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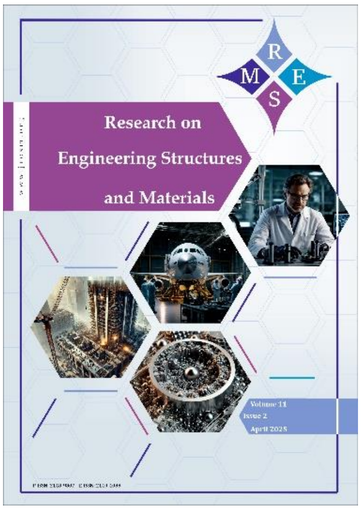

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Experimental investigation of bolted concrete column–foundation connections under lateral repeated loading

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Abstract

In this study, behavior of concrete column-foundation connections under repeated lateral loading was examined. Six column -foundation specimens were tested. The specimens consisted of one monolithic concrete column, three columns with the embedded stiffener connections, and two columns with the one-face welded (W1) connections (without stiffener). The stiffener connections were created by welding triangular plate to the longitudinal reinforcement and then welding them to the upper surface of the base plate, while in the W1 connections, the longitudinal reinforcement was welded directly to the upper face of the base plate. All specimens were similar in dimensions. The foundation measured 700 mm in length by 300 mm in width by 200 mm in thickness, and the columns a square cross section of 120mmx120mm with 700mm length. The study examined one of the most important factors affecting column foundations connection using steel base plates. The cyclic response of the specimens, such as failure load, failure type and lateral displacement histories, was examined under the effect of axial load of 80 kN. The experimental results showed that the columns using spatial stiffener connections were more resistant to lateral loads if compared to the W1 and monolithic column specimens under similar axial loading conditions.

1. Introduction

Columns in structural systems are among the most critical components in buildings when they were subjected to lateral repeated loadings, such as those induced by tremors, wind, and dynamic vibrations, as these loads directly challenged structural integrity and overall stability. Extensive scientific investigations have played a pivotal role in advancing the understanding of deformation mechanisms and non-linear behaviors of these members under cumulative effects, which enabled the development of more accurate and reliable design parameters. Consequently, research in this field is not merely an academic endeavor but a technical imperative aimed at minimizing human and material losses while ensuring the highest levels of structural safety in seismically active regions and under variable environmental loads.

Tasnimi investigate the response of prefabricated column footing connections under cyclic loading conditions to fill the information gap in the existing literature. Six quarter-scale models of prefabricated column footing connections, were tested under constant axial load and cyclic lateral displacement. The results showed that some of prefabricated column footing connections exhibited semi-rigid properties with higher strength than conventional monolithic connections, while other connection types showed moderate energy dissipation capacity and slight degradation of post-elastic stiffness. The hysteresis curves indicated that some prefabricated column footing connections exhibited a stable response with bond damage after yielding [1].

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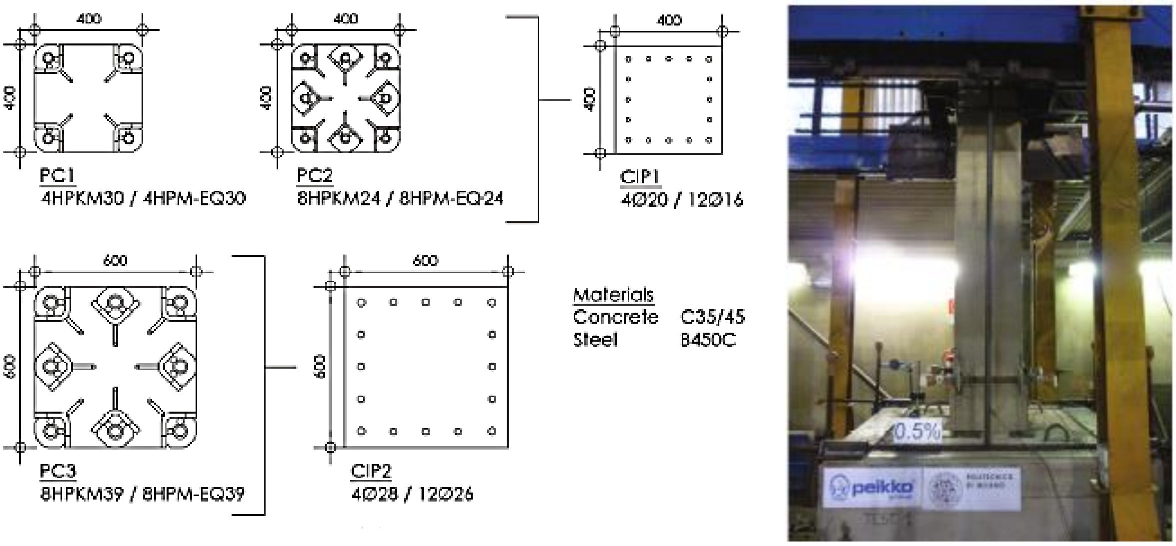
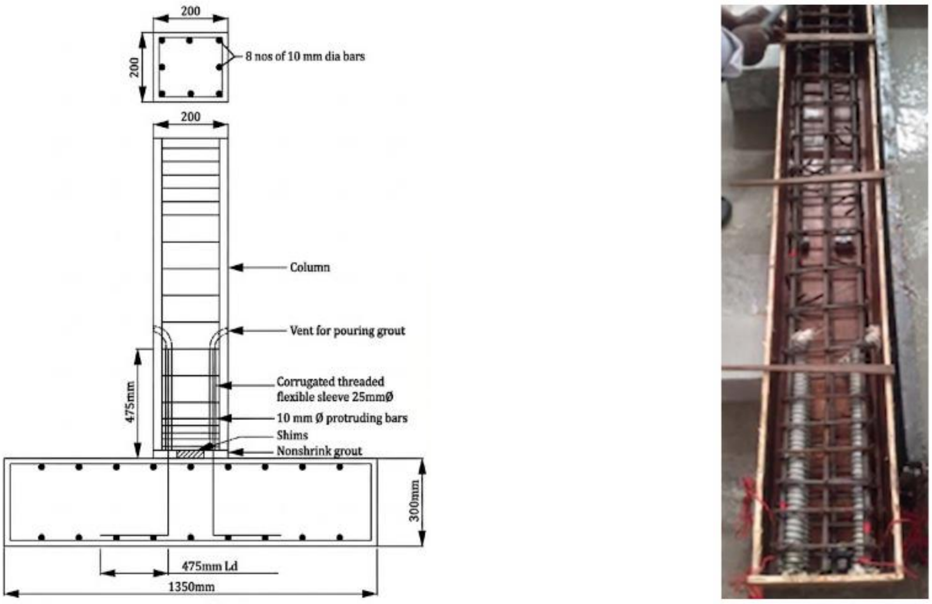


(a) (b)

Fig. 1. (a) Column cross-sections and material details of the specimens; (b) general view of the test setup

