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NONLINEAR EARTHQUAKE BEHAVIOR OF SPATIAL FRAME STRUCTURES UNDER THE INFLUENCE OF BI-DIRECTIONAL GROUND MOTIONS

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SUMMARY

The aim of this research is to analyze the behavior of spatial steel frame structures in the dynamic nonlinear field. These structures were designed in the elastic field so their response to exceptional seismic events could be tested. These tests yielded an assessment of the structure's resistance and structural ductility. A numerical analysis was conducted to determine the most important quantities. The structural response and the behavior under two horizontal components of the ground motions, depending on the seismic direction, were examined. Then, the structural response, depending on the structural eccentricity, was assessed. Accelerogram recordings of recent Italian earthquakes were chosen and used on the basis of their frequency content.

INTRODUCTION

Seismic building codes, also in Italy, prescribe structural dimensioning in the elastic field for medium intensity earthquakes. When violent earthquakes occur, structures can be badly damaged but should never collapse. It is then assumed that structures withstanding violent seismic events can become plastic. Plastic zones allow the dissipation of energy transmitted by the earthquake to the structure depending on the degree of structural damage. Some computer programs were developed to analyze structures in the non elastic field by Finite Element Method (FEM). The DRAIN-TABS program was used in this research (Ref.1). Numerous parameters need to be defined before conducting a structural analysis in the nonlinear field. In order to determine the influence of these parameters on the structural response, a previous numerical analysis was carried out. The parametrical analysis and the following numerical elaborations were conducted for a four storey steel tower which included a square base dimensions (Fig.1). This type of simple structure was analyzed so that a qualitative evaluation of the structural response to parameters' variation could be carried out. The structure, under investigation, was designed in the elastic field using well known methods of modal analysis and response spectrum technique. The structure was assessed by conducting a time-history analysis in the nonlinear field.
The ground motions records was chosen on the basis of its Fourier amplitude spectra of acceleration (Figs.2,3). The two horizontal components of the ground motion were used because they usually have similar response spectra. The structural behavior under violent seismic events was tested.
The test included an analysis of the two horizontal components while varying the direction of the main component. In order to assess the influence of this last quantity, three different eccentricity values of the mass center and the stiffness

center 3, 6, 9% of the structure's base dimension, were adopted. This choice was made in order to underline the torsional effects which characterises the building's response to different epicentral directions.

EARTHQUAKES USED IN THE RESEARCH

Seismic design usually takes into account the seismic horizontal components, because their effects on the structures are more relevant. Generally, the "design earthquake" is defined by determining the response spectrum. The design response spectrum is considered constant as the ground motion direction varies. The building is then tested in two non-contemporaneous orthogonal directions chosen by the designer. Actually, recordings indicate the contemporary presence of the two horizontal components in the ground motion. Many times the two response spectra ordinates are almost equivalent and in this case is incorrect to consider just one component at a time. This fact may be important if the analysis is done in the nonlinear field, and in particular when the effects of the seismic direction's variability are considered. Since a building's orientation is random with respect to an earthquake's direction, the assessment of the structure's response as the seismic direction varies becomes quite important. A good design should be based on the assessment of the maximum structural response caused by an earthquake. Previous researches (Refs.2,3) in the elastic field showed that it is not important to find the epicentral direction which maximises the structural effects because it is nearly equal to the two directions parallel to the principal directions of the structure itself, for well designed seismic buildings. Well designed seismic buildings are geometrically regular and demonstrate small eccentricity between the mass center and the instantaneous rotation center. The problem of evaluating the horizontal seismic action's value and the ratio between the two horizontal components has not yet been solved.

When a calculation method in the linear field, based on the modal analysis with response spectrum, is used for the acceleration design two different techniques can be applied: 1) the same spectrum is used for both the horizontal components; 2) the component hortogonal to the principal direction is reduced. The second technique is suggested by the the Eurocode and by the C.N.R.-G.N.D.T. seismic draft proposal. In the C.N.R.-G.N.D.T. draft proposal, a value of 0.3 is adopted for the ratio between the spectral values in the two directions.

A nonlinear analysis is far more complicated because it is necessary to consider both the effects of the two horizontal components and the variability of their directions. Accelerograms registered during Italian earthquakes (Friuli 1976, Campano-Lucano 1980) were used in the numerical analysis. The specific characteristics of these accelerograms were analized and these quantities were used to establish a relationship with the dynamic characteristics of the tested structure. The principal ground motion characteristics which influence the structural dynamic response are: (i) intensity: (ii) frequency content; (iii) duration. Tab.1 shows the selected accelerograms used for the numerical applications.

