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Seismic performance evaluation for steel MRF: non linear dynamic and static analyses

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Abstract. The performance of steel MRF with rigid connections, proportioned by adopting different capacity design criteria, is evaluated in order to highlight the effectiveness of static non-linear procedure in predicting the structural seismic behavior. In the framework of the performance-based design, some considerations are made on the basis of the results obtained by both dynamic time histories and push-over analyses, particularly with reference to the damage level and the structure ability to withstand a strong earthquake.

Key words: steel frame; seismic behaviour; overstrength; performance-based design; push-over analysis; time history analysis.

1. Introduction

During recent earthquakes unexpected serious damages to many buildings, even if designed according to seismic codes, have driven researchers to a conceptual revision of the methodologies currently adopted in practice. The criteria used to date in seismic design of structures, particularly devoted to ensure the no-collapse requirement, are no longer considered fully satisfactory, due to their inadequacy in predicting and limiting damage levels. Since 1995 the so-called performance based design (SEAOC 1995) is considered the most appropriate way to face the problem of controlling structure performance with reference to different levels of seismic hazard (Bertero 1997, Hamburger 1997). While this new approach is very attractive from a conceptual point of view, its practical application is not yet well defined and many researchers are now working to develop simple and reliable methodologies to be incorporated in the new codes (Gobarah *et al*. 1997). The prediction of structural performance related to a specified level of ground motion, in fact, requires the evaluation of displacements and force distributions also beyond the yield limit, which cannot be derived from simplified elastic calculations (Priestley 2000).

Inelastic dynamic time history analysis seems to be the most effective tool for evaluating the structural behaviour, but it is too cumbersome and time consuming. Furthermore this methodology is closely connected to the earthquake characteristics, and the use of a large ensemble of accelerograms is

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necessary to obtain meaningful results. Static push-over analysis, on the other hand, is considered a suitable procedure for predicting the non-linear behaviour of the structure in a more accessible way (Tso & Moghadam 1998).

In this paper, the performance of moment resisting steel frames with rigid connections, designed according to different criteria, has been evaluated in order to judge: the effectiveness of the non-linear static analysis if compared to the dynamic time history analysis, the influence of the overstrength given by different capacity criteria and the effectiveness of a displacement oriented approach.

2. MRF performance evaluation

Force-based design is nowadays the methodology used in proportioning seismic structures and required by present codes. Conventional horizontal forces are obtained by reducing elastic forces by means of a factor (R_w, q-factor and so on), accounting for the plastic deformation experienced during the ground motion. The resistance of the system under this load is then verified.

Obviously the necessary ductility has to be assured and, with this aim, the most recent codes suggest applying suitable methodologies usually called capacity design criteria. Whatever be the adopted criterion, it can be said that the whole design procedure is really effective only if the structure is able to withstand severe earthquakes without collapse.

The dynamic behaviour of MRFs, designed according to various criteria, has been already analysed by the authors (Calderoni *et al*. 1996, 1997b). The obtained results have pointed out that a judgement of the bearing capacity of structures, designed according to European seismic code - EC8 (Eurocode 8 1994), cannot be independent of the way the design has been really developed, with particular reference to the amount and distribution of the design overstrength along the height of the frame. It was highlighted that the capacity criterion, as proposed by EC8, sometimes can give also unsatisfactory results, while it can be more effective and simple in many cases to provide the column with a given overstrength, “correctly” distributed.

Frames designed in order to exhibit an ultimate global mechanism (Mazzolani and Piluso 1995), were also analysed (Calderoni *et al*. 1995): improvement of the structural inelastic performance has been shown, facing a significant overstrength given to the columns and the corresponding increase of structural weight.

Finally the conclusion was that the force-based design gives satisfactory results with reference to the collapse behaviour, when a suitable capacity criterion is applied, even if some uncertainties are still related to the force-reduction factor. The present criticism of this procedure is its inadequacy in predicting and limiting damage levels, particularly for the action of earthquakes having a larger probability of occurrence.

The displacement-based design (which falls within the framework of the performance-based design), on the contrary, is not oriented to ensure an assigned strength for the structures, but to control the damage level related to displacements expected to occur for different ground motion intensities. In practise structural performance is evaluated in terms of deformation rather than of resistance.

The application of the displacement-based design, however, requires the use of inelastic static analyses. Therefore the behaviour of some MRFs has been evaluated by means of both static push-over and dynamic procedures, and the corresponding results are compared and discussed also with reference to the adopted design criteria.

3. Analysed frames and design criteria

Four different steel MRFs (named 13-14-17-18) have been analysed. Their geometrical and load schemes are reported in Fig. 1, where the floor weight (W), constant at each storey, is also indicated. It must be noted that the mass has been assigned in such a way that all the frames exhibit the same first period, equal to 1.5 s, despite differences in the element cross-sections. Two levels of vertical loads have been considered for the beams: a lower one in the frames 13 and 17 and a higher one in the other two (frames 14 and 18).

All the frames have been designed according to EC8 requirements, for a seismic zone characterised by PGA = 0.35 g and soil type A. Multi-modal response spectrum analysis has been performed, by adopting a reduction factor $q = 5$. P-Δ effects have been also accounted for in the approximate way allowed by the code. The inter-story drift limitation, stated by the code, has deliberately not been accounted for.

Furthermore three different design criteria (named B, C and Z) (Calderoni *et al.* 1997a) have been considered in proportioning the frame elements, and so twelve different structures have been analysed.

