EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS
Earthquake Engng Struct. Dyn. 2003; **32**:985–998 (DOI: 10.1002/eqe.258)

Estimation of fundamental periods of shear-wall dominant building structures

Can Balkaya[1] and Erol Kalkan[2,*,†]

[1] *Department of Civil Engineering, Structures Division, Middle East Technical University, Ankara 06531, Turkey*
[2] *Department of Civil and Environmental Engineering, Rensselaer Polytechnic Institute, Troy, NY 12180, U.S.A.*

SUMMARY

Shear-wall dominant multistorey reinforced concrete structures, constructed by using a special tunnel form technique are commonly built in countries facing a substantial seismic risk, such as Chile, Japan, Italy and Turkey. In spite of their high resistance to earthquake excitations, current seismic code provisions including the Uniform Building Code (*International Conference of Building Officials*, Whittier, CA, 1997) and the Turkish Seismic Code (*Specification for Structures to be Built in Disaster Areas*, Ankara, Turkey, 1998) present limited information for their design criteria. In this study, consistency of equations in those seismic codes related to their dynamic properties are investigated and it is observed that the given empirical equations for prediction of fundamental periods of this specific type of structures yield inaccurate results. For that reason, a total of 80 different building configurations were analysed by using three-dimensional finite-element modelling and a set of new empirical equations was proposed. The results of the analyses demonstrate that given formulas including new parameters provide accurate predictions for the broad range of different architectural configurations, roof heights and shear-wall distributions, and may be used as an efficient tool for the implicit design of these structures.
Copyright © 2003 John Wiley & Sons, Ltd.

KEY WORDS: shear wall; tunnel form technique; torsion; earthquake-resistant design; fundamental period; finite-element modelling

INTRODUCTION

Shear-wall dominant buildings, constructed by using tunnel form techniques, are composed of vertical and horizontal panels set at right angles and supported by struts and props. The typical illustration for this special structural type is shown in Figure 1. There are no beams or columns and these structures generally use all wall elements as primary load carrying members. In this construction technique, the use of pre-cast load carrying members is avoided. The walls

* Correspondence to: Erol Kalkan, Department of Civil Engineering, Rensselaer Polytechnic Institute, JEC 4049, 110 8th St., Troy, NY 12180, U.S.A.
† E-mail: ekalkan@scorec.rpi.edu

Received 22 April 2002
Revised 26 June 2002
Accepted 30 August 2002

Copyright © 2003 John Wiley & Sons, Ltd.

Figure 1. Tunnel form technique and construction system.

and slabs, having almost the same thickness are cast in their place in a single operation. This reduces not only the number of joints, but also the assembly time. Consequently, the casting of walls and slabs can be completed in 1 day for each floor. The simultaneous casting of walls, slabs and cross-walls results in monolithic structures, which provides high seismic performance and, therefore, they meet seismic code requirements of many countries located in regions having high earthquake risk. In addition to their considerable resistance, the speed and ease of building make them preferable as the multi-unit construction of public and residential buildings.

In 1999, two severe urban earthquakes struck the Kocaeli and Düzce provinces in Turkey with magnitudes (Mw) 7.4 and 7.1. These catastrophes caused substantial structural damage, casualties and loss of life. In the aftermath of these destructive earthquakes, neither demolished nor damaged shear-wall dominant buildings constructed by tunnel form techniques were reported. Almost non-damaged conditions of these special structures drew our attention to focus on their dynamic properties.

Within the framework of this study, the consistency of design criteria for these buildings given by current seismic code provisions and building codes are investigated. Despite their high resistance and satisfactory behaviour under earthquake excitations due to their discrete structural and load transferring systems, the general trend is towards their acceptance as conventional R/C frame type shear wall buildings. For that reason, the reliability of given empirical equations, related to defining dynamic properties of these structures, are examined in detail and it is shown that the empirical equations given in the seismic code provisions for prediction of fundamental periods give unreliable results. Therefore, our effort was spent in the derivation of new empirical equations on the basis of their fundamental properties. These

Copyright © 2003 John Wiley & Sons, Ltd.

equations and the values of their predictor parameters were developed by an extensive three-dimensional finite-element analysis of 16 selected different plans for five different building heights (Storey levels: 2,5,10,12,15). The database obtained constitutes the analysis results of 80 different case studies and the calculations of their basic properties. The empirical equations for predicting the fundamental period of tunnel form structures were typically fit to this data set by applying non-linear regression analysis.

