FROM ONE-STOREY TO MULTI-STOREY ASYMMETRIC SYSTEMS: CONCEPTUAL DIFFERENCES AND PROBLEMS

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ABSTRACT

The seismic behaviour of asymmetric buildings has been up to now quite always investigated by the study of one-storey schemes, composed by shear-type resisting elements. This simplified approach gives useful information both on the elastic and the inelastic response but it is not able to catch all the aspects of the behaviour of actual multi-storey buildings. For this reason, the use of multi-storey models is now strongly advised. In this paper the peculiarities and the problems to be faced passing from shear-type one-storey models to framed one-storey schemes and from these to multi-storey systems are discussed. The importance of the overstrength and the influence of vertical loads and load conditions on it, the differences in the response due to the actual non linear force-displacement relationship instead of the simplified elastic-perfectly plastic model are pointed out. The difficulty in choosing the most appropriate parameters in order to significantly describe the inelastic response is finally discussed.

INTRODUCTION

In the past, the majority of the research investigations on the elastic behaviour of asymmetric buildings was based on the study of one-storey systems [4]. The reasons of this choice are quite obvious: the equations of motion for such a model may be easily written in function of few parameters (lateral and torsional stiffness k_x k_y k_θ, mass m, location of stiffness and mass centres C_S and C_M); static and modal analysis may be performed in an analytical way; the elastic response to any given seismic record may be evaluated by means of a simple numerical procedure suitable even for small, non powerful computers. In few years the adoption of this model allowed a thorough comprehension of the elastic behaviour of one-storey systems and of the differences between the results of static and modal analysis, suggesting the use of additional eccentricities in order to correct the results of static analysis and to make them equivalent to those provided by the modal one. Many seismic codes accepted these results and prescribed the use of simplified formulations for evaluating additional eccentricities which should be able to grant such equivalence (but in some case they failed in achieving this result, specially in the case of torsionally flexible schemes [1] [2]).

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One-storey systems were widely used also for analysing the inelastic behaviour. In most cases, the three-dimensional scheme was constituted by two sets of resisting elements having an elastic-perfectly plastic behaviour, disposed along two orthogonal directions. A larger number of parameters is here involved, because the inelastic response to a seismic record depends on the number and location of the resisting elements and, in a stronger way, on the strength of each element and thus on the design criterion adopted. This led in some cases to contrasting conclusions on some aspects, e.g. whether the maximum ductility demand was achieved at the stiff or at the flexible edge. These were overcome by the observation that the maximum displacements in the inelastic range are scarcely influenced by the strength of the elements: the ductility demand is thus directly connected to the strength provided to the element [7] [13] [14]. Moreover, differently from the elastic behaviour of asymmetric systems the rotational component of the motion of a structure well in the inelastic range appeared to be less relevant than the translational one. The use of modal analysis (or of static analysis properly corrected by means of reliable additional eccentricities) appeared to be strongly necessary in the case of torsionally flexible schemes in order to avoid dramatically large ductility demands [8]; the use of further design eccentricities was deemed appropriate, particularly in the case of torsionally stiff schemes [10].

In the last decade a criticism progressively arose against the use of one-storey schemes, which were charged to describe in an oversimplified and unfaithful way the behaviour of multi-storey buildings. A scheme constituted by two sets of multi-storey plane frames, mutually connected by floor slabs infinitely rigid in the horizontal plan, was in most cases considered appropriate [5] [12], although some researchers preferred a shear-type model, i.e. a spatial set of frames with stiff and strong girders in which at each storey the plastic hinges may be located only at the ends of the columns [3] [6].

The adoption of a multi-storey model may undoubtedly give more complete and reliable information on the seismic behaviour of a building. Nevertheless it should be used with great care and awareness, because the inelastic response of such a scheme is conditioned by a large number of facts and parameters (like the design approach and the overstrength of the single cross-sections), which in connection to slight differences in the seismic input may significantly alter the sequence of yielding and the global failure mechanism. At the same time, the complexity of the scheme often makes the numerical analysis so cumbersome and slow (even with powerful computers) to limit the researcher to few applications, from which it might be hazardous to take general conclusions. For this reasons we believe necessary, and thus we carry on in this paper, a thorough reflection about the conceptual differences and the problems which arise while passing from the idealised one-storey system (three-dimensional set of elastic-perfectly plastic resisting elements, which will be from now on named *shear-type one-storey system*) to the *framed multi-storey system*, through the intermediate model of *framed one-storey system* (three-dimensional set of one-storey frames); or, in a parallel way for the plane systems, while passing from a shear-type one-storey scheme to a multi-storey frame via the one-storey frame.

