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FATIGUE ANALYSIS OF MOMENT RESISTING STEEL FRAMES

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A procedure for the fatigue assessment of steel building structures subjected to earthquakes is presented. The procedure constitutes an extension of the present, high-cycle, fatigue assessment to cases of low-cycle fatigue. It may serve as a basis for the introduction of a fatigue limit state in the earthquake design of steel structures. It may be also used for the damage assessment of existing steel buildings subjected to past earthquakes. By means of parametric studies, the effects of various parameters on the fatigue susceptibility of several moment resisting steel frames are studied. The influence of a number of parameters such as the type of ground motion, type of structural typology, local fatigue behaviour, overall frame design and semi-rigidity of joints on the susceptibility to damage are investigated.

Keywords: Fatigue; steel structures; moment-resisting frames.

1. Introduction

It is well known that structures do not remain "young" forever, as they are subjected to deterioration due to use, climatic actions, etc, during their life. Besides inspection that reports on existing deterioration, there is a need to predict possible structural damage due to defined loading histories in order to ensure that the structure is unlikely to fail by fatigue. In addition, it is necessary to evaluate if and to what extend immediate repair to existing structures with damages is required or how far inspection intervals are to be shortened if repair is going to be postponed. A (well established) fatigue assessment exists for steel structures subjected to repeat dynamic loading, such as bridges, offshore platforms, cranes and crane girders, etc. For these structures, verifications in the limit state of fatigue are often more critical than in the serviceability or the ultimate limit state. This implies that fatigue resistance may be more critical than structural stiffness or strength. As a result, construction details often have to be modified in order to behave more beneficially with respect to fatigue. Contrary to the previous types of structures, building structures are normally excluded from a fatigue assessment [Eurocode, 1992].

A fatigue analysis is not presently required for steel building structures subjected to earthquakes. Seismic design, as currently prescribed in the relevant codes, as in Eurocode [1994], refers mainly to the provision of stiffness, strength and ductility. The latter is required due to the fact that structural elements are allowed to yield during strong earthquakes.

However, such a design methodology inevitably accepts structural damage due to the developing inelastic action [Akiyama, 1985]. In spite of the recognition of this fact, a fatigue limit state is not introduced in the codes. One reason, of a rather practical nature, is that from past experience steel structures were not considered particularly vulnerable to earthquakes. Another reason is that despite the similarities to the usual fatigue mechanisms, i.e. formation and growth of cracks, there exist differences to the fatigue due to the usual dynamic loading. More specifically, fatigue due to earthquake loading is not due to a large number of applied cycles of rather low nominal stress, but due to a small number of applied inelastic deformations cycles, i.e. it is a case of low-cycle fatigue. This difference calls for a different assessment procedure of the phenomenon, unlike for high-cycle fatigue, on which, a generally accepted agreement is still lacking.

This attitude changed after the earthquakes of Northridge, USA [Youssef *et al.*, 1995], and Kobe, Japan [Yamada, 1996] in 1994 and 1995, respectively. An observation of damage sustained by buildings in those two earthquakes indicates that contrary to the intended behaviour, in many cases brittle fractures are initiated within the connections at low levels of plastic demands (and in some cases, especially in Northridge) while the structures remained elastic. Extensive experimental and analytical investigations were subsequently undertaken to analyse and explain the relevant behaviour and recommend methods of repair and improvements in joint design [Roeder, 2000; Kato *et al.*, 1997; Kurobane *et al.*, 1997]. The results of the investigations indicate that poor fatigue resistance due to low material toughness, poor weld execution, high stress concentrations of details, restraint conditions, high strain rates, low temperature, etc, were the main causes of failure.

In Europe, a joint research project between 13 universities and relevant institutions in eight countries, entitled "Reliability of moment resistant connections of steel building frames in seismic areas", has been supported by the European Community [Mazzolani, 2000]. One part of the program in Athens dealt with the interaction between global and local behaviour of the frames. Based on the previously made observations, it was found that the overall structural performance could be evaluated by means of fatigue analysis. This paper presents the relevant methodology, presenting its applications for moment resisting frames and illustrates by means of parametric studies its implications in design. The method, when supported with sufficient experimental data with respect to the fatigue resistance, may be suggested when a fatigue limit state would be introduced in seismic design.

2. Fatigue Assessment Procedure

Methods for the fatigue assessment of dynamically loaded steel structures are well established. The application of fracture mechanics is possible, if sufficient information on the stress intensity factor, the expected pre-existing defects and the weld shape in the vicinity of these effects exists. However, not only are such data seldom available, but also that designers are not familiar with fracture mechanics, which is more or less applied by the specialists. The usual design procedure is based on nominal stress ranges for classified constructional details, whose fatigue resistance is represented by means of appropriate Whler or S-N curves. These curves relate the nominal stress range $\Delta\sigma$ that may be resisted for a certain number of applied loading cycles N. They are determined experimentally by means of fatigue tests performed under constant amplitude loading. Figure 1 shows the typical Whler curves, as proposed in codes. It may be observed that (a) the low values of the nominal stresses indicate an elastic response, (b) the minimum number of cycles is restricted to 10000 and (c) the slope constant m of the curves increases as the number of cycles increases.

