TENSILE BEHAVIOUR OF TAPERED EMBEDDED COLUMN BASES

M. Heristchian*, M. Motamedi, P. Pourakbar and A. Fadavi
1Islamic Azad University, South Tehran Branch (IAU STB), Iran

Received: 5 July 2013; Accepted: 10 February 2014

ABSTRACT

The customary exposed column bases of steel construction use anchor bolts. The anchor bolts may be subjected to various combinations of forces. The tensile (or pull-out) actions are one of these forces. The embedded steel sections can replace the anchor bolts in resisting pull-out forces. Depending on the shape of an embedded section, it can resist against pull-out forces by three mechanisms namely: the bond resistance, the interlocking force and frictional resistance. The embedded tapered section develops the resistance against the pull-out forces by the frictional resistance. The present paper, numerically and experimentally, studies the pull-out behaviour of tapered steel sections embedded in unreinforced concrete. The numerical models are generated with Abaqus 6.10-1. To support the numerical results, four tapered box and I-sections are tested under pull-out forces. The numerical models, study the effects of boundary conditions, the size of the concrete block, the tapering angle, and the coefficient of friction. The restraining boundary conditions prevent the splitting of the concrete block, which is the most common type of failure in embedded tapered sections, and could double its pull-out strength. Under proper confinement, the embedded tapered sections could have very large post-failure pull-out strength.

Keywords: Embedded column base; embedded tapered section; numerical modelling; pull-out test.

1. INTRODUCTION

Anchor bolts are, normally, used to connect steel members to concrete parts. The conventional steel column bases are such connections. In these connections, the anchor bolts resist the tensile and /or shear forces produced in the interface of the steel with concrete. Anchor bolts, however, cannot offer good resistance against shear forces.

*E-mail address of the corresponding author: heris@azad.ac.ir (M. Heristchian)
Instead of the conventional practice of using anchor bolts, the steel members may be directly embedded in concrete with no anchor bolts. In these connections the embedded steel part provides the resistance against tensile and shear forces. Figure 1 shows three types of embedded column bases, where, C1 has a uniform sectional area, C2 has an end-plate, and C3 has a tapered section with the angle $\alpha$. The connection types of Figure 1, also, can be used in composite construction, with the embedded part being steel element such as a girder, a truss chord or a truss web element.

The embedding depth and other geometric particulars of the steel element are crucial factors for the load transfer and integrity of such connections. Figure 1, shows the interface forces in C1, C2, and C3 under tensile force T. Depending on the shape of an embedded section, the resistance against pull-out forces are developed by three mechanisms namely: the bond resistance, the interlocking force and frictional resistance. In the case of C1, where the surface of the steel element is parallel to the direction of the load T, the resisting force $R_s$, is developed mainly by the bond resistance. In the case of C2, with the presence of the (anchoring) end-plate, a significant resisting component $R_a$ is present. The end-plate provides a large bearing area that helps resisting the tensile force. The force $R_a$, is closely related to the compressive and punching resistance of concrete, and for certain ranges of embedding depth and area of the end-plate, it has much higher values than $R_s$. The embedded tapered section, C3, develops the resistance against the pull-out forces by the frictional resistance, thus, the value of $R_a$, depends on combined effect of the coefficient of friction of the steel-concrete interface and the tapering angle $\alpha$, and it could be considered as a combination of $R_a$ and $R_s$.

Grauvilardell et al, 2005, [1], summarise and classify the experimental and analytical works on various types of column bases. Akiyama et al, 1984, [2], carried out experimental works on embedded H-section column bases under cyclic bending moment together with axial tension load. They discuss the modes of failure of the embedded column bases and propose an empirical formula to predict their collapse load. The authors conclude that sufficient embedding depth is necessary in order to achieve a required level of rigidity and strength. With the aim of post-earthquakes retrofitting of structures in Japan, extensive experimental studies were conducted on embedded steel column bases under cyclic shear and bending forces. Nakashima et al, 1986, 1987, 1992 and 1996, [3-6], in their studies
embedded the conventional exposed column base plates (together with anchor bolts) in reinforced concrete. They found that the embedding depth and detailing of the reinforcement around the embedded column base had significant effects on its ductility. Based on post Kobe research, Hitaka et al, 2003, [7], report that larger rotational stiffness is expected for embedded column base than conventional base plate connection. Further, they observed that anchor bolts had fractured or elongated severely in Kobe earthquake, whereas no damage was reported for the embedded column connections. Kohzu et al 1991, [8], studied very shallow embedded steel column bases under vertical and horizontal loading.

