Numerical investigation of Cover Plate in RCS connections

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Abstract

Due to the growing popularity of Reinforced Concrete column with Steel beam (RCS) moment frame system in recent years, there are lots of publications about the performance of this structural system. In this paper, fifteen RCS joints with practical details are studied using a verified finite element method. Joint details of the models include Cover Plate, Additional Bearing Plate, Steel Doubler Plate, and joint stirrups. The results show that Cover Plate can be used instead of a combination of steel doubler plate and joint stirrups; it improves the joint performance by increasing the confinement of joint region and contributing to joint shear strength.

Keywords: RCS connections; composite structural frames; Finite element method; Cover plate; Steel doubler plate
1. Introduction

Since the late 70’s, composite structural frames consist of steel and reinforced concrete have gained popularity in building construction. One type of these structural systems is called RCS moment frame (Reinforced Concrete column Steel beam moment frame) [1]. The RCS moment frames became common in United States and Japan in the late 1970’s and early 1980’s. Replacing the heavy wide-flange columns of a typical steel moment frame structure with the most cost effective reinforced concrete columns to resist the high axial compressive loads, brings about an economic advantage to the RCS system. On the other hand, there are many other advantages in using this system; for instance, concrete columns are more fire resistant, the energy dissipation capacity of steel beams is higher than concrete beams and the stiffness of concrete columns are greater than steel columns.

Several experimental programs have already been conducted for studying the performance of RCS connections. Bugeja tested five interior and one exterior beam–column–slab subassemblies subjected to cyclic loading in two principal directions at Texas A&M University [2]. The specimens consisted of steel beams passing through the reinforced concrete columns in the two orthogonal directions. The joint details included Face Bearing Plates (FBPs), erection steel columns, steel Cover Plates (CPs), and Steel Band Plates (SBPs). The composite beams typically composed of steel beams that mechanically connected to the RC slab using shear studs. The results indicated that the composite RCS beam–column–slab subassembly has excellent inelastic behavior and energy dissipation capacity under cyclic loading.

The problem of a partial interaction in steel-concrete composite beams connected by flexible stud shear connectors was discussed by Machelski and Toczkiewicz [3]. In this research, the problem was formulated according to the modified Trost’s theory of concrete ageing and the effects of the parameter characterizing flexibility of the connection and the authors’ own
coefficients on the distribution of internal forces was shown, but the phenomenon was not completely formulated and described.

Eghbali and Mirghaderi provide a beam-to-column connection with enhanced performance [4]. In the current paper, two interior connections at 3/4 scale were evaluated experimentally under cyclic lateral loading and a constant axial load on the column. In these specimens, the beams were connected to a vertical plate passing through the concrete column (Through Plate). Steel cover plates remove any separation potential of the rigid shear connectors from the concrete while increasing the concrete strength. The through plate involved with concrete provided a strong panel zone with elastic behavior, and the proposed connection was a fully restrained connection and also the tested specimens provided permanent hysteretic diagrams without any pinching.

Nguyen et al. studied on seismic performance of a new type of exterior RCS connection, in which a steel profile embedded inside RC column was directly welded to the steel beam [5]. A full-scale exterior hybrid joint was built and tested under reversed-cyclic and without compressive axial force. The test results indicated that after yielding the strength of the specimen still increased approximately 20%. The test specimen performed in a ductile manner with a ductility factor $\mu = 2.2$. This leads to the conclusion that the RCS joint may be used as dissipative element in the structures of ductility class medium (DCM).

Mirghaderi et al. suggest a new moment that two parallel beams pass from both sides of the column and were welded to the cover plates surrounding the concrete column in the joint area. This detail provides some advantages compared with previous constructions. One of them is that both the beam and column are continuous in the joint area, which provides more reliable performance. The proposed connection was studied in two experimental tests under cyclic loading. The test results indicated that both specimens sustained 8% story drift with stable hysteretic loops and that the suggested connection is acceptable as a special moment
connection. In addition, the test results demonstrated that the proposed design relationships were arranged properly such that the cover plates were maintained in the elastic phase, only slight cracks appeared in the column, and plastic hinges were formed in the beams in the vicinity of the column.