Field observations, as previously mentioned, reveal that fatigue failure occurs to structures subjected to strong earthquakes. However test results also indicate that failure is due to crack formation and growth in regions where high inelastic action develops, such as in the region of beam-to-column joints. This refers to rigid welded connections, as well as rigid and semi-rigid bolted end-plate connections, top and seat angle connections, and compact or slender column web panels, such as reported in Kato *et al.* [1997], Kurobane *et al.* [1997], Calado [2000], Dubina *et al.* [2000],

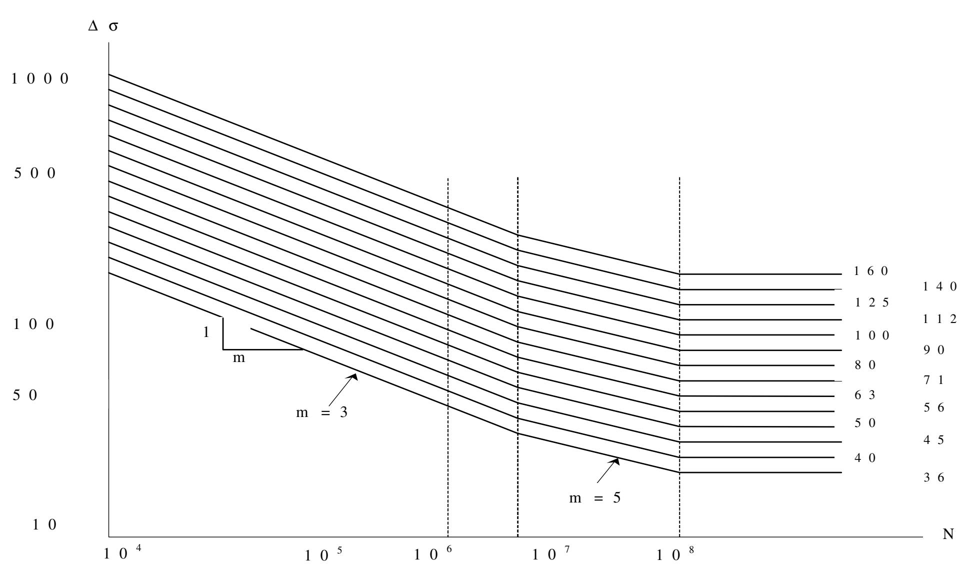


Fig. 1. Fatigue strength curves [Eurocode, 2000].

Kasai *et al.* [2000] and Vayas *et al.* [1995]. However, structural members outside joint regions have also shown to be prone to fatigue failure when subjected to cycles of inelastic deformations [Yamada *et al.*, 1989; Mateescu and Gioncu, 2000]. Finally, strain rate, material toughness and temperature effects on the connection fatigue strength could be observed [Beg *et al.*, 2000; Matsumoto *et al.*, 2000].

To accommodate for large inelastic deformations, seismic design currently relies on ductility. Ductility as conventionally defined, e.g. by means of rotation capacity for joints, is not nessesarily related to a certain number of cycles to failure. However, as past experience and analysis shows, the number of inelastic excursions reflecting the demand is related to the characteristics of both the seismic input and the structure. Such a correlation can be made possible by means of fatigue analysis. Again, methods based on fracture mechanics accounting for elastic crack propagation [Righiniotis *et al.*, 2000; Righiniotis and Hobbs, 2000], or on local notch strains are possible. However, for practical design purposes, a methodology based on experimental results on classified details seems to be most appropriate. Evidently, due to the differences in response between structures under the usual dynamic loading and buildings under earthquakes, the previously described fatigue assessment procedure for high-cycle fatigue has to be appropriately modified.

It is widely known that when structural members respond in the inelastic range, generalised deformations such as strains, rotations or displacements become more relevant than generalised forces, such as stresses, moments or forces. Therefore, it is straightforward to substitute stress ranges in the fatigue curves by appropriate deformation ranges. These may be strains or axial deformations, if the member is primarily subjected to axial loads, or rotations if the member is primarily subjected to moments. The relevant curves accordingly express the fatigue deformability rather than the fatigue strength (Fig. 2). Obviously, the most appropriate deformation quantity for connections or members in moment resisting frames is the rotation [Vayas *et al.*, 1999]. Furthermore, it is generally accepted that damage in building structures subjected to earthquakes is primarily associated to plastic deformations only, while the contribution of elastic deformations is negligible [Akiyama, 1985].

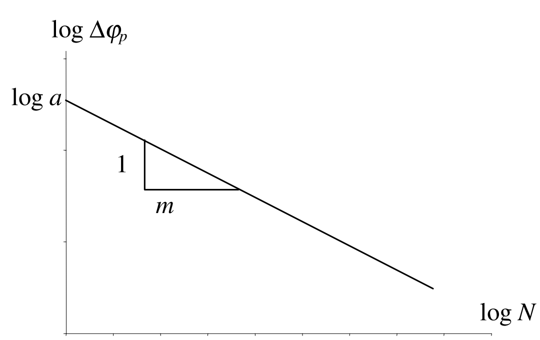


Fig. 2. Fatigue deformability curves.

This may be confirmed by re-elaboration of cyclic tests performed in accordance to both the ECCS and the ATC-24 loading procedures. The damage due to elastic deformations is indeed very small. Indicatively it may be stated that a re-elaboration of the test results described in Dubina *et al.* [2000], which were performed according to the ECCS-procedure, showed that the contribution to damage (Eq. (2)) of the deformations up to and including yielding was between 0.5% and 4%, depending on the slope constant of the fatigue curve. Similar figures resulted from a re-elaboration of the tests of Kasai *et al.* [2000], performed in accordance to the ATC-24 procedure. In all of these tests, the specimen failed at cycles between six- and ten-times the yield displacement. The relevant fatigue expression may then be writen as:

$$\log N = \log a - m \log \Delta\varphi_p \,, \tag{1}$$

where

$$\begin{aligned} \Delta\varphi_p &= \text{fatigue deformability (plastic rotation)},\\ N &= \text{number of rotation range cycles},\\ m &= \text{slope constant of the fatigue curve},\\ \log a &= \text{constant}. \end{aligned}$$

A transformation of elastic and inelastic deformations into equivalent stresses, as proposed in Ballio and Castiglioni [1995] is also possible. However, the fact that the resulting stresses may be well above the material tensile strength might be confusing for the application.

It should be noted here that there are applications where elastic and inelastic deformations are of similar orders of magnitude. In such cases that constitute intermediate problems between high- and low-cycle fatigue, elastic deformations cannot be ignored. As an example plate girders with slender webs may be referred in which fatigue fracture occurs due to web breathing at intermediate number of cycles [Machacek and Skaloud, 2000]. Table 1 summarises the various possible formulations of the fatigue analysis as well as examples of relevant applications as outlined before.

