ANALYSES OF SEISMIC BEHAVIOUR OF STRUCTURAL WALLS WITH VARIOUS REINFORCEMENTS AND BOUNDARY CONDITIONS

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SUMMARY

Within the framework of the ICONS-TMR research project two five-story buildings, the so-called CAMUS III and CAMUS IV specimens, have been tested under dynamic loading in CEA Saclay. Predictive numerical calculations have been made for the two projects using a multi-layered finite element code (EFICOS) and the DYNAFLOW code. The results are compared with those of the previous French research program CAMUS I, a mock-up with the same geometry. Special attention has been paid on the influence of different reinforcement ratios (CAMUS III - CAMUS I) and boundary conditions (CAMUS IV - CAMUS I).

INTRODUCTION

This paper deals with the influence of different reinforcement ratios and boundary conditions on the response of reinforced concrete structures submitted to seismic loading. Within the framework of the ICONS-TMR research project two five-story buildings, the so-called CAMUS III and CAMUS IV specimens, have been tested under dynamic loading in CEA Saclay. Predictive numerical calculations have been made for the two projects using a multi-layered finite element code (EFICOS) and the DYNAFLOW code. The behaviour of the specimens is compared with that of CAMUS I, a previous French research program.

CAMUS I was designed according to the French PS92 seismic design code and CAMUS III according to EC8 but for the same ultimate moment capacity at the base. The two mock-ups were anchored to the shaking table. The comparison aims to show the influence of the different reinforcement ratios and to oppose the "monofuse" and the "multifuse" concepts that govern the two codes.

CAMUS IV mock-up designed according to the French code PS92 (same reinforcement as CAMUS I) was sitting on a sand layer and uplift was allowed. The aim of the test is to analyse the effect on structural response of soft boundary conditions. The structural behaviour during uplift is compared to that of CAMUS I mock-up, where neither sliding nor uplift is allowed (stiff boundary conditions).

CAMUS I SPECIMEN DESCRIPTION

CAMUS I is a 1/3 scaled mock-up of two parallel 5-floor reinforced concrete walls without opening (Figure 1). 6 square floors link the walls. A heavily reinforced concrete footing allows the anchorage to the shaking table. The total height of the mock-up is 5.1 m and the total mass is estimated at 36 tons. Each wall is 1.70m long and 6 cm thick. CAMUS I was designed according to French PS92 seismic design code and the reinforcement is shown in

Table 1. More details about the experimental program and its results can be found in [CAMUS International Benchmark 1997], [Queval and al. 1998].

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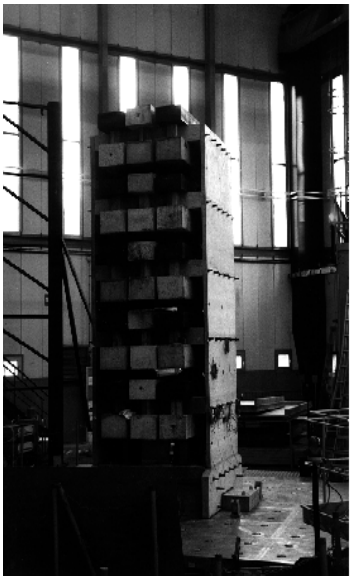


FIGURE 1. CAMUS I

TABLE 1. Reinforcement - CAMUS I mock-up

	Wall Boundaries	Central reinf.
5th storey	(1d4.5)*2=32mm^2	4d5=78.4mm^2
4th storey	(1d6)*2=57mm^2	4d5=78.4mm^2
3rd storey	(1d8+1d6+1d4.5)*2=189mm^2	4d5+2d4.5=110mm^2
2nd storey	(2d8+2d6+2d4.5)*2=378mm^2	4d5+2d4.5+1d6=138mm^2
1st storey	(4d8+2d6+2d4.5)*2=579mm^2	4d5+2d4.5+1d6=138mm^2

