Studying the Behaviour of Base Plates with High Degree of Rigidity

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SUMMARY:
Because of the large number of parameters involved in the behaviour of column base plates, the technical analysis of these connections has always had its special complexities. In addition to the complexities involved in the study of the interaction between steel and concrete, the existence of additional components which are normally added to the system to increase its rigidity makes the study of the system much more complicated. In this work, the behaviour of a series of commonly-used semi-rigid base plate systems with high degree of rigidity was studied through FEA methods. The interaction between concrete and steel, the distribution of contact stresses, the roles of various involving elements, and their significance in the strength as well as the rigidity of the connection were studied. This was done after making comparisons between FEA models of some specimens which were already tested by other investigators in order to increase the degree of confidence on the created FEA models.

Keywords: Base Plate, High Rigidity Connection, Interaction between Steel and Concrete, Failure Mode

1. INTRODUCTION

Base plates as one of the most important elements in structures can influence the total behaviour of structures. Behaviour of base plates as one of the connections that are used in buildings, has its own complexity. The existence of different materials such as steel and concrete, interaction between materials, existence of axial force, shear and moment are the most important problems in analysing these connections. The actual behaviour of steel base plates considerably different from assumed behaviour as rigid plates and design on the basis of rigid plate behaviour could be over-conservative (Krishnamurthy & Thambiratnam. 1989).

Much studies have been performed in this field. For example, the work done in 1992 by R. E Melchers. That study presents the results of 10 moment reversal tests on nominally ‘pinned’ steel bases for elastic stiffness of the connection. Both two-bolt and four-bolt connections were considered. It was argued that for the portal frames of interest, holding-down bolt extension, base plate deformation and prying force effects are most significant in influencing elastic stiffness (Melchers. 1992). It was showed that the stiffness of the base plate is a significant parameter, affecting the development of prying action at the active contact areas of the plate. The appearance of prying forces creates plastification zones at the interfaces of the connections, in areas that could not be considered using classical design and calculation methods (Kontoleon, Mistakidis, Baniotopoulos & Panagiotopoulos. 1998). In the past, due to the absence of computer facilities and numerical methods for analysing, the best way of studying the behaviour of connections was testing experimental models. Although the results of such experiments are closer to reality, their cost is too high. Also in order to understand the behaviour of connections with different geometries and dimensions, these experiments should be repeated several times. Therefore, understanding the behaviour of different types of base plates, failure modes and their advantages and disadvantages by using computer simulations can result in great savings. Some studies have done to show relation between experimental and analytical model of base plates. Most of these investigation were on simple base plates without stiffeners and the verification among results seemed to be satisfactory (Stamatopulos & Ermopoulos. 2011).

In this study we try to describe the behaviour of some base plates known as rigid or semi rigid connections under applied moment through finite element method. Materials were modelled based on available information and some are based on author's judgement. In order to validate the results of simulation, a model was made based on experimental studied made by R. M. Melchers (1992) under applied moment. Results of modelling and experimental results are compared with each other. Having reached an acceptable correlation between the result of the numerical and the experimental counterparts, the authors were ready to conduct similar studies on other type of base plates with enough confidence.

2. NUMERICALLY STUDIED BASE PLATES

In this study we focus on five types of base plates. These base plates are known to have rigid and semi-rigid. Some of these connections are common in use and some are with a little changes. Columns, base plates and anchor bolts in all the simulated situation are the same with added different elements. In all models I/wide flange were used as column sections. Columns were placed in the centre of the base plates. However, in these connections welds were not simulated and elements were tied together. All models were applied under direct moment. Base plates are connected to a concrete block by four anchor bolts.

2.1. Geometry of the Models

Section of columns, dimensions of base plates and diameter of anchor bolts are the same in all models. Column sections in all cases are HE-B 200, based on EuroNorm 53-62 (DIN 1025). This is a wide flange I section with moderate unit weight. Base plates dimension are 350×350×15 mm^3 and diameter of anchor bolts are 22 mm. Dimensions of concrete blocks are 700×700×400 mm^3. Bolts spacing's are 70 mm along column flange. Details and schematic drawings of all samples are shown in Fig. 1-5. Design of the first one was based on suggestion of AISC (DeWolf & Bicker. 1990).