The database, analyses and results of the empirical equations complementing this work are summarized in the remaining sections of this paper. With all this available information, this study provides a general methodology for developing estimates of the fundamental period based on specific parameters characterizing the primary structural and architectural properties with associated measures of uncertainty. Finally, this paper makes comparisons for various case studies between the proposed formula and empirical equations given by the Turkish Seismic Code [1] and the UBC [2] and illuminates the reasons for their differences.

DATABASE

In order to obtain a representative database for the analysis, as-built plans are intentionally selected and most of these plans have already been applied. The initial analysis results show that there is a clear difference in the fundamental period of those structures depending on their side ratios. Consequently, the complete database was categorized into two sub-data sets according to the plan dimension ratios. If the ratio of the long-side to short-side dimension is less than 1.5, these plans are accepted as square and those plans having the same ratio greater or equal to 1.5 are accepted as rectangular. These sub-data sets are listed in Tables I and II for rectangular and square plans, respectively. In these tables, side dimensions and shear-wall areas are given for each direction. The distributions of data in these data sets according to various building heights are illustrated in Figure 2.

ANALYTICAL MODELLING OF STRUCTURES

In the analyses part, all structural elements including shear walls and slabs are three dimensionally modelled by using finite-element modelling using shell elements. All elevator and stair case hollows and door openings are considered. Diaphragm flexibility was taken into account without making any rigid-floor assumption. This issue is taken up again with details in the section on structural importance of tunnel form buildings.

As a main part of this study, the three-dimensional finite-element dynamic analysis of 80 different cases performed by using ETABS (Ver. 7.22) [3] and obtained fundamental period results are listed for two district data sets in Tables I and II. Selective example plans in the database for rectangular and square cases are shown in Figures 3–5 to illuminate their architectural and structural concepts. In these figures solid lines demonstrate the shear walls in the plan. The typical three-dimensional mesh model is given in Figure 6 for a five-storey building (Plan No. 2).

Copyright © 2003 John Wiley & Sons, Ltd.

Table I. Rectangular plans* and their architectural properties.

			Plan dimensions (m)		Shear wall area (m^2)		Dynamic analysis
Plan no.	Storey #	Height (m)	Long side	Short side	Long side	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

* Ratio of long-side to short-side dimension is greater or equal to 1.5.

Copyright © 2003 John Wiley & Sons, Ltd.

Table II. Square plans* and their architectural properties.

			Plan dimensions (m)		Shear wall area (m^2)		Dynamic analysis
Plan no.	Storey #	Height (m)	Long side	Short side	Long side	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 less than 1.5.

STRUCTURAL IMPORTANCE OF TUNNEL FORM BUILDINGS

A desirable characteristic in an earthquake-resistant structure is the ability to respond to strong motion by progressively mobilizing the energy-dissipative capacities of an ascending hierarchy of elements making up the structure. In this connection, shear walls, when properly designed, represent economical and effective lateral stiffening elements that can be used to reduce potentially damaging inter-storey drifts in multi-storey structures under earthquake excitations. Their good performance has been demonstrated in a number of recent earthquakes. Therefore, the monolithic structural shape of tunnel form buildings, including only shear walls and slabs as

Copyright © 2003 John Wiley & Sons, Ltd.