Hongtuo Qi et al. [7] conducted experimental and analytical studies on the behavior of tubed SRC stub columns subjected to axial compressive load. In this paper twenty five tubed SRC stub columns were tested to investigate the failure mode and axial loaded behavior of tubed SRC columns. The results indicated that tubed SRC stub columns showed higher axial load capacity than usual SRC columns with the same volumetric steel ratio. Some Equations were also proposed to predict the axial load strength of tubed SRC stub columns based on the experimental results.

A series of test was performed on some CFDST columns with external steel rings by J.C.M. Ho and C.X. Dong [8]. The results exhibited that the elastic strength, elastic stiffness and ductility were increased by utilizing the steel rings as external confinement. In addition, a theoretical model was proposed to predict the axial strength of confined CFDST columns.

T.Wroblewski et al. [9] present estimations of shearing and axial stiffness of connecting elements, and substitute longitudinal modulus of elasticity of the reinforced concrete slab based on the results of experimental research. The analyses demonstrated that increasing the shearing stiffness of connection increased all the analyzed frequencies of flexural vibrations.

Cai et al. [10] numerically investigated the mechanical behaviors and failure mechanism of steel-reinforced concrete-filled steel tubular (SRCFST) to examine the effect of steel tube ratio( $t_α$), section steel ratio ( $s_α$) and etc. on the mechanical behaviors and ultimate resistance of the SRCFST columns under axial compression. The calculation results illustrated that the peak strength and initial stiffness of CFST columns increases with the increase of all parameters.
Alizadeh et al. [11] investigated the cyclic behavior of RCS connections. In their paper, two interior connections were tested under reversed cyclic loading. One of the specimens had a new proposed joint detail that consisted of additional bearing plates. Comparing the performance of two specimens showed that using additional bearing plates, increase bearing and shear strength of the joint. Furthermore, a modified method for modeling this type of connections was introduced using Open Sees software.

In another research conducted by Alizadeh et al. [12] some suggested joint details were simulated using verified FEM model to investigate the performance of steel band plates, FBPs, Wide Face Bearing Plates (WFBPs), ABPs and Steel Doubler Plates (SDPs). The results indicated that the performance of models directly depends on joint detailing, effectiveness of shear keys, and the amount of confinement provided for the joint region.

Some other research programs of RCS connections were conducted by other researchers such as Nguyen et al. [13], Men et al. [14], Chou and Chen [15], Shen [16], Zhang et al. [17], Li et al. [18], and Farahmand Azar [19]. Li et al. [20] reviewed some important researches in this field.

There are many researches on the performance of RCS connections with different joint details; however, a few of them focused on the behavior of the CPs in RCS joints. So, the performance of CPs and its effects on the joint shear stiffness and strength is not well known. The aim of this study is to clarify the influence of SDPs and CPs on the performance of RCS connections. In this research program, a complete FEM study is conducted to investigate the performance of different types of joint details in combination with CP. The connections are simulated using ABAQUS [21] software and are verified with the experimental results of Alizadeh et al. [12].

2. Numerical Modeling and Verification
The numerical models of this study are simulated using the assumption of a model, which is verified with the results of Alizadeh et al. experimental investigation. A brief description of the experimental test of Alizadeh et al. is presented. Experimental test set up is shown in Fig. 1. As it can be seen in this Figure, at the ends of the steel beams the roller supports were used. Bottom end of the concrete column was pinned to strong floor. Two hydraulic jacks were used at the top of the column to impose reversed lateral cyclic loading. To simulate axial load of column, 300 kN axial load was applied to the concrete column at the beginning of the test, which was about 4% of the column's gross axial strength. The ends of beams and columns were braced laterally to prevent out of plane movements during the test.

Fig. 1. Test setup

The loading pattern was consisted of 28 cycles started from 0.2% drift angle and maintaining 0.25%, 0.375%, 0.5%, 0.75%, 1%, 1.5%, 2%, 3%, 4%, 5% and 6% drift angles and were repeated each cycle two times.

2.1. Material Properties

The tested specimens are modeled with considering all of the interactions, boundary conditions, and materials properties. Based on the experimental reports, the ASTM A615 grade-75 [22] was utilized for longitudinal reinforcements and ASTM A615 grade-60 [22] was used for all transverse reinforcements. The steel beam materials were ASTM A572 grade-50 [23]. Based on test results, the yield stress and ultimate strength of steel beam were 362.7 MPa and 495 MPa respectively. The material properties that are used for numerical simulation are shown in Table 1, Fig.2 For reducing the computational costs, half of the models were simulated, using the symmetrical situation at the center of the column. Since the end parts of the columns and beams did not experience significant nonlinear deformations, these regions are modeled using one-dimensional beam elements that can be seen in Fig. 3.

