3D BEHAVIOUR OF RC LOAD BEARING WALLS

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SUMMARY

Shaking table tests on RC wall structures have been performed in the past as a part of French and European programs. The data provided by these experimental programs have been used to improve and validate refined models integrated on FE codes and to set up simplified modelling useful for engineers. Although extremely valuable, only loadings in one direction (in the plane of the walls) were applied. More recently, the CAMUS 2000 French program allowed to treat the case of a bi-directional loading.

An extensive numerical campaign was devoted to the analysis of these results. To reduce the time and the request of computing resources related to the realistic dynamic description of the material failure under such a complex loading, a simplified modellisations, coupled with explicit procedures, has been adopted.

After the verification of the reliability of both simplified models, based on the use of 3D beams only, and explicit methods in describing the seismic behaviour of reinforced concrete wall structures, the numerical investigation has been extended to the introduction of a seismic load in the vertical direction and to the effects of the soil conditions on the response of the structure.

INTRODUCTION

Lightly reinforced concrete bearing walls are commonly used in France and in other European countries for building structures. As reported by Mazars [1], research to date on this type of walls (the CASSBA and CAMUS French programs) has shown that such kind of construction exhibits good behaviour under seismic loadings, in spite of its limited ductility capacity (reduced amount of reinforcement concentrated at the edges of the walls with a large portion of the structural members with practically no reinforcement). The results of these experimental campaigns and of numerical simulations have also highlighted some unconventional mechanisms of earthquake resistance, such as the rigid block-type rotation of the walls, or part thereof, that take place at the interface between the foundation and the soil or at the level of cold construction joints. The opening and closing of horizontal cracks caused by this behaviour mechanism results in the excitation of high frequency vertical vibrations. On the other hand, in real structures such walls are likely to be loaded in the out of plane direction as well. Due to the out of plane loading, to the

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previously mentioned effects an additional fluctuation of the axial force may arise and more damage is to be expected since cracks may not completely close at load reversal. It is therefore easily understood that efforts have to be focused on the experimental study of these phenomena and on the calibration of numerical models able to capture in a satisfactory way the spatial seismic behaviour of such a kind of structures.

Based on the results of an experimental campaign carried out within the CAMUS 2000 French program (Bisch [2]), a simplified numerical model has been calibrated and successively used to investigate the effects of the application of seismic loads in multiple directions and of the influence of soil conditions on the global response of wall structures. The results of this study are presented in detail in the paper.

THE CAMUS 2000 RESEARCH PROGRAMME

The 3-years experimental and numerical research program CAMUS 2000 was launched in 1998 with the aim of evaluating the effects of torsion on the response of RC structures and the behaviour of lightly R/C walls subjected to bi-directional motions (Bisch [2]). As a part of the program, two scaled (1/3) models representative of 5 storeys reinforced concrete buildings have been tested on the major Azalee shaking table of Commissariat à l'Energie Atomique (CEA) in the Saclay Nuclear Centre. The design of the mock-ups was made according to the "multifuse" concept commonly used in France that privileges diffuse rupture at several stories instead of concentration of damage at the base of the building and creation of one plastic hinge. Low percentages of reinforcement combined with an appropriate distribution at several levels helps to dissipate energy via wide crack patterns at different heights of the wall and lead in that way to the multiplication of the dissipation zones [3].

The first structure (CAMUS 2000-1) has been subjected to a horizontal bi-directional excitation. A set of accelerograms of increasing amplitude (effective acceleration equal to 0.15g, 0.22g, 0.25g, 0.40g, 0.55g and 0.65g) was applied. The structural stiffness was provided by two identical shear walls in one direction and a steel bracing system in the orthogonal direction. For the second test (CAMUS 2000-2), an in plane excitation was applied. A torsional response was induced by the asymmetry in the horizontal dimensions of the two walls.

The paper presents the results of the extensive numerical campaign devoted to the analysis of the CAMUS 2000-1 experimental work.

Test Specimen Geometry
The 1\3-scale test specimen consists of two reinforced concrete shear walls with no openings, linked by 6 square floors (see Figure 1) supported on a reinforced concrete footing and tied down to a shaking table. The walls are 6 cm thick, 1.70 m wide and 0.90 m tall per storey. Construction joints are created just above the middle of each floor by casting the walls in separate pours. Floors are 0.21 m thick, 1.70 m long and 1.70 m wide. A transverse bracing system has been provided in order to increase the stiffness in the out-of-plane direction. Due to similarity laws additional masses of 6.55 t were positioned at each story as shown in Figure 1.

Vertical wall reinforcement is concentrated at each end of the walls and is confined by a light transverse reinforcement. At the base of the walls the reinforcement consists of six 4.5 mm bars, confined by 3 mm stirrups spaced at 4.5 cm. The reinforcing bars terminate at different distance from the top floor.

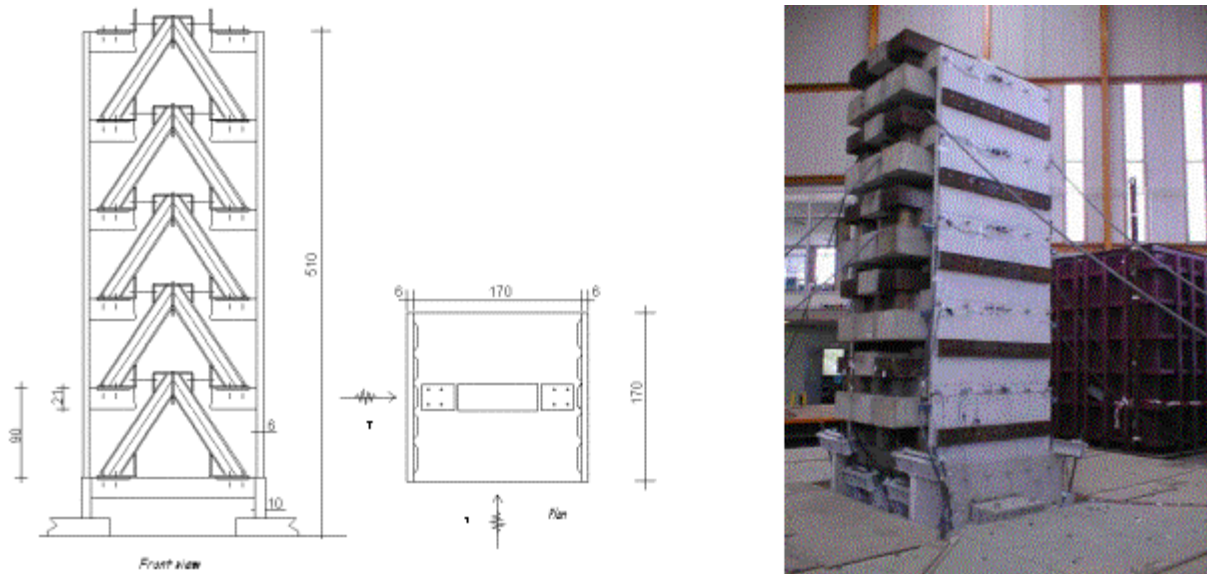# BASIC ASSUMPTIONS ADOPTED FOR THE NUMERICAL SIMULATIONS

Figure 1: Lay-out of the specimen and distribution of mass blocks on the floors.