The first adopted design criterion (B) consists in not applying any capacity criterion, i.e. in giving the frame no design overstrength. This means that the section resistance is exactly equal to the design internal forces in all structural elements. The second one (C) is the capacity criterion required by EC8, which states that the ultimate strength of the columns must be always higher than the beam one. The last design criterion (Z) gives the columns a prefixed overstrength, with a linear distribution along the height of the frame, and so is independent of the beam strength (Fig. 2). In greater detail, increases of column strength (compared to that necessary to withstand the design external loads) equal to 25% at the

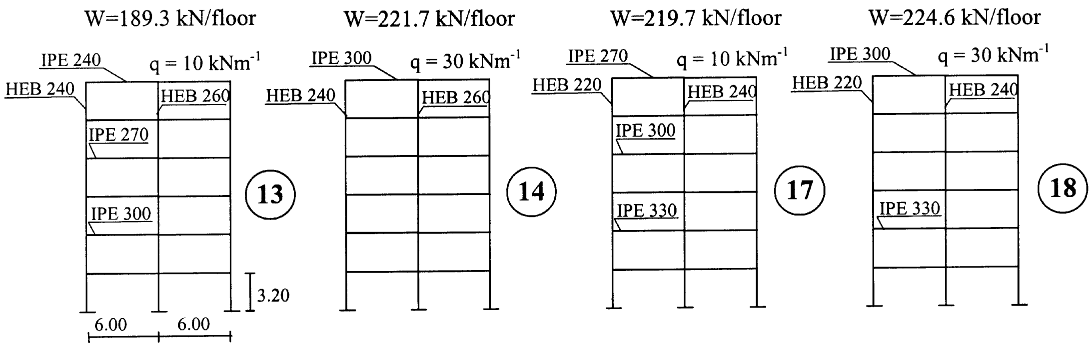

Fig. 1 Analysed frames - Geometrical and load schemes

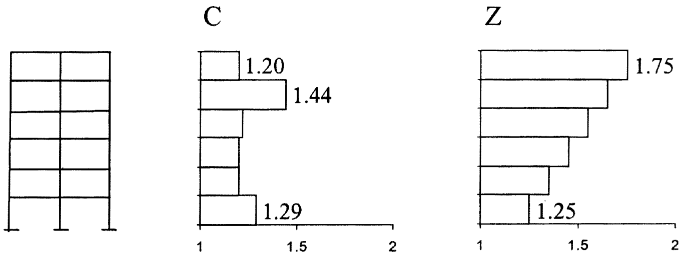

Fig. 2 Frame 13 - Overstrength distributions

first floor and to 75% at the top floor are given, with a mean value of 50% for the whole frame. Note that this overstrength distribution is consistent with the column failure pattern, usually resulting from dynamic analyses of frames designed without any overstrength (Calderoni *et al.* 1995).

The definition of design overstrength has to be clarified. In this paper this word refers to a section and points out its capacity to bear internal actions greater than those caused by the design load. In particular, it is simply evaluated as the ratio of the element cross-section bending moment resistance above the maximum bending moment due to the design vertical plus (or minus) horizontal loads.

It must be pointed out that the necessary resistance has been assigned to the frame conventionally, by giving to each section a proper (and different) value of the yielding stress, without changing the prefixed cross-section dimensions. In this way the structure is provided with the exact overstrength required by the design criterion used, and contemporarily no variation affects the fundamental period.

4. Dynamic analyses

The evaluation of seismic performance of the above described twelve frames was carried out by means of a non-linear dynamic time history analysis. As dynamic behaviour is strongly affected by ground motion, an ensemble of thirty actual records, selected from a large number of Italian earthquakes (Fig. 3), was adopted as seismic input. The seismic records were suitably scaled in order to improve their homogeneity and to obtain an average spectrum similar to the EC8 elastic one (Fig. 4), used in the design phase, with a PGA=0.35 g (Calderoni *et al.* 1996, Rinaldi 1997).

The dynamic response of the frames subjected to the whole set of accelerograms was evaluated by means of the Drain2DX code (Prakash *et al.* 1993). The obtained dynamic quantities (displacement, rotation etc), when statistically interpreted, refer to the mean values of the thirty results (corresponding to the thirty ground motions), as the design spectrum is analogous to the mean spectrum of the selected records. The performance of the frames was then judged in terms of both collapse and damage, as described below.

Furthermore incremental analyses were performed, for PGA values ranging from 0.1g to 0.8 g.

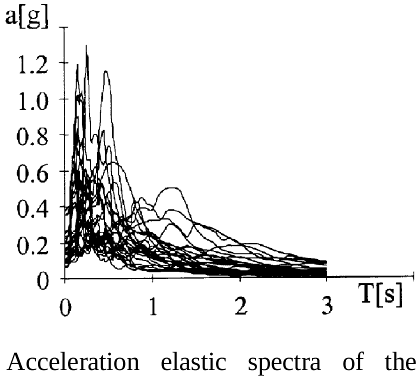

Fig. 3 Acceleration elastic spectra of the thirty selected earthquakes

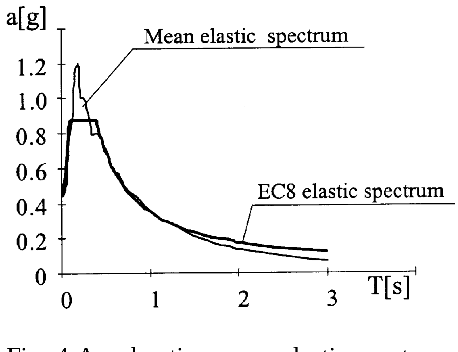

Fig. 4 Acceleration mean elastic spectrum

4.1. Collapse analysis

The dynamic behaviour of the frames was firstly evaluated in terms of ultimate resistance. The percentage of earthquakes, under which the structure reaches the limit fixed by a proper collapse criterion at least in one section, is assumed as failure index.

