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SEISMIC REEVALUATION OF STRUCTURES: A COMBINED ANALYTICAL AND EXPERIMENTAL APPROACH

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SUMMARY

Many buildings, critical installations and nuclear facilities have been constructed with seismic provisions of the sixties, which did not match the present seismic codes. The seismic assessment of these structures requires tools in order to quantify their actual capacity.
The paper presents the approach used in CEA. It is based on non-linear analyses simulating the actual behaviour beyond the conventional code limits. Different finite element models are used: global, fibre type or 3D refined meshes. An important point is to characterize the effect of different detailing, such as stirrups density, lap splices, node reinforcement, on the post elastic behaviour.
In parallel to the analytical approach experimental studies are performed to qualify and confirm the non-linear capacities of different structural types, such as reinforced concrete walls or frames, including masonry. Comparison to elementary tests results will be presented. The important experimental capacities of the CEA Seismic Laboratory, including a large 100t capacity 6 dof shaking table, will be presented together with some relevant experimental programs.

INTRODUCTION

In the oldest industrial countries specially with moderate seismicity such as France, the main part of the building stock has been designed before the application of the modern seismic code. Because of the absence of any seismic consideration at the period of the construction or the modification of the action levels, many reinforced concrete structures do not satisfy the actual requirements: inadequate stirrups spacing, insufficient lengths of anchorage and lap splices, low shear strength of columns or beam-column joints compared to the shear demand imposed by the global flexural mechanism…
Due to safety reasons, the case of industrial and nuclear facilities and plants may be critical and requires detailed seismic evaluations. The application of simplified procedures used for design tends to give unrealistic results and, overall, masks the critical points for the facility safety.
The seismic assessment of the RC frames can be performed using a non linear fibre type model. This model supported by a Timoshenko beam element with shear distortion is based on classical beam assumptions and uses uniaxial constitutive laws for concrete and steel. The strength of the brittle

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mechanisms - the modern seismic code intends to avoid- can also be checked using appropriate material laws and modelling assumptions.
This paper aimed at presenting some validation of these constitutive laws on experimental results available in the literature. The main principles of the modelling (type of finite elements, constitutive laws,…) will be reminded before the detailed analysis of the tests on elementary structural elements (columns and beam-column joints). The testing capabilities of the CEA Seismic Laboratory will be presented with some of relevant experimental works.
Some consideration about the modelling of the bending behaviour will be made specially on the failure criteria (length of plastic hinge to be used, influence of the detailing on the failure criteria, etc…).
In a second step, the simplified laws used for shear behaviour and the modelling of anchorage and lap splices will be particularly detailed. A simple model based on the equilibrium between the bond stress between concrete and steel and the axial stress in the steel bar is used for the verification of the anchorage and the lap splices. Furthermore a way to compute and check the shear strength of the beam-column joint using only simple beam elements will be described.
Comparison between numerical and experimental results will be made in order to validate the modelling approach. The experimental results used for this validation are part of testing campaigns performed on isolated structural elements under cyclic loading or on more representative mock-ups on the Laboratory shaking tables. RC columns with insufficient shear strength and lap splices and beam-column joints with insufficient anchorage length of the bottom beam steel bars were concerned by these experiments.

GENERAL DESCRIPTION OF THE LABORATORY

The seismic testing facility, TAMARIS, is part of the mechanical studies laboratory of the Systems and Structures Modelling Departement in CEA Saclay research centre. It has a permanent staff of 20 engineers and technicians. Its objectives are model development and validation, calculation methods development and qualification, codification and seismic qualification of components.

The TAMARIS facility includes the following equipment (Figure 1):
AZALEE : 6m x 6m shaking table : it allows testing of specimen with a mass up to 100 tons in 3 directions and has 6 degrees of freedom (translations and rotations). Independent excitations of any kind : (sinusoidal, random, shock, seismic... with a 0-100Hz frequency range) may be applied. Max acceleration (with max. weight) 1g ; max velocity 1m/s ; max displacement : ±125mm. These characteristics make it the largest shaking table in size and in capacity outside Japan.
VESUVE : 3.1mx3.8m shaking table. Maximal mass : 20 tons in one horizontal direction ; max acceleration (with max. weight) 1.2g ; max velocity : 1m/s ; max displacement : ±100mm.
TOURNESOL : 2mx2m shaking table . Maximal mass : 10 tons ; max acceleration (with max. weight) ; horizontal direction :1.0g, vertical direction 1.3g ; max velocity H 2m/s and V 1.3m/s ; max displacement : V±100mm and H ± 125mm.
MIMOSA : 2mx2m shaking table. Maximal mass : 10 tons in one horizontal direction ; max acceleration (with max. weight) 4.g ; max velocity 0.6m/s ; max displacement : ±12.5mm.
IRIS : pit allows testing of specimen up to 25m long. The diameter of the hexagonal pit is about 4m and lateral walls are equipped with anchoring devices.
REACTION WALL : (4m height, 5m length) equipped with anchoring devices.
All the equipment are connected to the acquisition system allowing recording and processing of 142 channels. It is connected to a scientific computing system with the mechanical analysis code Castem 2000 used linear and non linear analysis of structures and fluid mechanics.