OVERSTRENGTH

One of the main aims of the seismic design is the achievement of a global collapse mechanism, with plastic hinges at the ends of all the beams and at the bottom cross-section of the first order columns, so as to exploit in the best way the ductility of the scheme. Theoretically, an elastic behaviour up to the design value of horizontal forces and the subsequent contemporaneous plastification of all the above cross-sections would be acceptable. In practice, most cross-sections have an overstrength, i.e. a strength larger than the one strictly required by the design analysis. In the case of steel frames, technological and commercial reasons impose the use of a limited set of sections; in r.c. structures the use of single reinforcing bars allows to reduce, but not to wholly eliminate, the difference between required and provided strength. Even more relevant, and surely not eliminable, is the effect of using more load conditions (increased vertical loads without horizontal forces, reduced vertical loads with positive or negative horizontal forces). As a consequence of this, under the design value of horizontal forces plastic hinges arise only in few cross-sections and the frame may therefore hold larger forces before attaining a global collapse mechanism.

After defining the strength of all the cross-sections, the collapse multiplier of a given distribution of horizontal forces and the limit global shear at the base may be easily evaluated by means of push-over analysis or by using the theorems of limit analysis; it is to be noticed that it is not correct to calculate the limit global shear at any level as the sum of the limit shear of all the columns, $V_{\mathrm{lim}} = \Sigma\,(M_{\mathrm{lim,top}} + M_{\mathrm{lim,bottom}})\,/\,h$, because this would be meaningful only in the case of an undesired partial collapse mechanism with hinges at the top and at the bottom of the columns. The *global overstrength* of the frame may thus be defined as the ratio of limit shear to design shear at the base of the frame.

An example may point out how relevantly the overstrength is influenced by the use of more load conditions and, in particular, by the comparative entity of vertical loads and horizontal forces. Reference is made to the six-storey building with steel structure analysed in other papers [9] [10], subjected to a vertical load q=5.1 kN m^{-2} in seismic condition (dead load plus reduced live load) which corresponds, in absence of seismic action, to a design value q_d=9.6 kN m^{-2} (dead and live loads increased by the partial coefficients γ_g γ_q). The stiffness of the members of the transversal frames (fig. 1) has been assigned so as to obtain a first period of vibration T=1.0 s; the second moment I_b of the cross-section of the beams of each frame has been maintained constant at every level, so as that of the columns I_c, with a ratio $I_b\,/\,I_c = 0.364$; these values have been assigned independently of those of the available commercial sections. The seismic actions for a simplified (static) analysis have been evaluated according to the elastic spectrum proposed by Eurocode 8 for soil A with α=0.35, reduced by a behaviour factor q=5, obtaining a triangular distribution of horizontal forces.

The internal actions have been firstly evaluated only for one load condition (horizontal actions) and in a second way as the envelope of the results of more load conditions (increased vertical loads, reduced vertical loads plus or minus horizontal forces); different values of the vertical load have been in this case assumed, corresponding to an uniform distribution among the transversal frames q_u, to one half of this (as if the weight of the floor slab was equally divided between the two sets of longitudinal and transversal frames) and to the double of this (to check the possibility of further increase of vertical loads or reduction of horizontal actions). In the first case only the strictly necessary strength has been assigned to each cross-section, while in the second one other possibilities have been considered too (equal strength for both ends of a beam, equal strength for all the beams of a storey).

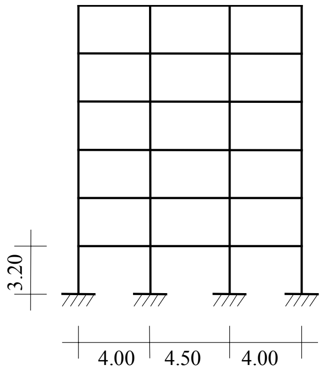


Fig. 1 – Transversal frame

The limit global shear at the base has been evaluated by means of limit analysis, thus obtaining the values of the global overstrength shown in table 1. Obviously, when no vertical loads are considered and the cross-sections are provided of the strictly necessary strength the frame is able to hold just the design forces, i.e. the global overstrength is unitary. The use of more load conditions, connected to the presence of vertical loads, increases the overstrength as much as the vertical load grows up. In the standard situation (vertical load equal q_u) the strength increment is slightly more than 50%, while it is about 25% for small vertical loads and largely more than 100% for high vertical loads, even if the strictly necessary cross-sections are used. It must be remarked that it is not important the value of vertical load by itself, but only its effect in comparison with hat of horizontal forces. If both vertical and horizontal actions were increased by the same factor the global overstrength should remain unchanged.

The effect of the unification of sections is in this case scarcely relevant, because of the regularity of the scheme (but it could have importance in case of larger variation of the spans). The use of commercial sections, together with the necessity of satisfying other conditions like the displacement serviceability limits, leads to a further increment of strength, which in other examples has been quantified as about 30%.

