

Experimental Study on Partially Restrained Beam-Column Connections

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Abstract: This paper presents an experimental investigation of the structural behavior of bolted semi-rigid beam-column connections. Three specimens with different connection parameters were created and tested under various loads up to failure. The main variables considered in the experimental test were the end-plate thickness and bolt diameter. The experimental results were used to obtain the moment-rotation ($M-\phi$) relationships for the tested connections, from which the connection stiffness, rotation capacity, and ultimate moment resistance were evaluated. The failure modes of the tested connections were observed and documented, showing that the connection behavior was mainly governed by end-plate deformation and bolt response. The results indicate that variations in end-plate thickness and bolt diameter significantly influence the stiffness and moment capacity of semi-rigid connections.

Keywords: Semi-Rigid Behavior, Beam-Column Connections, Experimental Results, Bolted End-Plate.

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I. INTRODUCTION

The structural performance of steel systems under different loading conditions is strongly affected by the behavior of beam-column connections. In many traditional design methods, these connections are often simplified into two idealized categories: nominally pinned or fully rigid.[6] However, in practice, most connections show behavior between these two cases.

Therefore, beam-column connections are generally classified according to their rotational stiffness and flexibility into three main types: nominally pinned, rigid and semi-rigid connections. These types are described as follows: [1] A pinned joint also known as "simply supported", the connected members are allowed to rotate freely at the joint. The pinned connection transfers only axial or shear forces, it is not transferring bending moments and is often referred to as a perfect hinge between the connected members. [3,7] While rigid connections are assumed to be perfectly fixed relative to each other. Rigid joint means that no relative rotation or translation is happening between the connected members during an applied loading. They transmit forces, moments and

deformations across the connection. [3,7] Semi-rigid connection is between rigid and pinned connection in terms of stiffness. Semi-rigid joints transfer moments between the connected members, but their deformation is not negligible, the rotations are allowed but less than the rigid connections.[5]

➤ Classification of Connections

Visual appearance does not determine a joint's classification. A standard approach for rigid and pinned joints is that the bolt row is located either outside or inside the beam flanges. This visual classification is not accurate. The proper classification of the connection is based on the system's stiffness, strength, and ductility shown in Figure 2. [1,8]

➤ Classification of Joints by Stiffness

This classification applies specifically to beam-column connection configurations. By comparing the initial rotational stiffness $S_{j,ini}$ with the classification limits, the connections can be categorized as rigid, semi-rigid or nominally pinned depending on their rotational stiffness. Zones illustrated in Figure 1.[4,7]

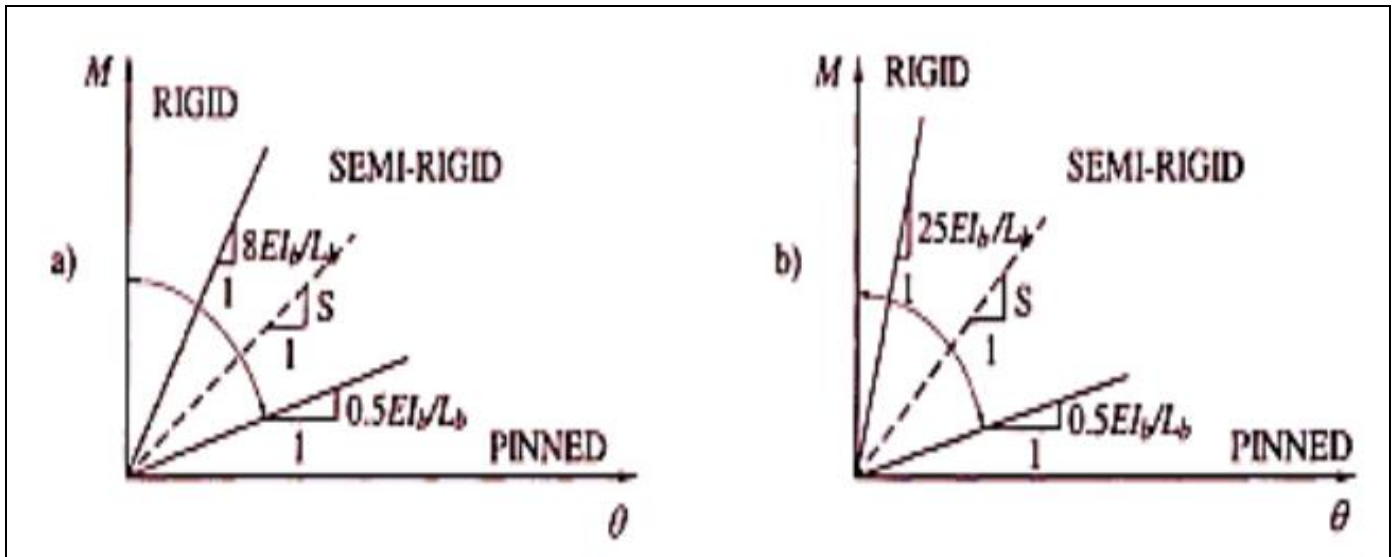


Fig 1 Classification of Beam-Column Connections by Stiffness According to the Eurocode 3 a) Braced Frames b) un-Braced Frames