General assumptions

Simplified models based on explicit formulations
Various modelling strategies have been proposed up to now for the non-linear analysis of R/C structures. Their level of complexity is usually proportional to the dimension of the problem. Detailed 2D or 3D finite element models with local constitutive relationships are typically used for predicting the response of structural elements or substructures, whereas simplified global or local models are useful for the dynamic response analysis of structures. The use of complex FE discretisation and advanced constitutive models demands for large scale computing, which must be feasible from the view of both time and available resources. In this light the use of simplified modellisations coupled with explicit procedures became very popular for solving time dependent problems. Together with the limited number of numerical elements used in simplified approaches, the employment of explicit procedures contributes to a general reduction of the time requested by the calculation, since they require no iterations and no tangent stiffness matrix. Even though explicit procedures are in general conditionally stable, their stability is automatically assured by the small time increments required by the solution of dynamic problems. The analyses reported in this paper have been carried out by using the explicit version of the commercial computer code ABAQUS version 6.1.

Modellisation of the shaking table
To account for the filter action due to the table, and the possible alterations of the structural response due to the interaction with the dynamics of the supported model, the shaking table has been represented in the numerical model as a rigid body supported by springs, the stiffness of which has been provided directly by CEA.

Input motion
For the reliability and reproducibility of the results, the accelerograms measured at the centre of the table have been imposed in the numerical analyses.

Material modelling

Steel
A classical metal plasticity model based on the use of the Mises yield surfaces has been adopted to describe the behaviour of the reinforcing bars. Hardening is defined by using the isotropic hardening model: the yield surface changes size uniformly in all directions such that the yield stress increases (or decreases) in all stress directions as plastic straining occurs.

Concrete
The cyclic behaviour of concrete has been represented by using the PRM model (Mazars [4], Rouquand [5, 6]). The analyses has been carried out with ABAQUS and the explicit version of the PRM model running in parallel. The PRM model is based on constitutive laws able to account for the effects of stiffness deterioration, recovery of stiffness due to crack closure, effects of damage and strain rate. The general formulation of the model is reported below:

$$(\boldsymbol{\sigma} - \boldsymbol{\sigma}_{ft}) = \boldsymbol{\Lambda}_0 \cdot (1\text{-}D) \cdot (\boldsymbol{\varepsilon}\text{-}\boldsymbol{\varepsilon}_{ft})$$

were $\boldsymbol{\sigma}_{ft}$ and $\boldsymbol{\varepsilon}_{ft}$ are the crack closure stress and strain thresholds used to manage permanent effects; $\boldsymbol{\Lambda}_0$ is related to the initial Young modulus and $D=\alpha_t D_t + \alpha_c D_c$ is the damage variable, defined as a linear combination of damage in tension (D_t) and damage in compression (D_c). A scheme of the model is reported in Figure 2.
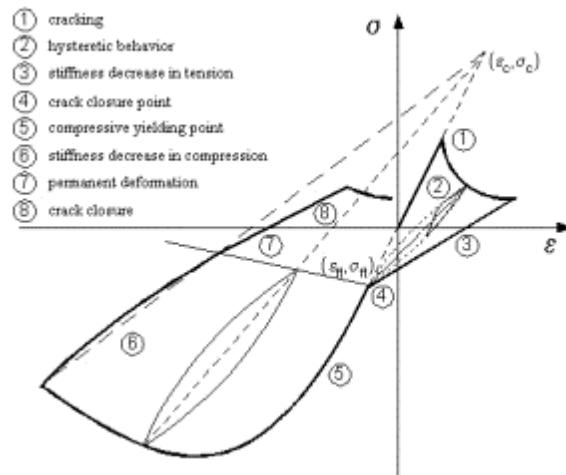

Figure 2: PRM model: stress-strain relationship.

Model description

Key characteristics of beam elements
The main characteristics of the 3D beam element chosen for the modellisation of the spatial behaviour of the structure are summarised in the following paragraphs.

- *Transverse shear behaviour.* The elements used in the analysis are Timoshenko beams that allow for transverse shear deformation. The transverse shear deformation in these elements is treated as if the response were linear elastic, independent of the axial and bending responses, even for non-linear material behaviour.

- *Mass formulation*. The linear Timoshenko beam elements adopted use a lumped mass formulation.

- *Integration*. The cross-section of each beam is integrated numerically to obtain the force-moment/strain-curvature relations for the section. The complete generality in material response is assured, since each point of each section (see Figure 3) is considered independently by the constitutive routines.

Modelling hypothesis

Even though the same type of numerical element has been used to represent the various structural components, a different section has been chosen for all of them, as shown in Figure 3. The mechanical properties of concrete and steel evaluated by performing standard tests (see Table 1) have been used in the analysis.

Table 1: Mechanical characteristics of concrete and steel.

