FAILURE MODES OF RC COLUMNS UNDER LOADING

G. Doğan, M. H. Arslan

Abstract—Failure of reinforced concrete (RC) columns has been a main cause of collapse of existing RC building frames so far. Similar failure patterns of buildings have been repeatedly observed in the investigation of past earthquake damages. As observed from these events, seven failure types cause completely failed buildings or partially damage the structural members. The names of them flexural failure, shear failure, shear failure of flat-plate construction (punching failure), bond splitting failure, splice failure of longitudinal reinforcement, creep failure and column-beam joint failure. The objectives of this paper are to highlight some of these observations, causes of failure and to find preventing method of these damages. Some analytical calculation is also given in this paper.

Index Terms— Earthquake, Failure of columns, Punching, Reinforced Concrete, Shear Effect

Notation

d	:	Core of constant diameter
l	:	Length of core
A_c	:	Gross section area of column
$N_{d\,m}$	:	Greater of the factored axial forces calculated under vertical loads only and under simultaneous action of vertical and seismic loads
f_{ck}	:	Characteristic compressive cylinder strength of concrete
ρ_l	:	Longitudinal column reinforcement
V_n	:	Column shear strength
V_p	:	Max. probable shear force required for the plastic hinge form. at column ends
M_p	:	Max. plastic moment capacity of the column
L	:	Clear height of the column
ℓ_b	:	Development length
f_{yd}	:	Design yield strength of longitudinal reinforcement
f_{ctd}	:	Design tensile strength of concrete
ϕ	:	Rebar diameter
V_c	:	Concrete contribution to the shear strength
P	:	Axial load
A_g	:	Gross cross section area
f_c`	:	Specified compressive concrete strength
b,d	:	Web width and effective depth of the section
v_c	:	Shear strength carried by concrete
A_{sw}	:	Transverse reinforcement area within a spacing
f_{ywd}	:	Yield strength of transverse reinforcement
s	:	Spacing in loading direction
N_b	:	Axial force on balanced point
D`	:	Distance measured parallel to the applied shear between centers of the peripheral hoop

1 INTRODUCTION

There have been observed different failure types in reinforced concrete (RC) building members such as columns, beams, shear walls, infill walls, slabs, connection regions etc. Performance of RC buildings has demonstrated that a concrete column has an important role for preventing totally collapse of building. As well as Japan, USA, India, Greece, in Turkey most of failure types have occurred due to,

- Earthquakes (for instance in Turkey, over than 500.000 building heavily damaged after the earthquakes (lateral load is dominant)
- Self-weight (in the last decade over than 200 building totally collapsed in Turkey (axial load is dominant))

Recent earthquakes (Erzincan-1972, Kobe-1995, Marmara-1999) show that, RC member failing may cause the total collapsing of a building and it has become an obvious realty that the column damage is the most serious and important failure type in all structural members. Summaries of the performance of RC buildings in past earthquake are provided in literature [1-2]. Lessons and prominent observations summarized in those documents and other earthquake reconnaissance reports [3-4] indicate that damage in poorly detailed columns is a primarily cause for significant structural damage including excessive permanent drift and building collapse.

Sometimes, buildings and structures have failed or suddenly sustained damage because of their own weight or other loads. The collapses occurred spontaneously, and were not related to an earthquake or other external causes. Kaltakcı et al. [5-7] studied about the collapsed RC building in their self-weight. The most dramatic failure example of self-weight was the Zumrut Apartment Building disaster: a 9-story RC building in Turkey that collapsed on February 2, 2004, leaving 92 people dead.

The last experimental and analytical investigations [3-8-9] have been concentrated on deficiencies in seismic shear resistance of RC columns.

Typical failure types in columns can be divided as flexure, shear, combined shear and flexure and bond failure.

From this point of view, the main aim of this study is to explain failure modes of RC columns. The other major objective of this research is to identify main factors contributing to shear failure and gravity load collapse of lightly RC columns. At the end of the paper, the significance of column failure for a RC structure is emphasized.

2 Failure Modes of RC Columns

As mentioned above, columns have the most important role in all structural members of a building. According to the base civil engineering concept, columns failure wants to be occurring after beams and the other components damages. Performance of existing buildings in earthquakes indicates that the beams are less vulnerable to damage during earthquakes and their damage appears to be less critical to performance as compared with that of columns and column – beam joints. Table 1 represents of damage level of a column [10-11].

