Capacity Analysis of Exterior Beam-Coolumn Reinforced Concrete Joints Using Midas FEA Software

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Capacity Analysis of Exterior Beam-Column Reinforced Concrete Joints Using Midas FEA Software

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Abstract. Planning of building structures, especially the construction of high-rise buildings requires structural analysis that can make the behavior of building structures remain optimal. The beam-column connection consists of interior, exterior, and knee joints. The methods that are often used in construction are usually the conventional method and the precast system. In this study an analysis will be carried out regarding the behavior of the joints on the exterior of the beam-column joints and look for the capacity of the structural strength of the beam-column using conventional methods. The connection type component will be carried out based on the results of laboratory test results and modifying the connection type based on the data search obtained, then compared with the Midas FEA Software. There are 3 models of beam-column joints with types, 1 without shear reinforcement, 2 using shear reinforcement and 3 with modified reinforcement. The results of the analysis of the capacity of beam-column joints based on ASCE 41 and finite elements in the yield condition yield smaller values with a difference of 1.34% - 4%. In the inside hook modification using laboratory and finite element test methods, the difference in capacity values is greater, namely 2.91% - 8.12%. The behavior of the beam-column joints capacity when two U-bar joint shear reinforcement was added did not increase significantly by 2.77% and the behavior of the beam-column joints when four U-bar joint shear reinforcement was added showed a high increase by 95.26%.

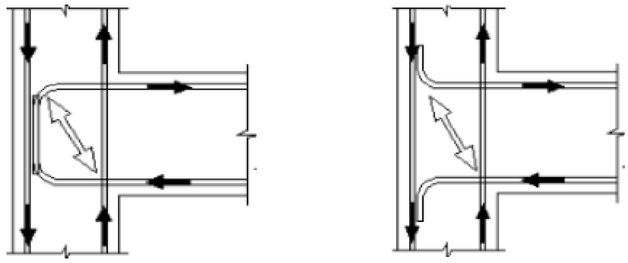
Keywords: Beam-column Joints, Exterior Joint, Finite Element Analysis.

1 Introduction

Connection is the meeting point of one component to another component. Reinforced concrete beam-column joints are located in column sections located between beam intersections. Beam-column joints are required to withstand alternating loads that develop the flexural strength of adjacent beams [2]. Structural components under different types of loading are based on the design and details of the reinforcement [9].

The function of reinforced concrete beam-column joints is to transfer loads and moments at the ends of the beams to the columns and provide stability to the structural system. Connection types consist of interior joints, exterior joints and knee joints. Exterior joints in a cyclically loaded frame will be subjected to shear strength of the longitudinal joints. Knee joint is required when restraints in beam reinforcement head along the top face of the joint [11].

Connection deformations are grouped into two parts, namely structures that do not tend to experience large inelastic deformations and do not need to be designed are called non-seismic structures and structures that must be able to accommodate large inelastic deformations are called seismic structures [11].



(a) (b)

Fig. 1. Types of beam-column joints (a) Inside hook, and (b) Outside hook [7].

With the above description regarding beam-column joints, the author wants to discuss connections with reinforced reinforcement models Inside hook and Outside hook using finite element analysis MIDAS FEA.

2 Current Design Method

To find the capacity of the exterior beam-column joints, three failure modes are considered as follows: (1) flexural yielding of beam reinforcement (2) joint crack failure, and (3) shear failure of beam reinforcement joints. The lateral force corresponding to each failure mode can be estimated as follows.

2.1 Structure Design Capacity

Estimating the failure of a structure based on the maximum load experienced by the structure. The behavior of the structure due to an earthquake will experience deformation depending on the amount of bending deformation, when combined with detailing to find the amount of ductility. Optimal deformation capacity energy dissipation will produce a structure that behaves plastically [10].

