DAMAGE CONTROL INDICES FOR REINFORCED CONCRETE BUILDINGS

BETONARME BİNALARDA HASAR KONTROL ENDEKSLERİ

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ABSTRACT

The interstorey drift limitations in earthquake codes of various countries around the world, have been reviewed. It is emphasized that, the aseismic safety of a reinforced concrete building as well as its susceptibility to nonstructural damages are primarily indexed to its ability of restricting the relative storey displacements, in addition to its adequate strength, ductility, and toughness.

A moment resisting frame building satisfying all requirements of strength and ductility may still be vulnerable to severe nonstructural damages, if the interstorey drifts are not restricted properly by means of shear walls.

The use of shear walls in buildings of any height is a very effective method of restricting the interstorey drifts thereby providing safety against excessive damages to nonstructural elements. A damage control index is introduced to be conscientiously determined and checked during the design calculations for the purpose of controlling damages to nonstructural elements.

Three real example buildings have been discussed to illustrate the use and importance of the damage control index values.

ÖZET

Çeşitli ülkelerin deprem yönetmeliklerindeki relatif kat ötelenmesi oranlarına ait sınırlamalar gözden geçirilmiştir. Betonarme binalarda deprem güvencesi ve özellikle ikinci derece elemanların hasar görme ihtimali, mukavemet, düktilite ve enerji yutabilme gibi yeteneklerin yanı sıra, relatif kat ötelenmelerindeki sınırlama ile çok yakından ilişkilidir.

Mukavemet ve düktilitesi çok yüksek bir betonarme çerçeve sistemde, eğer relatif kat ötelenmeleri perdelerle yeterince sınırlandırılmamışsa, depremde ikinci derece elemanlar çok ağır hasar görebilir.

Deprem perdelerinin kullanılması, relatif kat ötelenmelerini sınırlayan dolayısı ile, taşıyıcı olmayan yapı elemanlarına gelebilecek ağır hasarı önleyen en etkili bir metoddur. Bu çalışmada, yapısal olmayan hasarları kontrol altında tutabilecek ve tasarım sırasında bilinçli bir şekilde hesaplanacak olan bir hasar kontrol endeksi tarif edilmiştir.

Hasar kontrol endeksinin etkinliğini ve kullanılma tekniğini açıklamak üzere, deprem tecrübesi geçirmiş üç ayrı betonarme bina örnek olarak incelenmiştir.

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1. INTRODUCTION

1.1 Design Philosophy

Unless a building is equipped with a passive and/or active control system, the universal philosophy of aseismic design is based on assuring that the building will not fail or collapse structurally during the most severe earthquake likely to occur within the economic life span of the structure.

This universal philosophy however, inadvertently allows for major nonstructural damages. Sometimes the buildings have to be demolished after the earthquake, on account of the excessive damages to nonstructural elements.

Although, the lateral load carrying system itself is not expected to collapse as a whole, the structural elements, especially the beam-column joints are permitted to undergo minor damages to the extent of developing plastic hinges. But, what happens to the nonstructural elements, such as infill walls, false ceilings, partition walls, plasters, finishing materials, facade elements, elevators, etc. is not much of a concern of the designer.

1.2 Damage Control in Shear Buildings

There are no specific restrictions or governing constraints in the earthquake code provisions ensuring that the damages to nonstructural elements will not be extensive. It is only by experience gained from the past earthquakes that engineers have significant confidence in shear walls to avoid excessive damages to nonstructural elements. Various publications exist strongly advocating the use of shear walls and/or box systems for the purpose of controlling the secondary damages *[1], [2], [3], [4]*. In fact, based on his conscientious observations on a multitude of reinforced concrete buildings in the past earthquakes, *Fintel [3]* concludes one of his articles by saying that;

> *"...Safety against collapse has been the major preoccupation of earthquake engineering. In addition to safety, damage control should be our major goal.*
>
> *Judging from the behaviour of multistorey concrete buildings in earthquakes, it seems that to achieve damage control the ductile shear wall may be the most logical solution.*
>
> *Aetually, from observations in earthquakes, it seems that we can no longer afford to build our multistorey buildings without shear walls."*

1.3 Interstorey Drift Limitations

It is true that there are some limiting requirements for the interstorey deflections of buildings in the earthquake regulations of almost all countries in the world. But, these requirements are not sufficient to control the extent of damages to nonstructural elements, neither these limitations are intended to divert the structural system from moment resisting frames to shear walls.

A moment resisting reinforced concrete frame structure may very well satisfy all strength and ductility requirements as well as the interstorey drift limitations, but the extent of secondary damages during a strong earthquake may be so high that the structure may have to be demolished on account of the high costs of repair and rehabilitation.

If however, the same building is designed to carry the lateral loads largely by means of a shear wall system, the structural and nonstructural damages may be so minor that the building may be readily put into service immediately after the earthquake.

1.4 Managua, Nicaragua Earthquake

The Bank of America, an 18-storey reinforced concrete building, consisting a rigid core of shear walls, was put into service, immediately after the December 23, 1972 Managua, Nicaragua earthquake *(M = 6.5)*. There were no structural or nonstructural damages, except the cracking of a few coupling beams between the elevator shafts and spalling of the marble covers at a few places. This shear building was designed for a base shear of about 7 percent gravity.

Contrary to such a good performance, the 15-storey Banco Central de Nicaragua building, consisting of ductile moment resisting space frames was located diagonally across the corner of the same street and the top 12-storeys had to be demolished, because the nonstructural damages were too extensive *[1], [3]*. The frame building was designed for a base shear coefficient of 10 percent gravity.

1.5 Erzincan Earthquake

Similarly, the Yayla Apartment Block, a 5-storey high residential building composed of moment resisting frames suffered minor structural, but major nonstructural damages during the March 13, 1992 Erzincan, Turkey earthquake. The building was originally designed to meet fully the strength and interstorey drift requirements of the current Turkish Earthquake Code *(1975 TDY) [5]*. In order to strengthen this particular building, along with 350 other apartment blocks, in addition to repairing the plastic hinges at beam-column connections of the first floor, new shear walls have been added in two principal directions. The details of this strengthening are presented in a paper by *Ulker, et, al [6]*.

Based on various analytical investigations about the influence of lateral storey displacements on the extent of damages on real example buildings, some specific recommendations have been made in this paper, concerning the upper limits of interstorey drifts.

2. EARTHQUAKE CODE REQUIREMENTS FOR DEFLECTIONS

2.1 Base Shear Coefficient

The total lateral roof displacement, as well as the relative storey displacements of reinforced concrete buildings are restricted in the earthquake codes normally not to exceed certain upper limits. The limiting interstorey drift values recommended by the earthquake codes of various countries around the world *[7]* are summarised in ***Table 1***.

For the purpose of comparing the recommended level of lateral loads, the base shear coefficient, C, in percent of gravity, is also included in this table, for a typical reinforced concrete building. The C-values correspond to the highest earthquake hazard zone, hard soil, rigid low-rise building with $T \leq 0.2$ seconds and ductile moment resisting frame conditions.