TABLE 2. Reinforcement - CAMUS III mock-up

	Wall Boundaries	Central reinf.
5th storey	(2d8)*2=201mm^2	14d4.5=223mm^2
4th storey	(4d8)*2=402mm^2	14d4.5=223mm^2
3rd storey	(4d8)*2=402 mm^2	14d4.5=223mm^2
2nd storey	(4d8+2d6+2d4.5)*2=579mm^2	10d4.5=159mm^2
1st storey	(4d8+2d6+2d4.5)*2=579mm^2	10d4.5=159mm^2

CAMUS III - CAMUS I: INFLUENCE OF REINFORCEMENT RATIOS

Description of camus iii

The specimen CAMUS III has the same geometric characteristics as CAMUS I. The difference is that CAMUS III was designed according to EC8 recommendations. A bending moment capacity similar to that of CAMUS I have been adopted at the base (1st floor) but reinforcement ratios at upper levels are higher (Table 2).

Loading program

CAMUS III was tested dynamically on the AZALEE shaking table of CEA at Saclay. The loading program consisted of different signals and sequences up to collapse of the mock-up. In this paper numerical results for the Nice 0.4g sequence are presented. The synthetic Nice signal is representative of the French design spectrum and it was also used for the CAMUS I experimental program.

Numerical model

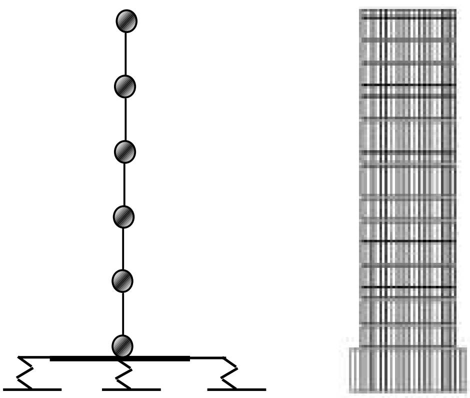


FIGURE 2. FEM model (EFICOS)

The finite element code EFICOS is used for the calculations. EFICOS was created in LMT and uses a multilayered finite element configuration. It has the advantage of using simple type finite elements (beams) divided in several layers. Constitutive laws are assumed at each layer based on damage mechanics for concrete or on plasticity for steel. The code uses the initial secant stiffness matrix algorithm where the non-linear behaviour appears in the second member of the equilibrium equation. Seismic loading is applied by the mean of an accelerogram at the basis of the structure [Ghavamian and al. 1998].

The 2-D finite element model used represents the wall by 24 beam elements with 37 layers each. A single wall is considered. To reproduce the flexibility of the shaking table three linear beams are used and the table is modelled by a horizontal stiff beam (Figure 2).

Constitutive models

The constitutive law used for concrete is based upon damage mechanics and is created in LMT [La Borderie 1991]. The model is elaborated for the description of microcraks and involves:

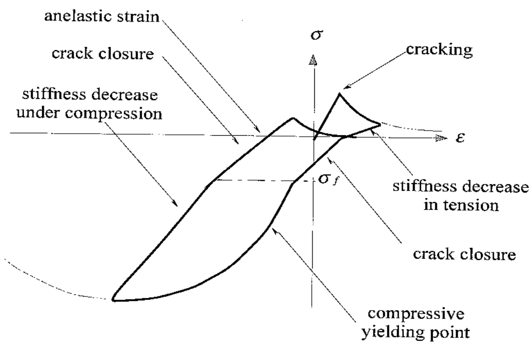


FIGURE 3. Tension-compression cycle

Two damage variables (damage in tension and damage in compression).

Inelastic (or permanent) strain coupled with damage.

Unilateral phenomena. Opening and reclosure of cracks with recovery of stiffness.

We are using a classical plasticity model with a linear cinematic hardening for the steel. Reinforcement bars are introduced with special layers, the behaviour of which is a combination of those of concrete and of steel by using a mixing homogenised law. Parameters chosen for the materials can be found in [Queval 1998], [Davenne and al. 1999].

