IN PLANE SEISMIC BEHAVIOUR OF SEVERAL 1/3RD SCALED R/C BEARING WALLS - TESTING AND INTERPRETATION USING NON LINEAR NUMERICAL MODELLING

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SUMMARY

Reinforced concrete bearing walls with limited reinforcement ratio are commonly used in France for the building structures. In order to show their seismic performance, a series of seismic tests on 3 large scale models have been performed on the largest shaking table of the Commissariat à l'Energie Atomique (CEA) in Saclay, France. These specimens have been designed according French and Eurocode Design Code. The seismic tests conducted to extensive damage in the R/C structure. This paper shows the main experimental results and some remarks about the interaction between experimental and numerical studies.

INTRODUCTION

Reinforced concrete bearing walls with limited reinforcement ratio are commonly used in France for the building structures. In order to show their seismic performance, a series of seismic tests on 3 large scale models have been performed on the largest shaking table of the Commissariat à l'Energie Atomique (CEA) in Saclay, France. The tests on the 2 first specimens has been supported by the CAMUS French national research project while the third specimen has been tested in the framework of the TMR (Training and Mobility of Researchers) program of the European Commission.

Three specimens with different reinforcement ratios have been built. The 1/3rd specimens are composed of two parallel 5-floor R/C walls without opening linked by 6 square floors and have a total mass of 36 tons. The 2 first specimens have been designed according to French constructions. The 1st model is slightly reinforced while the second one has almost no reinforcement. For the first specimen, the reinforcement ratio changes between two storeys in order to obtain steel yielding at several storeys. The tests have been performed up to obtain significant damage. Wide crack opening and extensive yielding and failure of the steel bars have been observed during the tests. The design of the third specimen follows the EC8 requirements. Although the ultimate bending moment at the base is the same one than the first specimen, its design aimed at concentrating damage at the base in a unique plastic hinge and not spread it on the height on the structure. The present paper presents the experimental results and the numerical modelling used for both the preparation and the interpretation of the tests. The computations performed with a fibre type model with non-linear constitutive laws for concrete and steel illustrate the interaction between experiments and numerical modelling.

MAIN CHARACTERISTICS OF THE CAMUS SPECIMENS

Three specimens with different reinforcement ratios have been built in the framework of both the French CAMUS research programme and the ECOEST II European programme (Fig 1). The 1/3rd scaled specimens are

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composed of two parallel 5-floor R/C walls without opening linked by 6 square floors (including the floor connected to the footing). A heavily reinforced concrete footing allows the anchorage to the shaking table.

The total height of each model is 5.10 m and the total mass is estimated at 36 tons. Each wall is 1.70 m long and 6 cm thick. Each part of the structures (walls, floors and basements) is cast separately. The walls are cast in two parts in order to reproduce the construction joint at the level of each floor. The different parts of the mock-up are assembled on the shaking table. The value of the additional masses fixed to each floor was chosen to obtain a vertical stress commonly found in such structures under the vertical static loading (1.6MPa in this case).

The 2 first specimens were designed according to French constructions. The 1st model CAMUS I was slightly reinforced while the second one has almost no reinforcement. 4.5, 5.0, 6.0 and 8.0 mm diameter steel bars have been used. Table 1 gives the steel rebars at each storey for the specimens CAMUS I and III. It was composed by vertical reinforcement concentrated in the 2 boundaries and the center of the section. The reinforcement ratio changes between two storeys in order to obtain steel yielding at several storeys and not a concentrated plastic hinge in the lower part of the wall. These 2 specimens had no horizontal shear reinforcement.

The last specimen CAMUS III was designed according Eurocode 8 but with the same ultimate bending moment at the base than CAMUS I. The drawings of 3 sections of CAMUS III are given Fig 2. The differences in reinforcement between CAMUS I and III concern mainly the shear reinforcement and the upper storeys. A comparison between the longitudinal reinforcement of CAMUS I and III is given in table 1.