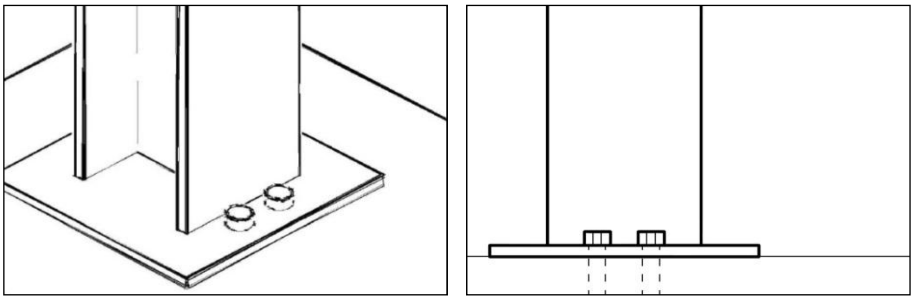

Figure 1. Schematic drawings and details of model 1.

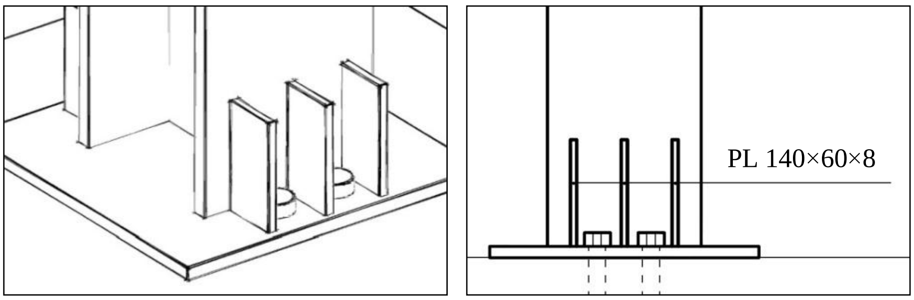

Figure 2. Schematic drawings and details of model 2.

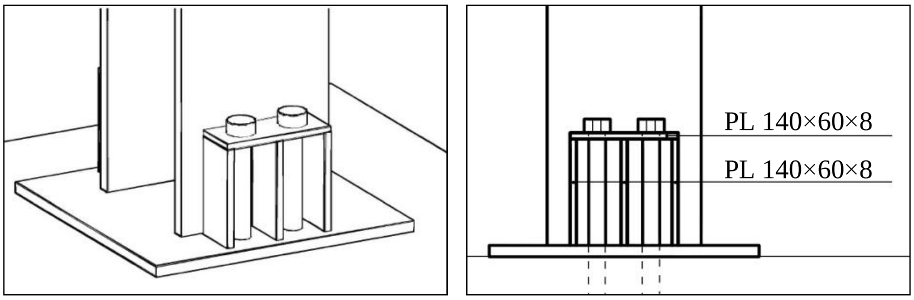

Figure 3. Schematic drawings and details of model 3.

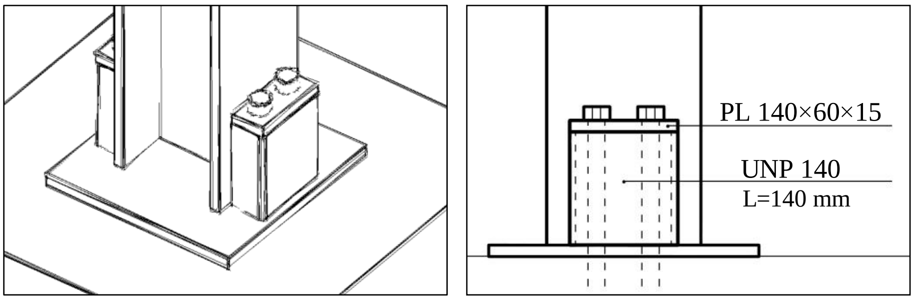

Figure 4. Schematic drawings and details of model 4.