➤ *Classification of Joints by Strength*

A joint is classified as a nominally pinned, if its design moment resistance, $M_{j,Rd}$, is not higher than 0.25 times the design moment resistance required for a fully rigid connection. The design resistance of a full strength should not be less than the connected beam and column. A joint doesn't have the criteria for nominally pinned or full strength; it must be considered as a partial strength resistance. [2,7,9].

➤ *Classification of Joints by Ductility*

In Eurocode 3, the classification of a cross-section is based on the width-to-thickness ratio of the components of the section. The classification of joints by ductility may be defined from the geometric and mechanical properties of the joint components (bolts, welds, plate thickness, etc.).

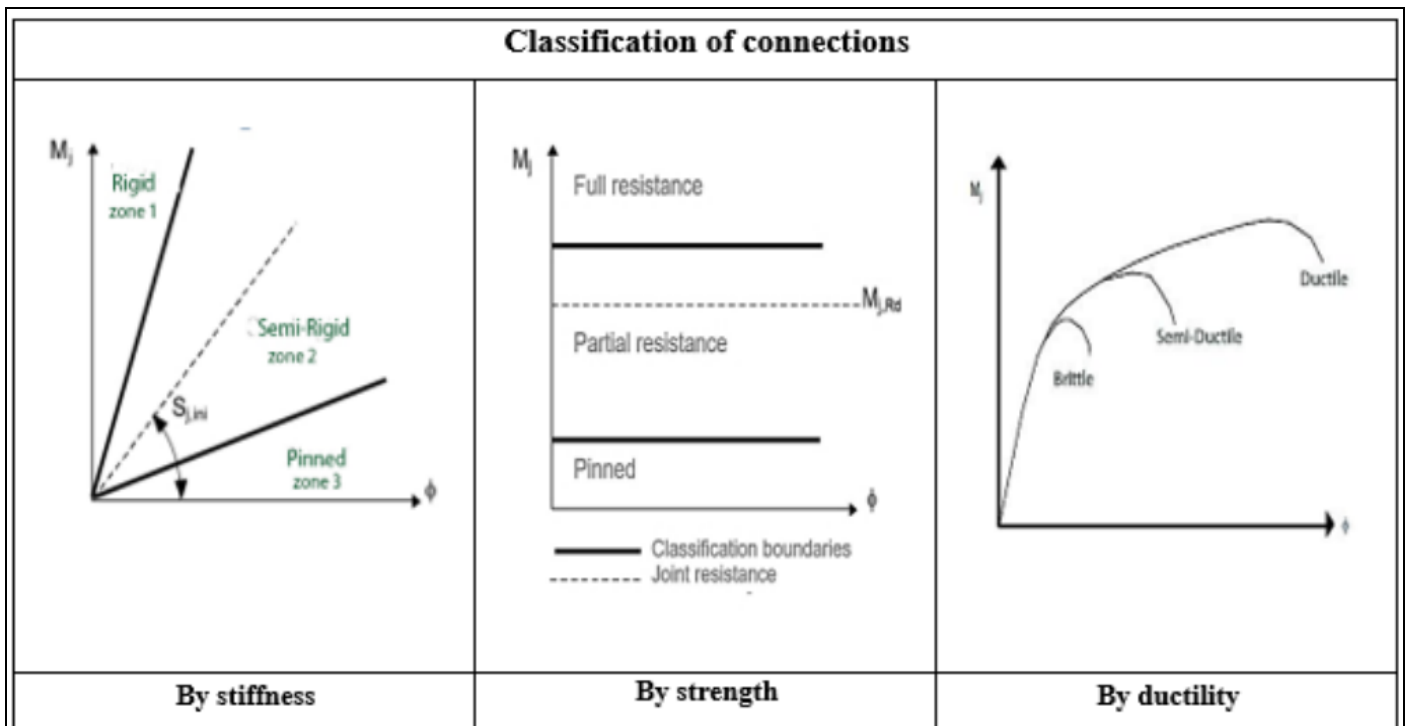


Fig 2 Connection Classification [8]

Understanding the failure mechanisms and ultimate capacity of structural connections has received increasing research interest in recent years. Therefore, this study aims to investigate the main parameters affecting the behavior of beam-column connections through laboratory

experimentation, where the moment resistance and stiffness of the connections are evaluated. This research aims to improve the design and performance of such connections in modern engineering practice.