DESCRIPTION OF THE TESTING PROGRAM

Loading program

The seismic tests have been performed on the AZALEE shaking table of the CEA at Saclay. This 6 m X 6 m biaxial shaking table has a maximum payload of 100 tons and is the largest in capacity in Europe. Monoaxial time excitations with increasing levels were applied to the specimens. A second series of tests on the first specimen were performed after its retrofitting with fibre carbon material by the TFC© Group. The main parts of the seismic tests – the high levels tests included - have been performed with the 10s long artificial signal Nice whose response spectra fit the French Design Code design spectra. Two recorded signals named San Francisco and Melendy Ranch have also been used for intermediate tests on the specimens CAMUS I and III. These signals are characteristics of near-field moderate earthquakes (important acceleration but short duration and different frequency content, [Sollogoub et al, 1998]). The theorical signals and their corresponding response spectra are given Fig 3. The following tests have been performed:

CAMUS I : Nice 0.24g, San Francisco 1.11g, Nice 0.24g, Nice 0.40g, Nice 0.71g

CAMUS II : Nice 0.10g, Nice 0.23g, Nice 0.52g, Nice 0.51g

CAMUS III : Nice 0.22g, Melendy Ranch 1.35g,Nice 0.64g, Nice 1.0g

Instrumentation

Up to 64 measurement channels were recorded during each test. The instrumental set-up was designed in order to give information on the motion of the shaking table, the global and the local behaviour of the specimen. Horizontal displacement, horizontal and vertical accelerations were measured at each floor.The internal forces have been deduced directly from the accelerations. Strain gages were fixed on the steel bars and 25 cm long transducers measuring the crack openings have been placed where damage were expected: at the construction joints of the 4 lower storeys for CAMUS I and II and at plastic hinges for CAMUS III.

MAIN EXPERIMENTAL RESULTS

Since the specimen CAMUS III is concerned by a blind International Benchmark, only the results on the two first models are prsented in this paper.

Motion of the shaking table

The horizontal accelerometer fixed in the centre of the table has been used for the control of the experiment. During the first tests, the imposed signal was not exactly the reference signal since the specimen behavior was strongly non-linear. But for the high intensity tests, the difference between the reference and the obtained signals remained small. Two vertical accelerometers were fixed at the two extremities in order to check if rocking or vertical motion occurred during the tests. The analysis of the signals in the frequency domain has been performed computing their response spectrum (with 2% damping). Rocking seems to be neglectable since the difference of the signal given by the two accelerometers has a high frequency content which does not correspond to any bending mode of the specimens. For CAMUS I, the shaking table had a vertical motion which has reached a 0.43g amplitude with a frequency of about 20Hz. This value corresponds to the frequency of the 1st vertical eigenmode of the system table+specimen.

Global behaviour of the specimens

Motion of the specimens

The horizontal top displacement is computed relatively to the base of the first storey. The maximum values are given table 2. One may remark the maximum values of top displacement correspond to 1% drift for the 2 specimens. For such drift, extensive damage was obtained without loss of stability of the structural system. The analysis of the top displacement time history shows the specimens responded mainly on its first natural frequency (Fig 6 for last test on CAMUS II). The decrease of natural frequency due to specimen damage can be estimated using :

- the transfert function computed by applying random excitations to the specimens between each test

- the response spectra with 2% damping of the top displacement time-histories which gives an apparent natural frequency

With the first method – it means at low level of excitation -, the fundamental frequency varies from 7.24 Hz to 6.60 Hz for CAMUS I and from 6.4 Hz to 6.05 Hz for CAMUS II: this decrease of frequency remains limited due to the importance, for such type of structure, of the vertical load which closes the cracks. At the opposite, with the second method – it means when the cracks are opened -, the frequency of the motion decreases down to 2 Hz (for CAMUS II). This great difference shows the difficulty to know the state of the structure (cracked, failure of steel bars …) with only the knowledge of the dynamic characteristics at low level of excitation.