Other aspects may condition the overstrength. Some examined cases show that the influence of the horizontal forces distribution (constant, linear or proportional to the first mode of vibration shape) is very small, while a reduction of the number of storeys give further increase to the overstrength, probably because it reduces the effect of horizontal forces in comparison to that of vertical loads. A more systematic analysis could be useful in order to quantify these considerations.

Table 1 – Global overstrength of the frame

load conditions	strength	vertical load		
		0.5 q_u	q_u	2 q_u
one	strictly necessary	1.00	1.00	1.00
more	strictly necessary	1.26	1.55	2.26
more	equal for both ends	1.27	1.58	2.33
more	equal for a whole storey	1.30	1.61	2.49

Such a long discussion about overstrength is not a pedantry and it has strict connections to the study of asymmetric systems. In actual, non regular, buildings different frames may have dissimilar vertical loads and may absorb a quite variable portion of the seismic actions. Their overstrength may thus be very different [8] and the sequence of plastification of the frames during the inelastic dynamic response to a seismic record may lead to a sharp reduction or increase of the instantaneous value of the uncoupled lateral-torsional frequency ratio Ω_θ which could dramatically condition the seismic behaviour of the scheme. A thorough knowledge of the overstrength distribution among the frames is therefore necessary in order to correctly analyse the results of the inelastic response of the system.

FORCE-DISPLACEMENT RELATIONSHIP

The word “shear-type” has been used to individuate schemes having an elastic-perfectly plastic force-displacement relationship. Framed systems could present such behaviour only if their overstrength was unitary, i.e. if all plastic hinges arise contemporaneously (fig. 2a). The behaviour of an actual frame is really different. An example is provided by a one-storey three bays frame with the same geometrical characteristics of the transversal frame examined in the previous section and with vertical loads such as to obtain an overstrength equal to 1.5. As expected, only some hinges arise when the design shear V_d is reached, thus reducing the stiffness of the scheme and the slope of the load-deflection curve (fig. 2b, point A) before the attainment of the collapse mechanism (fig. 2b, point B) from which starts the horizontal segment of the curve. It may be noted that the plastic hinges at the bottom of the columns and at the right end of the beams arise nearly contemporaneously (points A-A_1), while the plastic hinges at the left end of the beams arise much later, giving the curve an almost trilinear shape. In the case of multi-storey frames the plastifications are more spaced and the load-deflection curve varies in a soft way (fig. 2c).

When cycles of loading and unloading are performed, other differences may be observed between the shear-type and the framed schemes. In the first case all plastic hinges arise always at the same time and the force-displacement relationship remains unchanged (fig. 2d). On the contrary, for the framed structures the sequence of the plastifications in the first unloading phase differs from that of the loading phase, while it remains successively unchanged. It depends on the fact that in the first loading phase the internal actions caused by the increasing horizontal forces are added to those given by the vertical loads, while in the reversal after the full plastification only the effect of horizontal forces may give rise to new plastic hinges. This is particularly evident for the one-storey frame: in its unloading phase only the bottom cross-sections of the columns yield at first (fig. 2e, point C) and it is necessary a relevant variation of forces before achieving the limit moment in other sections or the full plastification (fig. 2e, point D); the slope of the curve is thus much greater in this phase, in respect to what denoted in the first loading phase. Analogous variations may be noted for multi-storey frames (fig. 2f), although in this case partially hidden by the rounded shape of the curve.

Once again it must be remarked that in irregular structures each plane frame has its own force-displacement relationship and that the different slope of the inelastic branches of the curves, possible even for frames having equal elastic stiffness, surely conditions the global behaviour of the building.

