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Autor: Jaspart, Jean-Pierre / Vandegans, Didier

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Modelling of semi-rigid connections : column bases.

Jean-Pierre JASPART
Research Associate FNRS
Doctor Engineer
University of Liège

Didier VANDEGANS
Civil Engineer
CRIF Steel Construction Dpt.

Liège
BELGIUM

Jean-Pierre Jaspart, born in 1962, got his Civil Engineering Degree in 1985 and his Ph.D Degree in 1991.
He is the author of more than 60 papers on connection and frame design, one of the member of the Drafting Group of EC3, Annex J and Chairman of the COST C1 Working Group on *Steel and Composite Connections*.

Didier Vandegans, born in 1971, got his Civil Engineering Degree in 1994 at the University of Liège. His main domain of activity is the semi-rigid connections.

Summary

The column bases have a high semi-rigid behaviour. In this paper, a mechanical model to predict their moment-rotation response is presented. To achieve this goal, the component method described in Annex J of Eurocode 3 (EC 3) is used and extended. Comparisons with experimental tests is performed.

1. Introduction

In the daily practice, the column bases are usually considered as rigid or pinned. Experiments have shown, that in fact, they can have a high semi-rigid behaviour which influences the frame response; in particular the frame lateral deflections and the frame stability in unbraced frames, the columns stability in braced frames. Taking this semi-rigid effect into account leads to significant cost savings linked to the reduction of the man power necessary to realise rigid column bases (less stiffening) or of that of the column size in case of pinned column bases.

Analytical formulas are now available to evaluate the strength of the column bases. The prediction of the stiffness is much more complex, because of the influence of the normal force and of the loading history.

A way to solve the problem of loading history is to develop a mechanical model, based on the component method, which can react by itself to the applied forces. Such mechanical model is described in this paper and the comparisons with experimental tests are shown.

2. Test description

Twelve experimental tests have been carried out recently in Liège [3]. The general configuration is the same for each of them and is described on figure 1

The column profiles are HEB 160 made of S355 steel.

Two sorts of configurations are considered :

- connections with two anchor bolts (traditionally considered as pinned);
- connections with four anchor bolts (traditionally considered as rigid)

Two different thicknesses for end-plate are used : 15 and 30 mm.

A normal compression force is first applied to the column; it remains constant during the whole test. In a second step, the bending moment is progressively increased.

Three different values of normal force are applied : 100 kN, 600 kN and 1000 kN.