Analysis of the internal forces

The inertial forces and so the bending moments, the shear forces and the axial forces can be computed with the horizontal absolute acceleration given by the accelerometers and the estimation of the masses of each floor. The maximal values at the base of the first storey are given in the table 2. The difference of reinforcement ratio explains the large difference of the measured bending moment between the 2 specimens. The importance of the variations of axial forces has to be highlighted. For the last test on CAMUS I, the amplitude of the variation of dynamic axial force is similar to the vertical static loading. This phenomenon can be explained by the mechanism of reinforced concrete. The variation of neutral axis due to concrete cracking induces a vertical motion of the floors. Compression force increases strongly when concrete recovers its stiffness at the crack closure, it means when the curvature and displacement are about zero. These shocks excite the first vertical mode of the system shaking table+specimen whose frequency is about 20Hz. Fig 7 and 8 show the interaction between bending moment and axial force for the last test on CAMUS II: the variation of axial force may induce important variation of bending moment when the plateau corresponding to steel yielding is reached.

Local behaviour

Damage pattern

During the first tests on CAMUS I, cracking occurred mainly at the construction joints. But large cracks appeared during the last test at the interruption of the steel bars of the second storey and developed in a diagonal direction (Fig 4). Visual inspection showed the steel bars were broken at this storey after the last test. For CAMUS II, cracking remained concentrated at the construction joint, mainly at the base of the 2 first storeys (Fig 5). Wide crack opening were observed during the test.

Strain and cracks openings

The strain gages gave high values of strain at the level of the construction joints (more than 2%). For CAMUS I, the maximal strain values were measured at the 3rd and 4th floors and not at the 1st floor (Table 3). This is due to the design used for this specimen which allows yielding at several floors and does not concentrate the damage at the lower storeys at the opposite of Eurocode 8. For the second model, large crack opening was observed at the 1st and 2nd storeys. In order to compare the values given by the strain gages and the transducers, the crack openings have been converted to equivalent strain by dividing the values given by the transducers by their lengths (250mm). For CAMUS I, the values given by the strain gages are higher than those deduced from the transducers: the steel bars yielded on a concentrated area. The transducers gives a mean value of vertical strain on their length. At the opposite, for CAMUS II, the values given by the strain gages are lower than the strain deduced from the crack opening (Table 3). Furthermore, except for one gage, the strain values remained lower than the yielding strain. This may be explained by the degradation of the bond interface between steel and concrete. For the second specimen, the diameter of the steel reinforcement bars does not exceed 5mm and for such diameter, adherence between steel and concrete was provided only by diameter variation.

Moment-curvature relationship

The moment-curvature relationships give information on the behaviour of the R/C section useful for the validation of the numerical models. The curvature is deduced from the steel strains or the crack openings. The moment-curvature relationships present a pinched aspect which increased for the lightly reinforced section (Fig 8). For lightly reinforced bearing walls, the static axial force can not be neglected to estimate the bending strength. Although the moment-curvature relationships do not exhibit large dissipative plastic cycles, it seems such a behaviour can provide good seismic performance without having large bending strength.

SOME REMARKS ON THE INTERACTION BETWEEN NON LINEAR MODELLING AND EXPERIMENTAL RESULTS

Such large scale experiments requires the utilization of numerical models for the preparation the tests –decide the levels of acceleration to be applied - and the interpretation of the experimental results. This exercice has been performed by several research teams belonging to universities and research centres during the French and the European programs supporting these experiments (GEO and ICONS research networks and CEA). The studies performed in CEA with the CASTEM 2000 general purpose software illustrate well the problems met during this program and the benefit of the interaction between computations and experiments.

A non linear fibre-type model on a Timoshenko beam elements with one Gauss point has been used for this purpose [Guedes, 1997]. In this model based on the assumption of plane section, uniaxial laws are considered for steel and concrete taking into account, for concrete, softening and unilateral effect in traction and the effect of confinement on the behaviour in compression and, for steel, hardening and Bauschinger effect. Note the behaviour of this structure which has low vertical stress is mainly dominated by cracking and yielding of the steel and not by the behaviour of concrete in compression.

