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SOME THOUGHTS FOR THE PREDICTION OF THE LOCAL INELASTIC CAPACITY OF MRF SUBJECTED TO SEISMIC ACTIONS

Anthimos Anastasiadis*, Marius Mosoarca, Cristian Petrus** and Federico M. Mazzolani*****

* Geostatic, Geotechnical & Structural Engineering, Greece
e-mail: anastasiadis@geostatic.eu

** Politehnica University of Timisoara, Faculty of Architecture, Romania
e-mails: marius.mosoarca@upt.ro, cristian.petrus@student.upt.ro

*** University of Naples Federico II, Dept. Structures for Engineering and Architecture, Italy
e-mail: fmm@unina.it

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Abstract. *Even 20 years after the Northridge and Kobe earthquakes, which represented a benchmark action particularly significant for the design and construction of steel structures, the prediction of the available local ductility of steel members still remains an open issue. Current design codes do not provide a clear procedure to evaluate the rotation capacity of a member. In order to verify directly the available ductility, based on the capacity-demand ratio, such a procedure should be set-up. The paper emphasizes the framework for the definition of the local ductility, considering that the component elements and their joints belong to a structural frame. Accordingly, it is important to distinguish different levels of influence, namely the available ductility under different loading conditions, monotonic, seismic (near-field, far-field), as well as under the effect of the conceptual detailing (strong column-weak beam, strengthening, weakening at the joint region) and finally under a given structural behavior.*

1 INTRODUCTION

The severe Northridge, USA, 1994, and Kobe, Japan, 1995, earthquakes unveil the deficiencies in the design of steel framed structures that was considered, until then, as invulnerable. Moreover, fractures at the joint region without any sign of ductile behavior were observed. As a consequence the issue of ductility and particularly the local ductility regained a leading role in the seismic design, not only based on the material ductility but also to the section, joint and member level of inelastic deformation. A great research effort was performed all over the world (e.g. FEMA/SAC, USA, RECOS Project [1], Europe, E-Defense, Japan) focused on the investigation of the unexpected brittle damage and further providing methodologies to predict the ductility; however the issue of the local ductility is not clearly specified in the design codes. In each case the implementation of the non-linear design requires the direct verification of the ductility, based on the capacity-demand ratio.

With regard to current Eurocode 8 (2004) [2], Chapter 6-'*Specific rules for steel buildings'*, it prescribes some vague limits in order to verify the local and global ductility, although does not specify a clear methodology. Thus, sufficient local ductility is assured by limiting flange and web width-to-thickness ratios, however taken from Eurocode 3 (2005) [3], which is mainly a structural code for the design of structures under static loading conditions. But neither this classification is well specified due to the fact that in the classification parameters the influence of the span was not taken into account, Anastasiadis et al. (2012a) [4]. Therefore, the local ductility classes should be redefined. In a more advanced step, the correlation between local and global ductility (ductility class – q factor – local ductility class) should also consider the differences in the seismic action (near-source vs. far-source earthquakes) for both the local and global level as well as the available and required capacities of inelastic behavior, Gioncu & Mazzolani (2002) [5], Anastasiadis et al. (2012b) [6]. Moreover, concerning the prediction of the local ductility of beam-to-column connection, only some limits of plastic rotation of the potential plastic hinge were specified, without providing a methodology for the calculation of those limits. Finally, the dissipative-non dissipative concept should be updated with current trends allowing for the implementation of the strengthening (use of cover plates, ribs, etc) or the weakening (´´dog-bone´´connection) joint detailing that moves the plastic hinge away from the column face; in this case member ductility is critical and not the connection ductility. In order to develop a ductile design for steel structures, it is obvious that a discrete process considering all the levels of influence (material, cross-section, connection, joint, and member) should be defined.