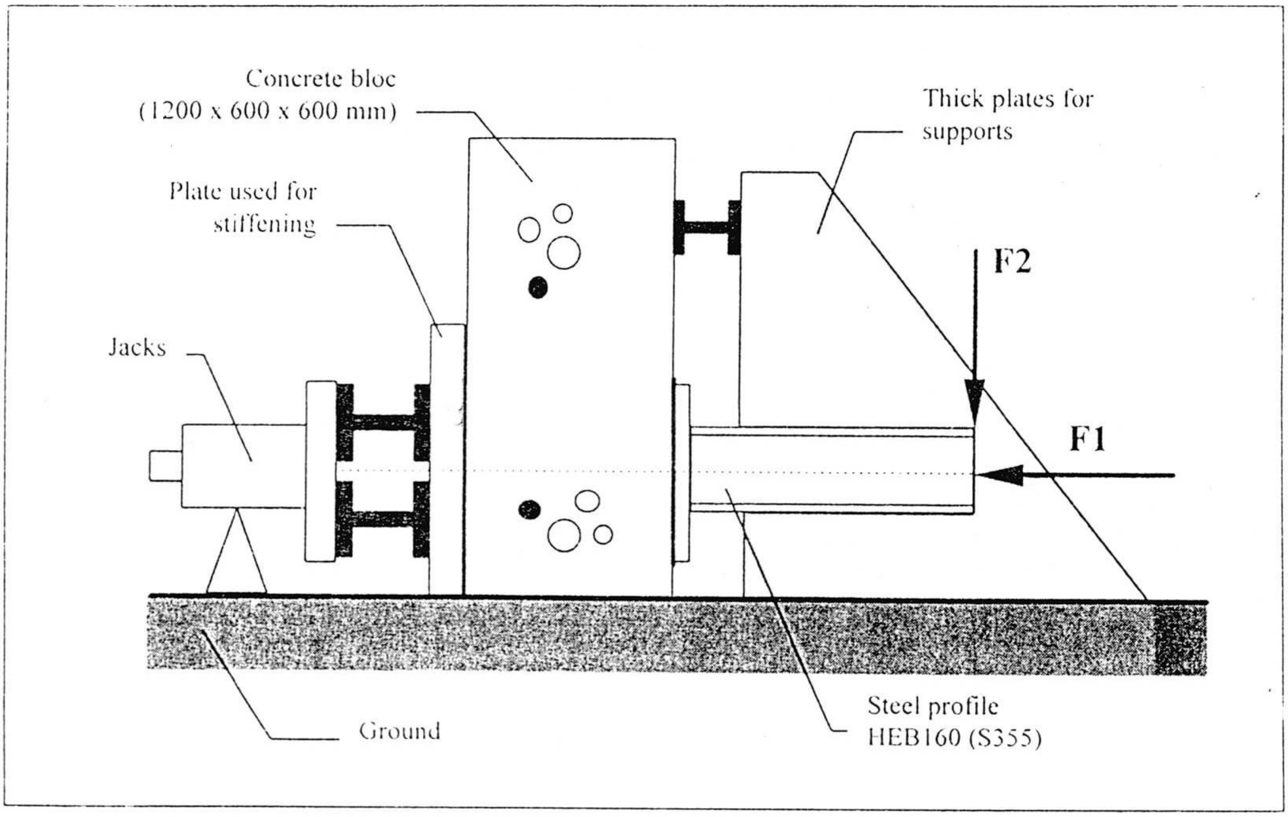


Fig. 1. Test configuration.

3. Experimental curves

Because of the connection deformability and of the high applied normal forces, second order effects cannot be neglected when interpreting the test results. Figure 2 shows how the bending moment in the connection is influenced by the compressive force in the column.

