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Model for the evaluation of the beam-column joint ultimate strength -a more simplified version

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Abstract. In this study, a well-established model and a new simplified version of it, that help avoid collapses in reinforced concrete structures during strong earthquakes, are presented and discussed. Using this model, the initial formation of plastic hinges and the final concentration of the damages only in beams are accurately assured. The model also assures that the columns and the beam-column joints can remain intact. This model can be applied for the design of modern R/C structures, as well as for the design of strengthening schemes of old R/C structures by the use of reinforced concrete jackets. The model can also predict the form of earthquake damages in old structures but also earthquake damages in the modern structures.

Keywords: beam-column; frames; connections; cyclic loads; reinforced concrete structural analysis

1. Introduction

Damage incurred by earthquakes over the years has indicated that many reinforced concrete (R/C) buildings, designed and constructed during the 1960s and 1970s, were found to have serious structural deficiencies today, especially in their columns and beam – column (b/c) joints. These deficiencies are mainly due to the lack of a capacity design approach and/or poor detailing of the reinforcement. The beam – column region of these buildings has often been found to contain no shear reinforcement. As a result, lateral strength and ductility of these structures were minimal and hence some of them have collapsed (Paulay and Park 1984, Karayannis *et al.* 1998, Park 2000, Karayannis *et al*. 2011, Kalogeropoulos and Tsonos 2014, Karayannis 2015, Kalogeropoulos *et al.* 2016, Tsonos *et al.* 2017, Golias *et al*. 2018, Karayannis *et al*. 2018). The studies by Tsonos (2006, 2007) provide structural engineers with useful information about the safety of the new reinforced concrete frame structures incorporating full seismic detail according to current building codes. Sometimes, this could be jeopardized during strong earthquakes by premature joint shear failures. The joints could at times remain the weak link of the structures even today.

Investigations into recent earthquake damage in Greece (Korinth 1981, Kalamata 1986, Aegion 1995, Athens 1999) have shown that, in many cases, damaged areas of reinforced concrete buildings were localised in beam-column connections. Furthermore, considering the commonly accepted idea that failure of joints may quickly lead to general failure (Park and Paulay 1975), the important issue of preserving the beam-column joints intact after strong earthquakes has arisen. A new model published in the studies by Tsonos (1999, 2006, 2007) assured the initial formation of plastic hinges in the beams but also the final concentration of damages only in these elements. This model also assures that the columns and the beam – column joints can remain intact during strong earthquake damages of the modern structures. This model can be applied to the design of modern R/C constructions, as well as to the design of strengthening schemes of old but also for modern structures by the use of reinforced concrete jackets (Tsonos 1999, 2001a, 2001b, 2002, 2008, 2010) offering in all cases higher level of safety. The model can also predict the type of earthquake damages of old structures but also the earthquake damages of modern buildings.

In the following paragraphs this model and especially the simplified version of it, are presented and discussed. Applications of the model are also presented.

2. A summary of the proposed model

New formulation published in the relatively recent studies by Tsonos (1996, 1997, 1999, 2006, 2007) predicts the beam-column joint ultimate shear strength.

Fig. 1(a) shows a reinforced concrete interior beam-column joint of a moment resisting frame and Fig. 1(b) shows the internal forces around this joint. The shear forces acting in the joint core are resisted: (i) partly by a diagonal compression strut that acts between diagonally opposite corners of the joint core (see Fig. 1(c)), and (ii) partly by a truss mechanism formed by horizontal and vertical reinforcement and concrete compression struts. The horizontal and vertical reinforcement is normally provided by horizontal hoops in the joint core around the longitudinal column bars and by longitudinal column bars between the corner bars in the side faces of the column. Both mechanisms depend on the core concrete strength. Thus, the ultimate concrete strength of the joint core under

