COLUMN BASE CONNECTIONS

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ABSTRACT: The paper presents comprehensive information about numerical modelling of column bases of steel structures. It deals with particular aspects of the modelling and describes their importance and influence on the overall behaviour of the model. It also introduces an effective 2-D numerical model and in addition, two experiments of two basic components of the column base connection which may be used as calibration examples of the calculation. Examples of numerical computation are presented.

1 INTRODUCTION

The present paper aims to contribute to the research efforts concerning the numerical modelling of column base connections. In particular, it deals with certain aspects of the modelling and describes their influence on the overall behaviour of the model. As known, the stress states of a column base connection under static loading has to be computed by appropriate models that are capable to take into account two critical parameters: the development of plastification zones, local crushing and cracking of concrete block, transfer of shear force under the base plate, the different anchoring influence and the unilateral (frictional or not) contact effects on the interfaces between connection members.

Through recent years, numerical simulations of steel connections have been described by one-dimensional to three-dimensional models. The one-dimensional (Bernoulli beam) models of such connections (bolts modelled as springs), taking into account only primary bending action, are naturally the most simple. On the other hand, a three-dimensional model can incorporate all the essential features of the steel connections, leading in general to the most accurate results. This is due to the fact that three-dimensional models, having been appropriately formulated and computed contain the correct stress distribution patterns, but in a form that requires great computational effort. The first attempts for two- and three-dimensional modelling of steel connections based on several simplified assumptions, are dated back to the seventies; these efforts are till nowadays continued by deducting one by one the simplifying hypotheses of the initially proposed models, thus producing more and more interesting and realistic results [5, 9, 14, 24]. In the present paper, we first present a two-dimensional finite element plane stress model which has been constructed for the analysis of the structural behaviour of a column base plate connection [13]. The model contains all the essential features that characterise the separation problem. Material yielding, contact interface slip and interface interaction are taken into account. Secondary bending effects are not present due to the static loading. The third dimension of the connection, is also considered by assigning different thickness values to the various regions of the FEM. mesh, thus achieving the most realistic response of the two-dimensional model.

The applied numerical method, does not use any a priori assumption on the flexibility of the several parts of the steel connection, to obtain first its deformed shape where contact (i.e. compressive reaction) and separation (i.e. tensile reaction) zones have been developed and next the actual stress distribution on the connection members by taking into account friction effects on the interfaces. The proposed finite element model is constructed in a way, that the interaction at any interface of the connection can be taken into account by means of unilateral contact boundary conditions [4-8, 13, 20]. Following this method, local separation zones between the interfaces of the steel connection are computed, whereas the deformed shape of the steel connection is with accuracy evaluated. Note also that the numerical results calculated by applying the proposed two-dimensional model, qualitatively conform well to those obtained by pilot experiments [12, 18]. Within such a theoretical framework the separation, the active contact, as well as the plastification zones are with accuracy calculated, leading thus to the computation of the exact stress state conditions holding on the steel connection under investigation.

2 FE MODELLING

The proposed numerical method, seems to be a reliable tool for the numerical simulation of the structural behaviour of most types of steel connections, because from one side the response of the different parts (column, plate, bolts) of the modelled steel connections are taken into account in an interactive way, whereas on the other side, the correct thickness of the various parts of the two-dimensional model are defined by applying an efficient and easy technique [19].

The numerical treatment of such problems also permits the investigation of the appearance of prying action forces. In the case of the column-base plate connection, the prying action phenomenon is directly connected with the flexibility of the connections. Exactly opposite is the reaction in the case of steel column-base plates, with underlying concrete foundation. The difference is due to the fact, that the one part of the connection (concrete foundation) is not deformable and this leads this way the deformable base plate to be locally separated from the concrete surface. As is obvious, the thickness of the base plate, is one of the most significant parameters that affects the response of such steel connections. Therefore, considering the thickness as a critical parameter for the analysis of the connection interface, under various axial external forces and moment rotation, such a sensitivity analysis should be considered as a contribution to the research of steel column base plate connections. The sensitivity study of a numerical application takes into account the base plate thickness as the critical parameter of the analysis. The obtained results are of great interest showing the different response of each connection of different base plate thickness under different loading conditions.

Applying an appropriate finite element model, the stress flow between the various components of a column base connection has to be followed up, whereas the main deformation and stress distribution patterns must be present and directly recognisable and interpretable. An appropriately defined 2-D plane stress model encompasses all the thickness effects, primary bending/membrane effects and the contact stress distribution on the connection interfaces between the parts of the connection. Stress concentration zones can be easily identified although the analysis remains incomplete, whereas secondary bending effects are not present in the model, but they cannot drastically affect the whole picture of the stress fields due to the static external loading. Such a 2-D model takes into consideration the dimensions of the joint also along the third direction by assigning different thickness values to the various regions of the FEM mesh, thus achieving a realistic simulation of the overall response of the model. In the regions where the thickness cannot be directly prescribed, as is e.g. the neighbourhood of the bolts, the washers and the zones with holes, several different assumptions have been made and the respective results have been compared to accurate numerical models which take into account the exact thickness value of each finite element. Although the later model gives the most accurate numerical results, the task of assigning the thickness of each region is a rather time consuming work. Thus, comparison of the results of the various tries of 2-D modelling is in any case unavoidable, in order to estimate the errors introduced by rougher assignments of the thickness of each part which requires less effort. To conclude with, we note that the applied 2-D model offers the following advantages: (i) Accurate numerical results in the cases that the geometry of the connection and the loading conditions, lead to 2-D deformed configuration, (ii) minimum computational cots and evaluation save of effort and (iii) they can be applied for reliable quick benchmark tests in validating commercial 3-D finite element codes for the analysis of steel connections.