In the absence of more specific information, the damage assessment for variable ranges of plastic rotation may be performed as for high-cycle fatigue in accordance

Table 1. Formulation of fatigue rules.

Structural Response	Elastic → Inelastic		
No of Cycles to Failure	$\sim 10^4$–10^8	$\sim 10^2$–10^4	$\sim 10^0$–10^2
Fatigue Curves for Ranges of	Generalised forces	Generalised total deformations	Generalised inelastic deformations
Usual Fields of Application	Bridges, crane girders, chimneys, masts etc.	Slender plate girders etc.	Buildings under seismic loading

to the linear Palmgren–Miner cumulative law:

$$D = \Sigma \frac{n_i}{N_i}\,, \tag{2}$$

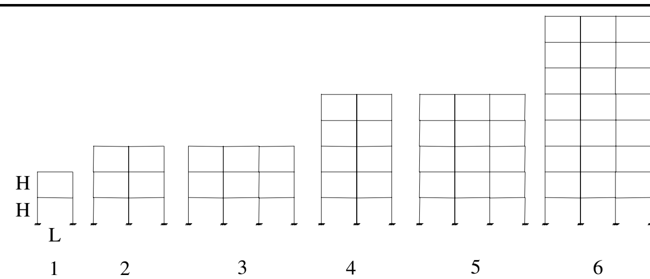
where

n_i = number of cycles for an applied range of plastic rotation $\Delta\varphi_i$,
N_i = number of cycles to failure for the same range of rotations.

The damage index D ranges between 0 (no damage) to 1 (complete damage). A validation of the applicability of the above law for low-cycle fatigue problems is still under discussion. However, a re-elaboration of limited test results with both constant and variable amplitudes result in values of the damage index at failure between 0.75 and 1.35, very close to the precise value 1. The formulation of more accurate cumulative laws based on wider experimental evidence could be envisaged.

For the determination of the design spectrum in the fatigue assessment, the *rainflow* or *reservoir* method [Eurocode, 2000] taking into account rotation- instead of stress ranges, for counting the cycles for a certain deformation history may be employed.

Table 2. Data of the first series of investigated frames.

Type of Frame					
Level of Vertical Loading	40%; 60%: Percentage of the beam moment resistance exploited for vertical loading				
Stiffness of Joints	Rigid; $0.8K$; $0.4K$ ($K = 25 \cdot EI_b/L_b$) EI_b = stiffness of the connected beam L_b = length of the connected beam				
Frame	L(m)	H(m)	T (sec)	Beam	Column
1	5	3	0.62 (0.76)	IPE 300	HEB 180
2	4	4	0.99 (1.21)	IPE 330	HEB 240
3	4	4	1.12 (1.37)	IPE 330	HEB 240
4	4	3	1.14 (1.39)	IPE 360	HEB 280
5	4	3	1.15 (1.42)	IPE 360	HEB 280
6	4	3	1.26 (1.89)	IPE 450	HEB 320

T = fundamental period for 40% (60%) vertical loading and rigid joints Yield stress of members $f_y = 235$ MPa

3. Parametric Studies

By means of parametric studies, a fatigue assessment of moment resisting frames subjected to earthquakes was performed. The parameters under investigation were the geometry of the frames, the flexibility of the joints, the level of vertical loading, the type of ground motion and the fatigue resistance (Tables 2 and 3).

With respect to geometry, a first series of two to nine stories, one to five-bay reference frames whose response has been evaluated elsewhere [Guerra *et al.*, 1990; JAC, 1999] were considered. A second series of frames (Table 3) studied in the SAC/FEMA Steel Project were also investigated.

With respect to joint flexibility, rigid and semi-rigid beam-to-column joints were examined. Joint flexibility is introduced by means of appropriate rotation springs adjacent to the joints. The degree of flexibility is expressed by a comparison of the connection stiffness with the parameter $K = 25 \cdot EI_b/L_b$ [Eurocode, 2000]. In the context of the present analysis three values of the relevant stiffness ranging from stiff to flexible were considered (Table 2).

The level of vertical loading stands for the degree of exploitation of the beams due to this type of loading. High levels of vertical loading indicate buildings in low seismicity areas, designed primarily against gravity loading. On the contrary, low levels indicate buildings designed to withstand larger earthquakes. In the context of the present analysis two levels of vertical loading, 40% and 60%, were taken into account.

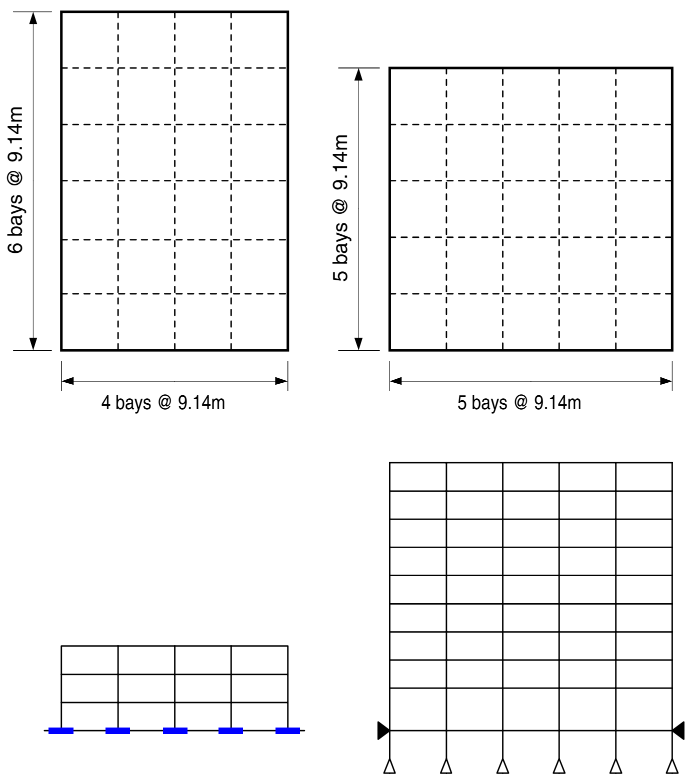
It is well known that the type of ground motion has a great influence on the structural response. In the present analysis, four records from Greece (Aigion 1985), Japan (Kobe 1995), Romania (Vrancea 1977) and USA (Northridge 1994) were considered.

