Structural integrity of buildings under exceptional earthquakes

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ABSTRACT: The paper tries to identify the effect of exceptional earthquake loading on Civil Engineering structures. Several reasons leading to exceptional action are identified and their effect on the structures is discussed. The main problems and the strategies to address them are summarized. The paper summarized the latest developments concerning the characteristic factors affecting the structural integrity and its evaluation. Three different cases are considered depending on the specific structural material (concrete, steel, masonry). Finally, conventional and non-conventional methods for the optimization of the structural response under the exceptional earthquake action are summarized.

1 INTRODUCTION

An exceptional action is an action that has not been considered in the initial design of the structure. Although, in principle, this condition should be avoided, it is a fact that in many cases the structures are asked to undertake larger forces than the ones for which they were designed. The reasons for that are connected with the character of the action (snow, wind, live load, seismic load) but also with the structural system itself, as it will be explained later. The paper tries to summarize the reasons leading to exceptional seismic action on structures and to give some material dependent solutions to the structural deficiencies that might occur.

The first part of the paper contains material introductory to the notion of exceptional seismic action on structures. The reasons than can lead to seismic loading not taken into account during the initial design process are summarized. The results of the exceptional action are identified and general procedures for the evaluation of the structural integrity are given. This part summarizes also the general methodologies used for the optimization of the structural response.

The next sections are related to specific materials (concrete, steel, masonry). Material related reasons leading to poor structural response are presented as e.g. poor detailing in concrete structures, brittle failure of concrete elements (shear), poor moment-resisting connections in steel, brittle fracture of welds in steel, combination of high stiffness and low strength in masonry structures, etc.. Also, material related procedures for the anticipation of the problems introduced in the first part of the paper are presented, as e.g. specialized procedures for the optimization of structural performance under earthquake action.

2 REASONS LEADING TO EXCEPTIONAL EARTHQUAKE ACTION ON STRUCTURES

The case of earthquake forces on structures is a rather characteristic case where an action can be exceptional. It is admitted that there exists a high probability that the value of the seismic forces will at some time exceed the value prescribed in the design. This fact is related to the inherent uncertainty nature of the seismic action but also to incomplete or inadequate knowledge of the structural behavior at the time of the design of the structures. In the following, several reasons for which a structure might be subjected to exceptional seismic forces are identified.

A very large number of structures were designed with a regulative framework, which today seems to be insufficient. This is for example the case in Greece, where the lateral forces prescribed by the later seismic codes are significantly higher than the ones prescribed by the seismic codes before the year 1985. Of course, this happens because, in the meantime, significant scientific knowledge has been embodied in the seismic codes. This knowledge has to do with the scientific field of seismology but also with the comprehension of the behavior of the structural materials and their combinations. For example, during the last 20 years, a huge number of tests have been performed in concrete, steel, composite, aluminum and wood specimens that have allowed the development of rational procedures for the calculation of the strength and the ductility of the various structural elements.

Also, due to the developments in both hardware and software that took place during the last 20 years, the design engineers are capable to perform more sophisticated seismic analyses, something that has also been reflected in requirements of the seismic codes.

From the viewpoint of the seismology, it is well known that it is difficult to obtain exact values for the seismic actions. But recently, there have been recognized several reasons for which the design forces should be increased in certain cases. For example, during the past 20 years, a significant number of recorded strong motion data has indicated that the characteristics of the ground motion vary significantly between recording stations. This phenomenon is magnified for stations located near the epicenter. As a result, two main regions with different types of ground motions are considered (Gioncu et al 2000).

- The near-source region that is the region within few kilometers of either the surface rupture or the projection on the ground surface of the fault rupture zone.
- The far-source region situated at some hundred kilometers from the source.

Unfortunately, the characteristics of the design spectra and the design methods adopted by the majority of the seismic codes have been based on records obtained by far-source fields. Therefore, they are incapable to describe the seismic intensity in the near-source region. Lately, the so called "near source factor" has been introduced in some advanced seismic codes in order to take into account the amplification of the earthquake forces in the near-source region (ATC-40 1996, SEAOC 1999). Moreover, the vertical component of the seismic action in near-source field could be greater than the horizontal ones. Notice that, until now, this vertical component is only in special cases taken into account in the design of buildings. Also, in near-source areas, due to the very short periods of the ground motion and the pulse characteristics of the loads, the significance of higher vibration modes increases. Due to the pulse characteristics of the actions, developed with great velocity and especially due to the lack of restoring forces, the ductility demands could be very high.

All the above have been dramatically verified after the Kobe earthquake. The observations after this earthquake show, by both ground motion recordings and observed damage to buildings, that earthquake loading conditions in the near-source region subject buildings to more severe conditions than previously assumed.

Another aspect of the seismic design whose significance has been recognized only during the last decades is connected to the ground conditions. It is now well known that the properties of the site soils affect the intensity of shaking that can be expected at the building site. Various parameters such as the thickness of the soft and stiff soil layers, the shear wave velocities of the rock and soil layers, the soil/rock impedance ratio, the layering properties of the soil layers etc. influence the amplification or attenuation of the seismic action on the structures. Moreover, landsliding, liquefaction and surface fault rupture has led to catastrophic results on buildings during the last decades and their significance has been only recently introduced and quantified in some seismic codes (ATC-40 1996).

Another reason leading to exceptional accelerations on structures (i.e. accelerations greater than the design ones) is connected with magnification that sometimes occurs in the short period range. This was for example the case in the earthquake that hit Athens in the September of 1999. This was a moderate intensity earthquake (5.9 on the Richter scale) that resulted in several collapses and deaths and a lot of damages in a large area. The accelerations recorded in various neighboring sites led to acceleration spectra, which presented a peak in the short period range (see Figure 1). This peak acceleration occurred in a period of 0.25 sec and was about 2 to 3 times greater than the design one for the specific site. As a result, a lot of rather stiff and limited ductile structures (low-rise buildings, some of them designed according to the latest seismic code) were significantly damaged. On the other hand, high-rise buildings with a dominant period greater than 0.4 sec appeared no damages at all.