2.1. Flexural Compression Failure

A flexural compression failure is a common failing type in RC especially high rise RC building columns. A RC column that subjected combined axial load and bending moment reaches capacity when concrete reaches the ultimate deformation level as about 0.003~0.0035. If concrete reaches the deformation capacity before yielding of longitudinal reinforcement bars in tension region, the failure mode is called as compression dominant flexure failure.

The main parameter of the deformation type under combined axial load and bending moment are; section area of column, concrete compressive strength, axial forces level and amount of longitudinal and lateral reinforcement bars.

With reference to the N-M interaction diagram given in Fig. 1, the design point on the compression axial load side should be made to lie at or below the balanced point, that is $N<N_b$ so that the failure of column is by yielding of steel and not by crushing of concrete.

If the column is idealized as a rigid body, the flexural deformation of the column can be represented by the rotation of the rigid body. Fig. 2.a shows this type of failure mode.

The bending cracks investigated at the socket level of the columns of a coffee-processing factory erected in 1996 in Izmit can be seen in Fig. 2.b. Formation of bending cracks at lower sections of the columns is a widespread failure type confronted. This type of failure is the indicator of column exceeding the ultimate elastic moment bearing strength at lower sections. The inadequacy of the column cross-sections especially in frame's orthogonal out-of-plane direction (the asymmetrical approach in column-design) and exceedingly ratio of the longitudinal reinforcement the total area of which reaches up to the ultimate value given in the building codes that a column might possess are the principal reasons for column lower section failures.

To compare concrete strength with code's requirement [12], six specimens were extracted from different axe's columns (one of them is shown in Fig. 2.b.). Core of constant diameter, d=8 cm, and different lengths, l. The results are listed in Table 2 where the core strength is converted into that of the standard cylinder of 15-30 cm.

Fig. 3 gives basic information about material strength. As illustrated in Fig. 3, even though the concrete strength must be emphasized upper than 30 MPa in the project, experimental study shows that real strength is lower.

In various studies on concrete strength in different regions of Turkey, it is concluded that the average concrete compressive strength in existing buildings is around 10 MPa [13-16]. Especially in the Kocaeli Earthquake, average concrete compressive strength that was taken damaged and undamaged buildings was as low as 1/3 of the design strengths very low and far from TEC-2007 [12] and TBC-2000 [17] requirements. Fig. 4 was taken from a damaged column that was made low quality materials.

According to TEC-2007 [12], Gross section area of column shall satisfy the condition given in equation 1, in this formulation N_{dm} refers to design axial load from load combination includes earthquake effect. In the drift version of TEC-2017 [12] the formulation are revised by changing 0.5 to 0.4.

$$A_c \geq \frac{N_{dm}}{\left(0.50 \times f_{ck}\right)} \quad (1)$$

$(N_{dm} = N_g + N_q \pm N_E)$

Shorter dimension of columns with rectangular section shall not be less than 250 mm and section area shall not be less than 75000 mm2. Although min. dimension of column has been 250 mm since TEC-1975 [18], in practice there has been observed too many application that contains column dimensions is less than 250 mm. In the chapter 21 of ACI 318-95 [19], the min. dimensions of columns is 300 mm. The reason for this is to satisfy minimum rigidity, decrease the axial load level, and thus increase in ductility.

In buildings, the redundancy is higher and the bending moment due to lateral loads per column may be small. Further, the efforts of designers to reduce column sizes to increase architectural appeal pushes the design point more towards the apex of the N-M interaction diagram. This is not desirable owing to possible brittle compression failure. In addition, smaller column sizes relative to that of the beams suggest that the beams are likely to be stronger than columns. Under lateral loads, this strong-beam-weak-column system leads to catastrophic storey collapse mechanisms (or sway mechanisms). Typical strong beam-weak column failure type represents in Fig. 5. According to all seismic codes, columns must be stronger 1.2 times than beams. In example buildings given in Figure, beams are 1.65 times stronger than columns. From these points of view, it may be required to the design point to a level marginally above the balanced axial load, if not at or below the balanced axial load. Fig. 6 shows column with buckled longitudinal bars that experienced high axial load.