Since the dynamic frame performance is defined by different response parameters, the choice of that related to the collapse is an initial problem. Once the parameter is chosen, the definition of its limit value is a further problem. A number of proposals on this matter can be found in literature. In this paper three different collapse criteria, frequently used, have been considered, which refer to the attainment, respectively, of:

- the maximum plastic rotation;
- the maximum accumulated plastic rotation;
- the maximum inter-story drift ratio.

According to many research results, failure is deemed to occur when the following values of the selected parameters are reached:

- 4% for beams and 2.5% for columns as regards the plastic rotation;
- 10% for beams and 5% for columns as regards the accumulated plastic rotation (Akbas & Shen 1998);
- 1.5% for the inter-story drift ratio (life-safe limit in SEAOC 1995).

In Fig. 5 the failure index, evaluated on the whole frame, is plotted with reference to the three adopted design criteria for each analysed scheme.

When maximum plastic rotation is used as collapse parameter (Fig. 5a), all the frames seem to behave well, as the percentage of failures is always very low, quite independently of the adopted design criteria. On the contrary, if we refer to the maximum accumulated plastic rotation (Fig. 5b), significant differences appear: in this case the adoption of a capacity design criterion (C and Z) proves to be very effective in reducing the failure probability. In fact accumulated plastic rotation is related also to cyclic behaviour and energy absorption, which are necessarily influenced by the adopted design criteria. It is evident that in the cases C and Z the columns are less engaged in cumulative plastic deformation, due to the overstrength given to them as compared to the beams. Furthermore, as already pointed out, the Z criterion, which is very simple to apply, is equivalent or even safer than the capacity criterion prescribed by EC8 (C criterion).

It is worth noting that collapse was reached at the top section in the columns of the top floor in a large

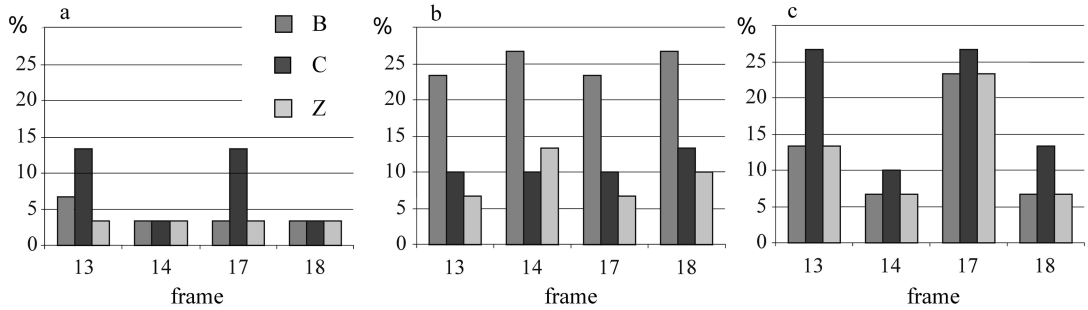

Fig. 5 Failure index based on maximum plastic rotation (a), maximum accumulated plastic rotation (b) and interstorey drift (c)

number of cases, only when this last criterion was adopted. This is probably due to the fact that the capacity design has not been applied “at the top floor of multi-storey buildings”, as explicitly stated by Eurocode 8, while overstrength has been given to the bottom section of the same columns. Then the failure index has been evaluated without including these failures, considering them as a local problem not significantly affecting the global behaviour of the structure.

As far as the maximum inter-story drift is concerned (Fig. 5c), the corresponding failure index has proved to be higher if compared with the first collapse criterion, but the pattern is similar. This result was expected since the inter-story drift ratio is practically equal to the plastic rotation in the columns (if elastic deformation is ignored). Collapse percentage values are higher, in this last case, due to the lower limit value adopted for the inter-story drift ratio (1.5% instead of 2.5% for plastic rotation).

4.2. Damage analysis

The damage level of the frames has been evaluated by means of a damage index (DI), defined as the ratio of the maximum accumulated plastic rotation attained during the earthquake above the limit value of it. The index DI consequently varies in the range 0÷1: the value 0 means absence of any yielding, while the value 1 corresponds to the collapse of the section.

Mean values of DI for all the examined cases, obtained from the thirty earthquakes, are plotted separately for beams, columns and whole frames in Fig. 6. It can be noted that the damage levels in cases C and Z are higher for beams and lower for columns, if compared to the B frames. Moreover, the DI for the whole frame is almost always lower in cases C and Z.

Similar results (not reported) have been obtained with reference to the maximum plastic rotation.

The adoption of any capacity design criterion is confirmed to be effective in reducing spread damages in the structure, shifting the plastic engagement from columns to beams.

It can be noticed that a low level of vertical loads on the beams (frame 13 and 17) reduces the effectiveness of the EC8 design criteria (C), both for failures and for damage, provided that the column strength is dependent on that of the beams. On the contrary the effectiveness of the Z criterion in reducing damage and failures seems to be independent of the vertical load level.