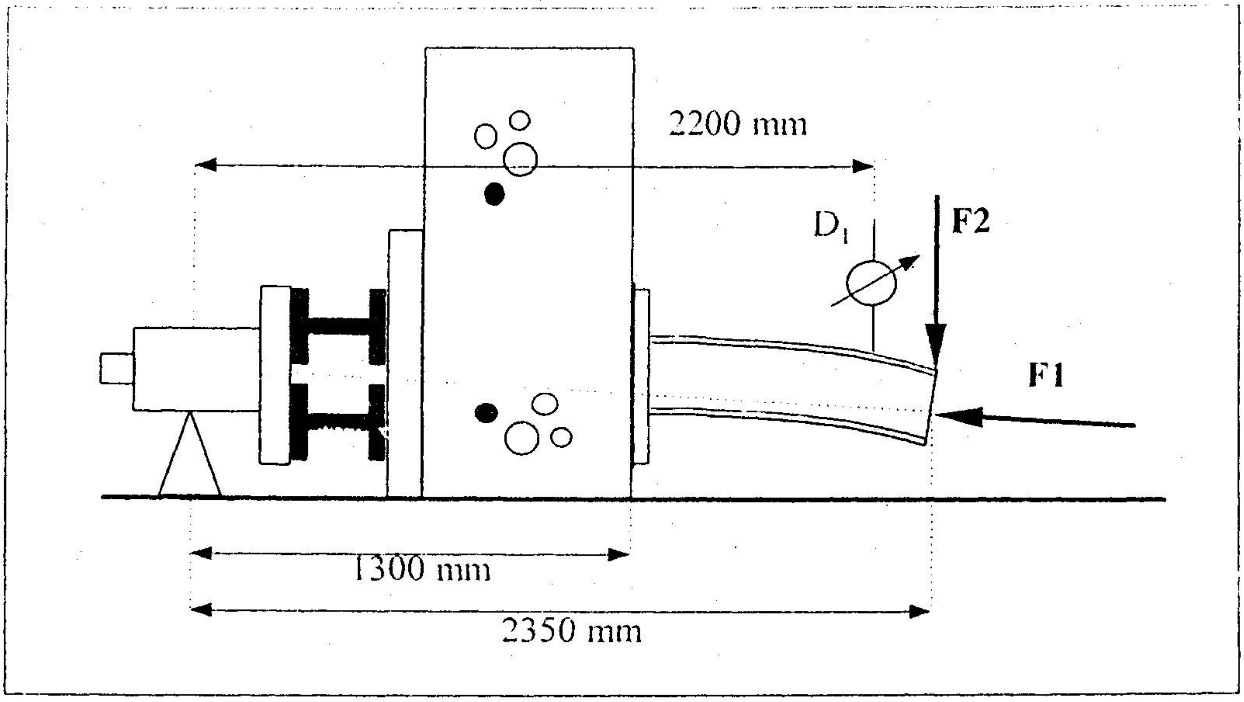


Fig. 2. Determination of the joint bending moment.

Figure 3 shows a comparison between the moment-rotation curves for the tests with 2 and 4 anchor bolts in the case of a 15 mm thick end-plate.

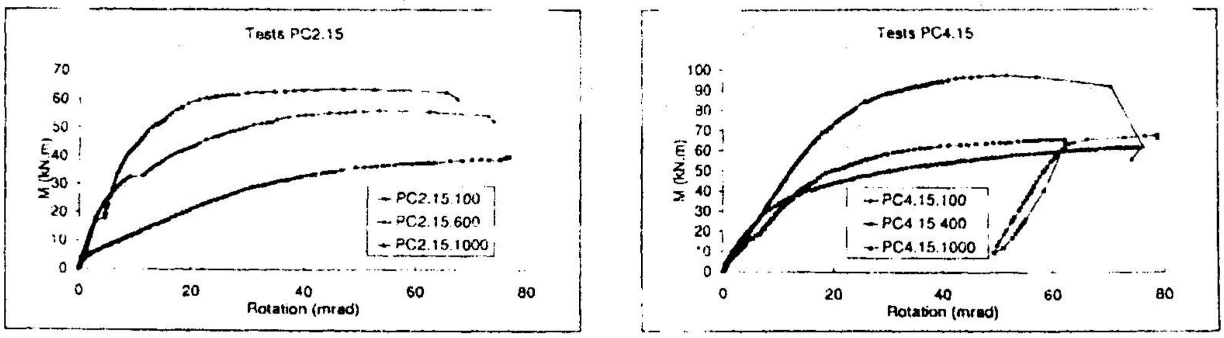


Fig. 3. Moment rotation curves

While examining these figures, it can be seen that the highest the normal force in the column, the highest the bending resistance of the joint. For tests with similar geometries, the initial stiffness seems not to be affected by the value of the normal force in the column. For higher moments, the stiffness of the connection decreases when there is a loose of contact between the plate and the concrete in the tension zone. This loose of contact appears earlier for a low normal force in the column.

4. Mechanical modelling

4.1 Generals

The aim is to develop a model for column bases, based on the component method described in Annex J of Eurocode 3 [1]. First it is necessary to identify the different behaviour aspects to be covered in order to describe correctly the behaviour of the column bases.

According the observations made during the experimental tests, it can be said that :
- the contact between the plate and the concrete is a complex phenomenon, which must be modelled in a very refined way;
- the bond between the anchor bolts and the concrete is quickly breaked. Therefore, it might be assumed that the anchor bolts are free to extend in tension, from the beginning of the loading;
- under the column flange in compression, the plate deforms significantly. Therefore, the pressure under the plate is far from being uniform, even under central compression. The concept of the equivalent rigid plate to which it is referred to in EC 3 Annex L [2] is kept in this model;
- in the compression zone, the extended part of the plate has a very high influence, because it prevents crushing in the concrete. The development of a plastic line is observed in the extended part during the test. This plastic line requires a large deformation energy and it is necessary to model it;
- a plastic hinge may form in the steel profile. This can lead to significant local deformations. In order to compare the mechanical model to the experimental moment-rotation curves (which include these deformations), it is imperative to take this source of deformation into account;
- the column base deforms during the loading. In particular the contact zone and the lever arm of the internal forces are changing.
  Furthermore, the behaviour of each component (concrete, anchor bolts, plate, profile, ...) is non linear. Therefore, only an iterative procedure allows to describe correctly the connection behaviour for the whole loading.