Nagaprasad et al. [2] investigated the post, installation of rectangular reinforced concrete (RC) columns using steel cage exteriors to provide better protection, specifically, to those old buildings whose structures had not been designed to meet today's seismic design codes. One of the columns was a typical reinforced concrete column, while the other two had been constructed to bear the combined action of constant axial compression and cyclic lateral displacement. The results showed that a steel cage turned the failure mode of the column from brittle to ductile and the bending resistance was therefore doubled. The use of end battens, especially at the locations of the plastic hinges, played a significant role in enhancing the strength of the concrete core under compression as well as in avoiding local buckling of steel angles. It was found that widening the end battens improved both plastic rotation and lateral strength, but total energy absorption was hardly affected. Most important thing is, it was not necessary to use bonding agents since the mechanical restraint exerted by the steel cage was sufficient to keep the retrofit fixed. While the previous study focused on strengthening individual columns, The project of Osman, M.H. and S.T. GRANT [3] focused on the earthquake resistance of exterior beam-column joint details in reinforced concrete moment-resisting frames, identifying them as the weakest link. Following BS8110 design criteria, it assessed joint vulnerability with variations in transverse reinforcement. Three reinforcement designs were evaluated: a reference specimen without shear links, one with closely grouped shear links, and one with inclined intersection bars. Results showed that the specimen with inclined bars performed best in terms of failure load, stiffness, and displacement capacity, while the reference specimen faced brittle failure due to anchorage issues. The findings underscored the importance of advanced joint designs for enhancing seismic safety, particularly in regions prone to distant earthquakes, such as Malaysia. In addition to experimental investigations on columns and beam–column joints, numerical approaches have also been employed to evaluate the behavior of column–foundation joints by Mahadik et al. [4]. The authors took a numerical approach to Column-Foundation Joints (CFJs) utilizing post-installed (PI) connections, addressing the need for European guidelines on seismic upgrading. The investigation utilized finite-element analysis, considering complex geometry, to assess the behavior of the PI systems, as they generally took better bond strength compared to the traditional cast-in (CI) rebar connections. The results demonstrated that, under seismic loading, CFJs utilizing CI and PI connections could fail, especially when embedment was low. However, once the embedment exceeded a certain level, the post-installed connections showed better robustness. While the CI connections experienced bond failure, the PI connections experienced ductile yielding. The results demonstrated the potential benefits of the PI systems, including the scenarios under which they might fail, to establish the framework for post-installed rebars in seismic areas .While the previous studies mainly examined cast-in and post-installed reinforcement systems, other researchers investigated alternative precast connection systems, particularly bolted column-to-foundation connections, Camnasio and Kriakopoulos[5] examined the performance of bolted precast concrete column-to-foundation connections subjected to cyclic loading. The goal was to achieve seismic performance that was similar to that of monolithic structures while also dealing with the risk of weak connections in areas with a lot of earthquakes. Quasi-static cyclic displacement tests on large specimens compared bolted systems using Peikko HPKM column shoes and HPM anchor bolts with a conventional cast-in-place (CIP) reference. Design modifications such as unbonded anchor bolts and fiber-reinforced mortar grouts were implemented to enhance structural performance. The findings indicated that bolted systems could match monolithic structures in ductility, stiffness, and energy absorption, meeting seismic design code requirements with a balance between precast construction speed and CIP energy dissipation capacity.



(a) (b)

Fig. 2 .Reinforcement details of grouted sleeve specimen (GS)

Further experimental studies have compared various precast column–foundation connection systems under cyclic loading conditions. An analysis of the cyclic performance of the precast column to foundation connections under reverse cyclic loading have been examined by Camnasio and Kriakopoulos). The study showed that there was considerable seismic susceptibility in precast structures. Also, it was found that comparing the performance of base plates, pocket connections, and grouted sleeves with monolithic structures, offered higher ultimate strength, they also experienced higher stiffness degradation. Notably, the performance of the PC I pocket connection was higher than that of monolithic structures, with a 184% increase in post-earthquake resistance and a 137% increase in energy dissipation. Although grouted sleeves were cost-effective, they also experienced premature failure due to bond loss. The results of the study, therefore, showed that precast structures could offer post-earthquake resistance and ductility that were equivalent to or higher than that of monolithic structures in seismic zones.

Beyond column–foundation systems, bolted connections have also been investigated in beam–column joints to enhance seismic repairability in prefabricated structures by Liu et al. [6] The investigation into a new bolt-connected prefabricated beam-column joint aimed to enhance seismic performance by optimizing damage distribution for quicker repairs post-earthquake. Two full-scale specimens, J1 and J2, were tested under low cycle loading with varying bolt strengths, compared against finite element analysis (FEA). Results demonstrated significant plastic deformation and energy dissipation, indicating resilience. Increased bolt strength improved load capacity, stiffness, and energy dissipation. FEA suggested a higher compression ratio accelerated concrete damage. A third design, J3, with integrated angle members was proposed, showing improved load transfer and reduced shear damage in the concrete core. Overall, bolted connections offer strong solutions for prefabricated constructions and enable localized earthquake repairability. It was also a well-established fact that repeated loading could cause cumulative damage under lateral repeated loading to structures. In areas prone to regular earthquakes, the bolt systems connecting columns with the footing were areas of concern, as they were subjected to side-to-side motions during quakes. Their performance depended on the properties, configuration, and unpredictability of the materials involved, thus requiring a combination of theory and practical experimentation to improve the strength of reinforced concrete buildings. Despite extensive analytical studies on the residual strength after repeated loading and ductility of single- and multi-degree-of-freedom systems, there was a lack of experimental data on the interaction of horizontal and vertical seismic components [7]. Furthermore, research on composite structural members has also contributed to understand load transfer mechanisms in column systems. Sheeba Ebenezer [8] investigate concrete-filled steel tube column members, encased in steel material, and its improvement in useful ways regarding the efficiencies of load transfer with the inclusion of shear connectors. In the study, the investigators developed and constructed full-scale concrete-filled steel tube column samples, each having a unique configuration of the shear connectors, and how these interacted and worked together in a composite manner, with the column members put through a series of axial and eccentric loading tests to ascertain the load capacity, total column stiffness, total column ductility, and failure mechanisms. From the study, the presence of properly designed shear connectors was highlighted as being very useful and significant, in the efficiencies of load transfer, the upgrade of the capacity of the column for axial and lateral loading, the retardation of the onset of tube buckling phenomena, and the upgrade of the column's ductility, compared to column members without shear connectors. These studies were highly useful and significant, particularly in relation to column connections and how the column and foundation were bolted together and subjected to repeated lateral loading regimes. Similarly, the behavior of precast reinforced concrete connections under seismic loading has been widely investigated in recent years. The behavior and performance of precast reinforced concrete connections under seismic and lateral loading were extensively investigated in previous studies [9] focused on five precast reinforced concrete wall-raft connection configurations subjected to cyclic lateral load, which proved that the grouted connectors with rods of a diameter of 12 mm could improve the lateral load capacity by 57% compared to the unconnected ones. This review aimed to synthesize the results of 30 highly pertinent research works on precast concrete connections, including experimental, numerical, and hybrid approaches. From the literature, it was revealed that there had been a significant evolution of precast concrete connections, from simple mechanical interlocks and shear keys in the 1990s to sophisticated energy-dissipation devices, post-tensioning systems, and hybrid connections in recent times. Among the principal types of connectors were grouted couplers and sleeves, bolted connections, post-tensioned tendons, tooth-groove connections, and engineered steel energy dissipaters, which exhibited various advantages in lateral load capacity, energy dissipation, ductility, and constructability. Despite the significant development in the subject, there were major research gaps in full-scale seismic validation, standardization of design guidelines, long-term assessment of durability, and comparison of results considering confounding factors. This review benefited structural engineers and researchers with an extensive overview of the state of the art, the performance of various types of connectors, and future research directions in seismic design of precast concrete connections. Despite numerous studies on column-base joints, significant gaps remained regarding the effects of connection type, base plate thickness, anchor bolt diameter, and count on the dynamic response of structures to lateral repeated loads. Notably, there was limited comparative research on various joint types and their impact on structural performance. The influence of base plate thickness under repeated loads was underexplored, while the correlation between anchor bolt diameter and cyclic behavior was inadequately understood. Additionally, the role of anchor bolt quantity and layout in load redistribution lacked sufficient investigation. There was a pressing need for integrated studies examining these parameters concurrently to better inform seismic designs and optimize structural detailing, Accordingly, the main aim of the present study is to examine the influence of the connection type, base plate thickness, number of anchor bolts, and bolt diameter on the seismic performance of column-foundation systems, with the final target of developing a strong structural configuration to withstand repeated loads.

2. Materials and Methods

2.1 Materials

2.1.1 Cement

Sulfate-resisting Portland cement, with trademark AL-JESR, manufactured by Lafarge Cement Factory, was used in mix. The compliance of the cement was carried out according to the Iraqi Standard No.5/1984 [10].