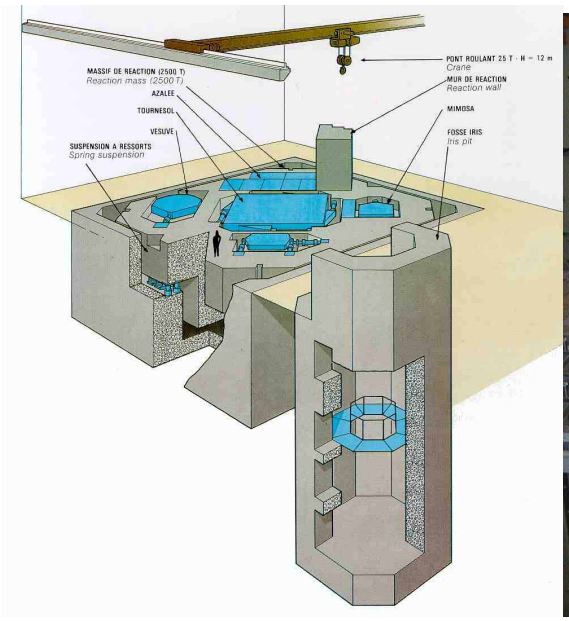


Fig 1 View of the experimental facility

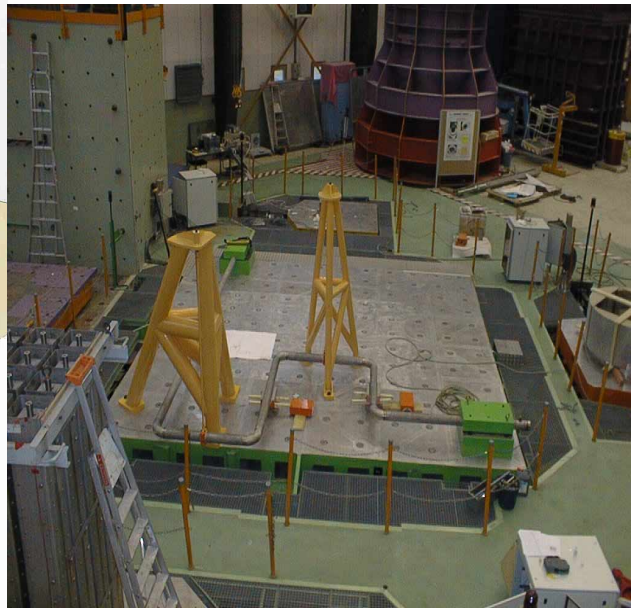


Fig 2 Seismic test of a nuclear power plant pipe

SOME RESEARCH ACTIVITIES

The laboratory is working on different national and international programs related to the seismic behavior of equipment, components and structures. Main activities are devoted to :

- Piping systems to evaluate the margins of current design methodologies.
- Industrial and nuclear equipment such as electrical cabinets, tanks (Figure 2), storage racks, nuclear fuel assemblies and core... to understand and codify their seismic behavior. These results participate to better understand and improve the overall safety of the critical industrial facilities.
- Civil engineering structures : analysis and testing of structural elements (shear walls, frames..) or structures in order to evaluate the margins of design methods or to improve them. Large scale tests on shear wall buildings have been performed for some years (French CAMUS program). One of the specimens was also tested after retrofitting with TFC Carbon fibre by Freyssinet (Figure 3).2 other similar structures were tested in 2001 in the frame of CAMUS-2000 program, with the objective of testing 3-D and irregular, torsional behaviour. A series of scaled 2 storey one bay RC frames designed according old seismic design provisions are also being tested in the framework of the seismic assessment of the RC structures of existing industrial facilities (Figure 4).
- The Laboratory has been involved in European programs since many years. It participates to ECOEST2 and ECOLEADER consortium grouping different Laboratories in Europe. In this frame, many test were performed on structures and components: shear walls, U-shaped walls in order to asses the effect of reinforcement between walls, silos, aseismic pads, damping devices, 3D effects on masonry…

In the nuclear field, Tamaris laboratory is also strongly involved in seismic assessment of existing nuclear facilities (mainly CEA laboratories). These expertises in seismic behaviour of industrial equipment, piping system and civil engineering structures require great interaction between experimental activities, linear and non linear finite element modelling and engineering.