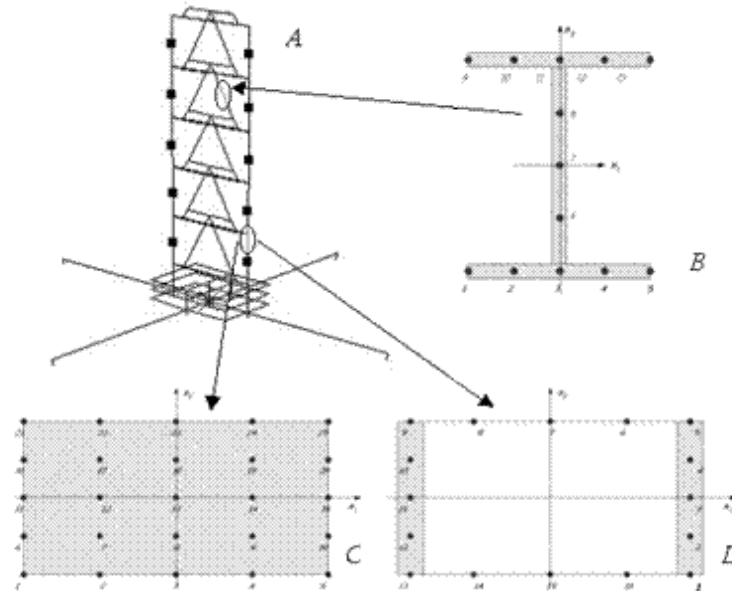| Concrete				
Wall	*Casting phase*	*Age (days)*	*Compression (MPa)*	*Young Modulus (GPa)*
1	1	28 / 210	16.2 / 24.9	/ 25.0
	2	28 / 209	24.4 / 35.0	/ 28.8
2	1	28 / 203	25.3 / 31.4	/ 25.7
	2	28 / 202	28.3 / 38.0	/ 30.4
Steel				
Diameter (mm)	Area (mm2)	Fy (N/mm2)	Fu (N/mm2)	Elong. 5 diam. (%)
4.5	15.90	664	733	13.3
6	28.27	585	617	16.7
8	50.27	563	591	15.9
10	78.54	562	629	26.0

Figure 3: A) Example of the model; B) Section shape adopted to model the bracing system; C) Rectangular section element for concrete members; D) Box section used to model the reinforcement and the mass blocks on the floors. Position and number of the integration points in each section are also shown.

- *Walls*. Rectangular section elements were chosen to model the concrete components of the walls. Each section is characterised by 25 integration points.
  The longitudinal reinforcement is accounted for by means of box section elements. Couples of bars at the same distance from the axis of the structural member were modelled by using the same numerical element – the area of the reinforcing bars has been transformed in an equivalent area of the box section-. This choice allowed the representation of the walls to be done directly by the geometrical description of their axis. Each box section is characterised by 16 integration points.
  The additional masses placed along the walls have been modelled as concentrated masses.

- *Slabs*. Rectangular and box section elements have been used to describe the behaviour of the six slabs. Box sections has been utilised in the central part of the slabs, where a hole has been realised to allow the connection between the bracing system of two adjacent storeys.
  As for the walls, the reinforcing bars have been accounted for through the use of box sections.
  The mass blocks added to the floors have been modelled by using fictive box sections: the density of the material has been chosen to represent the real weight of the blocks, whereas the stiffness of the elements has been limited, in order to avoid their influence on the dynamic behaviour of the structure.

- *Bracing system*. The bracing system has been modelled by using I-section elements. The geometry of the numerical model corresponds to the steel section adopted in reality. Each section is characterised by 13 integration points.

- *Basement*. A network of elements has been used to simulate the stiffness of the basement in the three directions. As for the other structural elements, rectangular sections were adopted for concrete, whereas the reinforcement has been modelled by using box sections.
  The basement of the specimen is connected to the table through four beam elements with circular section. A preload is applied to these components so as to be close to the reality.

- *Shaking table*. A rigid body composed by beams with rectangular section has been used to describe the behaviour of the shaking table. Circular beam elements, placed perpendicularly to the shaking table, have been utilised to simulate the deformability of the table. The value of the axial deformability of these elements has been provided by CEA. The mass of the table has been accounted for by defining an appropriate material density.

An example of the mesh used in the analysis is reported in Figure 3.

Type of analysis

The study presented in this document has been carried out in different steps. A modal analysis has been performed at the beginning of the calculation to test the validity of the adopted model. Dynamic analyses carried out by imposing a sequence of three earthquakes in the two horizontal directions allowed the reliability of the model to be demonstrated.

Following this validation, the model has been used to derive some information concerning the relative effects of the application of a seismic load in one, two and three directions. The influence of the flexibility of the shaking table on this comparison has been investigated by performing set of analyses corresponding to three different value of flexibility: the reference value provided by CEA, ten times and one third of it. The reason behind this part of the study is that even tough in an extremely simplified way, the flexibility of the table could represent the soil condition.
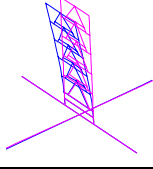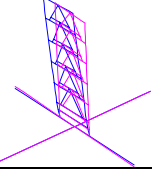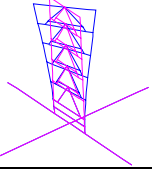
VERIFICATION OF THE RELIABILITY OF THE MODEL

Modal analysis

A first verification of the reliability of the model has been done by performing a modal analysis. For this analysis the concrete has been modelled to behave in an elastic way.
The first two frequencies obtained with the numerical model (5.5 and 6.02 Hz for the out-of-plane and the in-plane flexure) resulted to be quite close to those measured (5.45 and 6.00 HZ respectively) as shown in table 2.

Table 2: Measured and computed frequencies.

	In-plane flexion	Out of plane flexion	Torsion
Mode Shape			
Experiment	6,00 Hz	5,45 Hz	-
Computation	6,02 Hz	5,50 Hz	12,56 Hz

Dynamic analyses

Six seismic signals of increasing amplitude (0.15g, 0.22g, 0.25g, 0.40g, 0.55g and 0.65g) have been applied in sequence during the experimental campaign. Among them, only those corresponding to an intensity of 0.15g, 0.4g and 0.55g have been considered in the analytical study to reduce the time required by the calculation. As for in reality, the signals have been applied in series. Experimental and numerical results have been compared at local and global level.

As an example of the results obtained, the time histories of top displacement, axial load and shear load evaluated corresponding to the intensity of 0.4g and 0.55g are reported in Figure 4 and 5. From the graphs it can be observed that, in spite of the simplified approach adopted in the analysis, numerical and experimental data result in a satisfactory agreement. Some differences between experimental and numerical curves (mainly for what concern the top displacement) can be detected in the graphs corresponding to 0.4g. The authors believe that the reason for these dissimilarities can be found in the damage that might be suffered by the specimen during the application of the signals not considered in the analysis. The better agreement observed in the graphs corresponding to 0.55g, intensity of the last signal in the series of the imposed load, seems to verify this assumption.

As can be noted analysing the graph representing the axial load, the horizontal components of the accelerograms not only induce an overturning moment, but also a dynamic (vertical) axial force which can be of the same order of magnitude of the weight (plus or minus) of the structure (36 tons).