2.1.2 Fine Aggregate

Natural sand was used in producing the concrete mix with a maximum size of 4.75 mm, which is brought from Kerbala factories. The tests were implemented in the laboratory of the University of Kerbala, where the tests complied with the Iraqi Specifications No. 45 [11].

2.1.3 Coarse Aggregate

A black crushed gravel with a maximum size of 12.5 mm was used. This gravel was cleaned and washed by water and left out to dry in air. as well as the Iraqi Specifications No. 45 [11].

2.1.4 Water

Tap water was used in producing the concrete mix, as well as the curing process for all columns. This water was clean and free from unwanted materials like organic materials, salts, acids, and Alkalis.

2.1.5 Superplasticizer

The current experimental investigation included a high-performance concrete superplasticizer based on a modified polycarboxylate chemical compound. The concrete superplasticizer supplied from MBCC GROUP, which is Master Glenium 54, is suitable for a variety of concrete mixes and is compatible with all types of Portland cements and cementitious materials such as Microsilica, Fly Ash (PFA), and Ground Granulated Blast Furnace Slag (GGBS). The main benefit of using this concrete superplasticizer is its moisture-reducing properties, together with the following benefits: early compressive strength development, flexural strength, carbonation resistance, permeability, creep, shrinkage, and overall durability. A superplasticizer was added at a dosage of 1–2% by weight of cement to improve the workability of the normal-strength concrete.

Fig. 3. All specimens

Fig. 4. Curing of specimens

For the column specimens, Portland cement, fine sand, coarse aggregate, superplasticizer, and water were used in the concrete mixture. Compressive strength of about (45) MPa had been gained from these quantities. The American guidelines' recommendations for concrete mix design were followed in mixing and testing. In this study, the mix was designed according to the mix design approach described in (ACI 211.1-91). After finishing the casting process, all columns were submerged with water in the basin of College of Engineering of the University of Kerbala laboratory until the testing day After 28 days. Figure 4 showed the specimens in casting stage and curing.

Table 1. The quantities of materials for one cubic meter (Kg /m³)

Ingredients	Weight
Cement	410
Coarse aggregate	850
Fine aggregate	1010
Water	131
Superplasticizer	6 kg /m^3 (5.45 L/m^3)
w/c	0.319

Table 2. Gradation of sand and the chemical composition of sand

Sieve size (mm)	Cumulative Passing %	Passing accumulated %Limits of Iraqi specifications No. 45/1984, zone (3)
10	100	100
4.75	97	90-100
2.36	91	85-100
1.18	85	75-90
0.60	76	60-97
0.30	45	12-40
0.15	10	0-10
Specific Gravity	2.63	
Water Absorption	1.2%	
Fineness Modulus	2.71	
Materials passing from sieve 75 µ %= 3.6% (specification requirements up to 5%)		
SO_3 content=0.364% (specification requirements up to 0.5%)		

** These results were performed in the Karbala construction Laboratory*

Table 3. Coarse aggregate grading results and chemical characteristics

Sieve size (mm)	Cumulative Passing %	Passing accumulated %Limits of Iraqi specifications No. 45/1984, zone (3)
37.5(1.5 in)	------	-----
19(3/4in)	100	100
12.5(1/2in)	98	90-100
10(3/8in)	62	50-85
4.75(No.4)	3	0-10
Specific Gravity	2.68	
Water Absorption	0.8%	
Maximum Aggregate Size	12.5mm	
Mechanical wear =16.1% (specification requirements up to 35%)		
Materials passing from sieve 75 µ %= 0.29% (specification requirements up to 3%)		
SO_3 content=0.03% (specification requirements up to 0.1%)		

** These results were performed in the Karbala construction Laboratory*

2.2 Concrete Properties

Concrete had a significant test in fresh concrete, like slump test. In this test, the value of a slump measured is 35 mm for normal strength in the Abrams cone test. To determine the compressive strength of the concrete at the age of 28 days, twelve cubes of size (150mm x150mm x150mm) of the concrete mix were provided and tested as per (BS 1881 Part 116:1989) , three cylinders of size (Φ 100 x H =200mm) of the concrete mix were prepared and tested as per the [12]three cylinders of the concrete mix were tested for the splitting Tense test in order to determine the Indirect Tensile strength of the mix according to the [13]and three prisms of the concrete mix were of the dimensions (W=100xH=100 x L=500mm) in order to determine the rupture strength of the mix in accordance with the [14]. See the Fig. 5, and (Table 4). The compressive strength value reported in Table 5 (47.6 MPa) was obtained from tests conducted on cube specimens.

(a) (b) (c) (d)

Fig. 5. Fresh and hardened concrete test (a) slump test, (b) modules of rupture test, (c) splitting tensile strength test, and(d) compressive strength test

Table 4. Values of concrete compressive strength, splitting tensile strength and rupture tensile strength (MPa)

NO.	Compressive Strength (MPa)	Splitting Tensile strength (MPa)	Rupture Tensile strength (MPa)
1	48.7	4.8	5.31
2	49.8	5.2	5.63
3	46.0	3.9	4.28
Average	47.6	4.63	5.07

2.3 Steel Reinforcement

Two steel reinforcements bars diameter were used in the experimental, Ø10 mm steel reinforcements, which as main reinforcement and Ø8 mm steel reinforcements as ties. The steel reinforcement test had to be conducted with at least three specimens of each steel type, and each specimen had to be not less than 370 mm long, as required by its tensile test, as mentioned by [15]. The test was conducted at University of Kerbala by utilizing a tensile testing machine, as shown in figure 6. The values of tensile strength of each type of steel reinforcement are shown in Table (5).

Fig. 6. Tensile testing machine for steel reinforcement

Table 5. Properties of shear steel reinforcement

Nominal diameter (mm)	Actual diameter (mm)	Elongation % In 50 mm min%		Yield Stress fy MPa		Ultimate Strength Fu MPa		Modulus of Elasticity, E MPa*
		Result	Limit	Result	Limit	Result	Limit	
8	8.4	12.7%	9-7	434.4	420	640.3	550	200000
10	9.84	6.3%	6-7	574.6	550	701.6	690	200000

** Assumed value*

2.4 Experimental Work

2.4.1 Preparation Method

The experimental program presented in detail in this paper involved the evaluation of six specimens, including two connection types and a monolithic one. the present study encompassed three separate categories of specimens: The Reference specimen (first category), CF1, featured a monolithic column-foundation connection, the column and foundation formed a single structural unit with continuous concrete and steel reinforcement, allowing internal forces to transfer directly without separate bearings or mechanical fasteners. The second category included two columns labeled as 4P10A16W1-5 and 6P10A16W1-5, both specimens employed the same technique for connecting the columns with base plate. This technique (W1) was carried out by welding the longitudinal reinforcement bar of the column to the top surface of the base plate. In accordance with AISC specifications, a minimum base plate thickness of 10 mm was maintained for both specimens. Anchor bolts diameter size in both specimens were 16mm. Each column had five longitudinal reinforcement bars; four of them were positioned on the corners, while the fifth bar lay in the middle. However, the middle bar had a length of 300mm, which was 40% of the total length. Differences in this category lay in the fact that while the 4P10A16W1-5 specimen had at least four, the 6P10A16W1-5 had six bolts. The second category consisted of three columns designated 4P10A16S-4, 4P10A18S-4, and 4P15A16S-4. Each specimen used the same mechanism to attach columns to the base plate, which involved the welding of stiffeners to the longitudinal steel reinforcement near the top surface of the base plate. It should also be noted that the base plates thickness for this category is 15 mm thick. The diameter for all anchor bolts in specimens 4P10A16S-4 and 4P15A16S-4 was 16 mm, whereas in 4P10A18S-4, it was 18 mm. Each column specimen contained four longitudinal steel bars in the corners. The footing feature in each column of the specimens used at least four anchor bolts, but some specimens had six anchor bolts. Foundations of all specimens were produced with identical dimensions: 700 mm in length, 300 mm in width, and 200 mm in thickness, while the columns were designed with a square section of 120 mm x 120 mm and a height of 700 mm. All Specimen reinforcement followed the ACI Building Code minimums. The columns used four 10 mm longitudinal bars for specimens with stiffeners as the connection type and five 10 mm longitudinal bars for specimens with one-face weld connections(W1). the foundation had seven 10 mm bars spaced at 40 mm transversely and 10 mm bars longitudinally. Concrete cover was 30 mm for the footing and 20 mm for the column to protect the reinforcement and maintain integrity.

