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Analytical Study on Cyclic Behaviour of Simple Column-Base Connections

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Abstract- Column-base connections are one of the most important elements in steel structures which connect the steel frame to the foundation. Therefore, in structures erected in seismic areas, these connections are critical to convey cyclic inertia forces to the frame. In this paper the author performed a parametric study utilizing Finite Elements Method (FEM) to investigate the cyclic behaviour of the connection. Primarily, the modeling method is verified and calibrated by the use of available experimental data. Afterwards, the parametric computer models are made. Effect of the parameter of column dimensions on overall behavior of connection is investigated for the first time. The mentioned parameter plays an important role in simple column-base connection's behaviour as it alters the moment distribution in the base plate. Additionally, two other separate parameters are considered and their individual impact on the total behavior of joint is investigated.

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Analytical Study on Cyclic Behaviour of Simple Column-Base Connections

Gholamreza Abdolahzadeh[α] & Seyed Mostafa Shabanian[σ]

Abstract- Column-base connections are one of the most important elements in steel structures which connect the steel frame to the foundation. Therefore, in structures erected in seismic areas, these connections are critical to convey cyclic inertia forces to the frame. In this paper the author performed a parametric study utilizing Finite Elements Method (FEM) to investigate the cyclic behaviour of the connection. Primarily, the modeling method is verified and calibrated by the use of available experimental data. Afterwards, the parametric computer models are made. Effect of the parameter of column dimensions on overall behavior of connection is investigated for the first time. The mentioned parameter plays an important role in simple column-base connection's behaviour as it alters the moment distribution in the base plate. Additionally, two other separate parameters are considered and their individual impact on the total behavior of joint is investigated.

Keywords: column-base connections, finite element method, cyclic behaviour, parametric study.

I. Introduction

Column-base connections provide the overall stability of steel frame and additionally convey earthquake ground motions to the structure and back to the earth. Therefore, their dissipative cyclic response is vital to be considered integrally by designers. Even though, studies relating to rotational stiffness of beam-to-column connections historically go back to early 20th century, investigations on the mechanical properties of column-base connections have been commenced relatively late. Primarily, experimental researchers like DeWolf et al, Picard et al, and Thambiratnam et al realized the importance of the issue. Among those studies the investigation carried on at UC-Berkeley might be considered as the most impressive. It has proved that the degree of rigidity is related to properties and configuration of the connection itself as well as the amount of axial load exerted on column. The cyclic ductility has also been considered as an important parameter for seismic design of column-base connections for the first time. Additionally, the simple column-base connections' behaviour has been classified into three separate categories proportional to the base plate thickness [1-8].

In recent years, some researchers have studied the behaviour of this kind of joints through experimental as well as the Finite Element Method (FEM) and have tried to develop appropriate design models for the everyday engineering practice. These investigations, however, chiefly considered the joints under monotonic loading conditions, while to the cyclic behaviour much less efforts have been dedicated. S. Khodaie, M.R. Mohamadi-shooreh and M. Mofid performed a parametric study on initial stiffness of square hollow section (SHS) column bases under monotonic loading conditions. In this research the software SUT-DAM to model this type of connection and developed an analytical approach to study the parametric behaviour of it. Jaspart and Vandegans at university of *Liège* presented a mechanical model to investigate the component method described in Annex J of Eurocode 3. They provided comparisons of the model with their experimental laboratory tests under monotonic loading conditions. In the present research, the moment-rotation responses of those tests are utilized for validating models with monotonic loading conditions [9-12].

Adany et al conducted experimental studies on the cyclic behaviour of end-plate connections. They extended their results for column-base connections as well. They neglected the effect of concrete deformation by installing the base plate on a heavy steel beam instead of concrete pedestal. Their hysteresis moment-rotation curves are utilized to validate the cyclic models of the present research with some considerations which is explained in the succeeding sections [13].