For Concrete Filled Tube (CFT) column bases, Morino et al, 2003, [9], proposed and tested a semi-embedded column base, also, Park et al, 2005, [10], proposed an anchoring method for embedded CFT base. Kingsley et al, 2005, [11], carried out three tests on embedded CFT column bases under axial compression together with cyclic lateral loads. According to their findings, increasing the embedding depth allows the column to sustain large inelastic deformations and good energy dissipation.

Di Sarno et al, 2007, [12], carried out tests on partially encased composite steel-concrete columns. Pecce and Rossi, 2013, [13], have a numerical model for a type of partially encased composite columns. In their model, a nonlinear spring at the base takes into account the elastic rigidity and the plastic behaviour of the base joint.

Pertold et al, 2000, [14], carried out tests to measure the bonding and punching strength of embedded steel H-section and proposed a numerical model to simulate the punching resistance of embedded section.

![Figure 2. Methods for construction of embedded tapered column base](Archive of SID)
but in method (a), only welding is feasible. The top-plate in method (a), also plays a major role in transferring compressive and bending forces, and therefore, this method is advantageous. The method (c) embeds the bottom part of a full-length steel column in a previously prepared pocket with the aid of non-shrinkable grout (concrete), and is more appropriate for single story and/or low rise industrial buildings. The parameters ‘he’ and ‘hp’ play a major role in the tensile and compressive strength of the embedded column base. The present paper, investigates the effect of boundary conditions, block size, tapering angle and the coefficient of friction on the tensile behaviour of the embedded tapered steel sections.

2. EXPERIMENTS ON THE PULL-OUT STRENGTH OF EMBEDDED TAPERED SECTIONS

In order to support the numerical studies for investigation of the pull-out strength of the embedded tapered sections, four tests were conducted. Figure 3 shows the plan and section of the concrete block, and the embedded tapered section together with the load frame that was used to apply the pull-out load. In plan view of Figure 3, the sides normal to the Y-axis are free, but the other two sides, that is, the sides normal to the X-axis, are touching the other concrete blocks and are, therefore restrained. A plastic sheet separated the neighbouring blocks from each other. The four supports of the load frame were 1360 mm, distant from each other. The sections were built of steel S235, with a minimum nominal yield strength $f_{ys}=235$ MPa. The dimensions of the test specimens S1 to S4 are shown in Figure 3, where, b-b denotes the dimensions of the sections at the surface of the concrete. The embedding depths of all of the specimens were $he=360$ mm. The specimens S1 and S3, have I-section, and the specimens S2 and S4, have box sections. The depth of the webs of the specimens S1 and S2, were tapered 5% and those of S3 and S4 were tapered 10%, as shown in Figure 3. The specimens were embedded in unreinforced concrete blocks. Each concrete block had the plan dimensions $2000\times1800$ mm, with a height of $800$ mm and a lean concrete of 100 mm thickness. The concrete mix was designed to achieve the compressive strength of 30 MPa, and had proportions, per cubic metre:

\{sand (0-6 mm) 804 kg, gravel (6-20 mm) 952 kg, cement 416 kg, water/cement 0.42\}. 

www.SID.ir
The embedded sections of the concrete blocks were subject to a static tensile load. At the time of application of load, according to the tests, the matured concrete had characteristic cylindrical compressive strength of 34 MPa. Before the tests, underneath of the supports of the load frame, greased plates were used to facilitate its horizontal movements. During the load steps, a dial gauge recorded the vertical upward displacements of the top of the axis of the specimens. Additionally, a surveying total station, recorded the displacement of the face point at the top centre of the profile. Though the points of measurement for the dial gauge and the total station were at small distance from each other, nevertheless, the displacements measured by the two methods agreed.
Figure 4. The crack patterns in the concrete blocks