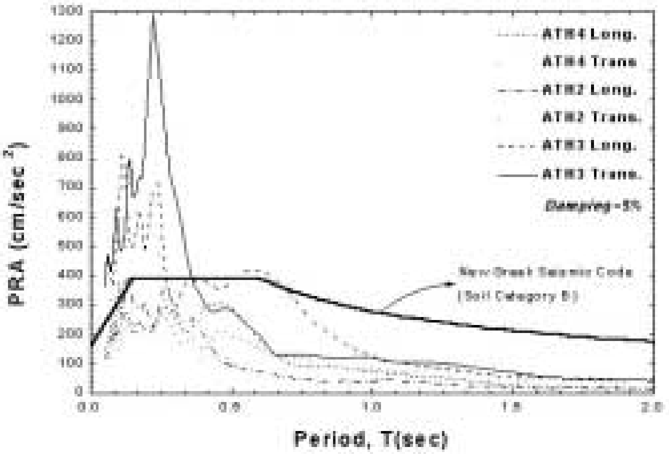

Figure 1. Acceleration response spectra (horizontal components) of the September 7, 1999 earthquake in Athens and elastic design spectra of the Greek Seismic Code.

The distinction of the seismic zones defined in the seismic codes is based on historical earthquake data. However, various difficulties arise concerning the determination of the faults that gave significant earthquakes during the historical ages. The most important difficulty is connected with the existence of extensive sea-covered areas in active seismic areas. In these areas, the faults that generate earthquakes cannot be explored with the usual geological methods. Another one is based on the fact that the biggest earthquakes in Europe are not very strong (usually less than 7.0 in the Richter scale) and for this reason, the surface indications of the faults are relatively small and therefore hardly recognizable. Therefore, some faults or the potential of existing faults for the generation of strong earthquakes, are identified only after giving new, relatively strong earthquakes. This was for example the case in Greece where a strong earthquake (Kozani-Grevena, 1995, 6.6 in the Richter scale) occurred in a region characterized in the lowest earthquake hazard zone. After this event, this area was classified in a higher seismic hazard zone, therefore, all the structures build before 1995 in this area may be subjected in an exceptional earthquake as they were designed for a lower value of the probabilistic acceleration.

It should be also realized that seismic design philosophy has dramatically changed in the past decade. Most traditional codes have been concentrated on life save performance during large, infrequent earthquakes, accepting significant damage as long as the collapse is prevented. However, there has been growing evidence (e.g. Northridge and Kobe) that such, frequently uncontrolled damage is not tolerated by the society. Consequently, new performance objectives demanding operational structures after fairly large earthquakes impose totally new requirements that have been seldom or never considered in the traditional design.

Except the above, there are also reasons more closely connected to the structural system, for which a structure might be submitted to an exceptional earthquake action. The most important is the fact that after moderate earthquakes, some damage occurs in the structure. In the case that some elements are significantly damaged, a procedure for repair is usually initiated. But, in the case that the damage is hidden or underestimated, the structure remains with some of its strength reduced; therefore, the next seismic motion will find it in a condition not considered in the design. Usually, this kind of damage is mainly connected with the degradation of stiffness. Studies that have been performed during the last years on this topic (FEMA307 1998) indicate that the main effect of prior damage to the seismic response of a structure is the increase of the displacement (and therefore the ductility) demands. However, it was found that a relatively limited prior damage does not play a dominant role on the overall seismic response.

Finally, arbitrary change of the usage of the structure should be mentioned as a case where the structure is submitted to forces for which it has not been designed. For example, a building designed for usual importance and later used as a hospital, may not meet the advance performance requirements for medical buildings. Also, the conversion of the floors of an ordinary building to storage compartments may lead to exceptional seismic action on it.

3 EVALUATION OF THE STRUCTURAL INTEGRITY

Cases as the ones described in the previous paragraph can be identified in almost any seismically active area and there is a strong need for some measures against this unfavorable situation. In this respect, a close examination of the situation is needed to identify the reasons leading to the possible exceptional seismic action on the structure. In general, the treatment of such a case requires the steps described in the following.

The first step is connected with the determination of the seismic action. As it has already been mentioned, modern seismic codes provide information about the consideration of the various unfavorable effects. For this reason, data on soil surface and subsurface conditions at the site shall be obtained from existing documentation, or from a program of site-specific subsurface investigation. In any case, the seismic hazard should be identified and quantified using the latest scientific knowledge.

In the second step, information should be gathered about the specific structure. A meaningful structural analysis of the probable seismic behavior of the building requires good understanding of the existing components and their interconnection. Therefore, the configuration of the structural system, as well as the type, detailing, connectivity, material strength and condition of the structural elements comprising the building shall be determined. It must be noticed that the strength and deformation capacity of existing components should be computed based on derived material properties and detailed component knowledge. As the conclusion that an existing structure does not meet some specified criteria can have considerable consequences, it is important that the evaluation is based on the best available information of the properties of the materials and the components, rather than on very conservative assumptions. Existing component strengths must be determined for two basic purposes: to allow calculation of their ability to deliver load to other elements and components, and to allow determination of their capacity to resist forces and deformations.

Finally, an analysis of the building should be performed in order to determine the forces and the deformations induced in the various structural components by the seismic action. Here, the following methods should be used.

- Static or dynamic elastic analysis. These are the most classical methods of analysis, however, concerning the analysis of existing buildings, various problems may occur. Although an elastic analysis indicates where first yielding will occur, it cannot predict failure mechanisms and account for the redistribution of forces during progressive yielding. For this reason, the results of the linear procedures can be very inaccurate when applied to buildings with highly irregular structural systems, unless the behavior of the building is nearly elastic.

- Static nonlinear analysis. This is generally a more reliable approach for the assessment of the behavior of a structure than are linear procedures. However, it is not exact, and cannot accurately account for changes in dynamic response as the structure degrades in stiffness or account for higher mode effects.
- Dynamic nonlinear analysis. This is the most accurate analysis, however, it is at present extremely time consuming. It involves time history analysis of a three-dimensional mathematical model using simultaneously imposed consistent pairs of earthquake ground motion records along each of the horizontal axes of the building.