Fig. 2 shows the support reactions and internal moments at the joint face in an exterior beam-column connection specimen subjected to a lateral load (P) at the top of the column. M_{nb} is the nominal flexural strength of the beam and l_b is the length of the beam from the centerline of the column to the support. The capacity of beam column can be calculated from the moment equilibrium using the following Eq. 1:

$$P_n = \left(\frac{M_{nb}}{l_b - 0.5\, h_c}\right)\frac{l_b}{l_c} \tag{1}$$

$$P_u = \phi\, P_n \tag{2}$$

The capacity of the beam-column connection in Eq. 1 can be calculated when the beam reaches its yield state and in Eq. 2 it can be calculated when the beam has reached its ultimate condition [5]. In determining the shear strength capacity of the beam-column connection, it can be calculated using the following Eq. 3:

$$V_{jn} = c\lambda\sqrt{fc'} \times A_j \tag{3}$$

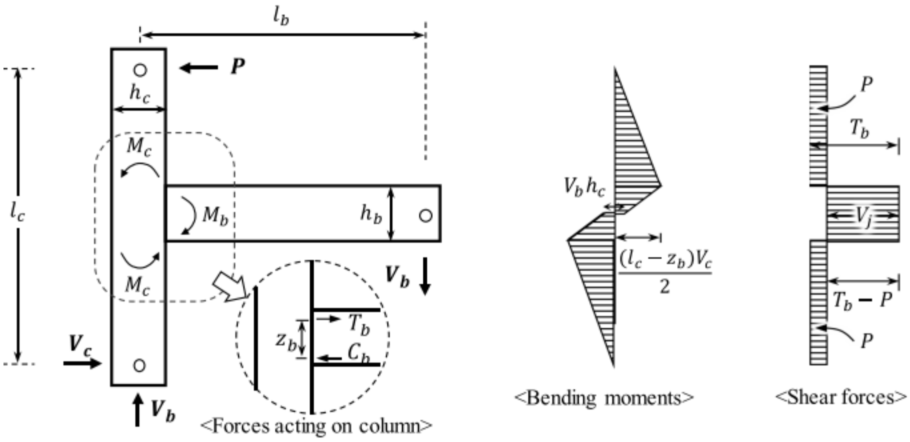


Fig. 2. Internal Forces and reaction forces acting on exterior beam-column connections [4].

Beam-column connections are considered if they are considered confined and cover at least three-quarters of the common faces. In sales, the effective area (A_j) in Eq. 3 can be calculated by adding the effective joint width to the joint height in the area of the reinforcement that produces shear [2].

2.2 Nonlinear Finite Element Method

Structural behavior in a case is said to be nonlinear if the strength matrix or load vector depends on the displacements. Changes in material properties, such as plasticity, are included in the nonlinear classification of materials. Configuration changes, such as large deformations and a beam whose elastic properties are included in the geometric nonlinear classification. Loads that cause large deformations in the structure can change the shape and behavior of the structure when the material stress reaches a certain limit, the material properties will also change [3].

Nonlinear analysis ignores the assumption of constant stiffness, stiffness which is defined as a change in behavior during the deformation process and the matrix stiffness will be updated continuously so that the structure converges with an iteration process that requires a long duration of time [12].

3 Research Method

The stages of the methodology and research flow can be seen in Fig. 3. Following are some explanations of the stages of this research method:

1. Conduct literature studies from various sources such as articles, books, scientific journals and other reliable sources related to research.
2. Collect and summarize data from the results of tests that have been carried out by [4].
3. Modeling and analysis using the finite element method and Midas FEA software.
4. Analyze the data obtained from the Midas FEA software to answer the objectives and formulation of the research problems.
5. Make conclusions to answer the objectives and formulation of research problems based on the analysis results obtained from the Midas FEA software.