3 COST-C1 BENCHMARKS

The experiments prepared for COST C1 calibration example represent both basic components of column base joint and anchor bolt. They were chosen from a wider set of experiments carried out at Czech Technical University in Prague, see [23].

Knowledge of material properties is important for numerical modelling therefore great care was taken to material tests, for details see [23]. The concrete used for the blocks was designed as C35. Compressive strength and modulus of elasticity were measured. The compressive strength $f_{cd} = 33{,}1\ MPa$ and modulus of elasticity $E_c = 46\ 789\ MPa$. The modulus of elasticity was measured by ultrasonic method. Material properties of the steel were obtained from tensile tests. The yield stress $f_y = 334\ MPa$ and ultimate strength $f_u = 460\ MPa$ were found for T-stub in compression, $f_y = 317\ MPa$ and $f_u = 400\ MPa$ for T-stub in tension and $f_y = 365\ MPa$ and $f_u = 443\ MPa$ for the anchor bolts.

The basic component influencing the tension part of column base is the anchor bolt. For COST C1 numerical simulation was chosen the example of typical steel frame anchoring, the anchor bolt with anchor head, see Fig. 1. The deflection was measured *25 mm* from the concrete surface, see Fig. . Two pull-out test of single anchor bolt were performed to find out strength and stiffness of the bolt. The load-deflection curve exhibits bi-linear relationship with stiffness *334,1 kN/mm* and ultimate strength *199,2 kN*. The bolts failed by damaging of the threads. The load to T-stub was applied in equal steps. Five loading/unloading cycles were performed prior loading up to collapse. At each load level, vertical displacement of two points on centre line of the base plate were measured. The load-displacement diagram is shown on Fig. 1 . Separation of the base plate from the concrete was clearly observed during the loading cycles. Plastic hinges in the base plate developed later which led to increasing of deformation. At load *272 kN* both bolts broke.

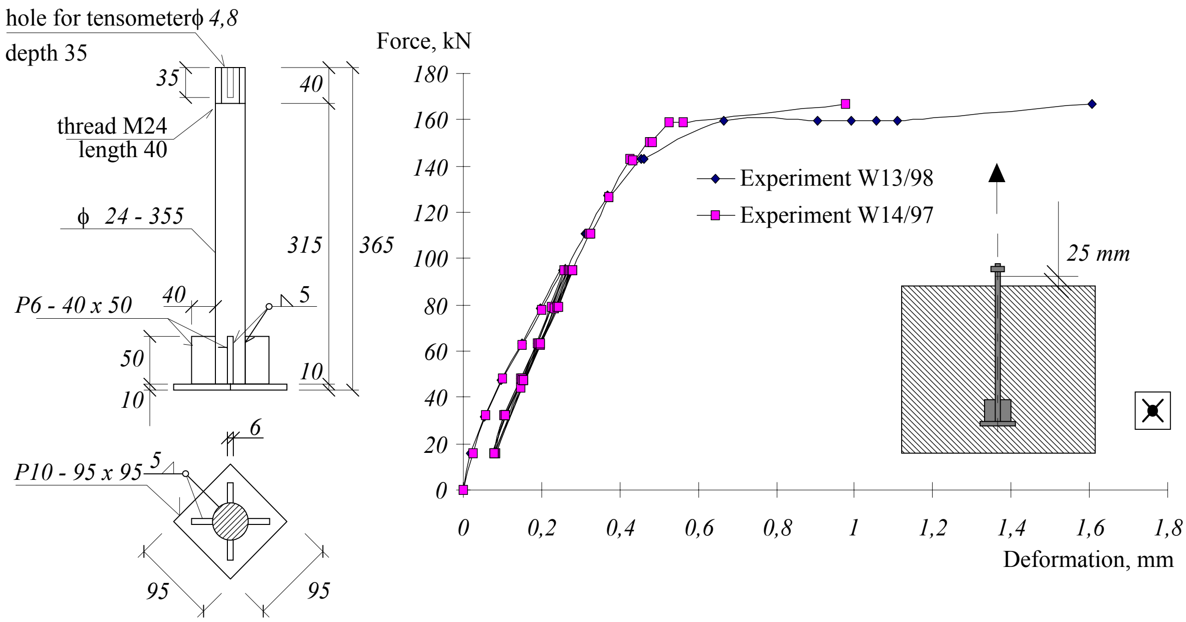


Fig. 1 The deformation of the anchor bolt, geometry of the anchor bolt [23]

This experiment for calibration example for the component in tension was chosen from twelve similar tests. It is referred as W97-03 in a comprehensive research report, see [23]. The experiment with T-stub in tension was especially focused on observing deformation of the T-stub loaded by tensile forces, evaluating the bolt force and prying effect of the anchor bolts, observing different failure modes of the T-stub (plate yielding, plate yielding and bolt failure, bolt failure), evaluating the ultimate load of the T-stub. The test specimen, see Fig. 2 , was attached to concrete block of size *550 x 550 x 550 mm*. No grout was used at the contact area but only thin layer of plaster was used to achieve smooth and level contact surface. The anchor bolts used for fastening of the T-stub are shown on Fig.. Each bolt was equipped with a tensometer placed in a hole in the bolt shank. Hand tightening of nuts and one washer was used for this test.

