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Application of the component method to column bases

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Abstract

Column bases transfer reactions from the structure to the foundation. When subjected to normal forces, shear forces and in-plane bending moments, they deform, particularly in rotation. This rotational behaviour is usually idealized as pinned or fully rigid. But in most of the cases column bases have a high semi-rigid behaviour which influences significantly the global frame response. 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 is used and extended. According to the component method, any structural joint is considered as a set of individual components and the determination of its mechanical properties as strength and rotational stiffness includes three main steps: (i) definition of the constitutive components, (ii) evaluation of their mechanical properties and (iii) assembly of the components to derive the joint properties. Lastly, comparisons of the mechanical model with experimental laboratory tests on column bases are performed.

Keywords: Component method; Column bases; Experiments; Mechanical model; Strength; Rotational stiffness; Eurocode 3

1. Introduction

The semi-rigid behaviour of column bases influences the structural frame response and in particular:

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- the frame lateral deflections and the global frame stability in unbraced frames;
- the column 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 to the reduction of the column and/or beam size in the case of pinned column bases.

The rotational behaviour of the column bases is known to depend on the normal force acting in the column and the loading history. Analytical formulae for strength evaluation are now available [3,6]. The prediction of the rotational stiffness is, however, much more complex. In this paper, a mechanical model based on the component method [1,4] and suitable for strength and stiffness evaluation is presented and comparisons with experimental tests are shown.

2. Experimental tests

2.1. Test set-up and configurations of the specimens

Twelve experimental tests on column bases have been recently carried out in Liège. They all feature an identical general configuration as shown in Fig. 1. For technical reasons the tests were carried out with a compressive force F_1 in the column acting horizontally, whereas the force F_2 generating bending moment was acting vertically.

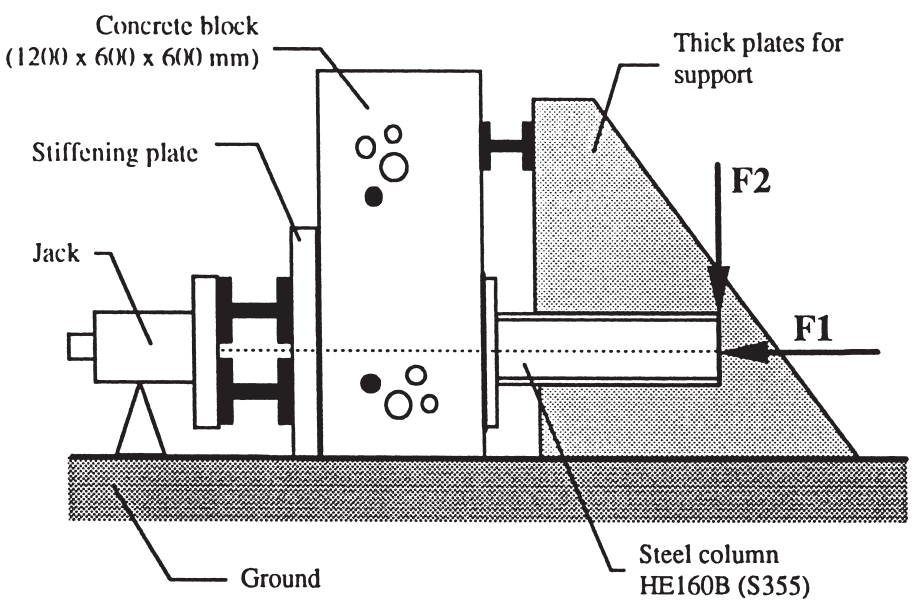

Fig. 1. Test configuration.

The compressive force F_1 is applied—through the use of high strength tensile bars represented by dashed lines in Fig. 1—by means of two jacks acting on the rear face of the concrete block. Thanks to the extremely thick stiffening plate (10 cm), the distribution of the stresses inside the concrete block may be considered to be the same as it would be if the block was placed directly on the ground.

Similar concrete blocks are used for all the tests: 1200 mm high for a 600 × 600 mm square base. All the blocks were concreted at the same time in order to ensure that their mechanical behaviour is as homogeneous as possible.

To prevent any movement of the block, an efficient support against the moment created by force F_2 is formed by two large thick steel plates placed on each side of the column profile (see Fig. 1). In order to be able to resist applied stresses, the concrete block has been slightly reinforced. The reinforcements, however, are placed so as not to prevent the possible formation of cracks under the action of the compressive force, as could happen in practical situations.

A thin layer of grout has been placed between the base-plate and the concrete block so as to ensure a good contact. Two types of test configurations are considered, with four (Fig. 2(a)) and two (Fig. 2(b)) anchor bolts respectively. In the first case, the column base is nearly rigid, while in the second it is usually modelled as pinned.

Only one steel column profile is considered in the test series: HE160B. Its steel grade is S355. The base-plates are welded to the column; the throat radius of the fillet welds is 6 mm. Two different thicknesses are used for base-plates: 15 mm and 30 mm. Their steel grade is S235; it is lower than that of the column profile, which allows to concentrate the collapse in the column bases themselves and not in the column steel profile.