A complex state of stress inside the walls caused by the combination of bi-directional flexure and shear similar to that observed during the tests was also highlighted by the numerical analyses. The damage theory implemented in the PRM model used for concrete allowed the study of the variation of damage suffered in different points of the section. As an example, the damage due to compression suffered by the

left wall in the four corners of the section, for the 0.55g excitation, is plotted in Figure 6. From the graphs it can be observed that compression causes damage only in the first storey, ed in particular close to the basement and to the connection with the slab of the first storey, where the maximum damage occurs (during the tests damage was observed in the same location). On the contrary, the effects of tension are maximum at the base of the walls, and decrease in the upper storeys.
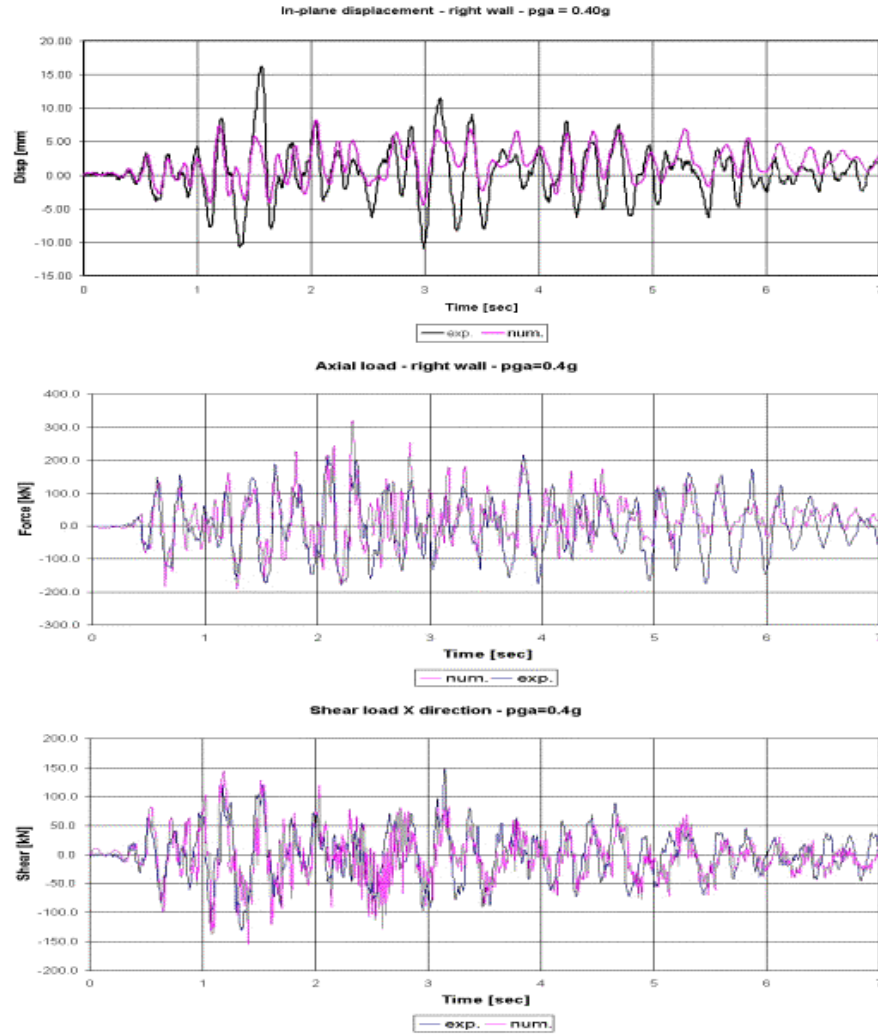

Figure 4: Comparison between measured and computed time histories of top displacement, axial load, shear load and moment corresponding to the signal of 0.4g.

EFFECTS OF THE APPLICATION OF THE LOAD IN MULTIPLE DIRECTIONS

As a consequence of the agreement between numerical and experimental results, the analytical model was used to derive information about the spatial behaviour of structures subjected to a given 3D earthquake. In order to better understand the effects of the introduction of the third direction of load, the structural behaviour caused by the application of the load in one, two and three directions has been investigated. The study is executed for the amplitude of 0.15g, 0.40g and 0.55g. The same series of seismic excitations has been used for the three situations. The magnitude of the load in the vertical direction has been chosen to correspond to the 70% of the horizontal one.
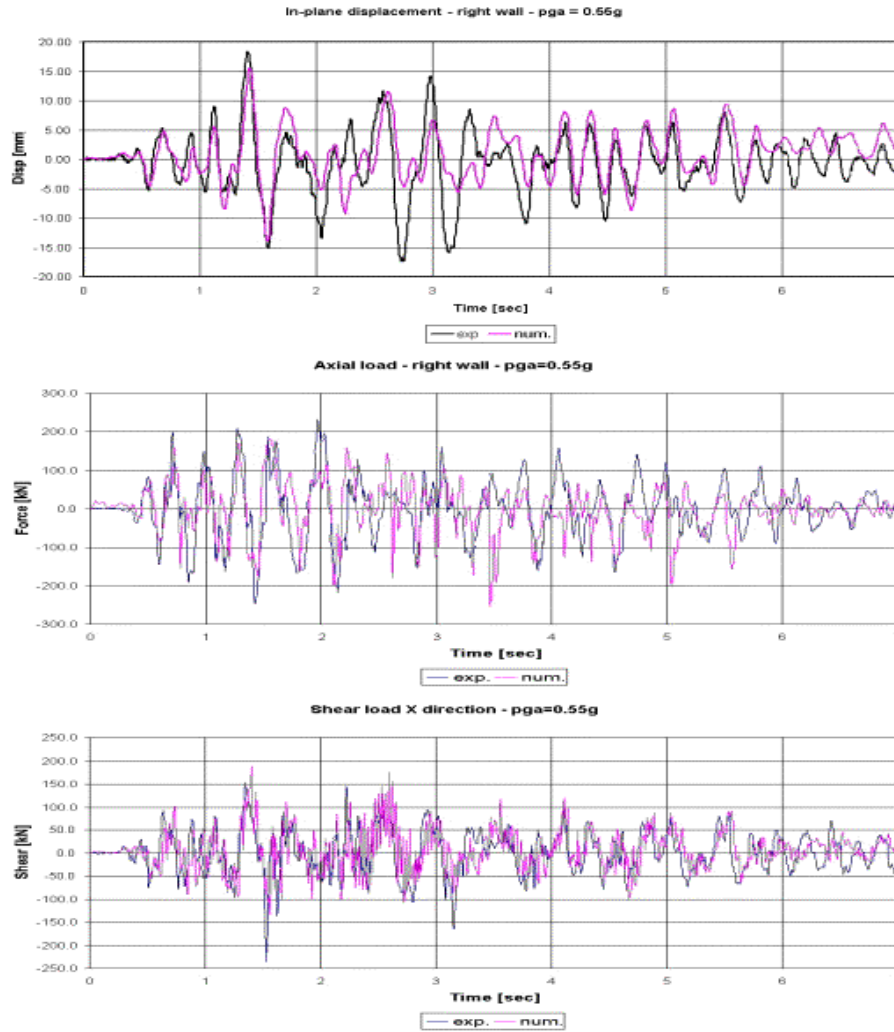
Figure 5: Comparison between measured and computed time histories of top displacement, axial load, shear load and moment corresponding to the signal of 0.55g.