2.2 Storey Drift Limitations

The *storey drift*, is defined as the difference of maximum elastic lateral displacements of any two adjacent floors, divided by the respective storey height. If there are any torsional action in the building, the influence of torsion should be taken into account in calculating the largest lateral deflections.

TABLE 1. – DEFLECTION RESTRICTIONS IN EARTHQUAKE CODES

Country	Year	Ref.	Base Shear Coef. C	$s_{max} = \frac{\delta_e}{h}$ Walls Integral	Walls Isolated	R $\delta_{ep} = R.\delta_e$ M	D	S	δ_{ep} cm	d_N [c]	Damage Index $i_{max} = \frac{s}{C} \cdot 10^4$
Cuba	1964	[7]	0.040	0.0020	0.0040	-	-	-	-	-	500
El Salvador	1966	[7]	0.072	0.0020	0.0040	-	-	-	-	-	278
Chile	1972	[7]	0.080	0.0020	0.0040	-	-	-	-	-	250
Turkey	1975	[7]	0.080	0.0025	-	-	-	-	-	-	313
Israel	1975	[7]	0.200	-	-	-	-	-	-	0.001H	-
Australia	1979	[7]	0.034	0.0050/R	-	1.5	1.3	1.0	-	-	986
Japan	1981	[7]	0.200	0.0050	0.0083	-	-	-	-	-	250
Colombia	1981	[7]	0.178	0.0150/R	-	11.2	6.5	5.0	-	-	76
Yugoslavia	1982	[7]	0.100	0.0029	-	-	-	-	-	0.0017H	290
Peru	1982	[7]	0.067	0.0100/R	0.0150/R	4.5	3.8	2.3	-	-	333
Venezuela	1982	[7]	0.089	0.0180/R	0.0240/R	6.0	5.0	4.0	-	-	337
Indonesia	1983	[7]	0.090	0.0050/R	-	1.0	1.0	0.8	2	-	556
Ethiopia	1983	[7]	0.075	0.0050/R	-	3.0	3.0	3.0	-	-	222
New Zealand	1984	[7]	0.096	0.0060/R	0.0100/R	3.1	2.5	2.5	-	-	200
India	1984	[7]	0.080	0.0040	-	-	-	-	-	-	500
UBC H<20m	1988	[7]	0.092	0.0400/R	-	12.0	12.0	8.0	-	-	362
			0.092	0.0050	-	12.0	12.0	8.0	-	-	543
UBC H>20m	1988	[7]	0.092	0.0300/R	-	-	-	-	-	-	271
			0.092	0.0040	-	12.0	12.0	8.0	-	-	435
Iran	1988	[7]	0.140	0.0050	-	-	-	-	-	-	357
Egypt	1988	[7]	0.054	0.0050/R	-	3.0	3.0	3.0	2	-	311
EUROCODE	1988		-	0.0020	0.0060	1.0	1.0	1.0	-	-	-
ATC/NEHRP	1991	[9]	0.180	0.0150/R	-	5.5	6.5	5.0	-	-	151
Romania	1992	[10]	0.160	0.0035/R	0.0070/R	5.0	4.0	4.0	-	-	44
Turkey	1995	[11]	0.125	0.0200/R	-	8.0	7.0	6.0	-	-	200
			0.125	0.0035	-	-	-	-	-	-	280

M = Moment Resisting Frame ; *D* = Dual System ; *S* = Shear Wall System

In terms of elastic deflections, the storey drift, s, is given by

$$s = \delta_e / h \tag{1}$$

$$\delta_e = d_{n+1} - d_n \tag{2}$$

in which, d_n = maximum elastic deflection of the n' th floor, δ_e = the difference of the elastic deflections between the two neighbouring floors, h = storey height.

Based on the type of connection of the nonstructural elements, such as infill walls, partition walls, etc. to the main structural system, the storey drift, s_m, has two different upper limitations as follows:

a) Integral Connections

A relatively lower limit of storey drift is defined for the case when the nonstructural elements are integrally attached to the structure, in which the nonstructural elements are susceptible to severe damages on account of the deformations of the structure. For this type of connection, the storey drift limitation, s_m, varies between 0.002 and 0.005.

b) Isolated Connections

When the nonstructural elements are isolated from and not integrally connected to the main structure, however, they are not affected by the lateral vibrations of the building, and as such are not susceptible to severe damages. Then relatively higher values of storey drift limitations, s_m, are used to vary between 0.004 and 0.008.

2.3 Elasto-Plastic Deflections

In some earthquake codes, very high values of storey drift limitations are specified corresponding to the elasto-plastic action of the structure. For the purpose of uniformity and normalisation in comparison, these elasto-plastic storey drift limitations have been divided by the respective structural factor, R, which represents in a way, the ratio of the elasto-plastic deflections to the elastic deflections.

Therefore, the upper limits of storey drift, s_m, separately listed in ***Table 1***, for a) the integrally connected, and b) the isolated cases, are to be tested only against the elastic deflections. The structural factor, R, is also listed in ***Table 1*** for a) ductile moment resisting frames *(M)*, b) dual system of frames and shear walls *(D)*, in which the frame alone is capable of resisting 25 percent of the lateral shears, and c) shear wall systems *(S)*.

2.4 Maximum Interstorey Deflection

In the earthquake codes of Indonesia and Egypt, the upper limit of the maximum relative elasto-plastic deflection, δ_{ep}, between any two adjacent floors, is restricted to be less than 2 centimeters. That is,

$$\delta_{ep} = R\,\delta_e \leq 0.02\,m \tag{3}$$

in which, δ_e = the difference of elastic displacements between any two adjacent floors. The structural coefficient, R, is given as $R = 1$ in Indonesia, but $R = 3$ in Egypt. In numerical calculations of the example structures in this paper $R = 5$ has been proposed, which means

$$\delta_e \leq 0.0040\,m \tag{4}$$

2.5 Maximum Roof Deflection

The maximum elastic deflection at the top of a building is restricted not to exceed a certain percentage of the building height as follows:

$d_N \leq 0.0010\,H$	*Israel, 1975*	*[7]*
$d_N \leq 0.0017\,H$	*Yugoslavia, 1982*	*[7]*
$d_N \leq 0.0007\,H$	*M. Fintel, 1973*	*[1]*

in which, d_N = the top floor elastic deflection of a building, N = total number of storeys, H = total height of the building.

It is seen that the limitation on the roof deflection is specified only in the earthquake codes of Israel and former Yugoslavia. Based on his conscientious observations, in the past earthquakes of the Central America, *Fintel [1]* recommended rather a very low limit for the maximum roof deflection. As will be illustrated in the three example buildings, the writer is in full agreement with *Fintel*'s recommendation. Unfortunately, this important deflection control index is not yet included in the earthquake provisions of the majority of countries around the world.

3. DAMAGE CONTROL INDEX

The storey drift limitation, s_m, specified almost in all earthquake codes, appears to be the only parameter intended to control directly the degree of damages to occur to nonstructural elements. In practice however, this parameter seldom becomes a governing criterium in affecting the design.

Further, the storey drift limit, s_m, is introduced for testing only against the elastic deflections. If the elastic design loads are relatively small, then the storey drifts of the building are calculated also to be relatively small. Therefore, it would be logically advisable to divide the upper limits of the storey drift, s_m, by the level of the elastic design loads, so as to obtain a normalisation among the earthquake regulations of various countries.

