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SEISMIC DESIGN CRITERIA FOR MULTI-STOREY PRECAST STRUCTURES

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ABSTRACT:

This paper presents a simplified method for the capacity design of multi-storey precast concrete frames with hinged beams. A parametric study shows that in the field of ordinary and less flexible structures, the proposed simplified method can be applied with full reliability, in the same way as prescribed by the seismic codes for monolithic cast-in-situ structures. Moreover, the displacement ductility capacity of a range of multi-storey precast concrete structures designed according to the proposed method is investigated.

KEYWORDS: precast frames, capacity design, behaviour factor, ductility

1. INTRODUCTION

Precast structures owe their optimum functionality and economy to dry-joint construction that minimizes the on-site complementary interventions. A good seismic behaviour is easily assured by the general regularity and simplicity of precast structures and by the high quality of their structural elements. However, the connections at the joints of precast structures are particularly important to ensure a good seismic behaviour for the whole structure and therefore, the necessary mechanical connectors must be appropriately over-dimensioned, because of their substantially brittle behaviour.

The modern codes for seismic design of structures, e.g. Eurocode 8 [CEN, 2004] and the Italian code [OPCM, 2003], systematically introduce the criterion of capacity design. Through this criterion, the resistance of some parts of the structure is assured to be higher than the resistance of others, so that the optimum configuration of the collapse mechanism is obtained. This configuration comprises the maximum number of plastic hinges, with high energy dissipation capacity, while brittle collapse modes, such as those due to shear, are implicitly avoided thanks to over-dimensioning.

As widely shown previously, e.g. [Biondini et al, 2001; Biondini & Toniolo, 2003b; Biondini et al, 2004b; Biondini & Toniolo 2004], in order to obtain a good seismic behaviour with high dissipation capacity, it is not necessary to orient the design of precast structures towards the emulation of cast-in-situ structures with monolithic connections. With appropriate dimensioning of elements and connections, precast structures are able to reach the same structural efficiency as monolithic cast-in-situ structures under seismic actions.

Obviously, the presence of hinged connections between beams and columns modifies the stresses and deformations and therefore also the ratios of dimensions among the elements. It is the objective of the research presented in this paper to verify the rules of structural analysis, since they have been calibrated on different values of the parameters that characterise the dynamic behaviour of the structures. In particular, the influence of the different modes of vibration on the structural response is studied in order to redefine the field of application of the two methods of linear analysis (static or modal dynamic). Also, a behaviour factor appropriate for precast frames is proposed, in the absence of specific provisions in modern seismic codes.

2. SIMPLIFIED DESIGN METHOD

2.1. Total base shear

A simplified method for the seismic design of multi-storey precast frames with hinged connections between columns and beams is described with reference to the structure shown in Figure 1.

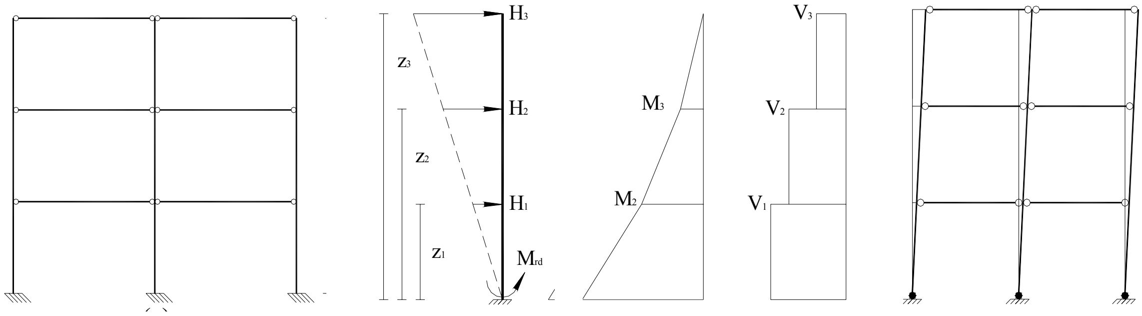


Figure 1. (a) Multi-storey frame with hinged beams, (b) distribution of storey forces, (c) bending moments, (d) shear forces, and (e) desired collapse mechanism.

