تقوية الأعمدة الحديدية ذات قطاعات على شكل I و H ضد الأنبعاج

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ملخص البحث

تبحث هذة الدراسة معمليا مقاومة الأنبعاج فى الأعمدة الحديد ذات قطاع على شكل I و H المقواة ضد الأنبعاج. تم تقوية الأعمدة بزيادة كلا من المساحة و عزم القصور الذاتى لقطاع العمود خلال جزء من طولة. تم تنفيذ ذلك عمليا بلحام شريحتين من الحديد بين الأطراف الحرة لألواح الشفاة و موازية للوح الجذع فى قطاع العمود. يمكن أيضا أستخدام المسامير أو اللحام لوصل أعضاء لها قطاعات مدرفلة على الساخن لقطاع العمود. تكون المساحة و عزم القصور الذاتى لقطاع العمود فى هذة الحالة ذات قيم مختلفة خلال طولة. تم أختبار عدد ٨ أعمدة تحت تأثير قوى ضغط محورية حتى الأنهيار. المتغيرات التى تم أخذها فى الأعتبار هى ١) النسبة بين قيمة عزم القصور الذاتى للعمود بعد التقوية و تلك الخاصة بالقطاع الأصلى للعمود ٢) النسبة بين الطول الذى تمت فية التقوية و الطول الكلى للعمود ٣) نوع وتفاصيل الوصلات بين العمود وعناصر التقوية ٤) طريقة تثبيت العمود عند الأطراف. تراوحت قيمة الزيادة فى حمل الأنبعاج نتيجة لعمليات التقوية التى تمت بين ١٨١% و ٣١٨%. تم أيجاد أحمال الأنبعاج للحالات التى تم أختبارها معمليا بأستخدام الحل النظرى. تمت مقارنة النتائج المستخرجة من التجارب و الحل النظرى وكذلك تم تقيم كفاءة طريقة التقوية المستخدمة ضد الأنبعاج.

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STRENGTHENING STEEL COLUMNS OF ROLLED SECTIONS AGAINST BUCKLING

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Abstract

This study investigates experimentally the buckling behavior of steel columns having rolled sections of I shape when strengthened against buckling. The area and the moment of inertia of column cross section was increased over limited length of the column. This was carried out by welding two plates between the free edges of the flanges parallel to the web. Bolting or welding rolled sections to column cross section is another method. The column in this case would have discontinuous variation in its cross section. A total number of 8 columns were subjected to concentric axial load until failure. The variables considered are: 1) the ratio of the new moment of inertia to the original one, 2) the ratio of the length where cross section is changed to column total length, 3) the type and details of joining system; i.e. bolts or welds and 4) the end restraining conditions. The enhancement gained in the buckling load value due to the strengthening technique implemented was between 181% and 318%. Analytical solution was used to calculate the critical load values of the cases considered. The obtained results from both the experiments and the analytical solutions were compared and discussed. The efficiency of this strengthening method in changing and controlling the buckling behavior of already existing columns is assessed.

Key words:

steel column, buckling - strengthening, buckling - discontinuous cross section, column

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Introduction

Situations do arise when strengthening of already existing steel columns of rolled sections is required. This would occur when the structure function and/or loading capacity is required to be changed. Satisfying new codes of practice is another reason. The authors used the concept of built-up members to solve this problem. The areas and the moments of inertia of the column cross section are to be increased in value over a limited length of the column. In practice, this can be carried out for columns of I and H shape cross sections by welding two plates between the free edges of the flanges parallel to the web, figure 1. Bolting or welding rolled sections to column cross section as shown in figure 1 is another method. The column in this case would have discontinuous variation in its cross section. The critical load and the buckling behavior would be changed. Timoshenko and Gere (1963) have presented an analytical solution for the problem. It depends on trial-and-error method. A semi analytical procedure for the buckling of structural components with continuous and discontinuous variations of cross sections has been presented by Arbabi and Li (1991). The only experimental results found in literature for a similar case was reported by Aslani et. al. (1991). They tested two specimens, each consisting of two angles back-to-back. The specimens were strengthened against local buckling by welding Inclined continuous plate between the free edges of each angle, specimens ABS7 and ABS8. The stiffening plates were stopped 6 inch before the gusset plate in one case and just before the gusset plate in the other case. This was carried out to avoid local buckling when the angles are subjected to seismic loads. The specimens showed superior buckling load values in comparison to other cases. No recommendations are given in codes of practice and specifications to calculate the contribution of the stiffening components to the column rolled section and the enhancement expected in the buckling strength and behavior.