These accelerograms were chosen from many samples in order to cover a wider frequency field, thereby including the principal frequencies of the tested structure. The accelerograms were normalised applying a value of 0.5 g for the

Tab. 1

Locality	Date	Hour	Comp.	a_{max} [g]	a(2)/a(1)
Tolmezzo	6.5.76	20.00:15	N-S	0.37 (1)	
	6.5.76	20.00:15	E-W	0.32 (2)	0.86
Tolmezzo	6.5.76	20.00:15	SE-NW	0.48 (1)	
	6.5.76	20.00:15	NE-SW	0.32 (2)	0.67
Bagnoli	23.11.80	19.34:54	N-S	0.19 (1)	
	23.11.80	19.34:54	E-W	0.13 (2)	0.68
Brienza	23.11.80	19.34:54	N-S	0.22 (1)	
	23.11.80	19.34:54	E-W	0.17 (2)	0.77
Calitri	23.11.80	19.34:54	N-S	0.14 (1)	
	23.11.80	19.34:54	E-W	0.12 (2)	0.86
Sturno	23.11.80	19.34:54	N-S	0.33 (1)	
	23.11.80	19.34:54	E-W	0.23 (2)	0.70

maximum accelerations of the larger component ($a_{max}(1)$) and the ratio between the maximum accelerations of the two components was not changed for any earthquake. The relationship between the tested structure's dynamic properties, in the elastic field, and the accelerogram's frequency content was assessed.

PARAMETRICAL ANALYSIS

A static scheme, consisting of a frame with material continuity for the beam-column joints, was used for dimensioning the structure in the elastic field. Some elements obtained by the DRAIN-TABS library were used for the numerical modelling: columns, beams and semi-rigid connections with a bilinear constitutive relationship. The plastic-collapse-limit-state was controlled following standards, set by CNR 10011/85 and Eurocode 3, in order to achieve the prestricted level of ductility performance of the structural elements before the local buckling phenomena.
During the structure's design, storey displacements were reduced using columns dimensions greater than the resistance limit, in order to have weak beams and strong columns. Theoretically, this design criteria allows plastic hinge formation only at the end sections of the beams. The collapse mechanism corresponded to the formation of a final plastic hinge at the base of the columns of the first storey and it is adopted to obtain acceptable structural ductility.

<u>Semi-rigid connections</u> Simplifying hypothesis were commonly applied to the behavior of the beam-column connections for the steel structure's design; in fact, joints are generally considered as rigid connections or hinges. Semi-rigid connections can be useful for the structural analysis, because the behavior of structural connections which have notable relative rotation can be described realistically. In the model used by the DRAIN-TABS program, the single joint behavior is influenced only by the connected elements relative rotation.
The following parameters were defined in order to evaluate the bending moment-rotation relationship: a) the first rotational rigidity K_e; b) yielding moment M_y; c) the second rotational rigidity K_i, or the strain-hardening (s.h.) ratio.
Experimental tests (Ref.4) provided the K_e and the s.h. ratio. The influence of the semi-rigid connections for two extreme configurations, characterised by small K_e values (end plates connections) and high K_e values (fully welded connections) was evaluated.
In the parametrical analysis the s.h. ratio was assumed variable from 0 (corresponding to an elastic perfectly plastic constitutive model) to 0.1 (the experimental superior limit). The connections in the steel structures must provide resistance equal or superior to that of the weakest connected element, usually the beam section; in doing so, the joint is not a weak point in the structure. It is possible to distinguish three different cases: (i) $M_{y,c}<M_{y,t}$ (i.e. partial restoration joint); (ii) $M_{y,c}=M_{y,t}$ (i.e. total restoration joint); (iii) $M_{y,c}>M_{y,t}$ (i.e. more than total restoration joint); where $M_{y,c}$ is the yielding moment of the connection and $M_{y,t}$ is the yielding moment of the beam end section.
A serviceable design should provide: 1) plasticization of beam's end section; 2) surpassing of the connection limit value $M_{y,c}$; 3) column end section's plasticization. In the present research, a ratio $M_{y,c}/M_{y,t}=1.12$ was used for the designed connections with $M_{y,c}>M_{y,t}$.