Fig. 2. Material properties
Table 1. Material properties

The mean compressive strength of concrete was 50.8 MPa that is modeled using Concrete Damaged Plasticity model, existing in ABAQUS library. This model is a continuum plasticity-based damage model for concrete and assumes that the two main failure mechanisms are tensile cracking and compressive crushing of the concrete material. Parameters, which are needed in this material model, are obtained from CEB-FIP model code 90 [24], based on the concrete compressive strength. The compressive and tensile damage parameters are considered as a linear function of inelastic strains. The nonlinear behavior of steel beams and reinforcement bars are simulated using an isotropic hardening model based on the von Mises yield criterion. The stress-strain relationship of the steel beams are defined according to the results of uniaxial tension tests.

Fig. 3. FEM model

2.2. Model Specifications

The 8–node solid elements, which are known as C3D8R elements in ABAQUS [21] software, are utilized for modeling steel beams and concrete columns. The reinforcements are modeled using one-dimensional two nodes truss elements (T3D2) and are fully embedded in concrete. These assumptions used for simplifying the finite element models. Separation of steel beam and concrete column at the joint region was allowed during the analysis for better simulation of interaction between the steel and concrete. The models are analyzed in two steps, at first, the axial force of column is applied, and then the column is pushed laterally up to 4% story drift.

The lateral load-story drift response of the verified model is indicated in Fig. 4. As it is shown, According to Fig. 4, the simulated model shows very good agreement with the backbone curve of the tested specimen. The backbone curve is plotted according to ASCE41-
13. Furthermore, as can be seen in Fig. 5, the overall crack pattern of FEM model is very similar to the experimental results. It should be noted that, since the numerical model is pushed monastically, the concrete cracks are formed at one side of the column.

Fig. 4. Test and FEM lateral load-story drift response

Fig. 5. Test and FEM crack pattern

3. Case Study

3.1. Simulated Models

Fifteen interior beam-through type RCS connections are investigated numerically in this paper. Various joint details are simulated using the verified model to investigate the effects of CP, ABPs, and steel doubler plates in RCS connections behavior. Simulated models are grouped into three main categories to ensure that all the possible failure modes are captured. Each of these categories consists of models with and without CP, SDP.

All of the models consist of 3000 mm-long concrete column, with 400x400 mm² square cross section. Columns are reinforced with sixteen Φ18 steel bars. Φ10 bars are used for joint and column stirrups. IPE 300 steel sections with 3900 mm length are considered for the beams. In six models, the thickness of the flanges of the steel beams are increased to 20 mm, for imposing larger forces to the panel zone. These models are specified by adding “(s)” after the model name. In four models, the flanges of steel beams are increased to 25 mm and the diameter of column’s longitudinal reinforcements are increased to 22 mm (Φ22) to closing up the flexural capacity of steel beams and columns. These models are named by adding “(ss)” after the model name.

In the joint region of different models, L-shaped stirrups, steel doubler plates, steel band plates and CP are used according to Table 2. Fig. 6 demonstrates typical joint details.
Model F-B-L-S is the same as the specimen 1 of Alizadeh et al. tests [12]. Model F-B-L-S (s) is similar to model F-B-L-S, while the flange thickness of the steel beam is increased to 20 mm to raise the beam flexural capacity. The flange thickness of steel beam in model L-S (ss) is increased to 25 mm and the longitudinal reinforcements are changed with Φ22 steel bars.

The performance of models are compared in terms of the lateral load-story drift response, joint shear forces, joint shear strain, bearing stress, and cracking pattern of concrete columns.

Fig. 6. Joint details of models

Table 2. Joints details

3.3. Lateral load-story drift response of simulated models

3.3.1. Investigating the performance of CP and steel doubler plate

The lateral load-story drift responses of two simulated models are indicated in Fig. 7a to compare the performance of CP and steel doubler plate. As it is shown in this Figure, the responses of models F-B-L-S and C are very similar. This indicates that using CP instead of the combination of SDP, FBP, Band Plate, and L-shape joint stirrups result in the same performance.