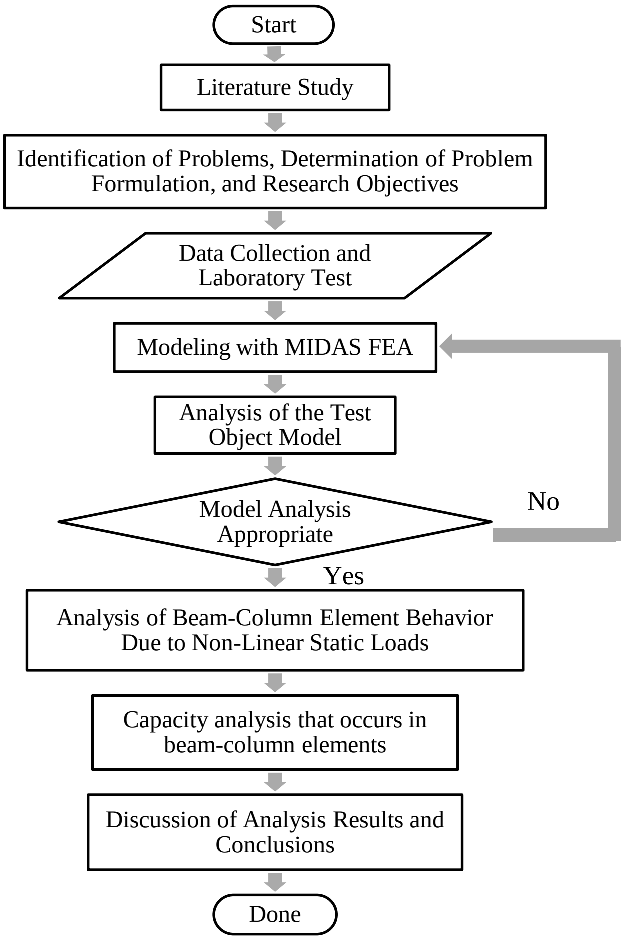


Fig. 3. Research flow chart

3.1 Type and Number of Test Objects

There are three (3) types of test objects for beam-column connections with each type of stirrups having different variants and dimensions. Table 1 presents the parameters of the specimen with code HBK 1 without joint shear reinforcement. Specimens with code HBK 2, HBK 3 use joint shear reinforcement. HBK 2, HBK 3 specimens use U-bar reinforcement details.

