On the design of tied braced frames

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ABSTRACT: A procedure for the design of tied braced frames is presented, aiming at favouring the structural collapse by global mechanisms in correspondence of a desired value of the peak ground acceleration. The procedure is applied to a twelve story tied eccentrically braced frame also at the aim of highlighting the good behaviour of such typology with respect to an analogous traditional eccentrically braced frame, which in the past have shown quite poor seismic behaviour. The procedure is very simple and governed by some behavioural hypotheses which find good agreement in the analysis of the numerical results.

1 INTRODUCTION

By the structural point of view, tied eccentrically braced frames (TBFs) may be conceived as constituted by two braced sub-structures coupled by means of horizontal elements (later on called *links*) generally undergoing shear and flexure. The braced sub-structures are always characterized by very high story stiffness and by quite zero base bending moment resistance. The capacity to resist overturning moments derives to the whole structure from the coupling action of the links and, hence, from the vertical component of the reactions of the lateral supports of the braced sub-structures. Differently from reinforced concrete coupled walls, which the above-mentioned systems may be ideally approached, coupling beams of TBFs have high plastic rotation capacity as well as stable and large hysteresis loops.

In order to favor a very ductile and dissipative global structural behavior, the modern design philosophy applied to tied braced frames pursues the aim of limiting, in occurrence of strong ground motions, the inelastic behavior to links only. The other resisting elements have to withstand, without yielding, the maximum internal actions which develop during the elastic and inelastic phase of the seismic response. In the view of such an approach, optimal may be considered a design procedure which leads to a contemporary collapse of the links and, therefore, to a complete exploitation of their deformative and dissipative capacities. In TBFs the presence of ties, which connect the corresponding ends of links belonging to contiguous floors, particularly simplifies the definition of a procedure which intends to grant such a behavioral goal.

In order to justify such an ascertainment some considerations may be useful.

Firstly, with reference to TBFs in which the capacity design criterion is supposed to be applied and verified, a whatever instantaneous distribution of inelastic rotations involving one or more links does not cause, for successive increments of the global deformation, large concentrations of plastic rotations. Indeed, differently from traditional eccentrically braced frames (EBFs), in TBFs the story lateral stiffness (related to a unity interstory drift) is only a little influenced by the rigidity of links and, therefore, by the presence of yielding links. Such behavioral aspect permits to avoid or, at least, to make very improbable the formation of partial collapse mechanisms involving only some stories located astride the plastified links.

Secondly, although the story lateral stiffness remains rather unchanged as links yield, the global stiffness of the structure (related to a unity top displacement) shows important decreases. The progressive yielding of links, which as previously referred occurs in absence of particular concentrations of plastic deformation, makes the structural motion governed more and more by the rigid rotation of the braced sub-structures. With reference to this aspect of the structural behavior, the high rotational capacity of links (particularly when yielding is governed by shear) would have to be able to allow the regularization of the lateral deformation (from the elastic response to the rigid motion) before failure of a whatever link occurs.

2 THE DESIGN OF TIED BRACED FRAMES

Owing to the above-mentioned considerations, if the elastic deformations of the braced sub-structures are neglected and the geometric characteristics of the generic story are supposed to be constant along the height of the structure, the maximum plastic deformations demanded to links (corresponding to the ultimate rotations of links) tends to assume equal values at all stories. In order to optimize the ultimate behavior of structures which fulfill the above-mentioned geometrical conditions (as the example shown later), such an observation suggests using for the links sections characterized by equal ultimate plastic rotations. Slightly different design suggestions may be analogously derived for structures which do not verify the above-mentioned geometrical characteristics. Unfortunately, in any case such design provision allows always the selection of a more or less large class of real sections, but only sometimes that of a particular section (because according to codes and laboratory experiments the correspondence between sections and ultimate rotations is practically univocal when referring to intermediate links only).

Once the choice of the link sections has been carried out, if higher modes of vibration are negligible, consistently with the capacity criterion the other members of the structure (columns, braces and ties) may be designed by means of static equations referred to the ultimate limit state configuration of the system. With reference to this phase of design, if the shape of the lateral displacements related to the ultimate limit state is assumed equal to that caused by the rigid rotation of the braced sub-structures, the design equivalent static forces may be considered directly proportional to the mass m and to the height h of the floors from the base of the system. Nevertheless, when higher modes of vibration are important, this first design procedure has to be corrected to account for the amplification of the story shears and overturning moments with respect to the previously obtained values. Such values (previously referred as amplified), predicted in the study by means of code expressions, may occur in a whatever phase of the structural response.