Each specimen had a code that showed its most important features. The first number showed the number of anchor bolts, P followed by a number showed the plate thickness in millimeters, A followed by a number showed the anchor diameter in millimeters, and the last part showed the connection type and the number of longitudinal bars used in the column.

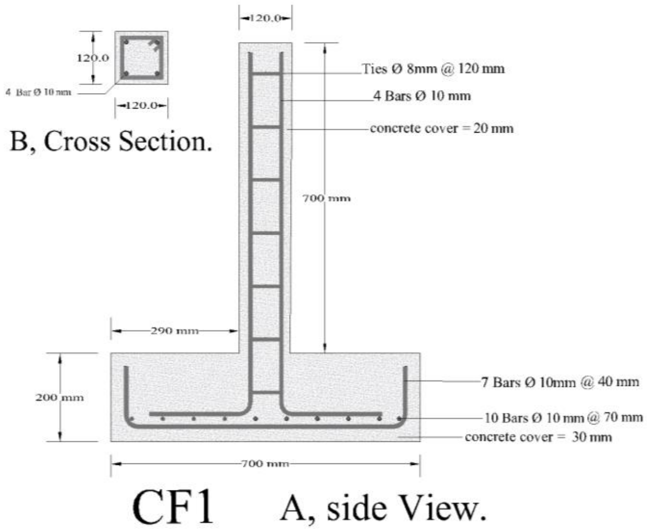


Fig. 7. Details of steel reinforcement for the column- foundation reference (A) Side view (B) Cross-section

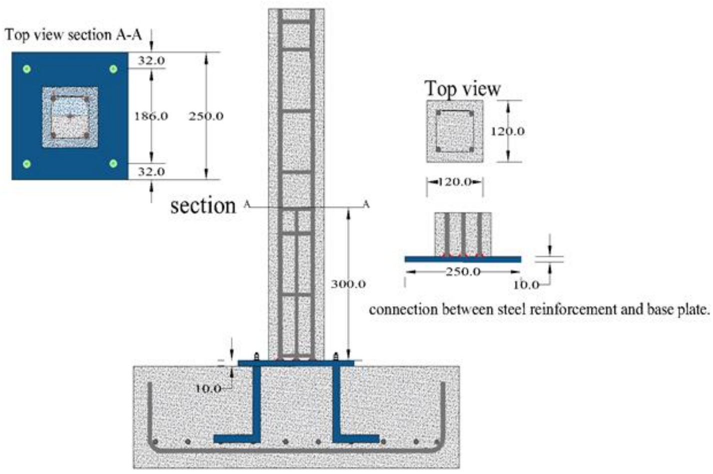


Fig. 8. Details of steel reinforcement for the column- foundation bolted connection one face welded type

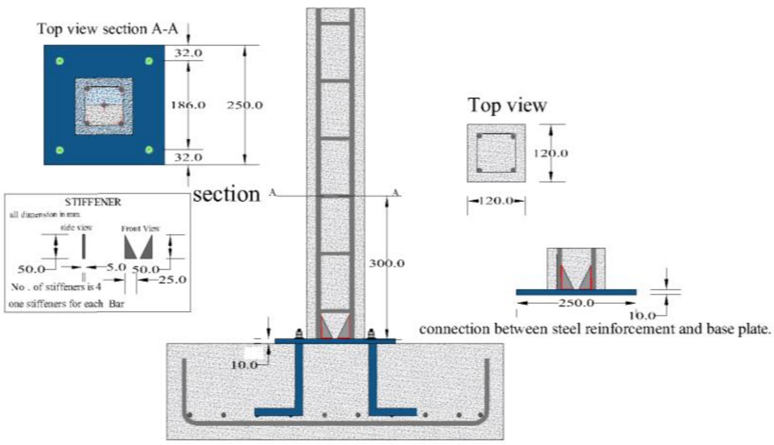


Fig. 9. Details of steel reinforcement for the column- foundation bolted connection stiffeners type

Table 6. Specimens' designation

Group No.	Specimen Label	No. of Bolt	Plate thickness (mm)	Bolt Diameter (mm)	No. of Column Bars	Type Connection
Ref	CF1	/	/	/	4 Bars	Monolithic
G1	4P10A16W1-5	4	10	16	5 Bars	One face Welded
	6P10A16W1-5	6	10	16	5 Bars	One Face Welded
G2	4P10A16S-4	4	10	16	4Bars	Stiffeners
	4P10A18S-4	4	10	18	4Bars	Stiffeners
	4P15A16S-4	4	15	16	4Bars	Stiffeners

2.4.2 Test Procedure

The process involved repeated lateral load tests that started from a displacement of 2mm and proceeded until failure with three cycles per displacement value. The displacement was increased by 2 mm at each subsequent step, so that the second displacement was (4 mm, 6mm, 8mm failure). An Excel-based program ran automatically on the computer to control the process. The nominal axial load strength (pn) and the maximum factored axial load resistance (Ø pn,max) for tied columns were calculated in accordance with ACI 318-19, The value of the axial load used was 40% of the design load, which was equivalent to 80 kN and used in all specimens

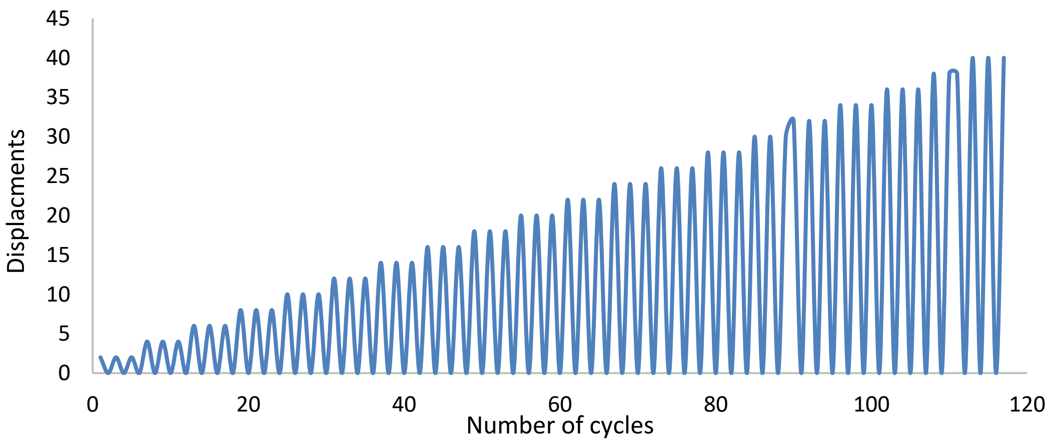
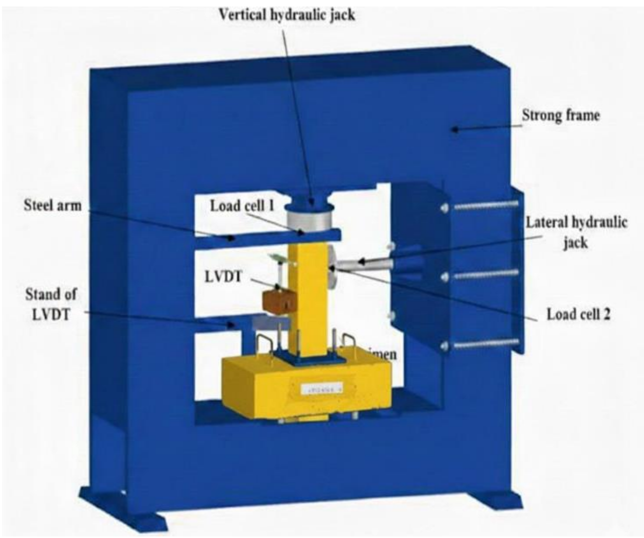


Fig. 10. Displacement history of all specimens



(a)

(b)

Fig. 11 . Test Setup for all Specimens (a) testing machine and (b) specimens in test

The effect of the connection type and other factors was experimentally investigated in Civil Engineering Laboratory in Kerbala University. To study the effect of each column subjected to axial and lateral force, two hydraulic jacks with two load cells were used for load application, also an LVDT was used to measure the applied lateral displacement, as shown in Fig. 11.