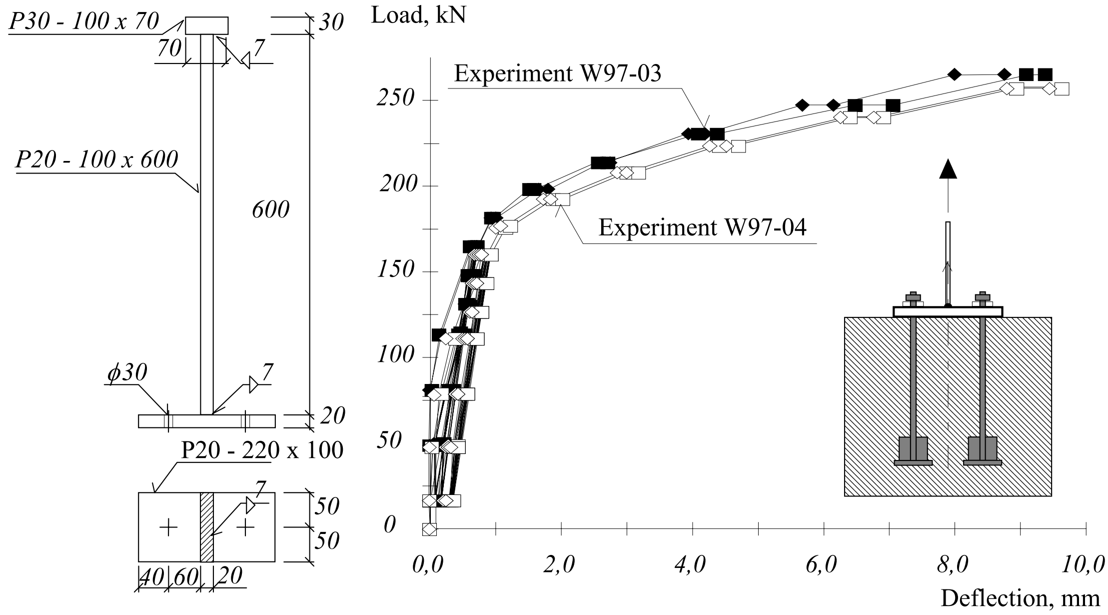


Fig. 2 Load-displacement curves of test in tension, geometry of the test [23]

The test simulates base plate loaded by the compressive force from the column flange. The test specimen consists from a steel plate and a rectangular steel bar representing the column flange. The bar is not welded to the plate but simply laid across, see Fig. . Thin layer of plaster was used instead of the grout to ensure proper contact of the plate and the concrete. Concrete block of the same size and quality as for T-stub in tension was used for this test. Measurements were taken at four corners of the plate and at both ends of the bar, see Fig. 3. The measured curves can be found on Fig. 3. No collapse of the test specimen was achieved.

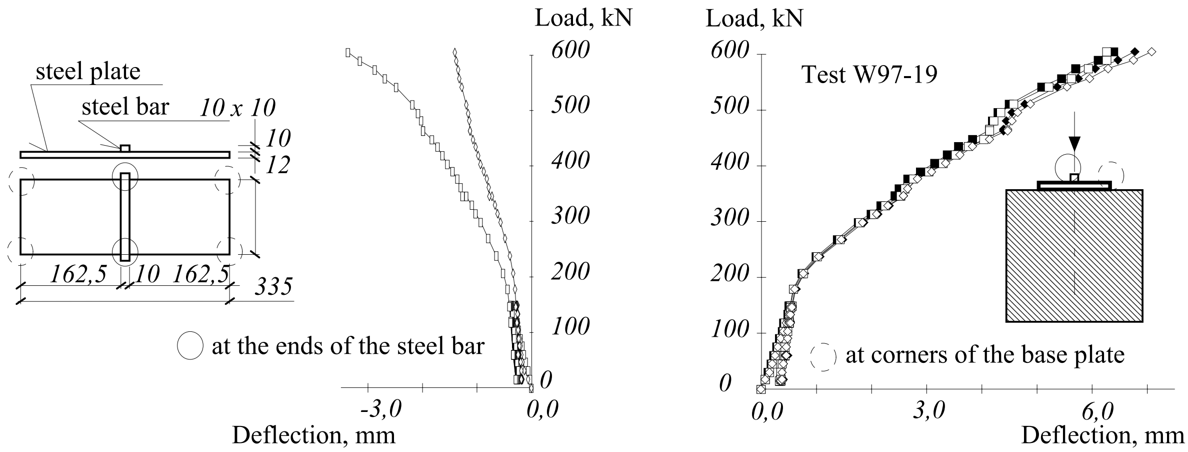


Fig. 3 Load-displacement curves of test of base plate T stub in compression, at corners of the base plate and at the ends of the steel bar [23]

4 COMPONENT SIMULATION

Modelling of the whole joint is complex due to the different deformability of the joint parts. The accuracy is mostly affected by the only one component. Therefore, modelling of the components is necessary to provide before solving of the whole joint. Separate modelling of components with different behaviour makes the work much easier and the modelling can be focused on few specific problems.

The basic components of the column base joint are the T-stub in compression and the T-stub in tension. The T-stub in compression represents compressed part of the column base joint where the load is transferred from the column flange to concrete block by bearing of the base plate. Development of contact zones is the main modelling problem of this component. The T-stub in tension represents the base plate anchored to the concrete block by anchor bolts. Modelling of anchor bolt, base plate in bending and contact of the parts of the joint are the main topics.

This following calculation shows behaviour of base plate in compression with different loading conditions. It represents a deformable base plate adjacent to a rigid plate under the column. The research was focused on investigation of the base plate deformation with variable position of the neutral axis.

The calculation was carried out using the code ANSYS 5.3. The model uses eight nodes brick elements SOLID 45 for the steel plate and SOLID 65 for the concrete block. The model enables plasticity of the steel and three dimensional behaviour of the concrete. 3D point-to-point contact elements CONTAC 52 were used at concrete-steel interface.

An infinitely long concrete block was considered for this calculation and therefore, only one layer of brick elements was used. The structure exhibits a two dimensional behaviour (in-plane strain) which requires applying of symmetric boundary conditions. With this approach, 3D non-linear behaviour of the concrete including cracking and crushing can be incorporated into two dimensional model, see Fig. 4 for finite element mesh. Note that the supports are not displayed on the picture.