Other relevant work is that of built-up compression members. In this case, the area and the moment of inertia of the cross section are increased along the column length. All the components of the cross section are assumed to behave as one integral unit. This is satisfied in the American Institute of Steel Construction specification, AISC (1973). The longitudinal spacing of fasteners connecting components of built-up compression members must be so limited that buckling of segments between adjacent fasteners would not occur at less load than that required to develop the ultimate strength of the member as a whole. Further, The maximum longitudinal spacing of rivets, bolts or intermittent welds connecting two rolled shapes in contact with one another shall not exceed 24 inch. Libove (1985) however, showed that the buckling load can be considerably below that corresponding to integral action, even if the slenderness ratio of the individual elements between connectors is less than that of the built-up column as a whole. Other studies were presented in literature concerned with width-to-thickness ratio of out standing legs, local buckling in seismic resistance (Aslani et. al.; 1991), and inplane and out-of-plane buckling of double angle bracing (Astaneh et. al.; 1984 and 1985). The effect of interconnection number in boxed angle compression members was studied by Temple et. al. (1987). Design rules and connectors type and spacing were considered by Duan et. al. (1988).

This study investigates experimentally the buckling behavior of steel columns having rolled sections of I shape when strengthened against buckling. The area and the moment of inertia of the column cross section was increased over a limited length of the column as described. The variables considered are: 1) the ratio of the new moment of inertia to the original one I_2/ I_1 , 2) the ratio of the length where cross section was changed to column total length L_2/L , 3) type and details of joining system (bolts or welds) and 4) end restraining conditions. The analytical solution presented by Timoshenko and Gere (1963) was used to calculate the critical load values of the cases considered. The obtained results from both the experiments and the analytical solution were compared and discussed. The efficiency of this strengthening method in changing and controlling the buckling behavior of already existing columns is assessed.

Analytical solution, (Timoshenko and Gere; 1963)

This method depends on the differential equation of the deflection curve for each portion of the column. The conditions at the column ends are considered. The critical load value P_{cr} for a column of hinged ends and symmetrical with respect to the middle cross section, figure 2, can be represented as follows:

$$P_{cr} = m\ E\ I_2 \ /\ L^2 \qquad (1)$$

where m is a numerical factor, E is the modulus of elasticity, I_2 is the new moment of inertia and L is the column total length. The value of m is calculated by the trial-and-error method from equation 2.

$$\mathrm{Tan}\ (K_1 L/2)\ \tan (K_2 L_2/2) = K_1/K_2 \qquad (2)$$

where

$$K_1^2 = P/\ E\ I_1 \qquad \text{and } K_2^2 = P/E\ I_2$$

When substituting equation 1 in the terms K_1 and K_2, equation 2 can be formulated as follows:

$$\tan\left\{\left[\left(1-\frac{L_2}{L}\right)/2\right]\left[\sqrt{\frac{m\,I_2}{I_1}}\right]\right\}\tan\left\{\left(\frac{L_2}{2L}\right)\left(\sqrt{m}\right)\right\}=\sqrt{\frac{I_2}{I_1}} \qquad (3)$$

Experimental programs

A total number of 8 columns were subjected to concentric axial load until failure. The variables considered are 1) ratio of I_2/I_1 , 2) ratio of L_2/L , 3) type and details of joining system and 4) end restraining conditions. The dimensions and details of the specimens are presented in figures 3-a, 3-b and table 1. The columns are classified into four groups. No strengthening was carried out for the columns' cross section in group A. Columns of group B were strengthened as described below. Two plates 750 mm length, 110 mm width and 4 mm thickness made of steel 37 were fillet welded between the free edges of the flanges parallel to the web, for each column cross section. In group C, two channels No. 65 were bolted back-to-back to the web of the column cross section. In group D, the channels were joined to the flanges of column cross section. They were bolted in column C7 and welded in column C8.