Modal analysis

Results of the modal analysis are presented in Table 3 for the CAMUS III model anchored to the shaking table. Results are very sensitive to the flexibility of the table induced by the three linear springs.

TABLE 3. Modal analysis

	1st horizontal mode (Hz)	1st vertical mode (Hz)
Preliminary Calculations	7.25	20.0

Analysis of results

EFICOS numerical results [Kotronis and al. 1998] for the CAMUS III model subjected to the Nice signal at 0.4g are analysed hereafter.

Time history of the displacement at the top and moment at the base of the mock-up are presented Figures 4 and 5. The behaviour of the model is dominated by the first horizontal mode so the two patterns are very similar.

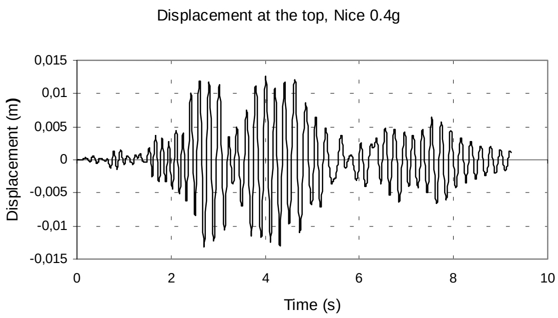


Figure 4.

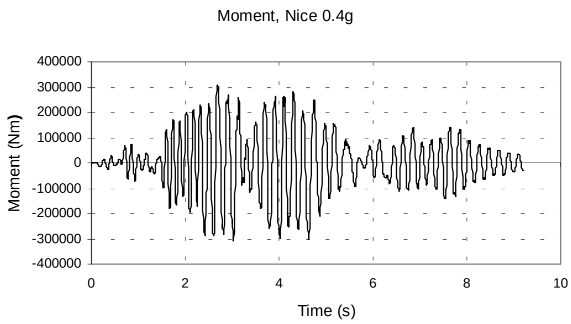


Figure 5.

Numerical calculations showed a variation of the axial force at the base of the structure for high frequencies (Figure 6: Dead weight –170,1 kN). As the cracks close, shock is induced, stiffness changes suddenly and the second mode (pumping mode) is excited. This variation of the vertical dynamic forces is important and for more important sequences it can even double or cancel the dead weight of the mock-up. Figure 7 presents the variation of the moment at the base and the dynamic variation of the axial force (referred to a zero initial value this time). When displacement reaches a value near zero, and so do moments, cracks closes and sudden oscillation of the axial force are induced.

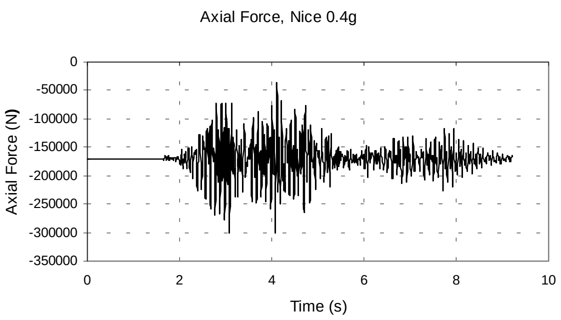


Figure 6.

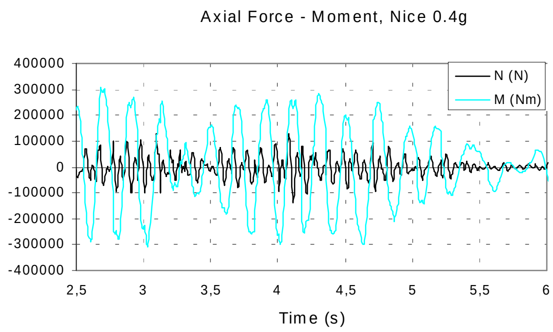


Figure 7.