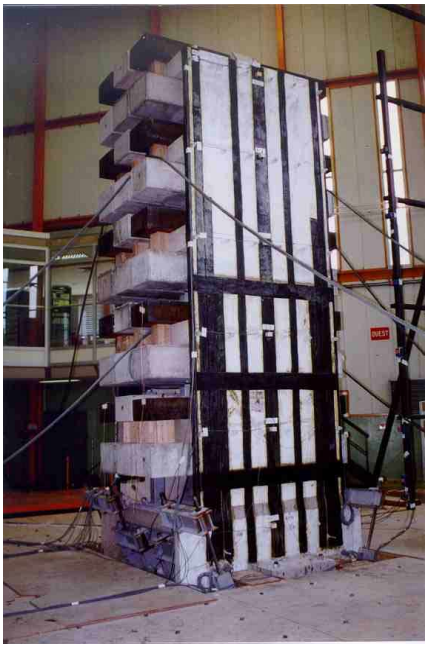


Fig 3 : CAMUS Structural wall with TFC repair

Fig 4 : Joint in RC frame structure

DESCRIPTION OF THE NON LINEAR MODELLING

SOME GENERAL CONSIDERATIONS ON THE APPLICATION OF NON LINEAR MODELING TO THE SEISMIC ASSESSMENT OF EXISTING STRUCTURES

The application of simplified procedures used for design for the seismic assessment of existing buildings –elastic computation and reduction by a q-factor- tends to give unrealistic results and, overall, masks the critical points for the safety of the structures. Non linear modelling which reproduce the physical phenomena in a better way can be a very useful tool during the seismic assessment process.
On one hand, the non linear computations allow a more realistic understanding of the global failure mechanism of the structure. Furthermore they can give a quite good estimation of the demands of local and global ductility, of shear in the critical members (columns, walls, beam-column joints…) and of bond forces in the steel reinforcement (rebar anchorages, lap splices…).
On the other hand, several points limit the confidence of the engineers and experts in this tool such as their complexity –number of parameters-, their limits of validity not often well declared –up to which damage level are they reliable and these models can really take into account the construction detailing ? - and the lack of common rules of utilisation. The acceptance of such modelling approach for the seismic assessment of existing buildings requires a strong interaction between experimental and numerical results.

A TIMOSHENKO BEAM ELEMENT WITH NON LINEAR SHEAR BEHAVIOUR

The seismic analysis of complete building structures with dynamic or simplified push-over analysis requires simplified non linear finite elements. The behavior of reinforced concrete members such as

columns, beams but also structural walls can be very well reproduced using non linear beam elements with fiber type assumptions at the section level.
This modeling is based on a geometrical description (Fig 5) of the beam section in fibers (or layers in 2D). The axial and shear strains in each fiber are deduced directly from the average axial ε_x and shear strains γ_y, γ_z, the curvatures (in flexion ϕ_y, ϕ_z and torsion ϕ_x) of the beam element and the section geometry.

$$(\varepsilon_x)_i = \varepsilon_x - y_i \cdot \phi_z + z_i \cdot \phi_y$$

$$(\gamma_y)_i = \gamma_y - z_i \cdot \phi_x \text{ and } (\gamma_z)_i = \gamma_z + y_i \cdot \phi_x$$

The normal force N_x, bending moments M_y, M_z, shear forces T_y, T_z and twisting moment M_x are calculated by integrating the axial and shear stresses in the section.

$$N_x = \int_S \sigma_x dS \,,\; M_y = \int_S z \cdot \sigma_x dS \text{ and } M_z = -\int_S y \cdot \sigma_x dS$$

$$T_y = \int_S \tau_y dS \,,\; T_z = \int_S \tau_z dS \text{ and } M_x = \int_S (y \cdot \tau_z - z \cdot \tau_y) dS$$

Each fiber supports a uniaxial law $\sigma(\varepsilon)$ representative of concrete or steel behaviour. Fig 6 shows the laws used in the present study respectively for concrete (with softening in compression and tension) and for steel (with hardening, Bauschinger effect and buckling [6]).
A simple Timoshenko beam element has been adopted in order to allow shear distortion and so the use of non linear constitutive laws not only for bending but also for shear and torsion. An example of a uniaxial law for shear $\tau(\gamma)$ with softening and empirical rules under cyclic loading is given on Fig 6d. In order to avoid shear locking, this 3D beam element has a unique Gauss point and the axial strain, curvature and shear strain remain constant on the element.
Details of this beam element and the uniaxial constitutive laws can be found in [5].