Longitudinal column reinforcement shall not be less than 1%, nor shall it be more than 4% of gross section area ($1\% < \rho_\ell < 4\%$). In the code, selection of low steel ratio is encouraged. The reason of that is; low steel ratio is an amplification of larger cross section. This situation effects ductility to increase. The author has observed that in all seismic regions column's ρl is ranges between 1% and 2%. 12-16 mm diameter smooth rebar are generally used but with respect to TEC-2007 minimum bar diameter must be 14 mm. The selection of minimum $\phi 14$ steel prevents buckling of reinforcement.

During an earthquake, however, exterior columns, especially corner ones, are subjected to varying axial force due to the overturning moment of a structure; the axial force level in these columns may become extremely high in compression, leading the flexural compression failure.

2.2. Shear and Torsion Failure

Shear and torsion failure after inelastic cyclic loading is often observed in RC beam or column whose shear strength is slightly larger than its flexural strength. In the AIJ Design Guidelines, 1999 [20], this kind of failure is attributed to the two reasons,

(1) reduction of effective compressive strength of concrete due to intersecting flexural-shear cracks, and

(2) reduction of aggregate interlocking due to wide flexural-shear cracks (Architectural Institute of Japan (1999) [20].

Shear and torsion failure which are the most brittle mode of RC columns are caused by especially lack of lateral reinforcements. As shear failure proceeds, degradation of the concrete core may lead to loss of axial load carrying capacity of the column. As the axial capacity diminishes, the gravity loads carried by the column must be transferred to neighboring elements. A rapid loss of axial capacity will result in the dynamic redistribution of internal actions within the building frame and may progressively lead to collapse.

A simple way to check for shear failure in a frame system with double-curvature columns is to compare the column shear strength, V_n, with the maximum probable shear force required for the plastic hinge formation at column ends, V_P (given in equation 2)

$$V_P = \frac{2M_P}{L} \quad (2)$$

M_P maximum plastic moment capacity of the column, L is clear height of the column.

The main reason of the shear failure is exceeding tension stress of the concrete tension strength. If shear deformation due to shear cracks is idealized as shown in Fig. 7.

After the concrete cracks under the tensile stress, the stress must be transferred to the lateral reinforcement. Brittle shear failure occurs in the diagonal tension mode when the minimum amount of lateral reinforcement is not provided in the member. Fig. 8 illustrated shear effect on a column after Adana Earthquake.

Transverse reinforcement in concrete columns is used to fulfill three main functions. These functions include restraining longitudinal reinforcement against buckling, increasing shear resistance, and confining concrete for improved deformability. Short and stubby columns attract shear stresses that may exceed diagonal tension capacity of concrete. The excess shear in these columns is resisted by transverse reinforcement. Shear reinforcement is usually designed following the 45o truss analogy employed in the ACI-318 [21] design code.

The lack of shear reinforcement was one of the main causes of collapse of the buildings and it is shown Fig. 9a [22]. Some RC colomns failed due to its shortening because of the effect of the masonry wall (short column effect) (Fig. 9b)

Fig. 10 is represents a shear failure type occurred after 1999 Marmara earthquake [24]. As well as ACI-318, during the last few decades several shear strength model have been proposed and used for the design and evaluation of RC columns. A short brief of this model is given in Table 3 for a sample column. In the Table 3, only concrete contribution and transverse reinforcement is given. According to the ASCE-ACI Committee 426 Proposals [25], the most critical mechanism were identified as the shear transfer by the transverse reinforcement and concrete. Shear transfer by uncracked concrete, interface shear transfer in the cracked concrete, aggregate interlock, dowel shear carried by the longitudinal reinforcement and arch action in deep members is commonly neglected.

In lightly reinforced columns after the shear failure degradation of the core concrete may lead to loss of gravity-load-carrying capacity. A sudden loss of column axial capacity will lead to transfer of column gravity loads to neighboring frame members with ensuing dynamic redistribution of forces within those members and a possible subsequent building collapse. Fig.11 shows shear failure effect in a lightly reinforced column [26] .