Fig. 7.b shows that the load-story drift response of model C (s) perform better than models F-L-S (s) and L-S (s), but it is just below than model F-B-L-S (s). The comparison between these models indicate that using CP instead of the combination of SDP and joint stirrups or the combination of SDP, FBP and joint stirrups improves the performance of connection. However, using SDP, joint stirrups with both SBP and FBP in model F-B-L-S (s), make its performance mostly the same as model C (s). From constructability point of view, model C(s) is more practical in this group, which makes it the best model of this group with the similar performance.
As it can be seen in Fig. 7.c, the load-story drift response of model C (ss) is higher than models L-S (ss), F- L-S (ss), F-B-L-S (ss). This shows that using CP instead of the combination of SDP and joint stirrups (comparison of models C (ss), L-S (ss)) or the combination of SDP and joint stirrups (comparison of models C (ss), F- L-S (ss)) or even the combination of SDP, joint stirrups, SBP and FBP (comparison of models C (ss), F-B- L-S (ss)) improves the performance of the joint. As mentioned before, using CP is more practical than each of these combinations.

Fig. 7. Lateral load-story drift response of simulated models

Overall, in all models with different types of details, models with CP have higher performance in terms of lateral load-story drift response.

3.3.2. Investigating the effect of ABPs in models with CP

Fig. 8 shows the performance of model C-A is slightly higher than model C, but in models with stronger steel beams (imposing higher forces to the joint region, models C (s), C-A (s)), the differences between model with ABP and model without this part is more remarkable. As can be seen, not only the ultimate capacity of model C-A (s) is higher than model C (s), but also the stiffness of model C-A (s) is higher than model C (s).

3.4. Comparing internal joint shear forces

In the simulated models, the joint shear strength is investigated by comparing the internal shear forces of joint shear mechanisms. The internal shear forces are captured from the middle of inner concrete panel, outer concrete panel, steel beam web panel, and cover plate. Inner concrete panel is the concrete region between two flanges of steel beam, and outer concrete panel is located outside the steel flange width. Table 3 shows the contribution percentage of internal shear forces of joint shear mechanisms of all the simulated models.
Table 3. Contribution percentage of internal shear forces of joint shear mechanisms

3.4.1. Investigating the performance of CP and steel doubler plate

The results of Table 3 indicated that using CP instead of the combination of Steel Doubler Plate and joint stirrups increase the participation of inner concrete panel and reduce steel beam web participation therefor the distribution of internal joint shear forces become more effectively.

3.4.2. Investigating the effect of ABPs in models with CP

Based on Table 3, and comparing models C and C-A indicates that using ABP reduces the concrete panel participation, while the steel beam web participation increase. on the other hand, the comparison between models C (s) and C-A (s) shows that using ABP transfers the forces from steel beam web to CP and makes CP more effective, in addition, it cause to reducing the concrete panel participations. The difference between these two groups is the beam capacity and, it can be understood that in beams with higher capacity, ABPs can help to transfer the forces from the steel web to CP.

Fig. 8. Lateral load-story drift response of models C, C (s), C-A, C-A (s)

3.5. Cracking and failure modes

The main failure mechanisms of the models are tensile concrete cracking and steel beam yielding. The tensile damage of columns and Von-Misses stresses of steel beams at 4% story drift are shown in Figs. 9 and 10 respectively.

The cracking pattern at the face of joint region indicates the formation of diagonal compression struts in the concrete panel. Fig. 9a shows the crack pattern of the models F-B-L-S, C, S-C, C-A. As can be seen, the joint region of model F-B-L-S experiences the most damage in terms of tensional cracks due to the insufficiency of concrete confinement at the
joint region, but in model C cracks at the joint region reduces because CP increase the concrete confinement. This reduction is more remarkable in model S-C with CP, SDP. However, differences between crack pattern in models C and S-C are very few and can be negligible.

In addition, Diagonal cracks of model F-B-L-S started at 0.6% story drift and in model C diagonal and flexural cracks can be observed at about 0.65% drift story while flexural crack started to occur at about 0.58% drift story and diagonal cracks started at 0.65% story drift in model S-C. As can be seen, using CP instead of SDP lead the cracks formation occur at higher story drift.