II. TEST RIG AND EXPERIMENTAL SETUP FOR THE SEMI-RIGID CONNECTION

A test rig with a bolted end-plate beam-column connection was set up for an experimental test of the semi-

rigid beam-column connection. The cantilever beam span was 90 cm as "IPE140" and the column section was "HEA100" with a height of 170 cm. The end-plate was welded to the cantilever beam and connected to the column flange with four bolts. Table 1 displays all the end-plate's dimensions.

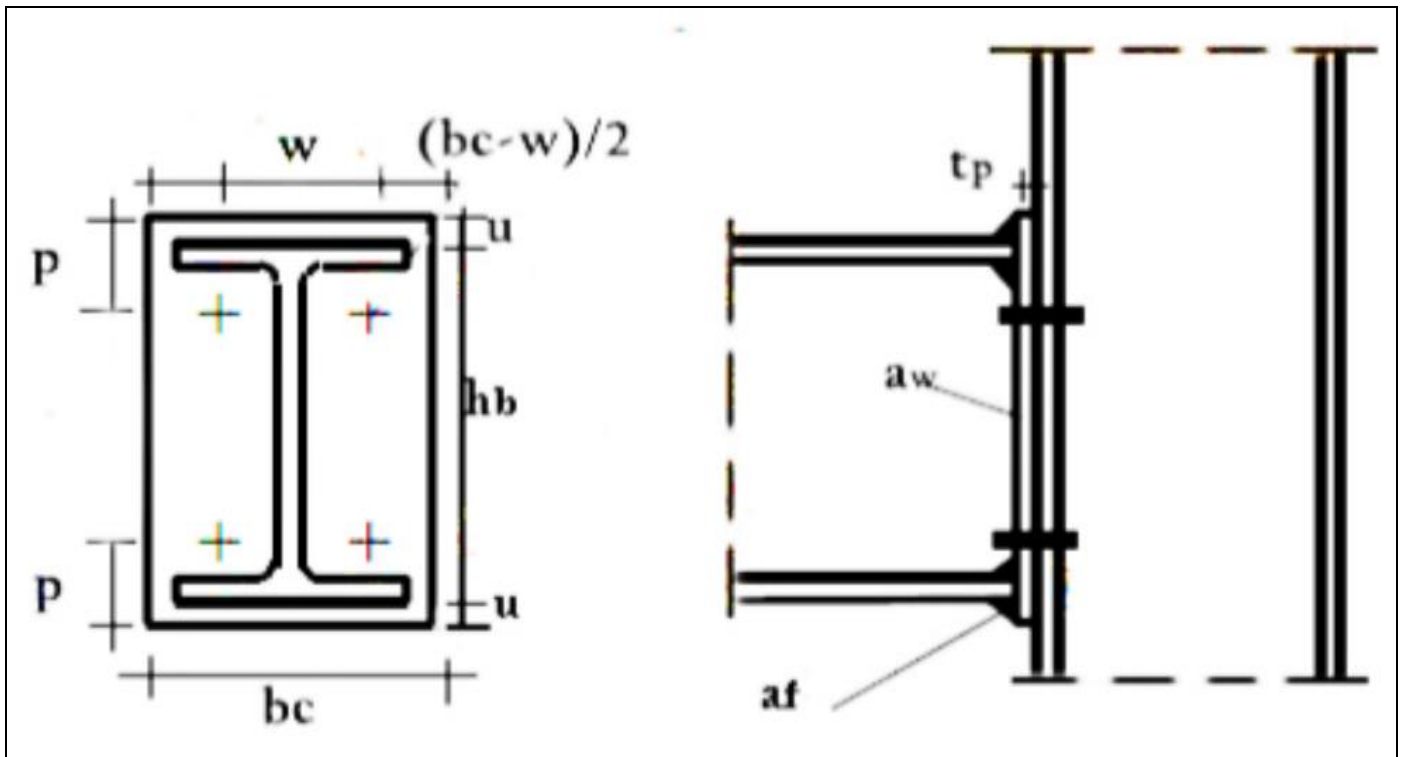


Fig 3 Single Side of Beam-Column Connection

Table 1 Definitions and Values of Symbols

Items	Description	Value	Items	Description	Value
W	Distance between bolts	60 mm	Bp	Width of end-plate	100 mm
P	Bolt pitch	40 mm	Hp	Height of end-plate	160 mm
U	Distance from bolt to edge of end-plate	10 mm	Aw	Web weld size	6 mm
Bc	Width of column	100 mm	Af	Flange weld size	6 mm

The experimental setup consisted of a loading frame fabricated from steel channel sections with pre-drilled 20 mm bolt holes, as illustrated in Figure 4. During the test, the column ends were considered to be fully restrained supports, while lateral movement of the beam was restricted to ensure stability of the system. The load was applied using a hydraulic jack positioned at 10 cm from the column face. Two dial gauges were used to record the deformation at the end-plate and the vertical deflection of the specimen. To evaluate the rotational capacity of the end-plate in semi-rigid connections, the loading position gradually moved farther from the column face, varying from 10 cm to 45 cm. The specimen was continuously loaded until failure occurred. Once a sudden, massive, and noticeable deformation occurred, the failure condition was considered. All experiments were tested in the Structural Engineering Laboratory at the Faculty of Engineering, Alexandria University.