Assuming a linear variation with height, a static linear analysis of the structure is performed for the horizontal forces equivalent to the seismic motion:

$$F_3 = \frac{F_{tot} z_3 W_3}{z_1 W_1 + z_2 W_2 + z_3 W_3} \quad F_2 = \frac{F_{tot} z_2 W_2}{z_1 W_1 + z_2 W_2 + z_3 W_3} \quad F_1 = \frac{F_{tot} z_1 W_1}{z_1 W_1 + z_2 W_2 + z_3 W_3} \tag{2.1}$$

where W_i is the weight of the i-th storey and the total seismic force (base shear) is calculated as:

$$F_{tot} = \alpha_g S_d (T_1) W_{tot} \tag{2.2}$$

with $W_{tot} = W_1 + W_2 + W_3$, and where α_g is the seismic intensity and $S_d(T_1)$ is the ordinate of the response spectrum for the first mode of vibration of the structure. An appropriate value of the behaviour factor q is required for the calculation of $S_d(T_1)$.

2.2. Behaviour factor q

It is known [Biondini & Toniolo, 2003a] that on the basis of a rotational ductility $\mu_\varphi \approx 7 \sim 8$, which for reinforced concrete sections in bending can be achieved by using appropriate steel, it is possible to obtain a displacement ductility μ_δ of the column:

$$\mu_\delta = (1+ \mu_\varphi) / 2 \cong 4.0 \sim 4.5 \tag{2.3}$$

Then, global ductility μ_Δ varies with the ultimate collapse mechanism according to the following formula:

$$\mu_\Delta = 1 + \frac{i}{n}(\mu_\delta - 1) \tag{2.4}$$

where n is the total number of storeys and i is the number of storeys above the plastic hinges. Assuming a value of local ductility $\mu_\delta = 4.0$ for the columns and the collapse mechanism shown in Figure 1e, the global ductility is $\mu_\Delta = 4.0$.

For relatively flexible structures with high vibration periods, as those considered in this paper, the equal displacement rule holds. According to this rule, the behaviour factor q is equal to the ductility factor which, for the values previously assumed, becomes:

$$q = \mu_\Delta = 4.0 \tag{2.5}$$

2.3. Period of vibration

In order to apply the simplified method, the first vibration period of the structure must be evaluated. The standard equations proposed by seismic codes [CEN, 2004; OPCM, 2003] give approximate default values. Within the simplified method, the following equation is used:

$$T_1 = c\left(2\pi\sqrt{\frac{W_{tot}/g}{\sum_j 3k_{1j}/h^3}}\right) = cT_0 \tag{2.6}$$

where the sum is extended to all columns, T_0 is the vibration period of a single degree of freedom system with all the mass concentrated at the last storey, $h = z_3$ for the structure in Figure 1, and c is a coefficient given in Table 1 as a function of the number n of storeys. The values of c are obtained by a parametric study under the assumption of equal storey masses and equal storey heights. The first line of Table 1 refers to the case of columns with constant flexural stiffness (k_{ij} = cost) throughout the whole height of the building, while the second line refers to the case of columns with flexural stiffness decreasing linearly with the order i of the storey ($k_{ij} = k_{1j}\,(n–i+1) / n$). It seems correct to calculate k_{1j} considering the stiffness of the cracked section [CEN, 2004].

Table 1. Coefficient c

n	1	2	3	4	∞
c	1.00	0.74	0.66	0.62	0.49
	1.00	0.78	0.71	0.67	0.57

2.4. Calculation of design action effects

This procedure represents a simple design method that for the ultimate collapse state and, in consideration of the overstrength of the upper parts of the single columns deriving from capacity design, leads only to the verification of the critical section at the base of the column:

$$M_{rd} \geq M_{\alpha d} = F_1' z_1 + F_2' z_2 + F_3' z_3 \tag{2.7}$$

where F_i' is the part of the seismic force at the i-th storey applied to the single column, in proportion to its stiffness.