SOME REMARKS ON THE INFLUENCE OF THE CONSTRUCTION DETAILS ON MODELLING

The accuracy of the modeling and the prediction of failure depends strongly on the capacity to take into account of the construction details specially for the reinforced concrete frame structures.
For example, the concrete law shown Fig 6a although it is uniaxial can be directly influenced by the confinement of the stirrups by modifying the ultimate compressive strength and the softening slope Z. A decrease of softening due to higher confinement ratio improves the curvature ductility capacity. The confinement can also be taken into account by modifying the local failure criteria (let say the concrete ultimate strain).
Another major difficulty in the modeling of frame structures up to flexural failure is the localization phenomena due to softening or limited hardening after yielding of the steel bars. This phenomena makes the local results (curvature and strain demands) strongly dependent on the mesh size and requires to fix the length of the elements –the plastic hinges- where damage may concentrate. This is equivalent to consider plastic rotations or chord rotations as failure criteria.
Priestley [9] gives some formulae to determine the length of the plastic hinge H_{hinge}.

$$H_{hinge} = 0.08\, H_{column} + 6\, d_{bar}$$

This length depends not only on the column height H_{column} but also on the steel bars diameter d_{bar} since spread of steel yielding in the footing has been evidenced by several experimental results.

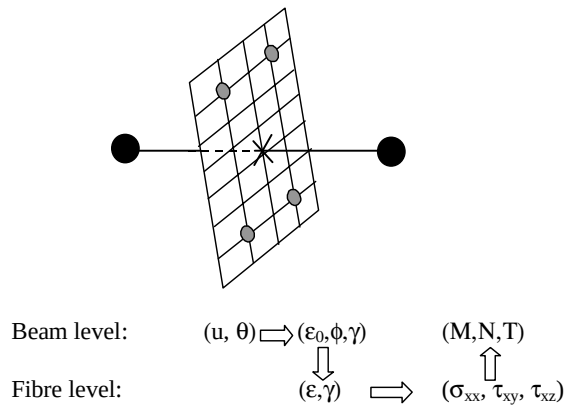
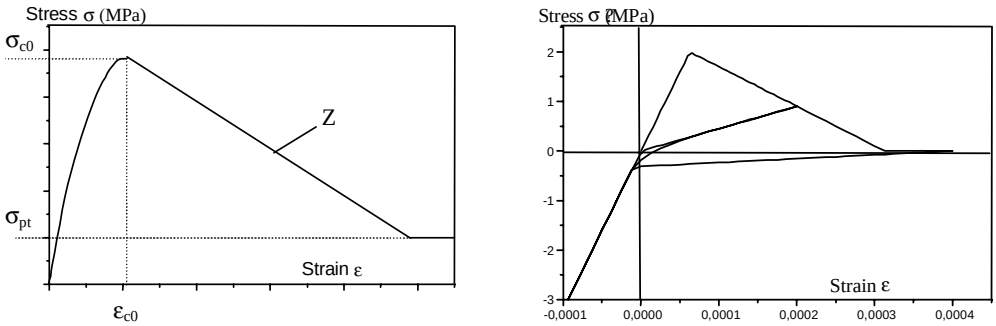
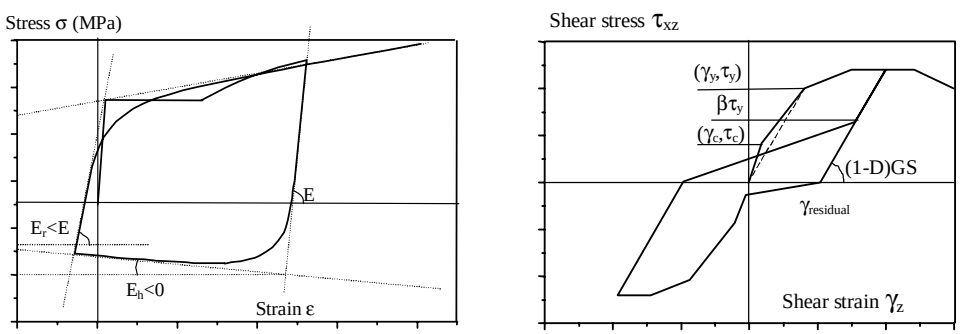


Figure 5: Non linear Fiber Beam Model



a/ Behaviour of concrete in compression b/ Behaviour of concrete in traction



c/ Menegotto-Pinto law for steel d/ Uniaxial law with softening for shear

Figure 6: Uniaxial Constitutive Law for Concrete, Steel and Shear

A CONSTITUTIVE LAW FOR ANCHORAGES AND LAP SPLICES

A specific constitutive law for has been introduced in the fibre model in order to check the possible failure of lap splices and anchorages. The approach already implemented for bridge piers by Monti [7] and Xiao [10] has been adopted.