Anchor bolts M20 10-9 are used. Such an unusal high steel grade has been selected

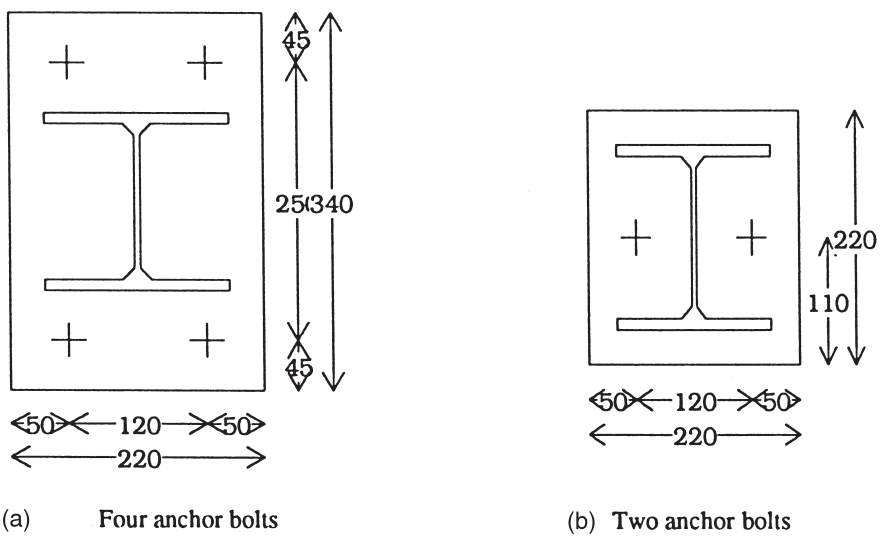

Fig. 2. Plate configurations.

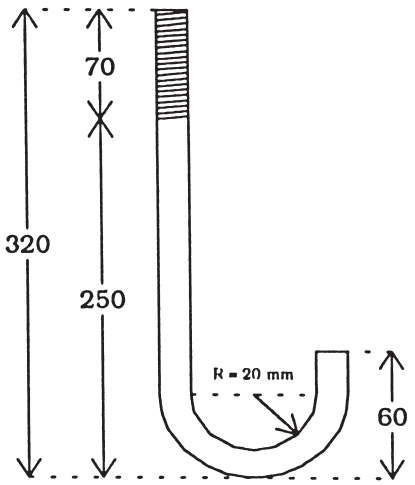

Fig. 3. Anchor bolts.

so as to avoid brittle failures during experimentation. Anchor bolts are made from steel rods curved as shown in Fig. 3. When they are subjected to tension, an unbending of the curved part of the anchor bolts is observed for a tensile force equal to 187 kN, i.e. before the tensile strength—which equals 250 kN—is reached. Such a collapse mode has been reported in some of the experimental tests.

The same loading history is applied to all the tests (Fig. 4): preliminary application of the full compressive force on the column (F_1), which remains constant, followed by a progressive application of the force F_2 until collapse.

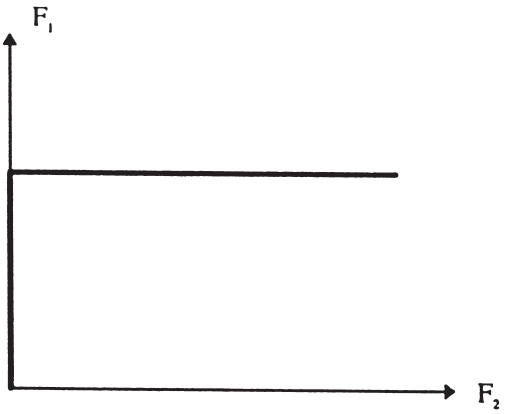

Fig. 4. Loading history.

Table 1 gives an overview of the test series and the value of the parameters which differentiate the test specimens. The designations given to the tests consist of the initial letters 'PC' followed by three numbers: the first indicates the number of anchor bolts (two or four), the second relates to the thickness of the plate (15 mm or 30 mm), while the last gives the value of the normal load applied to the column, in kN.

The actual geometrical and mechanical characteristics of all the column base components (anchor bolts, plates, profiles, etc.) have been measured in the laboratory. They are reported in Ref. [3]. In the same report, the instrumentation used in the laboratory is described and the way characteristic M-φ moment-rotation curves are derived from test measurements is carefully explained. In these curves, M is the moment acting at the interface between the base-plate and the concrete block, while φ is the relative rotation between the beam section located close to the base-plate and the concrete block.

2.2. Test results

Fig. 5 shows a comparison between the moment-rotation curves for the three tests PC2.15. It has to be noted that the higher the force in the column, the higher the bending moment resistance of the column base. However, this has not to be considered as a general rule, as demonstrated in [3].

The initial stiffness of the curves is quite similar for the three tests, as far as the very first loading steps are considered. In fact, the stiffness changes significantly as soon as a separation is observed between the plate and the concrete in the tensile zone. Obviously, the lower the initial compressive force, the more quickly this phenomemon occurs.

Fig. 6 relates to tests PC2.30. Unlike the previous figure it shows that the initial stiffness of test PC2.30.100 is much lower than that of the two other tests PC2.30. This might be explained as follows: when the blocks were concreted, the anchor

Table 1
Nomenclature of the tests

Name	Anchor bolts	Plate thickness (mm)	Normal force (kN)
PC2.15.100	2	15	100
PC2.15.600	2	15	600
PC2.15.1000	2	15	1000
PC2.30.100	2	30	100
PC2.30.600	2	30	600
PC2.30.1000	2	30	1000
PC4.15.100	4	15	100
PC4.15.400	4	15	400
PC4.15.1000	4	15	1000
PC4.30.100	4	30	100
PC4.30.400	4	30	400
PC4.30.1000	4	30	1000