The series of analyses has been carried out three times, corresponding to three different values of the flexibility of the shaking table: the reference value provided by CEA, ten times and one third the reference value. The comparison among the responses of the specimen in the different configurations allowed the evaluation of the influence of the flexibility of the shaking table on the obtained results.

Effects of the loading conditions

The effects of the loading condition on the seismic response of the specimen have been analysed in terms of maximum values. The values obtained for the two single horizontal directions are compared with those evaluated corresponding to an excitation in two and three directions simultaneously. The results of the analyses demonstrate that the application of two horizontal excitations simultaneously induces an asymmetric behaviour of the structure. The difference between the response of the two walls is mitigated by the introduction of the vertical component of the motion. This asymmetry becomes less significant during the application of the set of accelerograms at increased amplitude, i.e. on a damaged structure.
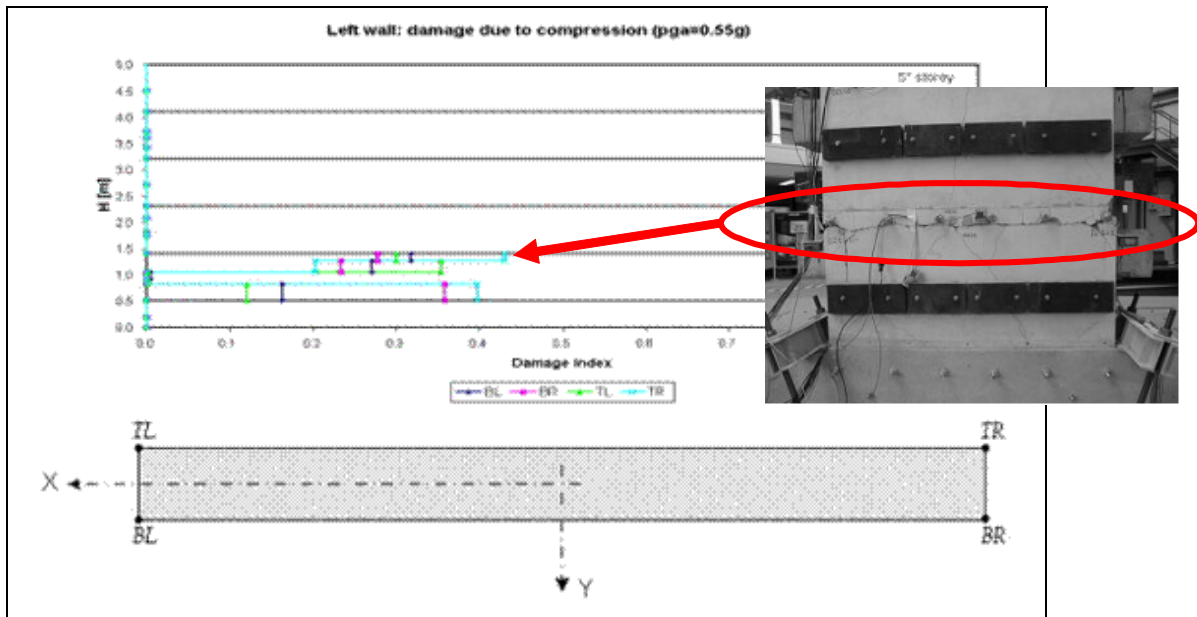
Figure 6: Comparison between damage observed during the tests and computed (component due to compression) at the four corners of the left wall (signal corresponding to an intensity of 0.55g).

The value of shear at the base of the two walls is represented in Figure 7. The loading condition, for an intensity of 0.4g, does not influence significantly the value of the shear force. Some differences appear only in correspondence to the bi-horizontal excitation when the amplitude of the load increases at 0.55 g.

One quantities strongly influenced by the loading condition is the axial load, as shown in Figure 8. The graphs show that a horizontal excitation in the direction parallel to that of the walls causes a negligible variation of the axial load. On the contrary, a seismic load in the direction perpendicular to that of the walls produces a variation of the axial load of the same order of magnitude of the weigh of the specimen. Due to this effect, during the excitation the walls are subjected to a complete unload (even to a negative load, i.e. tension), and to a total vertical load equal to the double of the dead load. For the case of a damaged structure, situation the results evaluated for 0.55g correspond to, the application of a bi- and a three- directional loading limits the variation of the axial load. In particular, the vertical excitation reduces the fluctuation of the axial load up to the half.
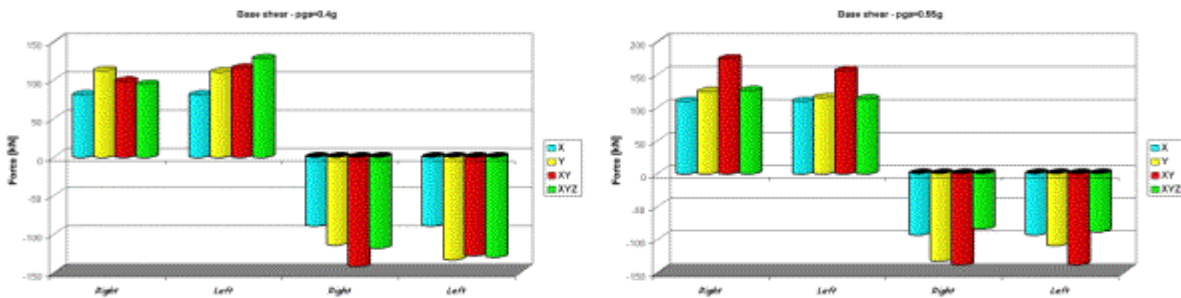