As expressed in the relevant acceleration histories and acceleration-energy spectra, Fig. 3, the characteristics of the three inputs are quite different. Aigion represents a near source, shallow, impulse type earthquake of low natural period. Kobe represents a near source, cyclic type earthquake of higher period. Northridge represents a near source, cyclic type earthquake of low period. Finally, the Bucharest earthquake, denoted as Vrancea, represents a far field, cyclic type record of very large period. The corresponding PGAs for the four records are equal to $0.54g$, $0, 85g$, $0.57g$ and $0.21g$ respectively.

As outlined before, the development of low-cycle fatigue lines for certain details relies on experimental data. Unfortunately, such experimental evidence is at the present rather scarce. Systematic constant amplitude fatigue tests have been carried out more or less recently [Calado, 2000; Kasai *et al.*, 2000; Bernuzzi *et al.*, 1997] and the criteria on the definition of the number of cycles to failure are still under discussion [Calado, 2000]. The fatigue behaviour, especially for joints, depends additionally on a large number of parameters connected to the detail overall configuration, the workmanship conditions, the loading speed, the temperature, etc. For beam and beam column sections, the fatigue lines depend primarily on the

Table 3. Data of the second series of investigated frames.

Type of Frame



(7) (8)

Store	L(m)	H(m)	T (sec)	Beam	Column Exterior	Column Interior
3-story (Frame 7)						
1	9.14	3.96		W33 × 118	W14 × 257	W14 × 311
2	9.14	3.96	1.0	W30 × 116	W14 × 257	W14 × 311
3	9.14	3.96		W24 × 68	W14 × 257	W14 × 311
9-story (Frame 8)						
−1	9.14	3.96		W36 × 160	W14 × 370	W14 × 500
1	9.14	5.94		W36 × 160	W14 × 370	W14 × 500
2	9.14	3.96		W36 × 160	W14 × 370	W14 × 500
3	9.14	3.96		W36 × 135	W14 × 370	W14 × 455
4	9.14	3.96		W36 × 135	W14 × 370	W14 × 455
5	9.14	3.96	2.2	W36 × 135	W14 × 283	W14 × 370
6	9.14	3.96		W36 × 135	W14 × 283	W14 × 370
7	9.14	3.96		W30 × 99	W14 × 257	W14 × 283
8	9.14	3.96		W27 × 84	W14 × 257	W14 × 283
9	9.14	3.96		W24 × 68	W14 × 233	W14 × 257

T = fundamental period Yield stress of members $f_y = 360$ MPa (Grade A36)

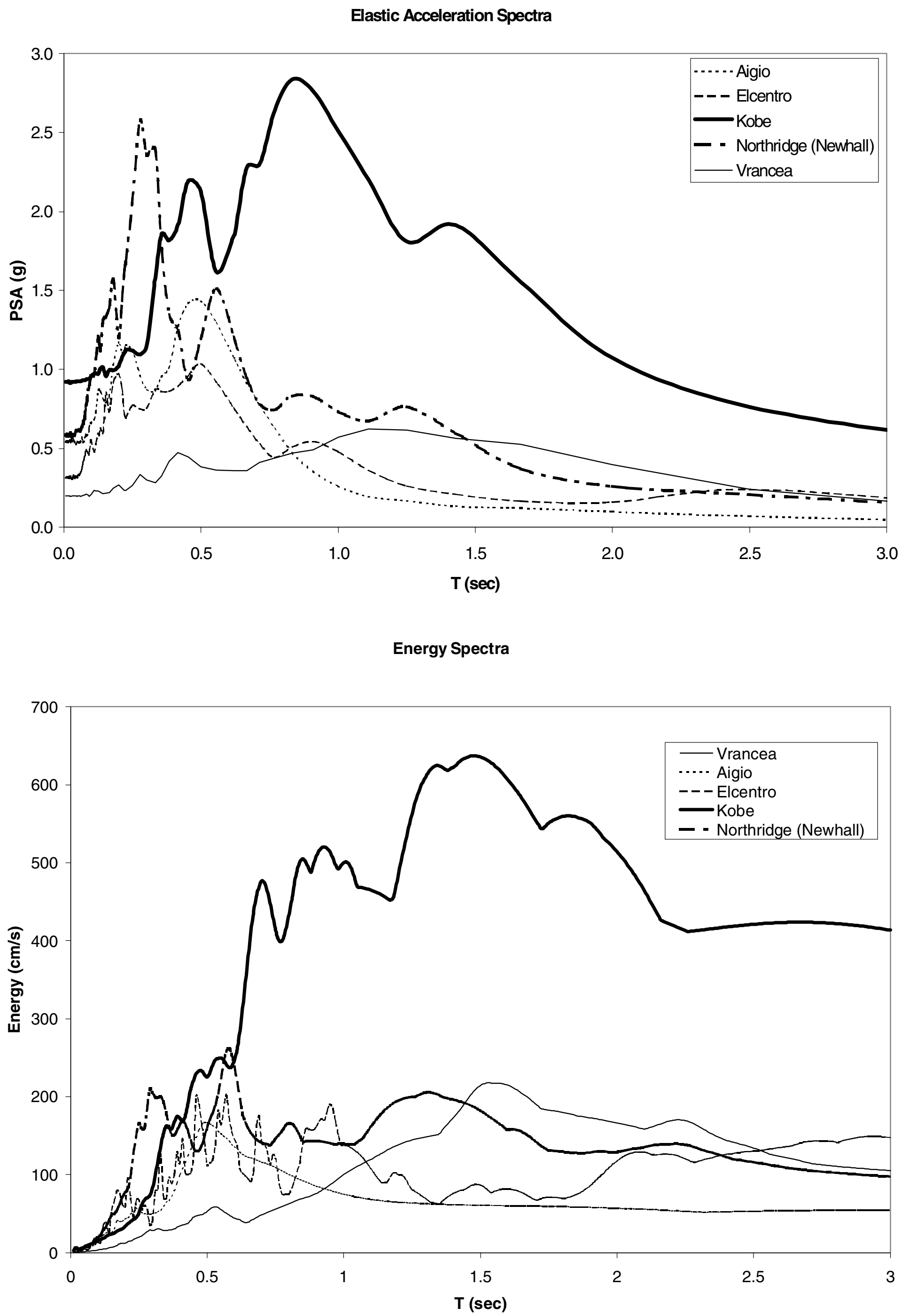


Fig. 3. Energy spectra of the records under consideration.