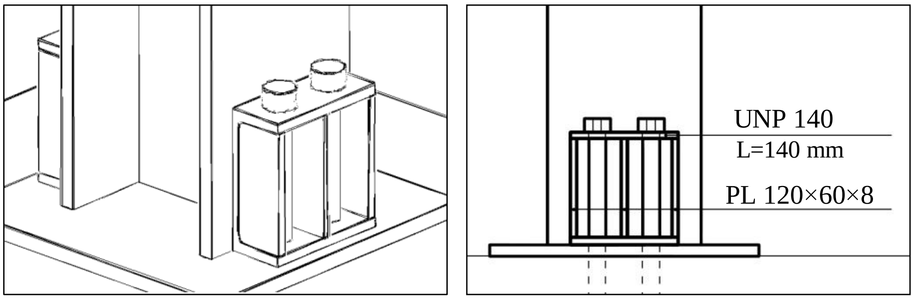

Figure 5. Schematic drawings and details of model 5.

The first type is the simplest type in which the column is directly connected to the base plate (Fig. 1). In order to increase the rigidity of the connection, three plates were added to each side of the second model. These plates are 140×60×8 mm^3 in size. The third model is almost like the second one with the difference that a plate has been used on the top of the three stiffening plates and nuts were placed on top of this plate. In the fourth and the fifth models, channel sections, UNP140, based on EN 53-62, were added to increase rigidity of the base plates. These channels were 140 mm long. In the fourth model a plate was placed on the top of each channel and in the fifth, three plates were used as stiffeners inside the channels. In the fifth model, the two channels with additional stiffeners are welded to the column flanges for the required moment from the hold-down bolts (Blodget. 1966). Dimensions of these plates are shown in Fig. 1-5.

2.2. Material Properties

The material of the columns, base plates and their attachments was assumed to be St37 hence having a yield point of $f_y=240\ \frac{N}{mm^2}$ and that of the anchor bolts to be AIII with the yield of stress of $f_y=400\ \frac{N}{mm^2}$. The nominal compressive strength of the concert was assumed as $f_c'=24\ \frac{N}{mm^2}$.

2.3. Finite Element Model

Since the connections are symmetrical, half of them were included in models, imposing the proper boundary conditions to the cut sections to account for the symmetry of the system. The bottom of concrete blocks were completely fixed. Elastic and plastic properties of concrete and steel in tension and compression were included in the models. The interaction between concrete and anchor bolts was modelled by '*friction*', based on studies by others (Tastani & Pantazopoulou. 2002). Eight-node elements (hexahedral elements) were used to mesh the models.

3. MODELLING VERIFICATION

Although computer models are carefully prepared, to ensure results, it should compare with some experimental tests. For this purpose one specimen (L8) that was built in 1992 by R. E. Melchers was modelled. They tested 10 specimens without axial force and used 200 UB 25 section as column. A simple test rig was used for experiments. A reinforced concrete block representing the footing and containing four (or two) anchorage bolts were constructed for each test and clamped to the laboratory strong floor. The base plates were welded to the 100 mm long 200 UB 25 column stubs with 6 mm fillet welds all round. This assembly was lowered over the bolts and seated on four 20 mm cubical steel packers. The column stub was provided with a loading lug usually placed 880mm above the top of the base plate and through which a horizontal load was applied using a hydrodynamic jack. The applied load was measured using a calibrated load cell. The rotation of the concrete base block was measured at a point 150 mm below the top of the concrete and that of the column 200 mm above the base plate (Melchers. 1992). The specimen that we modelled (L8) was connected to concrete block by four anchor bolts. Diameter of anchor bolts was 12 mm and dimension of the base plate was $300 \times 200 \times 10$ mm^3.

This model was simulated according to the existed information. This information included material properties, boundary conditions and loading conditions. In Fig. 6-a setup of test is shown. In order to compare the results of the test and the simulated model, moment-rotation (M-θ) curve was used. Results of both of them is shown in Fig. 6-b. As it is clear, there is a good correlation between the two. The existing correlation between these curves is an indication of the correct modelling of the system, including the material properties and the interactions between the engaging parts.

