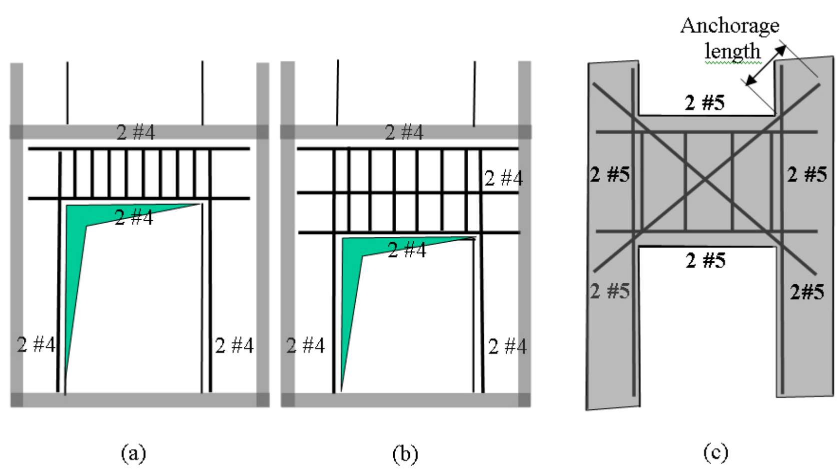


Figure 8. Reinforcement detailing around the openings of pierced shear-walls

RESPONSE MODIFICATION FACTOR (*R*) FOR TUNNEL FORM BUILDINGS

In many seismic design codes and guidelines, such as UBC (1997), NEHRP provisions (FEMA 1997b) and Turkish Seismic Code (1998), 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 (ATC-19, 1995):

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

Recent developments in displacement based design methodology (ATC-40 and FEMA 273-274) enable more quantitative evaluation of these factors. The relations exhibited in Figure 9 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 (1995) 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 (Whittaker et al., 1999). 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 (1998) 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 (V_d), envelopes the global effects of story drift limitations, multiple load combinations, strain hardening, participation of nonstructural elements, and other parameters (Uang, 1994). To quantify this value, Hwang and Shinozuka (1994) studied a four-story RC intermediate moment frame building located in seismic zone 2 as per the UBC (1994), and they reported an overstrength factor of 2.2. Mwafy and Elnashai (2002) performed both inelastic static pushover and time-history collapse analyses on 12 RC frame type buildings designed based on EC8 (CEN, 1994) 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 Figure 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 (e.g. Celebi et al., 1977).

The ductility factor ($R\mu$) 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 (e.g. Mwafy and Elnashai 2002) as illustrated in Figure 9. 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 (1997) and 4 or 6 (depending on the ductility level) in the Turkish Seismic Code (1998) 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.

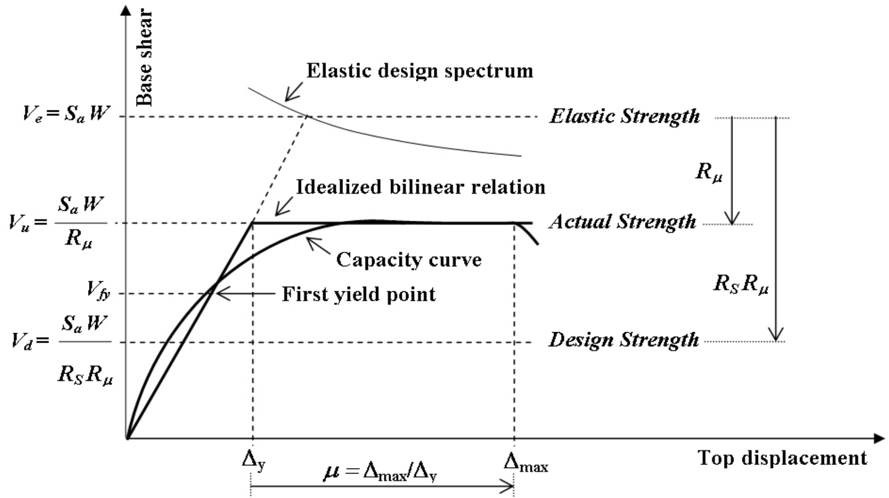


Figure 9. Relationships between the response modification factor (*R*), ductility factor (R_μ) and overstrength factor (R_S)

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.

CONCLUSIONS

This article reflects the breadth of the multi-scope study conveyed for tunnel form buildings to identify their most important seismic design parameters. It was also aimed to reveal the strong and weak points of tunnel form construction technique. The recommended empirical equations presented in this study through Table 1 can be considered to be appropriate for the estimation of fundamental periods of tunnel form buildings for 2 to 15 storey levels having various architectural configurations. Yet there exists a need of ambient surveys that can provide an effective tool for experimentally verifying the validity of the proposed formulas. For buildings having storey levels more than 15 the reader can refer to the study of Lee et al. 2000. The pushover analysis was utilized as a tool in this study within the perimeter of performance-based design concept. For structures that vibrate primarily in the fundamental mode like the two 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. 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 evaluation of response modification factors "*R*" used in current seismic design provisions and codes. 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 beneficial effects of nonstructural elements.

Notwithstanding the preferable seismic resistance of these buildings, tunnel form construction limitations and restrictions may facilitate the appearance of torsional effects in their dominant vibration modes. Selection of appropriate side dimensions and symmetrical configuration of shear-walls may help to eliminate these controversial effects. Although rectangular plans seem to be more preferable for avoiding torsion than square plans, they have weaker bending capacity along their short sides due to the architectural and constructional limitations. Designer should be aware of these observed handicaps of tunnel form buildings. The results of this study also show that both analyses and experimental studies conducted on shear-wall dominant buildings without paying attention to 3D effects of existing transverse walls and diaphragm flexibilities may yield inaccurate results. In this study, the stress flow and crack patterns around the openings of pierced shear-walls in the 3D models were observed to be significantly different than those for the 2D models. This was attributed to the nonexistence of the contra-flexure points during the 3D behavior. The deflected shapes obtained for the sections above the openings in the 3D models demonstrated more rigid forms than those in the 2D models. In general, considering the interaction effects of the slabs and transverse walls during the analyses increased the overall capacity of the pierced shear-walls. It is further observed that despite the existence of openings introducing a strong disturbance of the shear flow within the transverse walls, these walls provided a significant contribution to the formation of the T/C coupling mechanism. Even though, the membrane action was found to be a dominant force mechanism for the tunnel form buildings, use of a nonlinear isoparametric shell element rather than a plane stress element in the finite element models provides a better representation of this mechanism. Additionally, use of this element enabled various modeling of the reinforcement in the models based on the criticality of their locations. To investigate the local effects around the openings, simulation of reinforcement with discrete elements at such weak locations provided the detailed modeling of concrete cover for the development of more realistic crack patterns. Due to the nature of high stress concentrations around the openings, use of the diagonal shear reinforcement in addition to the edge reinforcement in these locations may lead significant contribution for retarding and slowing down the crack propagation. For that reason, reinforcement details given in this study are recommended for various shear-wall-opening configurations.

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