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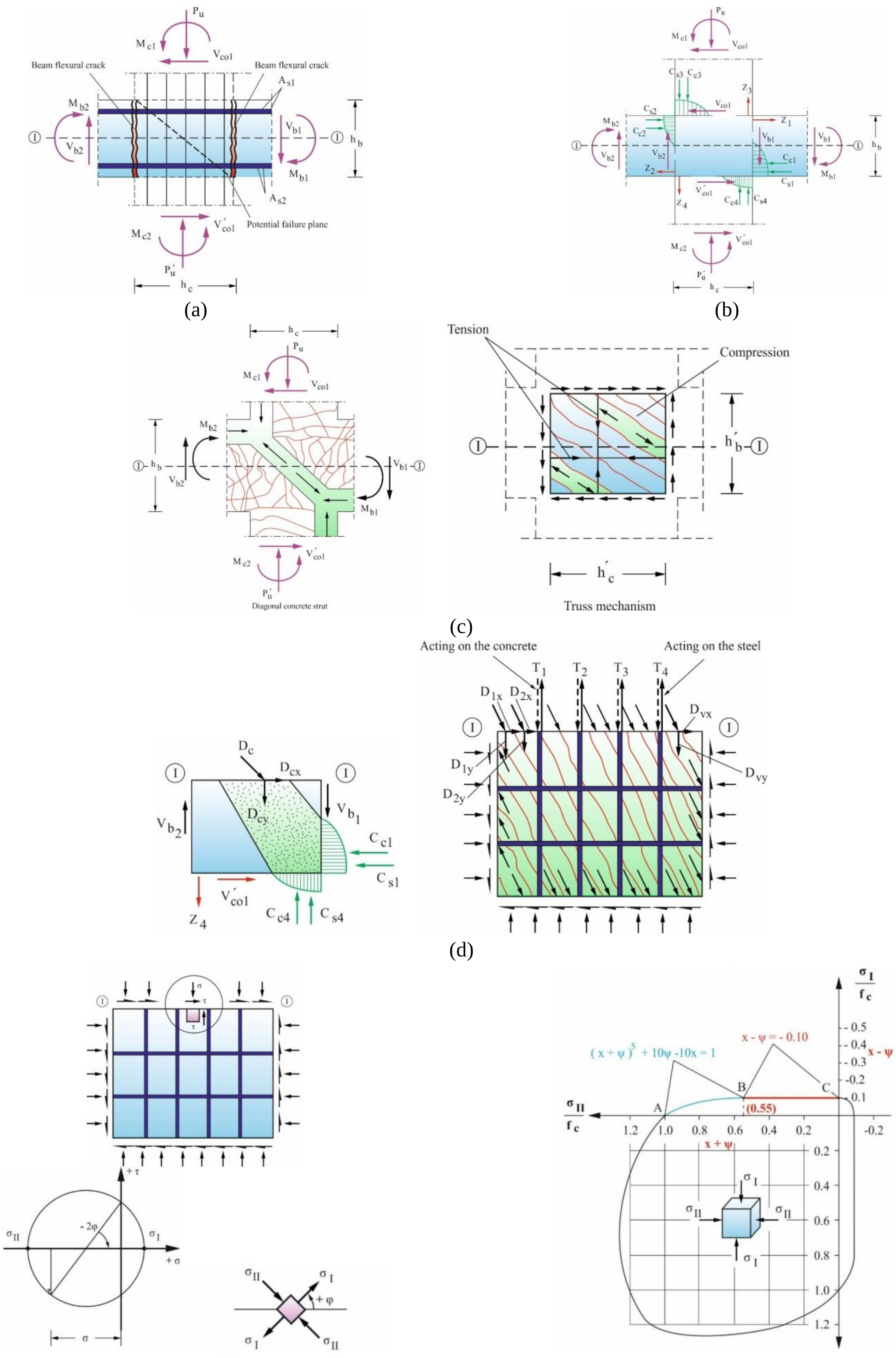


Fig. 1 (a) Interior beam-column joint, (b) Internal forces around an Interior beam-column joint as a result of seismic actions, (c) The two mechanisms of shear transfer (diagonal concrete strut and truss mechanism), (d) Forces acting in the joint core concrete through section I – I from the two mechanisms, (e) Stress state element of studied region and representation of the concrete biaxial strength curve by a parabola of 5th degree (Tsonos 2007) compression/tension controls the ultimate strength of the connection. After failure of the concrete, the strength in the joint is limited by the gradual crushing along the cross-diagonal cracks and especially along the potential failure planes (Fig. 1(a)).

For instance, consider the section I-I in the middle of the joint height (Fig. 1(a)). In this section, the flexural moment is almost zero. The forces acting in the concrete are shown in Fig. 1(d). Each force acting in the joint core is analysed into two components along the *X* and *Y* axes (Fig. 1(d)). T_i are the tension forces acting on the longitudinal column bars between the corner bars in the side faces of the column. Their resultant is ΣT_i. An equal and opposing compression force ($-\Sigma T_i$) must act in the joint core to balance the vertical tensile forces generated in the reinforcement. This compression force was generated by the resultant of the vertical components of the truss mechanism's diagonal compression forces $D_1, D_2 \ldots D_v$. Thus

$$D_{1y} + D_{2y} + \cdots + D_{vy} = \Sigma T_i = T_1 + T_2 + T_3 + T_4 \quad (1)$$