The lack of information about column-base connections might be considered as a result of their complex structures as they are made-up of different materials (Fig. 1). This may cause difficulties for experimental and numerical studies particularly. High order of inherent nonlinearities in the behaviour of these kinds of joints imposes high computational costs and extremely long run times especially in the case of cyclic analysis [14].

Due to the fact that there is limited information about seismic behaviour of column-base connections, a demand for more investigations on this issue is tangible. Additionally, the cyclic behaviour of these connections is rarely investigated by the use of FEM method.

The first aim of this study is to investigate the important characteristics of the simple H-shaped column-base connections in seismic cases. Initially, a complex numerical model with material, contact and geometric nonlinearities is made and validated with corresponding experimental data. Afterwards, the computer models are utilized in order to achieve a better

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understanding of the real seismic behaviour of mentioned connection type specifically. This aim is achieved by studying the effect of three important geometric parameters that have a great impact on the mechanical response and calculating and interpreting the amount of standardized seismic variables like rigidity, ductility, and energy dissipation capacities for each model's hysteresis response. The variable of column dimensions and its impact on overall behaviour of the joint is considered for the first time. The mentioned parameter plays an important role in connection's behaviour as it alters the moment distribution in the base plate.

II. Modelling of Connection

Simple column-base connections are widely used when the designer desires no moment to be transferred from steel structure to foundation . Even though, in accordance to former investigations of the behaviour of steel structure connections they can be called semi-rigid or semi-hinged [15] and their considerable rigidity should not be neglected by the designers. Whereas the Occupational Safety and Health Administration (OSHA, 2001) recommended a minimum of four anchor bolts for the simple column-base connections, such exposed type joints are considered to have the minimum rigidity as the most simple column-base connection types (Fig.1).

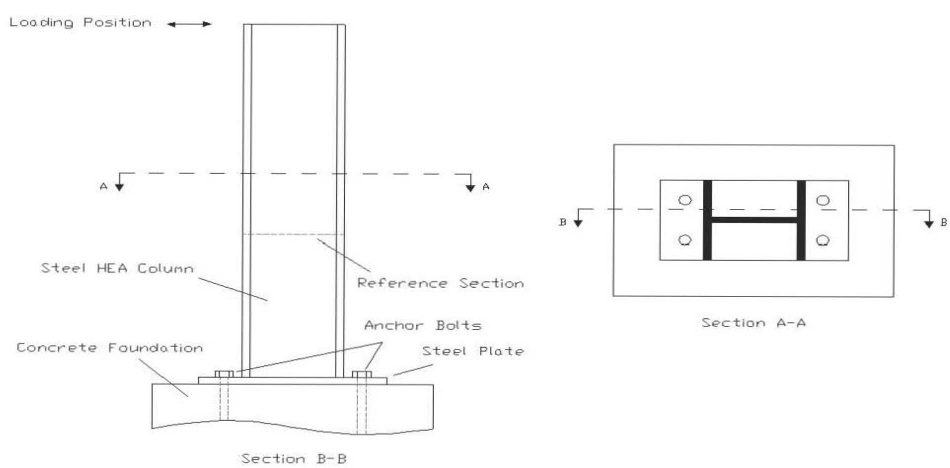

Fig. 1: investigated simple column-base connection

In this research, a FEM analysis was conducted utilizing SUT-DAM, which is nonlinear finite element software developed at the Sharif University of Technology. As there is a complex interaction between different components of connection made from distinct materials, it is important to make an appropriate model for contact areas. The convergence of FEM analysis process lies in the quality of meshing, material models, and contact formulations. Thus, a number of trial models are created and studied initially [14].

III. Generating the Model

By considering proper boundary conditions one half of the connection is simulated. The Lagrangian formulation is utilized to describe the kinematic behaviour of elements. All parts are modelled utilizing 3D eight-node brick elements which have 3 displacement degrees of freedom on each individual node. However, the element sizes are not identical in all components. The concrete pedestal is modelled with largest mesh as the base plate and the bolt heads are modelled with finest elements as shown in Fig. 2. The optimised element size is estimated by studying the changes and verifying responses of each case with the available experimental data [14].