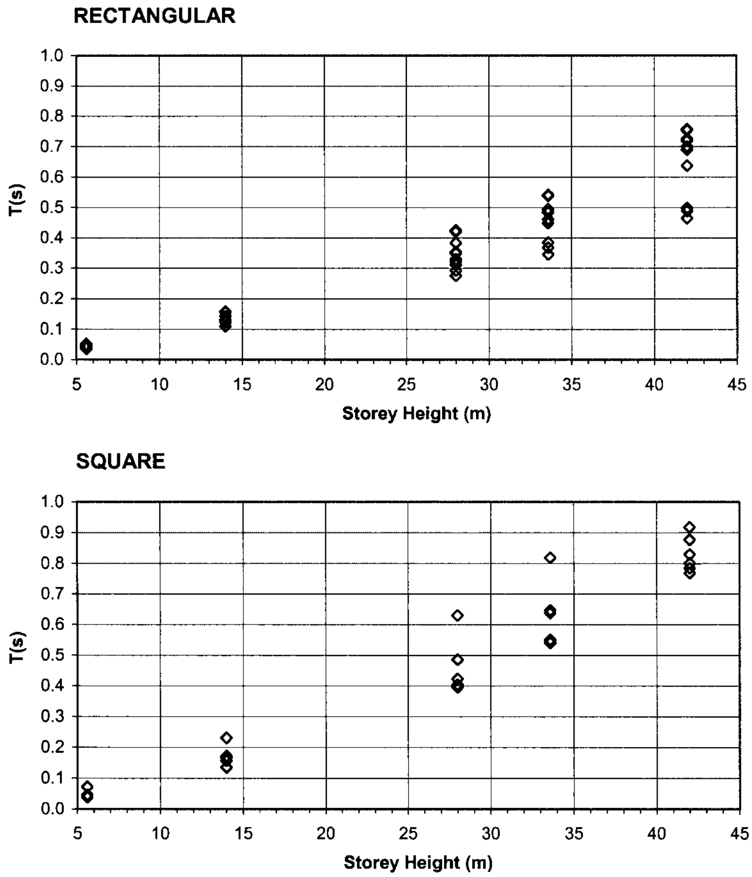

Figure 2. Distribution of rectangular and square plans in the database in terms of fundamental period and storey height.

load carrying and transferring members, provides considerable earthquake resistance. This conclusion was also experienced from the results obtained by three-dimensional non-linear seismic performance evaluations of tunnel form buildings by Balkaya and Kalkan [4]. For these structures, shear walls and slabs have almost the same thickness, less than those of standard building slabs. Therefore, diaphragm flexibility can modify dynamic behaviour considerably, it is recommended to keep the rigid floor assumption out of modelling, this phenomenon is also discussed by Tena-Colunga and Abrams [5], and also Fleischman and Farrow [6].

Transverse walls which are perpendicular to the main walls and the loading direction provide extra resistance and significantly increase the predicted load capacity as a result of tension/compression (T/C) coupling effect produced by in-plane or membrane forces in the walls even though their connection to the main walls is rather loose. In addition to wall-to-wall, wall-to-slab interaction is another issue that develops due to the membrane forces in the slabs. The lateral walls form a system with in-plane walls similar to a typical T-section whose behaviour through its 3D effects is similar to the section above the openings in the

Copyright © 2003 John Wiley & Sons, Ltd.

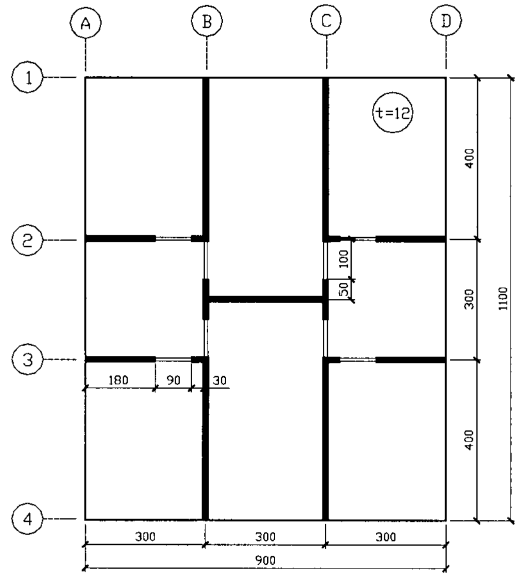

Figure 3. Typical plan view for a square case (Plan No. 11) (units are in cm).