In a first step, the numerical model has been used for predictive parametrical studies. Due to the lack of validation of this type of model by large scale dynamic experiments, the choice of the model parameters has been greatly influenced by the methods used for design. For exemple, it was considered an uncracked stiffness, a Rayleigh damping with 5% damping on the 2 first eigenmodes and a failure criteria of 1% strain in steel. These choices conduct to a strong underestimation of the predicted displacement -it was found a top displacement of 14.4mm for Nice 0.75g and 16.5mm for San Francisco 1.5g-.

Although the difficulties of the numerical model to predict the experimental displacements, one of the main aspect of the effect of near-field earthquake has been highlighted by the predictive computations: at low level, the displacement and so the damage remains, for the same level of acceleration, higher for the San Francisco signal than for the Nice signal but, at high level, after the beginning of cracking and yielding, the opposite is observed. This means the San Franscisco signal is less “damageable” but only at medium or high level of excitation. This conclusion contribute to the decision of the levels of excitation. A second phenomena observed during the predictive numerical studies is the strong variation of axial force although they were overestimated because of a “stiff” crack closure law.

The comparison between numerical and experimental results has conducted to some change in the numerical modelling. Firstly, the vertical rods supporting the shaking table –in term of stiffness- and the shaking table itself –in term of mass- had to be included in the model in order to find the frequency of the vertical eigenmode measured during the test which is at 20 Hz and not 40Hz like predicted by the model witout the shaking table. Secondly, supplementary flexibility had to be introduced at the anchorage of the wall and a lower value of damping (2% on the first mode like the measured frequency) in order to reproduce the global behaviour of the specimen (displacement, internal forces…). With these modifications, the predictive computations gave much better results for the second specimen.

It must be remarked that also with the modifications made after the tests, it was not possible to reproduce the experimental results at the local level: the computed strains in the steel remain much lower than the measured one. So the local results given by the numerical model must be considered very carefully if they are used as failure criteria. Another important –and obvious- remark is the impossibility for the fibre type model to reproduce the failure pattern of the first specimen CAMUS I – diagonal cracking because of the tensile shift due to shear-. Plane stress 2D or full 3D models are necessary in this case.

CONCLUSIONS

Two large-scale specimens representative of bearing wall structures were tested under seismic loading on the largest shaking table of CEA Saclay-France up to obtain extensive damage. A first analysis of the experimental results shows such kind of structure can support extensive damage without loss of stability although the reinforcement ratio remains low. Important variation of axial force was also observed during the test.

The numerical studies performed in parallel of these experiments shows the capacity and the difficulty of utilization of the non linear model and so the necessity of improvement of not only the models themselves but also in the methodology of utilization – including the interpretation of the results. The organization of blind International Benchmark such as the CAMUS International Benchmark which concluded on Septembre 1998 in Paris during the European Conference of Earthquake Engineering can be very helpful to this purpose [Camus International Benchmark, 1998].

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Table 1: Longitudinal reinforcement of the CAMUS I and III specimens

	Boundaries (each) – CAMUS I	Boundaries (each) – CAMUS III	Central reinforcement – CAMUS I	Central reinforcement- CAMUS III
5th storey	1φ4.5=15.9 mm^2	2φ8+2φ4.5=132 mm^2	4φ5=78.4 mm^2	2x5φ4.5/200=159 mm^2
4th storey	1φ6=28.2 mm^2	4φ8+2φ4.5=233 mm^2	4φ5=78.4 mm^2	2x5φ4.5/200=159 mm^2
3rd storey	1φ6+1φ8+1φ4.5=94.4 mm^2	4φ8+2φ4.5=233 mm^2	4φ5+2φ4.5=110 mm^2	2x5φ4.5/200=159 mm^2
2nd storey	2φ6+2φ8+2φ4.5=189 mm^2	4φ8+2φ6+2φ4.5=289 mm^2	4φ5+2φ4.5+φ6=138 mm^2	2x5φ4.5/200=159 mm^2
1st storey	4φ8+2φ6+2φ4.5=289 mm^2	4φ8+2φ6+2φ4.5=289 mm^2	4φ5+2φ4.5+φ6=138 mm^2	2x5φ4.5/200=159 mm^2