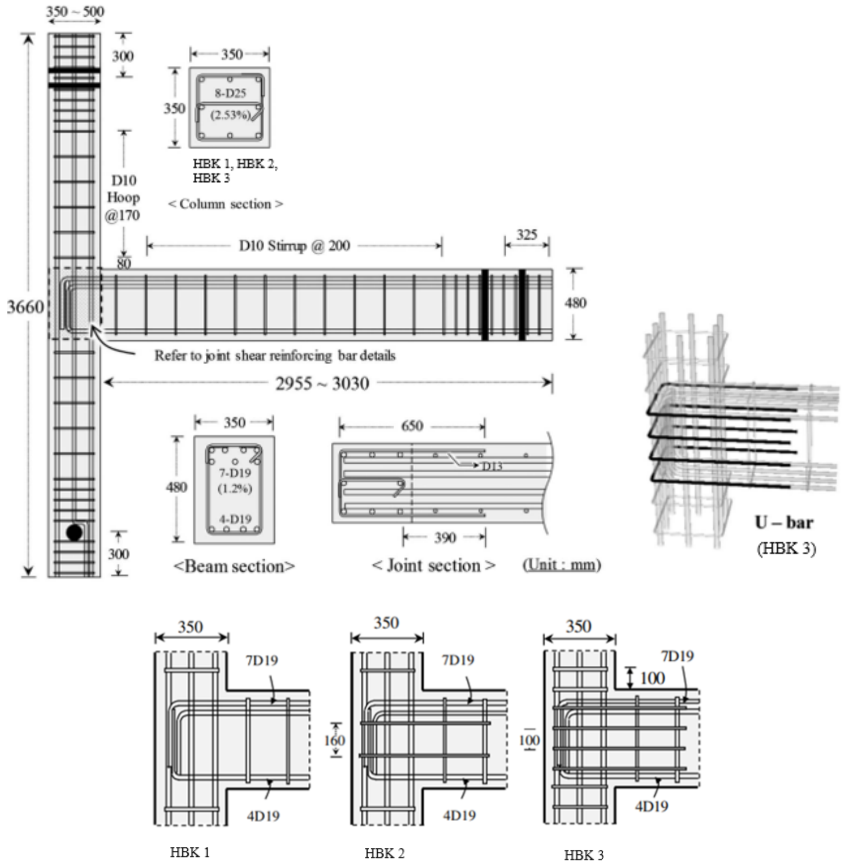


Fig. 4. Dimensions and reinforcement details of exterior connection specimens.

Table 1. Exterior Connection Test Object Parameters [4].

Specimen	Joint reinforcement		Column size, $b_c \times h_c$ (mm × mm)
	Number	Detail	
HBK 1	-	-	350 × 350
HBK 2	2	U-bar	350 × 350
HBK 3	4	U-bar	350 × 350

3.2 Test Spesimen

Dimensions of reinforced concrete beam-column connections tested with column height between pinned bearings as high as 3060 mm and beam length between roll bearings as high as 2880 mm. The column dimensions used for reinforced concrete specimens with codes HBK 1, HBK 2, HBK 3 are $h_c \times bc = 350 \times 350$ mm, and the beam size used for reinforced concrete specimens is $bb \times h_b = 350 \times 480$mm. The dimensions of the test object can be seen in Fig. 3.

3.3 Material Strengths

The quality of the concrete used by all codes of specimens is 31.4 and 31.3 MPa. The maximum aggregate size is 25 mm. Table 2, presents the yield strength f_y and ultimate strength f_u of steel bar reinforcement. For bars D10 and D13 with diameters $d_b = 9.5$ and 12.7 mm used for shear reinforcement, $f_{yt} = 449$~576 MPa and $f_{ut} = 577$~689 MPa. Each thick diameter db = 19.1, 22.2, and 25.4 mm for beam and column reinforcement D19, D22, and D25.

Table 2. Material strength of steel reinforcing bars [4].

Bar size	HBK 1 dan HBK 2				
	D_b (mm)	f_y (MPa)	ε_y	f_u (MPa)	ε_u
D10	9.53	449	0.00224	577	0.1018
D13	12.7	455	0.00227	628	0.1006
D19	19.1	540	0.00270	663	0.1056
D22	22.2	562	0.00281	707	0.1041
D25	25.4	569	0.00284	688	0.1147

Table 3. Material strength of steel reinforcing bars [4].

Bar size	HBK 3				
	D_b (mm)	f_y (MPa)	ε_y	f_u (MPa)	ε_u
D10	9.53	576	0.00288	689	0.0895
D13	12.7	528	0.00264	656	0.1075
D19	19.1	550	0.00275	685	0.1063
D22	22.2	577	0.00289	720	0.1001
D25	25.4	588	0.00294	702	0.1037

4 Result and Discussion

The results of the analysis of the yield capacity of the beam-column connections are calculated based on Eq. 1. The behavior of the increase in beam column connection capacity can be seen when the behavior of HBK is added 2 U-bar shear reinforcement and 4 U-bar shear reinforcement. The capacity of beam-column connections from the results of manual calculations, laboratory reference data and finite element analysis of the MIDAS FEA program will then be compared to one which can be seen in Table 4 and Fig. 6.

Table 4. Comparison of Melting Capacity and Increase in HBK Capacity.

Model	Yield Capacity (P_n) Chul-Goo, Park, & Eom (kN)	Yield Capacity (P_n) MIDAS FEA (kN)	Diffrence (%)	Ascension Capacity of HBK (%)
HBK 1	49.3	47.14	4.38	
HBK 2	49.3	48.45	1.73	2.77
HBK 3	99.1	94.6	4.54	95.26

Table 4 shows that the results of the analysis of the behavior of HBK 1 capacity when two U-bar joint shear reinforcement were added did not increase significantly by 2.77% and the behavior of the beam column relationship when four U-bar joint shear reinforcement was added showed a high increase of 95.26 % that occurs in HBK 3.