Fig. 9. Tension cracks in models at 4% story drift

In comparison of models C and C-A, it can be seen that Model C-A shows fewer cracks because of using CP and ABP which can increase the concrete confinement and transfer the imposed forces to the CP. Comparison of the performance of models S-C and C-A indicates that using ABP in combination with CP can noticeably improve the concrete confinement and decrease the joint region damages. Overall, it can be concluded that CP in combination with ABP can work better than CP in combination with SDP, and can improve the joint region performance and increase its strength and stiffness.

Fig. 9.b indicates the cracking patterns of second group of models with stronger beams than the first group. Therefore, the cracks in this group are more than the first group. In this group, model C (s) has the fewest crack because of using CP instead of SDP with or without FBP and SDP, which cause better confinement in concrete at joint region. Furthermore, lack of FBPs in 5s cause to weak transfer of force from steel beam to concrete and result in fewer cracks in model L-S (s) in comparison with model F-L-S (s).

Diagonal and flexural cracks of the column of model C (s) happened at 0.62% story drift and in model F-L-S (s) flexural cracks started at 0.6% story drift and diagonal cracks can be
observed at about 0.9% story drift, while flexural crack started to occur at about 0.73% story drift and diagonal cracks started at approximately 1% story drift in model L-S (s). It can be seen that lack of SBP or FBP cause the cracks to occur at higher story drift, because in these models, concrete cannot participate as properly as models with CP (model C (s)) and beam should tolerate more shear force.

The cracks of columns of third group is presented in Fig. 9c at 4% story drift. As it can be seen, the same result of Fig. 9b can be obtained from this Figure, but cracks in this group is a little fewer than second group because of using stronger beams and reinforcement bars in these models. As same as second group, the model with CP (model C (ss)) has the fewest cracks. As can be seen, in the first group of models, the governing failure mode is the beam yielding. In second group of models, the capacity of steel beam is increased by raising the flange thickness to 20mm. In these models, failure is happened in the panel zone because of increasing the imposed forces to the joint. In other models flange thickness raised to 25mm and the diameter of column longitudinal steel bars increased to Φ22. The third group of models are designed such that failure modes concentrated in the panel zone.

Fig. 10 shows the Von-Misses stress contours of beams of models. Based on Fig. 10.a, the steel beam web at the joint region of models without cover plate, experienced a higher level of stresses. On the other hand, the efficiency of cover plates in models C and S-C is lower than model C-A due to lack of using stiffeners for transferring the joint forces to the CP, also, when CP is used in combination with SDP in a model (model S-C) CP did not work effectively. Some joint details like ABP, increases the joint stiffness and cause to decline the joint shear lag and increase participation of cover plate in the joint shear strength and stiffness.
In addition, in model F-B-L-S beam yielding started at about 1% drift and plastic hinge formed in flange of beam at 1.9% story drift. Beam yielding in model C and model S-C started at approximately 0.95% and 0.85% story drift respectively.

Based on Fig. 10a, CP perform better in models with ABP (comparing model C-A with model C) because ABPs help to transfer force from steel beam to CP more efficiently.

As can be seen in Fig. 10b, the performance of model C (s) (with CP) is better than models L-S (s) (with SDP), F-L-S (s) (with SDP, FBP) and F-B-L-S (s) (with SDP, FBP, SBP) because CP transfer forces to concrete properly and because of this distribution, stresses in flanges of model C (s) is lower than other models of this group.

Fig. 10. Stress of beams in models at 4% story drift

In addition, beam yielding in model F-B-L-S (s) started from its flange at 0.55% story drift and steel web of beam started to yield at 1.14% story drift. In model F-L-S (s) yielding of steel web started at 1.08% story drift and yielding of flange started a bit later, at 1.17% story drift. In model L-S (s) flange started to yield at 0.63% story drift and in web it started at 0.83% story drift. However, in model C (s) yielding started simultaneously in web and flange of beam at 0.84% story drift. It can be concluded that models with CP start to yield at higher story drift levels than models with SDP.

In Fig. 10c can be seen that the third group of models shows the same results of second group. Also, the distribution of stresses in models of this group is very similar to second group.

The steel beam in models L-S (ss) started to yield in both flange and web of steel beam simultaneously at 0.76%, while yielding in model C (ss) originated in web at 0.73% story drift and at 0.93% story drifts flange yielding started. Since there is a direct correlation between these values and the joints stiffness, it can be concluded that the joint region of
model with CP instead of SDP (model C (ss)) performed better in terms of strength and stiffness than model with SDP (model L-S (ss)).

