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Toward a consistent methodology for ductility checking

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ABSTRACT: An accurate design of structure subjected to seismic loads has to consider the verification of the stiffness, resistance and ductility triad. Unfortunately, in the present codes only the direct checking for stiffness and resistance is required, the ductility demands being ensured just by detailing rules. During the last great earthquakes this provision has been proved to be inadequate, the damage of steel structures being very important. Thus, a consistent methodology for direct ductility checking is required by design practice. This paper presents a proposal for such methodology, which considers the interaction between local and global ductility.

1 INTRODUCTION

For an efficient seismic design is necessary to use plastic analysis in which ductility plays an important role. The behaviour of a structure depends on ductility requirements, comprising both the earthquake characteristics and the available ductility of the individual members, which is limited by buckling of compression plates or fracture of tension parts. Therefore, for a proper design of steel structures subjected to seismic loads, the ductility checking should be quantified at the same level as for stiffness and strength. Unfortunately, in the present codes there are only vague provisions "...when plastic global analysis is used, the members shall be capable of forming plastic hinges with sufficient rotation capacity to enable the required redistribution of bending moment to develop..." (EUROCODE 3, 5.3.1), "...sufficient local ductility of members or parts of members in compression shall be assured..." (EUROCODE 8, 3.5.3.1). These two examples show the very rough definitions given by codes. For the structural designer is essential to have a clear definition of what " sufficient rotation capacity" or " sufficient local ductility" means and how these terms can be quantified.

The EC 8 considers that sufficient ductility for members shall be assured by limiting the width-to-thickness ratio of compression parts, according to the cross-sectional classes specified in EC 3. For plastic global analysis, EC 3 requires that all members developing plastic hinges shall have class 1 cross-sections, and under special conditions also class 2. EC 8 gives limitations for the q-factor value in relation with three behavioural classes, being the use of class 4 sections not allowed in dissipative zones. For joints the provisions given in Annex of EC3 considers only some constructional details, without any explicit ductility determination. This methodology to assure a sufficient ductility by means of constructional rules only, contains many shortcomings and in some cases is proved to be not effective because:

(i) The local ductility of members depends not only on width-to-thickness ratios, but also on the flange and web interaction, member length, moment gradient, level of axial forces, etc. As a consequence of such additional factors, the concept of cross – section behavioural classes should be substituted by the concept of member behavioural classes (Gioncu & Mazzolani, 1994, Gioncu and Petcu, 1997, Mazzolani & Piluso, 1993, Anastasiadis, 1999).

(ii) The provisions of EC 3 concerning the ductility of members and joints refer to the static loads. In case of seismic actions, the local behaviour of cross-sections is considerably different due to cyclic and high velocity characteristics of loads (Gioncu, 2000, Gioncu et al, 2000a, Anastasiadis et al, 2000). These new factors reduce the local ductility and, in some cases, can transform the plastic deformations in a brittle fracture (for instance, the connection failure during Northridge and Kobe earthquakes). In addition, the ductility demand is strongly influenced by the earthquake type (near or far-source) (Gioncu et al, 2000b).

(iii) The code imposes that plastic deformations occur only at the beam ends and at the column bases, but without considering the joints, which under some conditions can show a stable behaviour. But in reality, the required overstrength of connections (the joint capacity must be 20% stronger than the adjacent member) does not assure the elastic behaviour of joints. As a consequence, the joint could be the weakest component of the node and its ductility cannot be ignored (Gioncu, 1999a, Gioncu et al, 2000a).

For these reasons, it is strongly required by design practice to have a comprehensive methodology for ductility checking. The present paper presents such a method in which all the above mentioned factors are considered.

2 DUCTILITY CHECKING IN SEISMIC DESIGN

Building in seismic areas requires the development of a particular design philosophy. The basic principle of this philosophy consists in considering that it is not economically justified that, in a seismic active area, all structures should be designed to survive the strongest possible ground motion without any damage. In the rare event of very strong ground motion, damage would be tolerated as long as the structure collapse is prevented. The main goal of seismic design and requirement is to protect life and structure collapse. However, the last earthquakes have been characterized by element collapses, interruption of functionality for many buildings, evacuation of people, losses in work places for varying periods, monetary losses and, therefore, they have shown that the above mentioned goal is not sufficient for a proper design methodology. So, in the last time the concept of multi-level design approach is proposed as a basic design philosophy. In the Vision 2000 Committee of SEAOC (Bertero, 1996) four levels of structural performance are proposed: *fully operational*, *operational*, *life safety* and *near collapse* for frequent, occasional rare and very rare earthquakes. Mazzolani and Piluso (1996) propose three levels: *serviceability*, *damageability* and *survivability* limit states. Contrary EC 8 proposes two levels verification: *serviceability*, and *ultimate* limit state. Among these proposals, the verification for three levels seems to be more reasonable for design practice. To be effective for design, these performance levels must be translated into seismic action values in term of design accelerations. In this context, it is necessary to decide the return period for each level. For three performance levels it is admitted that 10, 50 and 450 years correspond for the above mentioned limit states, respectively. The adequate accelerations result from recurrence relations established for each seismic area (Figure 1a).