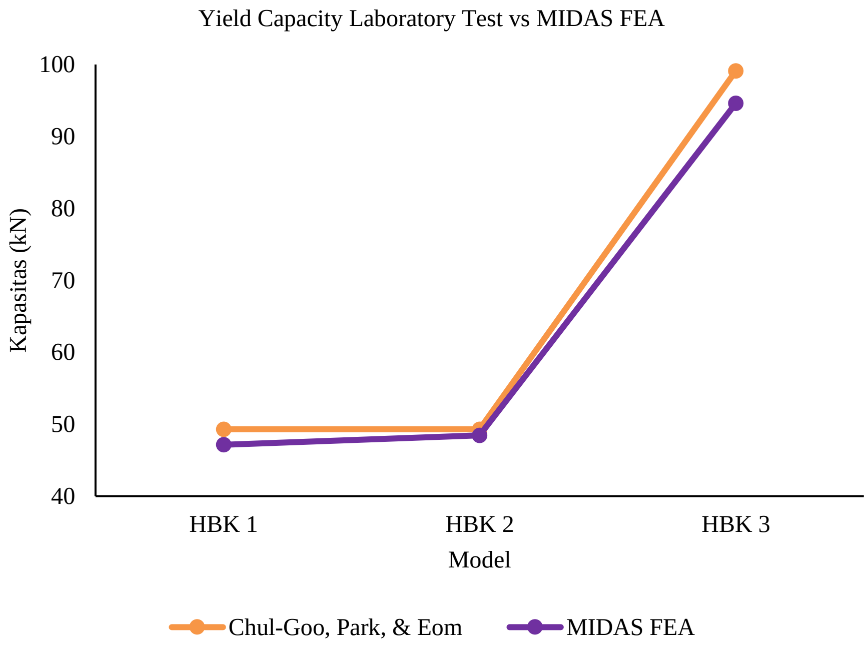


Fig. 5. Graph of Laboratory Test Melting Capacity vs MIDAS FEA

Table 5. Comparison of Melting Capacity of MIIDAS HBK 1, 2, 3 and HBK 4, 5, 6

Yield Capacity (P_n)		
HBK 1, 2, 3 (kN)	HBK 4, 5, 6, Modifikasi (kN)	Diffrence (%)
47.14	45.35	3.80
48.45	47.67	1.60
94.6	90.09	4.77

Table 5 shows that the yield capacity of the beam-column connections when hooked into the inside hook produces a greater capacity than the open hook outside hook, with the biggest difference being 4.77%.