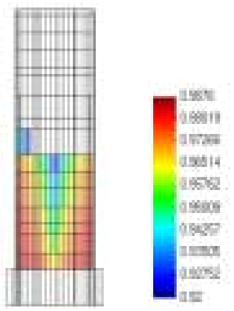


Figure 8. State of Damage

Damage of concrete due to tension is drawn Figure 8 at the end of Nice 0.4g sequence. The damage indicator varies normally between 0 (non – damaged) and 1.0 (completely damaged section). By filtering its values between 0.9 and 1.0 we omit the micro-cracks and have an image of the bigger cracks of the model. The wall is mainly damaged at the base and that is in accordance with the EC8 design philosophy ("monofuse" concept). In fact EC8 design provides a plastic hinge at the base where the damage is to be localised. In order to ensure the localisation mechanism through the formation of the plastic hinge it is necessary to provide sufficient reinforcement bars over the rest of the structure (see Table 2).

Comparison with camus i results

Table 4 compares the maximum values of forces issued from the CAMUS I experimental test and from the CAMUS III numerical predictive calculations. We note that the maximum bending moment is greater for the CAMUS III calculations. The dynamic variation of the axial force in compression is also much greater.

TABLE 4. Experimental CAMUS I and numerical CAMUS III results: maximum forces at the foundation level

	bending moment	Axial force (dyn variation in compression)	shear force
CAMUS I (0.4g)	279kNm	51.9kN	86.6kN
CAMUS III (0.4g)	309kNm	130kN	98KN

CAMUS I was designed according to PS92 French regulations that lead to lower ratios of reinforcement at upper storeys and to an "optimised" distribution of cracking. In fact, a wider crack pattern is allowed, not localised at the base, that leads to a multiplication of the dissipation zones. In opposition to the "monofuse" concept adopted by EC8, PS92 applies the "multifuse" concept which is more economical but also more difficult to master. The difference of the two concepts is obvious at Figure 9. At the end of the experimental program CAMUS I (Nice 0.71g) maximal steel strain values were measured at the 3rd and 4th floors.. This is not the case for the CAMUS III mock-up where calculations for Nice 0.71g showed plastic strains concentrated at the base.

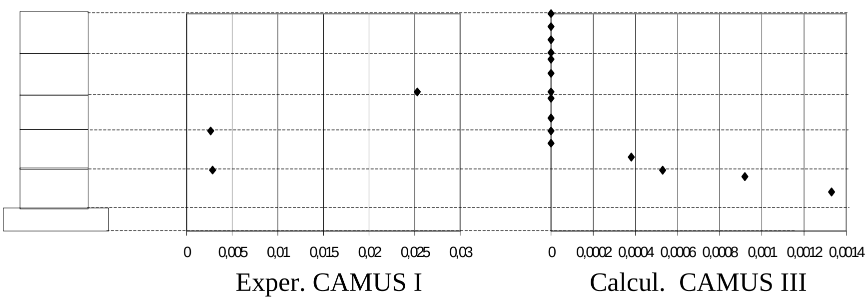


FIGURE 9. Steel Strain (Nice 0.71g)

CAMUS IV – CAMUS I: INFLUENCE OF BOUNDARY CONDITIONS

Description of camus iv test

In order to quantify the effect of soft boundary conditions, the CAMUS IV test is performed by setting the specimen on a sand container. To compare structural responses, the Camus IV mock-up is designed with the same reinforcement as Camus I. The foundations have been recalculated to accommodate the soil bearing capacity, and have been extended to two strip foundations of 0.8 x 2.1 m (Figure10).

The determination of the container height was guided by the willingness of reconstituting more or less realistic soil-structure interaction conditions. Because of the experimental conditions (dynamical test at scale 1/3), the thickness of the sand container was largely limited (security reasons and technical limitations). With such a limitation, it was not possible to design a system that described a realistic plastic behaviour of the soil and the foundation, regarding the development of the failure mechanisms. Therefore the container has been designed purely on elastic considerations. Following that philosophy, a sand layer of 0.4 m thick has been found to be appropriate [Cremer 1999a].