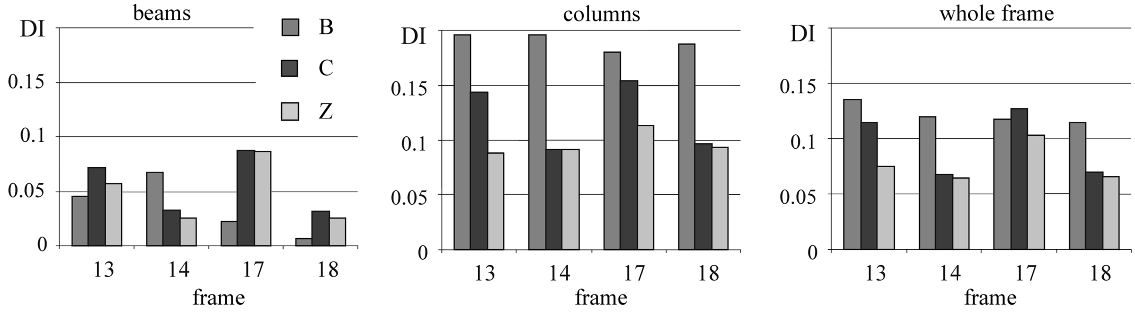

Fig. 6 Damage index: mean values for beams, columns and whole frames

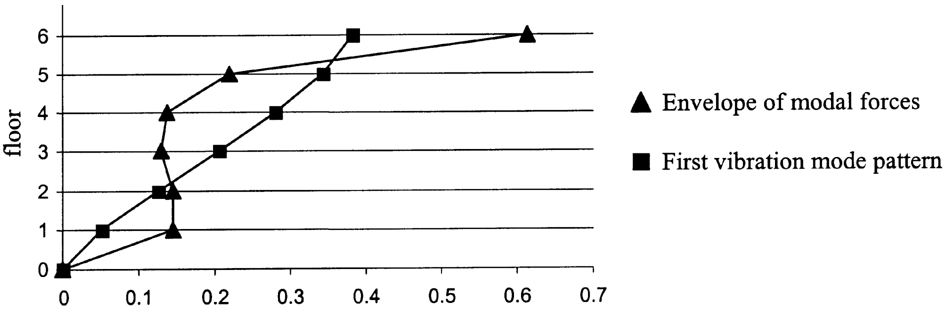

Fig. 7 Lateral load distributions along the frame height

5. Static non linear analyses

The behaviour of the above considered frames has been studied also by means of static inelastic procedure. Push-over analysis has been performed on the structures by increasing a fixed pattern of lateral forces. Since different criteria have been proposed in order to define the proper load pattern to perform an effective push-over analysis, in this paper two different lateral load distributions along the frame height are considered. The shape of the first one corresponds to the first vibration mode, while the second one is similar to the envelope of modal forces. Although these two patterns are quite different (Fig. 7), the obtained results do not differ significantly, and thus the reported results will be related only to the second adopted distribution.

The capacity curves (normalised base shear V_b / W versus roof drift angle δ_{top} / H) for all the frames and the considered design criteria are given in Fig. 8.

Three horizontal lines are plotted in each diagram, which are related to significant strength levels of the frames. The lower one highlights the design base shear (V_{DB}), obtained by means of the multi-modal analysis performed in the design phase according to EC8 requirements.

Since no overstrength was given to the beams, the first plastic hinge, for all the frames, should be developed at the attainment of this shear base value, as exhibited by frames 13 and 17, independently of the adopted design criterion, which gives overstrength only to the columns. Small shiftings are possible, due to the approximate way the P-Δ effects have been evaluated in the design phase.

However, frames 14 and 18, which bear more considerable vertical loads, experience the first yielding for a higher base shear, when criteria C and Z are applied. For these frames the beam design is governed by factorised vertical load condition rather than by seismic one, thus the beams exhibit some overstrength compared to seismic strength demand.

Then, if no overstrength is given to the columns (case B), the first yielding at V_{DB} occurs in these elements, while the elastic limit is clearly shifted up when a capacity design criterion is considered.

The higher horizontal line plotted in the diagrams points out the frame ultimate shear resistance (V_U), defined as:

$$V_U = \sum_{c=1}^{n} \left| \frac{Mu_b + Mu_t}{h} \right| \tag{1}$$

where Mu_b and Mu_t are the ultimate bending moments at the end sections of the column, h is the