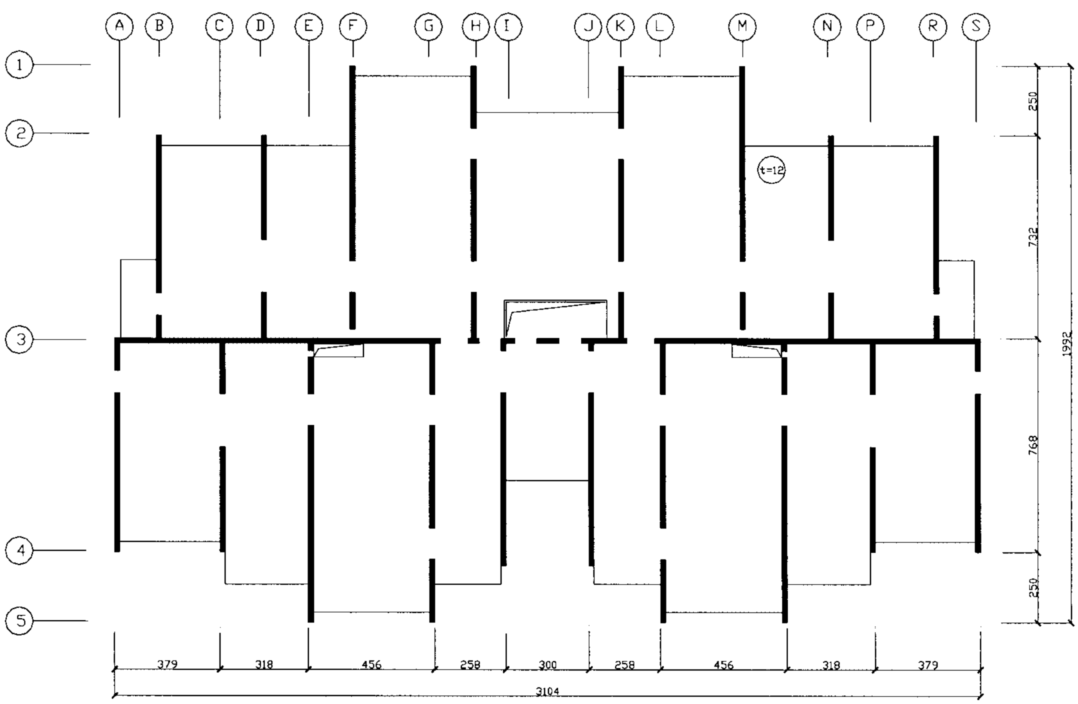

Figure 4. Typical plan view for a rectangular case (Plan No. 2).

Copyright © 2003 John Wiley & Sons, Ltd.

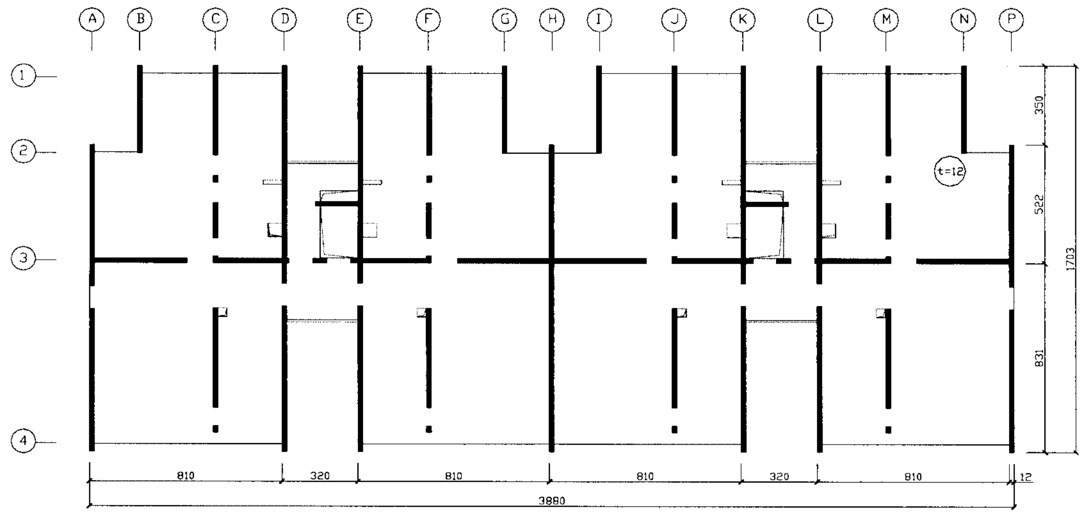

Figure 5. Typical plan view for a rectangular case (Plan No. 3).