Numerical model

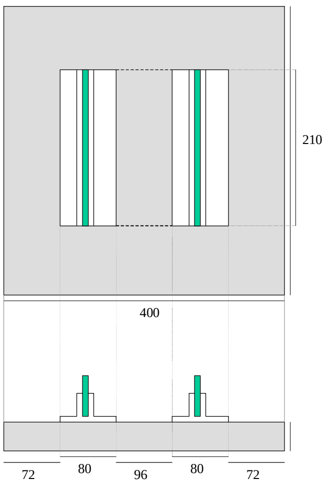


FIGURE 10.
Strip foundations and sand container

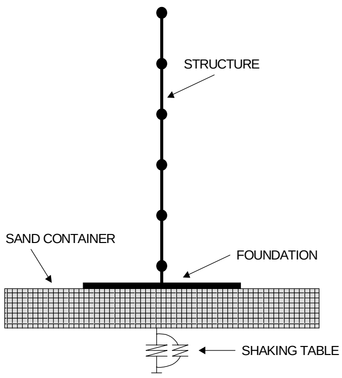
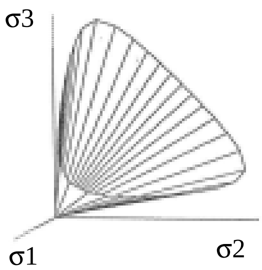


FIGURE 11.
FEM model (DYNAFLOW)

All numerical calculations have been performed with the finite element code DYNAFLOW, developed at Princeton University by J. Prevost [Prevost 1998]. That code has been chosen as it is specialised in soil dynamics problems and proposes well-adapted soil constitutive laws.

The two dimensional numerical model, composed of the structure, the sand container and the shaking table is presented in Figure 11 and detailed hereafter.

Sand



The soil behaviour is governed by the elasto-plastic Matsuoka-Nakaï criterion (Figure 12). It has been chosen because it is particularly suitable for granular materials as sand [Matsuoka, Nakai 1985]

FIGURE 12. Matsuoka-Nakaï criterion

Structure

As we are mainly interested in the uplift behaviour, and as the DYNAFLOW code is not appropriate to model complex structure, the specimen is simply modelled by a vertical beam with lumped masses. The geometrical characteristics of the wall have been applied to the beam. The additional masses and the floor masses supported by one wall as well as the calculated mass moments of inertia are lumped at each floor. The foundation consists in a horizontal beam, assumed very stiff, with uniform mass.

However, non-linear rotational springs have been introduced in the vertical beam to take into account the reduced bending stiffness due to concrete tensile cracks, with the assumption that non-linearities are concentrated in one point at each level. The springs stiffness has been determined by fitting them to the relationships overturning moment-curvature issued from EFICOS pushover calculations, for each reinforcement section, where the non-linear laws, described in the previous chapter, are used.

Shaking table

To reproduce the flexibility of the shaking table measured in CAMUS I test, two springs, one in the vertical direction, one in rotation, have been introduced under the container at the central node. The stiffnesses of the springs have been determined by fitting the calculated eigenfrequencies of the first horizontal and vertical modes with the measured ones, for the system without sand container.

Contact elements

To allow sliding and uplift of the specimen, contact elements are introduced along the soil-foundation interface. They are governed by the perfectly plastic Mohr-Coulomb criterion.

MODAL ANALYSIS

Results of the modal analysis are presented in Table 5 for the following systems: fixed base structure + shaking table, structure + shaking table + sand container (linear structure, elastic law for the soil).

With the sand container introduced in the system (no uplift allowed), the first horizontal frequency decreases from 7.23 Hz to 5.05 Hz and the first vertical frequency decreases from 20 Hz to 17.1 Hz.

TABLE 5. Modal analysis

	structure + shaking table (Hz)	Structure+shaking table+sand container (Hz)
1st horizontal mode	7.23	5.05
1st vertical mode	20.0	17.1

ANALYSIS OF RESULTS

DYNAFLOW numerical results [Cremer 1999b] for the structure submitted to the Nice accelerogram at 0.4g are analysed hereafter.