The paper emphasizes the framework for the definition of the local ductility considering that the component elements and their joints being part of a structural frame. Accordingly, it is important to distinguish different levels of influence, namely the available ductility under different loading conditions, monotonic, seismic (near-field, far-field), under the effect of the conceptual detailing (strong column-weak beam, strengthening, weakening at the joint region) and finally under the structural behavior. Based on the aforementioned general considerations and also selecting a proper methodology predicting the local and global ductility, it is possible to make a step forward for the direct ductility based design, thus providing a limit state for the further development of the non-linear analysis and design under more stable conditions.

2 THE FRAMEWORK FOR THE DEFINITION OF THE LOCAL DUCTILITY

2.1 Local ductility as a function of the loading condition

Generally, in function of a loading condition we can distinguish the monotonic ductility (mainly used for plastic design) and the seismic ductility (mainly used for earthquake design). Moreover, the second one can be influenced by a predominant cyclic action (cyclic ductility) or by a velocity and / or impulsive action (strain-rate ductility). Traditionally, only the first type of ductility was considered, namely the cyclic ductility; however the seismic events that had been experienced on the last twenty five years demonstrated the differentiation, as a function of the action, which has a direct effect on the inelastic capacity of the whole chain of the ductility levels. For instance, concerning the local ductility the levels are: material, cross-section, connection and member ductility. The cyclic ductility is usually connected with the far-field earthquakes, while stain-rate ductility is related to near-field earthquakes. Far-field earthquakes are characterized by a long duration and a strong cyclic-repetitive motion, while near-field earthquakes are described by a short period with a long acceleration pulse also joined with a great velocity. Figure 1a illustrates the difference of the motion (e.g the Tsukidate NS component from the 2011 Great Japan earthquake as a far-field type vs. the well-known JMA-NS component from the 1995 Kobe earthquake), while in figure 2 one can observe the level of the force application between the different types of earthquakes. The aforementioned have a reductive effect on the local available ductility of steel frames.

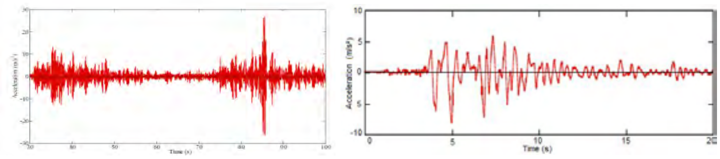


Figure 1. Far-field vs. Near-field earthquake type of motion.

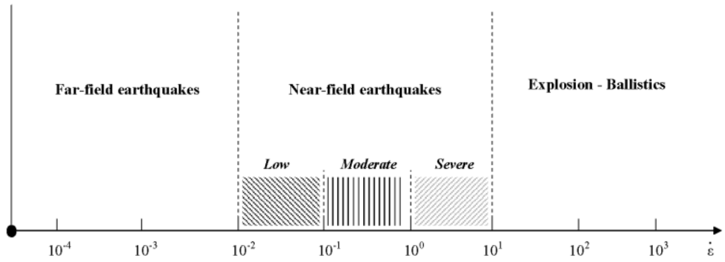


Figure 2. Strain-rate level for far and near field earthquakes.

The majority of studies have been performed with regard to the inelastic demands (e.g. drift demands), not considering explicitly the fact that the components of a steel structural system respond in a very different way when they stressed by a high repetitive action or by an impulsive motion. In any case both the available and the required ductility must consider the type of loading condition and further based on them we could verify the structure against the seismic action. From our studies, Anastasiadis et. al (2012b) [6], and a series of investigations that now are under review, as well as other recent research works, Hassouni et al. (2011) [7], D´Aniello et al. (2012) [8], Somja et al. (2013) [9], we can conclude about the very important differences in the post-elastic range of behavior; for the sake of brevity, those one are presented in tabular form, Table 1.