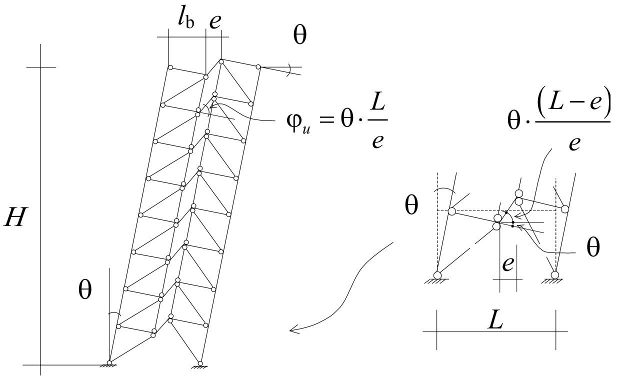

Figure 1. Displacements of the TBF caused by a rigid rotation of the braced sub-structures

Such considerations, which however constitute the basis for a design procedure aiming at an optimal seismic behavior, do not permit to force that the structural collapse occurs at a specified intensity of the design ground motion, unless the real behavior factor is known. At the aim of overcoming this problem, the knowledge of both the ultimate and maximum required top displacement (the last one as a function of the seismic intensity) is sufficient.

With regard to the first element, if a linear lateral displacement shape is assumed in correspondence of the structural ultimate limit state and elastic deformations of members are neglected, the rotation θ_u of the braced sub-structures (Fig. 1) related to the attainment of the ultimate rotation φ_u of the links may be easily evaluated by means of geometric considerations as:

$$\theta_u = \varphi_u \frac{e}{L} \tag{1}$$

being e and L the length of the link and braced span, respectively. Hence, the ultimate top displacement u_u^{top} may be derived by multiplying the ultimate rigid rotation θ_u by the total height H of the frame.

Differently from ultimate displacements, the inelastic displacements developed by structures during seismic events depend on the characteristics of the accelerometric signals and on both the dynamic and mechanic properties of the system under examination. In spite of such an apparent complexity, the mean value of the maximum required top displacements of TBFs may be evaluated with good approximation by means of constant ductility response spectra of an equivalent elastic-plastic with hardening single degree of freedom (SDOF) system. Such system may be defined as characterized by :

1 A natural period of vibration equal to the first period of vibration of the TBF.
2 An yielding force equal to the intensity of the external equivalent forces which, applied to the TBF according to an inverted triangular distribution, are in equilibrium with the yield shear of all the links.
3 A hardening ratio equal to the mean of the hardening ratios of the links.

In order to use the mentioned constant ductility response spectra, in the generic step of an iterative design process, the period of vibration of the equivalent SDOF system may be calculated by means of a modal analysis of the TBF or, more easily, according to the following simplified procedure.

The first period of vibration of the TBF may be assumed as characterized by eigenvector components ψ which are linearly increasing with height and calculated by means of the expression:

$$T^{(1)} = 2\pi \sqrt{\frac{M_\theta}{K_{\theta\theta}}} \tag{2}$$

being M_θ the first moment of the modal mass with respect to base; $K_{\theta\theta}$ the overturning moment corresponding to a unity value of the rigid rotation θ. At the aim of calculating the modal mass it is to be reminded that, owing to the assumed shape of the first mode of vibration of the TBF, the corresponding participation factor $g^{(1)}$ may be easily calculated as:

$$g^{(1)} = \frac{\sum m_i \psi_i}{\sum m_i \psi_i^2} = H \frac{\sum m_i h_i}{\sum m_i h_i^2} \quad (3)$$

The rotational stiffness $K_{\theta\theta}$, instead, may be derived by means of a standard static analysis or by a simple application of the virtual work principle. Indeed, once assigned an inverted triangular distribution of static forces F, an approximate distribution of shears in links may be easily evaluated which neglects the elastic deformations of the braced sub-structures. The rigid rotation θ consequent to such forces is reckoned as:

$$\theta = \frac{W^{\text{int}}}{\sum F_i \cdot h_i} \quad (4)$$

being W^{int} the internal virtual work. Hence, the rotational stiffness $K_{\theta\theta}$ may be calculated by dividing the overturning moment caused by the forces F by the rotation θ. In order to readily carry out the calculation of the rotational stiffness, the evaluation of the internal work may be usefully extended only to the elements of the structure which mostly contribute to the global internal work.