Numerical simulation of the test W97-03 was carried out to obtain additional data to evaluate the equivalent height of the joint for analytical prediction in elastic stage only. Because of symmetry, only one quarter of the experiment was modelled.
Point-to-point contact elements were used to model the steel-concrete interface. No friction between the concrete block and the steel plate was assumed. The model was loaded by deformation in the central part of the plate. Deformed shape of the model is included in Fig. 4. Fig. 5 shows principal stresses in the concrete with the steel plate removed and minimal i.e. compressive stress under the plate.

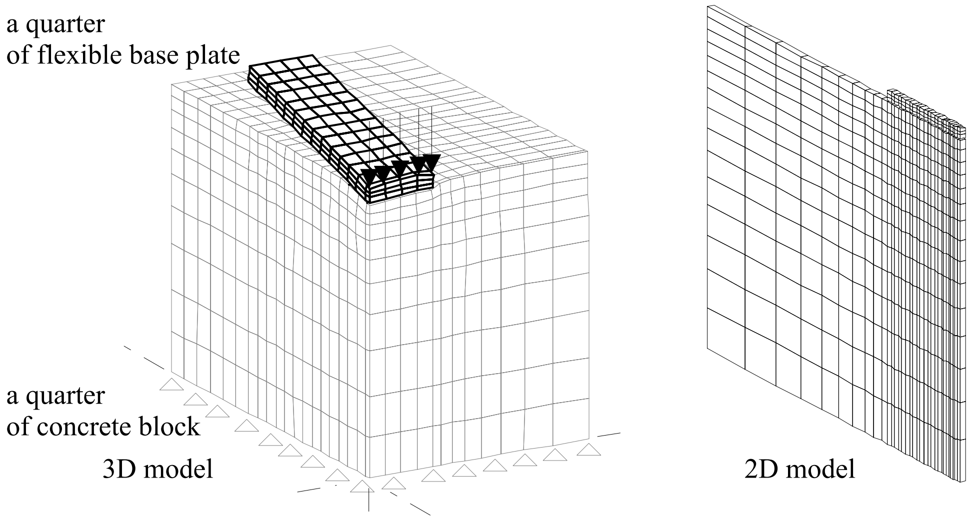
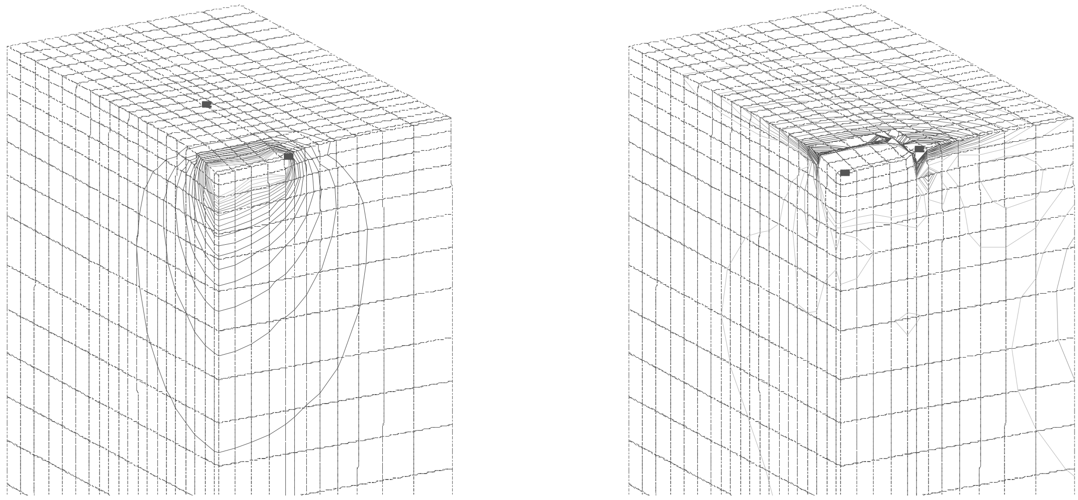


Fig. 4 The T-stub in compression loaded by deformation, the deformed mesh of the finite element model 3D model, 2D mesh for simplified solution

The other picture shows maximal i.e. tensile stress which developed mainly at the concrete surface adjacent to the contact area. The scale of the tensile stress is ten times smaller than the compressive stress.



a) *b)*

Fig. 6 Principal stresses in the concrete, a) compressive stress, b) tensile stress.

Position of the neutral axis was introduced by the loading. Prescribed displacement loading was applied on the rigid plate as shown on Fig. 6.

Five types of loading were considered: Type A, axial compression of the column, no bending moment, Type B, bending moment, the neutral axis is within the rigid part of the plate, Type C, bending moment, the neutral axis is at the edge of the compressed flange, Type D, bending moment, the neutral axis is 20 mm from the rigid part to the right, i.e. on the deformable part of the plate, Type E, bending moment, the neutral axis is 40 mm from the rigid part to the right, i.e. on the deformable part of the plate.
The work shown here represents only a part of an extensive study, Ref. [23]. The calculation was carried out with three different plate thickness *10 mm, 15 mm and 30 mm*. Variable concrete quality from C12 to C45 was assumed. Fig. 6 shows results for *15 mm* thick plate and concrete quality C30.

The analytical prediction of the anchor bolt elongation is sensitive to the prediction of the stress distribution along the anchor bolt, see Fig. 7. A special simulation of this subject shows small importance of modelling of a contact surface quality, the surface between bolt and the concrete block, under the elastic deformations and high influence under the loading close to the collapse. The model was calibrated against the tests of the embedded cast-in-situ anchor bolt with stiffened headed plate, see [13]. The main output, which can help in prediction model of the elongation - slip of the embedded part of the bolt, was the development of the stresses during the loading in elastic stage are presented in Fig. 8. The sensitivity study was steamed to the anchor bolt length and the headed plate influence on the elastic deformation at the concrete surface [28].