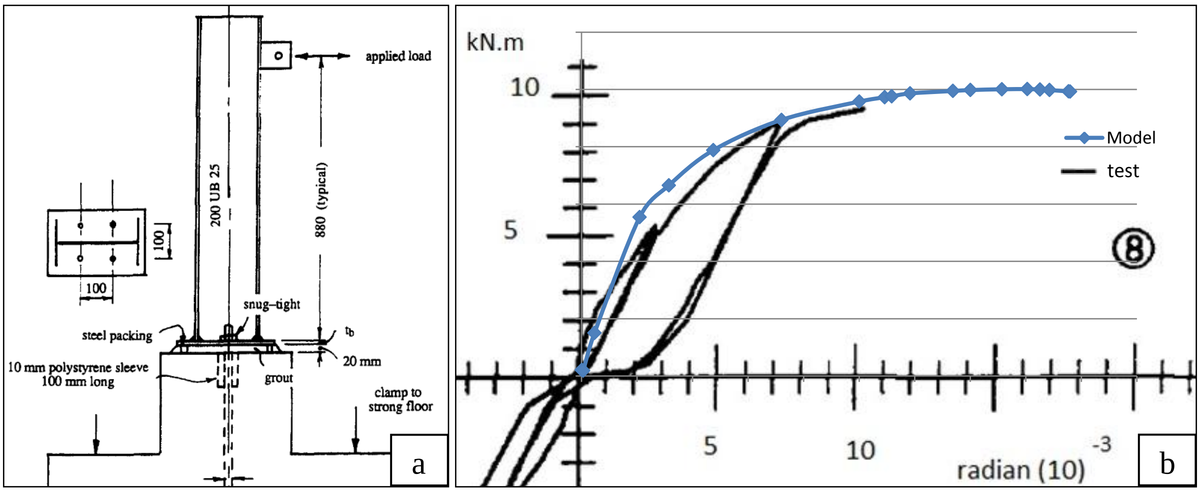

Figure 6. a) Experimental test setup, and b) M-θ curves of the tests of Melchers (1992) and the models of the authors.

4. RESULTS

4.1. Failure Modes of Studied Models

In this section the failure modes and the elements that are most vulnerable in the studied models are shown. Overall failure usually starts with failure in an element. Deformations of the models at their last studied stages are shown in Fig. 7. In the first model, excessive bending of the base plate was the cause of overall failure. In this model, critical zone was located just next to the flange of column (Fig. 7-a). It is concluded that base plate is the most vulnerable element in the design of this type of connection. In the second model, the large deformation of holding-down bolts was the cause of overall failure (Fig. 7-b). In the third model, the plate that was placed on the top of stiffeners, deformed quickly and the connection was unable to withstand more moment (Fig. 7-c). It seems that this connection, with the current design, cannot be considered as a rigid connection. Failure of the fourth model was started with large deformation of the web of the channel (Fig. 7-d). In the fifth model, flange of the channel was the most vulnerable element (Fig. 7-e).