This uniaxial law $\sigma(\varepsilon)$ is based on the partition of the total strain ε between the strain in the steel bar ε_s and the slippage between steel and concrete s (Fig 7-a). This partition can be written incrementally:

$\Delta\varepsilon = \Delta\varepsilon_s + \Delta s/L_{anc}$. with $\Delta\varepsilon_s=\lambda.\Delta\varepsilon$ and $\Delta s = L_{anc}.(1-\lambda).\Delta\varepsilon$

L_{anc}: Length of anchorage or splices

λ: Partition factor between the 2 types of deformations.

The axial stress in the steel bar σ_s and the bond stress τ are given by 2 appropriate constitutive laws respectively for steel rebar $\sigma_s(\varepsilon_s)$ and for bond slip $\tau(s)$. A law similar to the Eligehausen law [3] has been adopted for bond slip (Fig 7-c).
The partition factor λ can be calculated iteratively with the static equilibrium between the force in the steel bar F_{steel} and the bond stress F_{bond} which is supposed constant on the complete length of the anchorage or lap splices (Fig 7-b). An iterative modified Newton-Raphton algorithm is used to verify the equilibrium.

$\Delta F_{steel}+\Delta F_{bond}=0=f(\lambda)$

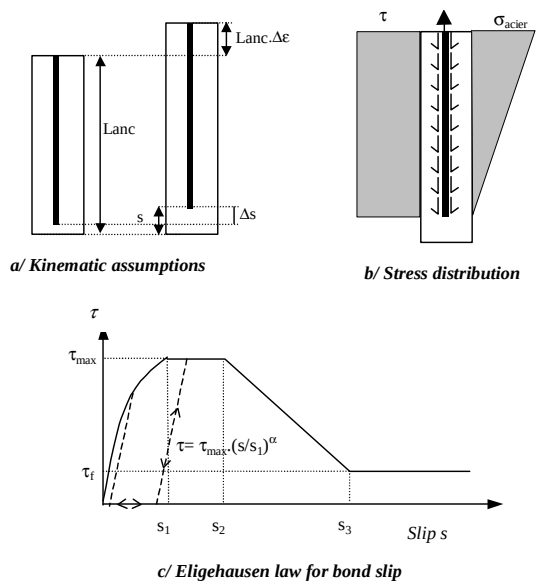


Figure 7: Phenomenological uniaxial law for anchorage and lap splices

APPLICATION TO RC COLUMNS UNDER CYCLIC LOADING

EXPERIMENTAL RESULTS

In a recent past, several experimental research programmes focused on the influence of detailing used between 1950 and 1980 in United States on the seismic behaviour of the structural members. The results of the tests performed by Aboutaha at Austin University on reinforced concrete columns under horizontal cyclic static loading have been used for the present study [1].
Aboutaha realized a series of tests on flexural columns (aspect ratio H/L=6) with square and rectangular sections characterized by insufficient lap splices: splice length of 20 diameters which used to be very common in US and only 2 stirrups in the lap splices region (Fig 8). Several columns have also been strengthened with different types of steel jacketing.

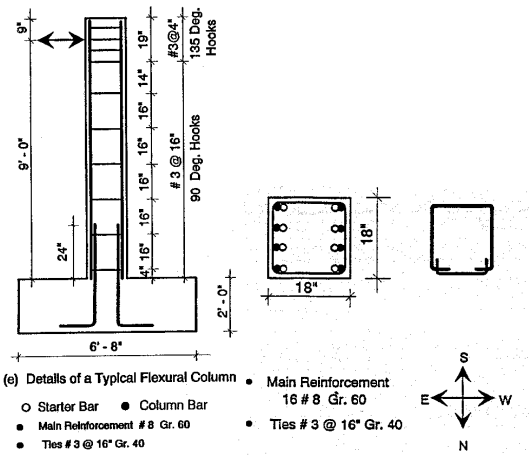
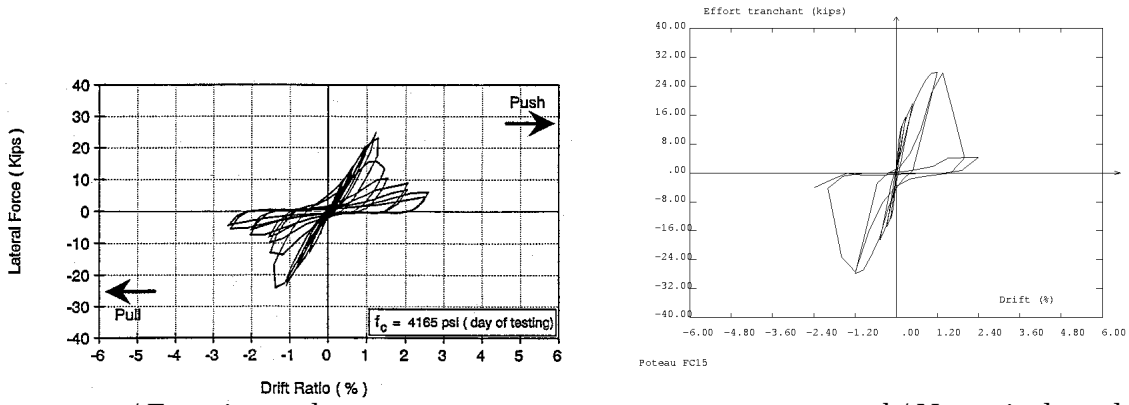