The mechanical model shown in figure 4 is based on these observations.

The following components can be identified :
1) extensional springs for the deformation of the profile. They are working in tension and compression;
2) extensional springs for the deformation of the anchor bolts and the plate subjected to transverse anchor bolt force. Only one spring is used for an anchor bolt row. It works only in tension;
3) extensional springs for the concrete under the plate; they work only in compression;

4) springs in bending for the plastic deformation of the plate in the compression zone(s). These springs are activated when the extended part of the plate in the compression zone is subjected to contact forces with concrete.

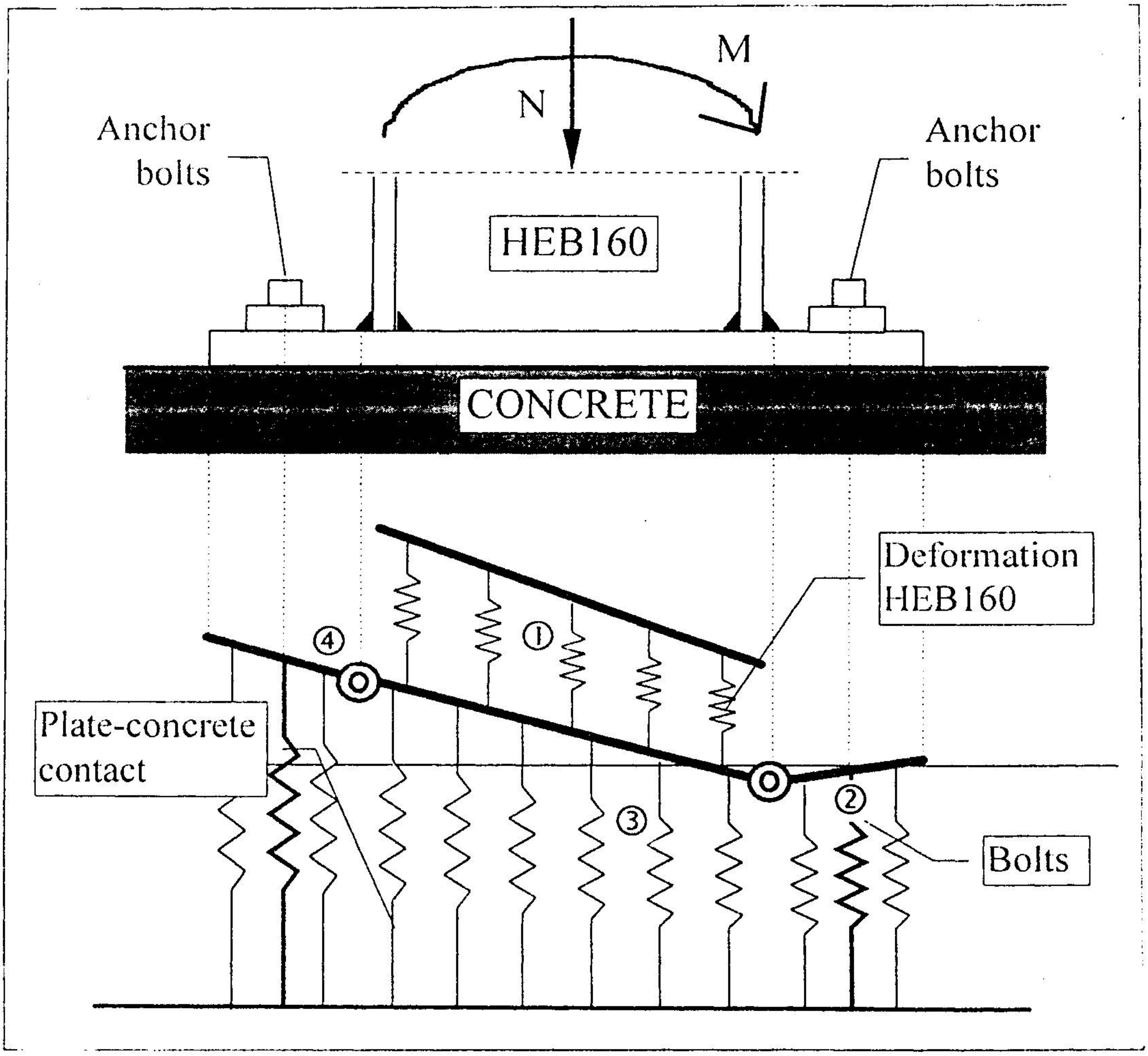


Fig. 4. Modelisation of the column bases connections

4.2 Behaviour of the individual components

4.2.1 Concrete

The plate-concrete contact is a very complex phenomenon, because the contact zone varies with the eccentricity of the compressive forces as well as with the flexibility of the plate, directly linked to its thickness.

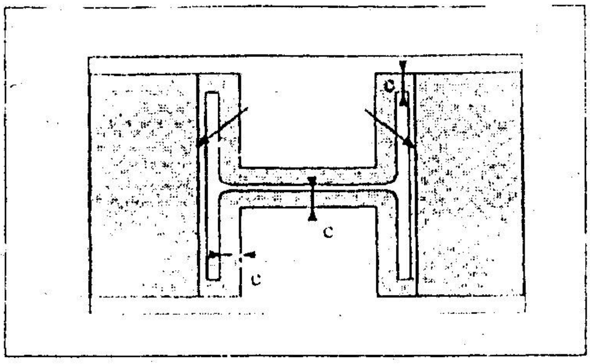


Fig. 5. Equivalent rigid plate

The concept of equivalent rigid plate described in Annex L of Eurocode 3 is kept. Figure 5 shows how the plate is idealized. It might be surprising to keep a so large equivalent plate outside the flange, but this part has not a very high influence on the connection behaviour.
The behaviour law σ - ε adopted in the model is the classical parabolic-rectangular law. The concrete-plate contact is modelled by a finite number of springs: each of them corresponds to a small part of the contact zone. A hundred of such springs leads to a good level of accuracy.

4.2.2 Anchor bolts and plate

The local response of the anchor bolts in tension and the plate depends on the thickness of the plate and of the position of the bolt rows : inside or outside the flange.