The column axial load is resisted by the compression strut mechanism. The summation of vertical forces equals the vertical joint shear force V_{jv}

$$\underbrace{D_{cy}}_{\text{compression strut}} + (T_1 + \cdots + T_4) = D_{cy} + \underbrace{D_{sy}}_{\text{truss model}} = V_{jv} \quad (2a)$$

The summation of horizontal forces equals the horizontal joint shear force V_{jh}

$$D_{cx} + (D_{1x} + \cdots + D_{vx}) = D_{cx} + D_{sx} = V_{jh} \quad (2b)$$

The vertical normal compressive stress σ and the shear stress τ uniformly distributed over section I-I are given by Eqs. (3)-(4) below.

$$\sigma = \frac{D_{cy} + D_{sy}}{h'_c \cdot b'_c} = \frac{V_{jv}}{h'_c \cdot b'_c} \quad (3)$$

$$\tau = \frac{V_{jh}}{h'_c \cdot b'_c} \quad (4)$$

where h'_c and b'_c are the length and the width of the joint core respectively.

It is now necessary to establish a relationship between the average normal compressive stress σ and the average shear stress τ (Fig. 1(e)). From Eqs. (3)-(4)

$$\sigma = \frac{V_{jv}}{V_{jh}} \cdot \tau \quad (5)$$

It has been shown that

$$\frac{V_{jv}}{V_{jh}} = \frac{h_b}{h_c} = \alpha \quad (6)$$

(Eurocode 8, Park and Paulay 1975, Paulay and Park 1984) where α is the joint aspect ratio.

The principle stresses (σ_1=maximum, σ_{11}=minimum are calculated as

$$\sigma_{I,II} = \frac{\sigma}{2} \pm \frac{\sigma}{2}\sqrt{1 + \frac{4\tau^2}{\sigma^2}} \quad (7)$$

Eq. (8) (Fig. 1(e), Tsonos 2007) was adopted for the representation of the concrete biaxial strength curve by a 5th degree parabola.

$$-10\frac{\sigma_I}{f_c} + \left[\frac{\sigma_{II}}{f_c}\right]^5 = 1 \quad (8)$$

where f_c is the increased joint concrete compressive strength due to confinement by joint hoop reinforcement, which is given by the model of Scott *et al.* (1982) according to Eq. (9).

$$f_c = k \cdot f'_c \quad (9a)$$

Also, f'_c is the concrete compressive strength and k is a parameter of the model (Scott *et al*. 1982) expressed as

$$k = 1 + \frac{\rho_s \cdot f_{yh}}{f'_c} \quad (9b)$$

where ρ_s is the volume ratio of transverse reinforcement and f_{yh} is its yield strength.

Substituting Eqs. (5)-(7) into Eq. (8) and using $\tau = \gamma \cdot \sqrt{f_c}$ gives the following expression (Eq. (10))

$$\left[\frac{\alpha\gamma}{2\sqrt{f_c}}\left(1 + \sqrt{1 + \frac{4}{\alpha^2}}\right)\right]^5 + \frac{5\alpha\gamma}{\sqrt{f_c}}\left(\sqrt{1 + \frac{4}{\alpha^2}} - 1\right) = 1 \quad (10)$$

Assume here that

$$x = \frac{\alpha\gamma}{2\sqrt{f_c}} \quad (11)$$

and

$$\psi = \frac{\alpha\gamma}{2\sqrt{f_c}}\sqrt{1 + \frac{4}{\alpha^2}} \quad (12)$$

Then Eq. (10) is transformed into

$$(x + \psi)^5 + 10\psi - 10x = 1 \quad (13)$$

The basic aim is how to substitute Eq. (13) (Fig. 1(c)) with a line equation in order to facilitate the procedure and to make it easier for the practising Civil Engineer. For this purpose, a large number of solutions of the system of Eqs. (11)-(13) was found for a wide range of joint aspect ratio values h_b/h_c, which are shown in Table 1. Each solution gives the corresponding values of x and ψ.

Then from the ratios of x and ψ the corresponding values of $x + \psi$ and $x - \psi$ are calculated. It is known that

$$\frac{\sigma_I}{f_c} = \frac{\alpha\gamma}{2\sqrt{f_c}} - \frac{\alpha\gamma}{2\sqrt{f_c}}\sqrt{1 + \frac{4}{\alpha^2}} \quad \text{and}$$

$$\frac{\sigma_{II}}{f_c} = \frac{\alpha\gamma}{2\sqrt{f_c}} + \frac{\alpha\gamma}{2\sqrt{f_c}}\sqrt{1 + \frac{4}{\alpha^2}}$$

Since $x = \frac{\alpha\gamma}{2\sqrt{f_c}}$ (Eq. (11)) and $\psi = \frac{\alpha\gamma}{2\sqrt{f_c}}\sqrt{1 + \frac{4}{\alpha^2}}$ (Eq. (12)).

