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Capacity Design for R/C Structures According to EN 1998-1:2004

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Abstract. Capacity Design is a design process in which it is decided which objects within a structural system will be permitted to yield (ductile components) and which objects will remain elastic (brittle components) .Material nonlinearity need only be modeled for ductile components, while components which will not yield need only consider elastic stiffness properties. In this design philosophy the capacity design approach that is currently used in practice demands strong column/weak beam frames or wall equivalent dual frames, with beam sway mechanisms, trying to involve plastic hinging at all beam ends.This paper aims to present Capacity design procedure for reinforced concrete (R/C) structures according to the EN 1998-1:2004.

Keywords: Capacity design, ductility, strong column/weak beam, plastic hinge

Introduction

The approach adopted by EC8 apply the principles of capacity design in order to design earthquake resistant structures (DCM and DCH) .Capacity Design is a design process in which it is decided which objects within a structural system will be permitted to yield (ductile components) and which objects will remain elastic (brittle components) .The capacity design approach that is currently used in practice demands strong column/weak beam frames or wall equivalent dual frames, with beam sway mechanisms, trying to involve plastic hinging at all beam ends.This paper aims to present Capacity design procedure for reinforced concrete (R/C) structures according to the EN 1998-1:2004.A frame structure 7 storey with grids : 4.0 ,5.0 and 6.0 m is used as example (Fig. 1) with Elements : slabs 20cm,Basement 60 cm,Beams 30/40 cm,Columns 50/50 cm, Surface support KR3 = 36000 kN/m3.Seismic actions are estimates according to the EC-8 elastic response spectrum ,Type I for ground C,ground acceleration equal to αg=0.20g,the importance factor γ_1=1.0. The accidental eccentricity is taken into account for each seismic actions Structure is regular by elevation, Frame system,-multistorey, αu/α1=1.3.Ductility class DCM: Behaviour factor: q=3.9.