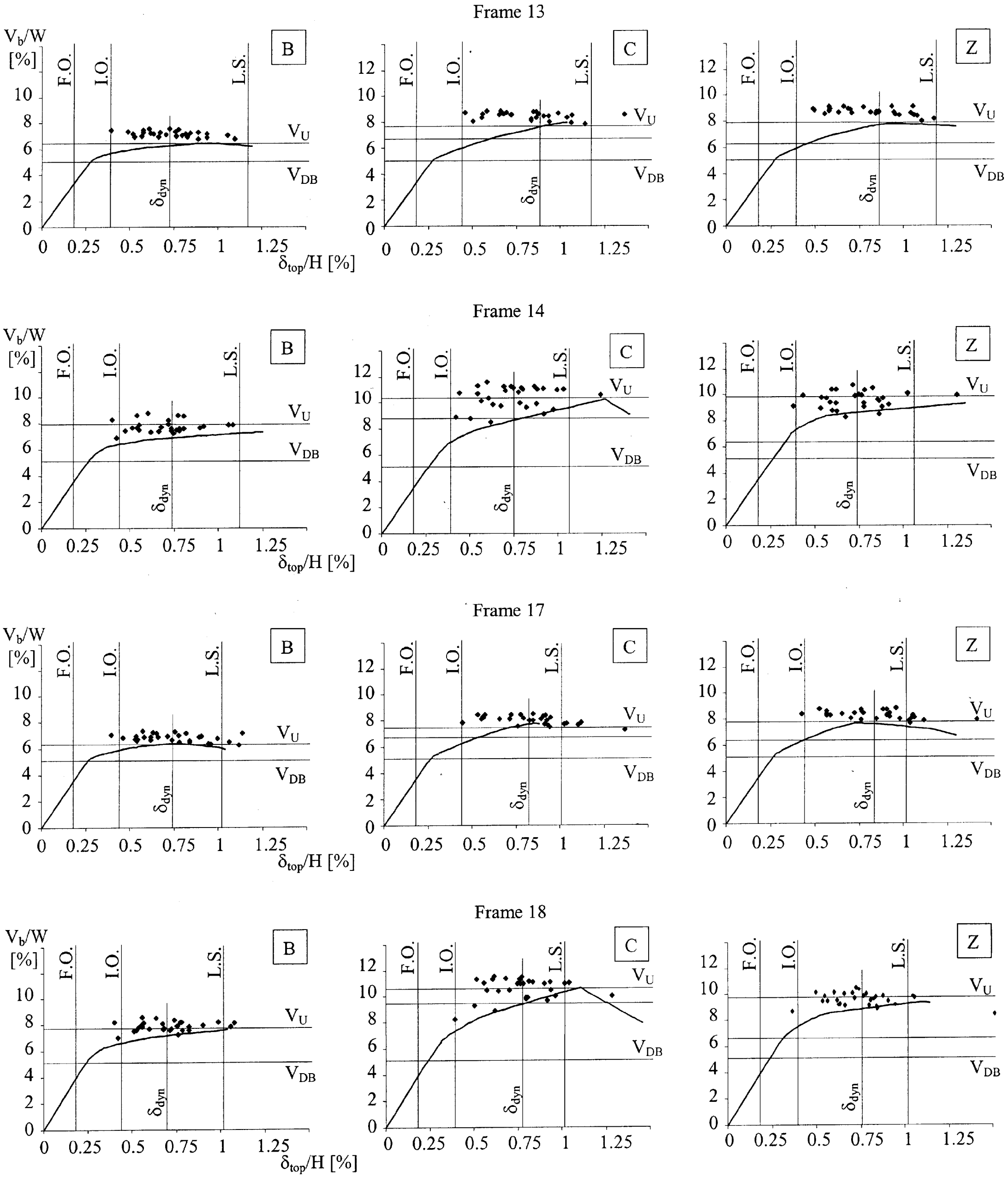

Fig. 8 Capacity curves of the frames for the adopted design criteria

inter-story height and n is the number of the columns in the floor.

In practise V_U is the ultimate shear strength provided by the first floor at mechanism development and it should coincide with the design base shear V_{DB} when no overstrength is given (case B). However an additional strength compared to the design global base shear is due to gravity load presence, and it cannot be removed. In fact, while the bending moments for horizontal forces change in sign when inverting the action, those for vertical loads are fixed, so forcing the columns to be oversized, when symmetrical cross-sections are adopted, as usual. This means that a frame exhibits an additional amount of horizontal shear strength simply because it bears gravity loads, even if the global base shear due to vertical action is null.

The push-over curves show that, in all the cases, the frames exhibit a maximum strength practically equal to the theoretical one (V_U).

The intermediate horizontal line indicates the increase in shear strength compared to V_{DB} correlated to the applied capacity design criterion (C, Z). Obviously, when no overstrength is given (B), this line is coincident with the lower one (V_{DB}). Note that, while the increase of resistance in the Z case is almost constant whatever be the frame, being fixed a priori, in case C this increase is influenced by the beams strength, i.e. by the vertical loads level, so that the resistance increase is higher for frames 14 and 18 as compared to frames 13 and 17.

On each curve the three vertical lines indicate the roof displacements related to the attainment of the maximum inter-story drifts ratio (δ/h) in the frame, which define the performance levels stated by SEAOC, as follows:

- Fully Operational (F.O.) - $\delta / h = 0.2\%$
- Immediate Occupancy (I.O.) - $\delta / h = 0.5\%$
- Life Safe (L.S.) - $\delta / h = 1.5\%$

In Fig. 9 plastic hinge patterns are depicted referring to this last limit state, for only two frames but for all the adopted criteria. It can be noted that no mechanism has yet been reached, even though the first floor failure is close to be attained. Yielding is widespread both in columns and beams, but not according to a global mechanism.

On the basis of the push-over results, it can be observed that, contrary to the expectation, no significant differences among the behaviour of frames emerged. In fact, despite the various adopted design criteria, which provide the structures with different resistance levels, the limit inter-story drifts, related to the above said performance levels, are reached in all the cases almost for the same roof displacements. Furthermore, even the maximum values of plastic rotation in the elements (not reported in this paper for sake of brevity) are quite similar and only slight differences have been shown in plastic engagement distribution.

However it must be borne in mind that the analysed frames have been designed without any additional design resistance, contrary to most cases reported in literature, which almost always refer to real frames. The unavoidable overstrength of these last ones can, in fact, influence in a random way the structural behaviour and can be misleading in the interpretation of the results.

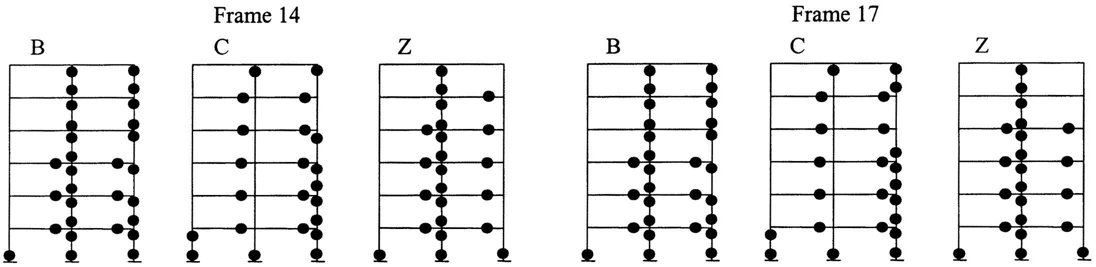

Fig. 9 Plastic hinge patterns at Life-Safe limit

6. Comparison between static and dynamic behaviour

The results of the dynamic non-linear analyses are also reported on the push-over diagrams. The solid square dots represent the couples V_{bmax} - δ_{topmax} obtained from the time histories performed for each of the thirty used ground motions. The few cases in which dynamic instability has occurred have been not considered. A vertical solid line points out the mean dynamic roof displacement (δ_{dyn}).