It should be emphasized that the inelastic analysis procedures should be preferred as they permit the comprehension of the behaviour of the buildings, the identification of the failure modes and the quantification of the potential for progressive collapse. However, it must be noticed that they should be applied only by engineers experienced in nonlinear analysis. As a balance between simplicity and applicability the static nonlinear analysis is promoted by various codes (FEMA273 1997, FEMA274 1997, ATC-40 1996, EC8).

Finally, the results of the analysis should be compared with the calculated force and deformation capacities of the structural members in order to verify the structural integrity under the considered seismic action. For this reason, the performance level should be defined, i.e. the requirements of the engineer should be quantified (see also the article "General methodologies for evaluating the structural performance under exceptional actions" in the same Volume). Depending on the results of this comparison, the following main deficiencies may be identified.

- Lack of stiffness
- Lack of strength
- Lack of ductility and dissipative capacity.

The following measures can be understood as a general description of the options provided to the design engineer for the optimization of the structural behavior, according to the deficiencies identified by the analysis procedure.

- Strengthening and stiffening. These are the most common methods for the enhancement of the seismic behavior of structures. They are applied mainly in cases where the lateral force resisting system is inadequate. The strengthening results to higher values of the lateral force that initiate damage events in the structure. However, the elastic characteristics of the structure do not change significantly. Stiffening changes the fundamental period of vibration. The modes of vibration might change as well. System stiffening and strengthening are in most cases performed as concurrent strategies, since most systems that will strengthen a structure also simultaneously stiffen it. Similarly, most stiffening techniques usually increase the strength as well. Typical systems used for strengthening and stiffening include the addition of new elements such as shear walls, braced frames, buttresses, etc. and also the strengthening of existing elements, e.g. column jacketing, strengthening of beams through external reinforcing materials (steel or composites), etc.
- Improvement of the deformation capacity. This method can be applied in order to improve the seismic performance of systems that include brittle elements. Methods of enhancing deformation capacity include the addition of confinement to existing elements (through shear jackets or fiber reinforced polymeric materials), strengthening of columns to avoid soft stories, etc. This methodology is mainly applied for the optimization of the behavior of concrete buildings and is effective when the required modifications involve a relatively small number of elements.
- Base isolation. This approach requires the insertion of flexible bearings at a single level of the vertical load carrying system of the building. Typically, these bearings are placed near the base and are designed to have relatively low stiffness and extensive lateral deformation capacity. In some cases the bearings may also have superior energy dissipation characteristics. The installation of an isolation system results in a substantial increase of the building's fundamental response period and subsequently to a dramatic reduction of the seismic forces applied on the building. As the isolation bearings have much greater flexibility than the structure itself, the seismic deformations tend to concentrate on the positions in which they have been installed. Together these effects result in significantly reduced deformations on the part of the building above the level of the isolation bearings. Base isolation has most commonly used in the past for the optimization of the seismic performance of historical buildings, because it usually allows the substantial reduction of the interventions in the superstructure.

  There are several base isolation systems, which are based on: viscous fluid dampers, sliding bearings, lead rubber bearings, sliding bearings with viscous fluid dampers, elastomeric bearings with controllable fluid dampers, etc. In the paper of Chang and Makris (2000), the efficiency of these dissipative mechanisms has been investigated. An alternative to these classical solutions is the use of the shape memory alloys (SMA). SMA's are special alloys, which can be made of various metals: Copper, Zinc, Aluminium, Nickel, Titanium, etc.). The new system based on this material is capable to provide a wide range of performances (Dolce and Marnetto 2000). However, despite this advances offered by base isolation, in recent years the actual efficacy of this technology has been also questioned by several seismologists, essentially because base-isolated buildings result to be vulnerable to large pulse-like ground motions generated at near-fault locations, due to the large displacement demand occurring for isolators. In order to reduce these displacements, supplementary dampers have been suggested. Nonetheless, existing results show that the use of supplemental dampers in seismic isolation is a misplaced effort and alternative strategies to solve the problem should be implemented (Kelly 1999).

- Addition of supplemental damping through energy dissipation systems. These systems increase the ability of the structure to dampen the earthquake response through either viscous or hysteretic damping. This approach requires the installation of energy dissipation units within the lateral force resisting system. These units dissipate energy and therefore, reduce the seismic displacements of the structure. The installation of these units often requires also the installation of vertical bracings to serve as a mounting platform for the units and therefore, typically results in a simultaneous increase in system stiffness. Energy dissipation systems typically have a greater cost than conventional systems for stiffening and strengthening but have the potential to significantly enhance the behavior of the structures.

4 REINFORCED CONCRETE

Engineered reinforced concrete (RC) buildings are in general considered earthquake resistant in comparison with some traditional structural systems. Unfortunately, many past events (e.g. Figure 2) have warned that this should not be taken as granted. Heavy damage of RC structural systems occurred since the demand was greater than expected and/or the structural capacity and integrity was not sufficient. Both factors are briefly analyzed below.

While all general conclusions regarding the reasons leading to exceptional seismic action on structures (Chapter 2) apply, some additional considerations relate to RC buildings.

- Design seismic action and related seismic force reduction (behavior) factors have been more or less empirically determined for the average and reasonable regular representatives of typical RC structural systems (like reasonably regular cast-in-situ frames for example). However, it should be realized that for various reasons many buildings do not possess the anticipated average dissipation capacity, which has been traditionally considered in the design. Therefore larger (exceptional) seismic forces should be used to verify such buildings.

Figure 2. Total disintegration of a RC frame (Kocaeli earthquake, 1999)

- This is particularly true for specific or less investigated existing systems (i.e. frames with infills, prefabricated industrial buildings and even structural walls) as well as for systems not designed for earthquake loads in the past. An example of numerous collapses of limited ductile infilled frames during the Kocaeli/Turkey earthquake is illustrative. Apparently regular structures lost the infills in the lower stories at the very beginning of the earthquake (Figure 3). Consequently they behaved like soft-story frames, which was never anticipated in the design (Dolšek, 2001). Such increased local ductility demand could not be withstood with very poorly detailed columns.
- Even more so, such considerations apply for innovative systems, where no empirical evidence exists. Special studies should be made for such systems to evaluate their dissipative capacity and related level of the design earthquake loading.
- It should also not be forgotten, that typical values for seismic force reduction factors (behavior

factors) of RC structures are based on the anticipated global ductility in the range of approximately 3 to 6. The related local ductility demand may well exceed the number of ten. Such large deformations of RC members are inevitably associated with large cracks and severe damage. There has been growing evidence that such damage is not tolerated by the society. This actually implies that larger (exceptional) earthquake action than the one considered in traditional design should be used to verify if the damage is within the acceptable limits.