type of the section, I- or box-section, the compactness of its walls and the level of applied axial force. As for high-cycle fatigue, where the slope of the line ranges between 3 and 5, the slope of the low-cycle curves seems to vary too. For beam column sections subjected to moderate compression, values for the slope $m = 2$ are proposed [Yamada, 1998]. Welded connections values between $m = 1.3$ to 3.4 have been experimentally determined [Calado, 2000]. For top and seat angle connections the test of Calado, 2000 revealed values ranging from 1.3 to 3.4, while the tests of Kasai *et al.*, 2000, values between 1.2 and 2.3. The value of the constant $\log \alpha$ in Eq. (1) is better understood when the resulting rotation capacities for monotonic loading (corresponding to $N = 1/2$) are given. For the usual beam-column I-sections, values ranging from 0.06 to 0.15 radians as a function of the flange slenderness are proposed [Yamada, 1998]. For connections, the values vary largely with the connection type, being higher for flexible than for rigid connections. In the context of the present work several fatigue lines have been considered in order to study the influence between local and overall frame behaviour. The lines are described by the slope m and one point that expresses the rotation capacity φ_{mon} for $N = 1/2$. It has to be stated that the symbol φ_{mon} is used only for the purpose of better understanding and is not necessarily equal to the rotation capacity under monotonic loading, especially when the mode of failure for this type of loading is different. For the parametric study, values of 1, 2 and 3 for the slope and rotation capacities between 0.03 and 0.05 radians have been considered.

4. Numerical Investigations

It is well known that deterioration processes in structures under earthquake loads are generally nonlinear. Past experience and associated numerical analyses indicate modifications of the structural properties with respect to stiffness and strength with increasing deterioration. Depending on the characteristics of the seismic motion and the structure, these modifications may be beneficial or detrimental to the structural response. In order to account for nonlinear processes, nonlinear analysis is required. Linear analyses, if and when applied, should be at least calibrated against nonlinear analyses. In the context of the present work, nonlinear dynamic analysis was employed, by application of the general-purpose software program DRAIN-2DX [Kannan and Powel, 1975].

The frames under consideration were subjected to the various records indicated above. Vertical loading was considered uniformly distributed along the beams. The damage of the structural elements was evaluated by application of the methods described before. The cyclic behaviour of the joints was considered to be elastic-plastic without pinching or strength degradation. In fact, the welded connections that failed during the earthquakes mentioned before were expected to exhibit such a behaviour, as they were designed as full strength, rigid connections. Damage may take place not only directly at beam-to-column joints, but also at intermediate

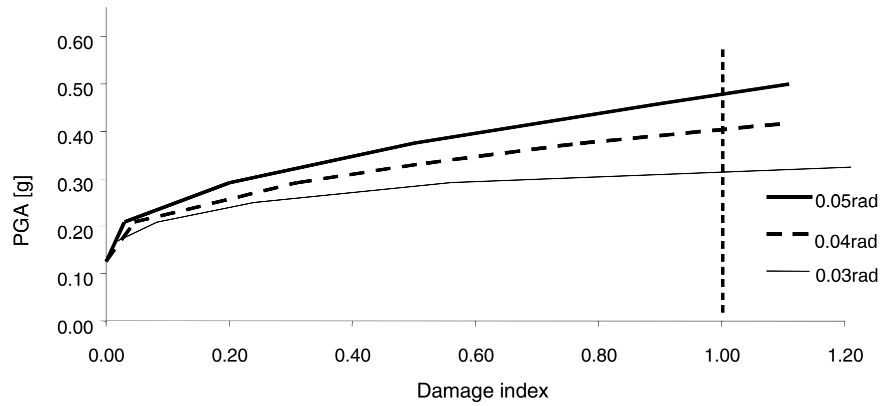


Fig. 4. Damage index versus peak ground acceleration.

positions along the members. Scaled and non-scaled records were considered. The records were scaled with respect to PGA, by keeping the level of vertical loading constant. Scaling was performed in order to evaluate the PGA (a) at which inelastic action, and therefore damage, starts and (b) at which a damage index of 1, corresponding to failure at the corresponding member, is first reported.

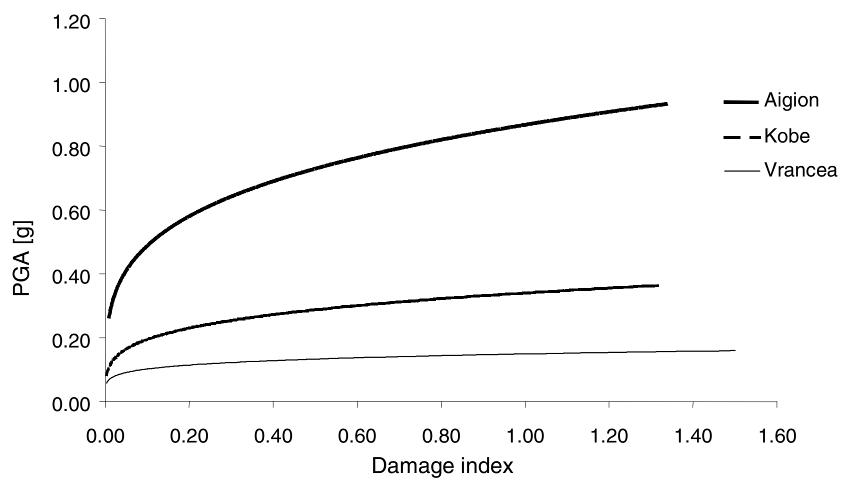
Figure 4 shows the characteristic acceleration — damage index curves calculated for fatigue curves with different rotation capacities. They show non-linear structural response analogously to load — deformation curves. The curves indicate that it might be possible to determine damage not in an absolute way, i.e. when the damage index becomes 1, but in a conventional way. In correspondence to the determination of the yield point of non-linear materials, such as high-strength steel, damage could be defined conventionally at the intersection of two straight lines. The first is the initial curve the other is the tangent of the nonlinear curve exhibiting a slope equal to 10% of the initial slope. However, in this paper damage was considered to occur when the maximum damage index within the structure became equal to 1.