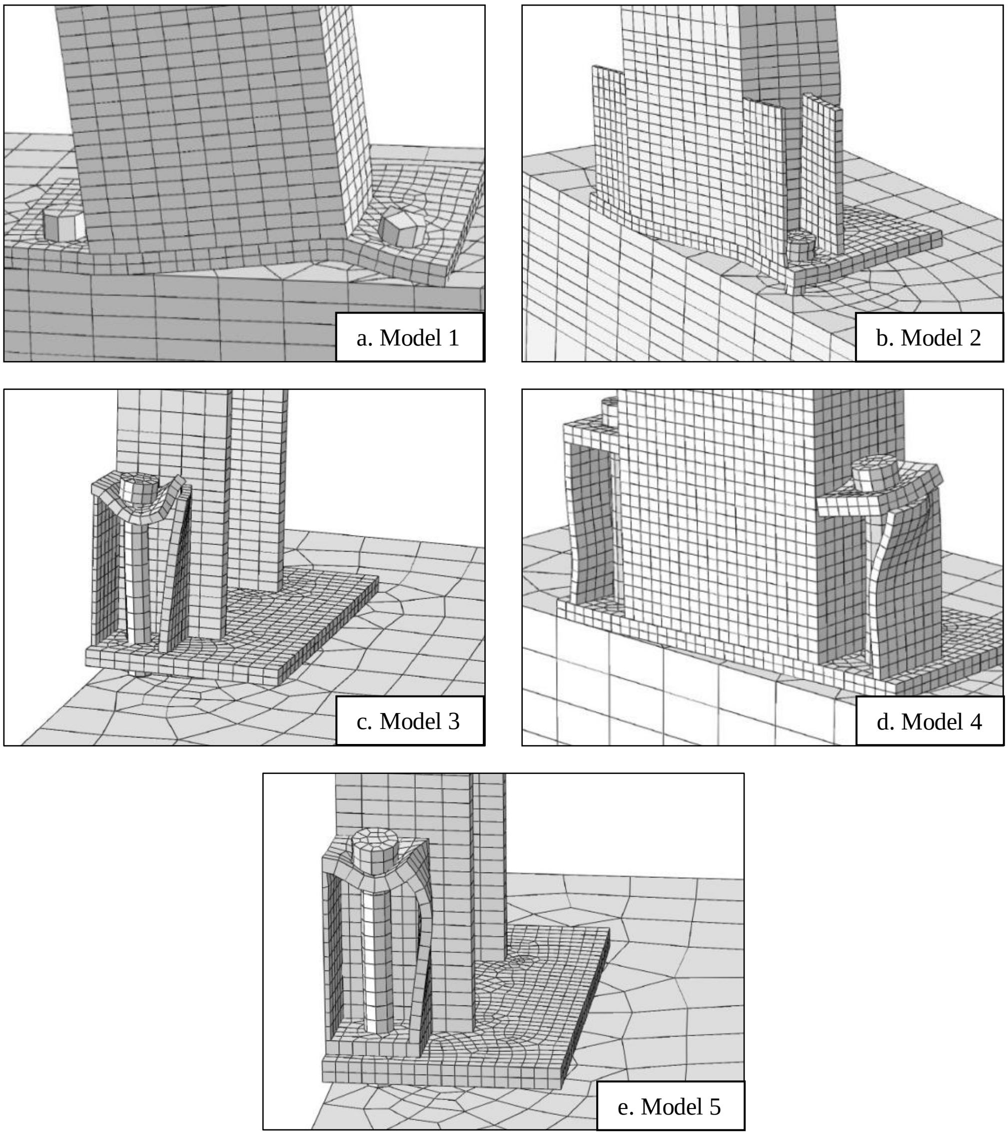

Figure 7. Failure modes of studied models.

4.2. Concrete Stress Distribution

Stress distributions in concrete is shown in Fig. 8. As can be seen, the maximum stress zone is different in each model. In the first model, the maximum stress occurred just below the flange of the column. In the third and the fourth models, it occurred at the tip of their base plates. In the second model that has the largest inelastic stiffness, and in the fifth model, maximum stress was located between the bottom of the flange of the column and the end of the base plate.