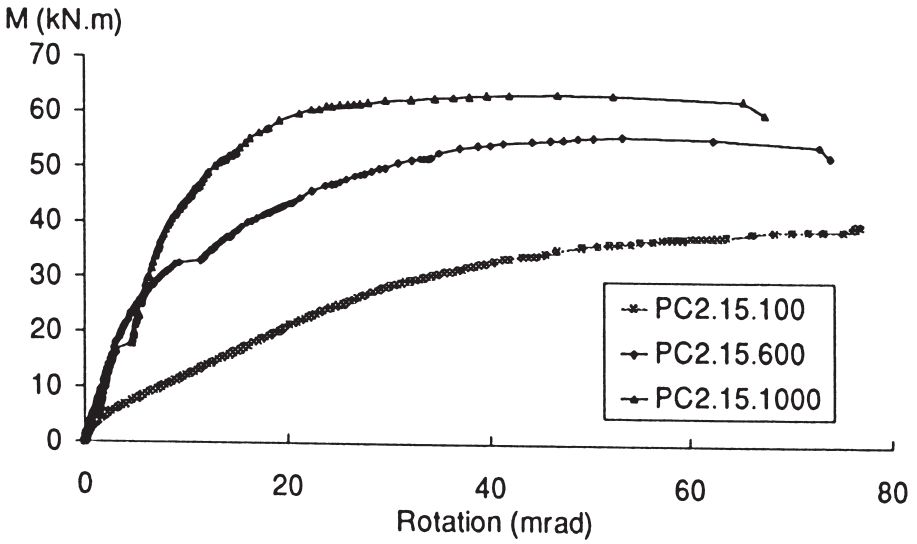

Fig. 5. M-φ curves for tests PC2.15.

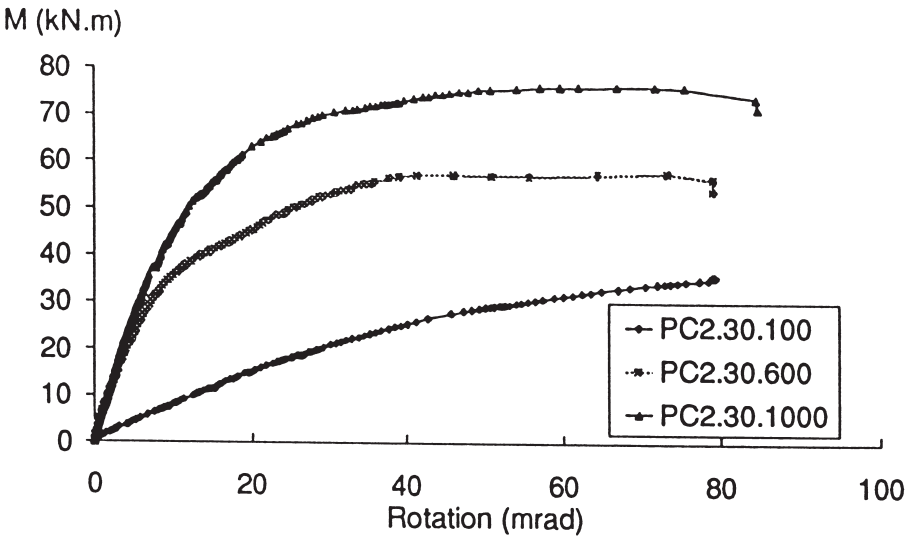

Fig. 6. M-φ curves for tests PC2.30.

bolts embedded in the concrete were held by a plywood board located at the level of the top face of the block and supported by the lateral shuttering. The fresh concrete just arrived at the level of the lower surface of the boards. As a consequence, the concrete was less well vibrated there and these less accessible zones exhibited a lower resistance, containing numerous air bubbles. The introduction of a compressive

force into the column had the effect of homogenizing the concrete located under the plate by reducing its porosity. The higher the compressive force, the lower default in the concrete, and the steeper the moment-rotation curve. This could explain the significant difference observed between the curves relating to test PC2.30.100 and the two other tests PC2.30.600 and PC2.30.1000.

By examining Figs 5 and 6, the particularly high value of the ultimate strength of these column bases traditionally considered as nominally pinned may be noted.

Fig. 7 presents the three curves for tests PC4.15. Test PC4.15.400 is the only one which has been subjected to an unloading/reloading cycle, because of various technical reasons. The significant difference in strength between test PC4.15.1000 and the two other tests PC4.15 has to be pointed out. This may be explained by the fact that the anchor bolts in tension are activated much later when the compressive force in the column is high.

Finally, Fig. 8 shows the M-φ curves for the four last tests, PC4.30. These are, a priori, the most rigid and the strongest, which is confirmed by their moment-rotation curves. Unlike the three previous figures, it has to be noted that test PC4.30.1000 does not exhibit a higher strength than test PC4.30.400. In fact, this is due to the yielding, at the end of the test, of the end section of the steel HE160B profile. In addition, local plate buckling is also observed in the column flange subjected to the higher compressive stresses.