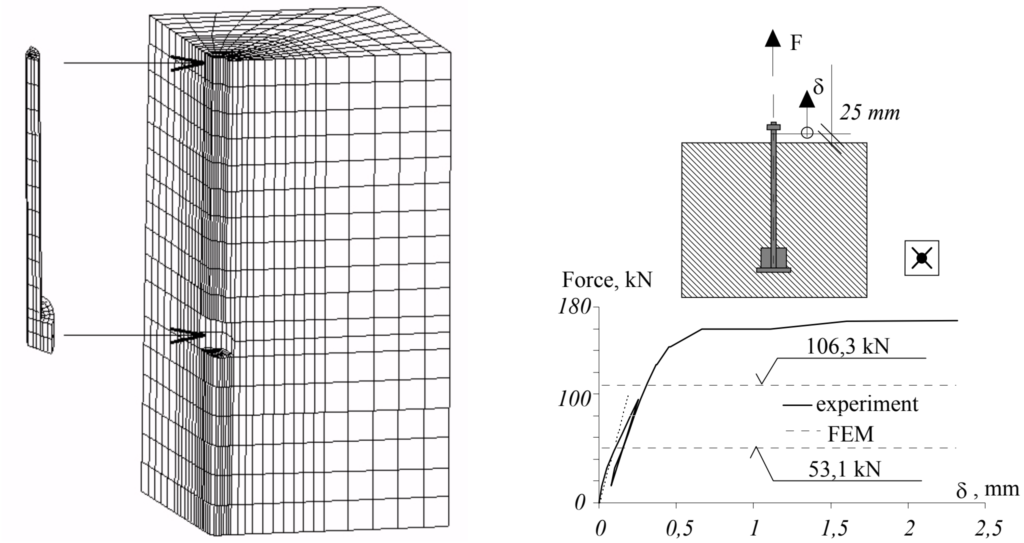


Fig. 7 The FE mesh for simulation of the anchor bolt, a quarter of the specimen, see test [13], bolt is glued by the contact elements (point to surface)

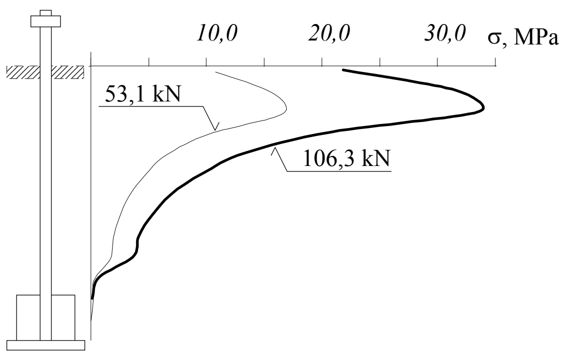


Fig. 8 The development of the contact vertical stresses during the loading

5 TWO DIMENSIONAL MODELS

5.1 Column Base with Base Plate

The described 2-D modelling technique is herein applied in order to simulate the behaviour of the base plate connection shown in Fig. 9. The steel column base connection consists of an RHS 120/200/10 steel column which is connected to a concrete block through a steel plate. The thickness of the base plate is considered as a critical parameter which varies taking the following values: *20mm, 25mm and 30mm*. Also six *M20- 5.6* bolts are used. The connection is simulated by means of a 2-D finite element mesh (Fig. 9) consisting of 4044 nodes and 2829 plain stress quadrilateral elements. The thickness of the plain stress elements are properly adjusted in order to take into account the three-dimensional properties of the structure compare [19]. Note that on the region of the holes of the plate, the plate and the bolt are overlapping. The interaction between the two bodies is taken into account by considering unilateral contact conditions between them. Unilateral contact conditions, are also assumed to hold between the plate and the concrete block. Elastoplastic stress-strain law for the steel profile and the plate is assumed to hold. A similar diagram is used to describe the material of the bolt. The material of the concrete block is considered as linear with modulus of elasticity $E_c = 29\ GPa$.

The model joint is loaded by applied displacements introduced as a sequence of 50 increments on the top of the edge of the column. At each increment, a displacement of *2 mm* is applied into the structure. Three groups of solutions are distinguished, each one corresponding to the three above mentioned plates with different thickness. In each set, the axial compressive loading of the connection consists of the following six cases: *0 kN, 100 kN, 200 kN, 300 kN, 400 kN, 500 kN*. From each group, moment-rotation curves are obtained which are compared separately, but as well as totally.

From the first set of results for base plate thickness $t = 20\ mm$, notice that very quickly a contact zone is established under the right end of the base plate. The resting part of the plate starts separating from the concrete foundation, tensioning the left bolt. The deformation of the base plate decreases for increasing axial force. The reduction of the deformation of the base plate, is followed with decreasing of its plastic strains. The opposite situation occurs for the column which develops greater plastic strains with the increasing of the axial force. It is obvious that the increasing of the axial force acts beneficially for the base plate, decreasing its plastic strains. This failure occurs, due to the fact that as the plate arises it causes tension at the left bolt, thus creating an additional critical member failure. From the deformed shapes it is observed, that the larger part of the base plate for the first 20 increments is in contact ($N = 300\ kN$)with the concrete. For the following increments, as the base plate uplifts, only the left edge node and the base plate nodes near the right bolt remain in contact with the concrete foundation. This contact area increases along with the axial loading from *0* to *500 kN*.