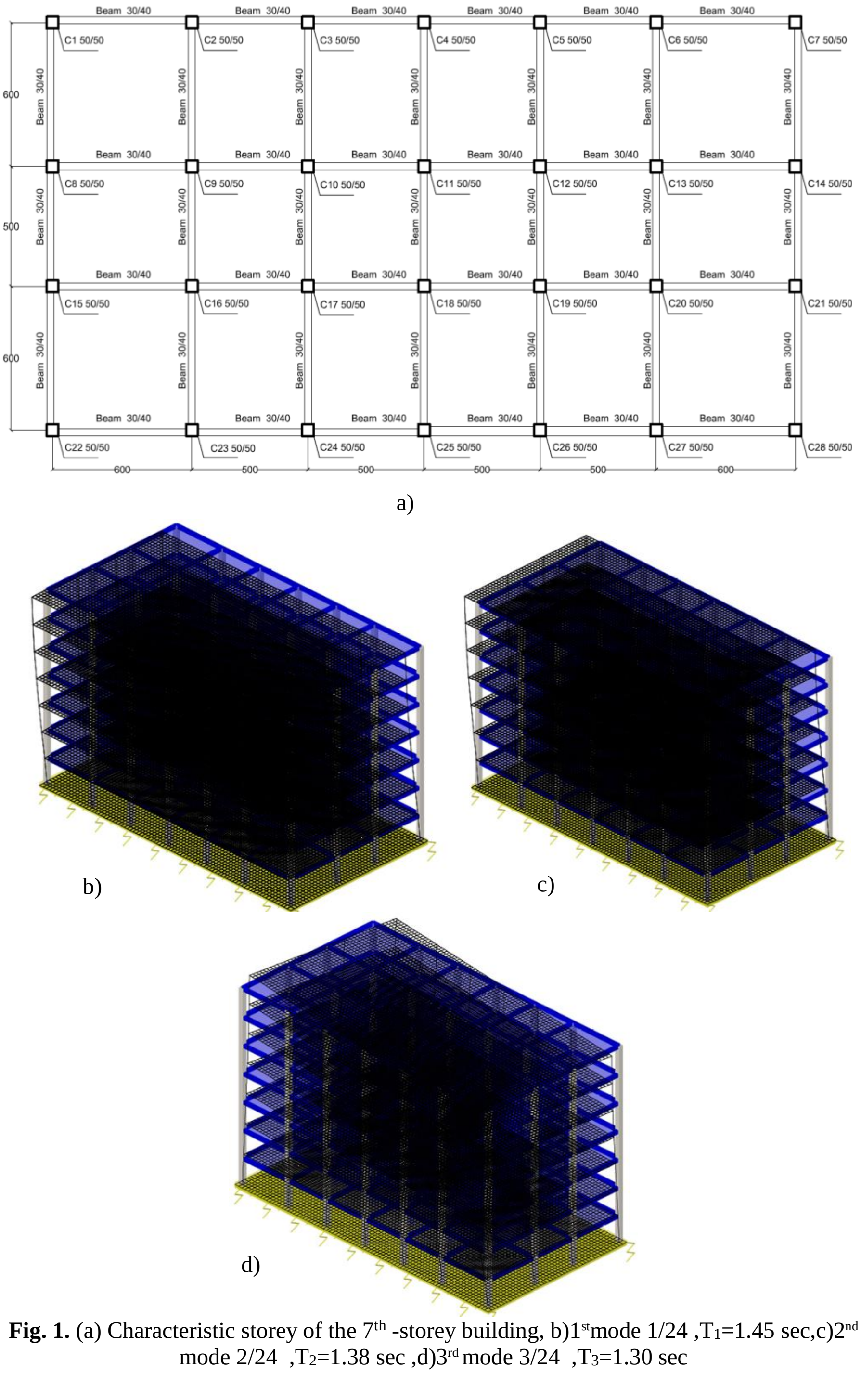

Fig. 1. (a) Characteristic storey of the 7th -storey building, b)1stmode 1/24 ,T_1=1.45 sec,c)2nd mode 2/24 ,T_2=1.38 sec ,d)3rd mode 3/24 ,T_3=1.30 sec

Design action effects

Beams

In primary seismic beams the design shear forces shall be determined in accordance with the capacity design rule, on the basis of the equilibrium of the beam EN 1998-1:2004 (E) under: a) the transverse load acting on it in the seismic design situation and b) end moments Mi,d (with i=1,2 denoting the end sections of the beam), corresponding to plastic hinges formation for positive and negative directions of seismic loading. At end section i, two values of the acting shear force should be calculated, i.e. the maximum VEd,max,i and the minimum VEd,min,i corresponding to the maximum positive and the maximum negative end moments Mi,d that can develop at ends 1 and 2 of the beam. End moments Mi,d may be determined as follows:

$$M_{i,d} = \gamma_{Rd} M_{Rb,i} min\left(1, \frac{\Sigma M_{rc}}{\Sigma M_{rb}}\right) \quad \textbf{(1)}$$

Where

γ_{Rd} is the factor accounting for possible overstrength due to steel strain hardening, which in the case of DCM beams may be taken as being equal to 1,0;

$M_{Rb,i}$ is the design value of the beam moment of resistance at end i in the sense of the seismic bending moment under the considered sense of the seismic action;

ΣM_{rc} and ΣM_{rb} are the sum of the design values of the moments of resistance of the columns and the sum of the design values of the moments of resistance of the beams framing into the joint, respectively (see 4.4.2.3(4)).[1]

The value of ΣM_{rc} should correspond to the column axial force(s) in the seismic design situation for the considered sense of the seismic action.

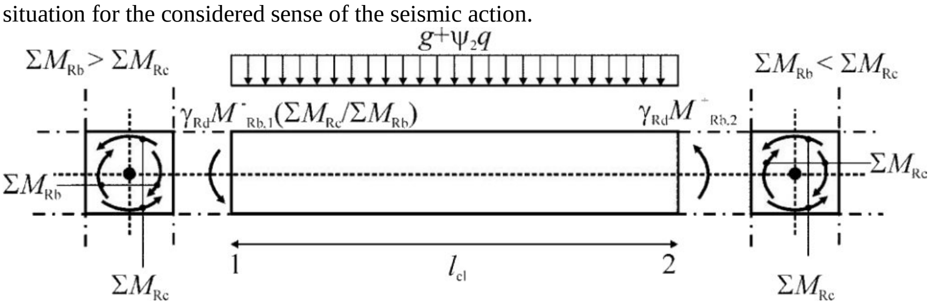

Fig. 2. Capacity design values of shear forces on beams

Equilibrium of forces and moments on a beam

$$V_{1=}V_{g+\psi q,1} + \frac{M_2+M_1}{l_{cl}} \quad \textbf{(2)}$$

$$V_{2=}V_{g+\psi q,2} + \frac{M_1+M_2}{l_{cl}} \quad \textbf{(3)}$$

Capacity-design shear in a beam weaker than the columns:

$$V_{CD,1=}V_{g+\psi q,1} + \gamma_{Rd} \frac{M^-{}_{Rd,b1}+M^+{}_{Rd,b2}}{l_{cl}} \quad \textbf{(4)}$$

$$V_{CD,2=}V_{g+\psi q,2} + \gamma_{Rd} \frac{M^-{}_{Rd,b1}+M^-{}_{Rd,b2}}{l_{cl}} \quad \textbf{(5)}$$

Capacity-design shear in beams (weak or strong) - Eurocode 8

$$maxV_{i,d}(x) = \frac{\gamma_{Rd}\left|M_{Rd,bi}{}^- min\left(1;\frac{\Sigma M_{rc}}{\Sigma M_{rb}}\right)_i + M_{Rd,bj}{}^+ + min\left(1;,\frac{\Sigma M_{rc}}{\Sigma M_{rb}}\right)_j\right|}{l_{cl}} + V_{g+\psi q,0}(x) \quad \textbf{(6)}$$

$$minV_{i,d}(x) = \frac{\gamma_{Rd}\left[M_{Rd,bi}{}^{+}min\left(1;\frac{\Sigma M_{rc}}{\Sigma M_{rb}}\right)_i + M_{Rd,bj}{}^{-} + min\left(1;,\frac{\Sigma M_{rc}}{\Sigma M_{rb}}\right)_j\right]}{l_{cl}} + V_{g+\psi q,0}(x) \tag{7}$$

in DC M γ_{Rd}=1.0, in in DC H γ_{Rd}=1.2 & reversal of V accounted for, depending on:
The sign of the ratio:

$$\zeta_1 = \frac{minV_{i,d}(x_i)}{maxV_{i,d}(x_i)} \tag{8}$$

Capacity design of columns Bending

The formation of plastic hinges in the columns during the earthquake should be avoided so that the energy is dissipated by the beams only (Park, 1986).
In the case of plastic hinges at columns, the total required plastic rotations are developed at the top and bottom of the columns of (soft storey), while in the case of plastic hinges, they develop at the beams and are spread to all storeys of frame(Fig.3.b)[3]

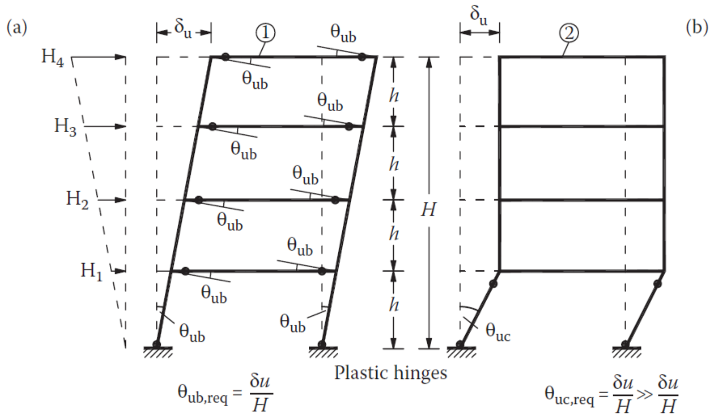

Fig. 3. Failure mechanism of a frame: (a) Beam mechanism; (b)soft- storey mechanism;

$$\theta_{uc}^{avail} \leq \theta_{ub}^{avail} \tag{9}$$

for the same δ_{ureq} req (i.e., the same μ_{ureq})

$$\theta_{uc}^{avail} = \frac{\delta_{ureq}}{h} \geq \theta_{ub}^{avail} = \frac{\delta_{ureq}}{H} \tag{10}$$

"Strong columns and weak beams"

Frames or frame-equivalent dual systems must be designed to have 'strong columns and weak beams' (Park, 1986; Paulay et al., 1990; Priestley and Calvi, 1991; Penelis and Kappos, 1997). This concept is adopted in the requirements of EC8-1 and other relevant Codes.