Thus

$$\frac{\sigma_I}{f_c} = x - \psi \quad \text{and} \quad \frac{\sigma_{II}}{f_c} = x + \psi$$

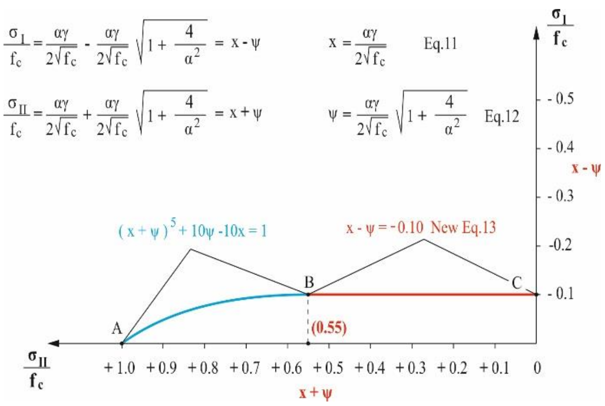


Fig. 2 The line equation of *x*–*ψ*=-0.10 (new Eq. (13)) was adopted for the realistic representation of the concrete biaxial strength curve in order to calculate the beam-column joint ultimate strength

Each couple of values $(x + \psi\ ,\ x - \psi)$ gives a point of the concrete biaxial strength curve according to Eq. (13). This curve is shown in Fig. 1(e). The initial equation is $(x+\psi)^5 + 10\psi - 10x = 1$ and describes the biaxial strength of concrete. When $x+\psi \leq 0.55$, then $(x+\psi)^5 \leq 0.051$, which is quite small (5%). Thus, it can easily be ignored without any important effect in the curve morphology; hence, the simplified form of the curve for its part BC becomes a line equation of $x - \psi = -0.1$ (new Eq. (13)).

$$x - \psi = -0.1 \qquad \text{new (13)}$$

$x + \psi \leq 0.55$ corresponds to aspect ratio values $h_b/h_c \leq 2.0$ (Fig. 2 and Table 1).

In order to estimate the error between the original nonlinear equation $(x+\psi)^5 + 10\psi - 10x = 1$ and the approximate one $x - \psi = -0.1$, we evaluate the root mean square error (RMSE) in the interval $p(= x + \psi) \in (0, 0.55)$, which is given by

$$E_{rror} = \sqrt{\frac{\int_0^p |q_1(p) - q_2(p)|^2 dp}{\int_0^p |q_1(p)|^2 dp}} = 0.0152 = 1.5\%$$

(initial equation)

$$q_1 = (1 - p^5)/(-10)$$

(approximation) $-10q = 1 \Longrightarrow\ q_2 = -0.10$

$$p = x + \psi$$

$$q = x - \psi$$

From the numerical estimation of RMSE it is derived that the error in this interval is 0.0152, i.e., 1.5%, which clearly is quite small, justifying further the adoption of the line equation as an adequate approximation of the true nonlinear equation. A characteristic example of the deviation between the nonlinear equation and the line equation is given in the following figure, where, for example, the real value of q from the nonlinear equation for $p = 0.55$ is $q = -0.09497$ and the approximated one from the line equation at $p = 0.55$ is $q = -0.1$, accordingly. As in the case of the RMSE, the error is again negligible.