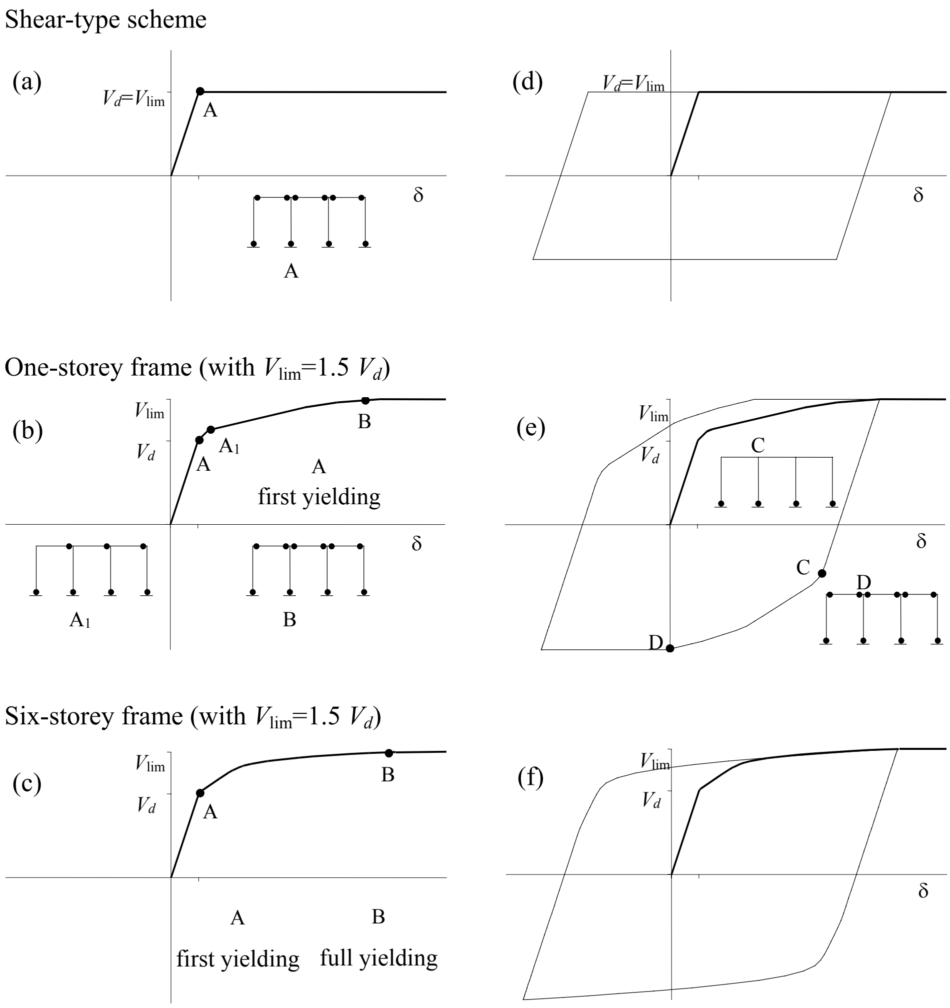
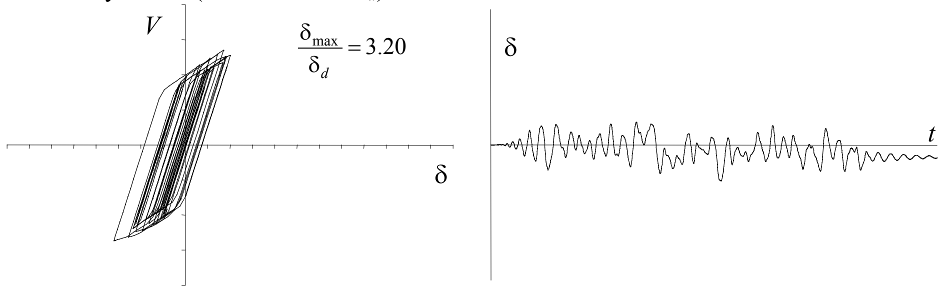


Fig. 2 – Force-displacement relationship for push-over analysis
and for cyclic loading and unloading

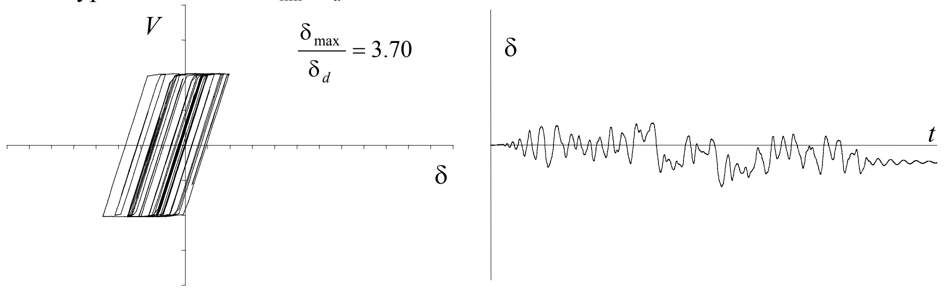
The differences in the force-displacement model may cause, both for three-dimensional and for plane schemes, relevant differences in the response to given seismic records and in the consequent ductility demand. In order to discuss this, the response of the one-storey frame previously examined (which presents a global over-strength 1.5) has been compared to that of two different shear-type schemes with the same elastic stiffness, having respectively $V_{\lim}=V_d$ and $V_{\lim}=1.5\ V_d$. The systems have been subjected to an artificial accelerogram corresponding to the design spectrum (Eurocode 8, soil A) with PGA=0.35 g. The analysis has been repeated with a tripled PGA (1.05 g) because in the first case the frame did not reach full plastification.

The time-displacement histories of the schemes (figs. 3 and 4) appear to be quite similar for most time, as it may be expected because of the coincident elastic stiffness.