Test specimen

Eight rolled steel members made of steel 37 having cross section of standard I-beam No. 100 were used for the columns. The dimensions of the cross section are shown in figure 4. The width-thickness ratio values for this section are in compliance with the Egyptian code of Practice for steel structures and bridges, (1989) and parts 1 and 2 of the AISC specifications, (1973). The column length was made 1850 mm. Rotation about the X-X axis was not allowed. Hinged supports were provided to allow rotation of the column about the Y-Y axis. The slenderness ratio about the Y-Y axis is equal to 172.9. This value exceeds the term Cc specified in the AISC specifications. The allowable axial loading capacity in this case, as specified in AISC, is equal to the Euler load of the column multiplied by a factor of safety. All the specimens were painted with lacquer.

Material Properties

i - steel

Tension tests were carried out on three specimens. Two were taken from the web of the standard I-beam section and one from the web of the channel No. 65. The yield and ultimate strength values were obtained and their average values were calculated and presented in table 2.

ii - bolts

Bolts of size M6 and grade 10.9 complying with DIN 931 were used to connect the channels No. 65 to standard I-beam sections No. 100 in close fit holes. The yield and ultimate strength values for grade 10.9 bolts specified in DIN 18800 are 900 and 1000 N/mm^2 respectively. The minimum ultimate tensile load and proofing load values for this size and grade of bolt specified in the International Standard ISO 898/1 -1988 (E) are 20.9 and 16.7 KN respectively. The M6 bolt is below the minumum size specified in codes of practice for structural bolting. However, the relatively small dimensions of the web depth and the flanges width in both standard I-beam No.100 and channels No. 65 and the minimum distance required for bolts installation necessitate the use of this size. This in turn again, necessitates the use of bolts of grade 10.9. The bolts were designed to satisfy the required shear resistance for load transfer from column cross section to the stiffening elements and vice versa. The ultimate shear strength is taken equal to 62% of bolts tensile strength specified in codes of practice. This percentage was found by Fisher and Struik (1974) and is independent of bolt grade. Nuts of grade 8 and nominal thread diameter of 6 mm complying with DIN 934 were used. This was due to the unavailability of nuts of higher grades. The specified proof load in the ISO 898/2 - 1980 (E) for this size and grade of nut is 16.3 KN. This value is comparable to that of bolt proof load. Hardened washers were used under the nuts in all the tests. Torque wrench was used for tightening the bolts to a torque of 12 foot-pound. This would produce tension force in each bolt equal to 13.56 KN.

iii - Welds

Fillet welds of 3.0 mm equal leg size were used in the experimental program. The welding was carried out by the manual metal arc welding process. The electrode was of 3.25 mm diameter and 350 mm length and of class E4332R complying with DIN 1913. For consistency, all the welds were carried out by one welder.

Instrumentation

Two displacement dial gauges were used to measure the axial deformation of the columns during the loading process. The gauges were fixed at equal distances from column cross section center. The dials were reacting against the angle shown in figure 5. The angle was welded to a steel plate 300 mm length, 200 mm width and 20 mm thickness. The angle and the steel plate became one unit and were positioned on the column top. They were made to coincide with the center of both the column cross section and the cylinder of the hydraulic jack used in the loading process. The average value of the dials readings is the column axial deformation. The comparison between the two readings of the dial gauges would indicate the start of column top rotation.

Test setup

Two plates of 150 mm length, 100 mm width and 5 mm thickness were fillet welded at the base and top of each column. Two rollers of 55 mm diameter were used to provide the hinged supporting conditions for the columns about the Y-Y axis. The columns were positioned in the test rig as shown in figure 6. Precautions were taken to prevent any transaltional movement for both the base and top of columns. A hydraulic jack of 60 tons loading capacity was used for the application of the load. The load was monitored using digital load pressure device having an accuracy of 0.3 ton. The load was applied in intervals and continued until failure. At this stage, the load drops and the deformations increase significantly.

Results

All the results obtained are presented in table 3. The experimental failure loads P_t are presented in KNs, col. 1 of table 3. Their ratios to the experimental buckling load P_o and the yielding load P_Y of specimen C1 are presented in col. 2 and col. 3 respectively. The actual yield stress value, obtained from the tension tests, was used in this comparison. In col. 4, P_y' is calculated using the minimum yield stress value of steel 37 specified in the Egyptian codes of practice (1989). The ratio of the critical load of each case, obtained using the analytical solution, to the yield load P_y of C1 is presented in Col. 5. The experimental failure load values were divided by the critical load values in col. 6. The weight of each specimen was divided by the weight of specimen C1 in col. 7. For each specimen, the value of P_t/P_o ratio was divided by its W/W_o ratio. The results were presented in col. 8. The produced value is considered herein as an efficiency coefficient. It is a non-dimensional factor and measures the efficiency of the steel added to the column in terms of column buckling loading capacity. Figure 7 compares between the axial deformation behavior of all the specimens. The ordinate represents the ratio of the applied load to the yield load P_Y. The abscissa represents the ratio of the axial deformation to the column total length. Figures 8, 9, 10 and 11 show the failure modes of specimens C3, C6, C7 and C8 respectively.