In Table 2, the ultimate bending resistance and the collapse mode is reported for each of the 12 experimental tests. Analysis of Table 2 confirms that, for the specific tests performed, the most rigid and resistant column bases, for a given configuration, are those for which the compressive force in the column is high. The only exception

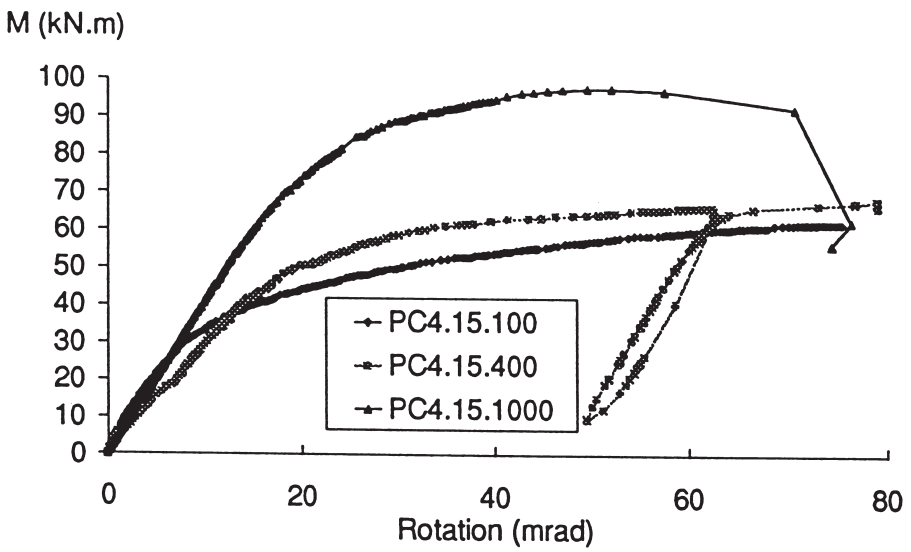

Fig. 7. M-φ curves for tests PC4.15.

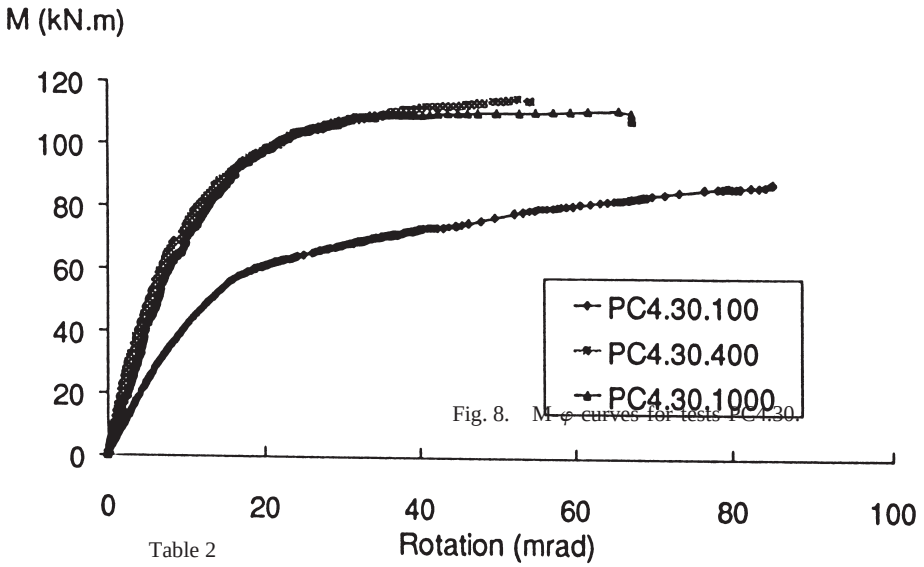

Fig. 8. M–φ curves for tests PC4.30.

Table 2
Ultimate resistaces and collapse modes of the experimental tests

Name	$M_{Ru,\ test}$ (kN.m)	Collapse mode
PC2.15.100	40	Failure of anchor bolts
PC2.15.600	56	Failure of anchor bolts
PC2.15.1000	63	Crushing of the concrete
PC2.30.100	35	Failure of anchor bolts
PC2.30.600	57	Failure of anchor bolts
PC2.30.1000	75	Failure of anchor bolts
PC4.15.100	62	Yielding of the plate
PC4.15.400	68	Collapse of the plate and of anchor bolts
PC4.15.1000	97	Yielding of the plate
PC4.30.100	86	Tearing of the anchor bolts
PC4.30.400	117	Tearing of the anchor bolts
PC4.30.1000	110	Yielding and local buckling of HEB160

is test PC4.30.1000 for which considerable yielding of the column cross-section and local instability are observed at failure.

Test PC2.30.100 differs from the other tests by a particularly low stiffness and strength. This point has been discussed here above.

3. Mechanical modelling

3.1. 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 aspects to be covered in order to describe correctly the behaviour of the column bases.

From test observations, it is seen 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 quickly vanishes. Therefore it might be assumed that the anchor bolts are free to extend in tension, from the beginning of the loading;
- under the column flanges in compression, the plate deforms significantly in bending. Therefore, the pressure under the plate is far from being uniform, even under axial 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 not to be disregarded as 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, etc.) is non linear. Therefore only an iterative procedure allows to describe correctly the connection behaviour for the whole loading.

The mechanical model shown in Fig. 9 is based on these observations. In this model, the constitutive components of the column base are represented by means of springs as follows:

1. extensional springs (tension or compression) for the deformation of the column end section;
2. extensional springs for the deformation of the anchor bolts and of the plate subjected to 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.