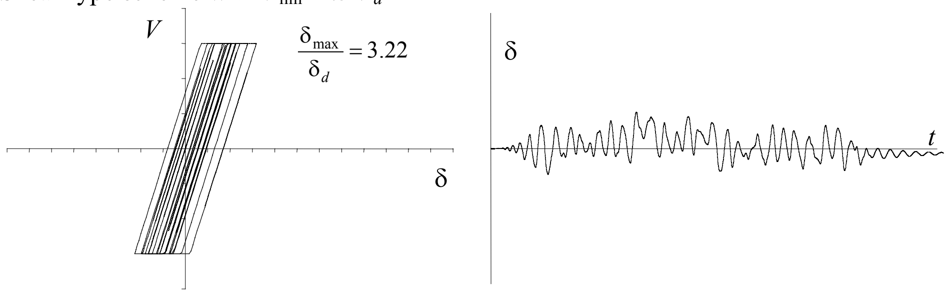
One-storey frame (with $V_{\lim}$=1.5 V_d)



Shear-type scheme with $V_{\lim}$=V_d



Shear-type scheme with $V_{\lim}$=1.5 V_d



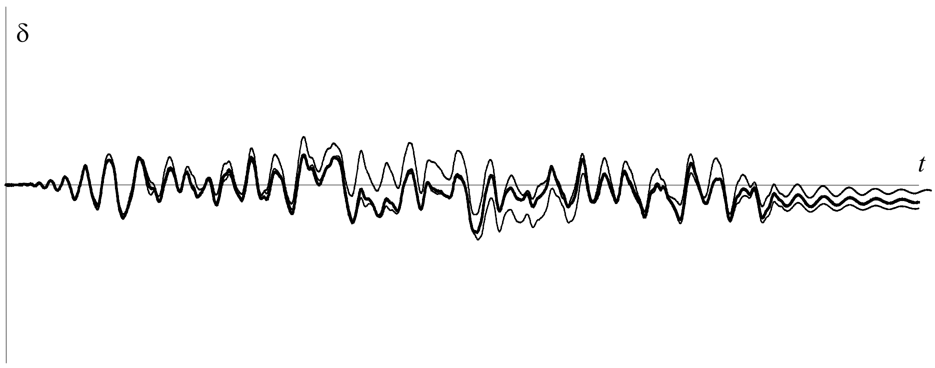


Fig. 3 – Force-displacement relationship and time-displacement history
for a seismic record with PGA=0.35 g