3. Results and Discussion

3.1 Load-Displacement Hysteresis Behavior

From the load-displacement response (Fig 12), the initial three loading cycles at each load level showed quasi-elastic behavior with significant displacement recoveries, meeting the seismic design codes such as ACI 318 and Eurocode 8 [16, 17]. As the load levels increased, the hysteresis loops became wider (Fig 12), signifying better energy dissipation and the occurrence of inelastic deformations, a characteristic of ductile reinforced concrete under lateral repeated loading [15, 18, 19]. The similarity in the shape of the hysteresis loops (Fig 12) with no sudden reduction in the load-carrying capacity confirmed the stable cyclic response of the system, as per the performance objectives of ACI 374.1 [19]. The three-cycle loading response was able to capture the cumulative damage and cyclic degradation of the system. The absence of significant reduction in the load-carrying capacity of the system over the cycles indicated good resistance to repeated loading, while the steady increase in the residual displacement indicated concrete cracking and yielding of the steel reinforcement, a positive response that met the code requirements for seismic resilience in reinforced concrete. [16, 19].

A Load reduction was recorded for some specimens, this reduction was -54.6% for 4P10A16W1-5 (see Fig 14), -36.8% for 6P10A16W1-5(see Fig 14), -38.9% for 4P10A16S-4(see Fig 15), -11.6% for 4P10A18S-4(see Fig 16), while an increment of 7.9% for 4P15A16S-4 was recorded and noted (See Fig 17) These results were consistent with those reported in ASTM International. The results indicated a decrease in the structure's load-bearing capacity compared to the control specimen. However, this reduction should be evaluated in light of the behavioral changes triggered by the advanced connection configuration, specifically the variations in base plate thickness and anchor bolt diameters. There is a considerable alteration in the load transfer mechanism at the joint between the column and the foundation due to this alteration. In the reference specimen, the load transfer had been mainly influenced by concrete behavior as it was more flexible in nature with high energy absorption capacity. Hence, it was able to undergo large deformation as well as withstand higher loads before reaching its failure stage, which was similar in behavior of reinforced concrete structures studied by Park and Paulay [20]. Unlike the monolithic connection the steel connection (base plates and anchor bolts) exhibited a lower bearing load capacity.

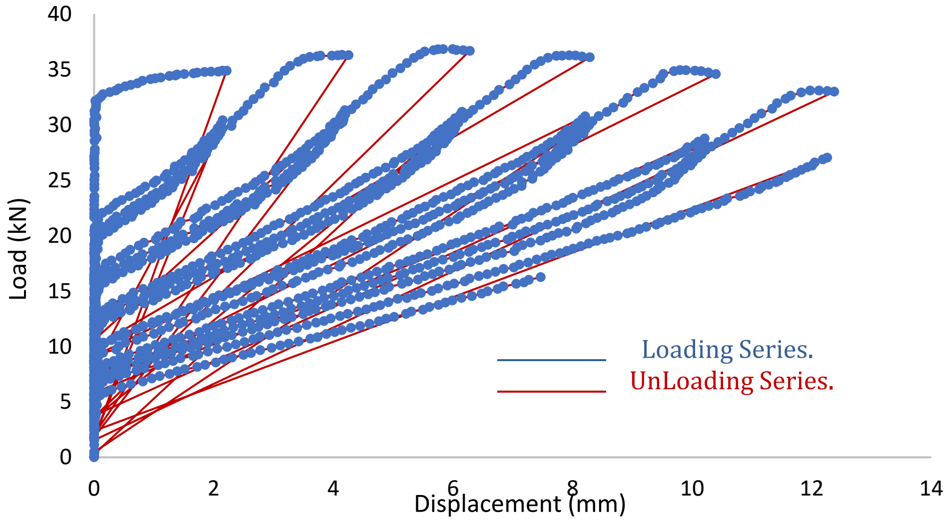


Fig. 12. Load - displacement curve of CF1 specimen

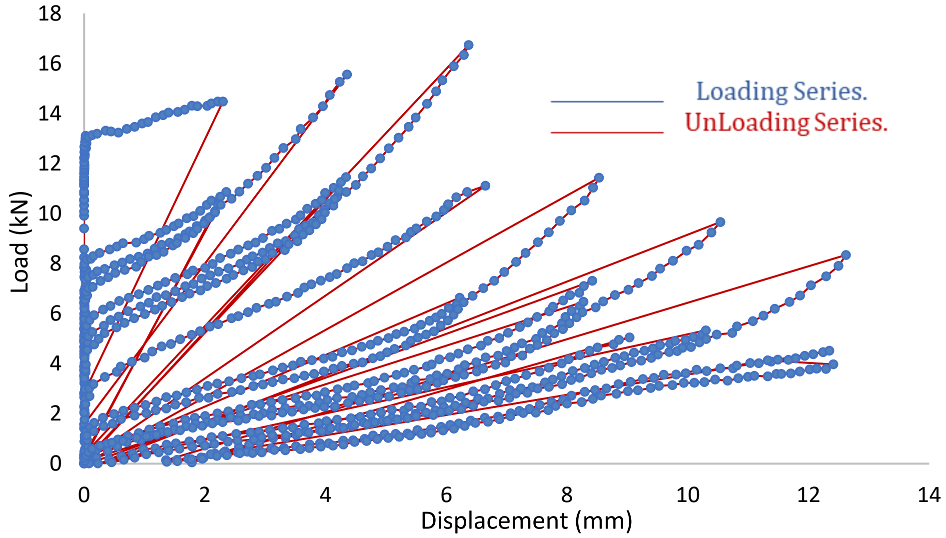


Fig. 13. Load -displacement curve (4 P10 A16 W1 - 5) specimen

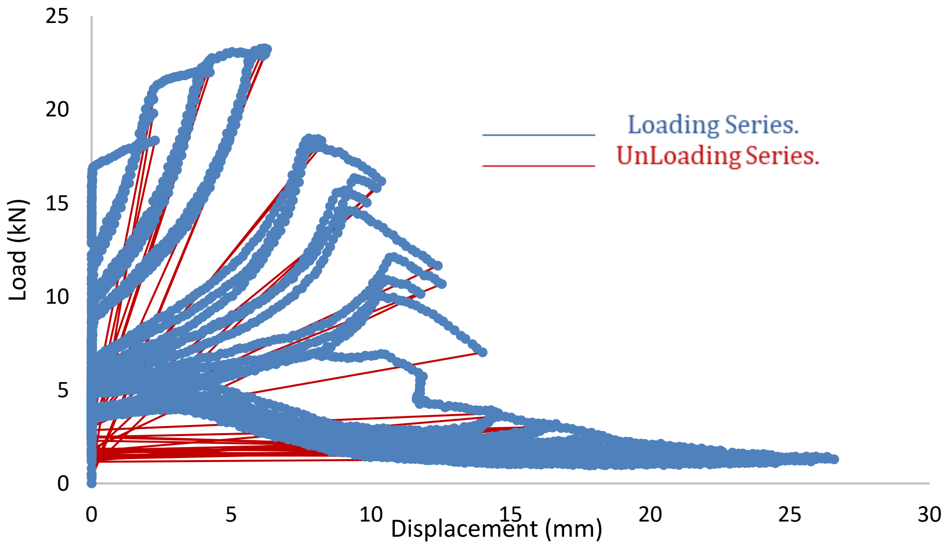


Fig. 14. Load -displacement curve (6 P10 A16 W1 -5) specimen

Thus, stresses were concentrated on the connection zone and reached the ultimate load sooner than they would have if monolithic concrete were used only. However, this connection exhibited a higher degree of deformability if compared to reference specimen. Grauvilardell et al findings support this behavior; his research points out that base plate and anchor details affect the distribution of stresses in steel/concrete connections as well as the mechanism of forces transferring. The ultimate load of strengthened specimens was less than before strengthening, but the ductility of the strengthened specimens increased as well as the continual cyclic performance [21].