Figure 4, shows the crack patterns of the concrete blocks of the tests for the failure state of the specimens. The photos on the left, (S1, S3), belong to the I-sections and the photos on the right, (S2, S4), belong to the box sections. Additionally, (S1, S2) and (S3, S4) are tapered 5% and 10%, respectively, as shown in the drawings on the left bottom corner of the photos. The drawings show the plan of the related concrete block together with the main crack lines which have led to the splitting of the concrete block. The plans of the concrete blocks and the location of the cracks are to the scale, but the steel sections are enlarged to be visible. Also, the constrained sides of the concrete blocks (the sides adjacent to the other blocks) are shown in double lines. The splitting cracks are developed in the tapering direction, that is, they almost coincide with the direction of the webs of the sections. The dashed lines of the drawings indicate the ‘shallow’ cracks, which do not extend through the full depth of the block, whereas, the solid lines belong to the cracks extending the full depth of the block. The nature and details of the cracks will be discussed later in the paper, but it is observed that, with the tapered embedded sections the splitting failure is the most likely failure mode in the unreinforced concrete block, compared to the conical type of breakage with the embedded profile with the end-plate. Furthermore, a noticeable feature of the failure behaviour of the tapered sections is that, after the drop of the load value, the specimen still sustains a relatively large portion of the peak load, with a large displacement, before its pull-out from the concrete block.

The peak loads related to the specimens were 713, 870, 842, and 871 kN, for S1 to S4, respectively. The specimen S1, has a markedly lower peak load than specimens S2, S3 and
Nevertheless, the peak load of the specimens S2, S3 and S4, irrespective of the difference in the shape and tapering angle, are about the average of 861 kN. The load-displacement records of the tests are shown in Figure 5. It is seen that the records of S1 have signs of instability. For test S1, there is a displacement lag (10.3=11.0-0.7) mm, between the ‘first’ and the ‘last’ peak loads. In other words, there is a ‘load plateau’ for S1. A smaller load plateau could be seen for test S2, with the displacement lag (2.4=10.0-7.6) mm. However, with a higher tapering angle, the plateau has disappeared and the overall failure displacement has also decreased. The displacements of the specimens before the load drop were 11.0, 10.0, 2.0, and 2.85 mm, for the tests S1, S2, S3, and S4, respectively. The average post failure strength of the specimens were about 45% and 22% of the average peak load, for the tapering angles 5% (S1, S2), and 10% (S3, S4), respectively.

3. ANALYTICAL MODELS AND DISCUSSION

The present section deals with the numerical models. The numerical models are generated for simulation of the experimental work on the embedded tapered sections. The numerical models were generated in Abaqus 6.10-1, 2010, [15], which has the capability of modelling nonlinear behaviour of both steel and concrete materials. Due to the static nature of the experiments, the static-general step of analysis was used.

The type of modelling of concrete is ‘damage plasticity’ which can simulate monotonic loading, failure in tension or compression due to cracking and crushing of the concrete. Figure 6 shows the modelled mechanical properties for concrete. Up to the compressive strength of 0.5 $f'_c$ the behaviour of concrete is assumed to be linear, then, the concrete stress-strain diagram follows the characteristic equation proposed by Park and Paulay, 1975, [16]. The final stress value corresponding to $\varepsilon_{cu}$ is 0.1$f'_c$. Poisson’s ratio is $\nu=0.2$ for concrete. The value of the compressive strength of concrete $f'_c=34$ MPa, was obtained from the compressive tests on the samples of the concrete blocks. The concrete tensile strength is assumed to be 2.8 MPa. For the tensile post-failure behaviour of the concrete, the fracture
The mechanical properties for steel are assumed to be linear with the modulus of elasticity $E_s = 208$ GPa, and Poisson's ratio $\nu = 0.3$. The use of linear steel material is in accordance with the fact that in the test specimens the stress of the steel material was below the yield point.

For the interface area between steel and concrete, the combination of ‘hard contact’ and ‘friction’, as defined in Abaqus, are assumed. The coefficient of friction is 0.4, except for the models where the effect of variation in the value of the friction coefficient on the pull-out strength was studied.