For this purpose, the maximum storey drift values, s_m, for each country have been divided by the respective maximum base shear coefficient, C_m, calculated as described in ***Paragraph 2.1*** above, and the resulting parameter,

$$i_m = 10^4 \; (s_m \; / \; C_m) \tag{5}$$

is defined as the *damage control index*, and listed in the last column of ***Table 1***. It is not a surprise that, similar to storey drift limitations, the damage control index, i_m, also varies within a very wide range. The lowest value is $i_m = 44$ in Romania, and the highest value is $i_m = 986$ in Australia, as illustrated in ***Fig. 1***.

The damage control index values in most countries, however, range within a relatively narrower band between 200 and 500. It is seen that, the earthquake codes around the world are not in an easy agreement with each other in respect to the storey drift or damage control index limitations, despite the fact that the elastic deflections are normalised.

In the current Turkish Earthquake Code *[5]*, the storey drift is limited to $s_m = 0.0025$ and the resulting damage control index is

$$i_m = 10^4 \; s_m \; / \; C_m = 10^4 \, (0.0025) \, / \, 0.08 = 312 \tag{6}$$

In the proposed draft code *[11]* however, the storey drift limitation, for a moment resisting frame *(R = 8)*, is $s_m = 0.02/R = 0.0025$ and the damage control index is

$$i_m = 10^4 \; s_m \; / \; C_m = 10^4 \; (0.0025) \; / \; 0.125 = 200 \tag{7}$$

Based on the numerical computations on various real example buildings, an effective range of damage control index, i_m , is recommended for use in practical design, in subsequent parts of this paper.

4. *BURDUR* HIGH SCHOOL BUILDING

4.1 Earthquake Damages

The high school building shown in ***Fig. 2***, was under construction at the time of the May 12, 1971 Burdur earthquake *(M = 6.2)*. Due to heavy structural damages, which occured at the basement and ground floor, this four storey frame building had to be demolished after the earthquake *[12]*.

The floor plan is given in ***Fig. 3***. The high school was one of the typical schools constructed elsewhere in Turkey and was designed and constructed under the strict supervision of the Ministry of Public Works. The typical structural system has been drastically revised, and extensive shear walls have been added in both directions, after this particular building was damaged during the Burdur earthquake.

4.2 Addition of Shear Walls

The lateral deflections, storey drifts a well as the damage control index values of the building frame along the axis *No. 15*, before and after the addition of shear walls are all listed in ***Table 2***. The base shear coefficient is assumed to be $C = 0.06$, as recommended by the Turkish Earthquake Code, valid at that time. The influence of infill walls are neglected in calculating the lateral rigidities. The modulus of elasticity of the concrete is assumed as $E = 21\ 000\ MPa$.

The floor area ratios of the columns were 1.6 percent and 1.2 percent in the basement and ground floors, respectively. The areas of shear walls added to the structure have been 1.2 percent and 1.6 percent in the X, and Y-directions, respectively.

5. *YAYLA* APARTMENT BLOCK IN ERZINCAN

5.1 Earthquake Damages

It is reported that about 3 290 residential units have been moderately damaged during the March 13, 1992 Erzincan earthquake *[13]*. The Ministry of Public Works, in collaboration with the Technical University of Istanbul, repaired and strengthened about 2 000 such residential units, through some disaster contingency funds and long term credit arrangements to the owners *[6]*.

The Yayla Apartment Block, shown in ***Fig. 4***, was one of the several hundred other buildings, which experienced mild structural damages *[14]*. The building is five storey high and consists of moment resisting frames in both transversal and longitudinal directions as outlined in the typical floor plan shown in ***Fig. 5***. No shear walls are included in the system.

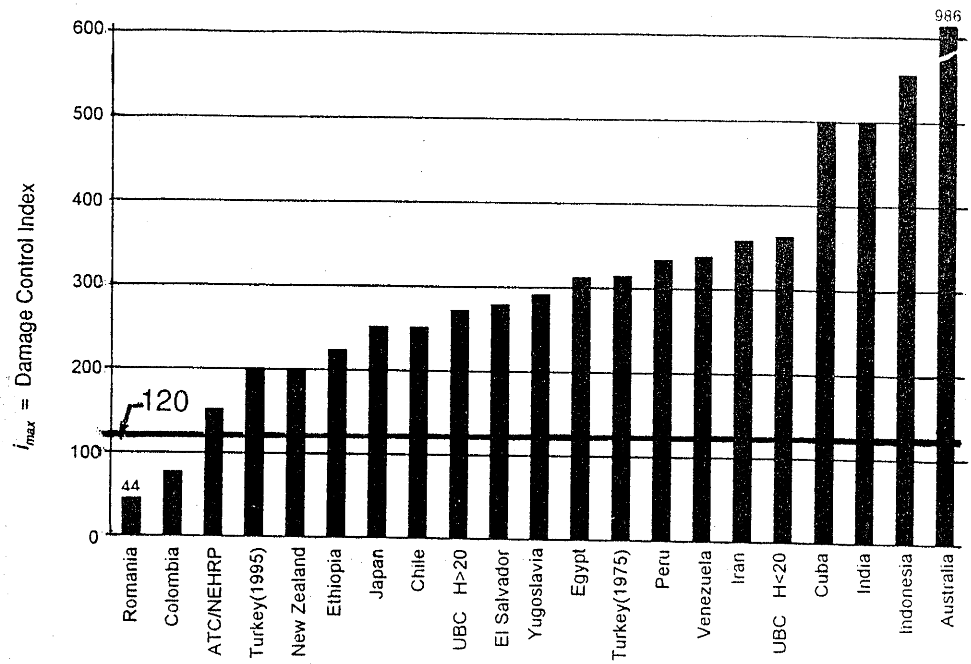

FIGURE 1. – **DAMAGE CONTROL INDEX BASED ON EARTHQUAKE CODES**

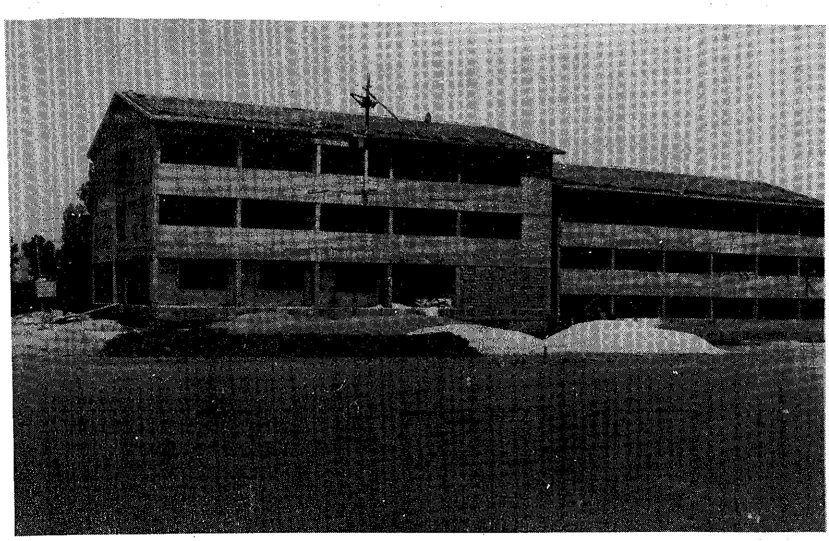

FIGURE 2. – **HIGH SCHOOL BUILDING**
(May 12, 1971 Burdur, Turkey)