5. Discussion of Results

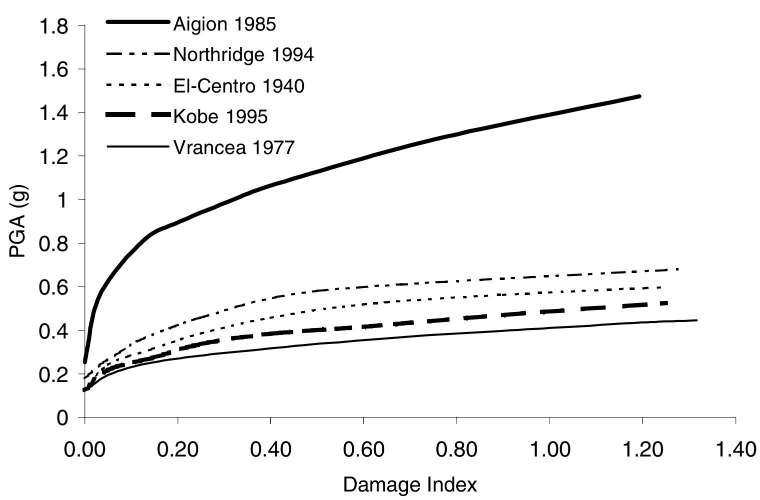
The results are based on a large number of numerical investigations, under consideration of the previously described parameters. Obviously each set of parameters that includes the frame geometry, the type of record, the joint characteristics, the level of vertical loading, the fatigue line, etc, provides its own results. As well known for such types of inelastic analyses, the results cannot be represented by smooth lines, as combinations of parameters may exhibit different behaviour. This is confirmed by field experience after strong earthquakes too, where buildings of similar configuration at the same location are often subjected to very different degree of damage. However, the numerical investigations indicate some clear trends in structural behaviour for the variation of some parameters. Therefore, the influence of some parameters will be discussed in the following by averaging the response for a larger number of frames.

5.1. *Influence of the type of ground motion*

The damage index of the frames subjected to the three seismic records under consideration is shown in Fig. 5(a) for the frames of Table 2 and in Fig. 5(b) for the frames of Table 3. The curves represent the average response of all frames.



(a) Frames of Table 2



(b) Frames of Table 3

Fig. 5. Damage index for various seismic records.

The fatigue curves used were determined by consideration of $\varphi_{mon} = 0.05$ rad and $m = 1$. It may be seen that the type of record, impulse or cyclic type, greatly influences the resulting damage. Aigion, an impulse type earthquake, is not able to produce high damage, unless it is scaled to very large PGAs.

In fact the original record with $0.54g$ PGA results in an average damage index of 0.10 and 0.05 for the two series of frames respectively. On the contrary, Vrancea and Kobe, which constitute the cyclic types of earthquakes, produced high damage at even low accelerations. It may be observed that the damage potential of the Vrancea record is higher than that of the Kobe record. However, Kobe was in fact more destructive because its actual PGA was four times higher than the corresponding one for Vrancea ($0.85g$ compared to $0.21g$). It may be noted that for the frames under investigation, the required PGA of the Northridge earthquake in order to result in a damage index 1 is nearly $0.6g$, a value very close to the actual acceleration of this record. This gives a confirmation of the proposed procedure.

5.2. *Influence of the fatigue resistance*

As outlined before, the fatigue lines are described by their slope and a point that corresponds to the rotation capacity under monotonic loading. The influence of the fatigue resistance of the structural elements to the damage of the overall frame is illustrated in Figs. 6 and 7.

Figure 6 shows the damage index of the frames for three values of the slope $m = 1, 2$ and 3. The rotation capacity is kept constant and is taken equal to $\varphi_{mon} = 0.05$ rad. When the slope is equal to $m = 1$, the cumulative law leads to an evaluation of the damage by algebraic summation of the plastic rotations. Values of $m = 3$ correspond to high-cycle fatigue lines which are valid in the range of 10000

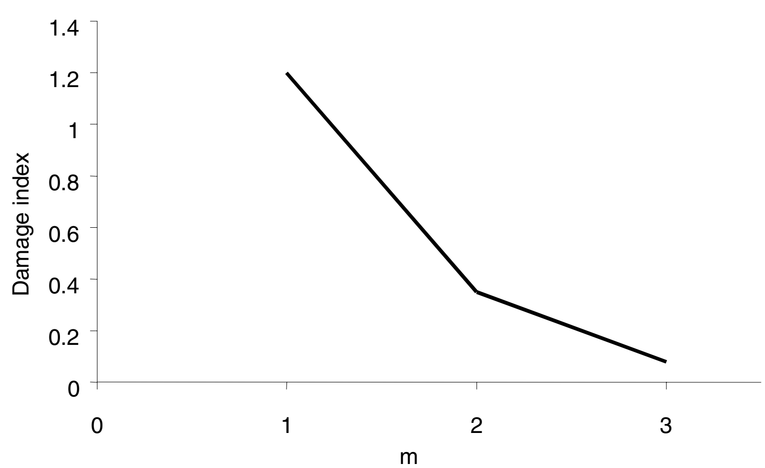


Fig. 6. Damage index for various slopes of the fatigue curve.