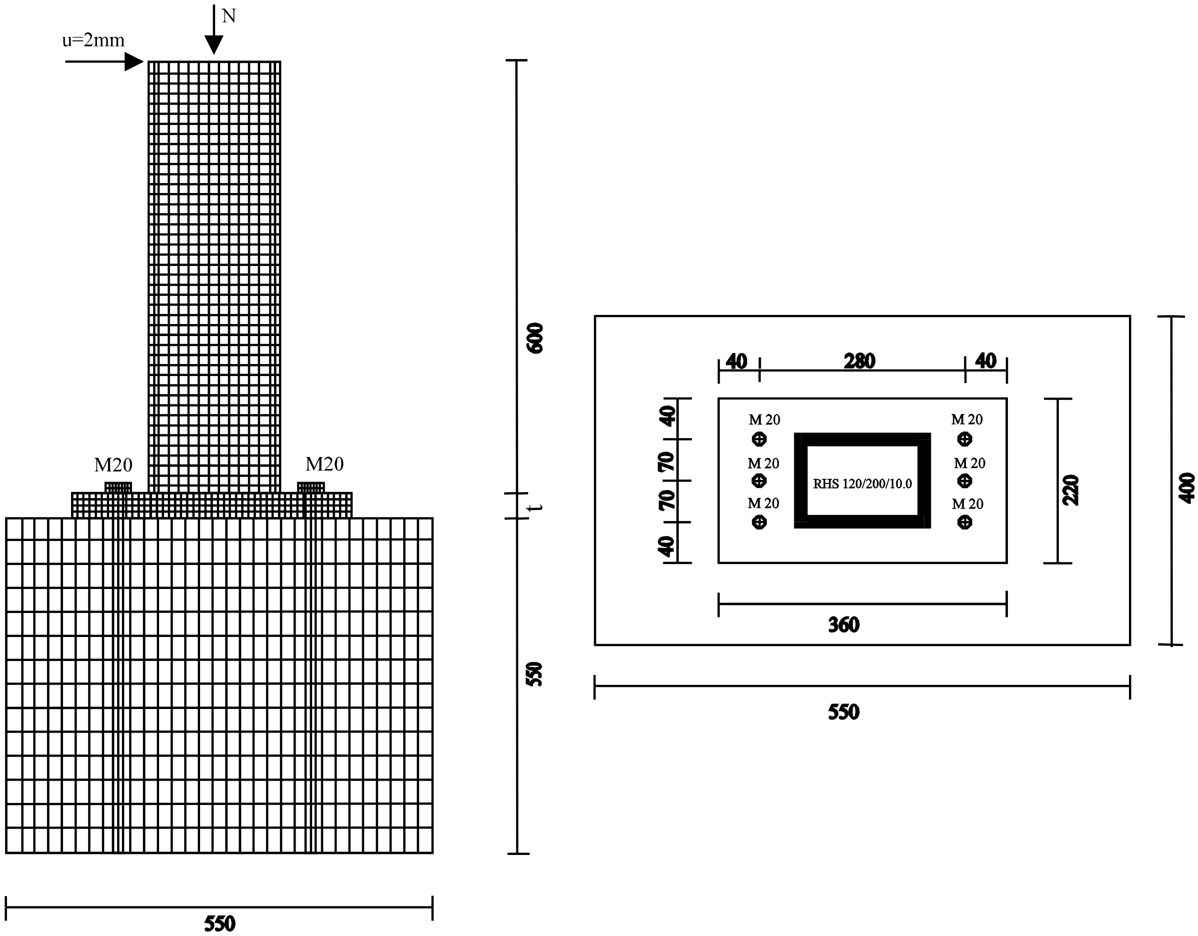


Fig. 9 Base plate connection the two dimensional example of calculation

Concerning the stress concentration on the joint, although the right region of the column and the base plate are naturally the first expected failure areas, similar stresses are developed in the left region after the 14^{th} - 18^{th} increment. This phenomenon occurs because of the prying forces which are developed in this area, when the left edge of the base plate comes in contact with the concrete base. For each case of axial load, the base plate for the first 10 - 18 increments separates from the concrete base in an area which extends, from the left edge of the plate to a certain point near the middle of the plate. For the following increments, the deformed shape of the base plate has as a result the contact of the left edge of the plate with the concrete base. This phenomenon creates through all the cases of axial load, a plastic area in the left region of the connection which develops similar final plastic strains with the right region. From the moment - rotation curves it becomes obvious that the moment capacity of the connection increases along with the increasing of the axial force. For axial force *0 kN*, the moment capacity is near *90 kNm*. In the case of *500 kN*, the moment capacity reaches 116 kNm. In Fig. 10, the differences between the six cases of axial loading are clearly observed. Concerning the results obtained for base plate with thickness $t = 25\ mm$, the stress condition, as well as the plastic strains fields do change because prying forces are not present in this case. This results obtained are logical, since the stiffness of the base plate increases for thickness $t = 25\ mm$, permitting smaller deformability and reducing its final plastic strains. This fact is also observed from the maximum node detachment of the base plate at the left edge of the base plate.

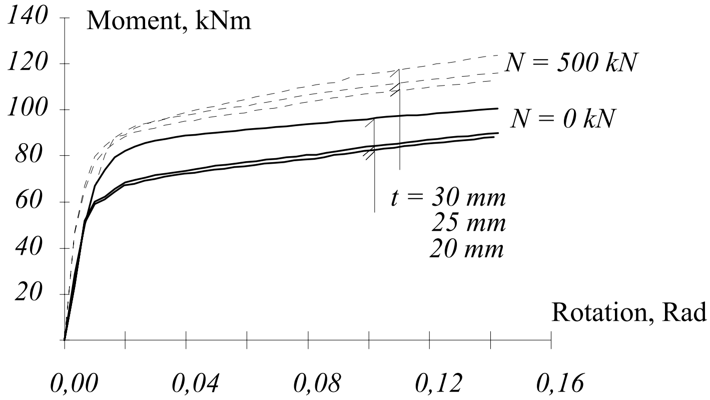


Fig. 10 Moment-rotation diagrams for various axial loads and base plate thickness (t = *20, 25 and 30 mm*)

The steel column for each case of axial force, fails around the area of its right foot near the 40th increment, with stresses that exceed its ultimate strength. The stresses appearing in the left region of the base plate are beneath its ultimate strength. This occurs because the prying forces which were developed in the previous analysis for plate thickness $t = 20$ *mm* do not appear.