In order to design a column, the capacity design is applied, assuming a linear variation of the storey forces with height:

$$H_2 = H_3\frac{z_2}{z_3} \quad H_1 = H_3\frac{z_1}{z_3} \tag{2.8}$$

where the storey forces are:

$$H_3 = M_{rd}\frac{z_3}{z_1^2+z_2^2+z_3^2} \quad H_2 = M_{rd}\frac{z_2}{z_1^2+z_2^2+z_3^2} \quad H_1 = M_{rd}\frac{z_1}{z_1^2+z_2^2+z_3^2} \tag{2.9}$$

A partial factor γ_R, divided in two components $\gamma_R = \gamma' \gamma''$, is introduced. The first component is related to the possible overstrength of the column reinforcement (e.g. $\gamma' = 1.5$), while the second one is related to the uncertainty of the assumed model of linear variation (e.g. $\gamma'' = 1.30$).

For the capacity design, the upper parts of the columns shall be proportioned for (see Figure 1):

$$\begin{aligned} M_3 &\geq \gamma'' H_3 (z_3 - z_2) = \gamma'' M_{rd} \frac{z_3(z_3 - z_2)}{z_1^2 + z_2^2 + z_3^2} \\ M_2 &\geq \gamma''[H_3(z_3 - z_1) + H_2(z_2 - z_1)] = \gamma'' M_{rd} \frac{z_3(z_3 - z_2) + z_2(z_2 - z_1)}{z_1^2 + z_2^2 + z_3^2} \end{aligned} \tag{2.10}$$

where, with $\gamma'' = 1.30$, the overstrength of the reinforcement is neglected, assuming that the same type of steel is used through the whole height of the columns.

This criterion, held for hinged frames by Eq. 2.10, should be extended also to cast-in-situ monolithic frames, for which, at present, no specific rules are given in design codes. Indeed, seismic design codes are limited to the well-known design rule of "strong columns – weak beams" that refers to the ratio of strengths at the nodes and not to the ratio of strengths available among the various storeys of the structure.

For the proportioning of the beam-column connections, the effect of the higher vibration modes must be considered. Higher modes of vibration produce forces in opposite directions at the various storeys, which give a little contribution to the column moments, but result in significant forces at the connections. For this reason, the maximum force, with $\gamma_R \approx 1.60$, is assumed to be applied to all storeys:

$$H = \gamma_R H_3 \tag{2.11}$$

Finally, the columns are dimensioned in shear according to capacity design (see Figure 1):

$$\begin{aligned} V_3 &\geq \gamma'' H_3 = \gamma'' M_{rd} \frac{z_3}{z_1^2 + z_2^2 + z_3^2} \\ V_2 &\geq \gamma''(H_3 + H_2) = \gamma'' M_{rd} \frac{z_3 + z_2}{z_1^2 + z_2^2 + z_3^2} \\ V_1 &\geq \gamma''(H_3 + H_2 + H_1) = \gamma'' M_{rd} \frac{z_3 + z_2 + z_1}{z_1^2 + z_2^2 + z_3^2} \end{aligned} \tag{2.12}$$

In this way, the shear forces are evaluated as a function of the resistant bending moment M_{rd} of the critical section, placed at the base of the column, where plastic deformation is concentrated at the ultimate collapse state of the structure.

3. VALIDATION OF THE SIMPLIFIED DESIGN METHOD

Given the high flexibility of the examined structures, the influence of the higher modes of vibration may be important and consequently the approximation of linear distribution of the storey forces could not be acceptable. In this section, a verification of the reliability of the simplified method is presented. The verification is made by varying the number n of storeys, the first vibration period T_1 assessed according to Eq. 2.6 and the type of variation of the flexural stiffness of the columns in height (constant or decreasing). The type 1 response spectrum for ground of category B of Eurocode 8 [CEN, 2004] is considered.

Table 2 reports the bending moments at the base of each storey obtained by means of modal response spectrum analysis, divided by the values obtained according to the proposed simplifies method. The actual moments at the base of storey 1 are always lower than those computed according to the simplified method. Therefore, the simplified method results reliable, under the condition that the integrity of the columns at the upper storeys is assured according to capacity design. For flexible structures with vibration periods above 2.0 sec, higher moments, beyond the possible coverage of the partial factor γ'', are obtained in the upper storeys. The simplified method is not applicable to these structures. In addition, these structures are subjected to more restrictive limits for the damage limit state, because of the excessive deformations due to their high flexibility.