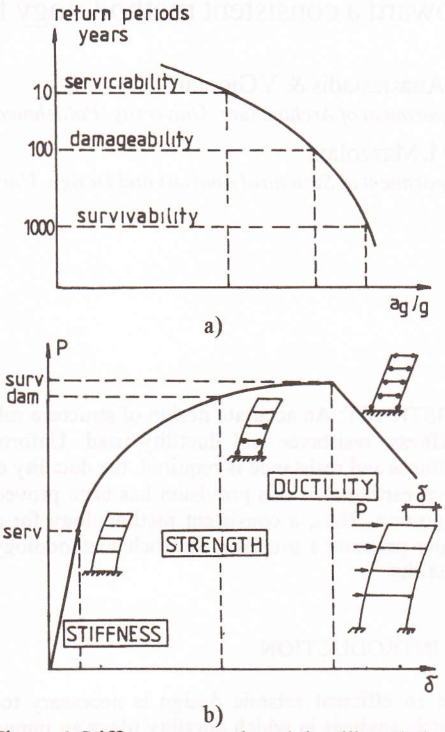


Figure 1 Stiffness, strength and ductility triad.

In the capacity design method a proper seismic design must consider the verification of structure stiffness, strength and ductility (Bachmann et all, 1995). Because the verification of this triad for each above limit states is too cumbersome, it seems that it is more rational to perform the stiffness, strength and ductility checks at different limit states: stiffness for serviceability, in case of frequent and weak earthquakes, strength for damageability, for rare and moderate earthquakes, and ductility for survivability, in case of very rare and strong earthquakes (Figure 1b).

The designer must verify the stiffness in elastic range (linear analysis), the strength by elasto-plastic analysis, using one of the well known methods (equivalent static analysis, push-over analysis, time-history analysis) and the ductility with the collapse kinematic mechanisms of structure (local and global mechanisms).

3 REQUIRED AND AVAILABLE DUCTILITIES

Ductility assessment of a structure is provided by satisfying the limit state criterion:

$$\gamma_r D_{req} \leq \frac{D_{av}}{\gamma_a} \tag{1}$$

where D_{req} is the required ductility, obtained from the global plastic behaviour of structure, and D_{av} is the available ductility determined from the local plastic deformation, while γ_r, γ_a are the partial safety factors for required ductility and available ductility, respectively. These two safety factors must be determined considering the scatter of data with a mean plus one standard variation. Values $\gamma_a = 1.3$ and $\gamma_r = 1.2$ are proposed for this verification, if the available ductility is determined by plastic deformation. If the available ductility results from local fracture, a greater value of γ_a must be used ($\gamma_a = 1.5$). The relationship (1) is presented in Figure 2a. In the range where this relation is not satisfied, the inelastic force redistribution is not assured and the structure may collapse. Another indicator of structure behaviour is the ductility index (Figure 2b):

$$I_D = \gamma_r \gamma_a \frac{D_{req}}{D_{av}} \qquad (2)$$

The elastic limit with minor damage corresponds to the ductility index of 0.1, superficial and repairable damage to the value of 0.4 and collapse limit to the damage index of 1.0. Values of the ductility index greater than 0.6 show unrepairable damage and values over 1.0 correspond to the extensive damage and progressive collapse state. This ductility index can be used as indicator of survivability limit state.

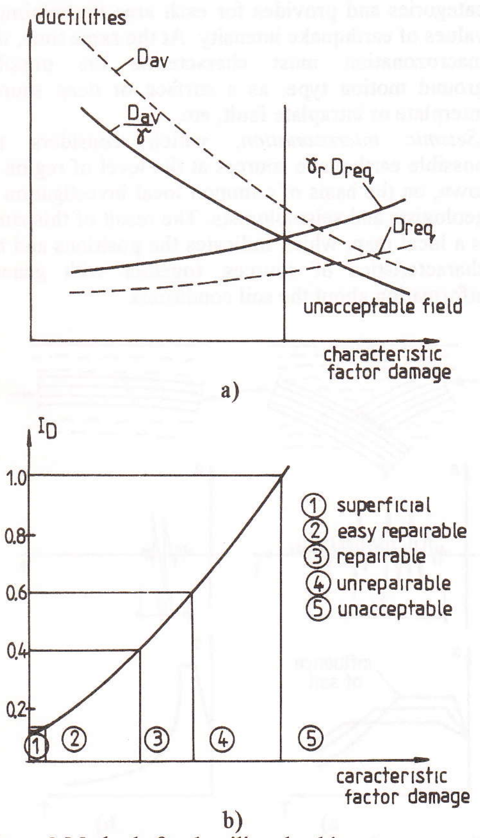


Figure 2 Methods for ductility checking.