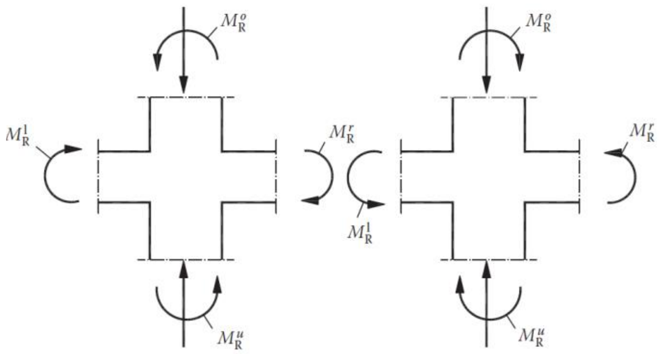

Fig. 4. Strong columns–weak beams

Detailing for local ductility

$$\left|M_{R,1}^{c,0}\right| + \left|M_{R,1}^{c,u}\right| \geq 1.30\left(\left|M_{R,1}^{b,l}\right| + \left|M_{R,1}^{b,r}\right|\right) \quad \textbf{(11)}$$

$$\left|M_{R,2}^{c,0}\right| + \left|M_{R,2}^{c,u}\right| \geq 1.30\left(\left|M_{R,2}^{b,l}\right| + \left|M_{R,2}^{b,r}\right|\right) \quad \textbf{(12)}$$

Factor 1.30 has been introduced in order to take into account the variability of the yield stress f_y of the reinforcement and the probability of overstrength factor.

$$\left.\begin{aligned} M_{S1CD} &= a_{CD1} M_{S1} \\ M_{S2CD} &= a_{CD2} M_{S2} \end{aligned}\right\} \quad \textbf{(13)}$$

Where

$$\left.\begin{aligned} a_{CD1} &= 1.30\,\frac{\left|M_{R,1}^{b,l}\right| + \left|M_{R,1}^{b,r}\right|}{\left|M_{S,1}^{c,0}\right| + \left|M_{S,1}^{c,u}\right|} \\ a_{CD2} &= 1.30\,\frac{\left|M_{R,2}^{b,l}\right| + \left|M_{R,2}^{b,r}\right|}{\left|M_{S,2}^{c,0}\right| + \left|M_{S,2}^{c,u}\right|} \end{aligned}\right\} \quad \textbf{(14)}$$

In the above relationships:

M_S^c is the action effects (bending moments) of the columns derived from the analysis for the seismic combination.

M_R^b is the design resisting moments of the beam derived from the design of the beams, which has already preceded column design.

EC8-1 allows a relaxation of the above capacity design criterion for wall-equivalent dual systems, uncoupled wall systems. The following cases are also exempted from the requirements of the above procedure:

• In single-storey R/C buildings and in the top storey of multi-storey buildings
• In one-quarter of the columns of each storey in plane R/C frames with four or more columns
• In two-storey R/C buildings if the value of the normalised axial load v_d at the bottom storey does not exceed 0.3 in any column

Capacity-design shear in Columns

Shear forces are determined by considering the equilibrium of the column under the actual resisting design moments at its ends (Fig. 5):

$$V_{sd,CD}=\gamma_{Rd}\frac{\kappa_A M_{AR}+\kappa_B M_{BR}}{l_c} \tag{15}$$

Where M_{AR} and M_{BR} are the actual resisting moments at the ends of the column,
In primary seismic columns the design values of shear forces shall be determined in accordance with the capacity design rule, End moments $M_{i,d}$ may be determined from the following expression:

$$M_{i,d} = \gamma_{Rd} M_{Rc,i} min\left(1,\frac{\Sigma M_{rb}}{\Sigma M_{rc}}\right) \tag{16}$$