<u>Damping</u> Usually in the equation of the dynamic equilibrium the damping matrix is expressed as a linear combination of the mass matrix and the stiffness matrix. The multiplying coefficients of the mass matrix and the stiffness matrix depend on the critical damping ratio. The same value of this factor was used for all the structure's free vibration.
Previous researches provided some indicative values of the damping factor, depending on: the structure's material, the predictable stresses level and connection's typology (in particular for steel structures). The following values of the damping ratio were used in the parametrical analysis: (i) ν=2%; (ii) ν=5%; (iii) ν=10%.
In order to completely define the damping matrix, a criterion for the choice of

the multiplying coefficients of the mass matrix (α) and the stiffness matrix (β) was used. The choice of α single non-zero coefficient is not theoretically acceptable, because: greater damping corresponds to the lowest vibration modes and the stiffness matrix cannot be used in the damping definition. Results of the numerical analysis (Ref.5) and experimental data show that the stiffness matrix is physically fundamental for a correct definition of the damping matrix because the variations of the stiffness greatly influence the damping. The analysis was then conducted in the cases of $\alpha=0$, $\beta \neq 0$ and $\alpha \neq 0$, $\beta \neq 0$.

Variability of the Epicentral Direction The influence of this variability was tested using different values of the angle formed by the seismic action's direction. This angle varied from 0^{o} to 90^{o} with a 15^{o} interval. This analysis was conducted using both horizontal components of the seismic recordings.

Structural Eccentricity In order to obtain information about the spatial behavior, the structural response was assessed using different values for the eccentricity between the mass center and the stiffness center. Three values for the eccentricity were used: 3%, 6%, and 9%, of the structure's base dimensions.

ANALYSIS OF THE NUMERICAL APPLICATION'S RESULTS

Results regarding the choice of the damping factor and the constitutive model of the beam-column connections are discussed in this paragraph. Results regarding the seismic action variability and the structural eccentricity value are discussed in the conclusion.

Damping Qualitative indications can only be obtained for the influence of the damping factor. As damping increases from 2% to 10%, the structure's displacements and stresses decrease. At the same time decrease the number of plasticized elements. This decrease is not dependent on the particular accelerogram or the type of semi-rigid connections used in the test. For structures which are about to collapse, the choice of the damping factor is fundamental. Despite this, the influence of the particular seismic event is more important, and it is only this parameter that can be considered responsible for the collapse. The conducted analysis also indicated the influence of the choice of the α and β coefficients for the damping matrix definition: 1) $\alpha=0$ and $\beta \neq 0$; 2) $\alpha \neq 0$ and $\beta \neq 0$. In case 2), the structure had more plasticized structural elements, but its stress state was worse than case 1). If in the case 1) structure was far from the collapse configuration, the passage from case 1) to case 2) did not cause the structure's collapse. If in the case 1) structure was near the collapse configuration, the passage to case 2) could cause collapse.

Semi-Rigid Connections The influence of the initial elastic stiffness, the s.h. ratio, and the yielding moment on the semi-rigid connections was examined. It is interesting to analyze the results obtained by varying only the initial stiffness. Numerical results indicate that the structure with bolts and flanges showed greater values for storey displacements. A 30% increase in these values was noted when other parameters were kept constant and a change from welded to bolted structures was tried. This increase of deformability corresponded to a worsened stress state for the columns and to a better stress state for the beams and semi-rigid connections. The structural response is therefore quite sensitive to the connection's initial rigidity. Interesting results were obtained when tests conducted with and without connections were compared. In the last case, structural stiffening provided less deformability. This fact does not never imply the reduction of the structural internal forces.
Actually, the accelerogram used in the analysis is the most influential factor. For example, using the Sturno accelerograms, plasticization was less in structure without connections. When using the Tolmezzo and Bagnoli-Irpin accelerograms, less plasticization took place in structure with connections. This result can be

explained by the presence of resonance phenomena between the dominant earthquake frequencies and the structure's main natural frequencies. In fact, the structure's natural frequencies depend on the type of connections used in the mathematical model. Previous researches have already noted that 5% is the best value for the s.h. ratio. Values greater than 5% do not involve significant variation of structural response, while smaller values created some problems. For example, there were some numerical problems when s.h.=0; in this case high values for the connection's rotations, which have no physical meaning, were calculated.