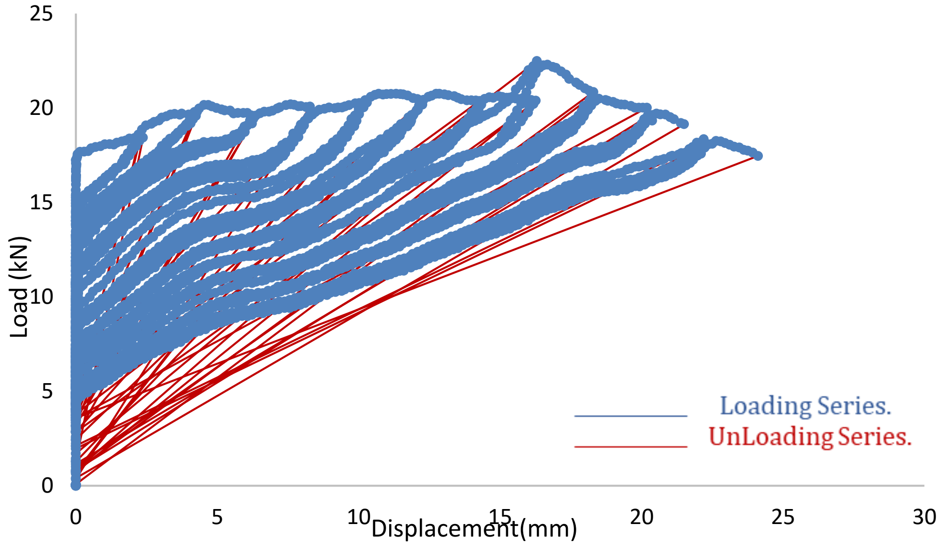


Fig .15. Load - displacement curve (4 P10 A16 S- 4) specimen

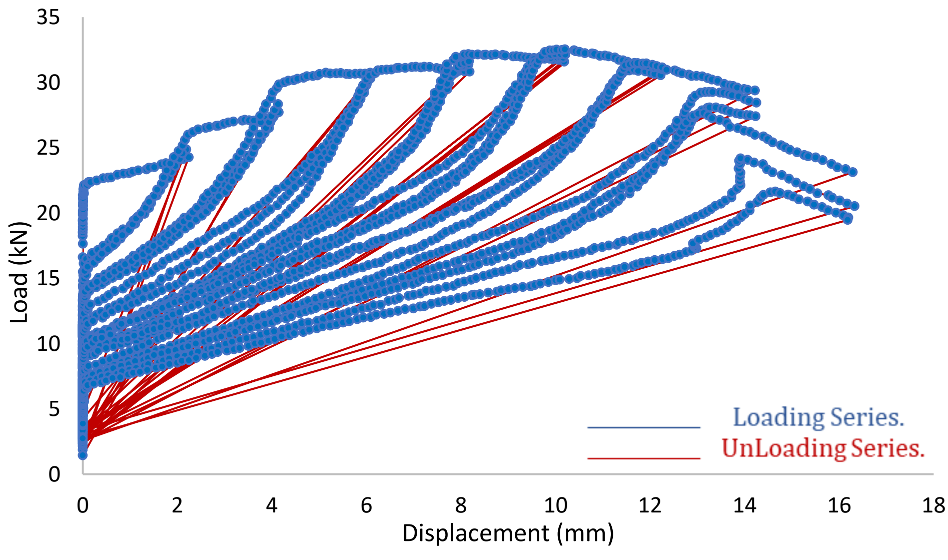


Fig. 16. Load - displacement curve (4 P10 A18 S - 4) specimen

This was due to the enhanced interaction of the steel and concrete as well as the confinement effect associated with the connection system, which helped to prevent significant cracking and prolong severe degradation. Calvi et al also found similar results in his work; he stated that improvements in seismic performance were often related to improvements in ductility and energy dissipation as opposed to simply providing higher ultimate strengths. Variations in the diameter of the anchor bolt and thickness of the base plate caused varying levels of confinement and methods of force transfer in the area where the connection is made. As a result, alterations in deformation properties and stress distributions occurred in each of the specimens [22]. The overall reduction of the deformability of the connection area dominated by steel explains why stress levels increased but afforded a stable and more predictable cyclic response, corroborating with the general research findings as identified in previous studies regarding composite construction between steel and concrete. Thus, the transition in the structural response from an entire concrete-based system to a whole steel and concrete connection system governed by base plates and anchor bolts produces improved ductility and increased cyclic stability, which is widely regarded as a desirable attribute when considering the seismic-resistance of a structure.

Furthermore, the reference specimen recorded a load retention of 46.6% (Fig 12), which was an indication of a significant degradation in load retention capacity These results were consistent with those reported in ASTM International, 2014 [23]. By contrast, the load retention ratios for the strengthened specimens were higher, confirming that the improved bonding systems were beneficial for load retention capacity These results were in agreement with those reported in (ASTM International, 2011a, European Committee for Standardization, 2004b [24, 25]. The load retention capacity of specimen 4P10A16W1-5 was recorded at 76.8% (See Fig 13), which was an indication of a significant improvement over the reference specimen. This was due to its ability to resist cumulative damage These findings were consistent with what was reported in American Concrete Institute (ACI), 2017, American Concrete Institute (ACI), 2019, ASTM International, 2014 [18, 19, 23]. The load retention capacity for specimen 4P10A16S-4 was the highest, recording a load retention capacity of 98.9% (Fig 15), which was an indication that its load retention capacity remained almost intact throughout the loading process. This indication that the specimen was more effective in load retention than the other specimens. The load retention capacity for specimen 4P10A18S-4 was also very high, recording a load retention capacity of 88.2% (Fig 16), which was an indication that it was more effective in load retention than the other specimens. By contrast , specimen 4P15A16S-4 recorded a moderate improvement, recording a load retention capacity of 57.9% (Fig 17), while specimen 6P10A16W1-5 recorded a load retention capacity of 54.6% (Fig 14), which was an indication that it was more effective than the reference specimen but less effective than the other strengthened specimens These results were consistent with those reported in(ASTM International, 2011a, ASTM International, 2014, European Committee for Standardization, 2004b [23-25]. The results indicate that the effectiveness of the improved bonding systems was dependent on the strengthening configurations [25].

The reference specimen experienced a stiffness degradation rate of 61%, reflected the level of degradation that occurred These results were consistent with those reported in ASTM International, 2014 [23]. Conversely, all the strengthened specimens experienced lower degradation rates, reflecting the effectiveness of the bond These results were in agreement with those reported in ASTM International, 2011a, European Committee for Standardization, 2004b [24, 25]. For instance, the specimen 4P10A16W1-5 experienced a degradation rate of 58% (Fig13), while the specimen 6P10A16W1-5 experienced a lower degradation rate of 53% (Fig 14). This showed that the specimens experienced less degradation compared to the reference specimen. The specimens 4P10A16S-4 and 4P10A18S-4 experienced degradation rates of 56% and 52% (See Fig 15, Fig 16), respectively. This showed that the specimens experienced less degradation compared to the reference specimen. The specimen 4P15A16S-4 experienced a moderate level of improvement by experiencing a degradation rate of 54% (Fig 17). Based on the results, all the strengthened specimens experienced decelerated degradation. The specimens that experienced the best performance experienced a reduction of up to 9% compared to the reference specimen. The differences that occurred among the strengthened specimens could be attributed to the bonding arrangements These results were consistent with those reported in European Committee for Standardization, 2004b [25].