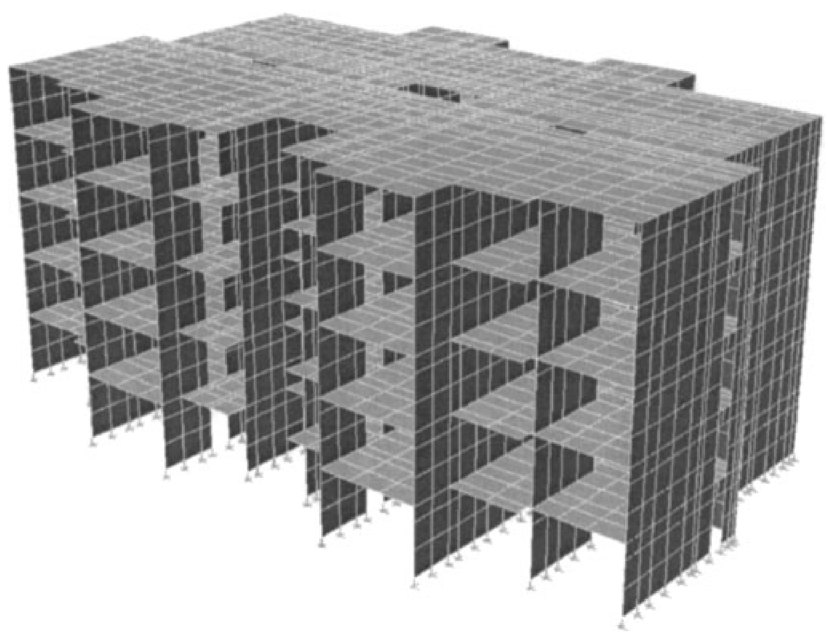

Figure 6. Typical three-dimensional mesh modelling for five-storey tunnel form building structure (Plan No. 2).

Copyright © 2003 John Wiley & Sons, Ltd.

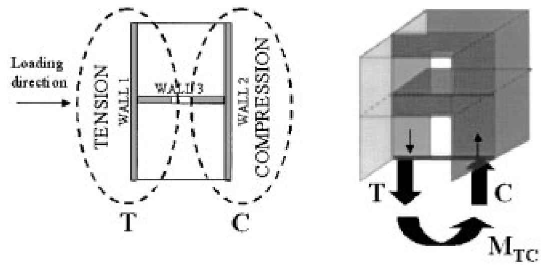

Figure 7. Slap-wall interaction due to tension and compression (T/C) coupling.

walls in the loading direction having a T-section contribution from the floor slabs as shown in Figure 7. This strong coupling occurs in spite of the door openings introducing a strong disruption of the shear flow between walls. The structural systems with these wall-to-wall and wall-to-slab interactions increase their lateral load capacity as well as their performance under earthquake forces (Balkaya and Schnobrich [7]). The analysis results in this study once again prove this judgement since these structures are capable of dissipating a significant amount of energy with tolerable deformations. This situation calls for fairly desirable seismic behaviour in order to keep them in the elastic domain when subjected to strong earthquake motions.

On the other hand, one observed handicap of this special structural type is their torsional behaviour, unless an appropriate side ratio is selected and shear walls are configured properly in the architectural plan. The dynamic analysis results, displaying the first three modes deformed shapes, are shown in Table III. Despite the short and long direction flexures, almost all plans show torsional behaviour under their natural vibration modes. Although the rectangular plan seems better to avoid torsion, when the total bending is taken into account, square plans due to their architectural characteristics show good behaviour. In fact, rectangular plans have weak flexural capacity along their short direction due to the construction and architectural limitations of the tunnel form technique. By considering this phenomenon, the design of those structures to resist the expected loadings is generally aimed to satisfy established or prescribed safety and serviceability criteria. Generally, tunnel form building structures show good performance under seismic forces as studied here. However, close to square architectural plans and symmetrically located shear walls are recommended in order to minimize aforementioned torsional disturbances. Those are the main points observed affecting the fundamental periods of those structures.

PREDICTION EQUATION DEVELOPMENT

Generally, predicted fundamental periods are used to obtain the expected seismic loads coming to structures. For that reason, accurate estimation of $T(s)$ is inevitably essential for the safety of the applied procedure in the next steps of design and consequently for the future performance of the structure in the post-construction period. In order to minimize this deficiency in current seismic codes, new empirical equations were proposed. Empirical equations for the prediction of fundamental periods were established by considering a great deal of alternative formulas by taking into account various structural and architectural parameters. As a result of these intensive exercises and analyses, the fundamental period estimation equation took the

Copyright © 2003 John Wiley & Sons, Ltd.

Table III. Deformed shapes of the first 3 modes according to FEM dynamic analysis results.

Plan	Mode-1	Mode-2	Mode-3
1	Torsion+long dir.	Torsion+short dir.	Torsion+short dir.
2	Long dir.+torsion	Torsion	Short dir.
3	Long dir.	Torsion	Short dir.
4	Short dir.	Torsion	Long dir.
5	Torsion	Short dir.	Long dir.
6	Long dir.	Short dir.	Torsion
7	Short dir.	Torsion	Long dir.
8	Torsion	Short dir.	Long dir.
9	Torsion	Short dir.	Long dir.
10	Long dir.	Torsion	Short dir.
11	Torsion	Short dir.	Long dir.
12	Torsion	Long dir.	Short dir.
13	Torsion	Short dir.	Long dir.
14	Torsion	Short dir.	Long dir.
15	Torsion	Long dir.	Short dir.
16	Torsion	Long dir.	Short dir.