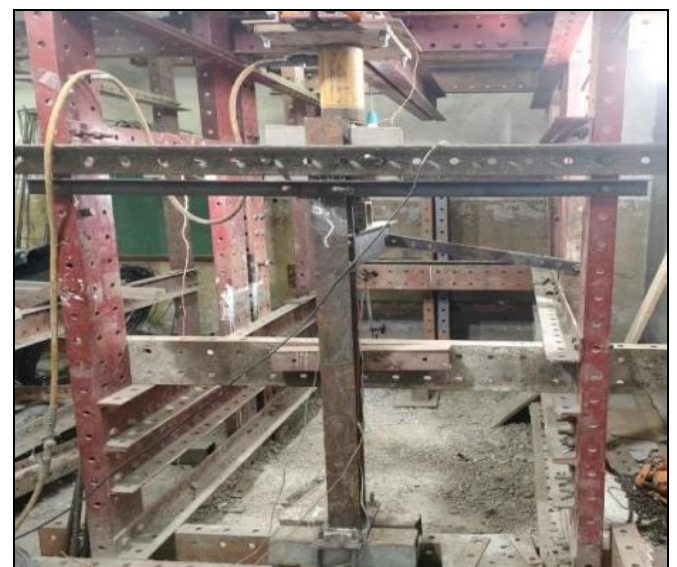


Fig 4 Experimental Test Rig for Semi-Rigid Beam-Column Connection

III. EXPERIMENTAL INVESTIGATION OF SEMI-RIGID CONNECTION

By changing the end-plate thickness and bolts diameter, three specimens were created to test and study the semi-rigid connection's end-plate. The geometrical configuration of the tested connection specimens is shown in Table 2. The connection components used in this study are

the column section HEA100, the beam section was IPE140, end-plate dimension was 160*100 mm, and the fillet weld thickness was 6 mm for welding the end-plate with the flange and the web of the beam, and there is one row of tension bolts. The study specimens depend on single sided beam-column profile, bolted end-plate connection and steel grade S235.

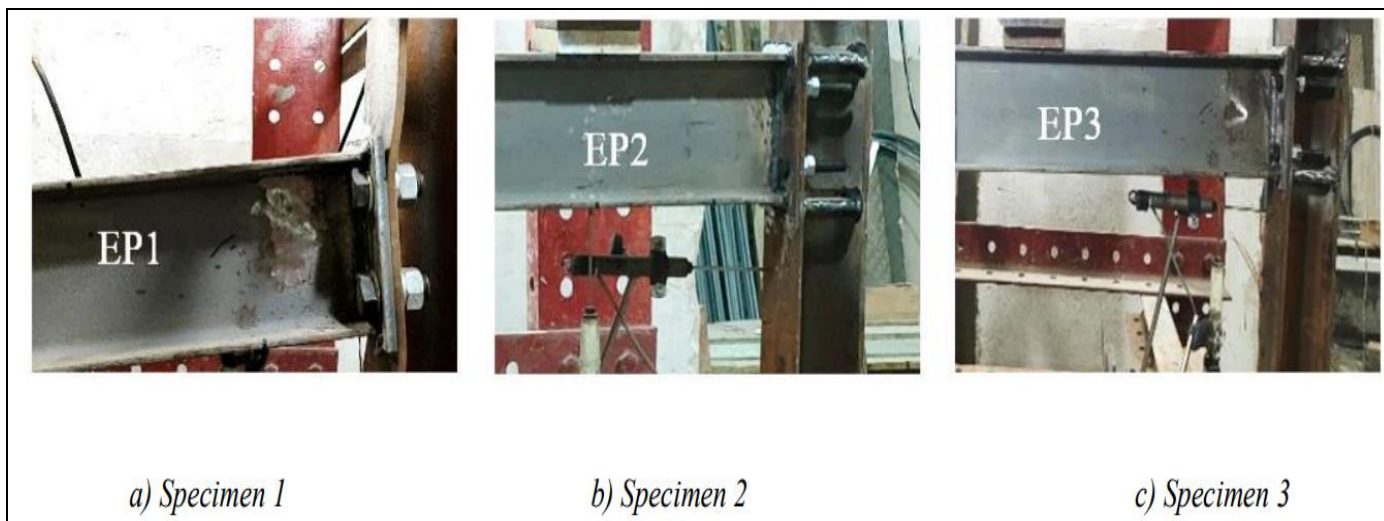


Fig 5 Experimental Specimens Used in the Study

Table 2 Geometrical Configuration of the Tested Connections

Model	Column Section	Beam Section	End-Plate Thickness in (mm)	Size of Bolt in (mm) and Grade
EP1	HEA 100	IPE140	12	M20 (10.9)
EP2			12	M12 (8.8)
EP3			8	M12 (8.8)

IV. EXPERIMENTAL RESULTS AND DISCUSSION

➤ Test Specimen 1

The experimental results for the first specimen obtained when the load was applied at 45 cm are shown in Table 3. The moment-rotation curves in Figure 6 illustrate the behavior of the other loading distances (10 and 25 cm).