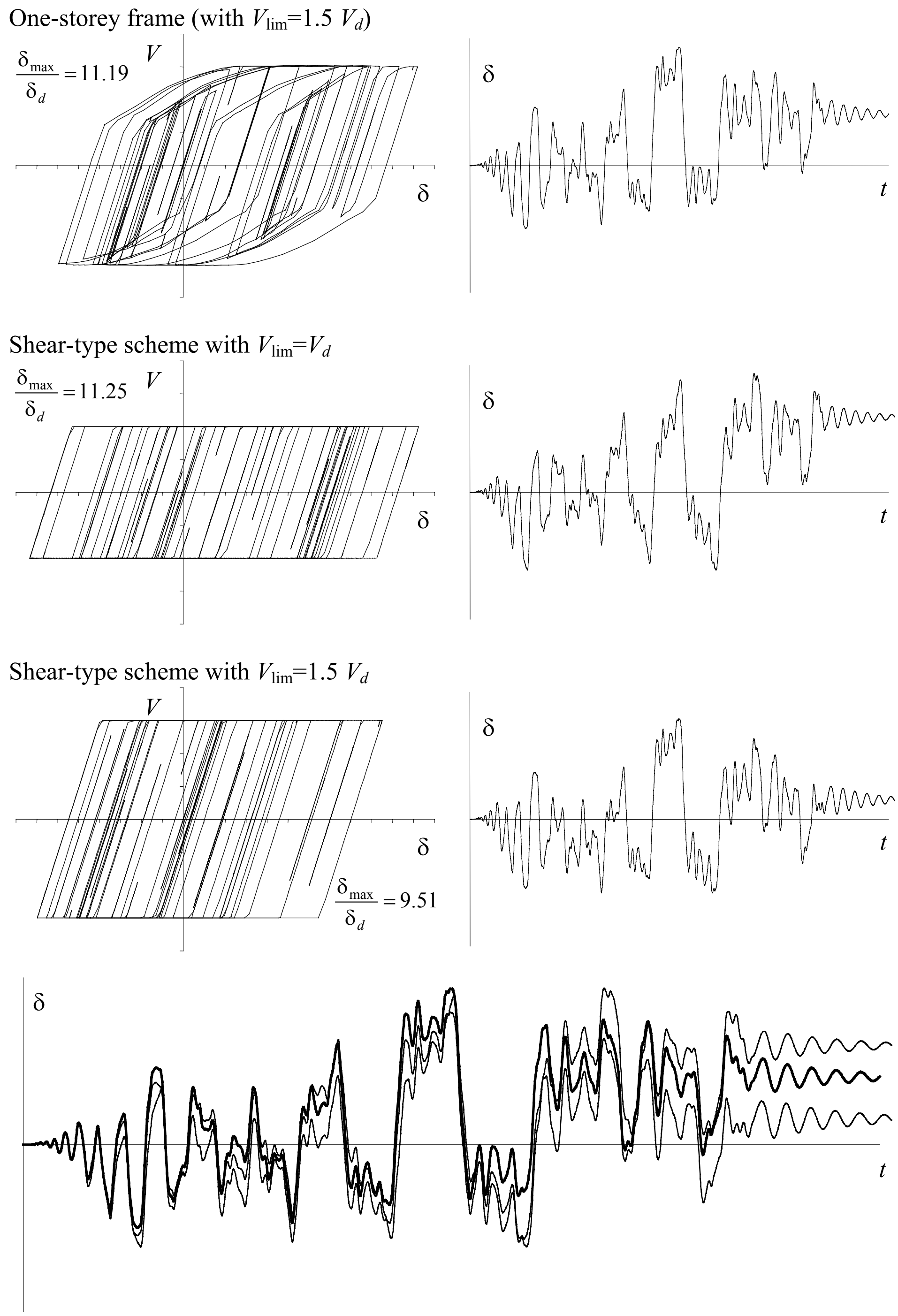


Fig. 4 – Force-displacement relationship and time-displacement history
for a seismic record with PGA=1.05 g

Nevertheless large and quite different plastic excursions may be observed during the inelastic phases. As a consequence, for PGA=0.35 g the displacements of the shear-type scheme with $V_{\lim}$=1.5 V_d are mostly positive, while those of the other models are generally negative. Differences of about 20% in the maximum displacements may be noted for both values of PGA; much larger is the discrepancy in ductility demand (up to 70%), owed to the different value of the yielding displacement (δ_d for the shear-type scheme with $V_{\lim}=V_d$, 1.5 δ_d for the other schemes, being δ_d the displacement provoked by the design forces). The contemporaneous effect of the overstrength and of the slope of the force-displacement relationship fully explains the fact, at first strange and unexpected, that the one-storey frame does not reach full plastification under a seismic record with PGA=0.35 g, although it was designed against such event with forces reduced by a behaviour factor q=5.

OUTPUT PARAMETERS

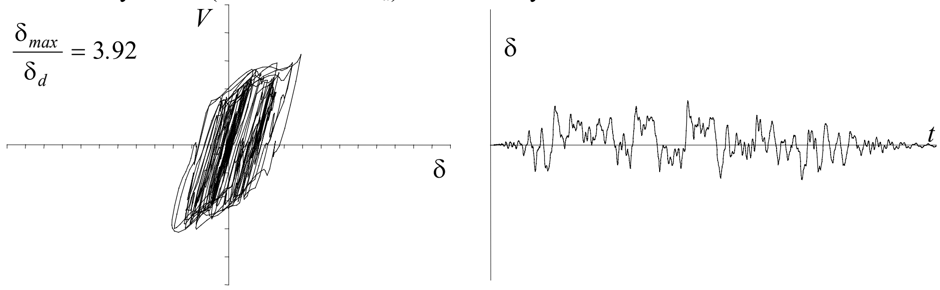
The results provided by the inelastic response analysis of a shear-type one-storey model are always easy to analyse and to describe. In the case of plane schemes two graphics (force-displacement and time-displacement) are sufficient to point out the whole time-history, but in most cases even a single value (the maximum absolute displacement, or the correspondent ductility demand) is able to provide the necessary information. In three-dimensional schemes, constituted by sets of elastic-perfectly plastic elements, a small number of graphics or parameters (like the maximum displacements of the elements) are necessary.

The situation is much more complex in the case of multi-storey frames. Even for plane schemes, each floor has a different horizontal displacement and each section has its own time-history and its ductility demand. The number of graphics and parameters necessary to fully describe the inelastic response dramatically increase. The choice of a limited set of parameters able to synthesise what is happening becomes one of the most important aspect of the work.