Figure 8: Geometry and characteristics of the FC15 column with insufficient lap splices

APPLICATION TO THE COLUMNS WITH INSUFFICIENT LAP SPLICES

The fiber beam element has been applied to the modeling of the flexural column FC15 whose lap splices failed before developing the flexural strength of the section. A unique beam element and the special law for anchorage and lap splices have been considered for the plastic hinge at the base of the column.

The Priestley formulae gives a length of plastic hinge of 37.2 cm for a column height equal to 274.3 cm and diameter of the steel bars of 25.4mm (#8).
The physical length of $20d_{bar}$ has been considered for the lap splices. The bond characteristics recommended by Eligehausen for unconfined concrete (bond strength τ_u=5 MPa) has been chosen for the bond slip model.
The upper part of the column has been discretized by 7 Timoshenko beam elements with non linear constitutive law for concrete and steel.
The failure mechanism and the global strength observed during the tests have been well captured by the numerical model (Fig 9). Important softening can be observed also in the computation after having reached the maximum strength which is equal to 124 kN (27.9 kips) in the calculation versus 111 kN (25 kips) measured experimentally. These values can be compared to the strength of the FC 17 flexural column which is equal to the FC15 column but strengthened with a steel jacket: 147 kN (33 kips) in the calculation and 142 kN (32 kips) experimentally.
Despite this good agreement between numerical and experimental results, the model adopted in this work for the anchorage and the lap splices can present some difficulties if columns with higher length of splices ($30d_{bar}$ or $40d_{bar}$ which are current values in some part of Europe such as in France) but insufficient numbers of stirrups are considered since the bond strength does not depend on the local ductility demand in the steel bars. The lack of experimental results on reinforced concrete columns with this type of detailing used in Europe must also be highlighted.



a/ Experimental curves b/ Numerical results

Figure 9: Force-displacement relationship for the column FC 15 (failure of the lap splices)

METHODOLOGY OF ANALYSIS OF THE BEAM-COLUMN JOINTS

EXPERIMENTAL RESULTS

A large series of static and dynamic tests on structural elements and frames has been performed in Buffalo (NCEER) in order to assess the seismic behaviour of the reinforced frame structures built without or with few seismic detailing in the United States [2].

Within this experimental campaign, several exterior and interior beam-column assemblages have been tested under horizontal cyclic loading. The tests have shown the external beam-column joints are the most critical and the present paper focused on such an assemblage.

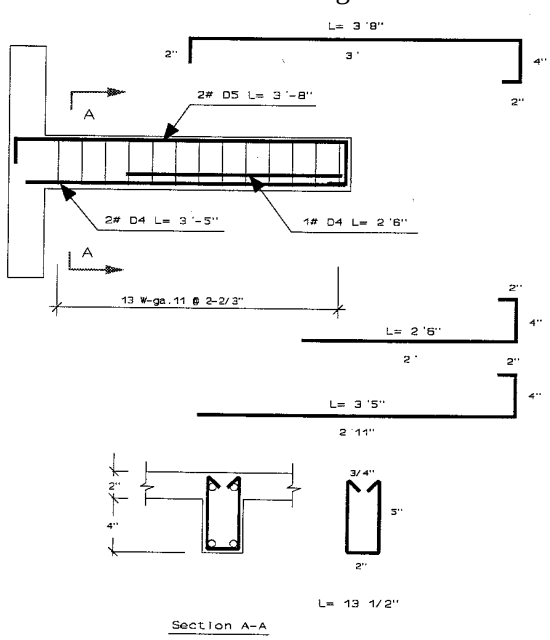


Figure 10: Reinforcement of the exterior beam-column assemblage

The steel reinforcement and the global force-displacement experimental curve are respectively given in Fig 10 and 11 for the assemblage studied in the following part of this chapter. This beam-column assemblage is made of a column with a square section and large stirrups spacing and a beam with a T-section (including the slab). Lap splices of the flexural column reinforcement are placed just above the