Table 3 Experimental Results of Specimen 1 at a Loading Distance of 45 cm

Load “KN”	Deflection” mm”	Strain	Moment” KN.m”	Rotation” m-rad”
0.113	0.003	0.95	0.011	0.03
5.635	1.330	30.4	0.564	13.3
10.819	2.723	54.15	1.082	27.23
15.891	3.573	69.35	1.589	35.73
20.962	4.765	85.5	2.097	47.65
25.019	5.481	101.65	2.502	54.81
30.542	6.754	125.4	3.054	67.54
35.275	7.977	153.9	3.528	79.77
40.121	9.360	184.3	4.012	93.6
45.08	12.901	277.4	4.508	129.01
45.869	13.525	292.6	4.587	135.25
46.094	14.841	329.65	4.609	148.41
46.432	15.115	334.4	4.643	151.15
46.545	16.81	378.1	4.655	168.1
47.109	17.266	387.6	4.711	172.66
47.447	22.289	497.8	4.745	222.89
48.010	23.443	521.55	4.801	234.43
49.025	30.481	652.65	4.902	304.81
49.588	36.894	760	4.959	368.94

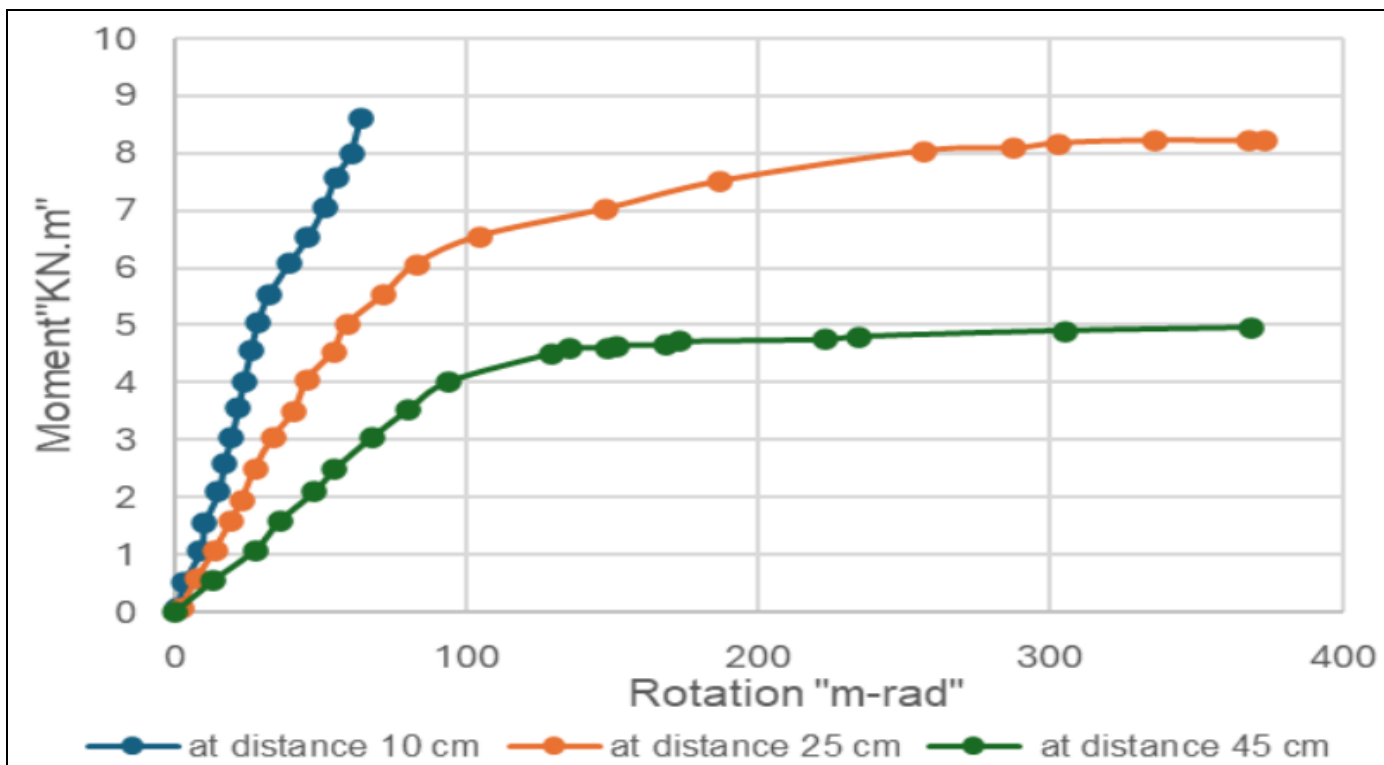


Fig 6 Moment-Rotation Curves of Specimen 1 at Different Loading Distances

Based on the moment-rotation curves for specimen 1 measured at 10 cm, 25 cm and 45 cm, it is evident that stiffness is highest at 10 cm and decreases at 25 cm and 45 cm due to increased distance from support. In the elastic region, all three distances exhibit a linear relationship between moment and rotation, where the slope represents flexural stiffness. After yielding, the curves enter a plastic region, indicating that significant rotational increases occur with minimal moment, reflecting a clear loss of stiffness.

• Discussion of Specimen 1 Results



Fig 7 Column Flange Deformation of Specimen 1 at a Loading Distance of 45 cm