4 GLOBAL DUCTILITY AS REQUIRED DUCTILITY

The global ductility is directly related to the earthquake characteristics. In the last time a great amount of information concerning the feature of earthquakes is collected and important databases are operative. Important activity in macro and microzonation has been carried out all over the World to identify and characterize all the potential sources of ground motions. For the structural engineers the interest of these results is focused in the source characteristics with direct influence on seismic action. Source depth has a considerable influence on the earthquake behaviour and may be classified as (Figure 3):

-*surface sources*;
-*deep sources*.

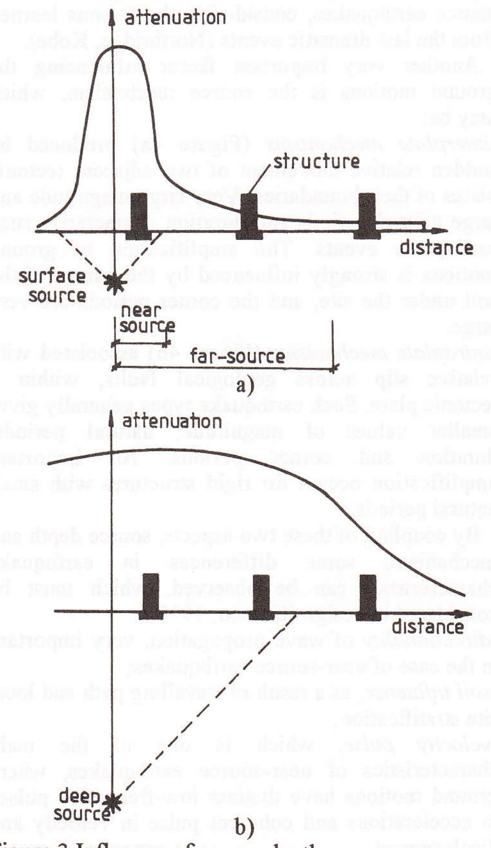


Figure 3 Influence of source depth.

Generally the surface sources are more frequent, over 85 percent of recorded earthquakes being ranged within 15Km. The importance of source depth is underlined by the attenuation low, which is very important for surface earthquakes (Figure 3a). So, the surface earthquakes have a great influence on the reduced area around the epicenter. In the last time, two main regions with different ground motion characteristics are considered (Figure 3a) (Iwan, 1996):

-*near–source region*, which can be defined as the region within few kilometres from either the surface rupture or the projection on the ground surface of the fault rupture zone. This region is also referred as near field region;

-*far-source region*, situated at some hundred kilometres far from the source.

For deep sources, the attenuation is reduced and the affected areas are very large (Figure 3b).

Unfortunately, the ground motions and the design methods adopted in the majority of codes are mainly based on records obtained from intermediate or far-source fields, being unable to describe in a proper manner the earthquake action in near-source field. Only the last UBC 97 has introduced some supplementary provisions concerning the near-source earthquakes, considering the lessons learned from the last dramatic events (Northridge, Kobe).

Another very important factor influencing the ground motions is the source mechanism, which may be:

-*interplate mechanisms* (Figure 4a) produced by sudden relative movement of two adjacent tectonic plates of their boundaries. Very large magnitude and large natural periods and duration characterize such earthquake events. The amplification of ground motions is strongly influenced by the nature of the soil under the site, and the corner periods are very large.

-*intraplate mechanisms* (Figure 4b) associated with relative slip across geological faults, within a tectonic plate. Such earthquake types generally gives smaller values of magnitude, natural periods, duration and corner periods. An important amplification occurs for rigid structures with small natural periods.

By coupling of these two aspects, source depth and mechanism, some differences in earthquake characteristics can be observed, which must be considered in design (Gioncu, 1999):

-*directionality* of wave propagation, very important in the case of near-source earthquakes;

-*soil influence*, as a result of travelling path and local site stratification;

-*velocity pulse*, which is one of the main characteristics of near-source earthquakes, where ground motions have distinct low-frequency pulses in accelerations and coherent pulse in velocity and displacement;

-*cyclic movements*, characteristic for far-source earthquakes, where the number of high value cycles is essential for the determination of ductility demands;

-*vertical components*, very high in the near-source region, being in many cases greater than the horizontal components;

-*velocity of ground motions*, with very important values in near-source regions, giving rise to very high strain-rates and impending the formation of plastic hinges in the structure members.