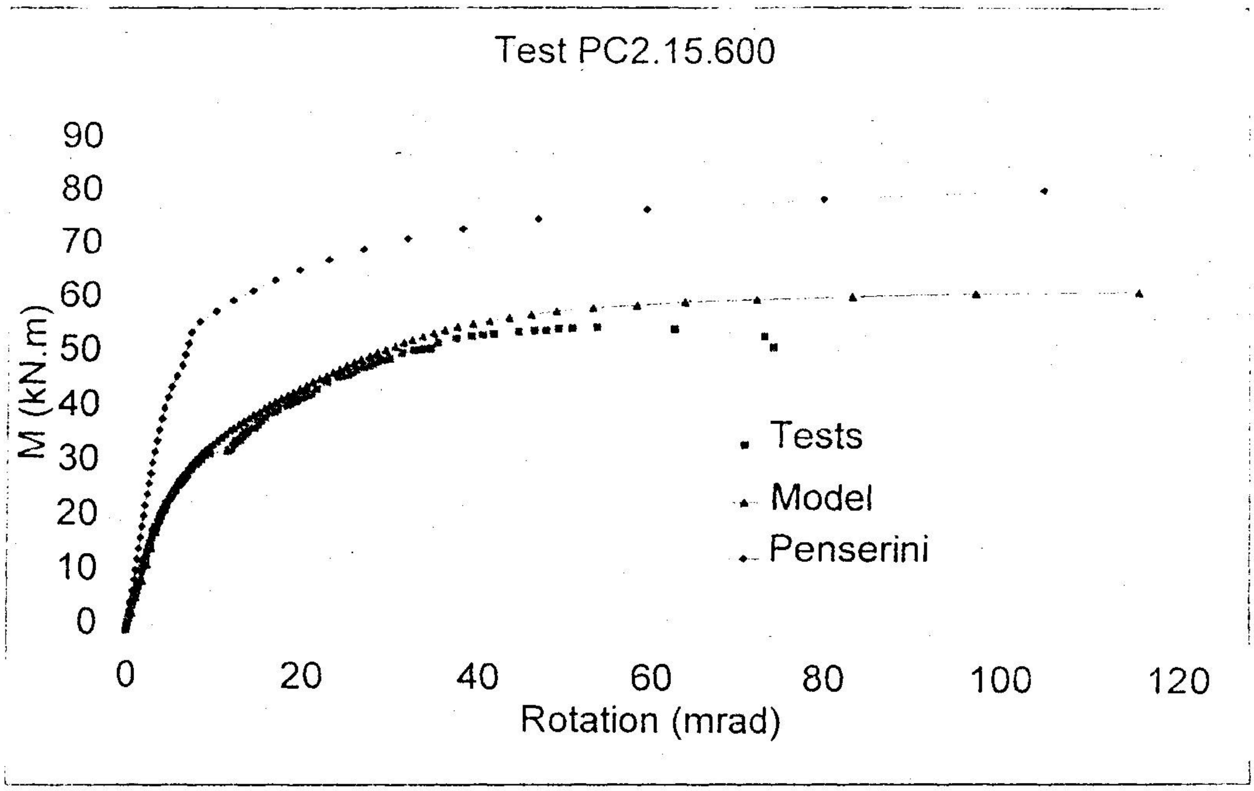
EC 3 Annex J is used for the determination of the behaviour curve of these components. For the end-plate deformability, it has been assumed that no prying effect occurs between the concrete and the edge of the end-plate in the tension zone. This assumption is justified as follows :

- the anchor bolts have a very high deformability. Therefore the resulting relative displacement between the plate and the concrete is significant; sufficiently to be considered as higher than that due to the flexural deformation of the plate, excepted for very thin plates, but these ones are usually not used for column bases;
- the prying effects result from a concrete-to-plate contact. Even if this contact develops, the high deformability of the concrete under these concentrated forces prevents an important prying force to develop as in case of steel-to-steel contact.

In the compression part, the plate also deforms. Tests have shown that this deformation is very local and can be assimilated to a plastic hinge. This one is modelled through the use of a spring in bending characterized by an elastic-plastic law in the compression zone. The spring is infinitely rigid in the tension zone.

4.2.3 The steel profile

Because of the high normal forces in the column, this one might partially yield. An elastic-plastic behaviour law is adopted for the related springs.