3.6. Shear strain

For better understanding the shear behavior of different joint details, the shear strains of joint shear mechanisms are depicted in Figs. 11, 12, and 13. The strains are recorded from the middle of steel beam web, cover plate and the inner and outer concrete panels.

Fig. 11. Shear strain in the joint shear mechanisms of models F-B-L-S, C, and S-C

The shear strains of the inner concrete panel are shown in Fig. 11a. It can be seen that shear strain immediately increased in all of the models at about 0.75% story drift. The shear strain of the inner concrete panel in model S-C is half of model F-B-L-S due to using CP. The shear strain of the inner concrete panel in model C is about 20% lower than model F-B-L-S because of using CP instead of steel doubler plate. But in Fig. 11b and with comparison between models F-B-L-S and C, it can be seen that using CP instead of steel doubler plate causes about 50% increase in shear strains of the outer concrete panel because CP produced a good confinement that cause more participation of outer concrete panel.

Based on Fig. 11c the shear strains in the steel beam web of model C shows the highest value, which is about 0.017, because of lack of steel doubler plate. In comparison the models F-B-L-S and S-C, the effects of using CP lead to about 2.6 times decreasing in steel beam web shear strain of this model. Fig. 11d shows the effects of removing the steel doubler plate in models with CP. Those Figure showed that eliminating doubler plate could help model to use CP more effectively by the comparison of models C and S-C.

The inner concrete panel shear strains in models with stronger beam are shown in Fig. 12a. As can be seen, the shear strains of the inner concrete panel in model C (s) indicate an increase at 0.5% story drift and reach to the peak value of about -0.03 at 4% story drift.
While, this increase in model F-B-L-S (s) happened at 0.75% story drift and reach to the peak value of about -0.041 at 4% story drift. However, in two other models we can see that shear strain of inner concrete is so lower than model C (s). The results show that using CP instead of SDP, (comparison models C (s), L-S (s)) can increases the contribution of outer concrete panel in joint shear strength and stiffness. The same results can be obtained from Fig. 12b.

The recorded shear strains of steel beam web are provided in Fig. 12c. Based on this Figure, the shear strains of models F-B-L-S (s), F-L-S (s), C (s) are nearly the same but model L-S (s) with SDP did not have the same result. Therefore, using CP instead of SDP can has the same result or even it can increase the contribution of steel beam in joint shear strength and stiffness.

As it can be seen in Fig. 13a and Fig. 13b, increasing the capacity of beams and column can increase the shear strain of concrete panel in models with SDP with or without SBP, but in model L-S (ss) with SDP and joint stirrups, the concrete panel did not experience any shear strain.

Fig 12. Shear strain in the joint shear mechanisms in models F-B-L-S (s), F-L-S (s), L-S (s), and C (s)

Fig 13c showed the same shear performance of steel beam for all models, that it should be the result of increasing in capacity of beam. Also, it showed that using each part of CP or SDP did not have any effect on shear performance of these models.

According to the Figs. 11d, Fig. 12d and Fig. 13d, it can be seen that increasing the capacity of beams without increasing the CP thickness increased the shear strain of CPs.

Fig. 13. Shear strain in the joint shear mechanisms in models L-S (ss), F -L-S (ss), F-B-L-S (ss), and C (ss)
Due to specific load path in RCS connections, shear lag occurs between steel beam web and the outer concrete panel. The shear lag effect varies with different joint details. In this research, this shear lag is investigated by defining a path in the middle of the joint, which is shown in Fig. 14. Shear strains are recorded from this path at 3% story drift and are provided in Fig. 15.

As can be seen in Fig. 15a, the shear lag in model C is the least because the CP provides confinement in the joint region, while the model F-B-L-S, shows highest shear lag due to using steel doubler plate without using CP, which makes the beam stronger without improving the confinement of concrete. Overall, using steel doubler plate can decrease the shear strains of steel beam web, and using cover plate, improves the concrete confinement and decline the joint shear lag.

Fig. 14. Selected path for recording joint shear strains

According to Fig. 15b, model C (s) shows the lowest shear lag, and concrete panels and CP contribute effectively to joint shear strength due to using CP instead of SDP. On the other hand, model L-S (s) (model with SDP without CP) showed the lowest shear strain because connection is strengthened using SDP but there is not any confinement provided for concrete panel in this model (Fig 15c).