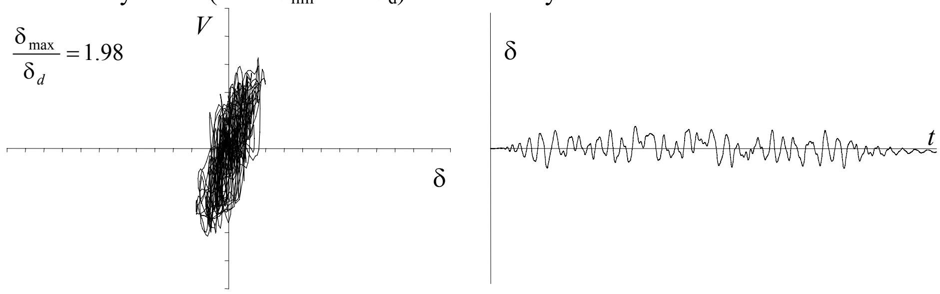
To point out some problems, the same six-storey frame (with $V_{\lim}$=1.5 V_d) studied in the previous sections has been subjected to the artificial accelerogram, once again scaled to PGA=0.35 g and 1.05 g. In order to draw a comparison, two shear-type one-storey systems have been examined too; their mass coincides with the total mass of the frame and their stiffness is evaluated so as to obtain the same period of vibration (T=1 s) while their strength is equal to V_d and 1.5 V_d respectively; being SDOF systems, their normalised response coincides with that already shown in the previous section. Figures 5 and 6 show the force-displacement and time displacement histories; in the case of the six-storey frame, the global shear at the base has been assumed as horizontal action, while, as displacements, the values at the first order and at the top of the frame have been separately considered (and normalised each one by the corresponding design displacement).

The first thing which strikes any observer is the total loss of linearity in the force-displacement relationship. In the global-shear versus first-order-displacement diagram, linearity should be maintained during the elastic phases if the girders were infinitely stiff; some approximately linear segments may still be individuated, but they are strongly distorted because of the effect of the higher modes of vibration.

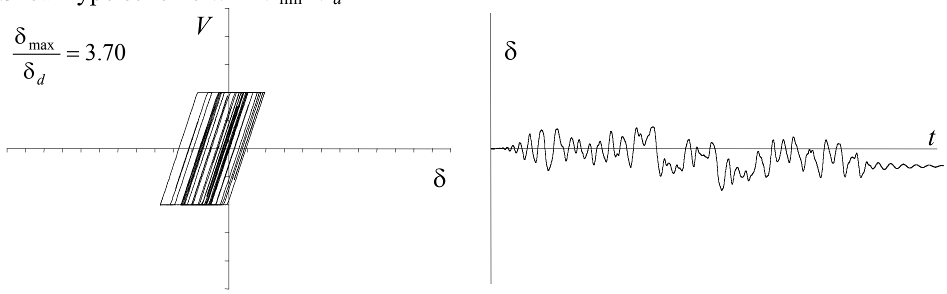
Multi-storey frame (with $V_{\lim}$=1.5 V_d) – first storey



Multi-storey frame (with $V_{\lim}$=1.5 V_d) – sixth storey.



Shear-type scheme with $V_{\lim}$=V_d



Shear-type scheme with $V_{\lim}$=1.5 V_d

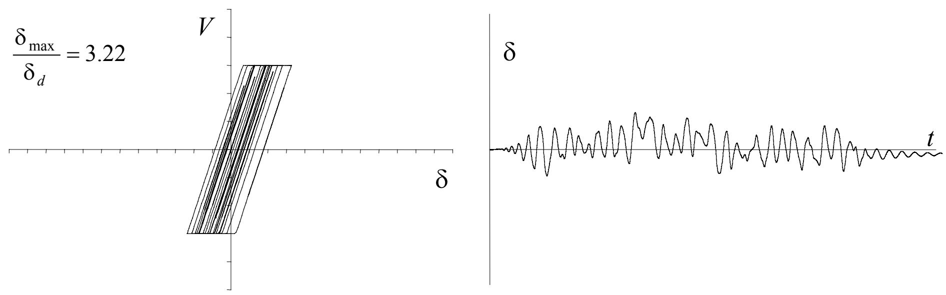
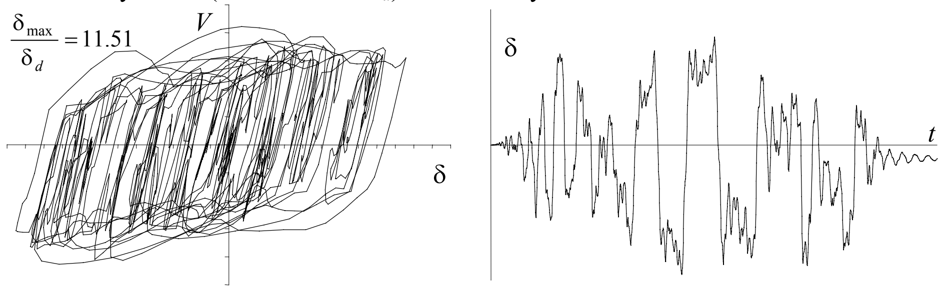
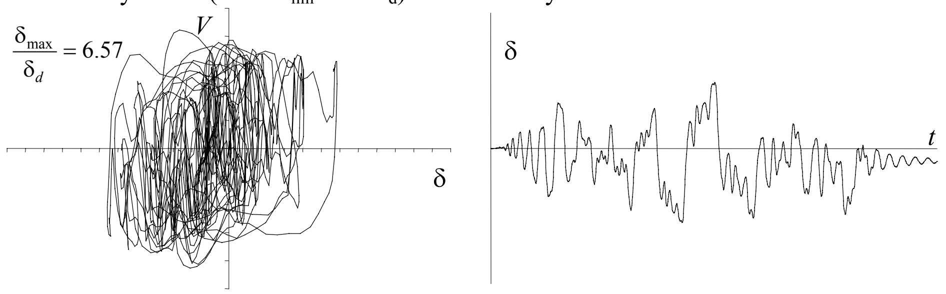