Figure 3. Soft story mechanism formed after the fall out of the infills in the lower two stories (the situation never considered in the design)

Fortunately, most well designed RC buildings posses enough dissipative capacity and overstrength that they maintain their structural integrity even in the case of exceptional earthquake loading. For example, most buildings in Kobe, built after 1981 survived well the earthquake, which was much stronger than anticipated.

Figure 4. Combined shear – axial compression failure due to the lack of confinement

In short, structural integrity of RC structures depends on two compatible yet very different materials - ductile steel and brittle concrete, as well as on the bond between them. In the case of earthquake loading, structure's integrity can therefore be ensured if (a) favorable global mechanism develops, (b) ductile reinforcement governs the response of elements, (c) bond between the two materials is maintained even in the case of repeated cyclic loading.

On the other hand, in addition to the bad conceptual design (i.e. soft story), which is a common cause of failure for all structural systems and materials, the most typical causes of structural disintegration in RC structures are shear failure, lack of confinement (Figure 4), and bond failure in anchorage zones (i.e. in beam-column joints; Figure 5). Other examples can be found in EASY – Eartquake Engineering Slide Information System (Fischinger, 1997).

The question arises what analysis is needed to identify such risks in the case of exceptional earthquake loading? It is clear that standard elastic analysis based on the behavior factor as a single empirically defined parameter to account for seismic energy dissipation, is not capable to provide an adequate answer.

Figure 5. Disintegration of a poorly detailed beam-column joint

Still in the range of elastic analysis, the capacity design procedure offers a viable solution, ensuring that ductile flexural response is achieved by inhibiting non-ductile deformation modes. Most typical application in RC structures is to preclude shear failure by suitable design and detailing of ductile flexural elements. Since the shear demand (in general any demand on the non-ductile mechanism or element) is limited below shear capacity regardless the level of the earthquake loading applied, this method seems to be perfect in the case of exceptional action. However, the problem is to know, if the capacity of the protecting mechanism (i.e. the rotational capacity of the RC beam) is large enough to survive the exceptional action. Very few countries possess a suitable data-base of their specific structural systems to solve this problem.

Therefore the most recent solutions in the advanced codes have moved towards new inelastic design methodologies. However, although relatively simple static (push-over) methods have been introduced, the question arises if at the present the modeling of inelastic behavior of RC structures is good enough to serve the purpose. Even in the pre-yielding stage the stiffness of the cracked section is difficult to evaluate, not to mention the complex behavior of confined concrete and bond-slip mechanisms in a post critical stage near failure.

In principle, two major groups of models exist. Micro models analyze inelastic stress-strain response of (confined) concrete and steel and are sometimes considered as "exact" models. Macro models consist of a finite number of discrete springs following prescribed force-displacement relationships. Which group is to be chosen, depends on the specific problem. Among plentiful analyses, it is worth to mention the international benchmark study CAMUS (Combescure, 2000). The blind prediction of the seismic response of a RC cantilever wall was made for four consecutive earthquakes, among which at least two could be considered as exceptional. Eleven participants of the benchmark study predicted the response with varying success. A number of predictions can be considered (surprisingly) good. In general micro models were somewhat more successful (if the problems like local failure criteria and bond slip were adequately addressed). However, some predictions with macro models were also good or at least acceptable (Fischinger, 2002), giving hope that such models could serve the purpose in the case of more complicated real buildings.

5 STEEL STRUCTURES

5.1 *Characteristic factors affecting the structural integrity and its evaluation*

5.1.1 *General*

Also in case of steel structures, seismic events recently occurred worldwide produced unacceptable damages and economical losses, destroying a number of buildings and bridges (De Matteis et al. 2001). In particular, the 1985 magnitude 7.3 Michoacan (Mexico City) earthquake evidenced the first collapse of an important high-rise steel building. Also, in magnitude 6.7 Northridge (Los Angeles) earthquake more than 100 low- and high-rise steel buildings suffered unacceptable damages and exhibited fractural failure modes. Similarly, in the magnitude 7.2 Hyogoken-Nanbu (Kobe) earthquake severe damages were detected in a number of modern steel constructions. Two main remarks have to be pointed out: (1) Most of such structures were built within the previous 10 years and therefore were designed according to recent design and constructional methods for seismic-prone Countries; (2) Damages interested a number of welded beam-to-column connections of moment resisting framed buildings, emphasising poorness of basic material and detailing.

In the whole, these events clearly shown that even a high-ductile material like steel could suffer brittle failure modes and something in customary design assumptions and construction technique had to be revised. Akiyama (2000) attributed the causes of the above damages to a combination of the following factors: poor quality of steel, poor welding, error in estimate of inelastic deformation capacity and excessive seismic input. In other words it could be concluded that the natural hazard related to earthquake was too high, high intensity earthquake actually, representing an exceptional loading condition for current steel building and bridges. This is particularly true for near-source earthquakes due to the effects of strain rate and superior vibration modes, which are not correctly taken into account by the present design codes (Gioncu and Mazzolani 2002). As a matter of the fact, aiming at guarantying the structural integrity of steel structures against earthquake attacks, several laboratory and field investigations as well as comprehensive research programs have been undertaken worldwide, promoting several detailing and design improvements. In particular, it has to be mentioned that a significant contribution to the improvement of the knowledge on the seismic response of steel structures has provided by the RECOS INCO-Copernicus European Research Project (Mazzolani 2000). A number of aspects have been clarified, with special emphasis given to (1) beam-to-column joint behaviour, (2) detrimental effect due to high strain rate and (3) detrimental effect due to welding, but a number of other aspects remain not yet completely solved, the present scientific research on this topic being still now very strong-minded.