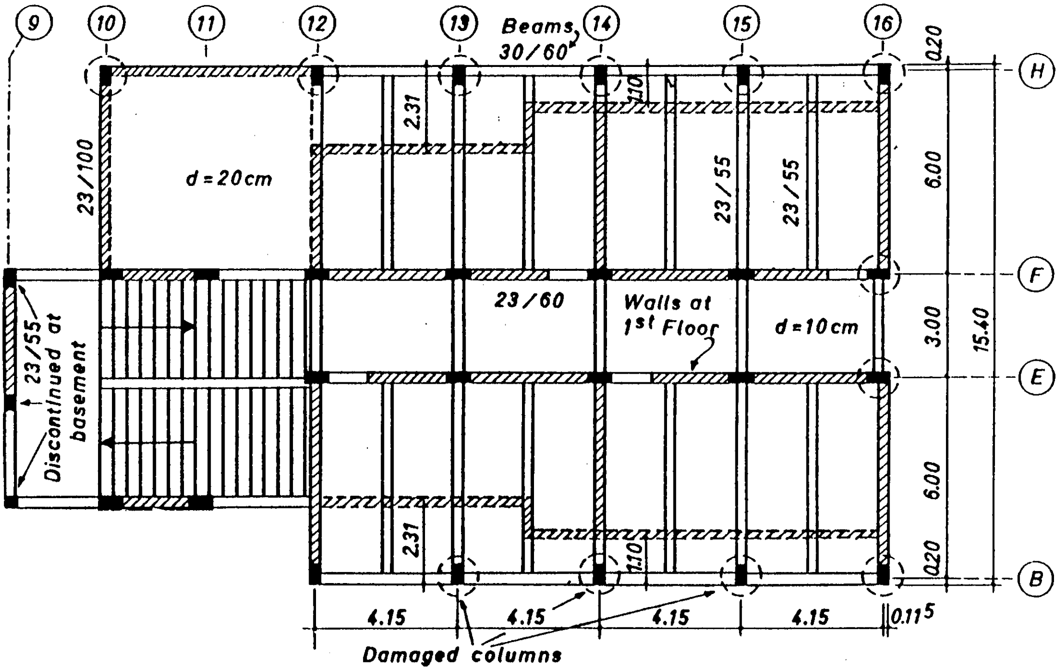

FIGURE 3. – **FLOOR PLAN - HIGH SCHOOL BUILDING**

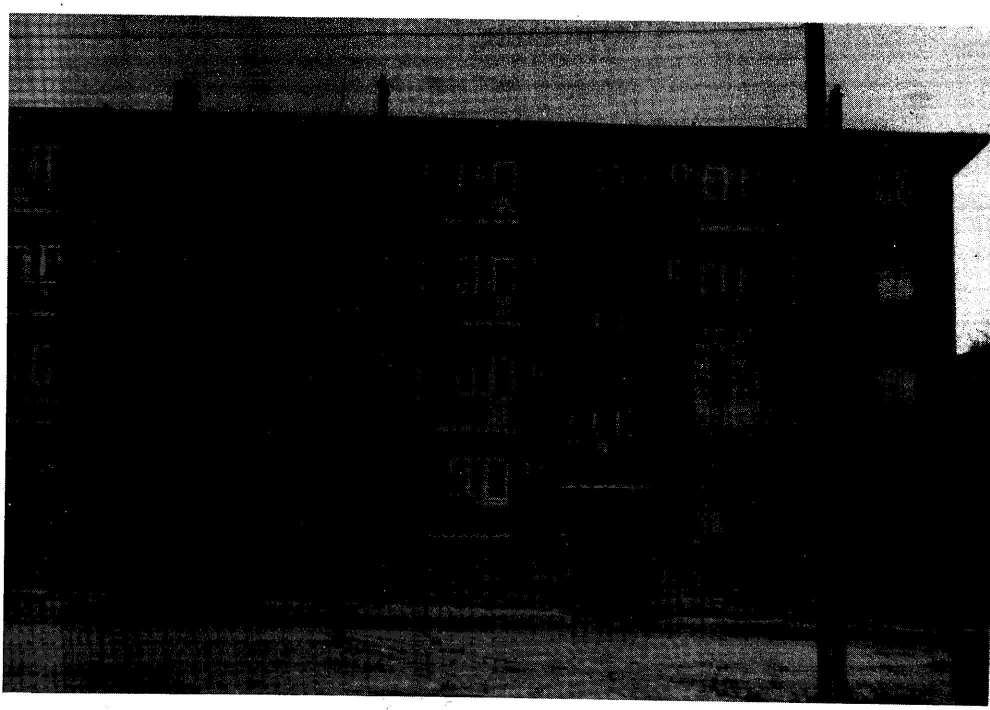

FIGURE 4. – **YAYLA APARTMENT BLOCK**
(March 13, 1992 Erzincan, Turkey)