In the test of the first specimen, the beam-column connection experienced nonlinearity on the third step of loading at distance 45 cm. Initial deformation was observed in the tension zone above the top bolt rows. Failure was characterized by deformation of the column flange in both tension and compression zones, as illustrated in Figure 7. The first test's results indicate that the end-plate's 12 mm thickness and the bolts' M20 (10.9) size had little effect on the connections' initial stiffness. So, stiffeners were added to the next two specimens to make the column section stronger.

➤ Test Specimen 2

Table 4 presents experimental results for the second specimen at a loading distance of 45 cm. The behavior of the other loading distances (10 and 25 cm) is shown by the moment-rotation curves in Figure 8.

Table 4 Experimental Results of Specimen 2 at a Loading Distance of 45 cm

load "KN"	Deflection "mm"	Strain	Moment" KN.m"	Rotation" m-rad"
6.311	0.403	23.75	0.631	4.03
10.481	0.7995	39.9	1.048	7.995
16.342	1.314	61.75	1.634	13.14
20.737	1.952	90.25	2.074	19.52
25.245	2.348	113.05	2.524	23.48
30.879	2.758	145.35	3.089	27.58
35.726	3.571	204.25	3.573	35.71

35.162	3.680	213.75	3.516	36.8
40.572	4.144	269.8	4.057	41.44
41.699	4.792	348.65	4.169	47.92
42.037	5.456	459.8	4.204	54.56
43.164	5.567	472.15	4.316	55.67
43.164	6.466	559.55	4.316	64.66
43.840	7.225	608	4.384	72.25
44.855	10.068	684	4.485	100.68
45.869	10.246	684	4.587	102.46
45.531	11.586	674.5	4.553	115.86