Stability under repeated loading was examined as one of the parameters through which the ability of each specimen to withstand successive cycles without significant reduction in its load-carrying capacity was examined and quantified These results were consistent with those reported in ASTM International, 2014, European Committee for Standardization, 2004a. The reference specimen took the lead in exhibiting the lowest stability by a load stability ratio of 0.47(Fig 12), which was indicative of a very large loss in load when subjected to the repeated cycles. By contrast, all the strengthened specimens in the study showed a higher load stability, which pointed to the great influence that the strengthening of the bonding configurations through the effectiveness of the improved relationships had on the performance of the structure being preserved These results were consistent with those reported in ASTM International, 2011a, ASTM International, 2014, European Committee for Standardization, 2004b [23-25]. Load stability of 0.77 was achieved by specimen 4P10A16W1-5(Fig 13), while 4P10A16S-4 attained the highest stability of 0.99(Fig 15), which meant the specimen kept its load capacity throughout the cycles almost entirely. Furthermore, specimen 4P10A18S-4 exhibited a load stability of 0.88(Fig 16), which was pretty good, whereas the load stability of specimens 4P15A16S-4 and 6P10A16W1-5 were 0.58(Fig 17) and 0.55(Fig 14), respectively, which were moderate improvements [23, 26]. The results obtained confirmed that the strengthened specimens were able to maintain a higher load level even after the repeated cycles, and the effectiveness of the stabilization depended significantly on the strengthening configuration These results were in agreement with those reported in European Committee for Standardization, 2004b [25].

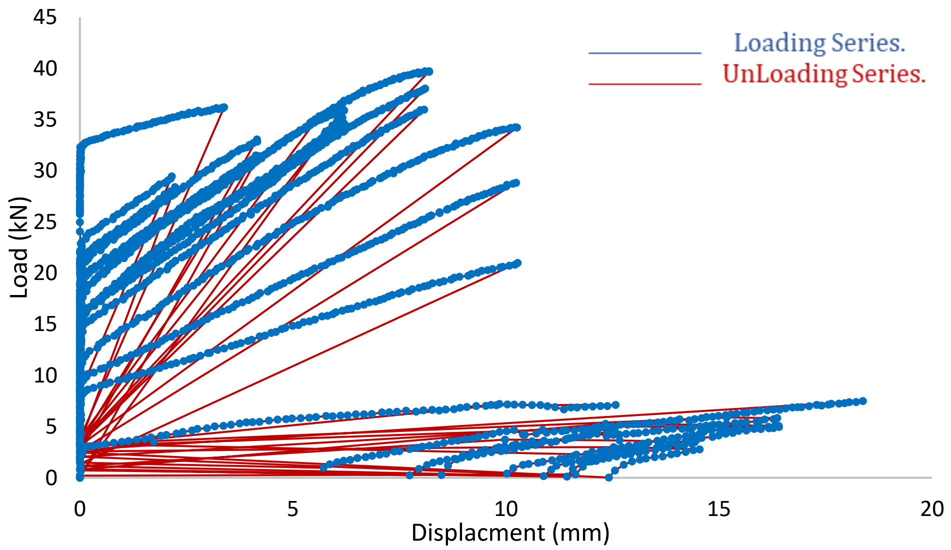


Fig. 17. Load - displacement curve (4 P15 A16 S - 4) specimen

Table 7. Comparison of Load Capacity, Stability, and Displacement Improvement for All Specimens

Label	Load Improvement Ratio	Load Retention	Stiffness Degradation Rate	Load Stability with Cycles	Maximum Displacement Improvement
CF1		46.60%	61%	0.39	
4P10A16W1-5	-54.60%	76.80%	58%	0.78	2.70%
6P10A16W1-5	-36.80%	54.60%	53%	0.55	3.78%
4P10A16S-4	-38.90%	98.90%	56%	0.99	1.18%
4P10A18 S -4	-11.60%	88.20%	52%	0.88	10.80%
4P15A16S-4	7.90%	57.90%	54%	0.58	21.03%

The reference specimen CF1 structural performance assessment utilized maximum displacement analysis to compared its results against the various improved test specimens (Fig 12) [16, 27]. The results demonstrated that different design alterations which designers tested showed distinct effectiveness to decrease displacement between them. The system improvement reached 21.03% according to specimen 4P15A16S-4 (Fig 17) because the combination of S-type connection with 15 mm plate thickness and bigger anchor bolt diameter resulted in better system stiffness and lower deformation [25, 27]and (4P10A18 S -4) specimen was 10.8% (Fig 16). The critical role of plate thickness and increased bolt diameter in improving displacement resistance was examined. The W1-type connection and increased bolt quantity both contributed to displacement reduction according to specimens 6P10A16W1-5 (Fig 14) and 4P10A16W1-5(Fig 13), which showed moderate improvement ratios of 3.78% and 2.70%, respectively. The original plate thickness maintenance together with connection type alteration produced a minor effect on displacement control according to specimen 4P10A16S-4(Fig 15), which achieved 1.18% as its lowest improvement ratio [27, 28]. The research results showed that design parameters such as plate thickness and anchor bolt diameter and connection type development all influenced maximum displacement response, while dual enhancements of bolt diameter and proper connection configuration worked to boost structural stiffness and system performance compared to the reference specimen [25, 27, 28].

3.2 Failure Mode

In the initial phases of repeated loading, the concrete column with the base showed stable structural behavior (Fig 20, Fig 23). A discernible decrease in lateral buckling and initial displacements demonstrated how well the developed confinement improved the stability of the column. However, a gradual nonlinear response developed as repeated lateral loads were applied; this was ascribed to cumulative damage in the reinforcing steel or concrete, especially in the vicinity of the column-to-base connection area (Fig 19, Fig 24). This area was important because it complied with seismic design specifications, highlighting its importance for structural integrity during seismic activity. The mode of failure was characterized by a gradual and non-brittle failure, with wide cracks at the base (Fig 19, Fig 24). Partial crushing of concrete (Fig 19, Fig 24), and steel (of steel (Fig 20). This mode provided clear warnings before failure, and this was a characteristic of a ductile failure with sufficient dissipation of energy, which was consistent with modern seismic design codes. It was acknowledged that the use of a single Linear Variable Differential Transformer (LVDT) posed certain limitations in the possibility of fully capturing the kinematic behavior of the column base connection, especially where localized deformations are more complicated and the level of detail is higher. In the same way, the lack of localized strain gauges was a reason why the detailed strain distributions inside the critical zones of the connection were not measured directly. One way of overcoming these instrumentation shortcomings was to complement the testing program with careful and continuous visual monitoring during each loading cycle. Early signs of local distress, such as the initiation and growth of cracks (Fig 19, Fig 21, Fig 24), steel yielding's (Fig 20), and potential buckling, were identified by means of a regular inspection of structural elements. This method of observation was a process of confirming in practice that there were no premature local failures that could have been hidden and might have compromised the global response recorded. When combined, the displacement data gathered by the LVDT and the visual inspections were considered sufficient to support the certainty of the findings from this experimental investigation.