form given below:

$$T = Ch^{b1}\beta^{b2}\rho_{\text{as}}^{b3}\rho_{\text{al}}^{b4}\rho_{\text{min}}^{b5}J^{b6} \tag{1}$$

$$J = I_{xx} + I_{yy} \tag{2}$$

Here, T is the period in s; h is the total height of the building in m; β is the ratio of long-side to short-side dimension; ρ_{as} is the ratio of short-side shear-wall area to total floor area; ρ_{al} is the ratio of long-side shear-wall area to total floor area; ρ_{min} is the ratio of minimum shear wall area to total floor area; C, $b1$, $b2$, $b3$, $b4$, $b5$ and $b6$ are the parameters to be determined by regression analysis; J is the polar moment of inertia of the plan given in Equation (2). The predictor coefficients in Equation (1) were determined by using non-linear regression analysis. Non-linear regression is a method of finding a non-linear model of the relationship between the dependent variable and a set of independent variables. Unlike traditional linear regression, which is restricted to estimating linear models, non-linear regression can estimate models with arbitrary relationships between independent and dependent variables. This exercise was performed separately on two distinct data sets. The coefficients for empirical equations for rectangular and square plans are given in Table IV. The resulting parameters can be used to find the fundamental period of tunnel form buildings over the full range of storey levels (2–15) for square to rectangular plans. The obtained results were also used to compute errors within the process of estimation. The standard deviation of residuals, σ_T, expressing the random variability of periods is almost equal to 0.025 for these two generalized plan shapes.

COMPARISONS WITH CURRENT SEISMIC CODE PROVISIONS

The Turkish Seismic Code concerning construction in seismic areas has been recently modified in 1998. In this code, the equations for predicting fundamental periods of structures were taken directly from the UBC (1997) with small modifications. The mentioned general empirical

Copyright © 2003 John Wiley & Sons, Ltd.

Table IV. Empirical equations for predicting fundamental periods of tunnel form buildings.

$$T = Ch^{b1}\beta^{b2}\rho_{as}^{b3}\rho_{al}^{b4}\rho\min^{b5} j^{b6}$$

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: Period (s)
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

equations prescribed in these provisions are as follows:

$$T = C_t(h_n)^{3/4} \tag{3}$$

Where $C_t = 0.0853$ (0.08) for steel moment-resisting frames, $C_t = 0.0731$ (0.07) for reinforced concrete moment-resisting frames and eccentrically braced frames, and $C_t = 0.0488$ (0.05) for all buildings. Alternatively, the value of C_t for structures where seismic loads are fully resisted by reinforced concrete structural walls, can be taken as $0.0743\ (0.075)/(A_c)^{1/2}$. The numbers within the parentheses show the corresponding values given in the Turkish Seismic Code (1998). The value of A_c shall be calculated from the following formula:

$$A_c = \Sigma A_e[0.2 + (D_e/h_n)^2] \tag{4}$$

The value of D_e/h_n used in Equation (4) shall not exceed 0.9. All calculations and given formulas are in the SI unit system. The estimates equations developed in this study were compared to those equations given by the UBC (1997) and Turkish Seismic Code (1998), and also compared with finite-element analysis results. These comparisons are illustrated for various selective cases from Figures 8 to 11 for plan numbers 2,5,13 and 15, respectively. As is observed from those figures, the obtained finite-element analysis results significantly differ from the code-referred values, whereas the general good agreement between all these curves gives support to estimated fundamental periods obtained from recommended equations. The significant deviation between current code given formulas and finite-element analysis leads to intolerable errors for the dynamic parameters and corresponding design loads. Generally, performing linear or non-linear detailed three-dimensional finite-element analysis for this structural type is difficult due to the existence of dominant shear-wall configurations and most of the time not conducted for design purposes. For practical applications code given simplified formulas are widely preferred. In order to compensate this error by considering the three-dimensional behaviour, a set of new equations is recommended.

LIMITATIONS AND UNCERTAINTIES

Uncertainty is a condition associated with essentially all aspects of earthquake-related science and engineering. In this study, principle sources of uncertainties are involved in the applied

Copyright © 2003 John Wiley & Sons, Ltd.