A good agreement emerges between the dynamic dots and the static curves, although the push-over results are always conservative, i.e. the static procedure tends to underestimate the base shear capacity at any particular roof drift angle.

As predictable, the dynamic results are grouped close to the mechanism line (V_U), proving the development of several plastic hinges in the columns during the quakes. A slightly higher scatter is shown when vertical loads are considerable and govern the beam design (frame 14 and 18). In these cases a number of dots can be found also below the mechanism line, although they almost always lay over the push-over curves.

In Fig. 10 the dashed line outlines the dynamic deformed shape of the frames 14 and 17. For each floor the mean values of the maximum floor displacements obtained from the thirty dynamic analyses are reported. The solid line, instead, represents the distribution of floor displacements from push-over static analyses at the roof deflection corresponding to the above defined dynamic one.

A satisfactory agreement between static and dynamic displacement shape can be noted. The slight

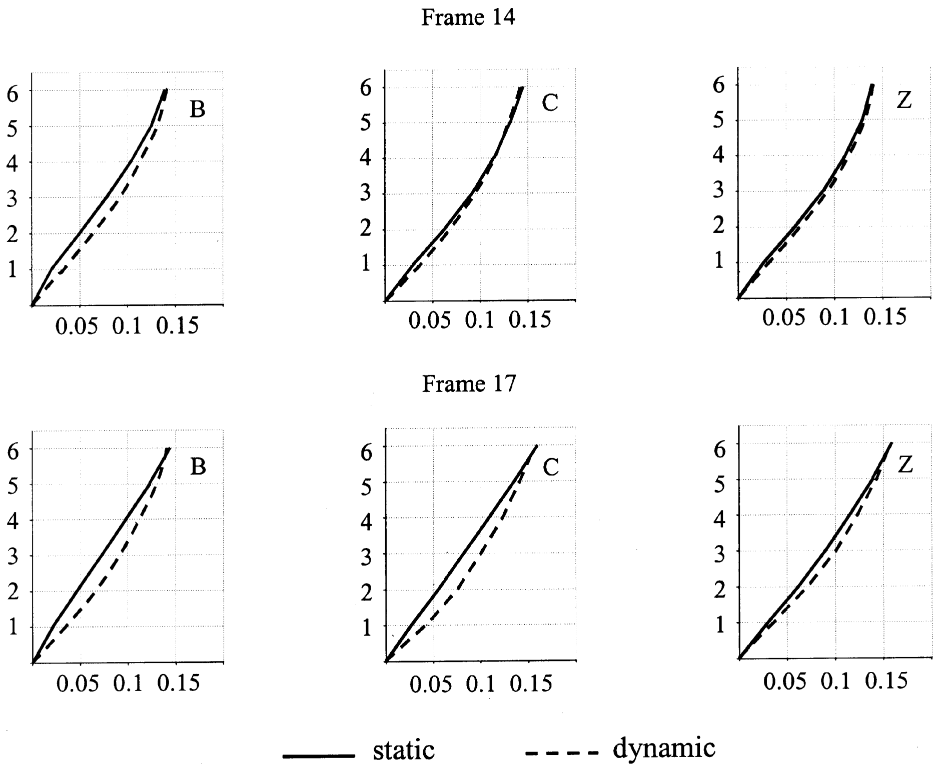

Fig. 10 Static and dynamic floor displacements [m]

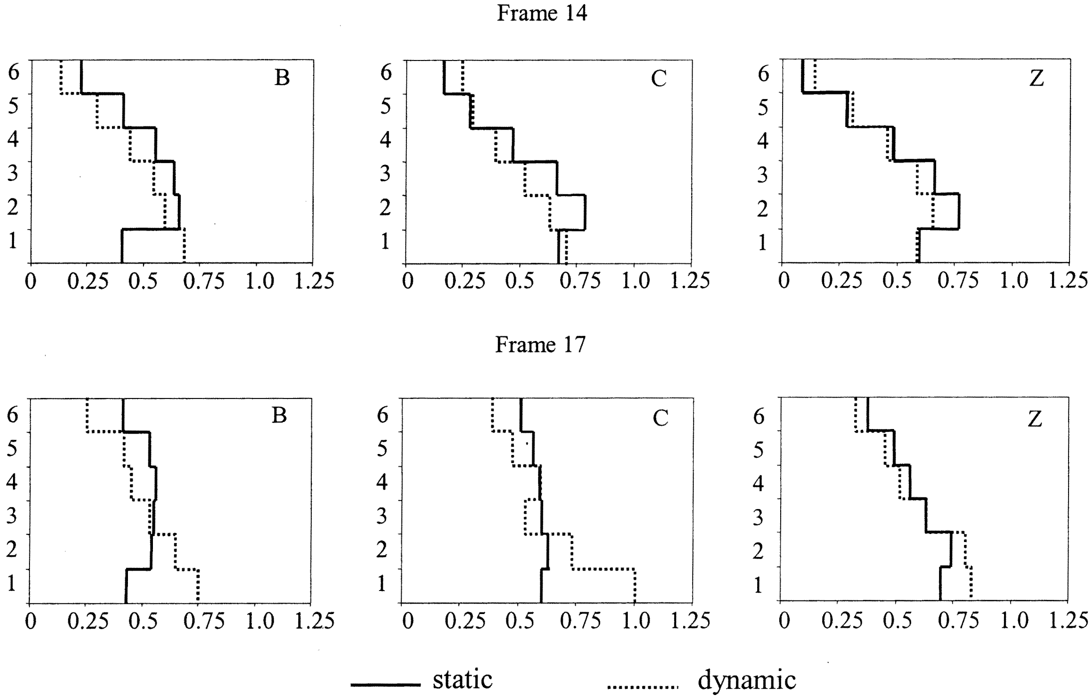

Fig. 11 Static and dynamic inter-story drift ratios [%]

scatters are due particularly to the different behaviour at the first floor. Anyway the figure confirms that the push-over method can provide lateral deflections similar to the ones obtained from inelastic dynamic analyses, and that the horizontal forces pattern used in performing the static analyses is quite suitable, at least for frames of such height.