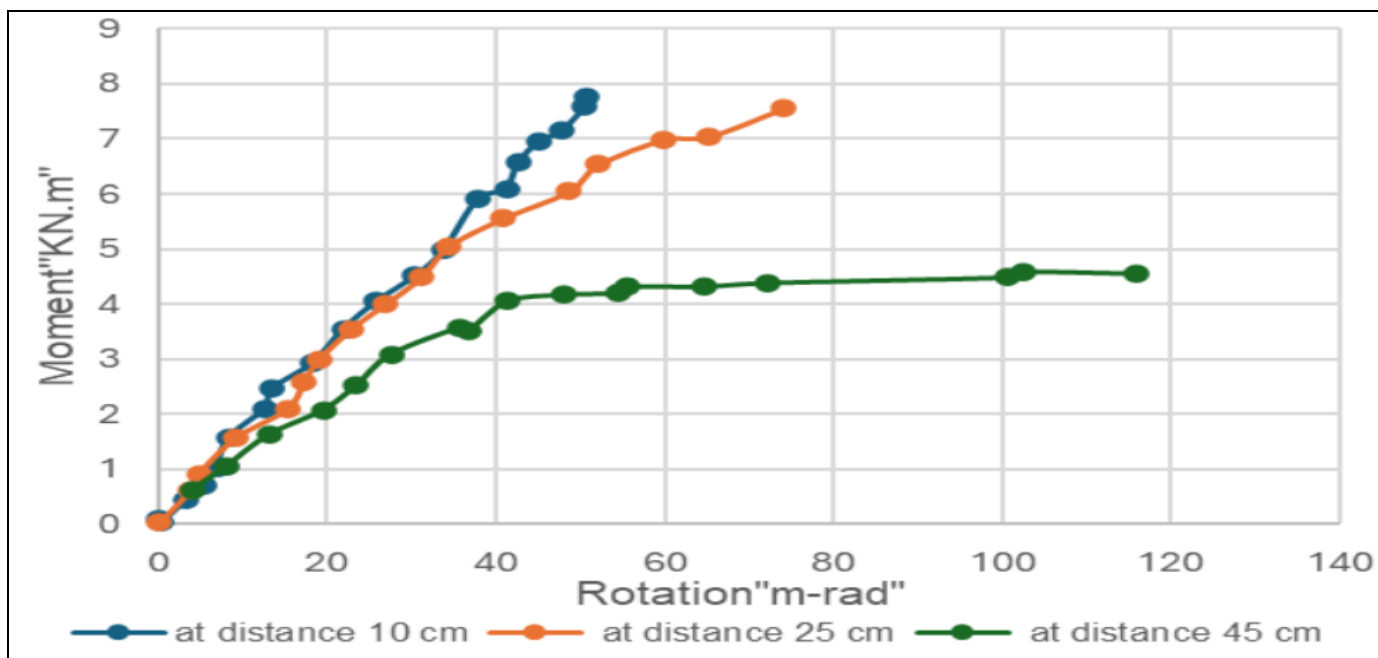


Fig 8 Moment-Rotation Curves of Specimen 2 at Different Loading Distances

The moment-rotation curves show that stiffness is highest at 10 cm and decreases at 25 cm and 45 cm due to increased distance from support. All specimens exhibit linear elastic behavior followed by yielding, after which they enter a plastic region where large rotations occur with minimal moment increase, indicating loss of stiffness.

• Discussion of Specimen 2 Results



Fig 10 Deformation of Column Flange and End-Plate, and Bolts Failure in Specimen 2 at a Loading Distance of 45 cm



Fig 9 Column Flange Deformation of Specimen 2 at a Loading Distance of 25 cm

In the test of the second specimen, the beam-column connection experienced nonlinearity on the second step of loading at distance 25 cm. The failure of the connection was identified as a deformation of the column flange in the tension zone, as illustrated in Figure 9. During the loading stage at 45 cm, several failure modes were observed. These included fracture of the upper bolt rows, deformation of the end-plate in the tension region, and increased deformation of the

column flange in the same zone. These failure modes are shown in Figure 10. The second test's results indicate that while the end-plate's 12 mm thickness has no effect on the connections' initial stiffness, the M12 (8.8) bolt size reduction has a major effect.

➤ *Test Specimen 3*

The experimental results for the third specimen obtained when the load was applied at 25 cm are shown in Table 5. The moment-rotation curves in Figure 11 illustrate the behavior of the other loading at distance 10 cm.

Table 5 Experimental Results of Specimen 3 at a Loading Distance of 25 cm

Load "KN"	Deflection "mm"	Strain	Moment "KN.m"	Rotation" m-rad"
0.225	0	0.95	0.023	0
6.199	0.889	0.95	0.619	8.89
11.27	1.681	14.25	1.127	16.81
15.327	2.270	19	1.533	22.7
19.723	2.787	21.85	1.972	27.87
24.907	3.414	24.7	2.491	34.14
30.654	4.247	40.85	3.065	42.47
35.162	4.839	51.3	3.516	48.39
40.685	5.854	55.1	4.068	58.54
45.418	7.004	95.95	4.542	70.04
50.377	8.635	274.55	5.038	86.35
55.674	11.589	878.75	5.567	115.89
60.182	14.232	1387.95	6.018	142.32
65.141	20.019	2516.55	6.514	200.19
66.493	21.655	2817.7	6.649	216.55
67.282	24.039	3305.05	6.728	240.39
68.071	27.296	4010.9	6.807	272.96
69.085	29.886	4192.35	6.909	298.86