The rotation at the foundation centre (Figure 13) describes large variations at low frequencies (1.5Hz to 3.5Hz), which corresponds to the oscillations of the mock-up on its base. As the natural rocking frequency of the system is 5 Hz, we observe that the non-linearities due to uplift and non-linear structure induce a significant decrease in the rocking frequencies. We also notice in Figure 14 that a vertical displacement of the foundation centre is rapidly induced, as soon as uplift occurs, because of the coupling existing between the rotational and vertical modes. The uplift, illustrated in Figure 15, reaches very soon 95% of the foundation width.

Figure 16 presents the variation of the overturning moment at the base, superimposed with the dynamic variation of the axial force (referred to a zero initial value). We observe a surprisingly great variation of the axial force at high frequency, which also affects the overturning moment. The excitation of that vertical mode, greatly emphasised, can only be explained as a result of the impact of the foundation on the soil after each cycle. Indeed, we observe that the greater peaks of the axial force occur exactly for a moment equal to zero, which corresponds to the instance when the foundation comes back in contact with the soil. A similar phenomenon has been observed in the Camus I test as soon as cracks appear. The cracks' closing induces also an important variation of the axial force.

Moreover, we notice that the overturning moment reaches 260 kNm, which is much greater than the ultimate value statically admissible for a constant axial force equal to the weight of the structure ($Mc = NB/2 = 180$ kNm). The variation of the axial force produces a similar variation of the admissible overturning moment. It is clear in Figure 17 where the M-θ dynamic curves are plotted, with the corresponding one coming from the static pushover analysis for a constant $N = 180$ kN.

The distribution of the vertical strains in the sand is plotted in Figure 18, at the time of occurrence of maximum response. The stresses reach 428kPa at the edges of the foundation and vertical strains 0.17.10-2. Yielding are concentrated in very local zones, located at the edges of the foundation.

FIGURE 13.

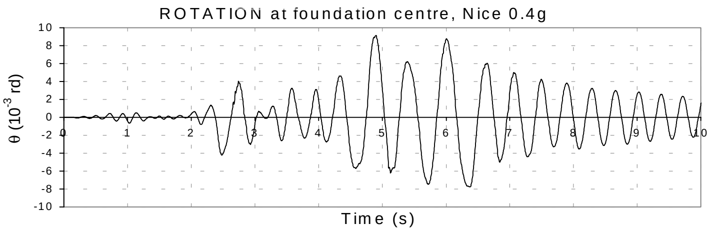


FIGURE 14.

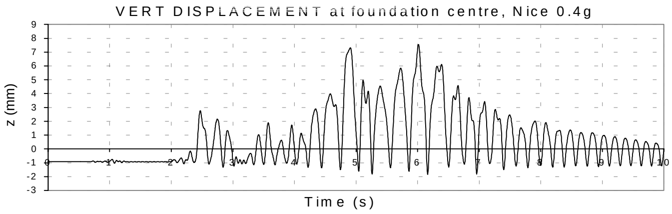


FIGURE 15.

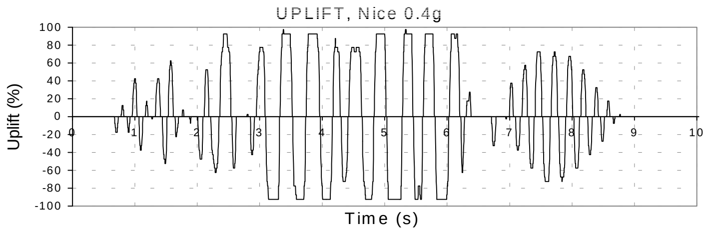


FIGURE 16.