beam-column joint. The non symmetry of the beam section (in geometry and steel reinforcement) makes it much weaker when the bottom fiber of the beam is in tension. Furthermore the bottom steel bars of the beam have insufficient anchorage in the joint (8 d_{bar}).
On Fig 11, the global force-displacement curve is compared to the horizontal forces corresponding to different failure mechanisms (yielding of the bottom steel bars in the beam, failure of the beam steel anchorage, yielding of the top beam steel with and without considering the steel reinforcement in the slab). This figure shows the importance of the slab in the estimation of the global strength of the structure and so the local force demand in the other structural members.

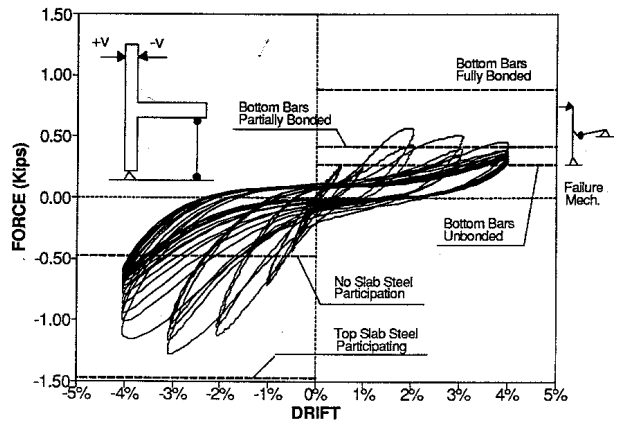


Figure 11: Experimental force-displacement curve for the exterior beam-column joint

MODELLING OF THE BEAM-COLUMN JOINTS WITH TIMOSHENKO BEAM ELEMENTS

The beam-column joints are usually considered as rigid in the non linear computations. Seismic assessment guidelines such as FEMA 273 [5]recommend to verify the shear demand in the beam column joints but few mean of computation of this forces is specified for the time-history or push over analysis.
It is possible to estimate the shear force and so the average shear stress to be transmitted by the joint using one vertical beam element in the beam-column joint and some kinematical restraints (Fig 12).
The displacements and rotations of the points at the faces of the joint are noted:
- Upper column (PN_{upper}): UX_{upper}, UY_{upper}, RZ_{upper}
- Lower column (PN_{lower}): UX_{lower}, UY_{lower}, RZ_{lower}
- Left beam (PN_{left}): UX_{left}, UY_{left}, RZ_{left}
- Right beam (PN_{right}): UX_{right}, UY_{right}, RZ_{right}
The kinematical relationships necessary to avoid the rigid body motions and the transfert of forces and bending moments between beams and columns can be written as:
- Restraint of the horizontal and vertical translations of the center of the beam-column joint:

$$UX_{joint}=0.5\ (UX_{right}+UX_{left})=0.5\ (UX_{upper}+UX_{lower})$$

$$UY_{joint}=0.5\ (UY_{right}+UY_{left})=0.5\ (UY_{upper}+UY_{lower})$$

- Continuity of the beam (there is no horizontal beam element crossing the joint)
- The nodes PN_{right} et PN_{left} have a rigid body motion (translations et rotation)
- The bending moments at the nodes PN_{right} and PN_{left} are transmitted from the beams to the columns only by horizontal shear forces to the points PN_{upper} and PN_{lower}. This can be insured by the following kinematic relationship

$$RZ_{right}=(UX_{lower}-UX_{upper})/H_{joint}\ (=RZ_{left})$$

H_{joint}: Joint height

These kinematical relationships allow to compute the shear force in the vertical beam element representative of the beam-column joint in accordance to the commonly adopted distribution of shear force in the columns and joints ([8] and Fig 12).
The previous non linear shear law with softening (Fig 6.d) gives the possibility to check the brittle shear mechanism of the beam-column joint.
It must also be noticed such modelling allows to check the anchorages of the steel bars of the beam into the joint and the lap splices in the columns since such verifications are performed in the plastic hinges.