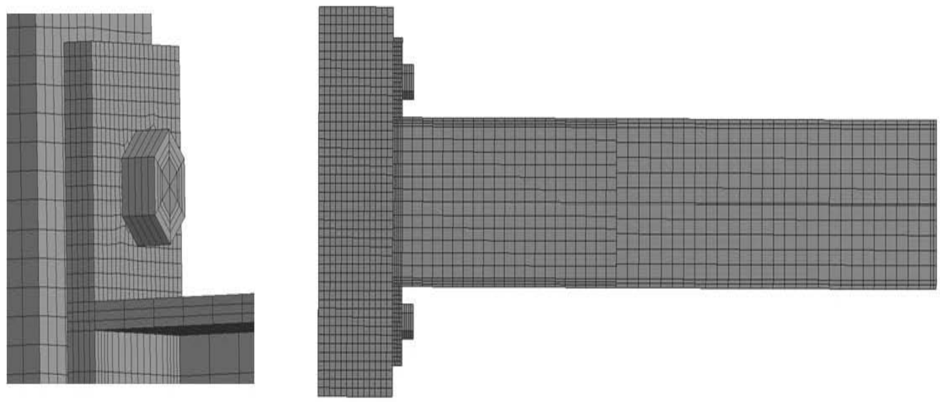

Fig. 2: Meshing

The interactions between components are modelled with 3D surface-to-surface contact elements. The nonlinear frictional contact between bolt head and upper surface of base plate as well as concrete pedestal and base plate can represent the sliding, separation and impact phenomena during the cyclic motions.

The solver utilizes an implicit approach as the load step increments are chosen adequately small and a smooth process of convergence is provided for it. A displacement-loading Newton-Raphson iteration approach is utilized in order to solve the nonlinear problem in each sub-step by reducing the errors. Additionally, a Minimum Residual Displacement Method is used for the analysis of cyclic loads. In order to curb the solution errors the Euclidean norm of displacements are considered as divergence criterion. In this case, the maximum value of vector of unbalanced displacements should be restricted. The limitation value is determined for each model by an empirical approach [14].

The material properties are basically defined in accordance to Adany's specimen which is utilized to calibrate the basic model of connection. However, in nonlinear region of stress-strain curves some adaptations are considered in order to achieve the best coincidence with the test results.

The loading is defined according to ECCS Standard which is repetitive cycles of horizontal displacements exerted on top of the column with increasing amplitudes which are defined as a function of the parameter δ_y-elastic limit of imposed horizontal displacement on the mentioned section. The parameter is calculated after studying F-δ response of an individual model for each connection under monotonic loading conditions. F stands for horizontal reaction force and δ for the imposed horizontal displacement of the mentioned section. The analytical-geometric approach utilized for calculating the amount of δ_y is mentioned in ECCS. Afterwards, the cyclic displacement loading history is applied on cyclic models as shown in the figure below.

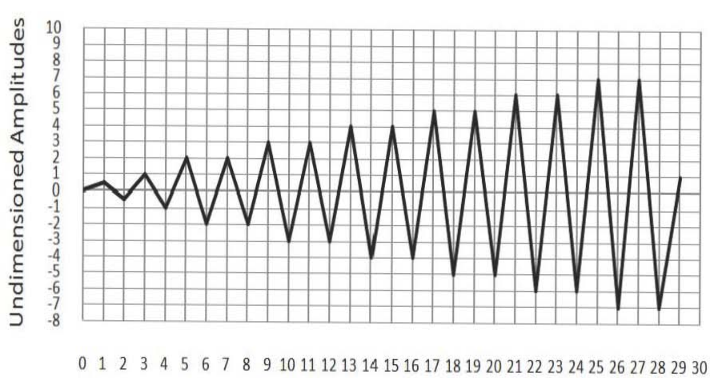

Fig. 3: Cyclic displacements as a factor of δ_y in loading cycles

It can be inferred from the previous paragraphs that the accuracy of cyclic loading in major models is related to the integrity of the corresponded model with monotonic loading condition. Therefore, a base plate model with monotonic loading condition is validated and calibrated separately utilizing available experimental data in the literature. The process is discussed subsequently.