In Fig. 3 is shown the aspect ratio $\alpha = h_b/h_c$ of a

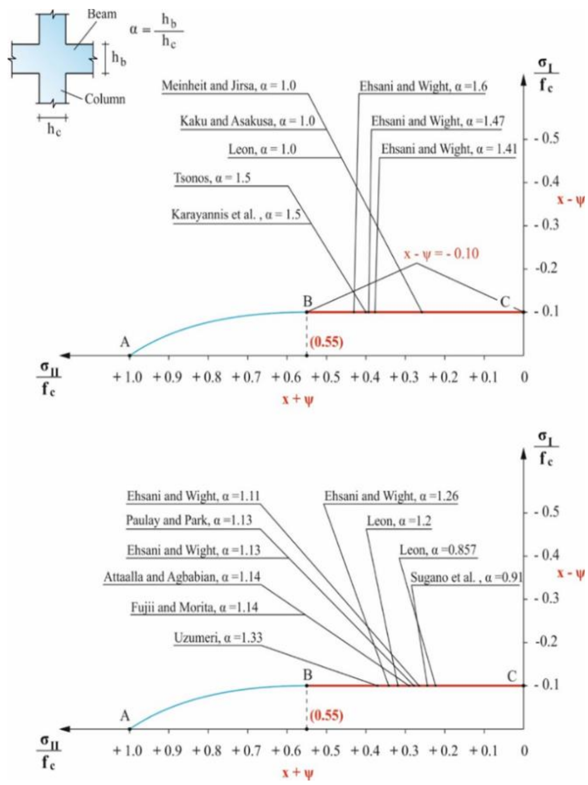


Fig. 3 The joint aspect ratio of a large number of beam-column subassemblages tested by international researchers

large number of beam-column subassemblages tested by international researchers. Their subassemblages are representative of the beam-column joints in real practice. It is worth noting that their joint aspect ratios values of all the subassemblages were lower than 2.0. Thus, the line equation of $x - \psi = -0.1$ (new Eq. (13)) was adopted for the realistic representation of the concrete biaxial strength curve in order to calculate the beam-column joint ultimate strength.

The aspect ratio values $\alpha = h_b/h_c$ in real practice are lower than 2.0 as is shown from representative beam-column subassemblages tested by international researchers. However, it is well known that the minimum dimensions of columns in real practice (in the common block of flats) are 40cm x 40cm due to beams framing at the joint and especially due to the beam bars passing generally from two directions through the joint region. These minimum column dimensions are also dictated by the best conditions of concrete casting in the joint region, where the beam and column bars are very congested. The beam height is usually between 70 cm and 75 cm in the common block of flats. Thus, α=70 cm/ 40 cm≤2.0 (Fig. 4).

Thus, the solution of the new system of Eqs. (11)-(new 13) gives the beam-column joint ultimate strength $\tau_{ult} = \gamma_{ult}\sqrt{f_c}\ (MPa)$. This system is solved each time for a given value of the joint aspect ratio using standard mathematical analysis. The joint ultimate strength τ_{ult} depends on the increased joint concrete compressive strength due to confining f_c and on the joint aspect ratio α.

Table 1 The solution of the system of Eqs. (11) to (13) for various values of $\alpha=h_b/h_c$

	$\alpha=h_b/h_c$	X	Ψ	$X+\Psi$	$X-\Psi$
1	α=0.50	0.032015609	0.132	0.164	- 0.09998
2	α=0.75	0.054091377	0.15405231	0.208	- 0.0999
3	α=0.88	0.0673829	0.167312	0.235	- 0.0999
4	α=1.00	0.080802804	0.1806	0.2614	- 0.0998
5	α=1.11	0.0940905	0.193892	0.288	- 0.0998
6	α=1.20	0.105663	0.205372	0.311	- 0.099709
7	α=1.25	0.11236060	0.21200155	0.324	- 0.0996
8	α=1.33	0.12346497	0.22296599	0.346	- 0.099
9	α=1.38	0.13063922	0.23002898	0.36	- 0.099
10	α=1.41	0.13502659	0.23433907	0.370	- 0.099
11	α=1.47	0.14397806	0.24310901	0.39	- 0.099
12	α=1.50	0.14853744	0.2475624	0.396	- 0.099025
13	α=1.60	0.16409353	0.26267781	0.427	- 0.099
14	α=1.75	0.18821416	0.28582056	0.474	- 0.0976
15	α=1.77	0.19147564	0.28891716	0.48	- 0.098
16	**α=2.00**	**0.22896255**	**0.32380**	**0.553**	**- 0.095**
17	α=2.50	0.30153823	0.38615	0.68768	- 0.0846
18	α=2.75	0.33010119	0.40816923	0.736	- 0.078
19	α=3.00	0.3536	0.425	0.7786	- 0.0714
20	α=3.25	0.3729286	0.43788525	0.810	- 0.065
21	α=3.50	0.3888	0.4478	0.8366	- 0.059
22	α=3.75	0.40201415	0.45561604	0.857	- 0.0536
23	α=4.00	0.413047	0.46180080	0.8748478	- 0.0488
24	α=5.00	0.44290408	0.4770223	0.92	- 0.034
25	α=6.00	0.45977187	0.48464211	0.9444	- 0.0248