APPLICATION

The exterior beam-column assemblage described in previous chapter has been analysed using the previous modeling approach with a non linear shear law for the joint and the uniaxial law for the anchorage of the lower steel bars of the beam and the lap splices of the upper column.
An horizontal loading controlled in displacement has been applied with a cyclic history. It must be noticed the same vertical loading than during the tests has been applied onto the column. The vertical load N depended directly on the horizontal reacting force V in the horizontal actuator:
N=5+2V (kips)
The joint shear strength τ_j has been chosen in accordance to FEMA 273:

$$\tau_t = \lambda \cdot \gamma \cdot \sqrt{f_c}$$

It comes for the present assemblage a maximum joint shear stress equal to 6.22 MPa. (γ=15).
For the flexural steel anchorage and lap splices, the maximum bond strength has been chosen equal to 5 MPa.
The Figures 13a and b show respectively the global force-displacement curve and the deformed mesh for the negative maximum displacement.
Under positive loading, the anchorage of the bottom steel in the beam is critical in both the computation and the experiment.
Under negative loading, higher value of force has been reached since all the steel reinforcement of the upper slab has been considered for the beam section. The numerical model has shown a brittle failure in the beam-column joint since the maximum shear stress has been reached in the beam element representative of the joint. During the experiment, the flexural steel bars of the column failed in tension at the interface between the lower column and the joint. The computation has also slightly overestimated the global strength of the assemblage.

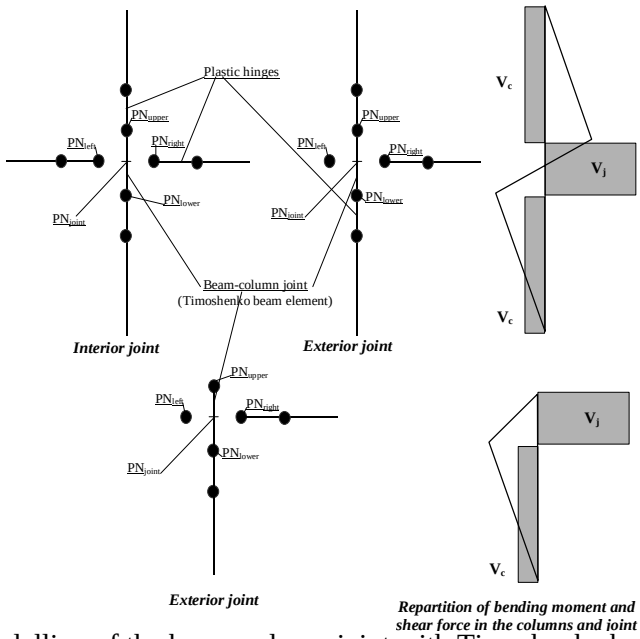
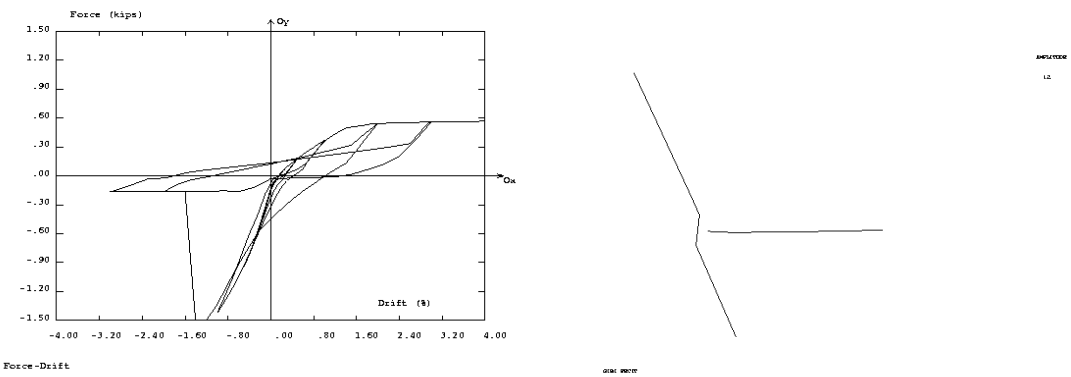


Figure 12: Modelling of the beam-column joint with Timoshenko beam finite element



a/ Force-displacement curve b/ Deformed mesh under negative loading

Figure 13: Main numerical results for the exterior beam-column joint

CONCLUSIONS

The present paper gives some general consideration about the application of non linear modeling to the seismic assessment of existing reinforced concrete buildings.

The modelling approach is based on non linear Timoshenko beam elements and uniaxial constitutive laws which can take into account some details of construction such as the anchorages, lap splices, confinement, etc… A methodology of verification of the beam-column joint has also been given. The application of these non linear models to several experimental results has shown their capabilities to catch some brittle failure modes although some limitation has been highlighted.

The present work emphasizes the necessity of experimental results for the validation of such numerical approach and their acceptance for the assessment of existing facilities. The experimental capabilities of

the CEA Laboratory allows to test complete structures in order to qualify and validate the analytical approach.

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