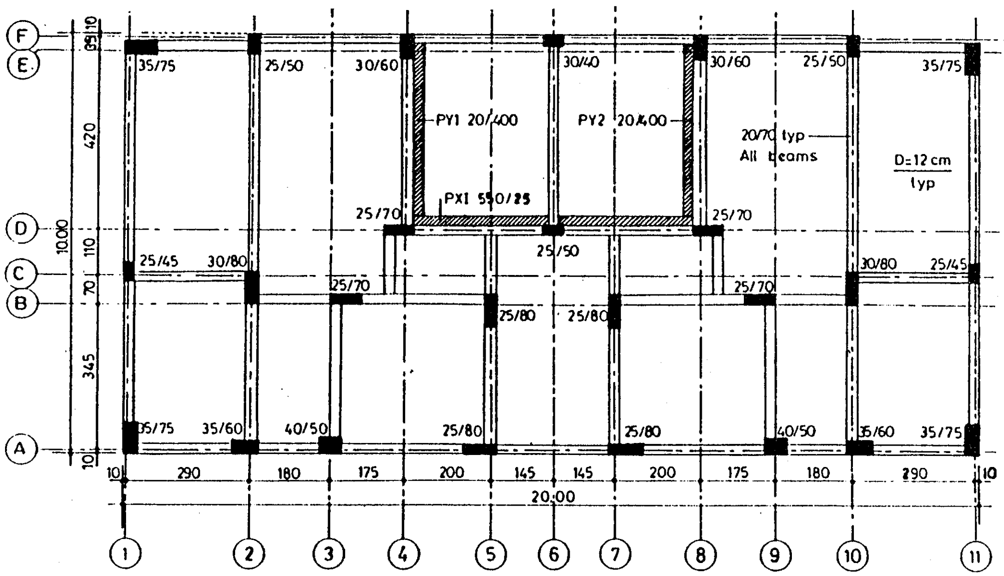

FIGURE 5. – **FLOOR PLAN - *YAYLA* APARTMENTS**

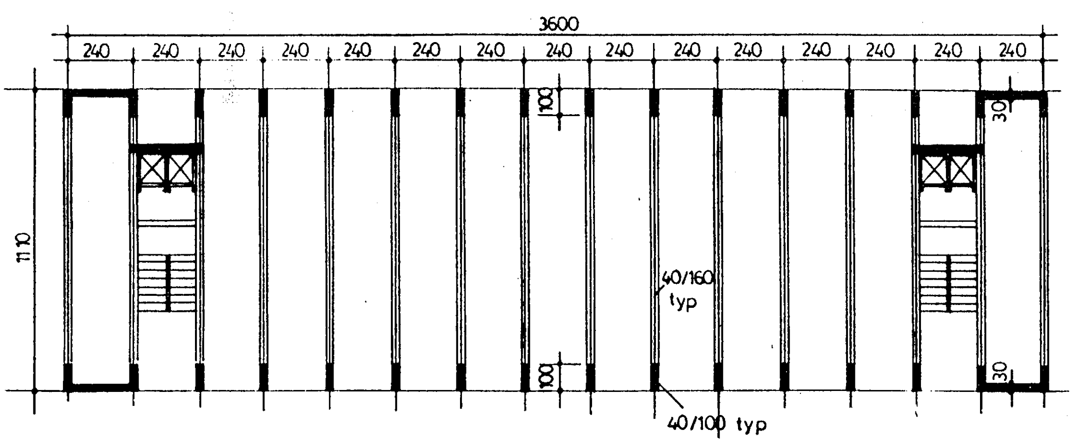

FIGURE 6. – **FLOOR PLAN - FRAME BUILDING *(20-Storey)***

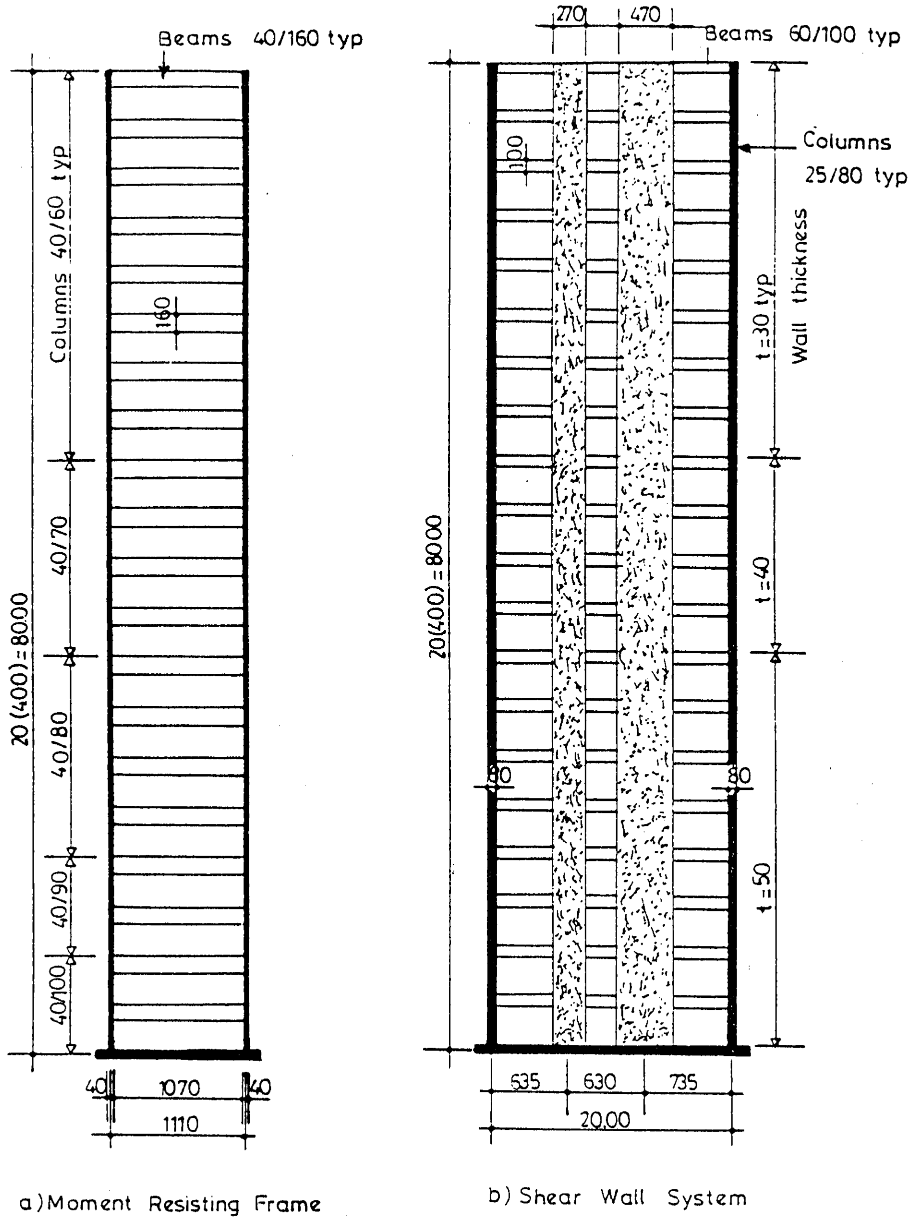

FIGURE 7. – **FLOOR PLAN - SHEAR BUILDING** ***(20-Storey)***

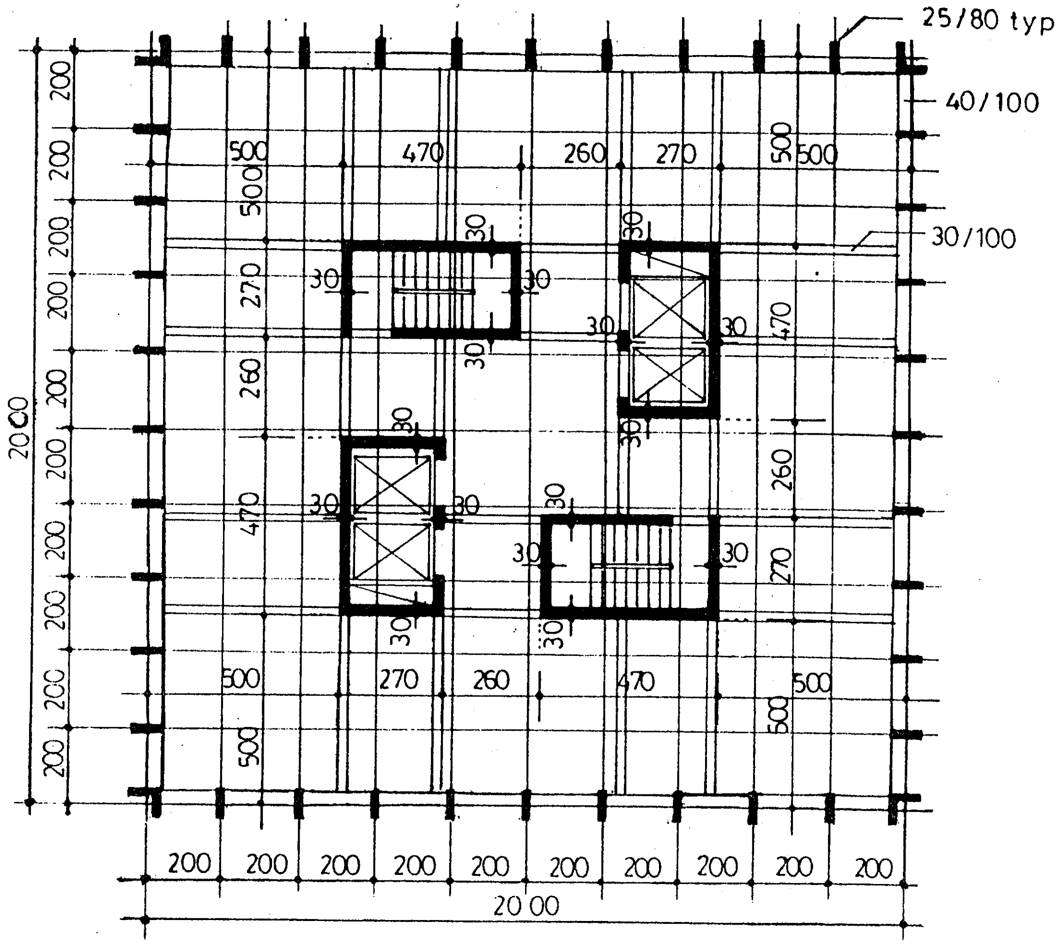

FIGURE 8. – **ELEVATIONS** ***(20-Storey Building)***

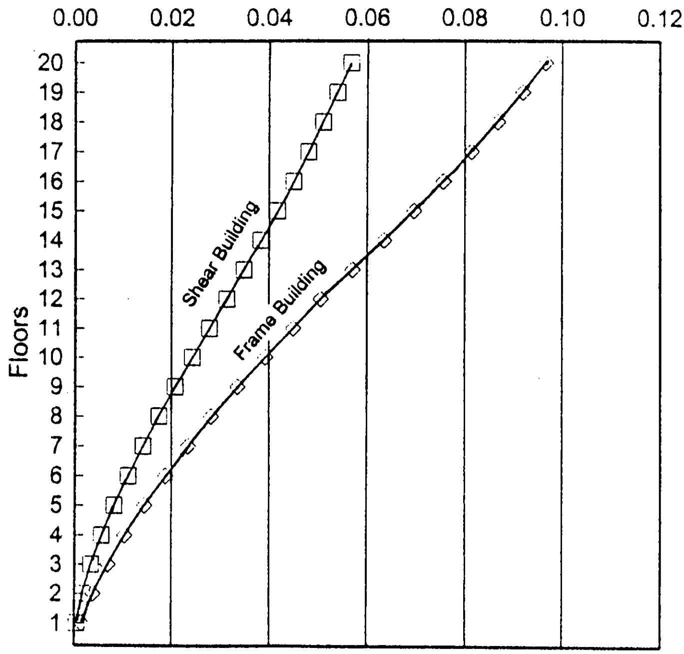

FIGURE 9. – **LATERAL DISPLACEMENTS** *(Meter)*

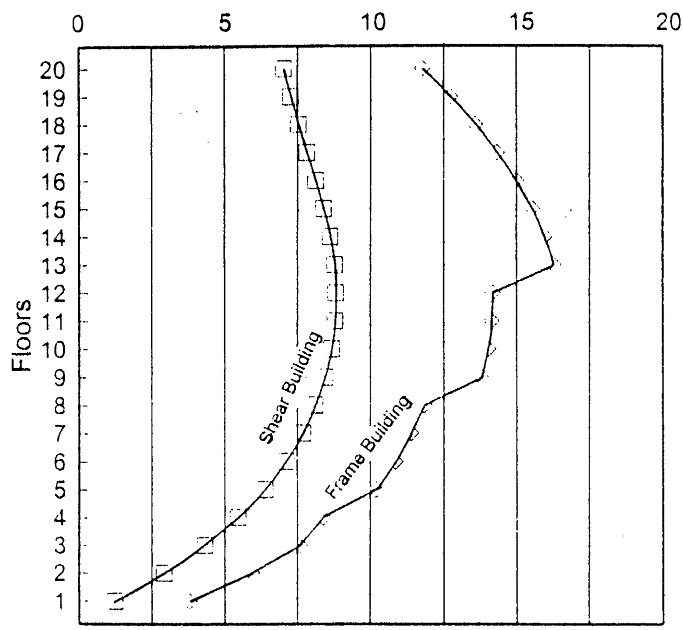

FIGURE 10. – **STOREY DRIFTS** *(10^{-4})*

5.2 Strengthening by Shear Walls

Immediately after the earthquake, the structural damages have been fully repaired. The cracks have been filled in with pressurised epoxy injections. The beam-column connections have been repaired first by using a special repair mortar, then galvanised steel laminates have been wrapped around the joints. In order to strengthen the building against future earthquakes however, shear walls have been added in both directions from the foundation level to the roof. The base shear coefficient was $C = 0.08$ for the frame structure, and $C = 0.15$ for the frame and shear wall structure *[15]*. However, for the purpose of uniformity in comparisons, the lateral deflections, storey drifts and the damage control index values have been calculated for both frame and shear building conditions, assuming $C = 0.08$, and the results have been summarised in ***Table 3***.