Similar consideration can be made also with reference to the inter-story drift ratios. In Fig. 11 the solid line refers to the values obtained from the push-over analyses, at the top displacement already reported in Fig. 10, while the dashed line refers to the mean values of the maximum dynamic drifts.

Again, significant scatters are shown particularly at the first floor, where the dynamic results are almost always higher, highlighting a larger plastic engagement of columns.

Anyway, for both floor displacement and inter-story drift ratio, no meaningful differences of behaviour have been pointed out among the frames, whatever be the adopted design criterion (B, C, Z).

It is also of interest to discuss about results obtained from the dynamic analyses performed by increasing the PGA from 0.1g to 0.8 g. In Fig. 12 the relationship between the mean value of the roof drift angle and the increasing PGA is plotted with reference to frame 13 C. This incremental dynamic analysis (IDA) can be considered as a sort of dynamic push-over.

In Fig. 13 the zone closer to the origin of axes is enlarged. The three vertical solid lines drawn in this figure indicate the roof displacements (from the static push-over analysis) related to the attainment of the maximum inter-story drift ratios (δ / h), which define the performance levels stated by SEAOC. The horizontal solid lines, instead, highlight the PGA values corresponding to different levels of ground motion.

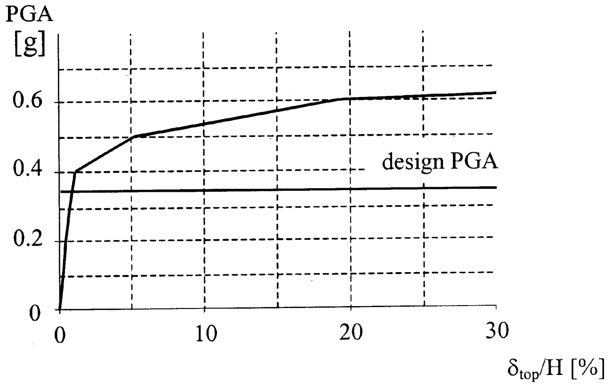

Fig. 12 Frame 13 C - Incremental dynamic curve (IDA)

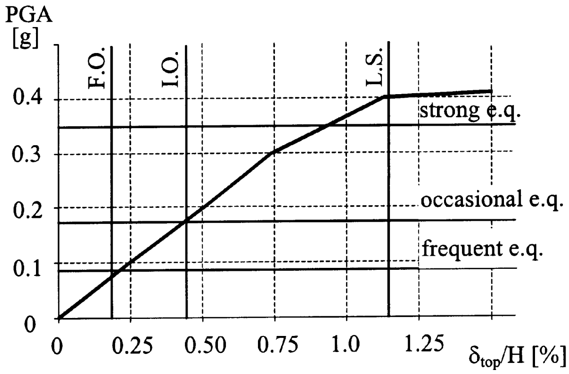

Fig. 13 Frame 13 C - Incremental dynamic curve (detail)

The higher line (PGA = 0.35 g) represents the design "strong" earthquake, for which no failure has to occur in the structure (no-collapse requirement in EC8 or "life-safe" in SEAOC).

The intermediate line (PGA = 0.175 g) represents the serviceability earthquake (the former reduced by two), for which the maximum inter-story drift should be not greater than 0.4 ÷ 0.6%, as stated in EC8. This no-damage requirement is equivalent to the "operational" SEAOC performance level related to an "occasional" seismic event.

The lower line (PGA = 0.0875 g) represents, in authors opinion, the "frequent" earthquake, for which, according to SEAOC, the structure must exhibit, at the most, negligible damages ("fully operational") with maximum inter-story drift ratio not greater then 0.2%.

The figures show that the limit value of inter-story drift ratio (1.5%), corresponding with the "life-safe" performance level, is not exceeded at the design PGA (0.35 g), while the maximum ground acceleration bearable by the frame is about 0.60 g, higher than the design value. The no-damage requirement is well fulfilled too, while the "fully operational" performance level is not reached, but by a small margin.

Finally the global behaviour of the frame can be considered quite satisfactory, if judged by using the dynamic results, both for serviceability and ultimate limit states.

If we refer to the static analysis, the value of the normalised base shear (V_b / W), corresponding to the

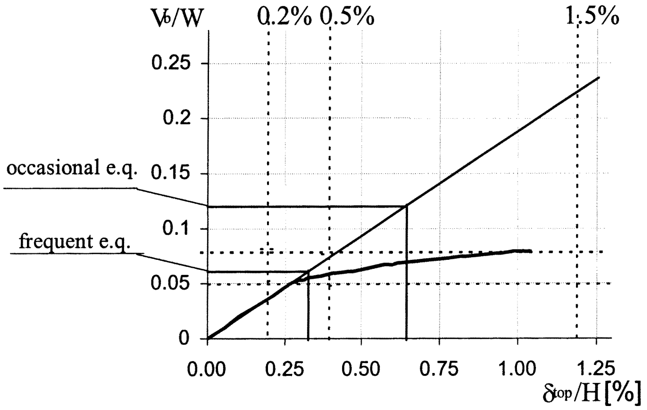

Fig. 14 Frame 13 C - Capacity curve

reduced (by means of q = 5) design forces for the "strong" earthquake, is about 5%, while the elastic (not reduced) one is about 25%. Consequently, the elastic values of V_b/W, corresponding to the "occasional" and "frequent" earthquakes, are about 12% and 6% respectively (i.e., ½ and ¼ of the elastic forces due to the "strong earthquake"). From Fig. 14, in which the push-over curve for frame 13C is plotted again, it can be easily seen that the serviceability requirements are not met.