Without considering all these aspects in the evaluation of the required ductility, every design methodology should be incomplete. But this is a very difficult task, which oversteps the possibilities of structural engineers. The co-operation with the seismologists, geologists and geotechnical engineers is necessary. The interaction between ground motion types and ductility demand requires to pay attention to some important aspects concerning the interaction between local-source conditions and structures (Table1):

-*Seismic macrozonation*, which is an official zoning map, at the level of a Country, based on a hazard analysis elaborated by seismologists and geologists. This map divides the national territory in different categories and provides for each area the minimum values of earthquake intensity. At the same time, this macrozonation must characterize the possible ground motion type, as a surface or deep source, interplate or intraplate fault, etc.

-*Seismic microzonation*, which considers the possible earthquake sources at the level of region or town, on the basis of common local investigation of geologists and seismologists. The result of this study is a local map, which indicates the positions and the characteristics of sources, together with general information about the soil conditions.

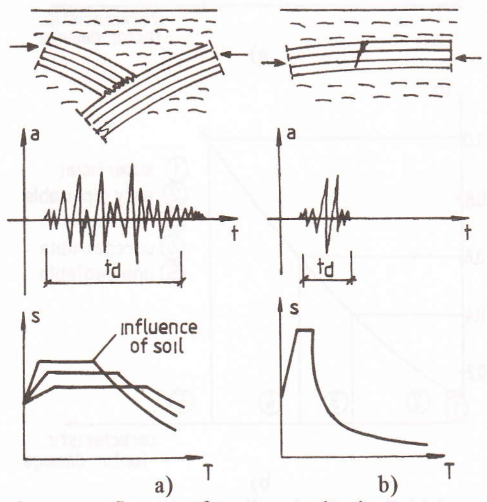
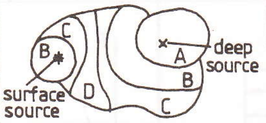
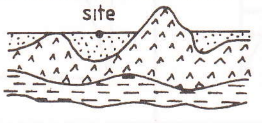
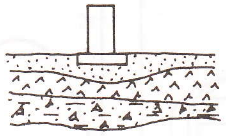
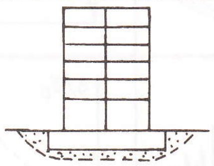


Figure 4 Influence of source mechanisms.

Table 1. Steps in control of required ductility.

Activity	Scheme	Specialits	Informations
Macrozonation		• seismologist • geologist	• earthquake types • intensities
Microzonation		• seismologist • geologist	• source position • intensities • attenuation • duration
Site conditions		• geologist • geotechnical eng.	• soil stratification • soil type • amplification • duration • time-history records • spectrum
Structure characteristics		• geotechnical eng. • structural eng. • architect • builder • owner	• level of protection • general configuration • materials • foundation type • structural system

-*Site conditions*, established by geologists and geotehnical engineers, from the examination of the stratification under the proposed structure site. The changing in ground motions (amplification of accelerations, modification of natural vibration periods, increasing of duration, etc) due to soil conditions must be specified as a result of site examination.

-*Structure characteristics*, which result from the collaboration among geotehnical engineers and structural engineers, architects, builders and owners. At this step the level of seismic protection is established and the ductility demand is fixed as a function of this level. General configuration, structural materials, foundation and elevation types, technology of erection, etc. are the results of this activity.

The definition of required ductility inevitably calls for a series of engineering judgements of seismology, safety policy as well as structural matters. For this reason, the required ductility should be established in close collaboration between seismologists and structural engineers. Unfortunately, there are some difficulties in communication between these professionals. Seismologists break their research works at the level of spectra without being interested in structure behaviour. In contrast, structural engineers have no sufficient knowledge in the seismological problems. So, an important gap exists between the view points of these two specialists category, impeding a reliable definition of seismic actions.

In order to establish the required ductility, the available methods for the designer are: monotonic static linear analysis (equivalent static analysis), monotonic static nonlinear analysis (push-over analysis) and dynamic nonlinear analysis (time-history analysis), presented in Table 2 with all the determinant factors:

-*Equivalent static analysis*, based on the assumption that the structural behaviour is governed by the first vibration mode. The characteristics of ground motions are described by means of linear elastic spectrum. For the inelastic deformations the design spectra are obtained by means of a reduction factor, namely q-factor. In this method the required ductility, D_{req}, is directly related to q-factor. These values are given by (Figure 5a):

$$D = \frac{\Delta_{pu}}{\Delta_e} \quad ; \quad q = \frac{Q_e}{Q_p} \qquad (2a,b)$$

In the literature there are some proposals for the relationship between D and q:

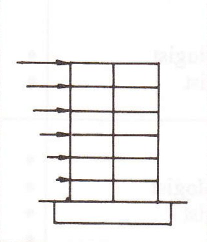
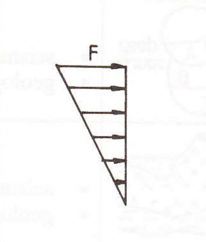
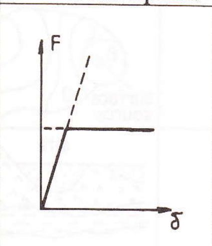
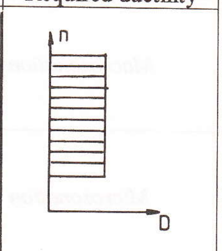
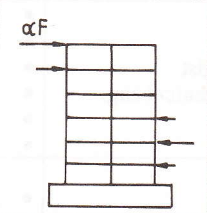
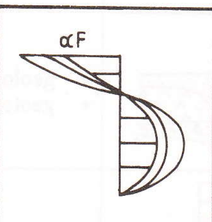
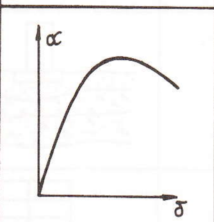
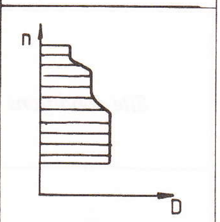
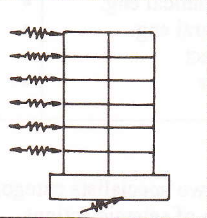
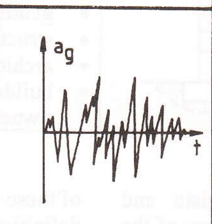
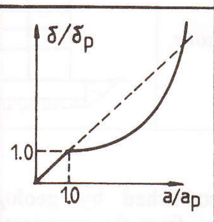
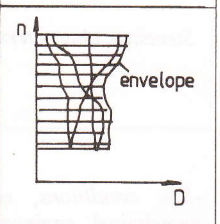
• Veletsos and Newmark (1960) for SDOF systems:

$$q = \sqrt{2(D+1)-1} \qquad (3a)$$

resulting

$$D_{req} = \frac{q^2-1}{2} \qquad (3b)$$

Table 2. Available methods for determining the required ductility

Method	Structure actions	Loading type	Structural response	Required ductility
Equivalent static analysis				
Push-over analysis				
Time-history analysis				

- Shinozouka and Moriyama (1989) for MDOF systems:

$$q = \varepsilon\sqrt{2(D+1)-1} \tag{4a}$$

resulting:

$$D_{req} = \frac{\left(\frac{q}{\varepsilon}\right)^2 - 1}{2} \tag{4b}$$

where ε is determined taking into account the scattering of numerical tests, using the average ± one standard deviation. For buildings with 3, 5 and 10 levels, after the examination of 711 cases, results ε ≈ 0.85.

- Mazzolani and Piluso (1993) for MDOF:

$$q = \frac{2}{3}D + 1 \tag{5a}$$

resulting:

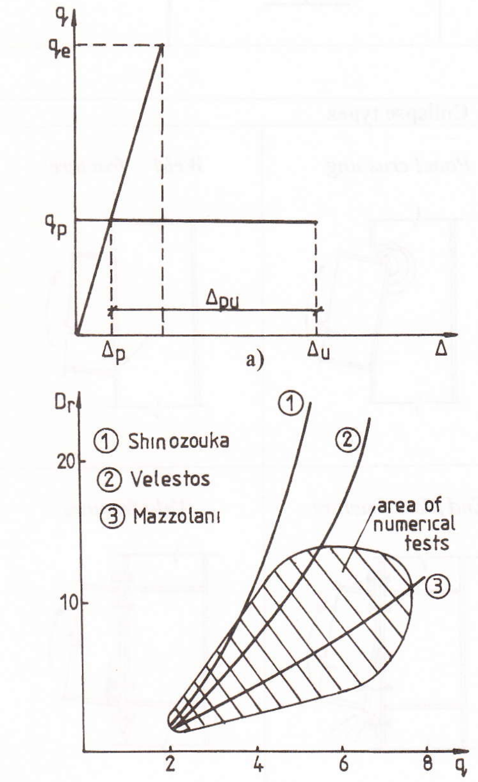
$$D_{req} = \frac{3}{2}(q-1) \tag{5b}$$

The proposals coming from Veletsos & Newmark and Mazzolani &Piluso correspond very closely with the medium values for required ductility. The Shinozouka & Moriyama relation gives the maximum values of ductility demands (Figure 5b).