The loading of the models were applied by imposing uniform displacements at the top end of the steel section. The loading continued up to the peak strength and was sustained beyond the failure of the specimen. The two opposite sides of the concrete blocks, adjacent to the other blocks, were constrained in X-direction of Figure 3. The bottom of block was constrained in Z-direction in compression only. Also, the position of the supports of the load frame were restrained in Z-direction.

The numerical models M1 to M4, simulate the tests S1 to S4, respectively. Figure 7 compares the load-displacement diagrams of the numerical models and the selected points from the records of the experiments. The peak load values $P_u$, for models M1 to M4, are 883, 909, 896, and 924 kN, at displacements 8.4, 7.7, 4.4, and 4.6 mm, respectively. The average of the calculated peak loads of M2, M3 and M4, have about 5% difference with the average of the peak load of the tests.
TENSILE BEHAVIOUR OF TAPERED EMBEDDED COLUMN BASES

4. PARAMETRIC MODELLING

There are many parameters that could affect the pull-out behaviour of an embedded section. The dimensions of the concrete block and its boundary conditions, the positions of the supports of the load frame, the embedding depth of the steel section and its geometry, the concrete and steel mechanical properties together with the steel-concrete interface parameters are among these factors. Therefore, in order to gain a deeper insight regarding the pull-out behaviour of the embedded sections, in addition to M1 to M4, additional models are generated in this section. The parametric studies include the effect of boundary conditions and the size of the concrete block, the tapering angle of the section, and the coefficient of friction at interface of steel with concrete.

4.1 Effect of boundary conditions
The effects of various boundary conditions of the concrete block on the pull-out behaviour of the embedded sections are studied in this part. In this regard, the following variations of the boundary conditions of the concrete block are considered for model M1:

The four sides and underneath of the block are assumed free. Under these assumptions, the peak load dropped 37% as compared with M1.

The two opposite faces, normal to Y-axis, Figure 3, were restrained in Y-direction only. The peak load dropped 32% as compared with M1.

The four sides were restrained normal to the faces. The peak load increased 21%. In particular, this case as compared with the case of all sides free, showed 92% higher strength.

The results of the analyses, provide information on the effect of boundary conditions and confinement of the concrete block on the peak loads. However, since the limited number of cases are studied, the conclusions should be regarded as reliable only within the range of similar cases.

In addition to the pull-out strength, the boundary conditions, affect the mode of failure of the concrete block. Figure 9, shows the failure mechanisms of the models under various conditions, where ‘R’ denotes a restraint normal to the respective plane. The models (a), (b), and M1 have split type of failure, but with changing of the boundary conditions, the direction of the splitting surface and the related strength are changed noticeably in model M1. Model (c), has the highest relative strength among the four models, and it has a
different type of failure mechanism, referred to as ‘biconical’ type of failure. In the biconical failure, the cracks on the faces of the flanges of the I-section, form the bases of two ‘half-conic’ shapes that have a common apex at the end of the embedded section, Figure 9. The biconical cracks cause failure and drop of strength without the cracks reaching the sides of the concrete block.

Figure 9. Failure mechanisms under different boundary conditions

4.2 Effect of concrete block dimensions
The effect of the concrete block dimensions on the pull-out behaviour of the embedded tapered I-section is investigated using fifteen analytical models. The section with the geometry of the specimen S1, with embedding depth 360 mm, and with the assumed free four sides, is used with different block sizes in the models. The positions of the supports of the load frame in all of these models are the same as in S1. The blocks have three different plan dimensions, namely, A×B=2000×1800, 3000×3000, and 4000×4000 mm. The depths of the blocks, (the parameter H, in Figure 3), vary in the range of [400, 2000] mm, with the steps of 400 mm.