Table 2 – Maximum values of top displacement, bending moment and axial forces (for one wall)

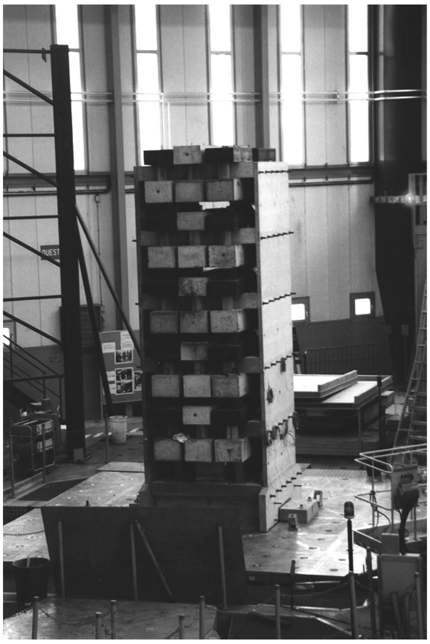
CAMUS I tests	Nice 0.24g	SF 1.11g	Nice 0.24g	Nice 0.40g	Nice 0.71g
Top displacement	7.0mm	13.2mm	6.3mm	13.4mm	43.3mm
Bending moment	211kN.m	280kN.m	164kN.m	279kN.m	345kN.m
Shear force	65.9kN	106kN	52.2kN	86.6kN	111kN
Axial force* – Traction	44.3kN	102kN	24.4kN	50.0kN	137kN
Axial force *– Compression	36.5kN	105kN	30.4kN	51.9kN	146kN
CAMUS II tests	Nice 0.10g	Nice 0.23g	Nice 0.52g	Nice 0.51g	
Top displacement	4.4mm	12.8mm	34.9mm	42.7mm	
Bending moment	110kN.m	149kN.m	178kN.m	186kN.m	
Shear force	34.1kN	51.9kN	73.8kN	70.7kN	
Axial force* – Traction	16.3kN	30.3kN	86.4kN	100kN	
Axial force *– Compression	16.9kN	43.3kN	104kN	125kN	

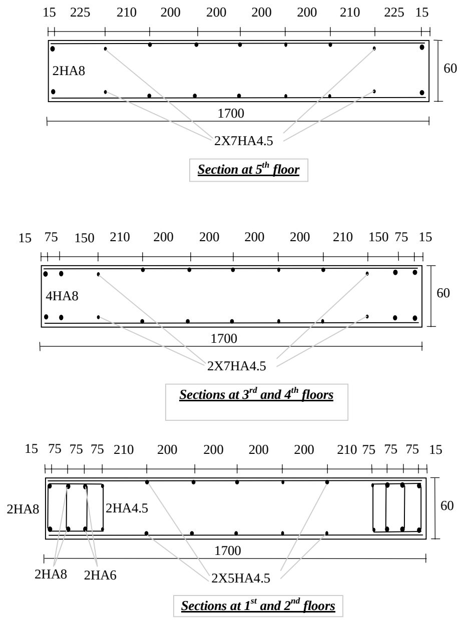
* The values of axial force must be added or substracted to the static vertical load (166kN)

Table 3: Maximal values of strain during the last test on CAMUS I and CAMUS II specimen

Storey	Left transducer	Left strain gage	Right strain gage	Right transducer
4th floor – CAMUS I	10.4/1000			5.71/1000
3rd floor - CAMUS I	20.2/1000	25.3/1000*	25.3/1000*	7.34/1000
2nd floor – CAMUS I	2.38/1000	2.64/1000	2.58/1000	1.69/1000
1st floor – CAMUS I	1.16/1000	2.85/1000	2.66/1000	1.52/1000
4th floor – CAMUS II	2.46/1000			1.35/1000
3rd floor - CAMUS II	7.26/1000	1.85/1000	2.85/1000	Out of service
2nd floor – CAMUS II	26.1/1000	4.42/1000	Out of service	39.2/1000
1st floor – CAMUS II	36.2/1000	2.15/1000	0.67/1000	16.6/1000