-*Push-over analysis*. The structure is subjected to incremental lateral loads, using one or more predetermined local patterns of horizontal forces. These load patterns are supposed to describe the lateral load distributions which occur when the structure is subjected to earthquakes (Mazzolani & Piluso, 1996). The determination of these patterns is a very difficult task, because it depends on the influence of superior vibration modes and the progressive plastic hinge formation. Mazzolani and Piluso (1997) develop a simplified methodology based on the rigid plastic collapse mechanism by substituting the actual curve with a tri-linear one (Figure 6). The first part corresponds to a linear behaviour, while the equilibrium curve of collapse is determined by second-order rigid-plastic analysis and can be described by the following relationship:

$$\alpha = \alpha_0 - \gamma_s \delta \tag{6}$$

where is α_0 is the collapse multiplier of the horizontal forces, obtained by rigid-plastic analysis and γ_s is the slope of the linearized mechanism curve, determined in function of mechanism type. The cusp produced by the intersection of elastic curve and mechanism equilibrium curve is cutted by a horizontal straight line, corresponding to a point of mechanism equilibrium curve with a sway displacement equal to 2.5 times the elastic displacement.



b)

Figure 5 Equivalent static analysis.

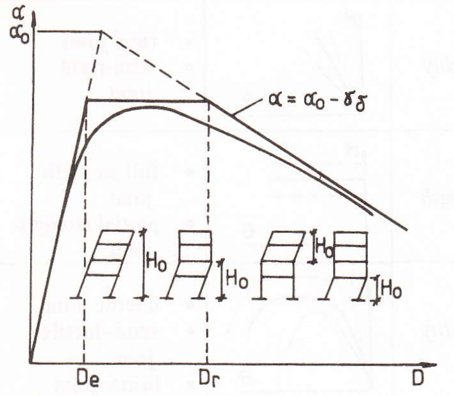


Figure 6 Push-over analysis.

The required rotation of plastic hinges can be determined by the relationship:

$$\theta_{req} = \frac{1}{H_0}(\delta_u - \delta_y) \tag{7}$$

where H_0 is the sum of the intrestorey heights of the storeys involved in the collapse mechanism. The ultimate displacement value can be determined corresponding to near collapse criteria (Gioncu 1999b). Using this methodology, the required ductility for each storey must be determined.

The push-over analysis is relatively simple to be implemented, but contains a great number of assumptions and approximations that may be reasonable in some cases and unreasonable in other ones. Especially, when the superior vibration modes have important effects, the obtained results can be very far from the actual behaviour of structure.

-*Time history analysis*. The structure is subjected to an artificial or recorded accelerogram and the structure response is determined by considering the nonlinear elasto-plastic deformation of structure. The result of this analysis is an envelop of required ductility for each structure levels. The maximum demands may occurs at different levels along the structure height, in function of the earthquake natural period. In the case of short periods, the structure top is more affected, while for long periods the maximum required ductilities occurs at the first levels (Figure 7) (Gioncu et al., 2000b). Due to the development of computer science, today is not a problem to perform such a complex analysis. But the real problem of this method is the option for an accelerogram, which adequately represents the earthquake at the structure site.

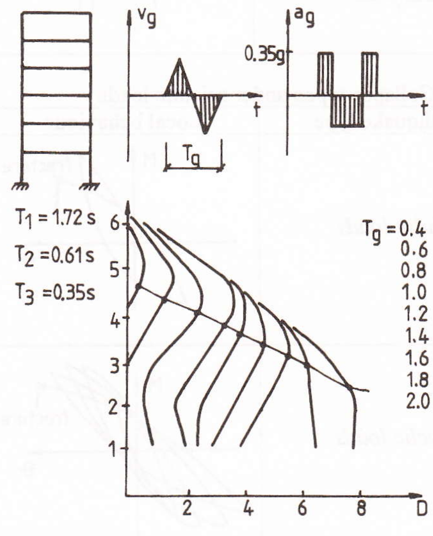


Figure 7 Time history analysis.

Table 3 Collapse modes of members

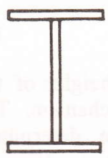
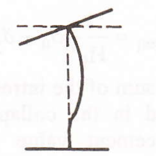
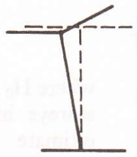
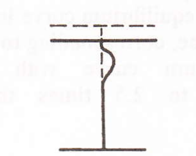
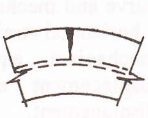
Member type	In-plane buckling	Out-of plane buckling	Flange induced buckling	Flange fracture
				

Table 4 Collapse modes of joints

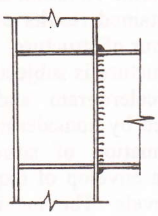
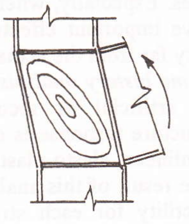
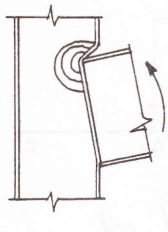
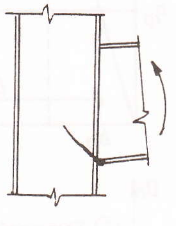
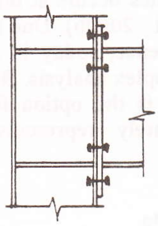
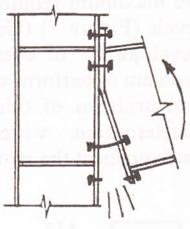
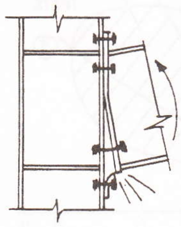
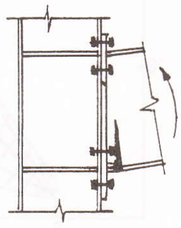
Joint type	Collapse types		
Welded joint	*Panel buckling*	*Panel crushing*	*Weld fracture*
			