2.3. Shear failure of flat-plate construction (punching failure)

The flat plate is a two-way reinforced concrete framing system utilizing a slab of uniform thickness, the simplest of structural shapes. A flat plate floor do not have beams supported the slabs. Serious shear failure of flat-plate (punching failure) was observed after the 1985 Mexico City and 1999 Marmara-Kocaeli Earthquake.

Pure punching failure capacity of the connections is defined using the eccentric shear stress model of TBC-500-2000 and new TEC-2017 draft code [27]. Slab-Column joints can cause failure for the buildings under earthquake effects and may even cause damage in some cases. [28-31]. The slabs and column components with greater flexibility can be observed collapse [29, 32].

Fig. 13 represents the pan – cake collapse of a RC building in Marmara region, after the 1999 Marmara-Kocaeli earthquake.

It can be seen that in many destructive earthquakes, the construction is totally collapsed because of the weak column or insufficient ductility [28-29, 33-37].

2.4. Bond Splitting Failure

In a reinforced concrete member, load is transferred between the reinforcement bars and the concrete through bonds under loading. High flexural bond stresses may exist in members with steep moment gradients along their lengths. The splitting failure of bond is influences many factors;

- Used deformed bars,
- the proportioning and positioning of the main reinforcement and lateral confining reinforcement within the member,
- material characteristics of the concrete,
- the concrete cover on the deformed bars,
- yield strength of the reinforcement bars.

In figures 14 and 15 bond splitting failure types are given.

2.5. Splice Failure of Longitudinal Reinforcement

The reinforce concrete is composed of steel and concrete material. While using lap-spliced design, it is very important to make sure that the bonding between concrete and reinforcement is sufficient. Because the lap-spliced steels strongly affect the column ductility [38-39].

The Chi-Chi Earthquake [40] caused severe damages in the central regions of Taiwan counties. In Taiwan, the traditional low rise buildings were designed without ductile details to resist strong earthquake ground motions, and suffered moderate to major damages, even collapse. According to the reconnaissance reports [39], it is believed that the failure mechanisms of steels lap-spliced at the plastic hinge zone and insufficient confinement of columns are fatal factors that bring to the structure to collapse. Particularly, the construction of lap-spliced steels, which play an important rule to affect the member behaviors, may led to brittle structural collapse [38-39].

Longitudinal reinforcement is spliced in various ways, including lap splice, mechanical splices and welded splices. Splices should located in a region where tension stress is low. Splices in order buildings were located in regions of higher tensile stresses because the implication for earthquake performance was inadequately understood. Splice failure reduces flexural resistance of the member often before yielding.

In the code, lap splices of column longitudinal reinforcement should be made, as much as possible, within the column central zone defined. In the drift version of TEC-2007 [12], TEC-1975 [18], TEC-2016 [27] lap splices of the columns proposed mid-section of the column (Fig. 16).

In this case, the splice length shall be equal to the development length ℓ_b given in TBC-2000 for tension bars. In equation 3, development length is given. Here, f_{yd} is design yield strength of longitudinal reinforcement, f_{ctd} is design tensile strength of concrete, ϕ is rebar diameter.

$$\ell_b = \left[0.12 \frac{f_{yd}}{f_{ctd}} \phi \right] \geq 20\phi \qquad (3)$$

According to TEC-2007 [12], in the case where lap splices of column longitudinal reinforcement are made at the bottom end of the column, the following requirements shall be met.

(a) In the case where 50% of longitudinal reinforcement or less is spliced at the bottom end of column, lap splice length shall be at least 1.25 times ℓ_b.

(b) In the case where more than 50% of longitudinal reinforcement is spliced at the bottom end of column, lap splice length shall be at least 1.5 times ℓ_b. The same condition shall apply to starter bars protruding from the foundation.

Lap splices in moment-frame columns were typically made immediately above the floor framing or the foundation. That means the lap splices in column were located in a plastic hinge zone that is the most critical region of RC members. The authors have observed this common mistake in many of buildings. This damage is due to inadequate lap splice length.

During the 1985 Mexico earthquake no confinement effect was observed in the columns where the transverse reinforcement was insufficient. In addition, there were inadequate construction joints on the shear walls, there was movement and damage throughout the joints[29-30]. In addition, inadequate construction and materials quality may cause structural failure or collapse.

Fig.17-18 is a good example of splice failure of longitudinal reinforcement. Most of buildings collapsed because of inadequate lap splices in column- base connection region after 1999 Marmara Earthquake.