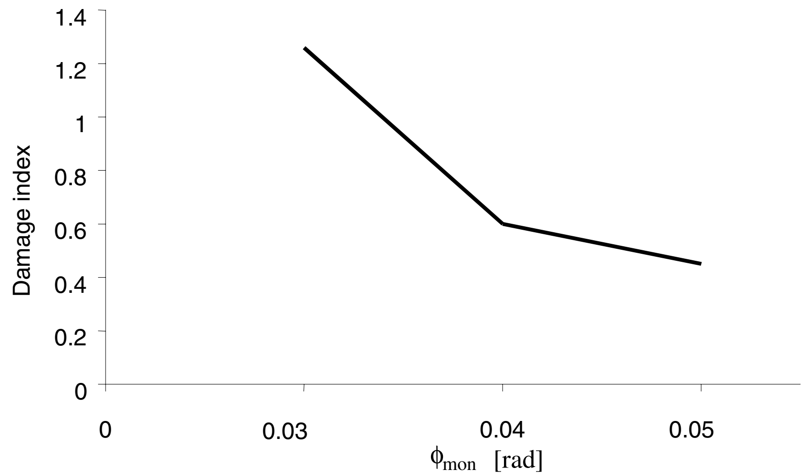


Fig. 7. Damage index for various rotation capacities.

to 10000000 cycles. Slopes with $m = 2$ appear to be more characteristic to low-cycle fatigue problems where only plastic deformations are considered. The other parameters being constant, a larger value of the slope indicates a better local fatigue resistance. Figure 6 shows that the overall damage of the frame decreases as the local fatigue resistance, expressed by higher values of m, increases. The benefits of a better fatigue resistance appear to be larger when the slope increases from 1 to 2 than from 2 to 3. This indicates that the implications in design will be not so much affected as long as the slope of the local fatigue resistance is higher than 2. On the other side it may be concluded that methods in which damage is evaluated by algebraic summation of the plastic deformations are rather on the conservative side.

Figure 7 presents the damage index of the frames for three values of the rotation capacity equal to 0.03, 0.04 and 0.05 rad. The slope of the fatigue resistance curve was taken equal to $m = 2$. It may be observed again that the frame behaviour is influenced by the local fatigue-resistance. However, the local fatigue resistance, expressed by the rotation capacity, and the damage of the frame are not linearly related.

5.3. *Influence of the level of vertical loading*

As stated before, the level of vertical loading, as introduced here, expresses the seismicity of the region. This level is high in low seismicity and low in high seismicity regions. To examine the influence of this parameter on the structural damage, two levels of vertical loading 40% and 60% were selected. Figure 8 presents, separately for beams and columns, the values of the damage index for 40% loading in relation to the corresponding ones for 60% loading. The results indicate that structural

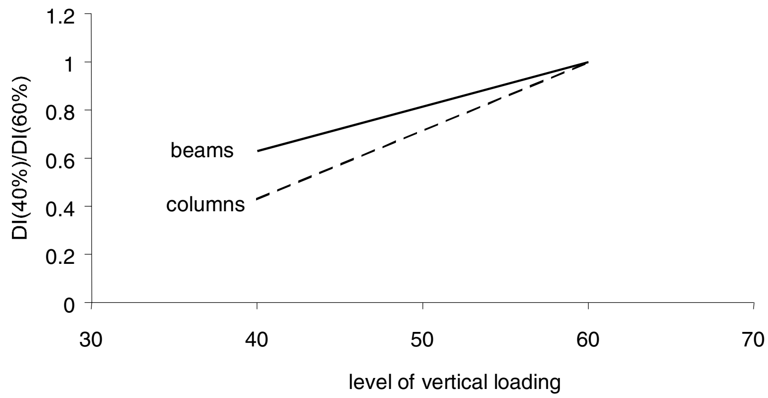
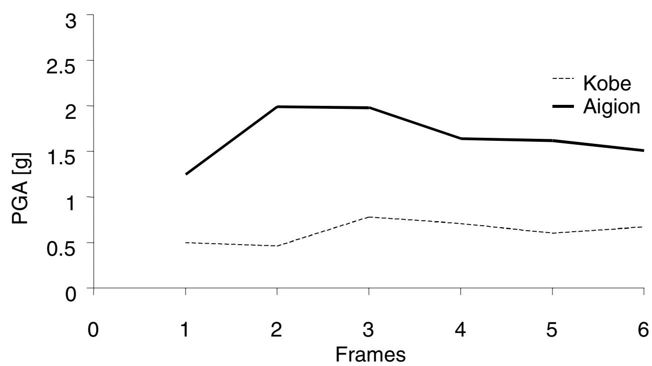
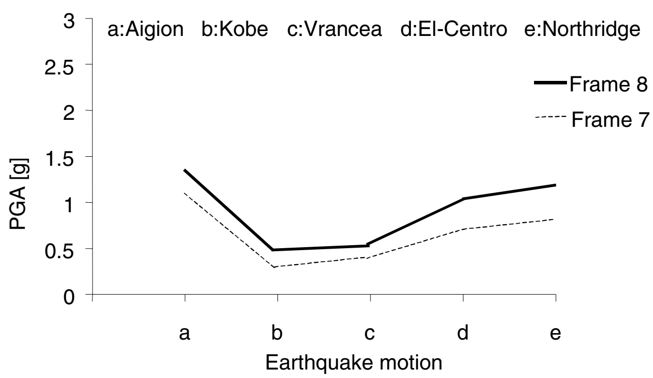


Fig. 8. Damage index for various levels of vertical loading.



(a)



(b)

Fig. 9. Damage index for various frame typologies.

damage decreases with decreasing level of vertical loading, and therefore when the structure is designed for higher seismic forces, it may also be seen that columns are more affected than beams.

5.4. *Influence of the structural typology*

For all the frames, the records were scaled with respect to acceleration until failure due to fatigue, expressed by a calculated damage index of 1, occurred. Figure 9(a) presents the limit accelerations for the Aigion and Kobe records considering rigid joints and 40% level of vertical loading. The curves show that the damage is associated to the characteristics of both the structure and the ground motion. However, for regular frames included in this study the differences in response do not appear to be very large. Evidently, the limit accelerations are much higher for the Aigion than for the Kobe record.