It is seen that the deflections are reduced significantly when the building is strengthened by means of shear walls. The deflections in the transversal direction *(Y-axis)* are relatively larger than the deflections in the longitudinal direction *(X-axis)*. Therefore, the results pertaining only the *Y*-direction are listed. The maximum values of deflections have been indicated in bold characters in ***Table 3***. The ratio of cross-sectional areas of columns at ground floor level, to the total area of floor plan was *(4.93/200)* = 2.46 percent. The floor area ratios of shear walls have been 0.7 percent and 0.8 percent in the *X* and *Y*-directions, respectively. The modulus of elasticity is assumed to be $E = 32\,500$ *MPa*.

6. A 20-STOREY BUILDING EXAMPLE

6.1 Structural Data

In order to simulate the performances of the Banco de Central Nicaragua and the Banco de America buildings, during the December 23, 1972 Managua earthquake, a 20-storey building example has been selected. Keeping the floor area to be the same as $A = 400$ square meters, two different structural systems have been envisaged, a) frame building, and b) shear building as shown in ***Fig. 6*** and ***Fig. 7***, respectively.

A typical moment resisting plane frame and also one half of the shear building elevations are illustrated in ***Fig. 8***. The weights of each floor are given in ***Table 4***.

The floor area ratios of columns in frame building are 3.2 percent and 2.5 percent in the basement and first floors, respectively. In shear building however, the shear wall ratio is 5.6 percent in ground floor and it gradually reduces to 4.1 percent in the upper floors.