Figure 7: Effects of the loading conditions on the value of shear at the base of the walls.
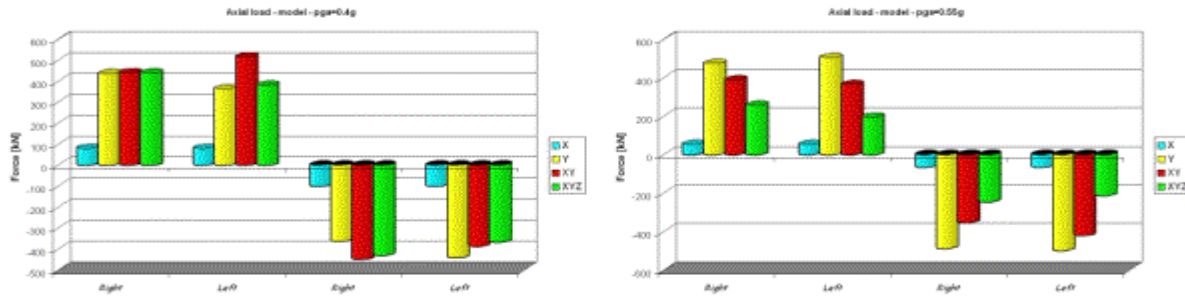
Figure 8: Effects of the loading conditions on the value of axial load in the two walls.

These results allow a verification of the procedure suggested by Eurocode 8 [7] for the combination of the horizontal components of the seismic action (paragraph 4.3.3.5.1 (3)) when the complying with the regularity criteria allow an analysis performed by using two planar models (action effects due to the application of the seismic load in one direction plus the 30% of the action effects due to the application of the seismic load in the orthogonal direction - $E_x + 0.3E_y$ and $E_y + 0.3E_x$).

The values of axial load and shear calculated at the base of the two walls corresponding to the application of a bi-directional motion are compared in Figure 9 with those evaluated respecting the methodology proposed by EC8. From the figure it can be noted that for the case of shear both formulations ($E_x + 0.3E_y$ and $E_y + 0.3E_x$) provide a value close to that evaluated with a 3D analysis for a structure in its original (undamaged) configuration. They underestimate the value of base shear by the 50% for the case of a damaged structure.

As already mentioned, the out-of-plane excitation is the main responsible for the variation of the axial load. According to this fact, the combination considering only the 30% of the effects due to the out-of-plane excitation provides a value four times lower than the real one. On the contrary, the other expression allows to calculate a variation of the axial load of the same order of magnitude of that corresponding to a bi-directional load. This consideration can be extended also for the case of a damaged structure.

The analyses demonstrated that the loading condition strongly influences the stress state of the two walls. An example of those effects is shown in Figure 10, where the values of damage due to tension suffered by the two walls are plotted for the three loading conditions. The graphs confirm that the introduction of a load in the second horizontal direction produces an asymmetric response of the structure due to the superposition of flexure in the two orthogonal directions. This causes a dissimilar concentration of damage in the structural members. An increase in the intensity of the applied earthquake results in the progression of damage caused by tension along the high of the structure, whereas damage in compression appears at the interface with the slab of the first storey. The introduction of the vertical seismic load intensify the differences, even tough to a less extent, in the stress-state of the two walls. Damage in tension progresses trough the high of the structures and cracks caused by tension stresses appears in correspondence to the highest storeys.

Effects of the deformability of the table

To account for the filter effect caused by the table, its behaviour has been represented in the numerical model by using a rigid body composed by two beams standing on a series of elements axially deformable. This allowed the study of the effects of the deformability of the shaking table on the response of the whole model. Even tough in an extremely simplified way, the table flexibility could be also considered to be

representative of the soil conditions. Three different values have been used: the reference value corresponding to the real flexibility of the shaking table (K_{ref}), ten times (K_{stiff}) and one third (K_{soft}) the reference value. The value of K_{stiff} has been chosen in order to represents the limit above which the first ten frequencies of the model remain practically constant. The value of K_{soft} is a compromise between the contribution of the mock-up and of the table to the global response of the model.
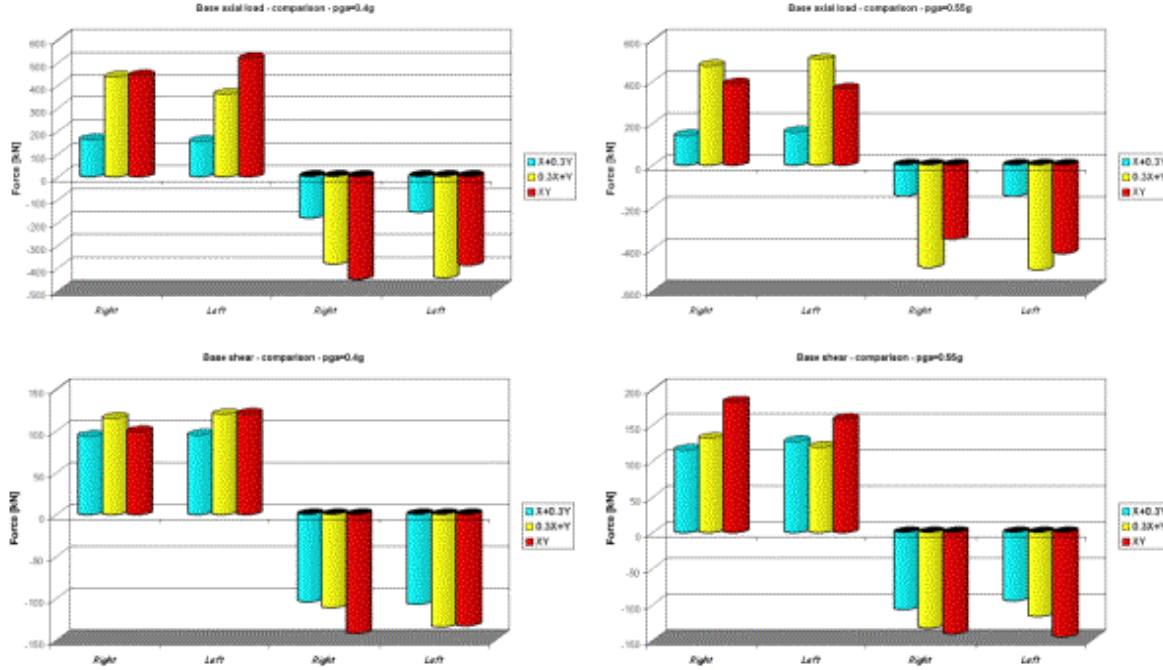

Figure 9: Variation of the axial load and of the shear force in the two walls for excitations of 0.4g and 0.55g: comparison between a 3D analysis and the combination of effects prescribed by EC8.