Fig. 5 – Force-displacement relationship and time-displacement history for a seismic record with PGA=0.35 g

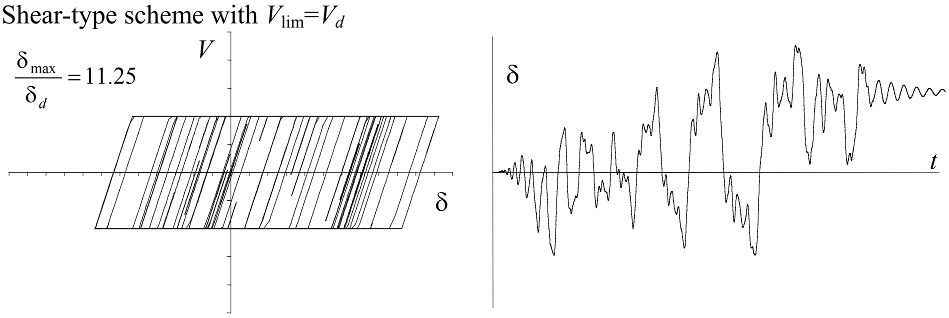
Multi-storey frame (with $V_{\lim}$=1.5 V_d) – first storey



Multi-storey frame (with $V_{\lim}$=1.5 V_{d}) – sixth storey



Shear-type scheme with $V_{\lim}$=V_d



Shear-type scheme with $V_{\lim}$=1.5 V_d

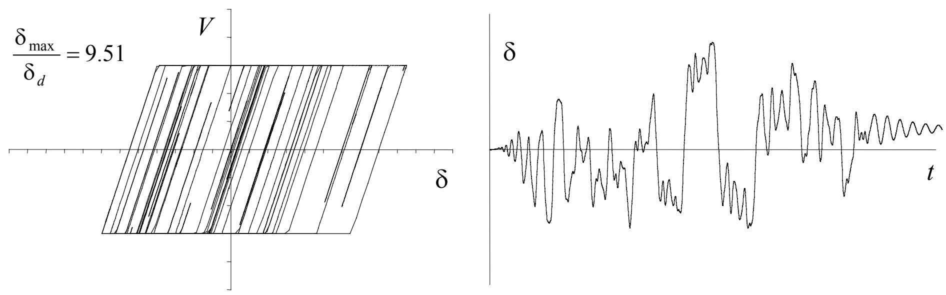


Fig. 6 – Force-displacement relationship and time-displacement history
for a seismic record with PGA=1.05

The global-shear versus top-displacement diagram is much more twisted, as it would be even in the case of stiff girders. A second important aspect is the strong differences in the normalised (i.e. divided by the design values) displacements of the first and the top floor. This is clearly owed to the effect of the plastification of the bottom cross-sections of the first order columns, which increases the horizontal displacements at all the storeys but in a way much more relevant, in proportion, at the first one. It is therefore apparent that the use of the first order normalised displacement instead of the top ones (or the referring to any simplified shear-type model) in evaluating the global ductility demand may lead to quite different conclusions. It must be finally noted that in the multi-storey frames the global base shear may significantly exceed the value $V_{\lim}$ obtained by the push-over analysis. Such peculiarity, which cannot be simulated by means of one-storey schemes, is clearly owed to the higher mode of vibration of the frame and may significantly increase the bending moment at those cross-sections of the columns which are deemed to remain elastic.

CONCLUSIONS

The aspects discussed in the paper (influence of overstrength and force-displacement relationship, choice of output parameters) individuate some of the main conceptual differences and problems which may be encountered while passing from the simple shear-type one-storey schemes to the complex framed multi-storey systems. Although none of these aspects is completely new or dramatically relevant, on the whole they significantly influence the capability of correctly using and interpreting multi-storey models, risking to make vane their great potentialities. A wide discussion and a deepening of this is really advisable.

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