2.6. Creep Failure

Creep has an unfavorable effect on the strength of concrete. Creep failure is the time dependent deformation of concrete that is subjected to permanent loads. Fig. 19 demonstrated that concrete specimen which are loaded up to 60%, 70% and 75% of their strength carry such loads forever without showing any failure. When the same specimen is loaded so that the stress in concrete is 80% or more of its strength, the specimen fails after a certain time [41]. Fig. 20 shows a creep failure. Creep causes significant deformation and makes some cracks occur. A harmful type of this failure is shown in Fig. 21. In Table 4 [42], experimental study results were given. It is clear that compressive strength of concrete is really very low according to the TEC-2007 and TBC-2000 design criteria.

Kaltakcı et al. [5-7] studied about the collapsed RC building in their self-weight. The main structural failing cause is creep deformation of the concrete. Creep failure, heavily damaged column in an apartment is shown Fig. 22 [42].

2.7. Column-Beam Joint Failure

Many of the beam-column joints are heavily damaged as result of

- insufficient lateral ties at the beam-column joints. A view of one of the damaged joints is shown in Fig. 23b. In the figure, beam-column connections consisted of weak reinforced concrete columns and strong beams. On the contrary, similar building that is shown in Fig.23a did not meet heavily damage after earthquake shock.
- Confinement reinforcement did not exist and beam reinforcing bars anchorage in the joint is inadequate. This type of damage was also reported for the September 21, 1999 Chi Chi earthquake [43].

Beam-column joints of frame systems comprised of columns and beams of high ductility that have been separated into two classes as confined and unconfined joints in the TEC-2007.

a) In the case where beams frame into all four sides of a column and where the width of each beam is not less than 3/4 the adjoining column width, such a beam-column joint shall be defined as a confined joint (Fig.24).

b) All joints not satisfying the above given conditions shall be defined as unconfined joint (Because it is rather difficult to satisfy this requirement practically).

It is rather difficult to satisfy this requirement practically. In practice, it is observed that additional stirrups were not provided near and within the connections in many cases [11].

To the TEC-2007, in structural systems comprised of frames only or of combination of frames and walls, sum of ultimate moment resistances of columns framing into a beam-column joint shall be at least 20% more than the sum of ultimate moment resistances of beams framing into the same joint.

3 What to Do for Preventing Failure in RC Columns?

- Shear reinforcement was lacking in most damaged columns, so the main failure reason is shear transverse reinforcement detailing and spacing. With respect to TEC-2007 and other country's codes, seismic detailing must be done carefully.
- Use of stirrups at column-beam connections should be adequate.
- Avoid poor bonding between concrete and steel.
- Hooks of stirrups into the concrete core must be satisfactory.
- Inadequate clear cover causes corrosion of steel. Clear cover must be arranged to the buildings and seismic codes.
- Drift rate is so important for stability of a structure, so designers avoid large displacement of structural system and large P-Δ effect. RC Shear wall must be used so as to prevent this effect.
- Discontinuity of vertical structural elements is very important because they cause structural failure or damage.
- Building with shear walls survived with limited or no damage.

4 Conclusion and Final Comments

Failure of a RC column makes complete collapse of a building. All failure types in a column (and building) can occur if it is not well equipped with transverse steel. Experimental studies have shown that in a damaged RC column, material quality is very low and insufficient. Especially in seismic region, to avoid future tragedies, a building stock inventory should be prepared, and short and long term measures planned for poor quality structures. An efficient control mechanism at any phase of concrete production should be established in seismic region as Turkey.

To sum up, thousands of existing RC structures in the world have a non-ductile character. If not these buildings should be retrofitted or strengthened immediately, many lives will be lost in future with strong earthquake.