Bolted joint	*Bolt fracture*	*End plate fracture*	*Weld fracture*
			

Table 5 Collapse types under seismic loads

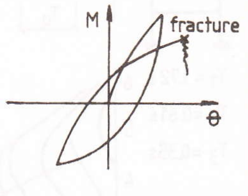
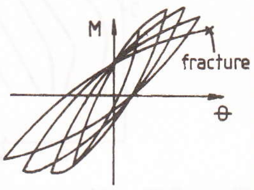
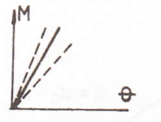
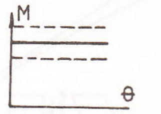
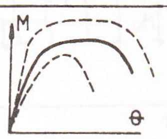
Earthquake type	Local behaviour
Pulse loads	
Cyclic loads	

Table 6 Classification criteria of joints

Properties	Behaviour	Joint type
Rigidity		• rigid joint • semi-rigid joint
Strength		• full strength joint • partial strength joint
Ductility		• ductile joint • semi-ductile joint • brittle joint

The choice of an accelerogram is a very complex task due to the fact that at the same site, as a result of the same source, the ground motions may be very different in characteristics for different events. Therefore, the method of amplification of the peak ground acceleration without changing other characteristics (periods, duration, velocities, etc), what in generally done according to this method, is very disputable.

All the above mentioned methods contain many assumptions which can introduce some errors in the evaluation of required ductility. Thus, the interpretation of results must be done within the context of the used assumptions. The determination of a realistic ductility demands is one of the most complex problem because contains many uncertainties and discussions beyond the current knowledge of a structural engineer. This may be an explanation why today the verification of structure ductility is more an exception than a rule. But these problems do not differ very much from the ones concerning the strength and rigidity verifications. However, in order to minimize the assumed risk in the prediction of the ductility requirements, it is necessary to estimate the seismic activity, to evaluate the local soil conditions as well as to assess the structural behaviour under the estimated and predicted conditions.

5 LOCAL DUCTILITY AS AVAILABLE DUCTILITY

The determination of local ductility is more related to the structural engineer judgements than to the required ductility and contains less uncertainties (Gioncu, 1997). Beams, columns and joints compose a framed structure. In seismic design some critical sections are chosen to form a suitable plastic mechanism able to dissipate an important amount of the input energy. Generally, it is considered that these sections are located at the beam ends, where plastic hinges occur during a strong earthquake. But the beam is joined to a node, which connects also the column. Furthermore, the local plastic mechanism in the structure can be located not only at the beam or column ends, but also at joints, or at both member ends and joints. Consequently, the local ductility has to be defined at the level of node, composed by panel zone (column web), connection elements (bolts or welds, plates, angles, etc) and member ends (Figure 8) (Gioncu, 1999, Gioncu et al, 2000a).

The collapse modes of the members are presented in Table 3: in-plane, out-of-plane, flange induced buckling types and flange fracture. The collapse modes for welded or bolted joints are shown in Table 4. It is interesting to notice that in case of welded joints the collapse mode is governed by the panel collapse, while for bolted joints, by connection elements. The main aspects of these collapse modes are presented in Gioncu and Petcu (1997), Gioncu et al (2000a), as well as, in the companion papers Anastasiadis et al (2000) and Gioncu (2000).

The types of local failure must be considered for available ductility under seismic loads (Table 5). In the case of pulse loads, characteristic for near-source earthquakes, the great velocity induces very high strain-rate and fracture of members or joints occurs at the first or second cycle. Contrary, if the action is characterized by cyclic loads, especially for far-source earthquakes and soft soils, an accumulation of plastic deformation occurs, producing a degradation in behaviour and the fracture takes place after a high number of cycles.

In order to establish the weakest component of a node, the joint properties must be compared with the properties of connected members in terms of rigidity, strength and ductility (Table 6). So, the joints may be classified according to their capacity to restore the properties of beams and columns. Based on the method of components, the overall behaviour of the node is dictated by the behaviour of the weakest component (Tschemmernegg, 1998), which is determined by the comparison of the two plastic moments. The node ductility is given by the component with the smallest value.

6 DUCTILITY CHECKING

The capacity design method is based on the concept that the available ductility, determined from local ductility, is greater than the required ductility, obtained from global ductility. A chart for determining the global and local ductilities, as well as, the conceptual ductility checking is illustrated in Figure 9.