The change of the flexibility of the shaking table results in a change of the natural frequencies of the model. A summary of the first three frequencies (out-of-plane flexure, in-plane flexure and torsion) of the model in the three configurations is reported in Table 3. The value of the frequency corresponding to the vertical excitation of the table (pumping) is also reported.

In spite of the change in the global response of the models caused by the variation of the spectral accelerations induced by the modification of the natural frequencies, only the values of relative storey displacement and axial load show significant differences as for the previous analyses. A reduction of the value of the displacement corresponds to the use of a stiff basement, whereas larger relative displacements appear in a soft model. The introduction of an excitation in the vertical direction tends to reduce the differences among the models. The response of the three models becomes comparable for the case of a damaged structure.

Table 3: Frequencies of the model corresponding to the three different configurations.

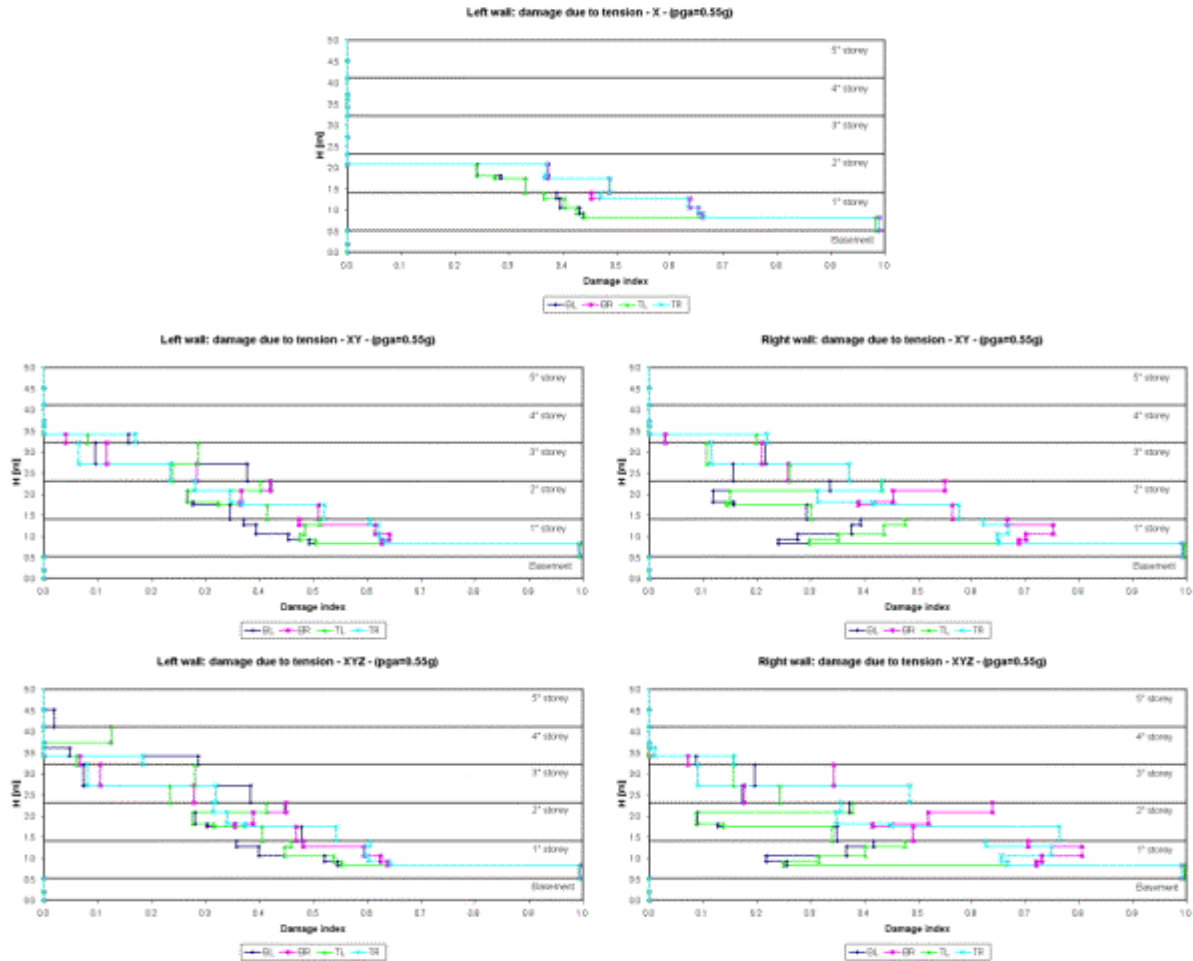|	Reference model K=K_{ref}	Flexible model K_{soft}=K_{ref} / 3	Stiff model K_{stiff}=K_{ref} * 10
Out-of-plane flexure [Hz]	5.50	5.14	5.82
In-plane flexure [Hz]	6.02	5.48	6.33
Torsion [Hz]	12.56	13.34	13.44
Pumping [Hz]	22.52	14.37	34.03

Figure 10: Comparison among the damage due to tension suffered during the application of different loading configurations (pga=0.55g) by the two walls.

The variation of the axial load has been analysed in two positions: at the base of the structure –this allowed the evaluation of the effects of the flexibility of the table on the structural response- and at the base of the walls –the analysis of the values in this second position provides an estimation of the effects of the damage in the response of the structure.

The extreme values of the variation of the axial load in the two positions is reported in Figure 13. The results corresponding to an excitation with pga = 0.4g in one and two horizontal directions show that the variation of the axial load is larger for a stiff basement. On the contrary, a three directional loading imposes an higher value on the soft model. This might be due to the excitation of the vertical mode (pumping). The differences among the three configurations tend to be reduced for the case of a damaged structure.