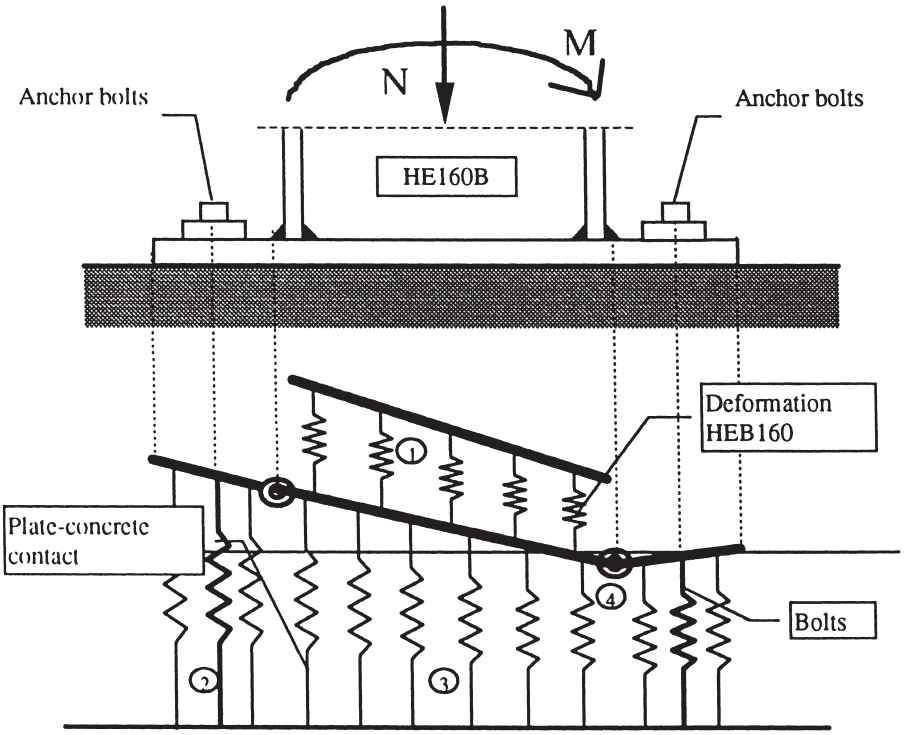

Fig. 9. Modelling of the column bases.

Each of these springs is characterized by its own deformability curve, as an individual component. In general, the stiffness and resistance properties of the springs may be derived experimentally or through specific numerical simulations. In the present work, analytical expressions have been suggested. They are all described in a detailed way in the report [3] which may be afforded to any interested reader. These expressions allow to derive the stiffness and strength properties of the components on the basis of the geometrical characteristics of the latter and the mechanical properties of the constitutive material (steel or concrete). It has to be noted that these expressions are fully based on theoretical approaches; no curve fitting or empirical procedure has been used in order to calibrate the model or to obtain a good agreement with test results. This work is briefly commented on in Section 3.2.

3.2. Behaviour of the individual components

3.2.1. 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.

The concept of equivalent rigid plate described in Annex L of Eurocode 3 [2] is

followed. The behaviour law adopted for concrete 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.

3.2.2. Anchor bolts and plate in tension

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

EUROCODE 3 Annex J [1] is used for the determination of the behaviour curve of these components. For some tests with thick base-plates, 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. More details about this particular aspect are available in [3].

3.2.3. Plate in compression

In the compression zone, 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. This spring is infinitely rigid in the tension zone, the deformation of the base-plate being covered then by that of the 'anchor bolts and plate in tension' component.

3.2.4. 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. The possible plate buckling of the column flanges and/or web in compression is not yet covered by the model.

4. Comparisons with experimental tests

Figs 10–21 present comparisons between the mechanical model described in the foregoing paragraphs and the 12 experimental tests results.

The curves shown in these figures are moment-rotation ones. They reflect the rotational behaviour of the column bases, but includes also the yielding and instability phenomena likely to occur in the end section of the column steel profiles. With regard to the mechanical model, the graph shows the variation of M as a function of θ, i.e. the rotation measured at the base of the steel profile, with due account being taken of the yielding phenomena occurring in the latter.

For the configurations with two anchor bolts, the theoretical curves obtained using the PENSERINI model developed some years ago [5] have also been plotted. It is clear that the agreement between the latter and the experimental tests is far from

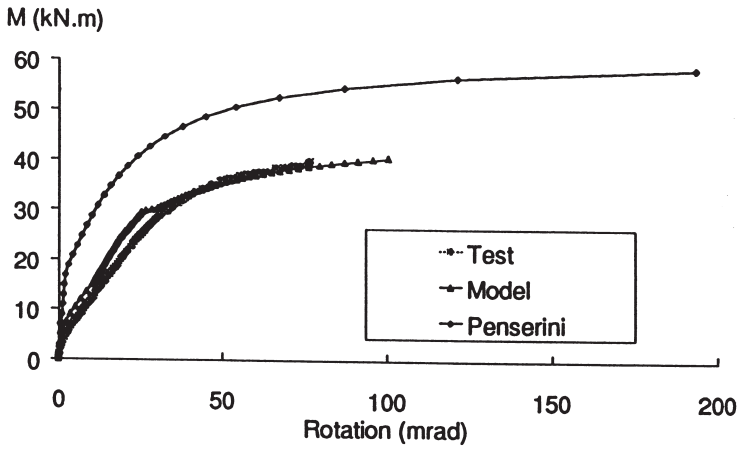

Fig. 10. Comparison between the tests and the mechanical model. Test PC2.15.100.