5.1.2 *Ductility criteria*

In case of high-intensity earthquakes, the main concept which recent code provisions are based upon is that the actual strength capacity of structures is lower than the one that could be developed if the structure itself behaved linearly elastic. Then, structures resist earthquake due to structural ductility and energy dissipation worked out by some crucial structural elements undergoing large plastic deformations. As a consequence, the performance of the whole structure is strongly dependent on the behaviour of these crucial elements and the dominant criterion for assessing the collapse of the systems is the ductility demand to ductility capacity ratio of the energy-dissipation zones. Ductility demand is strongly influenced by the structural typology and relative results are highly sensitive to adopted structural models. Ductility capacity essentially depends on the structural component itself, but is also related to the typology of the applied loading. In fact, local ductility (defined at level of members and joints), which is already difficult to be evaluated under monotonic loading, may be strongly affected by the type of seismic ground motion, essentially because: (1) cyclic action produces deterioration of mechanical properties due to repeated plastic excursions, (2) dynamic effects induced by high peak velocities affects the ductility of the material and may induce brittle fracture modes. A consistent procedure for ductility checking of steel structures in seismic areas is presented in Mazzolani and Gioncu (2001).

5.1.3 *Beam-to-column connections*

In case of steel moment resisting frames, the seismic response of the whole structure is severely influenced by the behaviour of beam-to-column connections, where plastic hinges are likely to occur. Hence, the assessment of the seismic performance is essentially based on the ratio between rotation demand imposed by earthquake and the rotation capacity available in connections.

In order to assess the available rotation capacity of connection details used in practice, accounting for low-cycle fatigue, several tests have conducted in recent years and fatigue (S-N) curves have been proposed (Calado et al. 1998). Among many others, excellent overviews on the relevant experimental evidence are presented in (Engelhardt & Sabol 1997, Tsai & Popov 1997, Mazzolani 2000, Roeder 2000). In particular, as an output of these investigations it has been arisen that there are several important issues that have a strong effect on the ductility of practised moment connection configurations under cyclic dynamic actions, such as the quality and detail of welding, stress concentration due to the adopted connection detail, concrete slab, randomness of actual mechanical properties, high strain rate.

In order to overcome failure of joints under seismic loading, recent specifications are aligning at addressing higher benchmark values of available rotation capacity for connections. As a consequence, procedures for improving connection behaviour so to allow for this higher ductility have been proposed and implemented in recent construction practice and code provisions. They include improved welding practice (higher toughness welding electrodes, modification of size and shape of weld access holes) and more accurate details, as well as different structural configurations, the latter being essentially based on two complementary strategies: (1) weakening of the beam cross section at the joint location; (2) straightening of the joint. The feasibility of these new-proposed connection typologies has been shown and discussed by several Authors and for instance by Suita et al. (2000), while a general study on their influence on the global seismic performance of moment resisting steel frames is presented in Anastasiadis et al. (1999). Anyway, the correct prediction of moment connection behaviour, both bolted and welded type, still represents a critical issue within the scientific debate, it being worthy of further research activity and laboratory investigation.

5.1.4 *Effect of welding*

The poor behaviour of welding was one of the main causes of brittle fracture of connections during the last important seismic events (Northridge, 1994 and Kobe, 1995). This has favoured the development of a number of experimental and analytical investigations on the performance of welded components (Mazzolani & Gioncu 2001).

Among the possible causes of brittle fractures in welded joints the following ones have been identified:

- workmanship (welding defects);
- detailing (stress concentration at the root or the toe of welds);
- design practices (larger beam and column sizes than those tested);
- poor welding practice (low-toughness weld metal, poor quality control);
- unusually high seismic input (high strain rates),

Various experimental and numerical investigations (Dexter et al (2000) Mao et al (2001), Dubina et al (2001)), show that, from the several factors affecting the welding behaviour, weld access hole and welding technology have the highest impact.

5.1.5 *Effect of strain rate*

As was previously presented in the introduction of this chapter, one of the main causes for the unsatisfactory behaviour of steel structures during the last great earthquakes was the poor ductility of basic material and detailing. It is well known that the mechanical properties of steel, such as yield strength and ultimate strength and thus the ductility vary with strain rate. The loading-rate effect during an earthquake was considered negligible, especially for earthquakes occurred before Northridge and Kobe events, where moderate velocities were recorded. However, after these very important and special earthquakes, when the recorded velocities have been very high, many specialists consider that the loading-rate may be a possible cause of the unexpected bad behaviour of steel structures.

The first research work concerning the effect of strain rate on the behaviour of metals performed during the third decade of last century indicated a very important increasing of the yield stress with an increase of strain-rate, especially for strain-rate greater than 10^{-1}/sec. The increase of ultimate tensile is moderate, the influence of strain rate being less important than the yield stress. Consequently, the yield ratio defined by the ratio between yield stress and tensile strength, increases as far as the strain-rate increases, with the tendency to reach the value 1.

More recent results (Wakabayashi et al 1994, Dubina et al 2001, Mazzolani 2000) have confirmed the previous results. More detailed research works have shown that the modulus of elasticity is not influenced by the strain-rate variation and the upper yield stress is more strain-rate sensitive than the lower stress. Furthermore, the increasing of strain rate produces a rapid increase of yield plateau, but contrary a slow increase of the ultimate strain.
Based on the theoretical and experimental results, it was also concluded that: (1) the mechanical properties such as yield strength and ultimate tensile strength, increase with increasing strain rate; (2) strain rate influence increases with decreasing the yield strength of the material.

5.1.6 *Hysteresis models for ductility demand evaluation*

Evaluation of seismic performance of steel structures is usually carried out assuming the elastic-perfectly-plastic model for plastic zones. This type of model may be associated with a conventional available ductility related to the maximum deformation beyond which strength degradation is likely to occur. This methodology leads to conservative results and is adequate to obtain reliable information on deformation demands only when the above ductility limit is not exceeded. However, when the evaluation of the actual structural integrity under exceptional loading is of concern, more realistic cyclic load-deformation characteristics should be considered, taking explicitly into account the actual features of members and connections likely to undergo large plastic deformations.

Existing relevant experimental tests show a number of complex aspects that could have a major impact on seismic demands and that therefore should be accounted for in accurate structural system modelling. In particular, strong non-linearity, kinematic hardening of the monotonic restoring force characteristic, cyclic hardening, damage of mechanical properties due to repeated plastic excursions, pinching of the hysteretic cycle are some of the essential aspects arising from laboratory tests.