Figure 10, shows the failure crack zones for each model. For block with plan dimensions 2000×1800 mm, all five models (a1) to (a5) have split type of failure. In model (a5), with H=2000 mm, though the side crack zones do not reach the bottom of the block, however, the failure mode is splitting. In the models of group (b), at higher depths (b4) and (b5), the splitting failure has changed into biconical failure. Similarly, in models (c3), (c4) and (c5), the biconical failure occurs. The biconical failure is the limiting failure mode, in other words, it corresponds to the highest pull-out strength. This fact is reflected in the graphs of Figure 11, showing the variation of the relative pull-out strength of the embedded specimens with respect to the block dimensions. It is seen that with the appearance of biconical failure, the pull-out strength has approached a limiting value. Further increase of the depth of the block, beyond the point of change of failure mode from splitting to biconical, is not economical and will not result in increase of pull-out strength. It is also, concluded that no major increase in strength is gained by changing the plan size from 3000×3000, to 4000×4000 mm, for H/he>4. In general, for a certain embedded specimen, the limiting pull-
out strength of a concrete block is a function of its dimensions $A \times B \times H$.

<table>
<thead>
<tr>
<th>Depth of the concrete block (H) mm</th>
<th>400</th>
<th>800</th>
<th>1200</th>
<th>1600</th>
<th>2000</th>
</tr>
</thead>
<tbody>
<tr>
<td>crack zone</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(a)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(b)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(c)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 10. Cracks in various concrete blocks

4.3 The effect of tapering angle ($\alpha$)
As shown in column base C3 of Figure 1, presence of a tapering angle in an embedded section, changes the proportion of the forces on the steel-concrete interfaces. Consequently, the tapering angle plays a major role in developing the pull-out strength and the mode of failure of the embedded section. To study the role of the tapering angle in the pull-out behaviour of the embedded section, eleven models are developed in this part of the paper, with the general assumptions of model M1. Referring to the results of the models of section 4.1, the maximum pull-out strength corresponds to the concrete block with four restrained...
sides. Therefore, the four sides of the concrete blocks of these models are restrained, in order to develop the maximum pull-out strength. The tapering angle of the embedded I-section varies in the range [0.25%, 15%]. The graphs of Figure 12, give the variation of the relative pull-out strength and displacement related to the peak load of the models with respect to the tapering angle. It is seen that, up to the value $\alpha=2.5\%$, the increase of the tapering angle, increases the pull-out strength noticeably, however, the tapering angles beyond $\alpha=5\%$, cause no further increase in the strength. In contrast with the pull-out strength variation, at tapering angles $\alpha<2.5\%$, the displacements related to the peak loads have much higher values. For tapering angle $\alpha>7.5\%$, the displacements almost remain the same and approach a limiting value. It should be reminded that, a low value of $(d/d_{\text{max}})$ corresponding to the tapering angle $\alpha>7.5\%$, is still quite a high value compared to the value of displacement related to peak load in other types of embedded sections such as a section with end-plate.

Figure 12. Variation of the pull-out strength and displacement with respect to the tapering angle

4.4 Effect of coefficient of friction (Cf)
The coefficient of friction plays an important role in developing of the steel-concrete interface forces, which affects the pull-out strength of the embedded section. So far in the models, the coefficient of friction $C_f=0.4$ is assumed. Evidently, the coefficient of friction is a function of various factors such as the preparation condition of the surface of the steel section, the properties of concrete and the stresses present at interface. However, the value 0.4 for the coefficient of friction matches with most of the normal conditions occurring in construction industry. The tests by Gomes et al, 2009 [17], report values 0.38, 0.45, and 0.46 for the coefficient of friction under three different force conditions. To study the effects of variation of the coefficient of friction on the pull-out strength of the embedded sections, a number of analytical models with various tapering angles and coefficient of friction are generated. The data for the models are similar to the models of section 4.3. The coefficient of friction varies in the range [0, 0.6]. The very low and the very high values of the coefficient of friction do not relate to normal construction conditions, however, such values are included only to investigate the extent of the effect of this parameter on the pull-out strength.
strength. The graphs of Figure 13, give the variation of the relative pull-out strength of the concrete blocks with respect to the coefficient of friction for different tapering angles. For normal range of coefficient of friction [0.3, 0.5], and with the tapering angle \( \alpha \geq 2.5\% \), there is no noticeable change in the pull-out strength, however, with lower \( \alpha \), the change in strength is magnified.