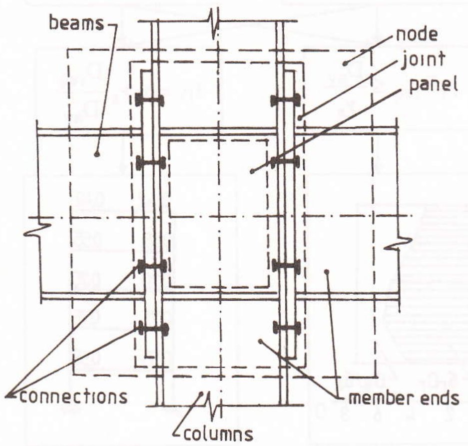


Figure 8 Node components

For global ductility the hierarchy is at the level of source, epicentral distance, site and structure, while for local ductility material, cross-section, member and connections are the main factors. The comparison between required and available ductilities, D_{req}, $D_{av.}$, can be performed in two ways as follows:

-*direct verification* using the equation (1);

-*calculation of ductility index* given by the equation (2).

The use of direct verification has the purpose to assure that the redistribution of forces after the formation of plastic hinges, in some predetermined sections, is going to be under stable conditions in order to prevent the structure collapse. Contrary, the use of ductility index has the advantage to limit the member and joint damage below an acceptable level, in order to allow for an easy repairing. The reserve

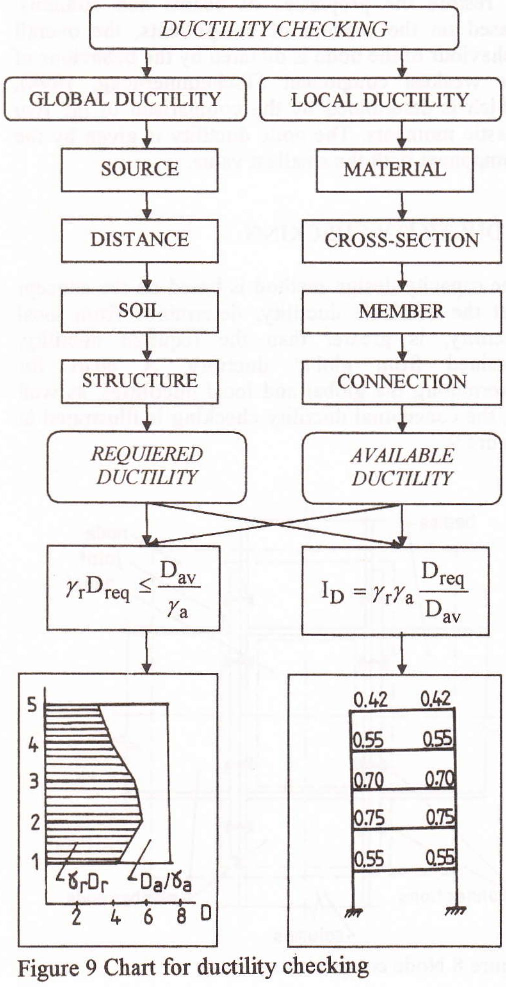


Figure 9 Chart for ductility checking

of ductility in frames subjected to some repeated earthquakes may be determined by using the latter approach.

7 CONCLUSIONS

In seismic analysis the most difficult problem is to predict in a proper manner the earthquake type and the seismic actions, because a great variability of these characteristics exists. The code provisions are normaly very poor, being based only on a reduced number of design parameters, which cannot cover the possible seismic actions. Due to this code lack, in many cases the structure behaviour is studied by structural engineers starting from a wrong distribution of lateral forces. Consequently the obtained results are far from the reality. Only the introduction in code provisions of more reliable methods to establish the seismic actions can solve this situation. Therefore, the co-operation with the seismologists must be enlarged. As usually the seismologists have a limited knowledge on the structure behaviour, it is the duty of structural engineers to fill the existing gap.

Recent developments of advanced design concepts, as the ones introduced in the capacity design method, are based on the scope to provide the structure with sufficient ductility, in the same way as for strength and rigidity, in order to minimize the aforementioned problems. For these reasons, a consistent, comprehensive and transparent methodology is developed here which considers the required and available ductilities determined at the levels of the overall structure as well as at the local levels of the structural components. The main factors influencing these ductilities are presented. One can consider that today the accumulated knowledge allows to elaborate a sufficiently simple, but consistent methodology, which can be implemented in the modern codes.

For instance in EC8, instead to refer to the provisions of EC 3 concerning the ductility of cross-section under statical conditions, it should be more useful to elaborate an Annex, in which the bases of ductility checking in seismic conditions are presented. At the same time, some constructional details, very important to assure an adequate seismic behaviour, preventing the local damages, are required to be introduced in this Annex.

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