As regards the evaluation of frame performance under the "strong" earthquake (no-collapse requirement) according to the displacement-based design, i.e. when adopting a push-over analysis, it is necessary to refer to a target displacement.

This displacement could be evaluated by performing a number of time history dynamic analyses, as made in this work; but, in this case, the static push-over would be useless.

At present some simplified criteria, suggested in order to make this kind of design procedure feasible in the practice (Faijfar & Gaspersic 1996, Chopra & Goel 1999), can be used.

If we refer in particular to the SDOF equivalence approach, the target displacement is obtained from the elastic displacement spectrum, considering the first period of the frame increased in order to account for the plastic engagement of the structural elements. According to Mendis & Chandler (1998) and setting the ductility index equal to the adopted q-factor, the shifted period of all examined frames becomes 3 s and, with reference to the mean displacement spectrum of the thirty adopted earthquakes (Fig. 15), the corresponding target displacement is 15 cm.

Note that this value is almost the same of the mean top displacements obtained for the frames from the dynamic analyses and displayed above, as roof drift angle δ_{top}/H, in Fig. 8, so highlighting a sufficient reliability of the adopted method, at least for frames of the analysed typology.

The target displacement thus evaluated is always lower than that corresponding to 1.5% inter-story drift ratio, as can be seen in the push-over curves (Fig. 8). This means that, whatever be the adopted design criteria (B, C, Z), all the analysed frames appear to be able to withstand the design "strong" earthquake without exceeding the SEAOC life-safe limit, as already found from the dynamic results.

On the basis of these remarks, static and dynamic procedures might be considered as perfectly equivalent in giving information about the behaviour of the frames at the ultimate seismic limit state. Nevertheless, some uncertainties can arise if we consider that no significant differences have been shown by the push-over analyses developed for the three different adopted design criteria.

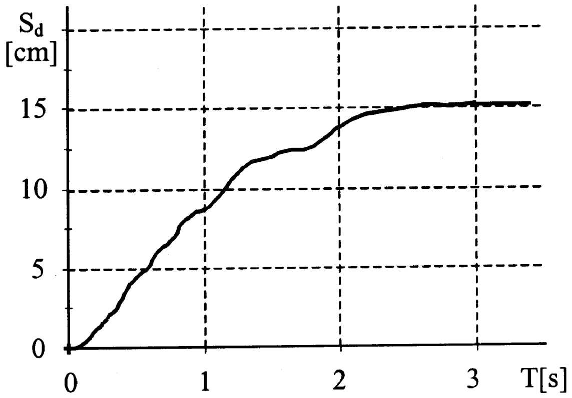

Fig. 15 Mean displacement spectrum of the thirty real earthquakes

On the contrary substantial differences among the design criteria are found when the damage index related to the accumulated plastic rotation, obtained by dynamic time history, is considered (see par.4).

In this case a better behaviour of the frame provided with some overstrength in the columns (C and Z criteria) is clearly pointed out. In authors opinion, this is due to the fact that, while the results of dynamic analyses are affected by structural cyclic behaviour, the static push-over procedure can never take account of this aspect.

With reference to the serviceability limit state, the static analyses have proved to be more severe than the dynamic ones. More in detail, the displacements related to moderate earthquakes, when obtained from the push-over curve, do not fulfil at all the code requirements, contrary to what is highlighted by the dynamic analysis, at least for the reported case.

7. Conclusions

The new performance-based design concepts appear to be useful and interesting in evaluating the behaviour of steel MRFs for different levels of ground motion.

The comparison of the static non-linear approach, which is the most suitable tool for applying the new criteria, with the dynamic time history methodology has shown a quite good agreement between the two procedures.

However, the developed analyses have pointed out that inter-story drift ratio is not sufficient to completely define the performance level of the structure, in contrast to what is stated in recent codes. This parameter, as well as the others related to a push-over curve, cannot account for the cyclic behaviour and the accumulation of plastic deformations.

In fact, the static non-linear analyses have been not able to highlight significant differences in the structural behaviour of frames having different amount and distribution of overstrength in the columns. On the contrary, cumbersome dynamic analyses have shown the effectiveness of adopting capacity design criteria: lower damage level has been obtained, as expected, for the frames provided with a more correct overstrength distribution.

In order to make the static inelastic methodology more reliable and effective for practical application, it is then necessary to improve the push-over analysis with an additional parameter or procedure related to cyclic behaviour. The authors suggest the introduction of a numerical index correlated to plastic energy dissipation, directly obtainable from the push-over curve. On the basis of the results reported here, it seems adequate to define it as the area below the static curve in the field of plastic deformations, suitably corrected. A specific research is now in progress with this aim.

As regards the serviceability limit state, since, at this stage, the global behaviour of the frame is not affected in a significant way by cyclic plastic behaviour, static non-linear analysis seems to be suitable to control the damage level when a moderate earthquake occurs. In this case, moreover, the push-over results appear to be more conservative than the dynamic results.

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