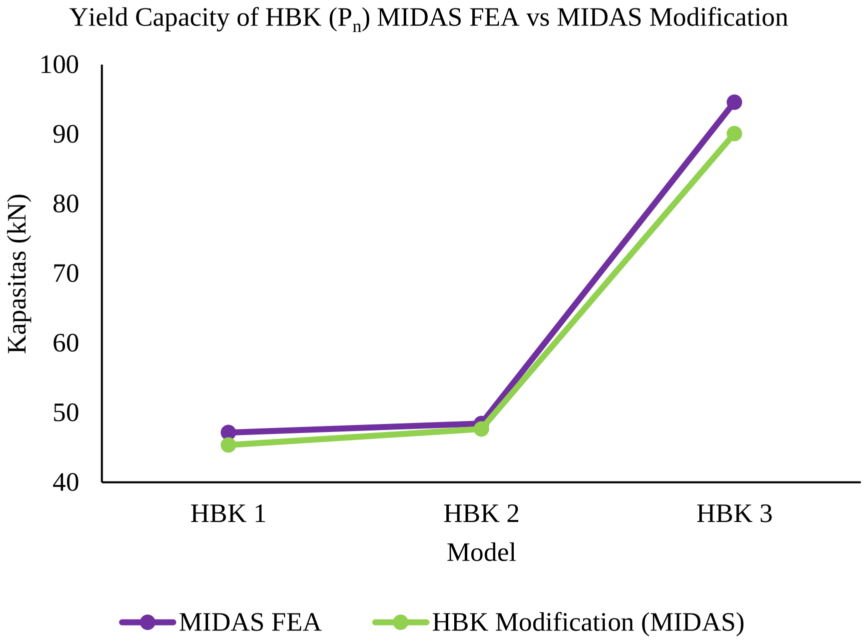
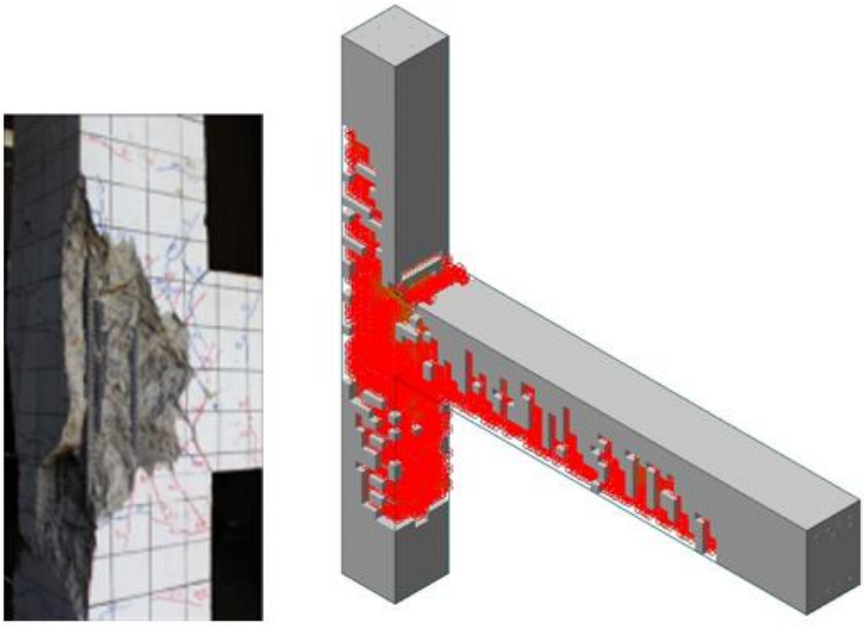


Fig. 6. Graph of MIDAS FEA vs Modified MIDAS Melting HBK Capacity

4.1 Column Beam Connection Crack Pattern

The crack pattern obtained based on the MIDAS FEA finite element testing and laboratory testing on the HBK 1 model without the U-bar joint shear reinforcement can be seen in Fig. 8, which shows an almost identical crack pattern.



(a) (b)

Fig. 7. HBK 1 Model Crack Pattern Laboratory Test Results; (b) 3D view of HBK Model 1 Crack Pattern MIDAS FEA.

5 Conclusion

1. The capacity of the beam-column connection when it reaches the melting condition based on the results of laboratory tests with the finite element method shows that the biggest difference is 4.54%.
2. The capacity of beam-column connections during melting conditions based on manual calculations with laboratory tests produces the largest difference of 4%.
3. The behavior of the beam-column connection capacity when two U-bar joint shear reinforcement was added did not increase significantly by 2.77% and the behavior of the beam-column connection when four U-bar joint shear reinforcement was added showed a high increase by 95.26%.
4. The bearing capacity of the beam-column connection in the ultimate condition when given a load with a modification of the outside hook open hook gives smaller results than the behavior of the beam-column connection capacity with hooks into the inside hook, which is 3.68%.
5. The crack pattern based on the results of finite element analysis with laboratory results shows almost the same crack pattern

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