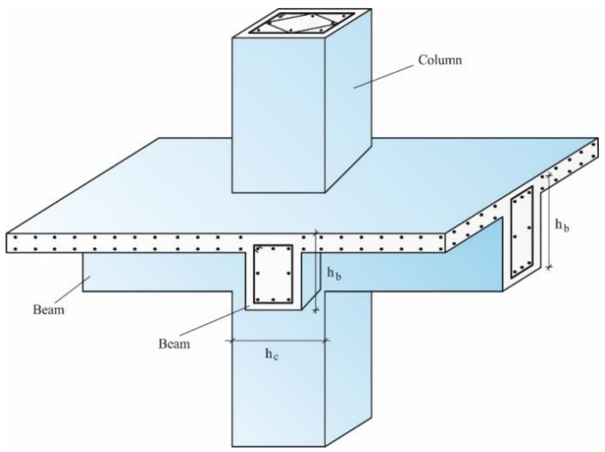
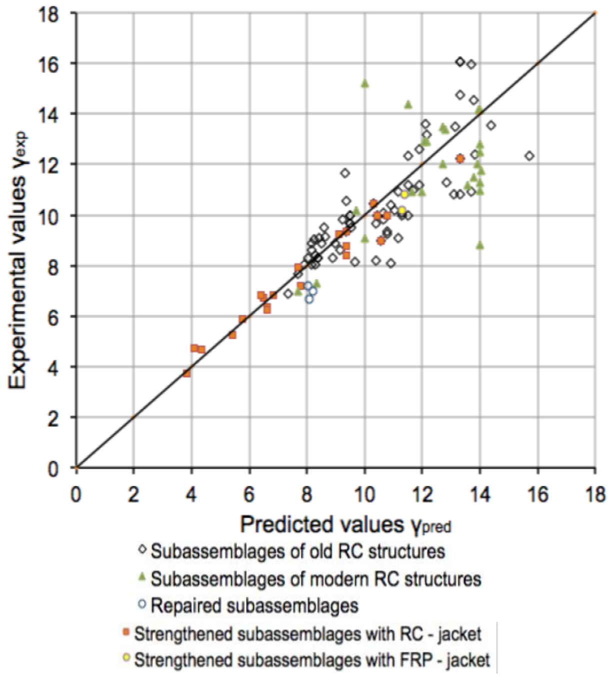


Fig. 4 Interior beam-column subassemblage with transverse beams

The validity of the formulation was checked using test data for more than 120 exterior and interior beam – column subassemblages that were tested in the Structural Engineering Laboratory at the Aristotle University of Thessaloniki, as well as data from similar experiments carried out in the United States, Japan and New Zealand (Tsonos 2007). In Fig. 5 the comparison is shown between experimental and predicted results by the preceding methodology. A very good correlation is observed (Fig. 5). The applied loading procedure used in these subassemblages was a typical one commonly used in this field of research. According to this method the subassemblages were subjected to many cycles of loading



$\left(\tau_{pred}=\gamma_{pred}\sqrt{f_c},\quad \tau_{pred}=\gamma_{\exp}\sqrt{f_c}\right)$

τ_{pred} is the predicted shear strength

τ_{exp} is the observed shear strength

Fig. 5 Predicted γ_{pred}, versus observed γ_{exp} values of various subassemblages

applied by slowly imposed earthquake-type loads or displacements. The vast majority of the experimental research studies in this direction adopt similar test implementation. The strain rate of the load applied corresponded to static conditions. Nevertheless during a real seismic excitation, the strain rate, $\dot{\varepsilon}$, is higher than the rate corresponding to static conditions. Soroushian and Sim (1986) showed that an increase in $\dot{\varepsilon}$ with respect to static conditions leads to a moderate increase in the strength of concrete

$$f_{c,dyn}=[1.48+0.160\cdot\log\dot{\varepsilon}+0.0127(\log\dot{\varepsilon})^2]\cdot f_{c,stat} \tag{14}$$

Scott *et al.* (1982) tested column specimens with various amounts of hoop reinforcement under strain rates ranging from $0.33\cdot10^{-5}\,\mathrm{sec}^{-1}$ (static loading), to $0.0167\cdot10^{-5}\,\mathrm{sec}^{-1}$ (seismic loading). Their test results conformed with the results obtained from Eq. (14). Using the aforementioned expression it is estimated that for a strain rate of $\dot{\varepsilon}=0.0167\,\mathrm{sec}^{-1}$ concrete strengths increase by about 20% (compared with the static one). An expression similar to Eq. (14) can be found in the C.E.B. Code (1990). Thus the strengths exhibited by the subassemblages used in this study during the tests are somewhat lower than the strengths they would exhibit if subjected to load histories similar to actual seismic events.