Table 2. Moments at the storey base

Storey	Constant stiffness					Variable stiffness				
	T_1 (sec)					T_1 (sec)				
i	2.5	1.5	1.0	0.75	0.50	2.5	1.5	1.0	0.75	0.50
2	1.27	1.00	0.94	0.91	0.90	1.26	0.99	0.94	0.91	0.90
1	0.91	0.86	0.84	0.83	0.83	0.97	0.85	0.83	0.82	0.82
3	1.77	1.13	0.99	0.91	0.88	1.71	1.12	0.98	0.91	0.91
2	0.88	0.83	0.82	0.82	0.82	0.84	0.81	0.81	0.81	0.81
1	0.89	0.81	0.79	0.78	0.78	0.92	0.80	0.78	0.78	0.77
4	2.07	1.23	1.03	0.91	0.87	2.05	0.84	1.21	1.22	1.22
3	1.56	1.23	1.17	1.13	1.12	1.41	1.09	1.17	1.17	1.17
2	0.79	0.78	0.78	0.78	0.78	0.77	0.76	0.76	0.76	0.76
1	0.86	0.78	0.77	0.76	0.75	0.88	0.73	0.76	0.77	0.77

A parametric study [Biondini et al, 2004a] covered the possible range of variation of the principal parameters of the problem. The number of storeys n was equal to 2, 3 and 4. T_1 was equal to 2.5, 1.5, 1.0, 0.75 and 0.5 sec. Two geometries of the columns, one with constant section through the height and the other with flexural stiffness decreasing in height, were considered. The results showed that the bending moments at the base of the precast columns always remain within a limit equal to $\gamma'' = 1.30$ times the values calculated according to the simplified method. As regards the forces at the beam-column connections of the hinged frames, the effect of the higher modes of vibration leads to exceeding the value $\gamma''= 1.30$ for the most flexible frames. For $T_1 \leq 2.0$ sec, the values of the forces are always within this limit.

4. ESTIMATION OF DISPLACEMENT DUCTILITY CAPACITY

The research is extended to examine the displacement ductility capacity of multi-storey precast concrete frames. The structures were designed according to the simplified method for $\alpha_g = 0.35$ and the type 1 response spectrum for ground of category B of Eurocode 8 [CEN, 2004]. The behaviour factor was $q = 4.0$, as discussed in Section 2.2. All storey weights were $W_i = 1200$ kN. The height of all storeys was 4.0 m and the length of the bays was 6.0 m.

The design parameters are given in Tables 3 and 4 for frames with constant and variable column stiffness, respectively. Concrete C30 and steel S500 were assumed for the columns. F'_{tot} is the design base shear of a single column. $M_{\alpha d}$ is the design bending moment calculated according to Eq. 2.7, b is the width and A_s is the total amount of longitudinal reinforcement, all referring to the rectangular column at the base of storey 1. The minimum reinforcement is considered for all columns, because the required reinforcement is less than the minimum amount corresponding to $\rho_{l,min} = 0.01$ [CEN, 2004].

For the calculation of the column dimensions, the stiffness k_{1j} is estimated from Eq. 2.6 so as to obtain the desired value of T_1. This corresponds to the stiffness of the cracked cross-section, which is equal to 0.5 times the stiffness of the uncracked cross-section [CEN, 2004]. Then, E_c being the Young modulus of concrete, the width of the column is calculated as:

$$k_{1j} = 0.5E_c I = 0.5E_c b^4 / 12 \Rightarrow b = \sqrt[4]{24k_{1j} / E_c} \tag{4.1}$$

Table 3. Design parameters for frames with constant column stiffness

	2-storey frame				3-storey frame		4-storey frame
T_1 (sec)	0.75	1.0	1.5	2.5	1.5	2.5	2.5
F'_{tot} (kN)	140	105	70	34	106	50	67
$M_{\alpha d}$ (kNm)	933	699	469	224	986	470	806
b (m)	0.81	0.68	0.56	0.43	0.79	0.61	0.79
A_s (cm^2)	66.3	46.4	30.9	18.6	62.1	37.3	62.2

Table 4. Design parameters for frames with column stiffness linearly decreasing with height