The complete modelling of a column-base connection under cyclic loads needs high amount of computer resources. Therefore, some simplifications are considered in order to save time and increase efficiency.

After any of these changes, the FEM model is calibrated and validated utilizing corresponding test results.

IV. Simplifications of Model

Due to the following reasons complete simulation of a column-base connection under cyclic loads cannot be possible with an ordinary computer

- Large number of DOFs
- Long cyclic loading history
- The highly nonlinear nature of the problem
- Lack of comprehensive information about material characteristics

Therefore, some verified simplifications are considered in order to achieve an efficient simulation of the joint. These simplifications can be divided into two parts: simplifications in material modeling and simplifications in geometry of joints and interactions between relevant parts.

The steel material model utilized for all plates as well as the high strength steel material model for the anchor bolts are modelled based upon a method explained by Diaz et al [16]. The confirmed model is established after utilizing trial and error method and ends with the best calibrated results. According to this method the modulus for segments are defined as:

- Region (a): elasticity modulus (E).
- Region (b): $E_{h1}=E/C_{wh}$, where C_{wh} is the work hardening coefficient.
- Region (c): $E_{h2}=E_{h1}/10$

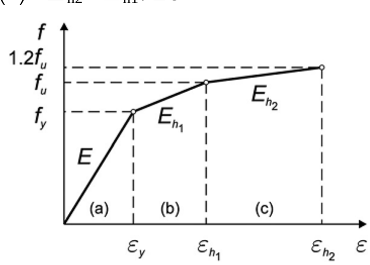

Fig. 4: Non-linear steel material models [16]

Another major type of simplifications is related to the modified geometry of anchor bolts and their interactions with the concrete pedestal. Three major parameters influence the behavior of an anchor bolt: material properties, contact forces between the bar and surrounding concrete, and the restraining of the bar. All these factors affect force-displacement curves which are the most significant characteristics of anchor bolts. In this research, in order to simplify the cyclic model the bar-concrete interaction is considered frictionless and the end of bar is assumed to be fixed. However, in a real case the interaction forces change along the bar length and this can be totally complicated in dynamic cases. This simplification contributed to better solver convergence and shorter run time for the software. On the other hand, reliable bolt material model is discovered by trying different stress-strain curves for the bolt material and studying the load-displacement response of anchor bolt specimens in comparison to the available experimental and theoretical data. These models include a discreet bar with some surrounding concrete which are fixed at the end. A normal tensile force is exerted at the top of the bar and increases monotonically. The force-deflection curve is verified utilizing available experimental data for anchor bars which is explained in the following [17].

V. Validations and Calibrations

Primarily, a discreet model of an anchor bolt with the considered simplifications is verified utilizing the experimental data available in the literature. As it can be seen in Fig. 5, agreement between the simplified numerical and experimental models is well. At the elastic parts of the force-displacement curve the slope of the FE model is less than the test results. This phenomenon is because of simplified bar-concrete contact that is considered frictionless and evidently cannot display the real characteristics of gradual separation between anchor bolts and the encircling concrete.