For instance, the importance of such effects has been evidenced in Krawinkler and Seneviratna (1997), where the evaluation of target displacement to be used for estimating deformation and strength demands to be compared with available capacities determined via pushover analysis is focussed on. In particular, the opportunity and feasibility of adopting simple either period-dependent or constant coefficient to amplify displacement demands of ideal elastic-perfectly elastic systems gathering the ones of actual systems, which are affected by pinching and strength deterioration, is shown and discussed. Further, a parametric study assessing the effect of the above phenomenological aspects on seismic demands via time history dynamic analyses, based on a complex but reliable mathematical model, has been carried out in Della Corte et al. (2001). Firstly, according to several existing test data, it has been shown that the proposed model is able to represent quite well all the above phenomenological aspects having a major impact on the load-deformation characteristics of elements (members and connections). Then a wide numerical study dealing with moment resisting frames has been performed, showing that, in some cases and depending on the type of strong ground motion under consideration, deformation demand evaluated through non-conventional hysteresis model may be remarkably different from the one evaluated by using elastic-perfectly plastic model, especially at collapse limit state of the building.

5.2 *Non conventional methods for improving structural integrity*

5.2.1 *General*

Structural integrity of steel structures may be improved by adopting several special design and constructional techniques on both new and existing structures. The majority of such methods aim at limiting deformation demands to main structural elements allowing for the satisfaction of higher performance levels under design and exceptional earthquake loading. In the following, some non conventional design strategies that can be adopted for both new and existing steel structures in order to reduce the effects of earthquakes on the main bearing structure are briefly presented and discussed.

5.2.2 *Smart use of cladding panels as shear walls*

Steel plate shear walls are being used more and more to resist earthquake and wind forces in steel structures. Major direct proceeds related to their application are improved structural energy dissipation capability and increase of building lateral stiffness. Such a system provides also several other advantages when compared to the other usual lateral load resisting systems, namely steel savings, speed of erection, reduced foundation cost, increased usable space in buildings. In contrast to these issues, in the field of steel structures the spreading out of this structural typology has been quite limited, both under the constructional point of view and with regard to code provisions and development of reliable design procedures.

Actually, there are several ways to profit of shear wall effect to improve economy and structural integrity of steel buildings. For instance, at European level, there have been many attempts to consider the diaphragm effect provided by common lightweight cladding panels as either the primary or the secondary lateral load resisting system of the building, giving rise to promising new trends for the design of steel structures. In such a case, due to the poor mechanical characteristics of the adopted system (both metal sheeting and connecting system), generally the contributing effect of shear panels is quite limited. Nevertheless, it has been already proved that it can be profitably taken into account in low- and medium-rise moment resisting frames for increasing the lateral stiffness of the whole structure, especially at the serviceability limit state (De Matteis et al. 2001). On the other hand, steel plates continuously and rigidly connected to the external frame have been successfully proposed and utilised worldwide. In this case, shear walls are definitively conceived as the primary system for absorbing external horizontal actions, while frame members have the main role of carrying out stationary loads. The main problem concerning this type of system is related to the earl occurrence of shear buckling phenomena, which produce a poor dissipative behaviour of the shear wall, resulting in a slip-type hysteretic response (Driver et al. 1997).

Lastly, some applications of shear walls based on stiffened plates made of special low yield strength steel have been proposed in Japan in recent years (Torri et al. 1996). In such a case, due to their exceptional hysteretic behaviour as well as their capability to early undergo plastic deformations, shear walls are mainly conceived as hysteretic dampers and therefore their employment is essentially based on the concept of seismic structural passive control. As a consequence, shear walls have to be designed in such a way to avoid any buckling phenomenon, checking that the yielding of the panel under shear is of concern.

All the above cited research works demonstrate that steel shear walls may remarkably improve the economy and/or the performance of steel buildings, especially providing both additional lateral stiffness and additional energy dissipation sources, which represent key aspects in the design of high-rise buildings under earthquake loading at serviceability and ultimate limit state, respectively. Nonetheless, two main features inhibiting a more extensive use of steel shear walls have to be recognised: lack of full understanding the design procedure and not full comprehension the actual seismic behaviour of the system. For these reasons, additional research efforts appear to be necessary.

5.2.3 *Use of specially designed eccentrically braced frames*

Eccentrically Braced Frames (EBF) may be classified according to the link length:

- short link
- long link

Depending on their length, the links will behave predominantly in shear or bending. EBF are configured so that the shear/flexural plastic hinges are conducted within the link. The beam outside the link, connections, braces and columns must then be proportioned to remain nominally elastic as they withstand the deformations.

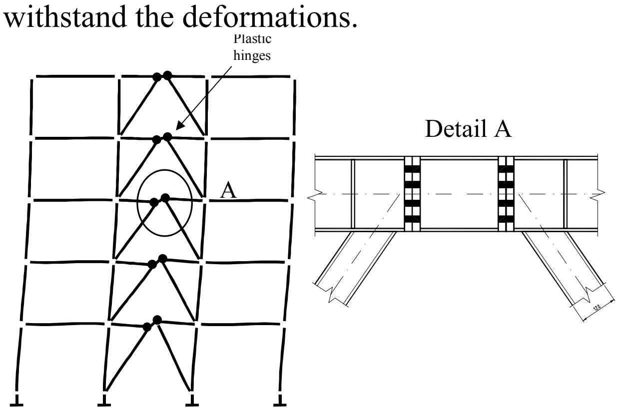

Figure 6. Structure and link configurations

For EBF with short links, it is possible, as suggested by Dubina et al (2002), to design removable dissipative links, by using a lower (compared to the rest of elements) yield steel (Figure 6). The inelastic frame behaviour is proportioned so that the required plastic deformation of the frame is accommodated through the development of shear plastic hinges within the links. The other structural elements – members and joints- behave elastically until “Operational” and “Damage Controlled” performance levels are attained. This is a repairable state of the structure, when the damage is located in links, only. In the “Life-safe” or “Damage State” levels, limited plastic zones are accepted in the other elements.