	2-storey frame				3-storey frame		4-storey frame
T_1 (sec)	0.75	1.0	1.5	2.5	1.5	2.5	2.5
F'_{tot} (kN)	140	105	70	34	106	50	67
$M_{\alpha d}$ (kNm)	933	699	469	224	986	470	806
b (m)	0.84	0.70	0.57	0.44	0.82	0.63	0.82
A_s (cm^2)	69.9	48.9	32.6	19.6	66.8	40.1	67.2

Non-linear static analyses under imposed horizontal displacement at the top of the structures were performed with the computer code Cast3m [Millard, 1993]. The base of the columns at storey 1 was modelled using a fibre model [Guedes et al, 1994]. Concrete behaviour was represented by a parabolic curve up to the peak stress, followed by a straight line in the softening zone, appropriately accounting for confinement [Mander et al, 1988]. An elastic-perfectly plastic constitutive law was used for the steel elements. All other elements were assumed to remain elastic.

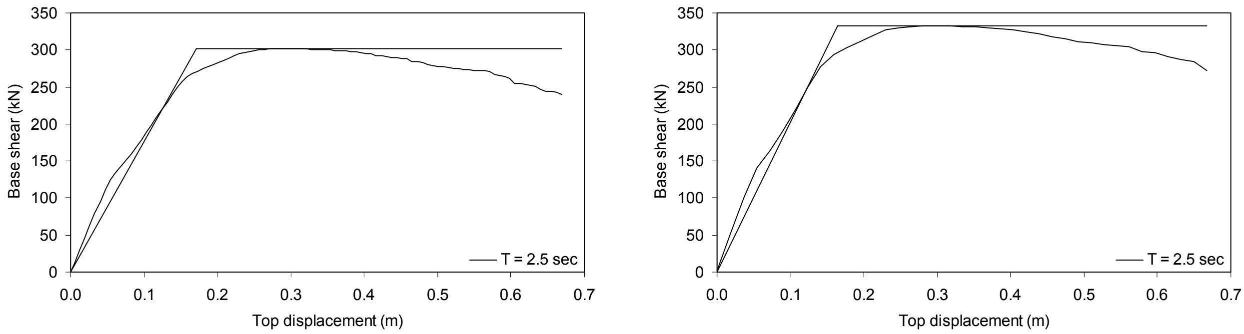
The capacity curves, i.e. the total base shear versus the horizontal displacement at the top of the structure, obtained by the non-linear analyses are shown in Figure 2. They can be used to assess the displacement ductility capacity of the structure as $\mu_\Delta = d_m / d_y$, where d_y and d_m are, respectively the yield and maximum displacements. The yield displacement d_y is estimated based on a bilinear elastic-plastic approximation of the capacity curve, where the areas under the bilinear and the actual curve, until the maximum force, are equal. The maximum displacement d_m is associated to the point of the capacity curve where the total base shear drops to 80% of the maximum value. The values of μ_Δ are given in Table 5.

Based on the obtained values of displacement ductility μ_Δ, a value of the behaviour factor q = 4.0 seems to be appropriate for structures with vibration period $T_1 \leq 2.0$ sec. For structures with $T_1 > 2.0$ sec it was already pointed out that the simplified method adopted for seismic design is not applicable. Finally, it is worth noting that the high values of displacement ductility obtained for structures with $T_1 < 1.0$ sec seem to be consistent with the equal energy rule, instead of the equal displacement rule.