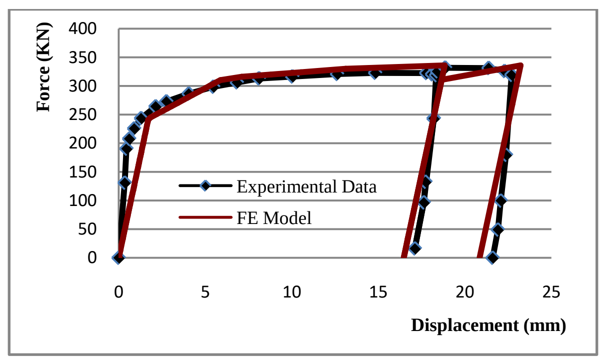

Fig. 5: Verification of discreet anchor bolt model

As explained in section 2.1, in order to define the cyclic loads, we should first calculate the elastic limit for each model from a monotonic test. In order to find the monotonic response of the connections FE models are made correspondingly. There is no available experimental data for calibrating the model of Adany's joint under such loading condition. Therefore, the modelling approach is validated and calibrated in comparison to an experimental research performed by Jaspart and Vandegans at University of Liege. As it is shown in Fig. 5 there is fine agreement between analytical and experimental curves. However, the numerical model has less initial rigidity due to the simplified frictionless interaction. At the non-linear part of curves the difference between curves is related to simplified material model in the lack of thorough information.

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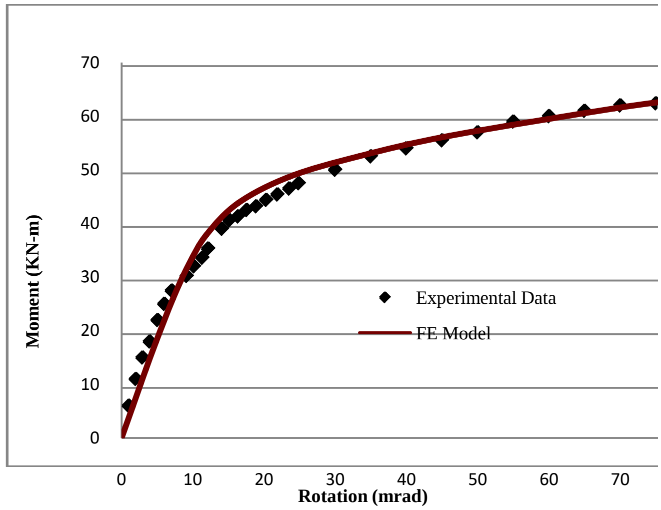


As mentioned before the test specimen utilized for validating the present numerical study was erected on a rigid base element instead of concrete pedestal. Therefore, the rotation caused by deformation of concrete pedestal under the base plate is subtracted from overall rotation of the joint in order to compare and verify the results. However, in the parametric models the concrete pedestal is considered in order to achieve more actualized response. Although such deformation may not seem considerable but it plays an important role in stress distribution under base plate which is the ruler factor in plate thickness design. [Dewolf]

The final hysteresis curve is shown in Fig. 5 that can be compared with the similar experimental curve of Adany et al. [6]. In this model the amount of elastic limit displacement is equal to 6 mm. It can be seen from the figure that at the initial parts there are reasonable agreement between numerical and experimental data. However, after 5 non-elastic cycles of loading, unloading and inverse loading the experimental curves start to fail because of fatigue phenomenon which is not considered in numerical models.

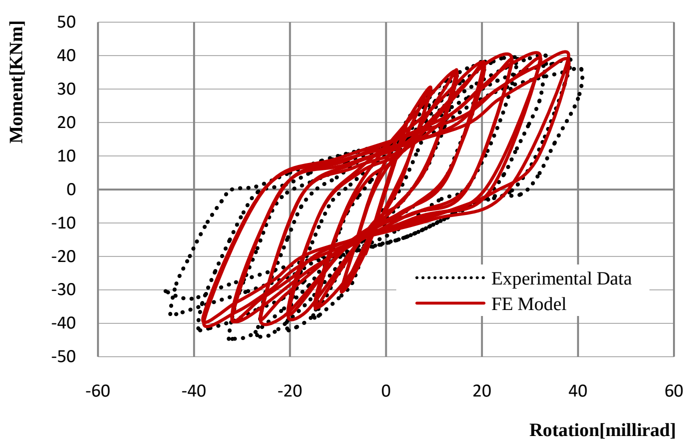


Fig. 7: Cyclic behaviour of calculated connection in the left and of the similar test of Adany et al. in the right

VI. Results

In order to study the overall behaviour of base plate connection $M\text{-}\theta$ graphs are utilized. In these graphs M is the total moment resisted by the connection under cyclic displacement exerted on column and θ is the rotation of the column at a reference section. This section is virtually located at a distance of twice the column section depth from the base plate. The definition is in accordance to Adany's ...