Example of Subassemblage A_1

i. According to the simplified methodology, the value $\alpha=1.5$ and the solution of the system of the above equations Eqs. (11)-(new 13) gives $x=0.15$,

$\psi = 0.25$, $f'_{c(A_1)} = 35MPa$, $k_{(A_1)} = 1.558$ according to Scott *et al*. model and

$f_{c(A_1)} = k_{(A_1)} \cdot f'_{c(A_1)} = 54.53MPa.$

Eq. (11) gives

$\gamma_{ult(A_1)} = \frac{2\cdot(0.15)\cdot\sqrt{54.53}}{1.5} = 1.48$ and finally

$\tau_{ult(A_1)} = 1.48 \cdot \sqrt{54.53} = 10.90MPa.$

ii. According to the initial methodology, the value $\alpha = 1.5$ and the solution of the system of Eqs. (11)-(13) gives $x = 0.1485$, $\psi = 0.248$, $f'_{c(A_1)} = 35MPa$, $k_{(A_1)} = 1.558$ according to the Scott *et al*. model and

$f_{c(A_1)} = k_{(A_1)} \cdot f'_{c(A_1)} = 54.53MPa.$

Eq. (11) gives

$\gamma_{ult(A_1)} = \frac{2\cdot(0.1458)\cdot\sqrt{54.53}}{1.5} = 1.46$ and finally

$\tau_{ult(A_1)} = 1.46 \cdot \sqrt{54.53} = 10.78MPa.$

It is obvious that the proposed simplified methodology is as accurate as the initial methodology.

3. The contribution of the proposed model to the safety of structures

3.1 The model assures the formation of plastic hinges in the beams and the integrity of the columns and the beam-column joints during strong earthquakes

Using this model we can predict with a high degree of precision the joint ultimate strength $\tau_{ult} = \gamma_{ult}\sqrt{f_c}$.

The improved retention of strength in the tested beam – column subassemblages as the values of the ratio $\tau_{cal}/\tau_{ult} = \gamma_{cal}/\gamma_{ult}$ decrease was demonstrated. For $\tau_{cal}/\tau_{ult} \leq 0.50$ the beam–column joints of the subassemblages behaved excellently during the tests and remained intact at the conclusion of the tests (Tsonos 1997, 1999, 2006, 2007).

$\tau_{cal} = \gamma_{cal}\sqrt{f_c}$ is the acting calculated joint shear stress.

$\tau_{cal} = \gamma_{cal}\sqrt{f_c}$ is calculated from the horizontal joint shear force assuming that the top reinforcement of the beam yields (Fig. 1(a)). In this case the horizontal joint shear force is expressed as

$$V_{jhcol} = 1.25(A_{s1} + A_{s2}) \cdot f_y - V_{col} \qquad (15)$$

Where A_{s1} is the top beam longitudinal reinforcement (Fig. 1(a)), A_{s2} is the bottom beam longitudinal reinforcement (Fig. 1(a)), f_y is the yield stress of these reinforcements and V_{col} is the column shear force (Fig. 1(a)). Development of inelastic relations at the faces of joints of reinforced concrete frames is associated with strains in the flexural reinforcement well in excess of the yield strain. Consequently, joint shear force generated by the flexural reinforcement, is calculated for a stress of $1.25 \cdot f_y$ in the reinforcement (ACI – ASCE 352 – 02).

It is clear that a significant innovation is introduced in the design of earthquake – resistant reinforced concrete structures which involves also an important contribution to the safety of these structures.

Thus in the old reinforced concrete structures the "strong columns and weak beams" rule resulted by the flexural strength ratio, which must be higher than 1.0, was completely unknown. Thus often in old buildings there are much stronger beams than the columns framing at the joints.

However, in the modern reinforced concrete structures the "strong columns – weak beams" method does not secure the safe formation of plastic hinges only in their beams and consequently the formation of the safe elastoplastic mechanism with only a beam failure mechanism. In studies by Tsonos (2006, 2007) failure modes of four subassemblages designed according to modern codes which were imposed to strong seismic loading are presented. However, two of them exhibited premature joint shear failure from the early stages of the seismic loading (Tsonos 2006, 2007). This occurs because sometimes the modern codes do not include any earthquake – resistant design of beam – columns joints for shear strength (new Greek Codes) or the methods which are adopted (Eurocodes 2 and 8) do not secure the safe and exclusive formation of plastic hinges in the beams of the structures (Tsonos 2006, 2007). Thus, even today the modern codes do not secure the safety of new structures. In some cases, safety could be jeopardized during strong earthquakes by premature joint shear failures. The joints could at times remain the weak link even for structures designed in accordance to current model building codes (Tsonos 2006, 2007).