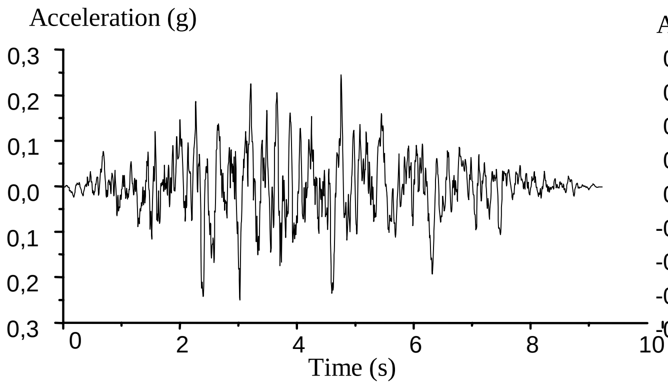
* Saturation of the strain gage



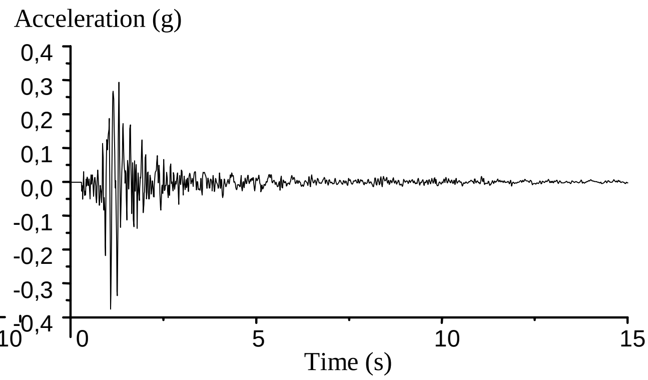


CAMUS III

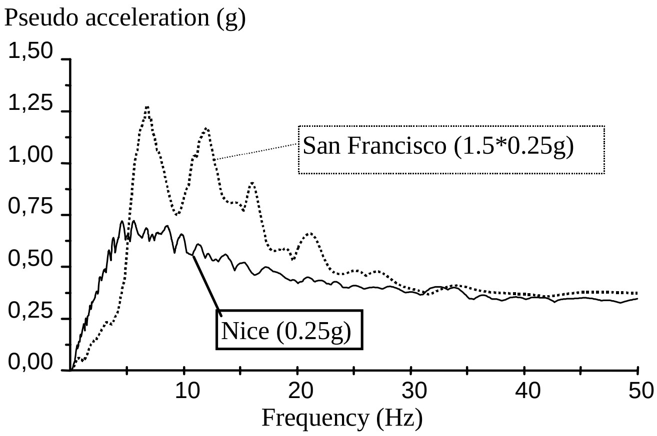
Figure 1 – View of one CAMUS specimen **Figure 2 – Reinforcement of CAMUS III specimen**



a/ Nice signal (a_{max}=0.25g)

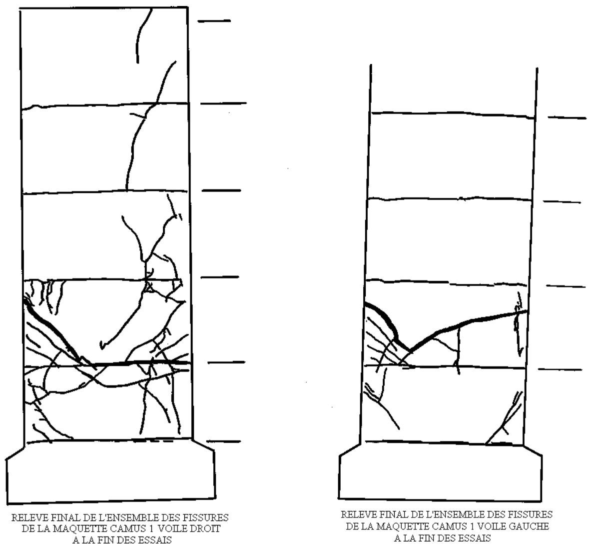


*b/ San Francisco signal (a_{max}=1.5*0.25g)*



c/ Response spectra (5% damping)