The time histories of the variation of the axial load for the three configurations corresponding to the three load conditions are reported in Figure 14. The responses for the two level of intensity are plotted in series. Observing the value of the load at the end of each excitation an estimation of the redistribution of the dead load between the walls and the bracing system can be obtained. The more important redistribution appears for the case of a load in the direction parallel to that of the walls. After the excitation at 0.4g the walls are

overloaded (the value of the axial load is positive). On the contrary, during the excitation at 0.55g an important part of the load is transferred from the walls to the bracing system. This might be caused by the appearance of a same localised damage in the two walls, phenomenon that could also explains the drop in the value of the axial load that can be observed in the time history after one second from the beginning of the excitation. The redistribution of dead load between walls and bracing system is less important when a load in two or three directions is applied.
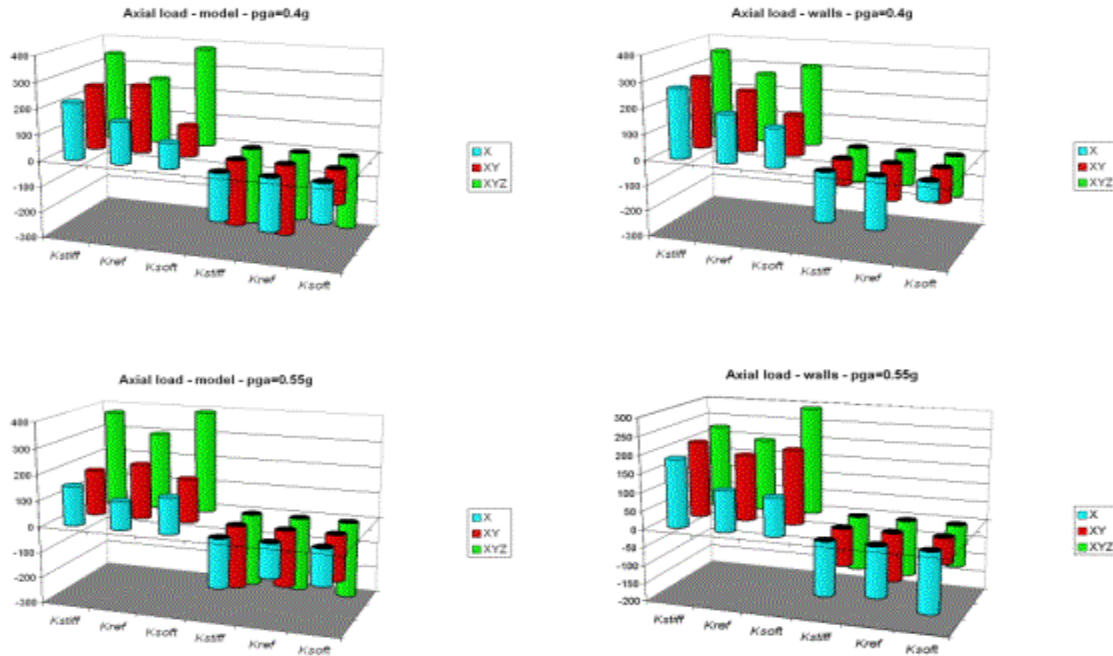

Figure 13: Effects of the table deformability on the variation of the axial load at the base of the specimen and at the base of the walls.

CONCLUSIONS

The dynamic behaviour of a five-storeys shear wall building has been simulated in this study by using a simplified analytical model based on the use of Timoshenko beam elements only. The results reported in the paper provide an additional verification of the efficacy of simplified modellisations in describing the global dynamic behaviour of reinforced concrete structures. The reduced accuracy characterising the results at local level (e.g. distribution of damage, elongation of reinforcement bars) is balanced by their reduced computational cost. This aspect takes a fundamental importance when there is a need for parametric studies or vulnerability analyses, as reported by Negro [8].

The analyses carried out provided some information about the dynamic behaviour of lightly reinforced concrete walls: a) The out-of-plane excitation can significantly increase the flexural demand of reinforced concrete walls with limited reinforcement, mainly because of the increase in the variation of the axial force. b) The effects of the different loading conditions are more significant on the value of the relative storey displacement and on the variation of the axial load. c) The combination of the effects of the horizontal components of the seismic actions proposed by Eurocode 8 provides values similar to that evaluated by performing a 3D analysis for undamaged structures. The value is underestimated by the 50% for the case of a damaged structure. d) The table flexibility, considered representative of the soil conditions, affects mainly the value of storey displacement and axial force. In particular, the variation of the axial load is more important corresponding to a stiff basement when a horizontal excitation is applied

in one and/or two directions. On the contrary, a tri-directional motion excites the vertical mode (pumping) of structures placed on soft basements, causing a large variation of the axial load.
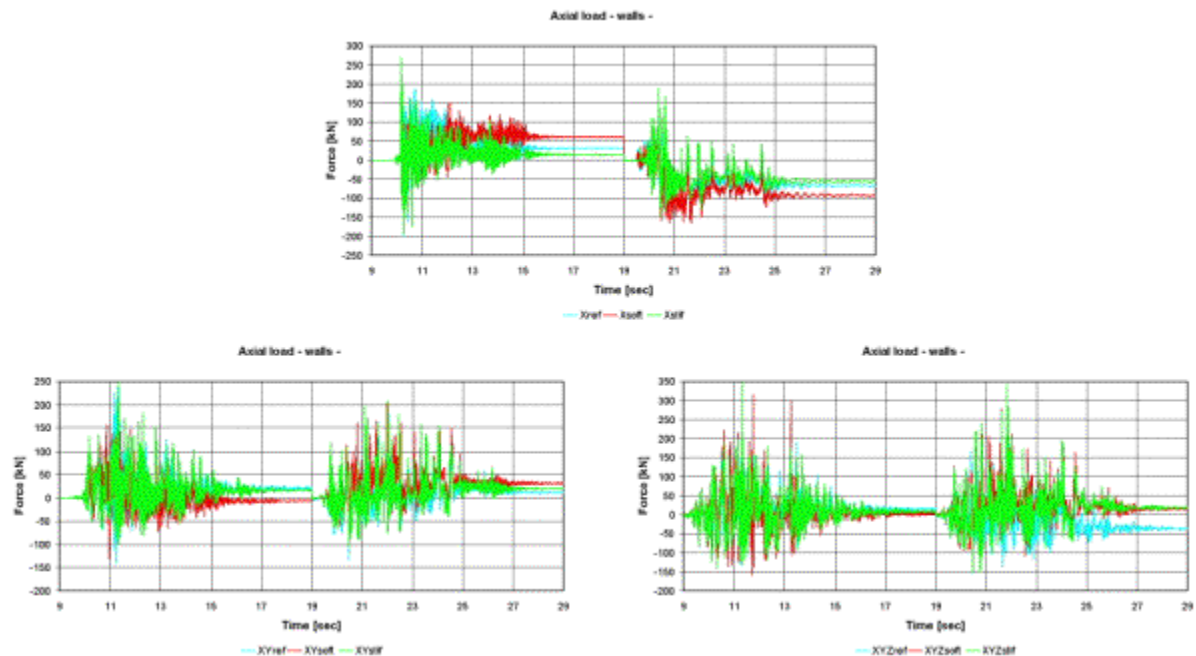

Figure 14: Time histories of the variation of the axial load at the base of the walls.

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