6.2 Storey Drift Calculations

Assuming the building is located in the highest earthquake hazard zone, and following the methodology as contained in the new draft Earthquake Code of Turkey *[11]*, the lateral deflections as well as the storey drifts have been calculated and listed in ***Table 5***. The same values are also diagramatically illustrated in ***Fig. 9*** and ***Fig. 10***.

In frame building, only one typical interior frame is analysed. In shear building however, one half of the building is analysed as a plane frame, due to symmetry. The peripheral columns have been considered also as a plane frame and attached to the shear walls by hinges at floor levels.

7. COMPARATIVE EVALUATIONS

7.1 Upper Limits of Deflections

There are basically three types of restrictions in the earthquake code regulations, involving lateral elastic deflections, as follows:

a) Maximum Deflection at Top Floor ?

There is no provision in the current Turkish Earthquake Code *[5]* about the top floor deflection. The most stringent requirement however, exists in the Israel Code to be $d_n \leq 0.0010\,H$. In order to encourage the use of shear walls, the maximum deflection, d_n, at top floors is recommended not to exceed

$$d_N \leq 0.0007\,H \tag{8}$$

b) Maximum Interstorey Deflections ?

The difference of maximum elasto-plastic deflections between any two neighbouring floors is limited to 2 centimeters only in the earthquake codes of Indonesia and Egypt. No such provision exists in other codes. The structural factor is given as $R = 3$. But, for reasons of damage control $R = 5$ is recommended for the calculation of elasto-plastic deflections as

$$\delta_{ep} = R\,\delta_e \leq 0.02\,m \tag{9}$$
$$\delta_e \leq 0.004\,m \tag{10}$$

c) Maximum Storey Drift ?

In the current Earthquake Code of Turkey *[5]*, the maximum storey drift at any floor, calculated on the basis of elastic deflections, is not allowed to exceed $s \leq 0.0025$. In the proposed new code *[11]*, this upper limit is revised to be smaller of $s_m = 0.0035$ and $s_m = 0.02/R$. For reasons of effective damage control, however, it is recommended that the upper limit of the storey drift is to be taken as

$$s \leq s_m = 0.0014 \tag{11}$$

7.2 Effective Damage Control

Maximum storey drift limitation, s_m, is very much dependent on the level of base shear coefficient and the magnitude of elastic lateral loads. If however, the maximum storey drift limitation, s_m, is divided by the maximum effective base shear coefficient, C_m, the recommendation of various earthquake codes may be normalised as already discussed above in ***Paragraph 3***. The damage control index is calculated to be $i_{max} = 250$ in the current Turkish Earthquake Code, and it is $i_{max} = 200$ in its draft new version.

None of the earthquake codes, except those of Colombia and Romania, provide sufficiently stringent requirements for damage control. For the purpose of encouraging the use of shear walls in practice, and also disqualifying the moment resisting frames in most cases, the effective value of the damage control index is recommended, in this presentation, not to exceed $i_{max} = 120$. That is,

$$i = 10^4\,s / C_m \leq (i_{max} = 120) \tag{12}$$

7.3 Correlation of the Example Buildings

The lateral elastic deflections of the above mentioned example buildings, with frame and shear type structures, have been correlated against the upper limits of the recommended requirements. The results of this correlation, are summarised in ***Tables 6, 7, 8*** and ***9.***

It is seen that in most cases, the moment resisting frame building is disqualified because the recommended upper limits of deflections, storey drifts and/or damage control index are exceeded. In shear wall buildings however, all of the deflection requirements are properly satisfied.

8. COST-BENEFIT CONSIDERATIONS

It is vividly demonstrated in the three example buildings discussed above that the lateral deflections are reduced significantly if the structural system is composed of shear walls. The use of shear walls however, may increase the amount of concrete consumption per unit floor area. The shear wall floor area percentages in the typical first floors of the three example buildings are as follows:

Building	Shear Columns %	Walls %	Total %
High School	1.20	1.6	2.80
Yayla Apt.	2.46	0.8	3.26
20-Storey Building	2.50	5.6	8.10

Supposing the shear wall areas in a particular floor is about 3 percent more than the corresponding column areas of a frame system, the additional cost due to increase in the volume of vertical elements is on the order of about 1 percent of the total cost of the building.

Some basic advantages of shear walls may be summarised as follows:

a) Shear walls substantially increase the lateral stiffness of buildings and thereby reduce the deflections,
b) Shear walls effectively support the gravity loads,
c) Shear walls act as partition walls at the same time,
d) Shear walls meet the twin requirements of safety and damage control, and thus prevent excessive damages to nonstructural elements,
e) Shear walls reduce the need for ductile moment resisting frames,
f) Buildings with shear wall structural systems may be put into service immediately after a severe earthquake.

9. CONCLUSIONS

Based on the observations of damages occured to the example buildings in the past earthquakes, and also considering the methods of rehabilitation applied thereon, various concluding remarks may be stated as follows:

a) Ductile moment resisting frames, although satisfying the necessary requirements of strength and ductility, may not escape experiencing major nonstructural damages. Collapse may be prevented but the cost of repair and rehabilitation after the earthquake may be prohibitive to the extent of demolishing and total reconstruction. Therefore, the use of ductile shear walls, almost in all reinforced concrete buildings, will help to reduce the nonstructural damages.

b) Paradoxical to the above comment, there are no specific requirements or provisions in the earthquake codes discouraging or discarding the use of moment resisting frames in favour of shear wall systems. On the contrary, the shear wall systems are penalised with respect to their base shear coefficients. By making special arrangements however, in the limiting values of the lateral deflections and the storey drifts, the use of shear walls may be encouraged and preferred.

c) None of the lateral deflection restrictions of the current earthquake codes would provide a clear prediction for the excessive nonstructural damage patterns of the three real example buildings treated in this paper. In order to overcome this loop hole, some new and stringent requirements have been recommended for the upper values of the lateral deflections.

d) The top floor elastic deflection is recommended not to exceed 0.0007 times the building height, the relative interstorey elastic deflections should not possibly exceed 0.0040 meters, and the upper value of the storey drift should be limited preferably to 0.0014.

e) In addition to the above recommendations, a damage control index is introduced for the intention of controlling the extent of damages to nonstructural elements.