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List of Figures

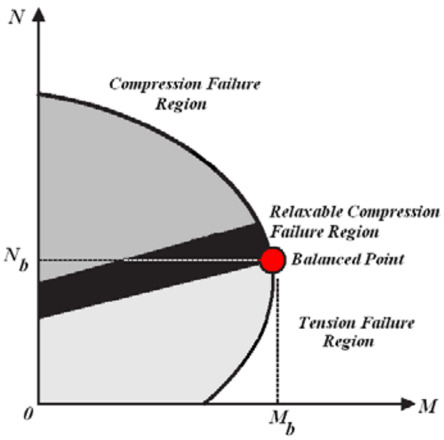


Fig. 1. Interaction curve of a column

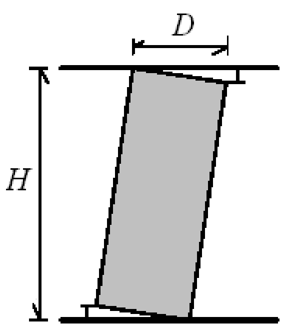


Fig. 2a.Deformation mode

Fig. 2b. Bending cracks witnessed in a coffee-processing factory in Izmit

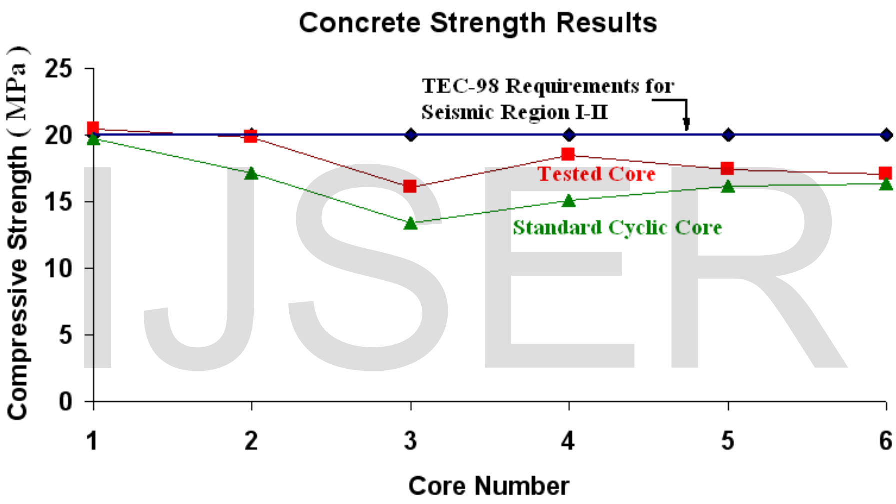


Fig. 3.Comparative experiment of concrete strength

Fig. 4. Typical column failure

Fig. 5. Strong beam-weak column failure example

Fig. 6. Columns failure with buckled longitudinal bars

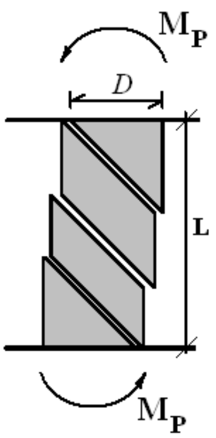


Fig. 7.Shear failure mechanism

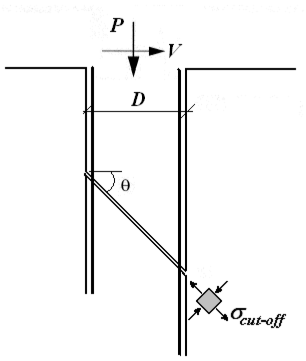


Fig. 8.A damaged column by shear effect

Fig. 9a. The Lack of Shear reinforcement failure of the column [22]

Fig.9b. Short column effect [23]

Fig. 10.Shear failure at column Top [24]

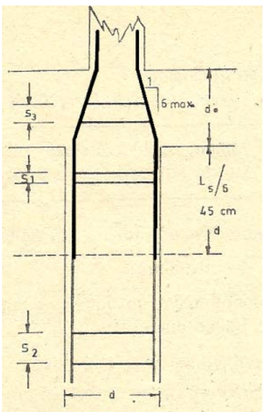
Fig.11. Shear failure of column [26]

Fig. 12. Shear failure of flat-plate construction (punching failure)

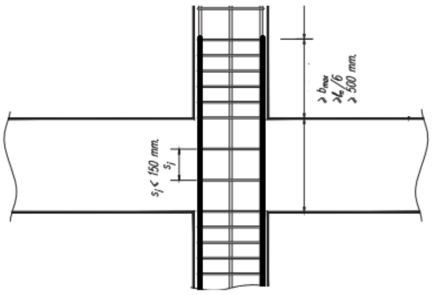
Fig. 13. The pan-cake collapse

Fig. 14. Longitudinal reinforcement of a column is not closely spaced (450-600 mm)