Finally, once defined the above-mentioned constant ductility spectra S_d^μ in terms of displacement and the ultimate top displacements u_u^{top}, the maximum first period of vibration T^* of the multistory system for which the required top displacement, evaluated for the desired peak ground acceleration a_g, equals the corresponding ultimate value may be obtained by means of the expression:

$$u_u^{top} = g^{(1)} \cdot S_d^\mu (a_g, T, \mu) \quad (5)$$

More in detail, from Eq. (4) the period of vibration T^* may be expressed as a function of the design peak ground acceleration a_g, of the maximum displacement demanded to the equivalent SDOF system (equal to $u_u^{top} / g^{(1)}$) and of the displacement ductility μ of the same SDOF system, or:

$$T^* = T^*\left(a_g \,;\, u_u^{SDOF} \,;\, \mu\right) \quad (6)$$

If the constant ductility spectra S_d^μ are practically independent of the ductility displacement for a wide range of values (as it is shown in the application), the required period of vibration T^* may be expressed as a function of a_g, and u_u^{SDOF} only.

In any case, the choice of the sections has to be verified with regard to the serviceability limit state.

3 APPLICATION

3.1 *The structural model*

The Authors examine a building (Ghersi et al., 2000) characterized by a squared plan (24 x 24 m) and by floor mass and interstory height (3.3 m) equal at all stories. The structural scheme of the system is defined by the intersection of eight plan frames (4 per each direction) having three spans each. Among such frames, the perimetral ones are devoted to the resistance of the seismic actions and endowed with tied eccentric braces in the central span. In order to test the design procedure in systems in which higher modes of vibration may influence the structural behavior, a twelve story TBF has been herein analyzed.

At the aim of nullifying the interaction between deck and links, two members are introduced at each level instead of the traditional single section. While the first resists the vertical loads transmitted by the deck, the second resists the horizontal actions and constitutes the link itself. Such technical choice also allows, both in the phase of design and in the evaluation of the response, a more reliable determination of the actions which apply to the link, otherwise affected by the imprecision due to the interaction between the deck and the links. In order to simplify the construction of the joints and to reduce the stress state at the lower story column, hinged connections have been considered between beams and columns, columns and foundation and at the ends of the braces. The deck has been considered as constituted by a corrugated sheet and by lightweight concrete, so as to limit the weight of the deck. For the same reason, all the non-structural elements have been considered to be made of rather light materials. On such a basis, a global weight of 5.0 kN/m² has been assumed as representative of the dead and live loads present in the building in occurrence of earthquakes.

3.2 *The constant ductility spectra*

The elastic response spectrum proposed by Eurocode 8 for soil class C (plotted in Fig. 2 in terms of displacement) has been considered representative of the expected seismic event.

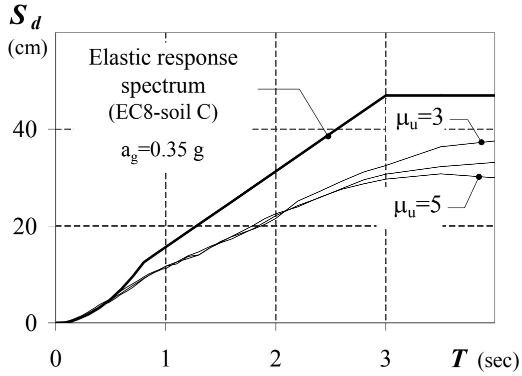

Figure 2. Elastic response spectrum and constant ductility response spectra in terms of displacement.

Hence, some constant ductility spectra have been calculated by using ten artificial accelerograms, generated by means of the program SIMQKE, compatible with the EC8 elastic response spectrum. The accelerograms are modeled by means of a trapezoidal envelope function individuated by a central part of 22.5 sec (stationary part of the accelerograms) and by initial and ending branches of 3 and 5 sec, respectively. Consistently with the provisions of Eurocode 8, no value of the mean response spectrum of the ten generated accelerograms is lower than 90% of the corresponding value proposed by EC8. Furthermore, the mean of the pseudo-accelerations in the constant pseudo-acceleration region of the spectrum is not lower than the value fixed by EC8.