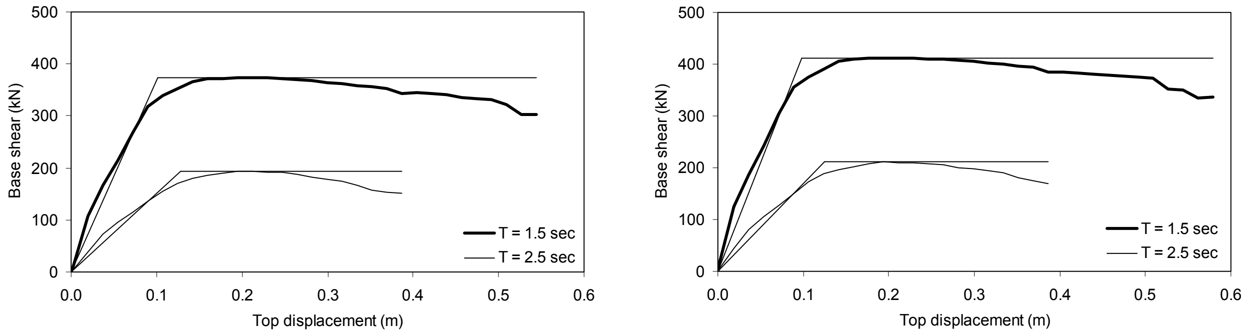
Clearly, to generalize these results, the seismic performance of these structures should be investigated by means of step-by-step non-linear dynamic analyses for a large set of accelerograms. However, despite the necessity of further investigations, the obtained results indicate that with appropriate dimensioning of elements and connections, precast structures are able to reach seismic performance as monolithic cast-in-situ structures.

Table 5. Displacement ductility capacity μ_Δ

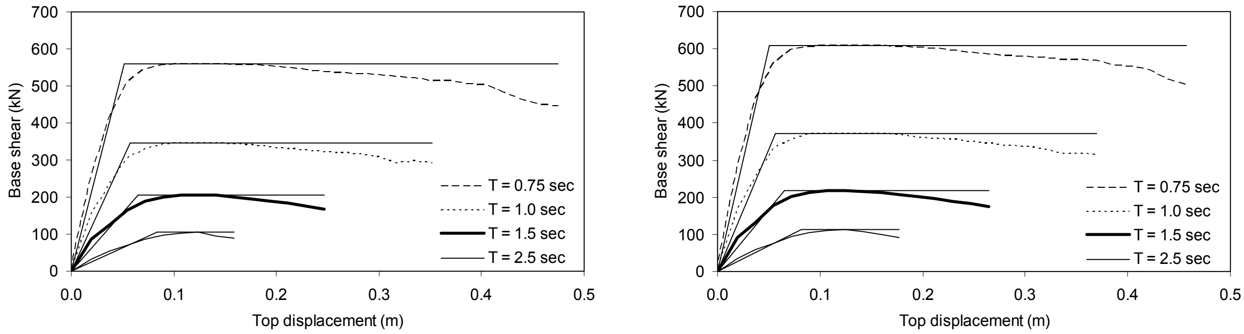
	2-storey frame				3-storey frame		4-storey frame
	T_1 (sec)				T_1 (sec)		T_1 (sec)
	0.75	1.0	1.5	2.5	1.5	2.5	2.5
Constant stiffness	9.1	6.2	3.8	1.9	5.4	3.0	3.9
Variable stiffness	9.1	6.6	4.1	2.2	5.9	3.1	4.0



4-storey frames



3-storey frames



2-storey frames

Figure 2. Capacity curves for frames with constant (left) and variable (right) column stiffness.

5. CONCLUSIONS

Based on the results of the parametric analyses, it is concluded that in the field of ordinary and less flexible structures, i.e. those with vibration periods indicatively less than 2.0 sec, the simplified method can be applied with full reliability in the same way as prescribed by the seismic codes for the similar monolithic cast-in-situ structures.

A safety factor for the model uncertainty equal to $\gamma'' = 1.30$ is introduced in order to cover the dispersion of the values resulting from the parametric analyses described in Section 3. Then, the calculations for capacity design, with the assumption of linear height-wise increase of the seismic forces, result reliable in all cases for which design according to the simplified method is permitted. This allows to verify the resistance at the base of columns only, as the sections at the upper storeys and the connections with the beams are over-designed, assuring at the same time the ductile collapse mechanism coherent with the basic value of the behaviour factor.

The displacement ductility capacity of a range of multi-storey precast concrete structures designed according to the proposed method is finally investigated. Based on the obtained values of displacement ductility, a value of the behaviour factor q=4.0 seems to be appropriate for this type of structures with vibration periods less than 2.0 sec. Further investigations are however needed to generalize this result. In any case, despite the necessity of further investigations, the obtained results indicate that with appropriate dimensioning of elements and connections, precast structures are able to reach seismic performance as monolithic cast-in-situ structures.

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