The cyclic parameters defined by the ECCS are considered as standard criteria for analogies between available results. Formulae and notations for the parameters are illustrated in Fig. 6. In these formulae the quantities of ductility, resistance, rigidity and absorbed energy are divided to the corresponding idealised quantities that are related to perfect elastic-plastic behaviour. As in this case all the ratios are equal to zero, their subtraction from the unity represents the deviation from the perfect elastic-plastic behaviour.

Partial Ductility	$\mu_i =$ θ_i [redacted] θ_y
Full Ductility Ratios	$\psi_i =$ $\Delta\theta_i$ [redacted] $2\theta_i - \theta_y$
Resistance Ratios	$\varepsilon_i = \frac{M_i}{M_y}$
Rigidity Ratios	$\xi_i =$ $\tan\alpha_i$ [redacted] $\tan\alpha_y$
Absorbed Energy Ratios	$\eta_i =$ A_i [redacted] $2M_y(\theta - \theta_y)$

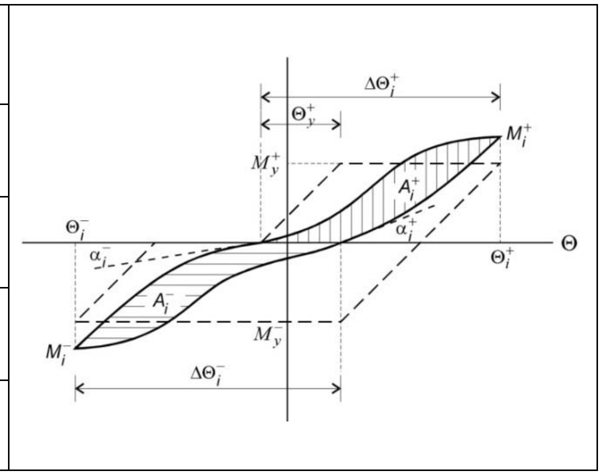


Fig. 5: Cyclic parameters defined by ECCS standard

Defined Variables

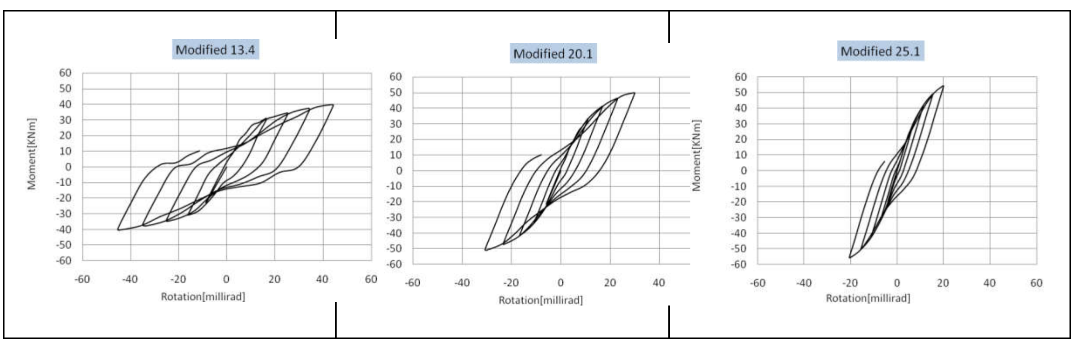
Three geometric variables are defined that each of them are varied two times. (Tab. 1)