Spectra have been then approximated by means of analytical expressions. Hence, owing to the similarity of the curves for a range of medium-high values of the displacement ductility (Fig.2), a unique linearized diagram of the inelastic displacements has been considered later for the design procedure.

3.3 *The design of links*

The geometrical length of the links, equal at all stories, has been fixed to 0.8 m. Short links having equal sections at all stories have been adopted, so as to obtain simplicity in the construction and very high dissipative capacity. According to the code definition of short links the section of such elements has been selected so as to fulfill the expression:

$$e \leq 1.6 \cdot \frac{M_P}{V_P} \tag{7}$$

being V_p and M_p the plastic shear and the full plastic bending moment of the cross-sections.
According to some codes (UBC 1997), the ultimate plastic rotation of links R_u has been assumed equal to 0.09 rad. The ultimate design value of the shear V_u, obtained in correspondence of the ultimate plastic rotation, is considered amplified with respect to the plastic value V_p to account for the steel hardening. On the basis of laboratory tests carried out by other researchers this amplifying factor has been considered equal to 1.5.

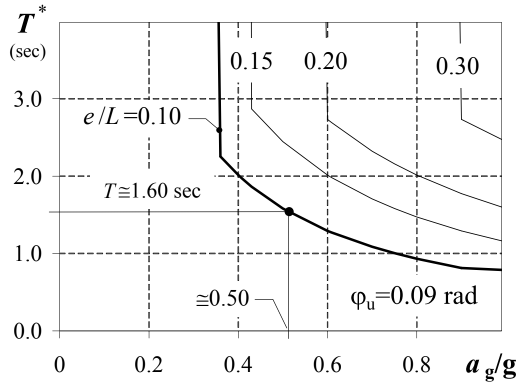

Figure 3. Maximum period of vibration as a function of the desired peak ground acceleration.

3.4 *The maximum period of vibration*

Once known the structural geometry and the mechanic properties of links, the value of the period of vibration T^* has been obtained by Eq. (4). As evident from Figure 3, where the period T^* is plotted as a function of a_g, if a peak ground acceleration equal to 0.50 g is desired at collapse the first period of vibration of the TBF must be, at most, equal to 1.60 sec.

3.5 *The design of braces*

According to the capacity design criterion columns, braces and ties have been designed so as to remain in the elastic field till the collapse of the structure (individuated by the attainment of the ultimate plastic rotation of a whatever link). Owing to the initial observations on the presumed behavior of tied braced frames, the design equivalent static forces corresponding to the collapse of the structure have been schematized by means of an inverted triangular distribution. Their intensity has been determined by taking into account that the maximum force transmitted by the generic link may be higher than the ultimate design value previously mentioned owing to the scattering of the values of the tensile strength of steel. In evaluating the intensity of such forces a second amplifying factor, equal to 1.2, has been considered. On such basis the intensity of the static equivalent forces corresponding to the collapse of the structure, to be used for the design of columns, braces and ties, may be expressed as:

$$F_i = \sum_{k=1}^{n} \left(1.2\, V_{u_k} L \right) \cdot \frac{h_i}{\sum_{k=1}^{n} h_k^2} \tag{8}$$

The value of the design internal actions of braces, corresponding to the collapse of the structure, has been obtained by means of equations of equilibrium in the horizontal direction of the part of the structure which is upper to the story under consideration. Such axial forces may be defined as:

$$N_i^b = \frac{\sum_{k=i}^{n} F_k}{2\cos\alpha} = \frac{0.6\, L}{\cos\alpha} \frac{\sum_{j=1}^{n} V_{u_j}}{\sum_{j=1}^{n} h_j^2} \sum_{k=i}^{n} h_k \tag{9}$$

being α the inclination of the brace with respect to a horizontal plane.
In order to account for the amplification of the story shear within the elastic-plastic response, an amplifying factor has been considered for the internal actions represented in Eq.(9), as recommended by EC 8 for the similar reinforced concrete coupled wall structures. The design internal actions corresponding to the yielding of all links (obtained by dividing Eq.(9) by 1.8) have been amplified by the factor:

$$q\sqrt{\left(\frac{1.2}{q}\right)^2+0.1\left(\frac{S_e(T_c)}{S_e(T)}\right)^2}\leq q \tag{10}$$

being q the behavior factor (assumed here equal to 5.5) and $S_e(T_c)$ the ordinate of the elastic response spectrum in correspondence of the period T_c which divides the constant acceleration region from the constant velocity region of the spectrum. The Authors have adopted the previous relation failing some other provision formally expressed so as to be used in a displacement-based design procedure (i.e. independently of the knowledge of the behavior factor).