Fig. 15. Shear strain at the middle of connection

3.7. Bearing stress

The failure mechanisms of beam-through type RCS connections are joint shear failure and joint bearing failure. In the previous sections, the shear behavior of this type of connection is investigated and the aim of this section is to study the bearing performance.

Bearing failure in RCS connection is due to high compression stresses in the concrete adjacent to steel beam. In this section, the bearing behavior of RCS connections is studied by
recording the bearing stresses of concrete. Fig. 16 shows the selected element for capturing the stress.

Fig. 16. Selected element for recording bearing stresses

The recorded bearing stresses are depicted in Fig. 17. According to Fig. 17a, the comparison between models F-B-L-S and C show that using CP instead of steel doubler plate increases the stiffness and reduces the bearing stress. The comparison between models C and S-C shows that using steel doubler plate in the model with CP can reduce the bearing stresses but cannot improve it significantly. In Fig. 17b, it can be seen that in models with stronger beam, the same result can be obtain.

According to Fig. 17c, model C (ss) shows the lowest bearing stresses by providing concrete confinement until drift 2.2%, while at higher drifts, model C (ss) has the same result as models L-S (ss), F -L-S (ss), F-B-L-S (ss) that have SDP. At drift level higher than 3.5, models with SDP, show better performance.

Fig. 17. Bearing stresses in concrete

4. Conclusion

In this paper, fifteen interior beam-through type RCS connections are simulated using a nonlinear three-dimensional finite element method using ABAQUS [21] software and are verified with the experimental tests of Alizadeh et al. [11]. The aim of this study is to investigate the performance of CPs, and ABPs. The performance of the models are compared in terms of load-story drift response, joint shear forces, joint bearing stress, shear strains at joint region, steel beam stresses, and concrete cracking pattern. Based on the results of this study, the following conclusions are made:

It is obvious that using CP and removing joint stirrups is more practical than using SDP and joint stirrups with or without SBP and FBP. Based on the lateral load-story drift
responses of the simulated models, using CP instead of the combination of steel band plates, FBP, and joint stirrups is useful to keep the model at the same performance. On the other hand, based on this study, using CP causes steel beam web participation in joint shear force to reduce and CP and inner concrete panel participations to increase. At 4% story drift, fewer cracks are observed because CP confines concrete well. Using CP instead of steel doubler plate causes shear strains in outer concrete panel and steel beam web to increase and shear strain in the inner concrete panel, bearing stress and shear lag in the joint region to decrease.

Results show that using ABP with CP can improve the performance and reducing the participation of steel doubler plate and increase the participation of CP. In addition, it helps CP to confine the concrete better, so that fewer cracks form in the joint region. The steel doubler plate in these models tolerates less stress because ABPs help to transfer force from steel beam to CP more efficiently.
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Figure captions:
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  b) Steel beam
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  c) Models L-S (ss), F -L-S (ss), F-B-L-S (ss), C (ss)
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  d) Cover plate
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  d) Cover plate
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  c) Steel beam web
d) Cover plate

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Fig. 10. Stress of beams in models at 4% story drift

11-a) Inner concrete panel
11-b) Outer concrete panel

11-c) Steel beam web

Cover Plate

Model c
Model S-C
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17.c) Models L-S (ss), F-L-S (ss), F-B-L-S (ss), C (ss)

Fig. 17. Bearing stresses in concrete
Table captions:

Table 1. Material properties
Table 2. Joints details
Table 3. Contribution percentage of internal shear forces of joint shear mechanisms
**Table 1. Material properties**

<table>
<thead>
<tr>
<th></th>
<th>F&lt;sub&gt;y&lt;/sub&gt; (MPa)</th>
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F: Face Bearing Plate, B: Band Plate, L: L-shaped Joint Stirrup, S: Steel Doubler Plate, C: Cover Plate, A: Additional Bearing Plate, (s): 20 mm Flange Thickness, (ss): 25 mm Flange Thickness
Table 3. Contribution percentage of internal shear forces of joint shear mechanisms

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<tr>
<th>Model</th>
<th>Inner Concrete Panel</th>
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<th>Steel Beam Web</th>
<th>Cover Plate</th>
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<td>3%</td>
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<td>model C (ss)</td>
<td>21.46</td>
<td>21.69</td>
<td>18.27</td>
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</table>

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