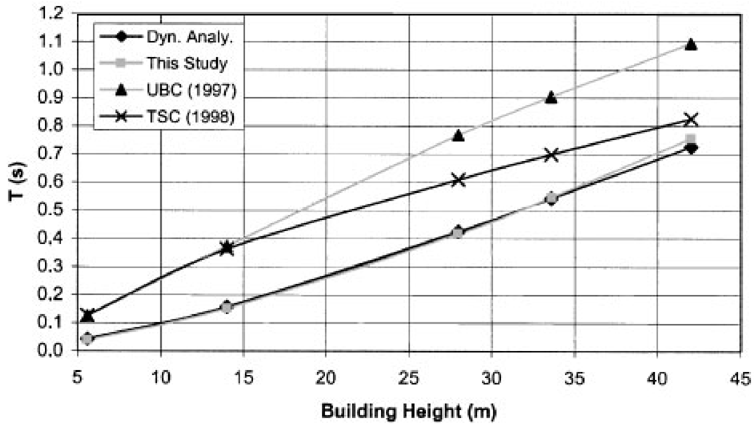

Figure 8. Comparisons of fundamental periods in terms of various building heights for selected rectangular case (Plan No. 2).

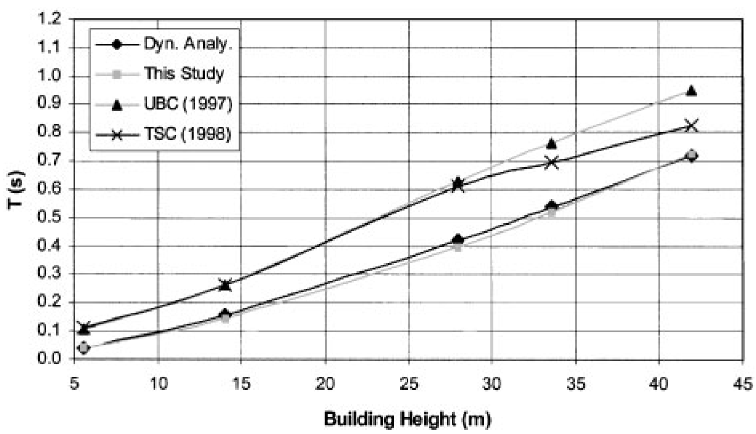

Figure 9. Comparisons of fundamental periods in terms of various building heights for selected rectangular case (Plan No. 5).

stochastic analysis methods, dynamic modelling of buildings, and strength and deformation capacities of elements and structures. However, through the use of non-linear regression analysis, it provides a more sophisticated and direct approach to address the uncertainties than do traditional linear analysis procedures. Part of these uncertainties in the analysis results were induced due to the performed evaluation procedures and tools.

For a given empirical equation, results are very sensitive to parameters used for the building height, shear-wall ratio in the plan and polar moment of inertia, it is found that those parameters have more considerable effects on the period of the building, as well as on its general response than the other unpronounced factors. The results we have presented in tab-

Copyright © 2003 John Wiley & Sons, Ltd.

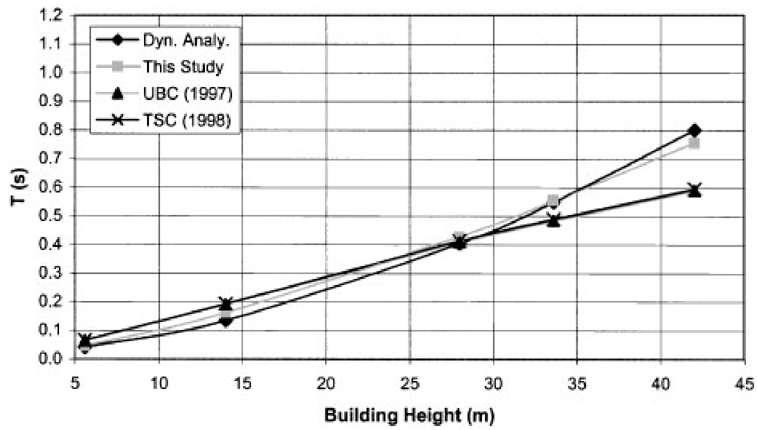

Figure 10. Comparisons of fundamental periods in terms of various building heights for selected square case (Plan No. 13).