3.6 *The design of ties*

The design value of the axial force of ties may be obtained by means of equations of equilibrium in the vertical direction of parts of the braced frame which involve links and braces from the top of the frame to the story under examination:

$$N_i^t=\sum_{j=i}^{n}1.2\,V_{uj}\left(1+\frac{e}{2l_b}\right)-\sum_{j=i}^{n}N_j^b\sin\alpha=$$
$$1.2\left[\sum_{j=i}^{n}V_{uj}\left(1+\frac{e}{2l_b}\right)-\frac{L\,tg\alpha}{2}\,\frac{\sum_{j=1}^{n}V_{u_j}}{\sum_{j=1}^{n}h_j^2}\sum_{j=i}^{n}\sum_{j=i}^{n}h_j\right] \tag{11}$$

As shown for braces, the internal actions of ties corresponding to the yielding of all links (obtained by dividing Eq.(11) by 1.8) have been amplified by the factor of Eq.(10).

3.7 *The design of columns*

The design value of the internal actions of columns has been evaluated by considering the vertical and horizontal forces present at each story as:

$$N_i^c=\sum_{j=i}^{n}\left[F_{qj}\mp V_j\frac{e}{2l_b}\right]\pm\sum_{j=i+1}^{n}N_j^b \tag{12}$$

being F_{qj} (equal to 160 kN at each story) the vertical forces transmitted by the deck to the columns.

3.8 *Sections*

Sections of columns, braces and ties (Table 1) are kept constant for two stories each (obviously there is no tie at the first story). In particular, section T1 of the columns of the two lower stories is constituted by a HEM500 (Fe510) section and by two IPE200 (Fe510) orthogonally disposed with respect to the HEM500 and welded to its web. Sections of links are characterized by plastic shear of 449.0 kN and by full plastic bending moment of 446.2 kNm.

Table 1. Sections of members

Storey	Links	Columns	Braces	Ties
12	HEA360	HEA180	HEB300	HEB300
11	HEA360	HEA180	HEB300	HEB300
10	HEA360	HEB240	HEB320	HEB340*
9	HEA360	HEB240	HEB320	HEB340*
8	HEA360	HEB320*	HEB320*	HEB360*
7	HEA360	HEB320*	HEB320*	HEB360*
6	HEA360	HEB450**	HEB320**	HEB320*
5	HEA360	HEB450**	HEB320**	HEB320*
4	HEA360	HEM450**	HEB360**	HEB280
3	HEA360	HEM450**	HEB360**	HEB280
2	HEA360	T1	HEB360**	HEB240
1	HEA360	T1	HEB360**	--------

* Fe430 ** Fe510 other sections Fe360

3.9 *The dynamic properties of the TBF*

The fundamental period of vibration of the TBF, calculated according to Eq.(2), is equal to 1.59 sec. The participation factor (Eq. (5)) is instead equal to 1.44.

4 NUMERICAL ANALYSES

4.1 *Modeling*

The structure has been analyzed by means of the program DRAIN-2DX. The shear hardening ratio of links has been fixed so as to reach a shear equal to 1.5 V_p in correspondence of the ultimate plastic rotation (0.09 rad). The flexural hardening ratio, instead, has been assumed so that a bending moment equal to 1.5 M_p is reached for a flexural plastic rotation of 0.03 rad. For the columns a value of the flexural hardening ratio equal to 0.03 has been considered independently of the section. This aspect has not been investigated further because yielding of columns has been avoided till the collapse of the structure. Braces and ties are supposed to have infinite strength so that a comparison between the maximum values of the internal actions required by dynamic analyses at collapse and the values predicted by design may be carried out.

4.2 *Results*

The first period of vibration of the TBF and the participation factor are in very good agreement with the results of Eqs. (2)-(5), even though the eigenvector shape is not perfectly linear. The peak ground accelerations corresponding to the first yielding of the structure a_{gy} and those related to the collapse a_{gu} are reported in Table 2. In the same table is also present the behavior factor calculated for each accelerogram as the ratio of a_{gu} to a_{gy}. As evident, the peak ground acceleration predicted in the phase of design for the structural collapse (0.50 g) is lower than the real value (≅0.6 g).