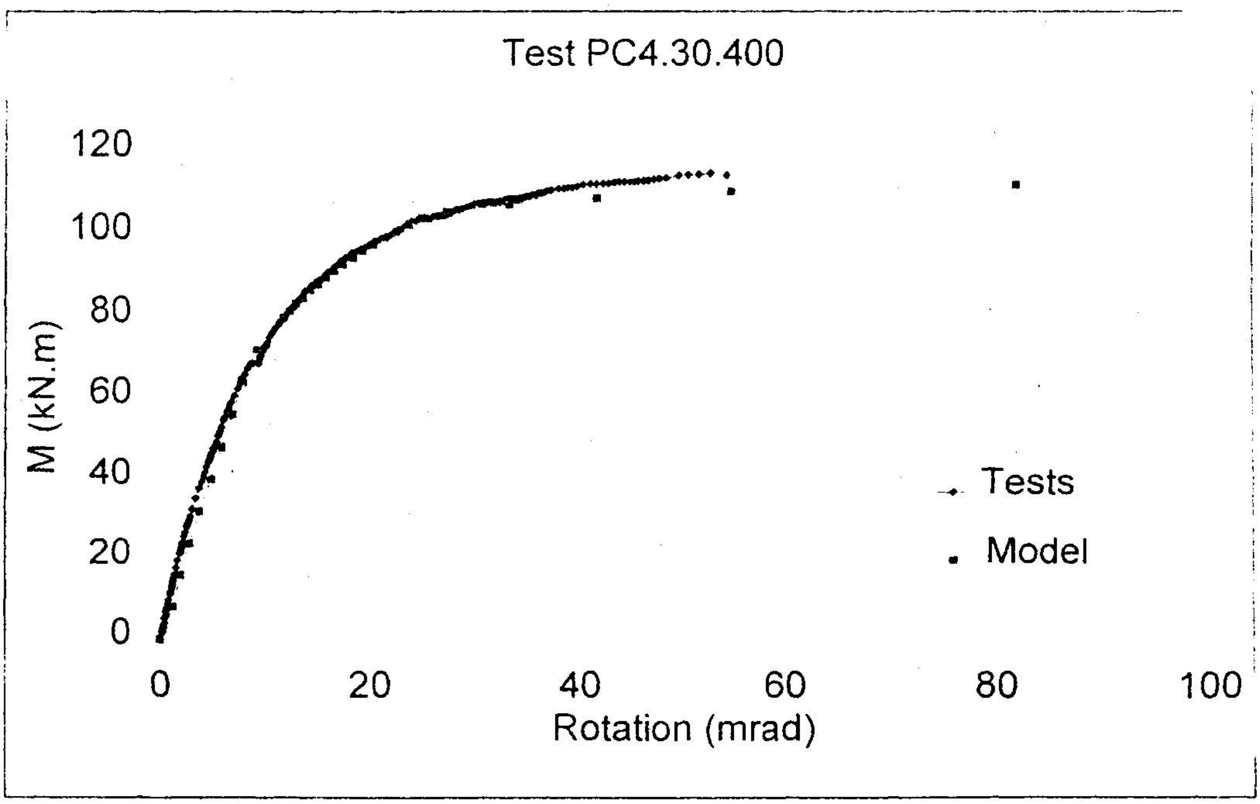


Fig. 6. Comparison between the model and the experimental curves

5. Comparisons with experimental tests

It is not possible to report on all the experimental tests in this paper. A full comparison can be found in the original research report [3]. Figure 6 shows the comparison for test PC2.15.600 and PC4.30.400.

In figure 6, the response predicted by the Penserini model [4] is also given for tests with two anchor bolts. The agreement between that model and the experiment is far from being satisfactory. It has however to be said that the tests considered here are outside the range of validity of the Penserini model.

For certain tests, some problems related to the execution has occured (concrete, anchor bolts). Therefore, the comparison between those tests and the model was difficult. More details are given in [3].

As a conclusion to the full comparison, it can be stated that :

- For the tests with two anchor bolts, the initial stiffness is very well predicted by the model. The progressive yielding of the connection is also well covered by the theory.
- There are small discrepancies at ultimate state (5 to 10 %). This can be explained by the quite complex ultimate behaviour of the different connections components at ultimate state.
- Close to ultimate loading, the deformation of the column bases are quite important, this leads to modifications in the geometry, which are not taken into account in the model.

6. Conclusions

Experimental tests have been carried out on column bases with two or four anchor bolts. They have shown that the column bases have a very high semi-rigid behaviour, even for so called nominally pinned connections; this is known to be potentially beneficial when designing building frames.

A mechanical model is developed, based on the component method described in EC 3 Annex J. The non-linear behaviour of the different components is taken into account. Therefore, only an iterative procedure allows to describe correctly the connection behaviour for the whole loading. Furthermore, with such model, the history of the loading can be taken into account.

A comparison between the experimental curves and the model is given. The accuracy may be qualified as good, even if some small discrepancies occur at the ultimate state.
Such a model is helpful in view of further investigations which would be aimed at developing a far more simple design procedure for practitioners.

7. References

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