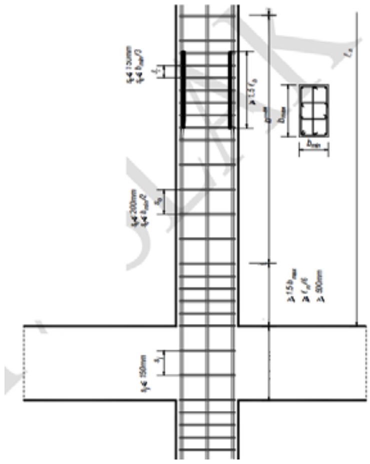
Fig. 15.The concrete cover on the deformed bars is thin



TEC-1975 [18]



TEC-2007 [12]



TEC-2017 (draft) [27]

Fig. 16. Lap splices of columns in TEC

Fig. 17. Inadequate details at the base [19-23]

Fig. 18. Failure of short lap splices in moment-column frame

Fig. 19. Short columns failed

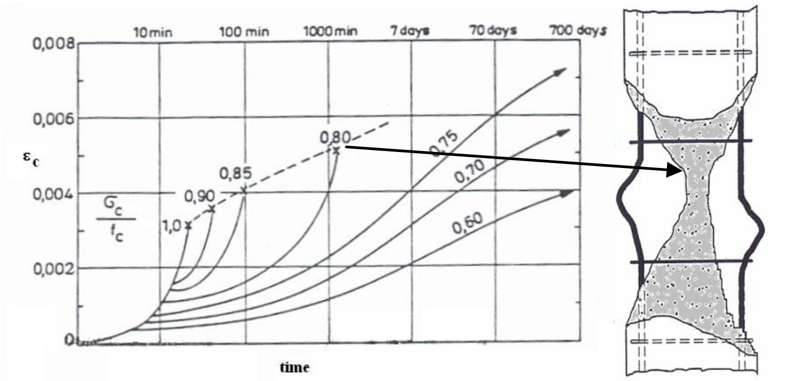


Fig. 20. Creep has an unfavorable effect on the strength of concrete [41]

Fig. 21. Cracks caused by creep

Fig. 22. Creep failure, heavily damaged column in an apartment [42]

Fig. 23a.Undamaged column-beam joint

Fig. 23b.Figure Damaged column-beam joint

ISSN 2229-5518

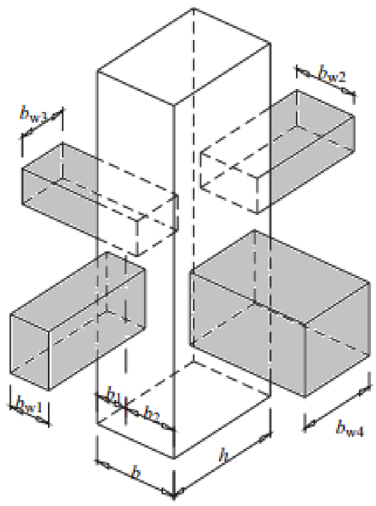
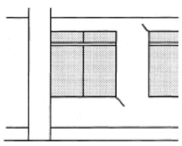
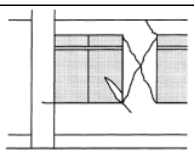
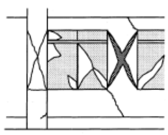
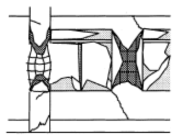
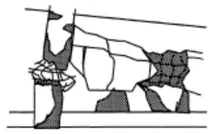


Fig. 24. Beam-column joints of frame systems

LIST OF TABLES

ISSN 2229-5518

TABLE 1
EVALUATION OF DAMAGE LEVEL [10-11]

Sketch					
Damage Level	Light	Minor	Medium	Major	Collapse
Description	Very Light or no damage	Light damage on columns, shear cracks on RC non-structural walls	Shear or flexural cracks on columns, appreciable damage on non-structural walls	Reinforcement exposed and buckled in columns, large shear cracks especially in shear walls	Significant damage on columns and shear walls, a part of entire building collapsed
Column Cracks Definition	Cracks with smaller than 0.2 mm	Cracks of 0.2-1 mm are found	Heavy cracks of 1-2 mm wide are found	Many heavy cracks are found, crushing of concrete and buckling of reinforcement	-------------