![Figure 13. Variation of pull-out strength with respect to coefficient of friction](image)

**5. CONCLUDING REMARKS**

The post earthquake studies show that embedded column bases can have a good capacity of energy dissipation, offer higher rotational stiffness, and sustain large inelastic deformations compared with the conventional base plates. From the constructability point of view, embedded steel sections can potentially simplify the detailing and construction of column bases. The paper proposes three methods for construction of embedded column bases.

The numerical and experimental studies of the pull-out behaviour of the tapered steel box and I-sections embedded in unreinforced concrete reveal that:

- The test specimens sustain a relatively large portion of the peak load with a large displacement, before pull-out from the concrete block. Also, between the ‘first’ and the ‘last’ peak loads, the tests show a ‘load plateau’. However, with a higher tapering angle, the plateau disappears; also the overall failure displacement is decreased.

- Splitting of the concrete block is the most likely failure mode in the embedded tapered sections as compared with the conical type of breakage that occur in the embedded sections having end-plates. According to the analytical models, the splitting mode of failure changes to the biconical type of failure under proper confining conditions. An unreinforced block with \( A=B \geq 8h_e \), and \( H \geq 3.5h_e \), with four free sides provides such a confinement. For a block with four restrained sides, the required dimensions of the block for development of the biconical failure are \( A=B \geq 5h_e \) and \( H \geq 2.0h_e \). For a reinforced block, the required size is anticipated to be smaller. The biconical failure is the limiting failure mode, in other words, it corresponds to the highest pull-out strength, for a specimen. The occurrence of the biconical
failure, indicates that further increase of the size of the block beyond the point of change of
failure mode, is not economical and will not result in increase of the pull-out strength.

Up to the tapering angle $\alpha=2.5\%$, the increase of the tapering angle, increases the pull-out
strength noticeably, however, the tapering angles beyond $\alpha=5\%$, cause no further increase in
the strength.

In contrast with the pull-out strength variation, at tapering angles $\alpha<2.5\%$, the
displacements related to the peak loads have much higher values. For tapering angle
$\alpha>7.5\%$, the displacement do not change significantly. In comparison with the embedded
sections having and end-plate, the tapered embedded sections have much higher
displacement at peak loads.

For normal range of coefficient of friction and with the tapering angle $\alpha\geq 2.5\%$, there is
no noticeable change in the pull-out strength, however, with lower tapering angles; the
change in strength is magnified.

Acknowledgements: The support of IA University South Tehran Branch, Azaran Industrial
Structures Company and Arian Sanat Company is acknowledged.

REFERENCES
1. Grauvilardell JE, Lee D, Hajjar JF, Dexter RJ. Synthesis of design, testing and analysis
research on steel column base plate connections in high-seismic zones, Structural
Engineering Report No ST-04-02, Department of Civil Engineering, University of
Minnesota, October 1, 2005.
2. Akiyama H, Kurosawa M, Wakuni N, Nishimura I. Strength and deformation of column
3. Suzuki T, Nakashima S. An experimental study on steel column bases consolidated
with reinforced concrete studs: Part 1–Tests on steel column base under bending
moment and shearing force, *Journal of Structural and Construction Engineering, AIJ*,
4. Nakashima S, Igarashi S. Behaviour of steel square tubular column bases of corner
columns embedded in concrete footings, *Proceedings of International Conference on
5. Nakashima S. Mechanical characteristics of steel column-base connections repaired by
concrete encasement, *Proceedings of the Tenth World Conference on Earthquake
6. Nakashima S. Response of Steel Column Bases Embedded Shallowly into Foundation
Beams, *Proceedings of the Eleventh World Conference on Earthquake Engineering*,
Acapulco, Mexico, 1996.
7. Hitaka T, Suitaa K, Kato M. CFT Column Base Design and Practice in Japan,
*Proceedings of the International Workshop on Steel and Concrete Composite
Construction (IWSCCC-2003)*, Report No. NCREE-03-026, National Centre for
Research in Earthquake Engineering, Taipei, Taiwan, October 8-9, 2003, pp. 35-45.
embedded type steel column-to-footing connections, *Journal of Structural and