Through the deformed shapes of the base plate for the six cases of axial loading at increments 10, 20, 30, 40, 50, we notice the different response the connection exhibits in comparison with the connection of plate thickness $t = 20$ *mm*. The nodes of the base plate, have detached from the concrete foundation in an area that extends from the left edge of the plate reaching near the right bolt. We also observe that the bending of the base plate, is reduced in comparison with base plate of *20 mm* thickness. This proves that the increasing of the stiffness of the base plate, affects significantly its response under the applied axial loading and bending moment combination. For axial force *0 kN*, the base plate reaches first its ultimate plastic strain. The left bolt exceeds its yield strength, as the base plate uplifts causing tension on it. The bolt develops stresses near $380\ N/mm^2$ adding a critical failure of the left bolt. For axial force *100 kN*, the ultimate plastic strains are developed at the same time at the plate and at the column. The left bolt in this case develops stresses near $362\ N/mm^2$ failing due to the tension forces that the plate transfers. Finally, for axial forces from *200 kN - 500 kN* the column begins to be plastified from the outer to the inner parts. From the moment -rotation curves we notice that the moment capacity of the connection increases along with the increasing of the axial force. In Fig. 10, the obtained moment-rotation curves for the six cases of axial load are compared each other. Increasing the base plate thickness, the stiffness that it possesses permits limited deformation.

As a result, the base plate does not fail for any case of axial load. Significant stresses are developed mainly at the area of the right foot of the column, which fails first exceeding its ultimate plastic strain. It can be noticed that the bending of the base plate is limited and is slightly visible only for the last increments. Through all the cases of loading, the column fails after the 40th increment. The developed plastic areas create a plastic hinge at the lower right part of the steel column. Thus, the collapse of the structure occurs due to the plastification of the steel column and the other parts of the connection are not critical.

5.2 Embedded Column Base

The anchoring of the structural steel frame is possible to provide by embedded of the column footing in concrete. The improvement of prediction model of this connection was based on experimental program and on FE simulation [21].

When the model was successfully calibrated a wider range of sensitivity studies were conducted. This has provided satisfactory data for the establishment of the design model. The numerical modelling and was conducted by using the finite element code SBETA [11]. The program SBETA is a special finite element package for concrete problems 2D simulation. It has strong function to model the crack progression and descending branch of the load deflection curve. This is important for obtaining information about failure mode. The concrete properties modelling is important for the program includes non-linear behaviour in the compression including softening, crack generation in the compressed concrete, bi-axial criterion for crack geneses, decrease of the compressive stress and shear rigidity after crack initiation. The two dimensional, non-linear analyses were used. The load was applied by deformation. It is element with four nodes with Gauss integration points were used for modelling concrete and steel. The elements modelling the concrete base had the material properties shown on Fig. 11.

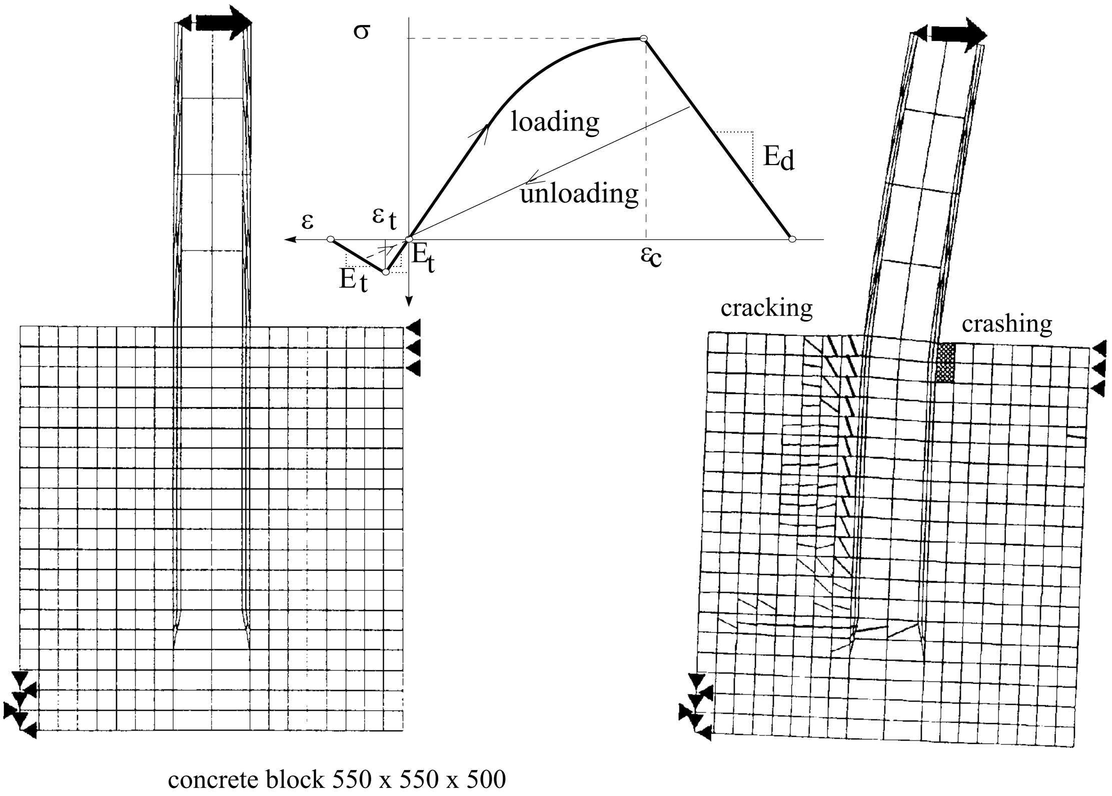


Fig. 11 The influence of the cracking of the concrete block on the embedded depth; material properties of the concrete elements, deformed shape including element cracking the last step before the collapse

In order to eliminate the error of two dimensional modelling, the finite element model was calibrated on the experimental results. The thickness of concrete element was changed in order to achieve the same resistance of the model as the experiments. The resulting thickness was *765 mm* instead of original *550 mm*.