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TABLE 2. – **DEFLECTIONS OF THE *BURDUR* HIGH SCHOOL** *(C_m = 0.06)*

a) Frame Building							
Floor	h	W	F	d	δ_e	$s = \delta_e / h$	$i = 10^4\ s\ /\ C_m$
–	m	ton	ton	m	m	–	–
4	3	43	4.54	**0.0153**	0.0024	0.0008	133
3	3	50	3.33	0.0129	0.0033	0.0011	183
2	3	50	2.48	0.0096	0.0043	**0.0015**	**250**
1	4	51	1.27	0.0053	**0.0053**	0.0013	217
b) Shear Building							
4	3	43	4.54	**0.0085**	**0.0024**	0.0008	132
3	3	50	3.33	0.0061	0.0024	**0.0008**	**133**
2	3	50	2.48	0.0037	0.0018	0.0006	100
1	4	51	1.27	0.0019	0.0019	0.0005	83

TABLE 3. – **DEFLECTIONS OF THE *YAYLA* APARTMENT BLOCK** *(C_m = 0.08)*

a) Frame Building in *Y*-direction							
Floor	h	W	F	d	δ_e	$s = \delta_e / h$	$i = 10^4\ s\ /\ C_m$
–	m	ton	ton	m	m	–	–
5	2.8	200	27.8	**0.0098**	0.0011	0.00040	50
4	2.8	240	26.5	0.0087	0.0019	0.00066	83
3	2.8	240	19.7	0.0068	0.0024	0.00087	109
3	2.8	240	12.8	0.0044	**0.0027**	**0.00094**	**118**
3	2.4	240	6.0	0.0017	0.0017	0.00071	88
b) Shear Building in *Y*-direction							
5	2.8	200	27.8	**0.0073**	0.0012	0.00045	56
4	2.8	240	26.5	0.0061	0.0016	0.00059	74
3	2.8	240	19.7	0.0044	**0.0017**	**0.00062**	**90**
3	2.8	240	12.8	0.0027	0.0016	0.00056	70
3	2.4	240	6.0	0.0011	0.0011	0.00046	57

TABLE 4. – **FLOOR WEIGHTS** *(20-Storey Building)*

Floor	h	Frame Building	Shear Building
	m	ton	ton
20	4	32.7	245.3
.....	.....	.....	.....
1	4	32.7	245.3
Σ	**80**	**654.0**	**4 905.6**

TABLE 5. – **LATERAL LOADS AND STOREY DRIFT** *(20-Storey Building, C_m = 0.125)*

Floor	Frame Building			Shear Building		
	Forces F	Deflection d	Drift s	Forces F	Deflection d	Drift s
	ton	10^{-4} m	10^{-4}	ton	10^{-4} m	10^{-4}
20	5.40	966	12	40.51	567	7
19	2.27	919	13	17.04	539	7
18	2.15	868	14	16.15	510	8
17	2.03	813	14	15.25	480	8
16	1.91	756	15	14.35	488	8
15	1.79	696	15	13.46	416	8
14	1.67	634	**16**	12.56	382	**9**
13	1.55	570	16	11.66	848	9
12	1.44	505	14	10.76	812	9
11	1.32	448	14	9.87	277	9
10	1.20	391	14	8.97	242	9
9	1.08	335	14	8.07	207	8
8	0.96	280	12	7.18	173	8
7	0.84	233	11	6.28	141	8
6	0.72	187	11	5.38	110	7
5	0.60	144	10	4.49	82	6
4	0.48	103	8	3.59	56	6
3	0.36	69	8	2.69	34	4
2	0.24	39	6	1.79	17	3
1	0.12	15	4	0.90	5	1
Σ	**28.12**	–	–	**210.94**	–	–

TABLE 6. – **TOP FLOOR DEFLECTION CRITERIUM** *(NO. 1)*

BUILDING	Top Floor	Israel, 1975		Tezcan Proposal	
	d_N	0.0010 *H*	Safe	0.007 *H*	Safe
High School *(13 m)*					
Frame	0.0153	0.0130	*U*	**0.0091**	***U***
Shear	0.0085		1.53		1.07
Yayla Apt. *(13.65 m)*					
Frame	0.0098	0.0136	1.39	**0.0102**	1.04
Shear	0.0073		1.86		1.40
20-Storey *(80 m)*					
Frame	0.0966	0.0800	*U*	**0.0560**	***U***
Shear	0.0567		1.41		1.00

TABLE 7. – **INTERSTOREY DEFLECTION CRITERIUM** *(NO. 2)*

BUILDING	Interstorey Deflection	Egypt, 1988 $R = 3$		Tezcan Proposal $R = 5$	
	δ_e meter	0.02/*R*	Safe	0.02/*R*	Safe
High School					
Frame	0.0053	0.0067	1.26	**0.0040**	***U***
Shear	0.0024		2.79		1.67
Yayla Apartment					
Frame	0.0027	0.0067	2.48	**0.0040**	1.48
Shear	0.0017		3.94		2.35
20-Storey Building					
Frame	0.0065	0.0067	1.03	**0.0040**	***U***
Shear	0.0035		1.91		1.14

U = Unsafe

TABLE 6. – **TOP FLOOR DEFLECTION CRITERIUM** *(NO. 1)*

BUILDING	Top Floor	Israel, 1975		Tezcan Proposal	
	d_N	0.0010 *H*	Safe	0.007 *H*	Safe
High School *(13 m)*					
Frame	0.0153	0.0130	*U*	**0.0091**	*U*
Shear	0.0085		1.53		1.07
Yayla Apt. *(13.65 m)*					
Frame	0.0098	0.0136	1.39	**0.0102**	1.04
Shear	0.0073		1.86		1.40
20-Storey Bldg. *(80 m)*					
Frame	0.0966	0.0800	*U*	**0.0560**	*U*
Shear	0.0567		1.41		1.00

TABLE 7. – **INTERSTOREY DEFLECTION CRITERIUM** *(NO. 2)*

BUILDING	Interstorey Deflection	Egypt, 1988 $R = 3$		Tezcan Proposal $R = 5$	
	δ_e meter	$0.02/R$	Safe	$0.02/R$	Safe
High School					
Frame	0.0053	0.0067	1.26	**0.0040**	*U*
Shear	0.0024		2.79		1.67
Yayla Apartment					
Frame	0.0027	0.0067	2.48	**0.0040**	1.48
Shear	0.0017		3.94		2.35
20-Storey Building					
Frame	0.0065	0.0067	1.03	**0.0040**	*U*
Shear	0.0035		1.91		1.14

U = Unsafe

TABLE 8. – STOREY DRIFT CRITERIUM *(NO. 3)*

BUILDING	Storey Drift	Turkey, 1975 *(Turkey, 1995)*		Tezcan Proposal	
	$s = \delta_e / h$ meter	0.0025 $(0.0200/R)^1$	Safe	0.0014	Safe
High School					
Frame	0.0015	0.0025	1.67	**0.0014**	*U*
Shear	0.0008		3.12		1.75
Yayla Apartment					
Frame	0.0009	0.0025	2.78	**0.0014**	1.55
Shear	0.0006		4.17		2.33
20-Storey Building					
Frame ($R = 8$)	0.0016	0.0025	1.56	**0.0014**	*U*
Shear	0.0009		2.78		1.55

(1) The twin requirement of s_m = 0.0035 of the draft new Turkish Code does not govern, since R = 8 for frame buildings, s_m = 0.02/R = 0.0025 ≤ 0.0035.

TABLE 9. – DAMAGE CONTROL CRITERIUM *(NO. 4)*

BUILDING	Damage Control Index	Turkey, 1975 *(Turkey, 1995)*		Tezcan Proposal	
	$i = 10^4\ s / C_m$	312 (200)	Safe	120	Safe
High School *(C_m = 0.06)*					
Frame	250	312	1.25	**120**	*U*
Shear	133		2.35		*U*
Yayla Apt. *(C_m = 0.08)*					
Frame	118	312	2.65	**120**	1.02
Shear	90		3.46		1.33
20-Storey *($C_m = 0.12^5$)*					
Frame	128	200	1.56	**120**	*U*
Shear	72		2.78		1.67

U = Unsafe