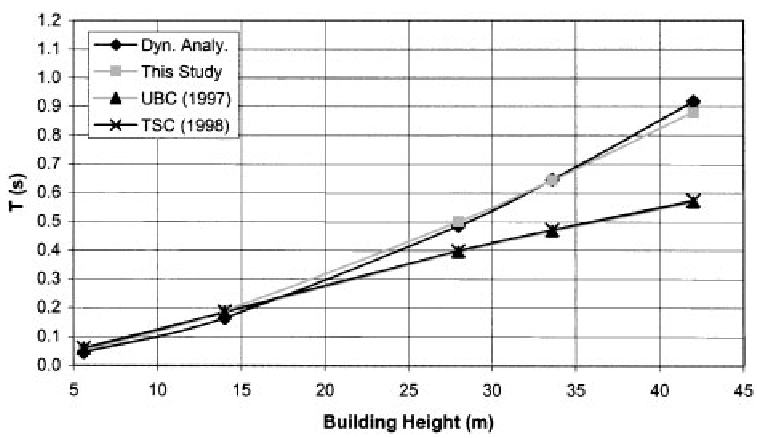

Figure 11. Comparisons of fundamental periods in terms of various building heights for selected square case (Plan No. 15).

ular and graphical form become meaningful only in the context of the error distributions that are associated with each variable. Generally our results possess a maximum 15% deviation in comparison to finite-element analysis results. This is plausible because of the limited number of more-pronounced parameters from which they have been derived.

DISCUSSION AND CONCLUSIONS

Estimation of earthquake forces generally by using design spectra presented in current seismic code provisions requires either implicitly the use of empirical equations for determination of

Copyright © 2003 John Wiley & Sons, Ltd.

the fundamental period of a structure or more specifically detailed dynamic analysis. In this study, the consistency of empirical equations in current seismic code provisions related to dynamic properties of shear-wall dominant buildings constructed by using tunnel form techniques are investigated. It is demonstrated that current earthquake codes overestimate the performed finite-element analysis results for rectangular plans and most of the time underestimate them for square plans. This is most likely due to ignorance of torsional disturbance as a parameter in the code-given predictive equations.

Actually, torsion is an exceptionally important criteria appearing in the dynamic mode of those structures that should be taken into account for the design. It is to be expected that this phenomenon is the result of tunnel form construction restrictions, since part of the outside walls should be opened in order to take the formwork back after the casting process. For that reason, these buildings may behave like thin-wall-tubular structures where torsional rigidity is low. Another important issue that must be mentioned is the bending capacity of these structures; generally rectangular plans have weaker bending capacity along their short sides than that of square plans due to their architectural and constructional limitations. The designer should be aware of these observed handicaps.

The recommended empirical equations presented in detail in this paper through Table IV are considered to be appropriate for the estimation of the period of tunnel form building structures for 2–15-storey levels with various architectural configurations. The results of the proposed equations agree well with finite-element analysis results, and are consistent with the expectation level of increasing vibration period for increasing roof height and decreasing lateral stiffness of structures, therefore, they can be preferred in order to calculate well-grounded seismic loads from current design spectra.

It should be noted that the proposed equations in this paper, which to date are empirical in nature, are based on a general consensus of engineering applications. Pending the accumulation of new data from the analysis of different as-built plan configurations, the derived equations in this study can be progressively modified and improved, and their uncertainties reduced. Soil–structure interaction and foundation effects will also be included in further studies.

The intent of this study was to bring the good performance of these structures forward and recommend new empirical equations for the purpose of revising seismic code provisions. It is more desirable to have detailed guidelines related to their design and construction conditions in the near future.

REFERENCES

1. Ministry of Public Works and Settlement. *Specification for Structures to be Built in Disaster Areas*, Ankara, Turkey, 1998.
2. Uniform Building Code. *International Conference of Building Officials*, Whittier, CA, 1997.
3. Computer and Structures Inc. ETABS Ver. 7.22, Berkeley, CA, 2001.
4. Balkaya C, Kalkan E. Nonlinear seismic response evaluation of tunnel form building structures. *Computers and Structures* 2002; in press.
5. Tena-Colunga A, Abrams DP. Seismic behavior of structures with flexible diaphragms. *Journal of Structural Engineering* (ASCE) 1996; **122**(4): 439–445.
6. Fleischman RB, Farrow KT. Dynamic behavior of perimeter lateral system structures with flexible diaphragms. *Earthquake Engineering and Structural Dynamics* 2001; **30**:745–763.
7. Balkaya C, Schnobrich WC. Nonlinear 3-D behavior of shear-wall dominant RC building structures. *Structural Engineering and Mechanics* 1993; **1**:1–16.

Copyright © 2003 John Wiley & Sons, Ltd.