Where
γ_{Rd} is the factor accounting for overstrength due to steel strain hardening and confinement of the concrete of the compression zone of the section, taken as being equal to 1,1;
$M_{Rc,i}$ is the design value of the column moment of resistance at end i in the sense of the seismic bending moment under the considered sense of the seismic action; ΣM_{rc} and ΣM_{rb} are as defined in 5.4.2.2(2).[1]

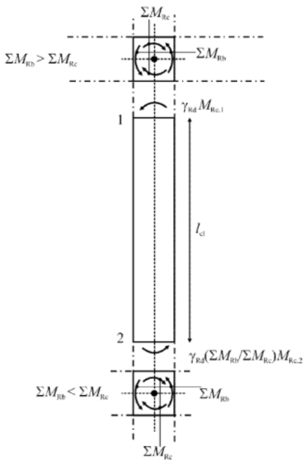

Fig. 5. Capacity design shear force in columns

Dimensioning procedure

TheSRSS method is used to combine the results of the modes considered for structure shown in (Fig. 1).From (Table 1.) conditions for interstorey drifts are fullfilled.
The storey shear forces in each direction are shown in (Fig.4),while dhe influences of second order theory will be taken into account by using influence multiplicator 1.20 for -4.Ex(-e)(0^0) and 1.23 for 5.Ey(+e)(90^0) (Table 2,3).Steps for dimensioning procedure are shown in (Fig.7).

Table 1. Interstorey drifts -7.SRSS:MAX(III,IV)+MAX(V,VI)

Level	Z(m)	Height(m)	$d_r(0^0)$(mm)	$d_r(90^0)$(mm)	$d_{r.k}(mm)$	$d_r lim$
Storey 6	18	3.00	10.14	11.51	15.34	75.00
Storey 5	15	3.00	16.66	18.19	24.66	75.00
Storey 4	12	3.00	21.92	23.51	32.14	75.00
Storey 3	9	3.00	26.12	27.69	38.07	75.00
Storey 2	6	3.00	29.55	31.10	42.90	75.00
Storey 1	3	3.00	31.26	32.66	45.21	75.00
Ground floor	0	3.00	23.31	24.11	33.53	75.00

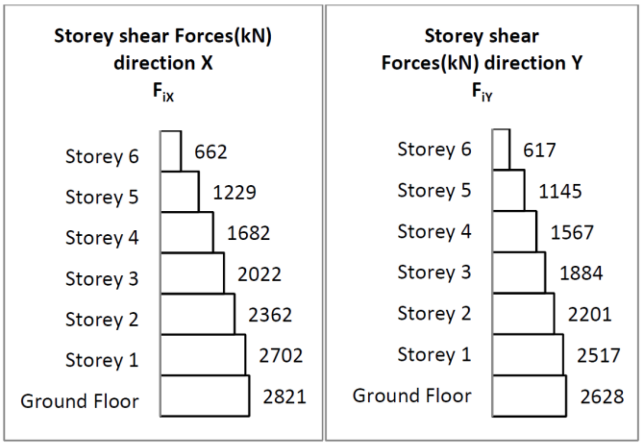

Fig. 6. Storey shear forces along the elevation for two horizontal directions

Table 2. Interstorey drift sensitivity coefficient -4.Ex(-e)(0^0)

Level	Z(m)	Height(m)	Weight(m)	Seis.Force(Kn)	$\Delta s(mm)$	θ
Storey 6	18	3.00	7434.70	829.84	10.14	0.030
Storey 5	15	3.00	15066.62	1434.59	16.66	0.058
Storey 4	12	3.00	22698.54	1884.11	21.92	0.088
Storey 3	9	3.00	30330.46	2241.39	26.12	0.118
Storey 2	6	3.00	37962.38	2552.85	29.55	~~0.146~~
Storey 1	3	3.00	45594.30	2819.88	31.26	0.169
Ground floor	0	3.00	53617.90	2998.52	23.31	0.139

Table 3. Interstorey drift sensitivity coefficient -5.Ey(+e)(90^0)