The integrity of the welds was tested systematically at each stage of the test process, as well as through the extensive post-test analysis. It helped draw the conclusion about the flawless condition of the connections up to the last stage of the test process without fractures, separation or any deterioration of the zone of contact between the longitudinal reinforcement of the column and the steel base plate. Cracks or any sign of deterioration were not found in the area of the weld, which means that there was no occurrence of structural failure or any other critical failure mechanism in the weld under lateral repeated loading. No crackling sounds or any other signs of the weld breakage during the experiment process have been recorded. Based on the findings made as a result of the analysis conducted after the test, one can conclude that the reasons for the appearance of failure or damage lie in the field of structural behavior and not in metallurgical failures, thus the failure was induced intentionally in accordance with the structural design of the element. In the case of the W1 connection, the concrete was stripped off in the column to base plate region to facilitate a more precise assessment of the connection performance based on the load transfer through the steel base plate and anchor bolts. The W1 connection had relatively lower performance compared to the second connection type, implying that the mechanism of load transfer through the connection region was relatively inefficient in the case of the former. However, all the welds performed well during the entire loading regime without any failure or damage occurring. Therefore, the relatively poor performance of the connections could be attributed to the load transfer mechanism of the connection rather than poor welding quality Fig 18.

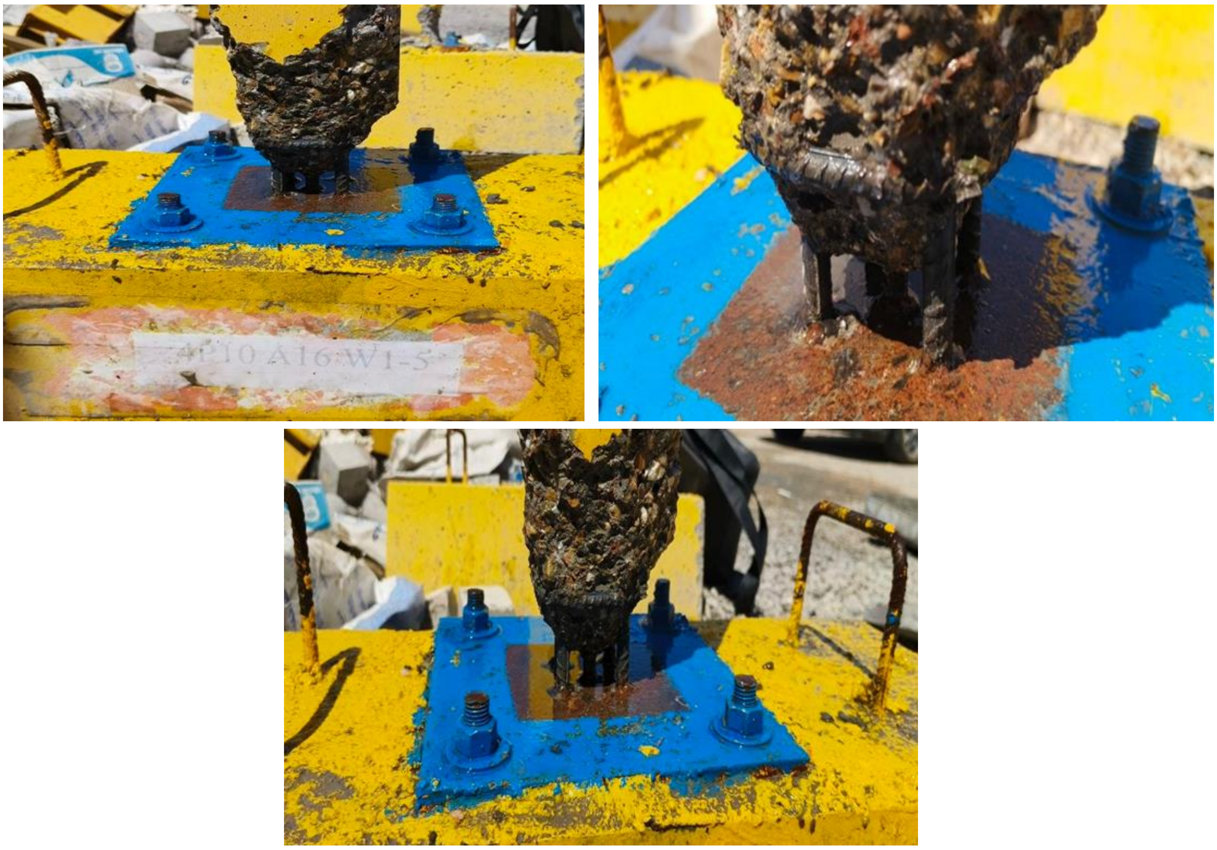


Fig. 18. Specimen (4P10A16W1-5) after removal of concrete from the contact zone between the concrete and the base plate

Fig. 19. Failure Mode of CF1 Specimen

Fig. 20. Failure mode of (4 P10 A16 W1 -5) specimen

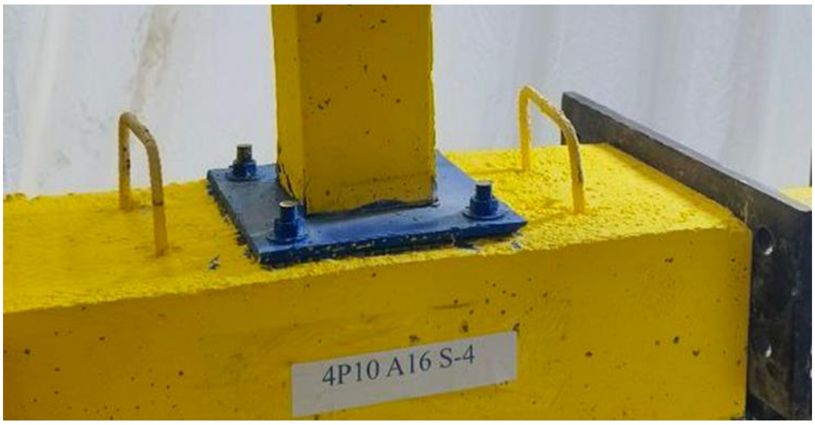


Fig. 21. Failure mode of (4 P10 A16 S -4) specimen

Fig. 22. Failure mode of (4 P10 A18 S -4) specimen

Fig. 23. Failure mode of (4 P15 A16 S -4) specimen

Fig. 24. Failure mode of (6 P10 A16 W1 -5) specimen

4. Conclusion

Six concrete column-foundation specimens were subjected to repeated loading, and their performance was assessed based on peak load capacity, retained load capacity, stiffness degradation, load stability, and maximum displacement. Although the reference specimen demonstrated a higher peak load capacity, it failed quickly and lost significant load-carrying capacity under repeated loading. Most of the strengthened specimens demonstrated a reduction in peak load capacity. This indicated that the specimens had lower peak loads than the reference one,; instead, the bond between the column and foundation had been enhanced to improve the efficiency of load transfer.

The strengthened specimens demonstrated greater load stability compared to the reference, with significantly less cumulative damage during the loading process. The retained load capacity of the strengthened specimens increased significantly, from 46.6% in reference to between 54.6% and 98.9% in the strengthened specimens, with 4P10A16S-4 retaining its load capacity. The rate of stiffness degradation was lower in the strengthened specimens, indicating a higher resistance to progressive deformation and damage under repeated loading. The load stability of the strengthened specimens also increased significantly, with values ranging from 0.47 in reference to a maximum of 0.99, indicating that the specimens maintained their load-carrying capacity better under repeated loading cycles.

An examination of the peak displacement values of the various specimens revealed that there were three significant factors that affected the degree of resistance to deformation of the column base joint and the stiffness of the connection. The first was the thickening of the base plate. The second was the use of larger diameter anchor bolts. The third was the use of the S-type connection, which had the most significant effect. Of all the specimens, 4P15A16S-4 had the highest resistance to deformation, with the greatest improvement in displacement at 21.03%. The use of the W1-type connection in the specimens had a moderate improvement in stiffness. The use of more bolts to connect the column to the base had a moderate improvement. The use of the connection type had a moderate reduction in the peak displacement value. The improvement in the connection between the column and the base, making the connection stiffer and stronger, had a positive effect on the stiffness, degradation, and load-carrying capacity, resulting in a lower value of maximum displacement.

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