Table 2. Peak ground accelerations corresponding to the first yielding and to the collapse of the structure (1st and 2nd column). Behavior factor of the structure (3rd column).

Acc.	a_{gy}	a_{gu}	Qfactor
	(g)	(g)	
1	0.089	0.466	5.268
2	0.089	0.717	8.026
3	0.096	0.510	5.283
4	0.091	0.5701	6.287
5	0.093	0.689	7.370
6	0.094	0.636	6.781
7	0.086	0.667	7.740
8	0.092	0.638	6.913
9	0.094	0.509	5.394
10	0.093	0.644	6.915
mean	0.092	0.606	6.598

Such difference is probably caused by the effect of the higher modes of vibration. Indeed, owing to the amplification of the base shear with respect to the value related to a unique mode of vibration characterized by equivalent static forces linearly increasing along the height of the frame, the hardening ratio of the equivalent SDOF system would have to be increased (at least in some ranges of the horizontal displacement) if exact results are desired.

The large values of the behavior factor anyway highlight the good seismic behavior of the TBF. The behavior factor (6.6) is much higher than that obtained in the past with reference to analogous eccentrically braced systems (in any case less than five).

The maximum plastic rotations corresponding to the collapse are quite uniform within the structure (Fig.4). The lower values of the upper storeys are due to the elastic deformations of the braced substructures. Indeed, owing to the axial forces of the members, the relative vertical displacements of the ends of the links are somewhat different from those depicted on the basis of the rigid movement of the braced sub-structures.

Furthermore, the displacements corresponding to the collapse of the structure (Fig. 5) show that their distribution may be with good approximation represented by a linear function increasing along the height (assumption of the phase of design). The mean of the maximum demanded top story displacements (0.41 m in mean) is certainly in some agreement with the ultimate value calculated in the phase of design. Indeed, while the ultimate value corresponding to the rigid behavior of the braced sub-structures is equal to 0.36 m, that one which also derives from the consideration of the elastic deformation of the members is equal to 0.39 m.

Finally, as expected, the maximum internal actions which develop in braces (and thus the story shears) are greater than those predicted with reference to the collapse configuration but smaller than those adopted in the phase of design.

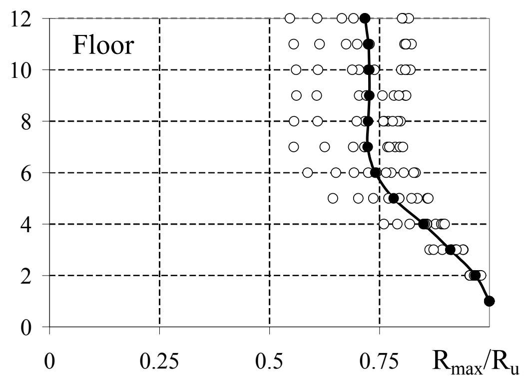

Figure 4. Ratios of maximum plastic rotations to ultimate plastic rotations.

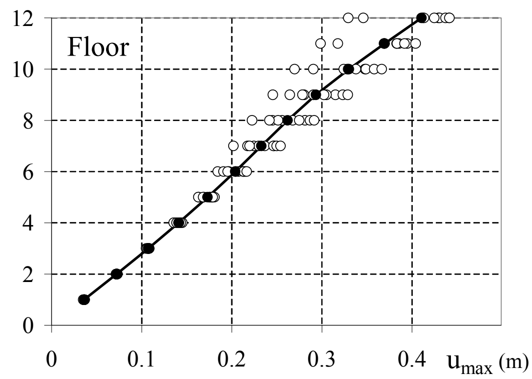

Figure 5. Maximum displacements in correspondence of the peak ground acceleration of collapse.

The maximum internal actions of both columns and ties are instead in better agreement with those adopted in the phase of design.

CONCLUSIONS

The assumptions of the presented design procedure find good agreement in the results of the numerical analyses. Nevertheless, some little differences remain in the prediction of the peak ground acceleration corresponding to the collapse, probably due to the effects of the higher modes of vibration which, in the structure under examination, are duly important. The response of the structure seems to be quite optimal, as required in the design.

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