LevelZ(m)		Height(m)	Weight(m)	Seis.Force(Kn)	$\Delta s(mm)$	θ
Storey 6	18	3.00	7434.70	812.66	11.51	0.035
Storey 5	15	3.00	15066.62	1386.68	18.19	0.066
Storey 4	12	3.00	22698.54	1801.64	23.51	0.099
Storey 3	9	3.00	30330.46	2127.23	27.69	0.132
Storey 2	6	3.00	37962.38	2417.57	31.10	0.163
Storey 1	3	3.00	45594.30	2675.10	32.66	0.186
Ground floor	0	3.00	53617.90	2851.28	24.11	0.151

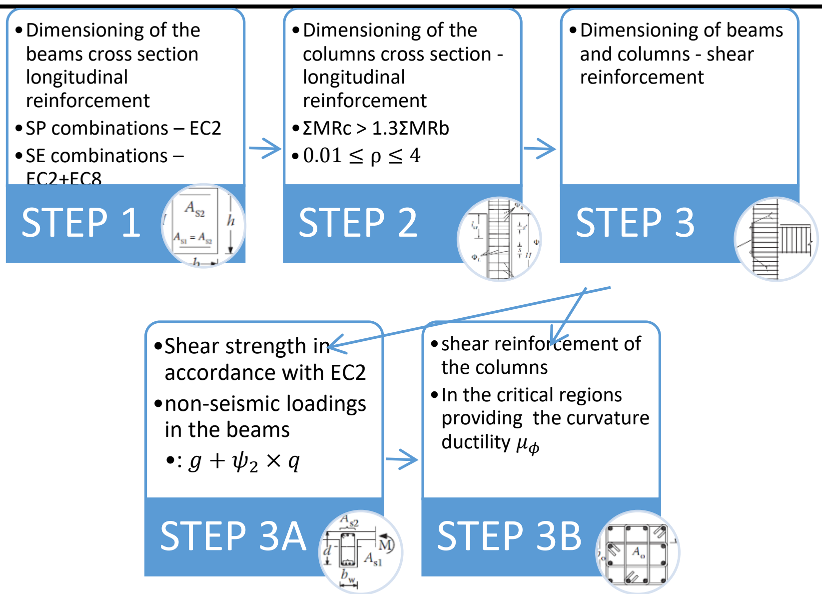

Fig. 7. Steps for dimensioning procedure - capacity design

Table 4. Critical zone length for ductility classes

Critical zone length	**DCL**	**DCM**	**DCH**
l_{cr}	$l_n/4$	$l_n/4$ h_w **(5.4.3.1.2(1))**	$l_n/4$ $1.5\ h_w$ **(5.5.3.1.2(1))**

Note:ln – span clear length.

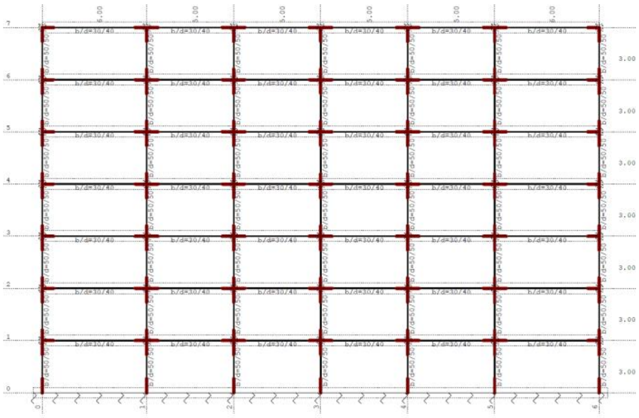

Fig. 8. Critical regions for frame structure

Conclusions

- Avoiding column failure is much more crucial for the overall safety of the structure than avoiding beam failure in bending.
- The formation of plastic hinges in the columns leads to significant inter-storey drifts, so that the relevant second-order effects may lead to a premature collapse of the structure
- In order to decrease the probability of plastic hinge formation in the columns, frames or frame-equivalent dual systems must be designed to have 'strong columns and weak beams'
- The torsionally flexible frame or frame-equivalent systems should also comply with the capacity design procedure because of their additional vulnerability, which is attributable to the torsional behaviour of the system even no reference is made to these systems in the EC8-1/2004
- As shown in (Fig.7) Longitudinal reinforcement of Columns depends from longitudinal beams reinforcement ,Shear reinforcement depends from longitudinal reinforcement ,all elements are linked with „ interdependence“

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