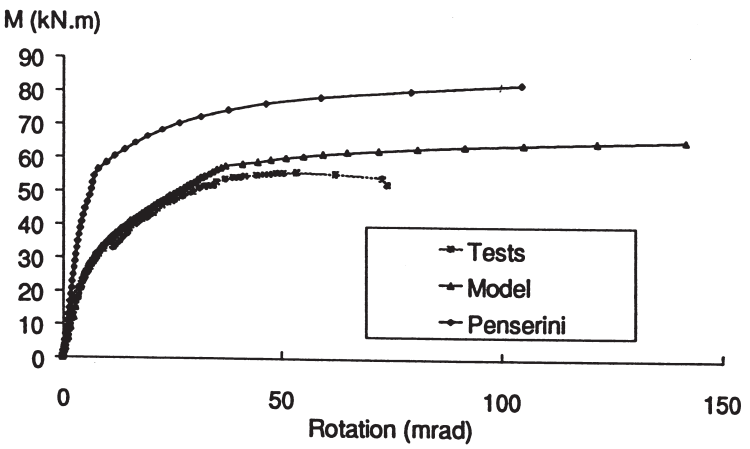

Fig. 11. Comparison between the tests and the mechanical model. Test PC2.15.600.

being convincing (Figs 10–15). This is simply because the tests carried out in Liège are largely outside the scope of the PENSERINI model.

Examination of Figs 10–12 presenting tests PC.15 (two bolts—15 mm plate) reveals an excellent agreement between the mechanical model and experimentation. The initial stiffness of the moment-rotation curves and the progressive yielding of the column bases are accurately predicted by the model. On the other hand, the agreement is seen to be less satisfactory for the ultimate load; its remains however quite acceptable (scatter of 5–10% maximum). This is due to the great complexity of the behaviour when the different components of the column base are close to collapse:

- concrete is a material whose mechanical properties can vary considerably, accord-

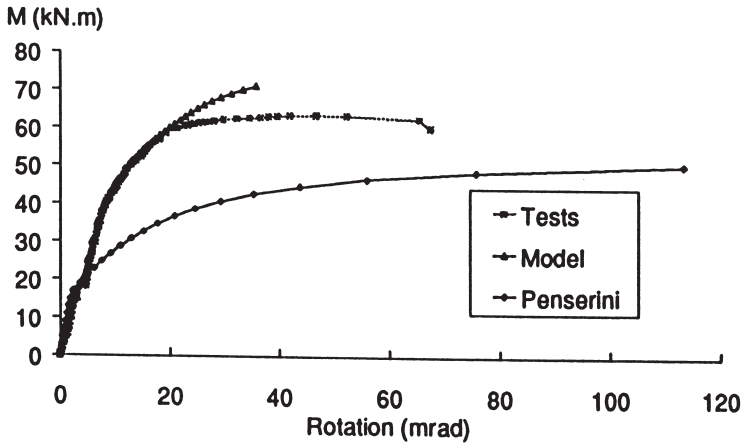

Fig. 12. Comparison between the tests and the mechanical model. Test PC2.15.1000.

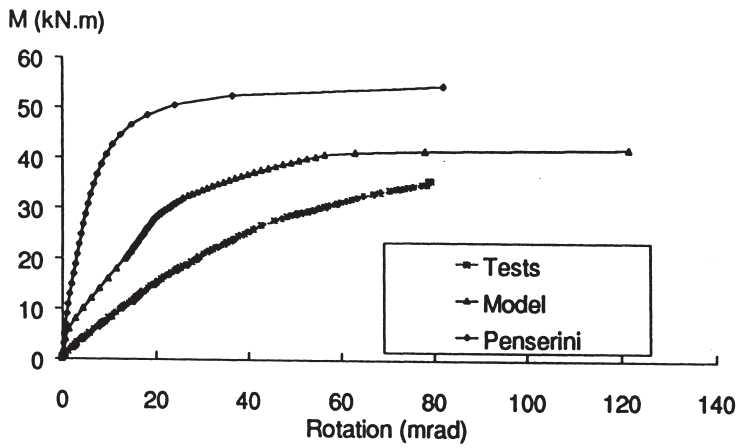

Fig. 13. Comparison between the tests and the mechanical model. Test PC2.30.100.

ing to the quality of the compaction. Furthermore, the crushing of the concrete under local forces is not an easy phenomenon to model;

- the anchorage of the bolts in the concrete is aleatory. In fact, contrary to what was expected, the anchorage of the bolts by the concrete was not sufficient enough to prevent a relative overall movement between the bolt and its support. Fortunately, these movements only occurred in the case of very high tension forces at the end of the test, and thus only altered the ultimate load.
- the displacements at the end of the test become quite significant, which leads to geometry changes which are not correctly taken into account in the mechanical model.

Despite this, and considering the curves as a whole, the general agreement between

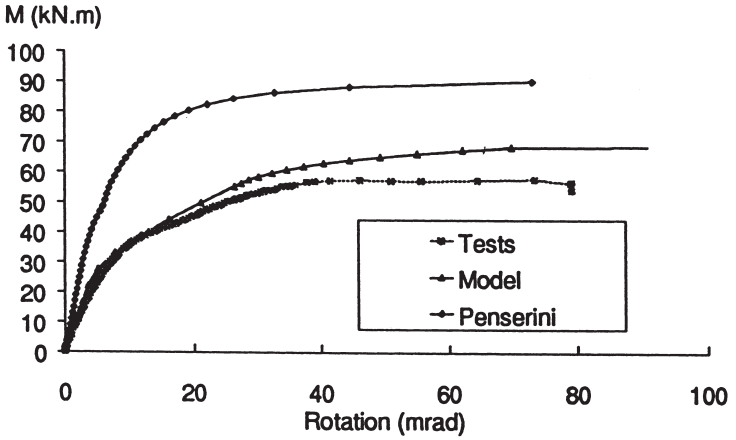

Fig. 14. Comparison between the tests and the mechanical model. Test PC2.30.600.