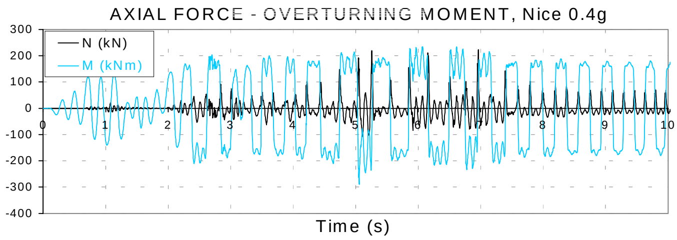


FIGURE 17.

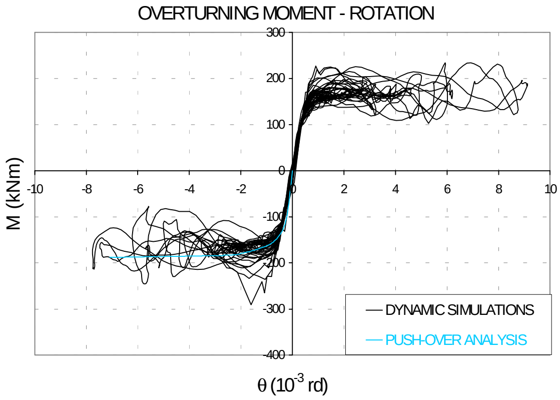
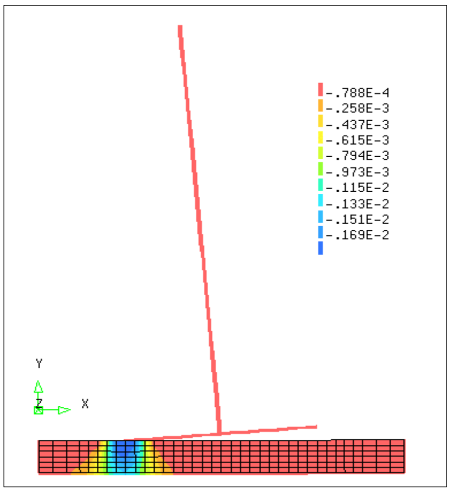


FIGURE 18.

Distribution of strains in the soil



COMPARISON WITH CAMUS I RESULTS

Table 6 compares the maximum values of forces issued from the CAMUS I experimental test and from the CAMUS IV numerical predictive calculations. We note that the maximum bending moment as well as the shear force is very similar, while the dynamic variation of the axial force in compression is much greater for CAMUS IV calculations. That observation stresses the effect discussed in the previous chapter.

TABLE 6. Experimental CAMUS I and numerical CAMUS IV results: maximum forces at the foundation level

	bending moment	Axial force (dyn variation in compression)	shear force
CAMUS I (0.4g)	279kNm	51.9kN	86.6kN
CAMUS IV (0.4g)	260kNm	158kN	92KN

CONCLUSION

Predictive calculations on the CAMUS III test gave us the opportunity to compare two different design philosophies. EC8 design code chooses to localise the damage at the base of the wall and keep the upper storeys linear (“monofuse” concept). PS92 opts for a “multifuse” design where the damage is distributed at different areas and leads to a multiplication of the dissipation zones. We have also shown the importance of the variation of the axial force. The phenomenon is due to the closing of cracks and the excitation of the vertical mode.

Predictive calculations on CAMUS IV test have allowed to highlight the effect of soft boundary conditions, and particularly uplift, on structural response. We have observed that uplift induces a significant decrease of the rotational apparent frequency. Moreover the excitation of the first vertical mode of the system is greatly amplified and kept up by the impact of the foundation on the soil. An important variation of the axial force is observed which allows for a high overturning moment at the base. That moment can be much greater than the critical static one calculated with the weight of the structure. As a result, the maximum moment and shear force supported by the structure with soft boundary conditions (CAMUS IV) are in the same order as those measured for the fixed base structure (CAMUS I) and are not reduced as we may have assumed considering the non-linear effect of the uplift.

Further research is still going on and more experimental and numerical results will be presented in forthcoming papers.

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