ISSN 2229-5518

TABLE 2

CONCRETE STRENGTH RESULTS OF DAMAGED BUILDINGS IN MARMARA REGION

		Compressive Strength	
Core	l/d	Tested Core(MPa)	Standard Cyclic Core (MPa)
I	1.52	20.41	19.75
II	1.34	19.83	17.13
III	1.28	16.10	13.40
IV	1.40	18.50	16.55
V	1.33	17.45	15.05
VI	1.39	17.02	16.15
		Average	16.34

TABLE 3

A SHORT BRIEF OF SHEAR STRENGTH MODEL

	Model for Shear Failure	V_C	V_S	V_n
1	ACI-318-02	$V_C = 2\left(1+\frac{P}{2000A_g}\right)\sqrt{f_c^{'}}bd$	$V_S = \frac{A_{sw}\times f_{ywd}\times d}{s}$	$V_C + V_S$
2	ASCE-ACI Committee 426 Proposals	$V_c = v_c\left(1+\frac{3P}{f_c^{'}A_g}\right)bd$ $V_c = v_c\left(1+\frac{P}{6\sqrt{f_c^{'}}A_g}\right)bd$	$V_S = \frac{A_{sw}\times f_{ywd}\times d}{s}$	$V_C + V_S$
3	SEAOC	$V_C = 3.5\sqrt{f_c^{'}}\sqrt{1+0.002\frac{P}{A_g}}A_c$	$V_S = \frac{A_{sw}\times f_{ywd}\times d}{s}$	$V_C + V_S$

4	Ascheim and Moehle	$V_C = \alpha'\left(1+\frac{P}{2000A_c}\right)\sqrt{f_c'}bd$	$V_S = \frac{A_{sw}\times f_{ywd}\times d}{s}$	$V_C + V_S$
5	Caltrans	$V_c = F_1F_2\sqrt{f_c'}(0.8A_c)$	$V_S = \frac{A_{sw}\times f_{ywd}\times d}{s}$	$V_C + V_S$
6	Priestly	$V_C = k\sqrt{f_c'}A_e$	$V_S = \frac{A_{sw}\times f_{yw}\times D'}{s}\times\cot 30^0$	$V_C + V_S$
7	Kowalski	$V_C = \alpha\beta\kappa\sqrt{f_c'}0.8A_g$	$V_S = \frac{A_{sw}\times f_{yw}\times(D'-c)}{s}\times\cot 30^0$	$V_C + V_S$
8	Konwinski	$V_C = \alpha\sqrt{f_c'}\sqrt{1+\frac{P/A_g}{12\sqrt{f_c'}}}0.85A_g$	$V_S = 0.85\times\frac{A_{sw}\times f_{ywd}\times d}{s}$	$V_C + V_S$
9	FEMA-273	$V_C = 3.5\lambda\left(k+\frac{P}{2000A_g}\right)\sqrt{f_c'}bd$	$V_S = \frac{A_{sw}\times f_{ywd}\times d}{s}$	$V_C + V_S$

TABLE 4
CONCRETE STRENGTH RESULTS OF DAMAGED BUILDINGS DUE TO CREEP FAILURE [42]

		Compressive Strength	
Core	l/d	Tested Core(MPa)	Standard Cyclic Core (MPa)
I	1.5	13.96	12.89
II	1.5	13.67	12.62
III	1.5	5.03	4.65
IV	1.5	6.36	5.87
V	1.5	8.91	8.23
VI	1.5	14.62	13.49
VII	1.5	12.57	11.60
VIII	1.5	8.02	7.40
IX	1.5	16.75	15.46
X	1.5	8.67	8.01
XI	1.5	6.25	5.77
XII	1.5	7.46	6.88
XIII	1.5	9.23	8.52
XIV	1.5	4.79	4.42
XV	1.5	8.79	8.12

XVI	1.5	11.72	10.81
XVII	1.5	9.59	8.85
XVIII	1.5	8.91	8.23
XIX	1.5	8.91	8.23
XX	1.5	4.67	4.31
		Average	8.69