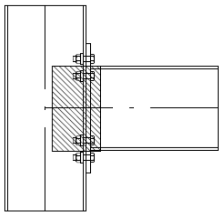
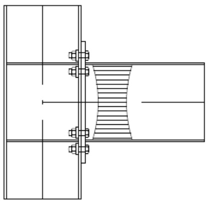
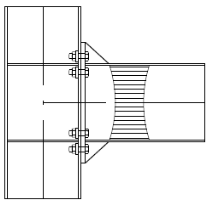
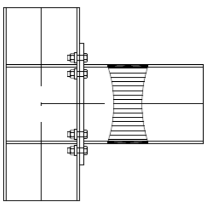
Table 1. Basic influences of the local ductility under different seismic loading conditions.

Local Ductility Levels	Far-field earthquakes	Near-field earthquakes
Material ductility (ε_y, ε_u, f_y, f_u, ρ_y)	Low loading velocity, Bauschinger effect, Random material variability	High velocity, strain-rate, rapid increasing of yielding strength as compared with the ultimate one, material embrittlement
Cross-section ductility (b_f, t_f, d_w, t_w)	Flange / web local buckling. Gradual cross-section stiffness and strength degradation	Fracture of flanges, no time for local buckling
Joint ductility (panel zone and the connection components) $\mu_{\theta,joint,av}$	Gradual stiffness, strength and ductility degradation of the component elements. Alteration of the SC-WB mechanism due to the additional strength of the slab	Fracture of welds and / or bolts. Alteration of the SC-WB mechanism due to the uncontrolled increase of the yield strength
Member ductility (b_f, t_f, d_w, t_w, L) R_{av}, $\mu_{\theta,member,av}$	The slab effect not permitting the buckling of the upper flange under the alternation of the moment action. Low cycle fatigue behavior (10-12 cycles)	The slab effect develops stress concentration at the lower flange. Ultra low cycle fatigue behavior (3-6 cycles)

2.2 Local ductility as a function of the conceptual design

According to the current seismic design, in case of steel framed structures, we can distinguish two types of conceptual conformation, namely, (i) the strong column-weak beam, SC-WB, and (ii) the weak column-strong beam, WC-SB. The joint detailing plays an important role showing the position where the potential plastic hinge will be formed (e.g. at the face of the columns in the connection or at some distance from the column flange), and additionally specifying the dissipative or the non-dissipative element. The prediction of the local ductility is strongly dependent with the local structural detailing; this means the proper selection between joint ductility and the member ductility. Current Eurocode 8 (2004) [2] specifies the ductility classes connecting the global behavior with the local one; however, the classification regarding local ductility should be replaced by the one considering the member and joint ductility as a function of the structural detailing. There is a stable basis in order to proceed for a further enhancement of the Eurocode 8, chapter 6, Gioncu & Mazzolani (2002) [5], Anastasiadis et al. (2000, 2012a,b) [4,6,10], Grecea et al. (2004) [11], Beg et al (2004) [12]. Table 2 briefly presents the aforementioned statements.

Table 2. Selection of the proper type of local ductility in function of structural detailing.

Constructional detailing	Design conformation	Type of the local ductility
	Weak column – strong beam (WC-SB) The dissipative zone is mainly located at the panel zone as well at the connection and under certain circumstances also in the column	Joint ductility (panel zone + connection deformation) + Member ductility (beam / column deformation)
	Strong column – weak beam (SC-WB) The dissipative zone is located in the beam (capacity design)	Member ductility (beam deformation)
	Strengthening solution of SC-WB concept by using ribs, haunches, cover plates. The dissipative zone is located in the beam (capacity design)	Member ductility (beam deformation)
	Weakening solution of SC-WB concept by reducing the beam flanges at some distance from the column face (“dog-bone”) connection. The dissipative zone is located in the beam (capacity design)	Member ductility (beam deformation)