Tab. 1: The variables and their practical ranges

Name of Parameter	Diameter of Anchor Bolts	Thickness of Base Plate	Size of Column
High	M30 (d=25.5)	22mm	HEA200
Medium	M24 (d=20.1)	16mm	HEA160
Low	M16 (d=13.4)	12mm	HEA120

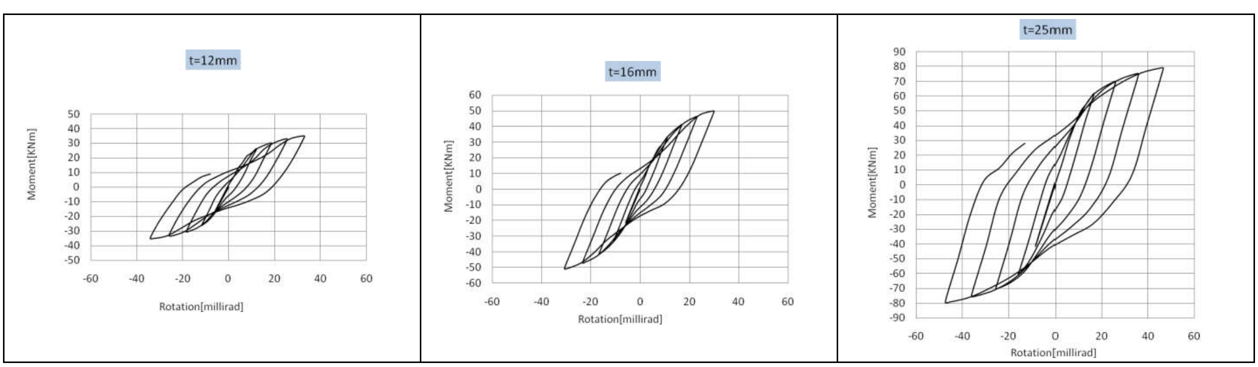
Influence of Anchor Bolt Diameter

In this section the impact of th our of connection is investigated.



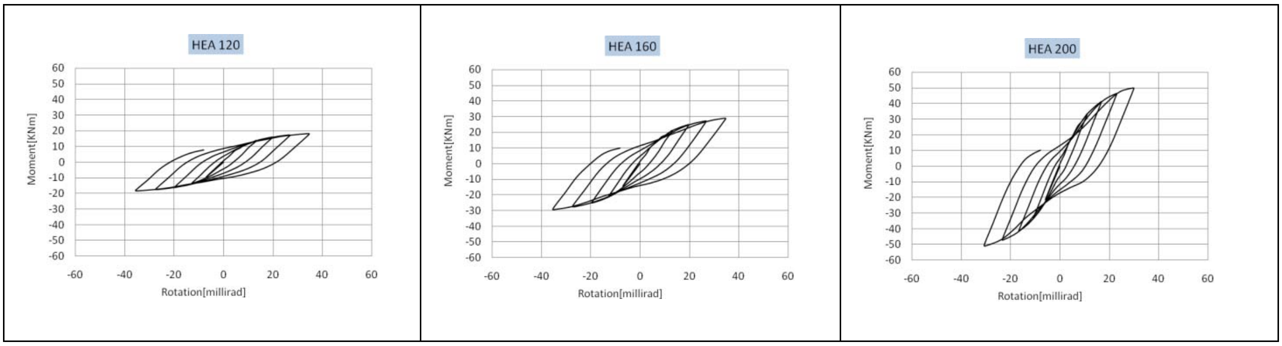
Influence of Base Plate Thickness

As it is shown in Fig. the thickness of b lic behaviour.



Influence of Column Size

In this study the change in d depth of HEA columns are investigated as an eff on the joint's cyclic response. The two altered amou'nts for mounts for column depths are both smaller than or That is because the basic model's depth is 200 millimetre which is the largest possible column for a plate with such dimensions.



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