Figure 3 – Nice and San Francisco signals



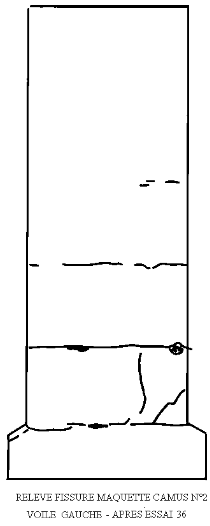


Figure 4- Cracking at the end of CAMUS I tests (Left and right sides)

Figure 5- Cracking at the end of CAMUS II tests (left side only)

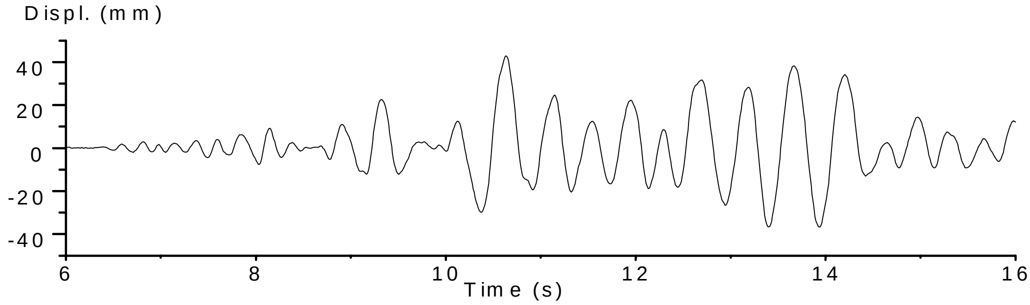


Figure 6- Evolution of top displacement (last test of CAMUS II)

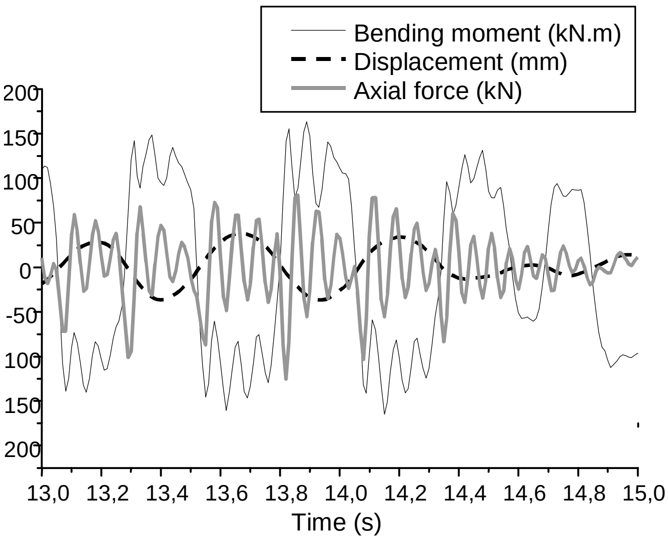


Figure 7 – Evolution of axial force and bending moment (last test of CAMUS II)

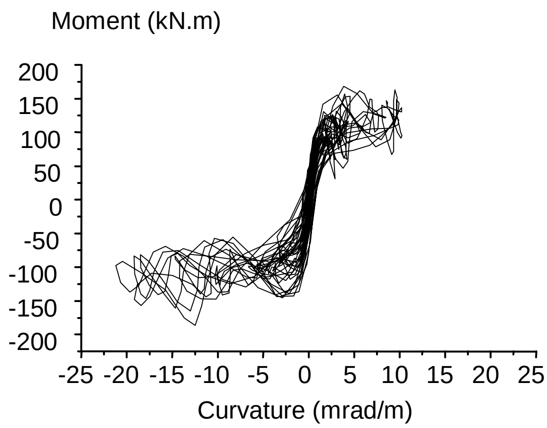


Figure 8-Moment-curvature relationship (Last test of CAMUS II)