In case of WC-SB the potential zone for the formation of a plastic hinge will be one of the following or a combination of them: the joint region (panel zone and connection), the beam at the column face as well as the column at the upper or bottom parts (the formation of plastic hinges should be avoided as possible and only at the base can be formed). Therefore, in order to assure a suitable level of ductility preventing the local brittle failures or structural collapse it should be checked both the joint and the member ductility. With regard to connection the Eurocode 8 (2004) [2] specifies the limits for the plastic rotation (e.g. 0.35 mrad for DCH, 0.25 mrad for DCM), however does not provide any methodology predicting the local ductility. As was mentioned earlier, concerning the member ductility there is not a clear definition, only the cross-section classification, which is inadequate, was provided by the code, Gioncu & Mazzolani (2002) [5], Anastasiadis et. al (2012a,b) [4,6].

A step forward in the Eurocode 8 is the implementation of the design and detailing of the strengthening and weakening concept in order to ensure a global plastic mechanism, avoiding the undesirable formation of plastic hinges in critical elements. Recent earthquakes reveal that only the application of the capacity design is not enough, e.g. due the slab effect, random material variability and exceptional-unpredictable loading, thus some conventional constructional techniques are absolutely necessary in order to ensure the initial capacity design. Therefore, a direct verification of the local ductility should consider the constructional detailing and depending on this one the type of the predicted local ductility (joint vs. member ductility) could be properly selected.

2.3 Local ductility as a function of the structural behavior

The steel moment resisting frames are composed by beams and columns, generally, rigidly connected forming frames of high, medium or low ductility classes, Eurocode 8 (2004) [2] (an equivalent of special, intermediate and ordinary MR-frames as defined in US practice). Considering the design of SC-WB frame a beam can be deformed in a different way depending on the loading conditions namely, the action of the vertical-gravity forces as well as the horizontal-earthquake forces.

Firstly, for the study of the local ductility it is necessary to assume that the element components belong to a frame. Hence in order to take into account the effect of the frame to the beams and /or columns the ´´standard beam´´ concept was proposed by Prof. Gioncu and further extended assimilating the deformational behavior, Fig. 3, Anastasiadis & Gioncu (1999) [13]. In this manner the influence of gravity loads and the level of severity of the horizontal loads were introduced, improving the traditional concept of the three point beam.

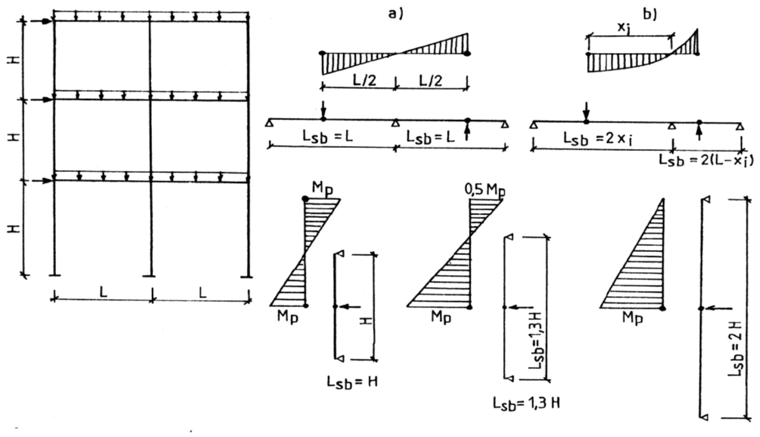
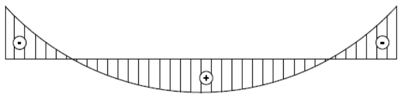
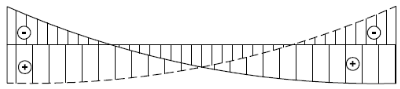
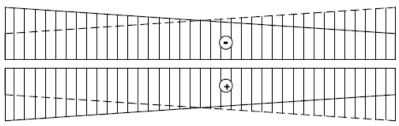
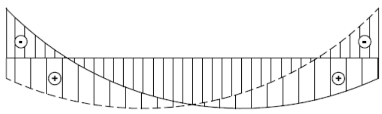


Figure 3. Definition of the standard beam as a tool to introduce the frame effect on a member.