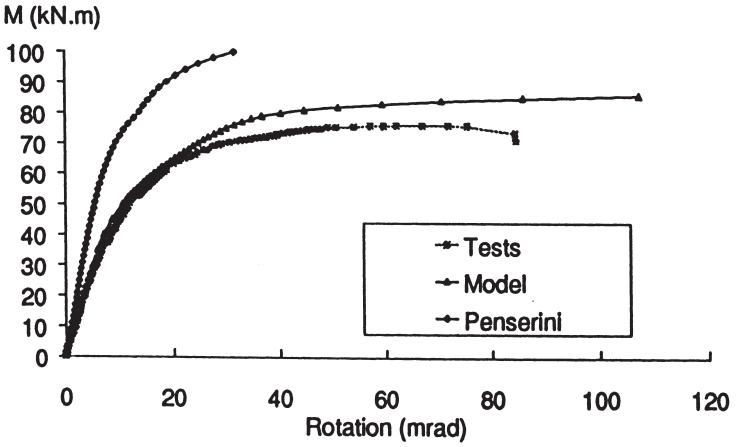

Fig. 15. Comparison between the tests and the mechanical model. Test PC2.30.1000.

the mechanical model and the tests may be regarded as quite good. A similar conclusion may be drawn from Figs 13–15 relating to tests PC2.30, except for test PC2.30.100 (Fig. 13).

Some investigations have shown, however, that the only way to come to a better agreement for tests PC2.30.100, with the model, was to reduce the stiffness of the concrete without modifying its ultimate strength. In other words, the lack of stiffness in the concrete block appears as the main reason to explain the rather poor agreement obtained in this case. In fact, that is what happened in test PC2.15.100 as explained in Section 2.2.

Figs 16–18 relate to PC4.15 tests. Again, the agreement is seen quite acceptable.

Finally, Figs 19–21 show a moderate agreement between theory and experimen-

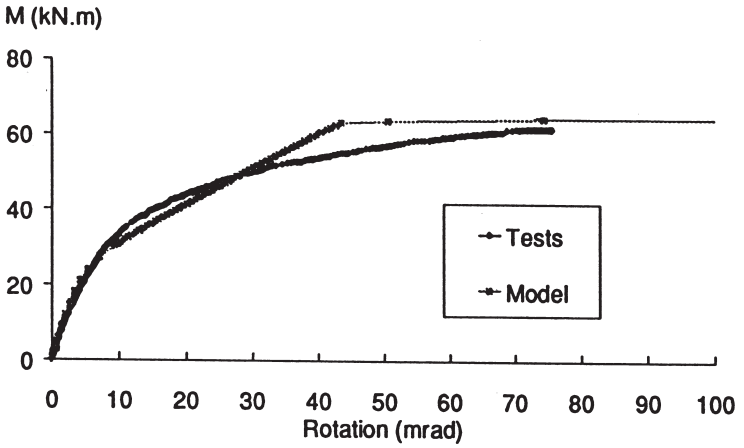

Fig. 16. Comparison between the tests and the mechanical model. Test PC4.15.100.

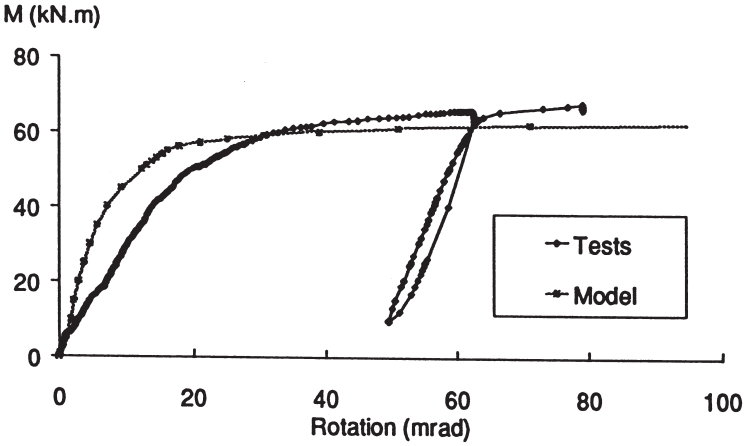

Fig. 17. Comparison between the tests and the mechanical model. Test PC4.15.400.

tation. While test PC4.30.400 (Fig. 20) gives excellent results, test PC4.30.100 seems to suffer from the same problem as test PC2.30.100, i.e. an actual stiffness for the concrete lower than the theoretical prediction. Finally, test PC4.30.1000 seems to be well described by the mechanical model (Fig. 21) when the moment is lower than half its ultimate value. But, as already said, a progressive buckling of the column flange in compression has been observed during the test and this failure mode is not yet included in the model. This probably explains the additional deformability reported during the test.

In conclusion, the agreement between the mechanical model and the experimentation may be considered as quite satisfactory, even good. Few discrepancies are observed. These, however, can be justified and they do not put the general validity

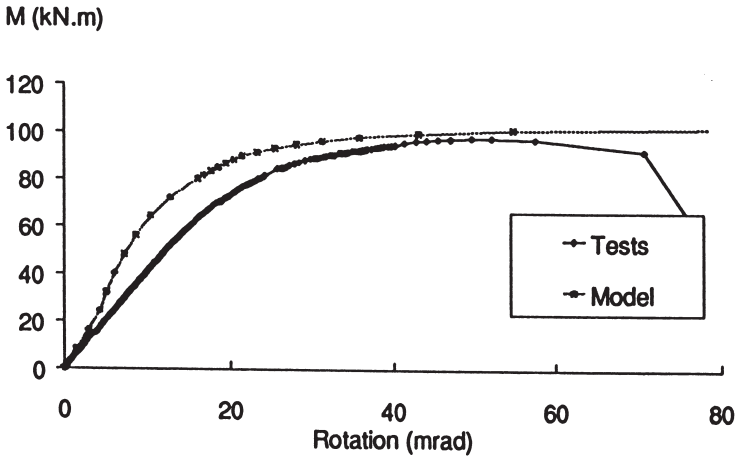

Fig. 18. Comparison between the tests and the mechanical model. Test PC4.15.1000.