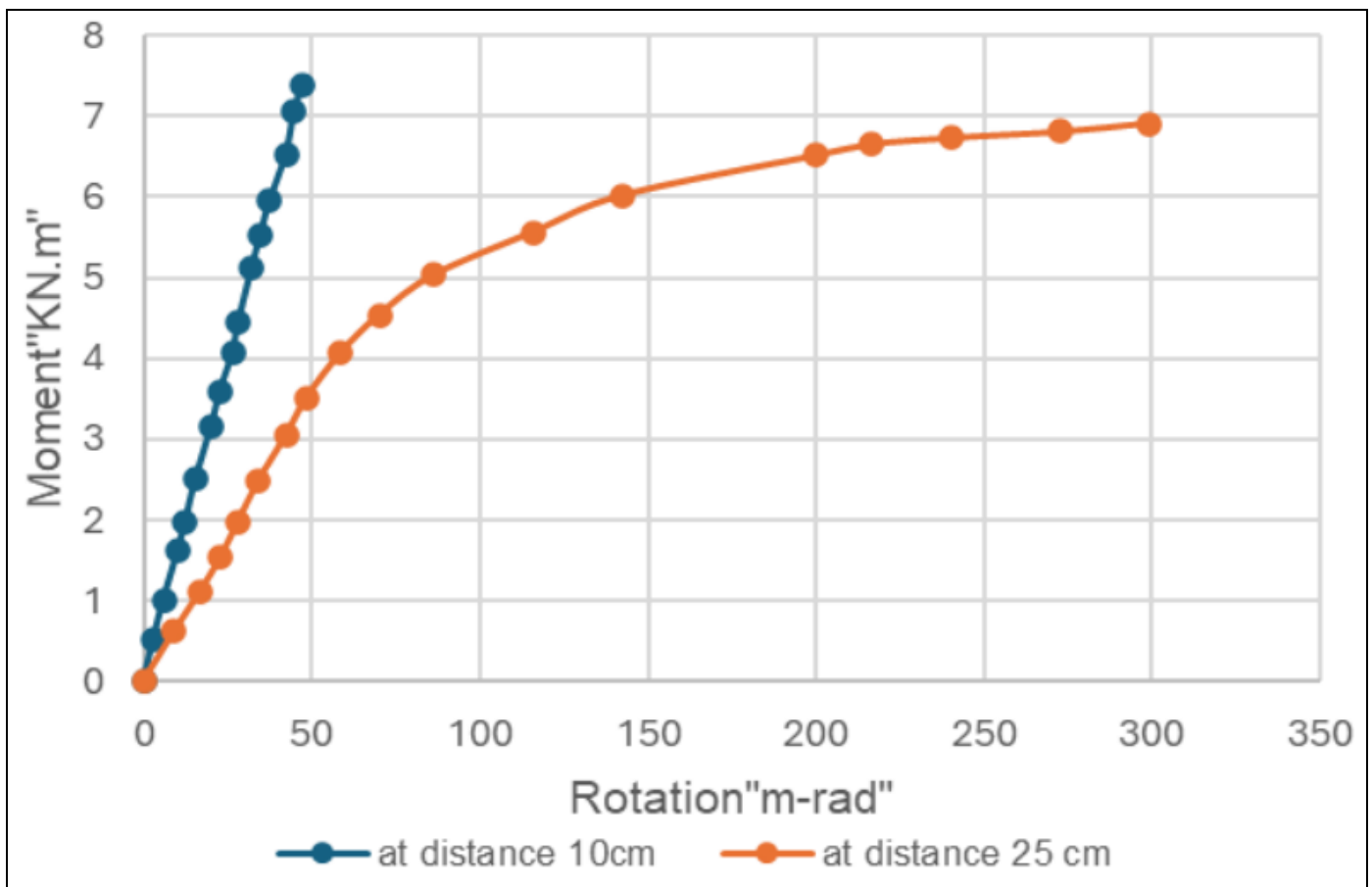


Fig 11 Moment-Rotation Curves of Specimen 3 at Different Loading Distances

At 10 cm, the specimen shows the highest initial stiffness, stiffness decreases with distance. In comparison to the 10 cm curve, the 25 cm curve displays greater rotation

values. The non-linear part shows plastic deformation or failure, while the linear part represents elastic behavior.

- *Discussion of Specimen 3 Results*



Fig 12 Deformation of Column Flange with End-Plate and Bolts Failure in Specimen 3 at 25 cm Loading Distance

In the test of the third specimen, the beam-column connection experienced nonlinearity on the second step of loading at distance 25 cm. The end-plate was the first to demonstrate deformation, and when loads increased, two different failure modes have been determined, as shown in Figure 12: the top bolt rows fracture and the end-plate's deformation increases. The third test's results demonstrate that the end-plate's thickness of 8 mm and the bolts' M12 (8.8) size have a major effect on the connections' initial stiffness.

V. CONCLUSIONS AND SUMMARY

➤ *Summary*

The failure was mainly in the column flange in the tension and compression zones using bolts M20 (10.9) and a plate thickness of 12mm. We decided to use stiffeners to strengthen the column section because there was no collapse in the bolts or the end-plate because the larger size of bolt and the end-plate was thicker than the column section. We repeated the experiment by using bolts M12 (8.8) and the same plate thickness of 12mm. The connection collapsed, as the bolt was broken when the moment increased too much. A slight formation occurred in the plate, and the column was only slightly affected. When we reduced the end-plate's thickness to 8 mm, while keeping the same specifications as the previous bolt, the connection could not bear the increased load and collapsed quickly as a result of the end-plate's small thickness. The bolt then broke. The following conclusions were reached based on the results of the experiments.

➤ *Conclusions*

The experimental tests demonstrated that if the end-plate thickness is increased, the column section used must also be increased so that the semi-rigid connection can support the moment and not collapse.

Increasing the end-plate thickness improves the connection's rigidity and load-carrying capacity. While reducing the plate thickness causes rapid collapse.

This study showed that the connections with larger bolt diameters and increased end-plate thickness exhibited greater deflection and higher bending moment capacity.

According to the study, semi-rigid connections are preferable for structural practice.

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