Secondly, for the complete framework definition of the local ductility one can consider the type of deformation and the predominant action, Table 3. Thus, the predominant action dictates the type of local ductility that can be calculated (monotonic vs. seismic ductility). Therefore, in case were the gravity forces are predominant and /or in case of low earthquake action the monotonic plastic ductility could be prevailed. In case were the effect of earthquake forces are very important the seismic ductility gets the control and in function of the earthquake region could be of the far-source (cyclic) or near-source type (strain-rate).

Table 3. Selection of the proper type of local ductility in function of structural detailing.

Type of deformation	Observations
	Cases were the gravity forces control the behavior. The level of gravity forces are high combined with a long span. The local ductility could be determined by the plastic monotonic one.
	Cases were the earthquakes forces control the behavior. The local ductility could be determined by the seismic one. Plastic hinges are formed at both ends.
	Cases with exceptional severity of earthquake forces, also in case with high level of seismic forces and short beam span. There is no point of inflexion. Plastic hinges can be formed initially at one end; during the evolution of the seismic action also it is possible to be formed a plastic hinge at the other end.
	Cases with high level of gravity and earthquake forces, where the potential plastic hinges can be formed at the end as well at the mid-span. During the seismic loading a third plastic hinge it is possible to be formed, at the other end, transforming the beam into a collapse mechanism

3 DESIGN FORMAT FOR A DIRECT DUCTILITY BASED DESIGN

For a safe design against earthquake action the stiffness, strength and ductility must equally be verified. The first two criteria are well defined; however the third one for the majority of the current codes of practice is under discussion being an open topic. A huge effort has been made, from the early work of Baker, regarding the plastic design, Driscoll and Lay & Galambos, focused on the rotation capacity, until the last 25 years of experimental and theoretical research, studying the seismic ductility, although we have not obtain a direct verification of the local and global ductility. The transition from the force based design to the deformational based design (displacement and rotation), or displacement based design as usually marked, requires the design control of the available local ductility, where the potential beam plastic hinge posses sufficient inelastic capacity allowing the development of a suitable global plastic collapse mechanism. Without such a check it is not possible to be assured a predetermined and desired dissipation mechanism. Therefore, a codified framework requires the definition of both the global, local and their interaction; however the paper is focused only to the second one.

The main difficulty is to determine the local available ductility under seismic conditions; hence due to the fact that the monotonic ductility is relatively well studied and by using correction factors that take into account the reductive effect of the seismic action, we can obtain, for design purposes, the following conceptual relationship:

Local Available Seismic Ductility = (Correction Factors) x (Available Local Monotonic Ductility) (1)

The prediction of the available monotonic ductility can be made by using the local plastic mechanism methodology initially developed and further extended by the Prof. Gioncu and his collaborators, Anastasiadis & Gioncu (1999) [13], Gioncu & Mazzolani (2002) [5], Anastasiadis et al (2012) [4,6], while the correction factors will be of different nature such as constructional (hot-rolled vs. welded sections) and loading (near-source vs. far-source). Moreover, the seismic influence on the local monotonic ductility is under investigation by the team. According to the current philosophy of the ultimate limit state and the format of the Eurocodes we can provide:

$$\text{Seismic Required Ductility, } D_{req} \leq \text{Seismic Available Ductility, } D_{ay} \tag{2a}$$

$$\gamma_{req}\,(\gamma_{fs}\,;\,\gamma_{ns})D_{req}\,(D_{req.fs}\,;\,D_{req.ns}) \;\leq\; \frac{D_{av}(D_{av.cyc.};D_{av.str})}{\gamma_{av}(\gamma_{cyc.};\gamma_{str.})} \tag{2b}$$