CONCLUSIONS

The damping factor choice is directly influenced by the structural typology and the stress level caused by the seismic action.
When strong ground motions are applied to steel structures, a damping factor equal to 5% can be used. Results that are numerically stable and less problematic can be obtained using a damping matrix which depends only on the stiffness matrix.
Semi-rigid connections must be included in the structural scheme when an analysis with strong accelerograms is conducted, thereby, taking the structural connection's deformability into consideration. In fact, when strong dynamic forces are applied, these connections cannot be considered rigid.
The tested structure was not particularly influenced by variations in the epicentral direction. In the cases which the structure didn't collapse, a more heavy stress state were obtained when the epicentral direction was parallel to one of the building's principal directions.
The obtained results were not quantitatively influenced by different values used for the eccentricity between mass center and stiffness center. In fact, in the examined cases, nearly storey displacement and total internal structural stress state values were obtained. Only the value of the storey rotation showed some differences.
In the end, the structural response was mainly influenced by the applied accelerogram's characteristics, and by its spectral content. Structural collapse takes place when the dominant earthquake frequencies are very similar to the first natural frequencies of free vibration (Figs.4,5). Analysis indicate that the structure "recalls" its elastic behavior even when it is in the nonlinear field. This result is very interesting, especially for structural analysis conducted in the time domain.
Accelerograms can be obtained by seismic event recordings or by response spectra, and should be chosen for emphasising significant responses of the structure. Further research should be conducted so that also new seismic design codes proposals addressing the above stated needs.

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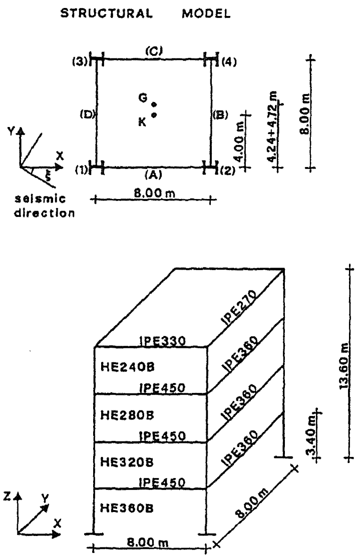
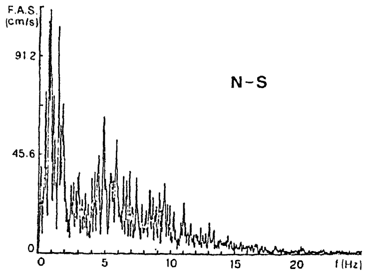


← Fig.1
$f_1 = 0.99$ Hz



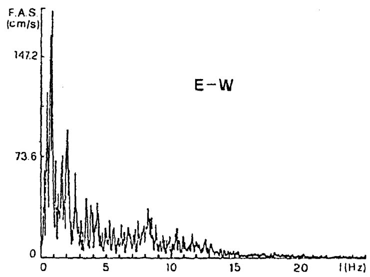


Fig.2 - Fourier amplitude spectra of acceleration (Bagnoli Irpino Earthquake)

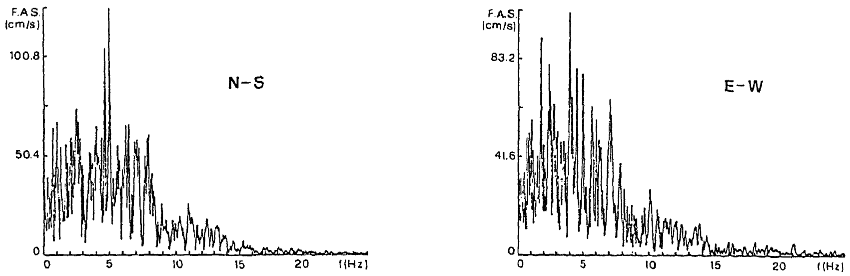
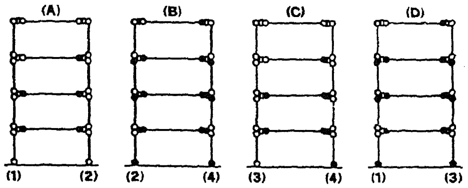


Fig.3 - Fourier amplitude spectra of acceleration (Brienza Earthquake)

Accelerogram	Direct. ξ	Damping	Connections	Eccentr.
Bagnoli Ir. N-S E-W	90	ν=5% α=0 ; β≠0	Welded	3%



← Fig.4

No. yielded columns	13	No. yielded beams	24	No. yielded connections	6

Xmax: 10.9 cm Ymax: 45.1 cm Rmax: $19.8 \cdot 10^{-4}$ rad

Accelerogram	Direct. ξ	Damping	Connections	Eccentr.
Brienza N-S E-W	90	ν=5% α=0 ; β≠0	Welded	3%

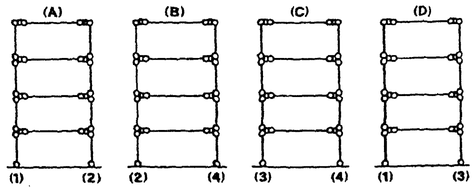


Fig.5 →

No. yielded columns	0	No. yielded beams	0	No. yielded connections	0

Xmax: 4.8 cm Ymax: 7.4 cm Rmax: $5.8 \cdot 10^{-4}$ rad