This solution permits, at the same time, an easy and low-cost intervention for replacing the damaged links.

The experimental tests carried out worldwide have shown a very good behaviour of this innovative system under static and cyclic loading, both in terms of strength and ductility.

6 MASONRY

6.1 *Introduction*

In contrast with other construction materials, masonry has much longer tradition as other materials have. That is why masonry heritage as well as contemporary masonry strongly depend on available materials, climatic and functional requirements, technical knowledge and traditional practices specific to different regions. Classification of masonry buildings can be done according to: materials used for construction (adobe, stone, brick, block), structural systems (unreinforced, confined, reinforced, prestressed and as infill), place of construction (rural, urban) and use of buildings.

Allthrough its history, masonry was usually seen as a compression type of material and while designers often used arches, vaults and domes to use compression to span spaces, the role of the structural walls was solely to support the floors and roofs. Little or no attention was paid on the resistance of the structural walls due to the seismic loading. Although some of the monumental masonry buildings were sometimes designed on the basis of experiments and the simple theory of structures, all through the history the rule of thumb was predominantly used by designers and architects and measures for improvement of structural integrity and resistance of the buildings due to seismic loading were more the exceptions than the rule.

6.2 *Masonry structures under seismic loading*

The previous discussion is leading us to some general overwhelming opinion among designers that masonry is a stiff, heavy and low resistance material with almost no ductility and ability for energy dissipation. It is true that some of the great number of masonry buildings subjected to exceptional earthquakes, many were severely damaged and collapsed. On the other hand, there were cases where some buildings survived the earthquake only slightly damaged or even undamaged, although they have been built at the same location as the damaged buildings.

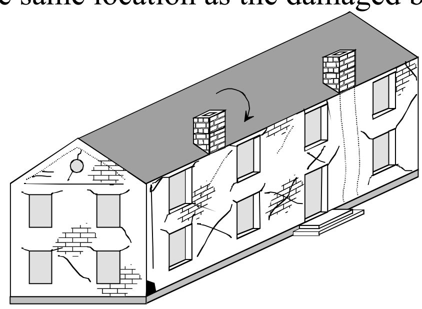

Figure 7. Typical damage of masonry building.

Learning from the past and awareness of the necessity to preserve our historical heritage, improved our knowledge of the mechanical behavior of structural masonry. By analyzing the crack patterns and damage of structural masonry we were able to clearly identify the weak and good points of different structural systems. Once the damage analysis was clearly evaluated and a failure mechanism was defined, the forces that were developed under seismic actions were determined.

Depending on the region, different types of masonry structures with different morphologies of their structural elements can be found. Nevertheless, single and multistory structures are representing the majority of our masonry heritage and when exposed to seismic loading some basic crack pattern (Figure 7) both of the structure and its elements can be generalized and classified as follows (Binda et al. 1999):

- cracks between walls and floors
- sliding between r.c. tie beams (or plates) and the masonry
- cracks in presence of discontinuity in the masonry (closed openings, chimney pipes)
- cracks and failure mechanisms in masonry vaults
- overturning of standing out or overhang elements (balcony, eaves, chimney pot)
- overturning of the gable wall
- cracks at the corners and at wall intersections
- out-of-plane collapse of outer leaves (external panels)
- cracks in spandrel beams (lintels) and/or parapets
- diagonal cracks in structural walls
- partial disintegration or collapse of structural walls and
- partial or complete collapse of the building.

However, for better understanding of masonry buildings under seismic loading, also additional experimental investigations have to be carried out. The results of these tests have consistently revealed that masonry, when properly proportioned, detailed and constructed with good workmanship, provides adequate resistance against seismic forces. Today, contemporary masonry buildings, designed and constructed according to requirements of modern seismic codes, behave adequately.

6.3 *Architectural and structural design concept*

The following basic principles should always be considered when designing a seismically resistant masonry structure:

- simple and regular plans, with symmetry where is possible
- regularity in elevation (both of the geometry and the variation of story stiffness)
- integrated foundation system
- rigid floor diaphragm and
- robustness.

When subjected to seismic forces, the structural walls in masonry buildings should be always distributed in two orthogonal directions. They should be firmly connected (either by steel ties or r.c. ring beams) to a rigid floor diaphragm and their number and strength should be sufficient to resist seismic load. Only in the case of rigid floor action the seismic forces will be evenly distributed to the individual shear walls proportionally to their lateral stiffness. The distance between structural walls should be also limited according to the structural system and the zone of seismicity.

In order to provide torsional stability due to differences in ground motion along the length of the building in the case of earthquakes, the recommendation is that the length of masonry buildings of all masonry structural systems or their separated parts should be limited up to 40 m in the zones of high ($a_g \geq 0.3$ g), and up to 50 m in the zones of moderate and low seismic intensity ($a_g < 0.3$ g).

The distribution, size and position of wall openings, such are windows and doors, have a strong effect on the in-plane resistance of shear walls. Furthermore, when subjected to lateral loading, the concentration of the stresses first occurs in the corner of the openings, thus provoking large strains and consequently resulting in cracking of masonry elements and degradation in stiffness (Bosiljkov et al 1997).

Finally, an important factor in designing masonry structures is also the design of details, connections and non-structural elements such as: partition walls, chimneys, masonry veneer, ornamentations, etc.

6.4 *Seismic resistance verification*

6.4.1 *Structural walls (Tomaževič 1999)*

Masonry buildings are typical shear-wall structures. Masonry shear-walls in two orthogonal directions of the building, which are linked together with floors, represent the basic resisting elements for both vertical gravity loads and horizontal seismic loads. Consequently, basic principles, hypothesis and mathematical models used for seismic resistant design of r.c. shear walls and shear-wall structures can also be applied to masonry buildings. However, mathematical models developed for seismic resistance verification of r.c. structures should be modified to take into account the specific mechanical characteristics of masonry and constituent materials, as well as specific structural characteristics of various types of masonry construction .