where the required ductility could be evaluated directly from a time-history analysis, push-over or simplified relationships, while the available one by using a proper software (e.g DuctRot or other specialized computer program) or by simplified relationships suitably adjusted taking into account the seismic action, Anastasiadis & Gioncu (1999) [13], Anastasiadis et al (2000) [10], Gioncu & Mazzolani (2002) [5]. Moreover the available and the required ductility could be evaluated in terms of the ultimate rotation, θ_u, or in a non-dimensional format given by the rotation capacity, as a ratio of the ultimate to plastic rotation, $R = (\theta_u/\theta_p)-1$. In any case also the drift as well as the roof displacement could be used independently or in combination with the aforementioned in order to control the deformational capacity. Therefore, $D_{av} = (\theta_{u.av}; R_{av}; \mu_{\theta.av}; \mu_{\delta.av})$ and in the same direction $D_{req.} = (\theta_{u\cdot req}; R_{req}; \mu_{\theta.req}; \mu_{\delta.req})$ could be defined. Focused on the available ductility we can distinguish the cyclic ductility, $D_{av.cyc}$, as well as the strain-rate ductility, $D_{av.str}$. In this direction and accounting for the conceptual relationship (1) as well as the main earthquake detrimental effect, the following could be defined:

$$D_{av.cyc} = (\text{factors introducing the cyclic effect}) \times D_{av.mon} \tag{3a}$$

$$D_{av.str} = (\text{factors introducing the strain-rate effect}) \times D_{av.mon} \tag{3b}$$

With respect to safety factors the γ_{av} should be determined taking into account the cyclic and strain-rate effect, while the γ_{req} could be evaluated taking into account the global frame behavior, the local soil conditions and also the characteristics of the action as defined from the far-source, γ_{fs}, and near-source, γ_{ns}, earthquake motion. A conceptual way in order to evaluate the local available ductility under the different action of earthquakes is presented in Table 4.

Table 4. Steps for the evaluation of the seismic available local ductility.

Determination of the "standard beam" introducing the frame effect	
Prediction of the local available ductility under monotonic conditions	
Factors influencing the local ductility	
Constructional, geometrical	Loading (near, far source earthquakes)
Prediction of the available local ductility under seismic loading conditions	
Safety factors	Correction factors
Local available seismic ductility	
Cyclic available ductility (*Cyclic* effect)	Strain-rate available ductility (*Strain-rate* effect)

4 CONCLUSIONS

The prediction of the local ductility is still an open topic, not clearly defined in the current codes of practice. Actually, the difficulties rise from the inherent variability of the loading, the material and the geometric parameters that can be evaluated in a post-elastic range. The experimental and theoretical background cumulated, particularly, in the last twenty five years creates the condition for a straightforward verification of the local ductility of steel moment resisting frames. The paper evidenced the framework under which should be worked in order to obtain a ductile design based on the inelastic capacity of the element components. Of paramount importance is to recognize the different effect on the available deformational capacity that has an earthquake of near-source with impulsive characteristics with very few inelastic cycles against a far-source earthquake with much more cyclic action, within the inelastic range, and longer duration. Future editions of the codes (e.g. EC 8) must clearly specify, the joint and member available ductility, based on the past existing experimental and theoretical work. Furthermore, having in mind all the aforementioned, new experimental protocols should be defined, after extensive time-history analysis with the corresponding structural systems and accelerograms, in order to introduce both the main effect of the earthquake as well as to distinguish between low and high seismicity for any action of prequalification.

ACKNOWLEDGMENT

This paper is dedicated to Professor V. Gioncu who passed away on March 2013 and along with Prof. Mazzolani initiated the STESSA Conference early on 1994, in order to gather all the ideas regarding the design of steel structures in seismic areas. His contributions in the field of stability and ductility were recognized from the international engineering community by numerous publications and honors and further on we feel the need to acknowledge him for his warm friendship and not only scientific but also practical ideas shared with all of us.

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