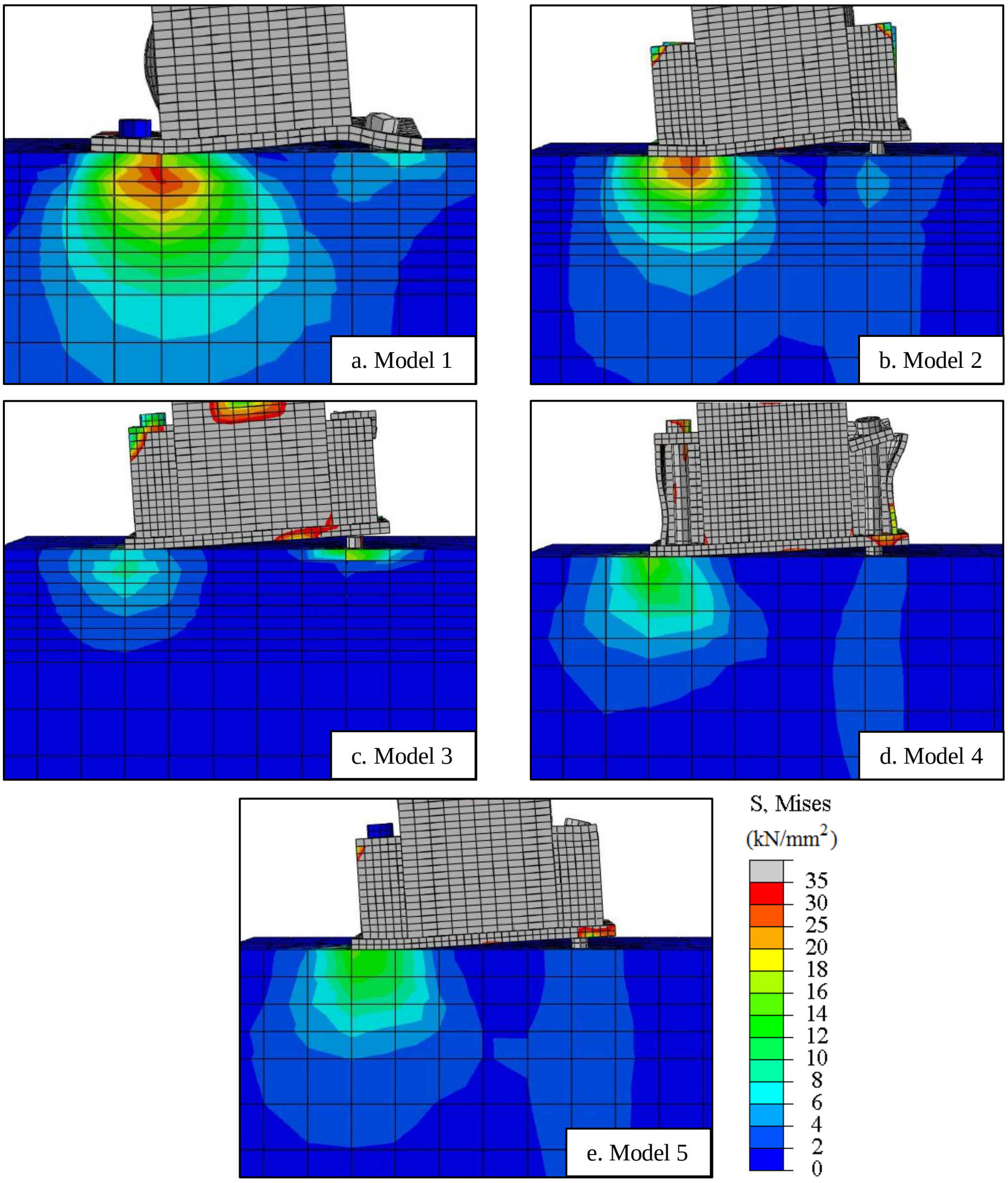

Figure 8. Von Mises stress distribution of models of various models.

4.3. *M-θ* Curves

The initial stiffness for the connections is known by the slope of their *M-θ* curves at $\theta = 0$. To measure rotations of the connections, rotation of the section that is 200 mm above the concrete block was considered. Model 2 has the largest initial elastic stiffness. As it was mentioned about in section 4.1 above, in model 3, the large deformation of top plate was the cause of its reduced initial stiffness. However, the ultimate capacity of model 5 was greater than those of others except model 2, while its initial stiffness (K_e) was less than those of models 1 & 4.