Figure 9(b) presents similar results for frames 7 and 8 for the five different ground motions taken into consideration. It may again be observed that damage depends on the structures' and ground motion characteristics. Both frames 7 and 8 reach a damage index 1 for the Northridge earthquake with about 0.6g PGA, which is almost equal to the actual ground motion value. This reveals again the effectiveness of the method proposed in estimating the damage.

5.5. *Influence of the joint flexibility*

As stated before, the joint, or rather connection, flexibility is introduced by means of appropriate rotational springs at the beam-to-column intersections. The flexibility is expressed by means of the spring stiffness as a percentage of the stiffness parameter

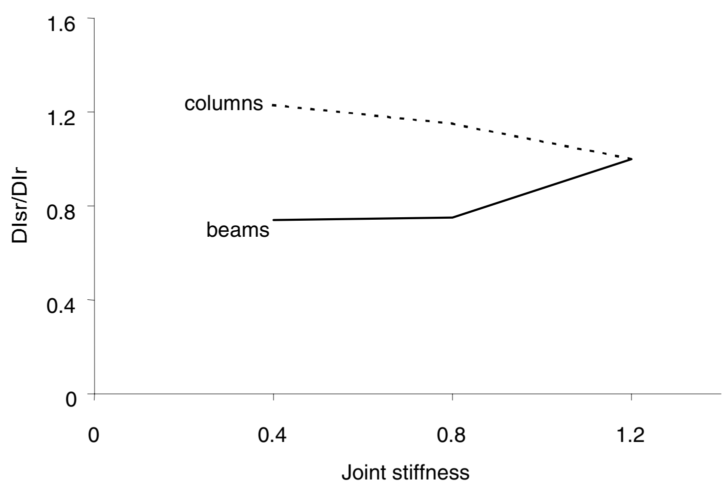


Fig. 10. Damage index for various frame typologies.

K of the connected beams (Table 2). The cyclic models for the connection response were identical, in order to study the influence of the flexibility only. Figure 10 presents, separately for beams and columns, the values of the damage index for semi-rigid joints related to the corresponding values for rigid joints. It may be seen that joint flexibility affects differently the beams and the columns. Flexible joints resulted more damage in the columns and less damage in the beams at their connection to the columns. This is due to the moment redistribution from the beams to the columns with increasing joint flexibility. However, the limit PGA at which fatigue failure occurred was generally little affected by the joint flexibility. It may be therefore concluded that for the frames investigated here the fatigue behaviour was not greatly affected by the semi-rigidity of the joints.

5.6. *Behaviour factors*

As well known, behaviour factors are defined as the ratio between the ultimate ground accelerations and the yield accelerations [Eurocode, 1994]. For the fatigue limit state, the ultimate ground accelerations are those at which the damage index is equal to 1. The yield accelerations are those at which first yielding occurs in the structure. The values of the behaviour factors of the different frames are shown in Fig. 11. The curves were derived for the Kobe earthquake, separately for the two levels of vertical loading. The behaviour factors are higher for 60% loading although the ultimate accelerations are lower for this level compared to 40% loading. This is due to the fact that first yielding appears at much lower accelerations for 60% loading so that the ratios increase. The results indicate that the behaviour factors have values within the usual limits proposed in codes (between 4 and 8).

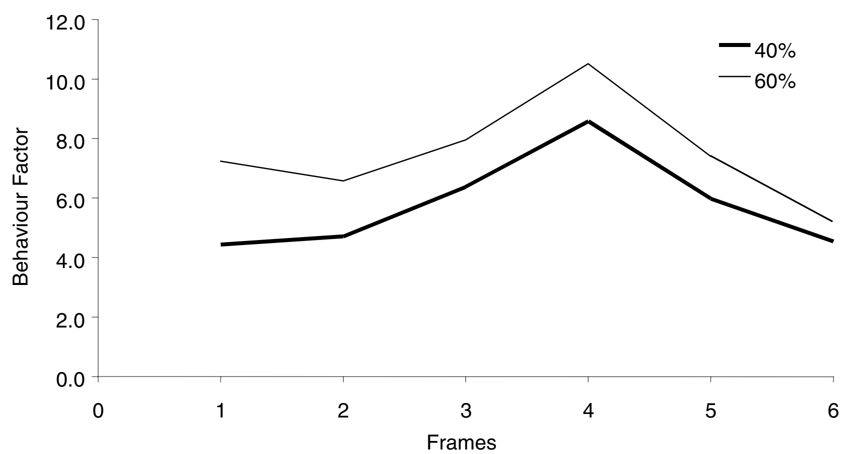


Fig. 11. Behaviour factors for various frame typologies.

6. Conclusions

A procedure for the fatigue assessment of steel building structures subjected to earthquakes was presented. The procedure constitutes an extension of the well-known (high-cycle) fatigue assessment methodology to cases of low-cycle fatigue. It may serve as a basis for the introduction of a fatigue limit state for the seismic design of steel structures as well as for the assessment of the damage occurred in buildings when subjected to specific earthquakes. By means of parametric studies the effects of various parameters on the fatigue resistance of buildings could be investigated. For the range of the parameters considered here following conclusions may be drawn:

(a) The peak ground acceleration — fatigue damage curves indicate a non-linear structural response and correspond to load-deformation curves.
(b) The fatigue damage is strongly influenced by the type of ground motion. Cyclic type, long duration motions result in more damage than impulse type motions.
(c) The overall structural damage may be reduced significantly if the local fatigue resistance of its elements is higher.
(d) Structures designed for low levels of seismic action are generally more susceptible to fatigue damage than corresponding ones for higher seismic loading.
(e) Semi-rigid beam-to-column connections lead generally to less damage in the beams and more damage in the columns. However, the overall damage is not highly affected by this rigidity.
(f) The behaviour factors determined on the basis of the fatigue resistance are of the same order of magnitude to those proposed in current seismic design codes.
(g) The application of the method requires for joints and members the knowledge of the relevant fatigue curves, as is the case in high-cycle fatigue, and must be therefore supported by additional experimental evidence.

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