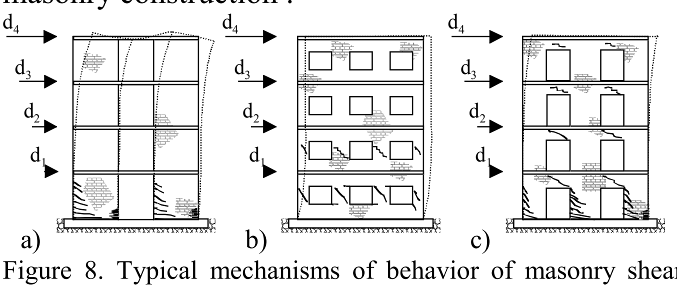

Figure 8. Typical mechanisms of behavior of masonry shear walls (after Tomaževič 1999)

Concerning the structural configuration, shear walls can be either solid or pierced by window and door openings in each story. They represent the basic structural elements of a masonry structure which resist the seismic loads. According to their configuration, type of construction and resulting seismic behavior as well as failure mechanism, masonry shear walls are classified into three main categories: cantilever walls, coupled walls with pier hinging and coupled walls with spandrel hinging (Figure 8).

In the case of cantilever walls (Figure 8a), two or more walls are connected together in the same plane with floor slabs, which distribute the lateral loads among the walls in proportion to their stiffness, but do not transfer any moments resulting from the bending of the walls. It represents a structural system preferred for ductile seismic response. To provide ductile behavior of the structure, strong reinforcement and careful detailing at the lowermost sections of the cantilevers is required due to large bending moments developed in the walls.

Traditionally, masonry shear walls are pierced by window and door openings. Above and below the opening, spandrels connect the walls and transfer the seismic forces. Depending on the proportion of openings, either piers are relatively weaker than spandrels (Figure 8b) or spandrels are relatively weaker than piers (Figure 8c). In the first case, which is most often the case of traditional unreinforced masonry (URM) construction, the damage will first occur to the piers, which may be considered as fixed above and below them. Depending on the geometry, boundary conditions and quality of masonry materials, the piers will either fail in shear due to diagonal compression or rock until crushing of masonry will occur at the compressed zones. Although the pier action of URM walls is not so fatal for a masonry structure as is hinging of columns in the case of a r.c. frame, the non-ductile behavior of weak piers can be improved by means of adequately distributed bed joint reinforcement.

6.4.2 *Analytical models*

In general, the analysis of masonry structures can be done either through usage of lumped parameter models (LPM), structural element model (SEM) or finite element models (FEM). Since for ordinary masonry buildings, the first vibration mode shape is the predominant one, there is usually no need for sophisticated non-linear dynamic and the LPM models are quite rare.

SEM approximates the actual structural geometry more accurately by describing individual structural elements such as piers and walls. In the case of single or multistory buildings due to their regularity and simplicity an equivalent static analysis in two orthogonal directions by using SEM can provide reliable information regarding the seismic safety under expected seismic loads. Nevertheless, since seismic loading can exercise the structural system to and beyond its maximum resistance capacity, the SEM models usually has to be used with static non-linear analysis. In that case a step-by-step procedure is followed, using decreased stiffness values under increasing lateral loads. Nonlinear element behavior is prescribed in the form of nonlinear lateral deformation-resistance relationships, depending on the boundary conditions and failure mode of masonry elements. Usually the bi-linear or tri-linear behavior of SE is considered. The storey resistance envelope is calculated by stepwise drifting of the storey for small values. The SE's are deformed equally (due to the rigidity of floor structure) and internal forces are induced according to the assumed shape of resistance envelope of each SE. In the case of torsional effects (due to relatively displacement of the mass centre to the centre of stiffness of the storey) the displacements of individual SE are modified.

Masonry is a composite, heterogeneous, non-linear structural material. As with other composite materials, also with masonry the mechanical properties are conditioned with the properties of composite elements, their volume ratio and the properties of bond between the units and the layers of mortar. Moreover, the properties and behavior of masonry is strongly affected by the orientation of the main principle stresses towards the bed joints. Following the aforementioned, the main strategies for application of FEM can be adopted for the masonry as follows:

- simplified micro-modeling – usable for small elements and shear wall with openings, where expanded units are represented by continuum elements whereas the behavior of the mortar joints and unit-mortar interface is represented by discontinuous elements;
- homogenization – is aimed to solve the problem of modeling of large masonry structures, by treating masonry as a homogeneous material. Mechanical properties of masonry are predicted from the properties of its constituents, i.e. units and mortars.
- macro-modeling – for modeling whole structures where masonry is regarded as an anisotropic composite material (constitutive models).
- Each of those FEM strategies has advantages and disadvantages. Nevertheless, when analyzing old historical building even a simple elastic FEM analysis should be very useful as an anticipation for effective and less time consuming SEM seismic verification.

6.5 *Optimization of the seismic response*

Improving the seismic response of the structure can be done through repair, strengthening or seismic upgrading of the existing or damaged structural system of building. The main goals are always the same – to increase the strength and ductility. The measures that will be applied depend on numerous factors, but basically the seismic resistance of the building is the governing one.

6.5.1 *Methods of strengthening of masonry walls*

Strengthening of the building elements in existing masonry constructions should be always done systematically. Some of the basic methods of strengthening of masonry walls are:

- repair of cracks
- repointing the joints with stronger or the same mortar as it has been already built in
- application of reinforced cement coating (jacketing) on one or both sides of the walls
- application of FRP or CFRP stripes on the walls
- grouting with cement, modified cement, or epoxy grout
- prestressing the walls in one or both directions (vertical or horizontal) and
- reconstruction of most damaged portion of the wall.

Special attention should be paid in the case of historical monuments, where the proposed methods should fulfil the basic requirements of restoration and conservation of cultural monuments. In that case the application of advanced materials should fulfil the requirement of compatibility as well as durability with the original architectural and mechanical elements of the existing structure.

6.5.2 *Methods of improvement of structural integrity*

In order to improve the structural integrity of the building system, beside the strengthening of individual structural elements, the following measures should be considered:

- tying of walls with steel ties
- interventions in floor structures and roofs
- repair of corners and wall intersection zones
- strengthening of walls by confinement and
- seismic isolation.

In the practice, we usually have to combine several of those measures together. Nevertheless, our final goal is always the same: to achieve monolithic response of the structure with increased load bearing capacity and energy dissipation.

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