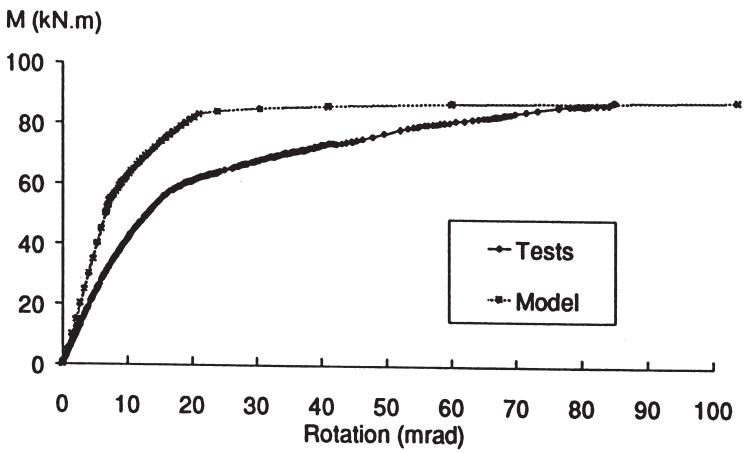

Fig. 19. Comparison between the tests and the mechanical model. Test PC4.30.100.

of the proposed rules in question again. It must not be forgotten that these rules have been rigorously derived, i.e. without a single empirical parameter, and are identical for the 12 tests being considered.

5. 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.

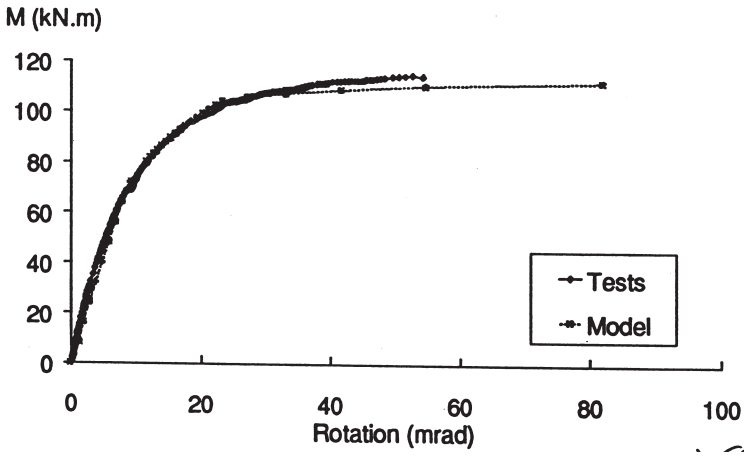

Fig. 20. Comparison between the tests and the mechanical model. Test PC4.30.400.

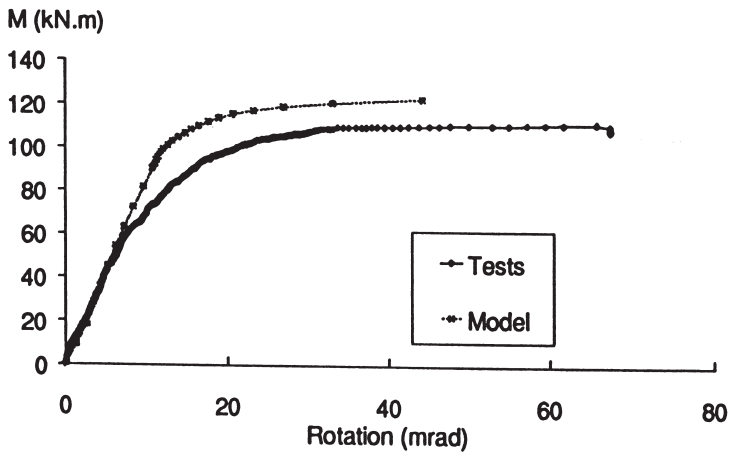

Fig. 21. Comparison between the tests and the mechanical model. Test PC4.30.1000.

On the basis of the knowledge obtained from these tests and from the available literature, a simple analytical model aimed at predicting the ultimate and design resistances has been developed and validated through comparisons with the experiments.

This model, which is described in Ref. [3], appears as an application of the principles of the component method, with references to annexes L and J of Eurocode 3 [1,2] for what regards the characterization of the individual basic components.

A complete model also requires nowadays the prediction of the rotational stiffness of the column bases.

However, the complexity of the problem is such that the development of a simple and reliable stiffness model appears still now as quite contingent. It has therefore

been decided to focus on the development of a scientific tool, i.e. a mechanical model, allowing, through rather long and iterative calculation procedures, to simulate accurately the non-linear response of the column bases from the first loading steps to failure.

Such a tool, the validity of which is demonstrated through comparisons with experiments, allows to deeply understand the behaviour of the column bases and their constitutive components, the inherent interactions and the way failure occurs.

Through the use of this tool in the frame of parametric studies, the development of a reliable stiffness model may be contemplated in the future, and more especially in the frame of the ongoing CRIF project on column bases where complementary experiments on column bases with two anchor bolts subjected to uniaxial major or minor axis bending have been recently performed.

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