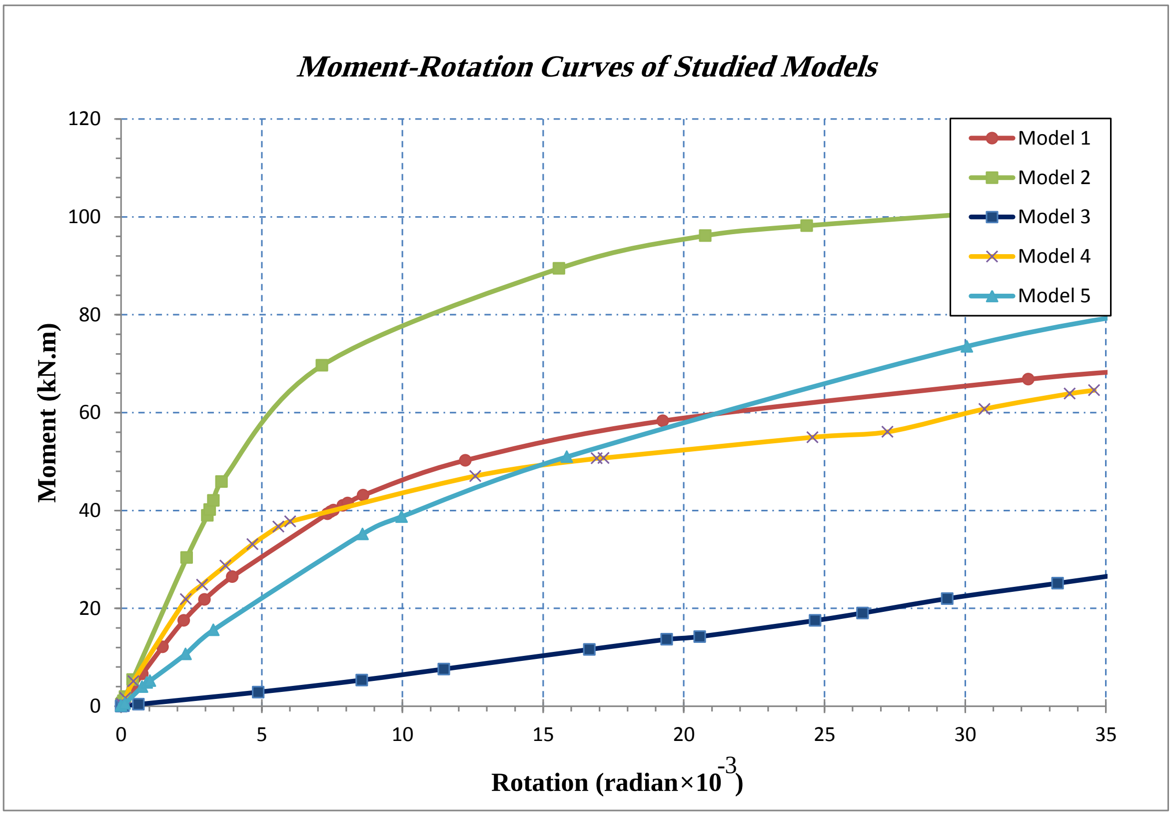

Figure 9. Moment-Rotation Curves of Studied Models.

Table 1. Initial Stiffness and moment at $\theta = 0.03^{rad}$ for various Studied Models.

Model	Initial Stiffness (K_e) (kN.m/radian)	Moment at $\theta = 0.03^{rad}$ (kN.m)
Model 1	7,369	65.33
Model 2	12,789	101.38
Model 3	588	22.49
Model 4	8,735	59.80
Model 5	4,675	73.47

5. CONCLUSION

Based on the numerical studies carried out in this work, the following conclusions can be made. However, despite the verification made against an experimental work, still some experimental studies are needed to support the results reported here. Nevertheless, within the context of the results obtained in this work the following conclusions can be drawn.

1- In more rigid systems (models 2 & 5), the role of anchor bolts is crucial.
2- In circumstances where the anchor bolts are connected to the system through an added top plate (models 3 & 4), such plates are the most vulnerable elements in the system. Besides, such configuration has an adverse effect on the rigidity as well as the strength of the system. Apparently, increasing the thickness of such plates increases the rigidity and the strength.
3- With moderate sized added top plates, systems designed based on model 3 cannot be considered as rigid.
4- Amongst all studied systems, model 2 has the highest, and model 3 has the lowest rigidity.
5- Models 4 & 5 result in better stress distribution in the concrete hence less destructing effects in it.

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