This serious problem can be solved by the use of the new model and new method proposed in this study. Thus using this model and taking into consideration in the joint regions that $\tau_{cal}/\tau_{ult} \leq 0.50$ and of course using the rule "strong columns – weak beams" it can be secured that the exclusive formation of plastic hinges in the beams of a reinforced concrete structure but also the damage concentration only in the beams. The columns and the beam – column joints would remain intact during strong earthquakes. The safety of the structures and the lives of the people inside them are significantly secured. This innovative method significantly improves the design approach of the modern codes.

The question arises: How can a model that gives the ultimate strength of a reinforced concrete beam-column joint and predicts the actual value of the joint shear stress also be used for the prediction of the actual values of shear forces and moments developed in the columns and in the beams, framing at the joint, during strong earthquakes? The answer can be found in Paulay and Priestley (1992), who clearly demonstrated that shear forces acting in beam – column joints are significantly higher than those acting in their adjacent columns. Thus, joints fail earlier than columns during a strong earthquake motion.

Consequently, a model that predicts the actual value of the joint's shear stress could also generally predict the actual values of shear forces and moments resisted by the structure in the beams and columns, in the vicinity of the joint region during strong earthquakes.

It is worth mentioning at this point that predicting the actual values of connections shear stresses during an earthquake also involves predicting the actual values of the structures flexural strength ratio M_R ratio with the same accuracy.

3.2 The model can be used to predict the failure mode of the structures in the vicinity of the joint regions

When the calculated joint shear stress τ_{cal} is greater or equal to the joint ultimate strength, ($\tau_{cal} = \gamma_{cal}\sqrt{f_c} \geq \tau_{ult} = \gamma_{ult}\sqrt{f_c}$) , then the predicted actual value of connection shear stress will be near τ_{ult}. This is because the connection fails earlier than the adjacent beam(s). When the calculated joint shear stress τ_{cal} is lower than the connection ultimate strength ($\tau_{cal} = \gamma_{cal}\sqrt{f_c} < \tau_{ult} = \gamma_{ult}\sqrt{f_c}$), then the predicted actual value of the connection' s shear stress will be near τ_{cal} because the connection permits its adjacent beam(s) to yield.

$\tau_{ult} = \gamma_{ult}\sqrt{f_c}$ is calculated from the solution of the system of Eqs. (11)-(13).

From the above analysis it is clear that the model gives the failure mode of structures (beam failure, joint failure, column failure). However, for $\tau_{cal}/\tau_{ult} \leq 0.50$ a pure beam failure mode is secured.

This methodology is very useful for the old reinforced concrete structures in order to determine their failure mode during strong future earthquakes after finding their reinforcing details and their concrete compressive strengths e.g. to find out the crucial beam – column joint or column failures in case it would happen which would involve dangerous collapses for these structures.

After the determination of the predicted failure modes, the retrofitting schemes of the old structures with the use of the model would be easily and more safely designed. So strengthened buildings in future strong earthquakes would behave in a safe beam failure mode (Tsonos 1999, 2001a, 2001b, 2002).

The proposed model is also very useful for new structures designed according to modern codes in order to determine what mode of failure would be exhibited and how dangerous this would be for the inhabitants living inside them during strong earthquakes (Tsonos 2006, 2007). Crucial questions about the seismic behavior of new structures would be raised:

i. Whether they would exhibit an exclusive beam failure mode as they were theoretically designed to behave according to the modern codes or

ii. Whether the beam – column joints of these modern structures are potentially their weak links, as in the old structures, and would demonstrate a premature joint shear failure during strong earthquakes, which could lead to dangerous collapses.

With the use of the new model the potential inadequacy of modern structures could be detected and a safer retrofitting scheme for these modern structures could be introduced.

4. Conclusions

1. The design assumptions of modern codes do not soundly help avoid a premature joint shear failure and do not secure the development of the optimal failure mechanism with plastic hinges in the beams near their adjacent column according to the requisite "strong column/ weak beam". The proposed model and method secure the exclusive formation of plastic hinges in the beams and the concentration of the damage only in the critical regions of the beams. According to the proposed model the columns and the beam – column joints remain intact during strong earthquakes.

2. The proposed model can be used both for old structures and the modern structures in order to determine their failure mode during future earthquakes. The model can also be used in order to choose and propose the best retrofitting scheme for both old and modern reinforced concrete structures.

3. The proposed simplified methodology is much easier to use than the initial methodology. However, the simplified methodology is as accurate as the initial methodology.

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