The FE simulation was streamed to observe the participation of the second flange, behaviour of the end plate embedded into concrete and the stress distribution in the concrete alongside the column loaded by moment and horizontal force. The high influence of the boundary condition was observed. The model represents the most unfavourable case of possible conditions, see Fig. 11.

Comparing the stress distribution of with different embedded depth, the following influence of the embendment depth can be observed. In the case of the embendment height $3{,}5{*}d_c$ the stress in concrete along the lower part of the steel column vanish due to the column flexibility. The stress main direction in concrete alongside the column flanges varies from the horizontal direction depending on the position and depth of embendment, see Fig. 12. As a result, the horizontal stress is in average 60% of the maximum concrete stress resistance.

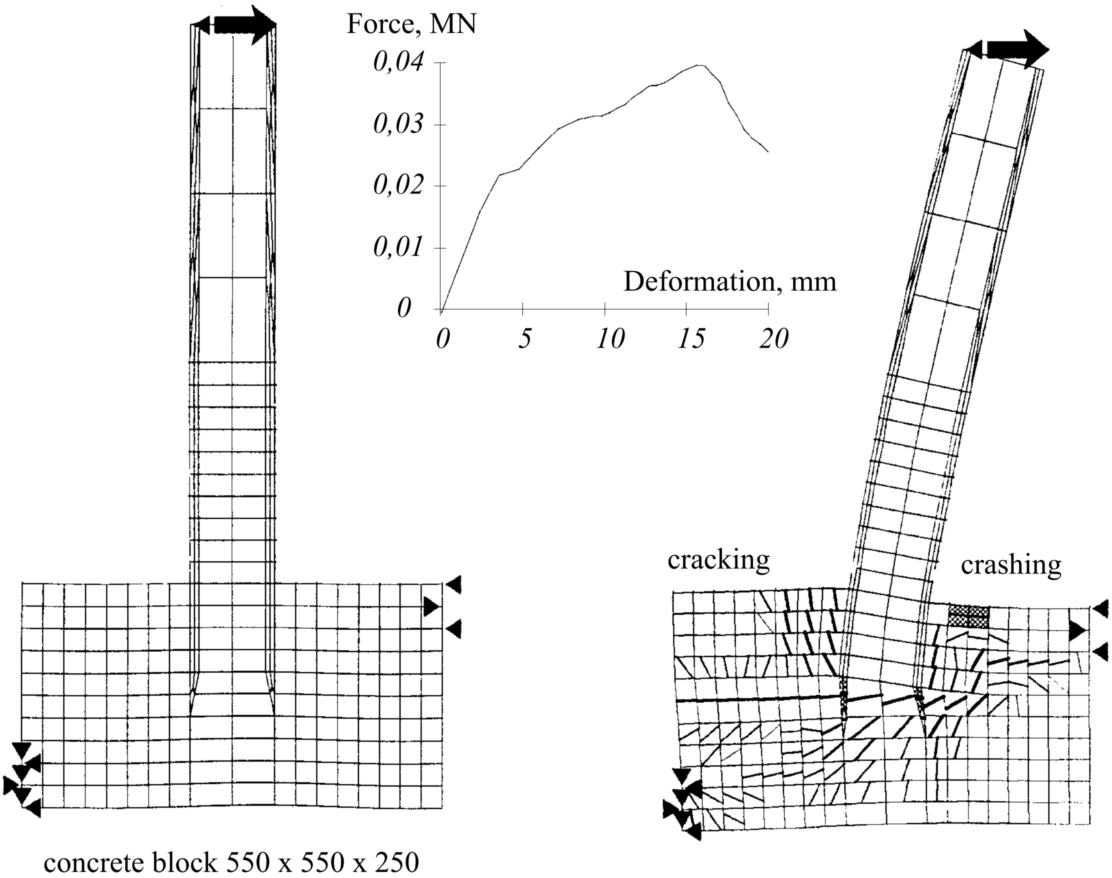


Fig. 12 The influence of the cracking of the concrete block on the embedded depth; deformed shape including element cracking the last step before the collapse, force - deformation diagram, the concrete cracking and the unloading part of the curve shows the non-linear influence of the material

6 CONCLUSIONS

- The FE simulation allows for the simulation the column base behaviour with good accuracy in initial stage of loading . The accuracy decreases in the non-linear part of the load deformation curve and the simulation of deformational capacity is a difficult task similar to experimental observations in this case of very high forces and very low deformations due to high loads.

- The present state of the hardware and software informatic support of the FE simulation enables to prepare a sensitivity study of different problem, but is limited for use in practical design to special particular problems. The solid 3D models are limited by very high number of freedom, but 2D models are applicable to study a particular special problems.

- The FE models checked to experiments are important tool for understanding the stress distribution during the loading, in positions where is impossible provide the direct measurement.

- Modelling of the whole joint is complex due to the different deformability of the joint parts. The accuracy is mostly affected by the only one component. Therefore, modelling of the components including their particular needs as contact, friction, concrete cracking and so on is necessary to provide before solving of the whole joint.

- For the evaluation of the simulation models a special tests are necessary to run with additional measurements of tests data compare to needs for check of the traditional technical models.

- The particular problems are efficient to study by two dimensional solution only based on calibration of the depth to the experiments.

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