Observed Behavior

No strengthening was carried out for the specimens of group A. Hinges were provided to support specimen C1 at its base and bottom. This is considered the basic case which is required to be strengthened. Concentric axial load was applied in intervals. Buckling occurred suddenly and in a symmetrical mode similar to that of a hinged-hinged column. At buckling, the load dropped significantly. After unloading, the specimen regained nearly its original shape. Specimen C2 was supported directly on its base plate without a hinge. Buckling occurred suddenly in a mode similar to that of a hinged-fixed column. After the test, permanent deformation was noticed but no local buckling was observed.

All the strengthened specimens were supported by hinges at their base and bottom plates. The specimens satisfied the requirements of the AISC as the spacing between bolts in C5, C6 and C7 and between intermittent welds in C3, C4 and C8 is less than 24 inches. The specimens showed similar behavior but different failure load values. They failed by buckling which occurred suddenly in an unsymmetric mode. It is similar to that reported by Aslani (1991). Buckling occurred in C3, C5 and C7 to the left. Specimens C4, C6 and C8 buckled in the other direction. Figures 8, 9, 10 and 11 show specimens C3, C6, C7 and C8 respectively in the test rig at buckling. Yielding was observed at the web and at the concave side of the flanges of the original cross section. This occurred just above the stiffening plates in C3 and C4 and above the channels in C5 and C8. In specimens C6 and C7, yielding was noticed in the original cross section at the first row of bolts from below. Yielding was indicated by the cracks that appeared in the painting at the specified areas. No local buckling was observed in the cross sections of both the original column and the stiffening elements. Furthermore, non of the stiffening plates or the channels buckled individually. No failure was observed in the welds and the bolts in all the specimens tested.

Analysis of results

The enhancement gained due to the strengthening technique implemented in this study was between 181% as in C3 and 318% as in C8. In all the specimens tested, the failure load was less than the yield load of C1. Generally, the specimens can by classified in two groups. In the first group, specimens C1, C2, C4 and C5 exhibited experimental failure loads nearly equal to their critical loads, col. 6 of table 3. Specimen C2 was strengthened against buckling by changing its restraining conditions. This caused stiffer axial behavior as shown in figure 7 and 190% enhancement in the buckling load compared to C1. The difference between P_t and P_{cr} may refer to the imperfections in the specimen and/or test arrangements. Specimens C4 and C5 were strengthened by welding and bolting additional stiffening elements to the original cross section over a limited length of the specimen, respectively. They failed at the critical load values obtained from the analytical solution. This indicates that the stiffening elements and the column behaved as one integral unit and that the details of the welds in C4 and the bolts in C5 are appropriate. The results in general show the success of the implemented technique to change and control the buckling loading capacity and the behavior of these specimens.

In the second group, specimens C3, C6, C7 and C8 failed by buckling at loads less than their critical load values, obtained using the analytical solution. This is explained as follows. The applied axial load causes axial deformations in the column. The bolts or the welds used transfer part of these deformations to the stiffening elements. This depends on the axial stiffness of the stiffening elements EA/L_2. When bolts are used, bolts bending resistance and bolts clearances if exist are other factors. When welds are used, welds elastic straining action should be considered. As a result, part of the applied load is transferred from the original column to the stiffening elements and vice versa. The value of this part of the load is proportional to the axial stiffeness of the stiffening elements in addition to the factors listed above. The joining system whether bolts or welds, does not provide full continuouty between the column and the stiffening elements. Hence, the load transfers through limited areas of the column cross section such as the free eddges of the flanges in C3 and C4, the web in C5 and C6 and the flanges in C7 and C8. This does not allow equal contribution of all the column cross section elements in the load transfer process. Areas of stress concentration are expected to exist at the sections where the column cross section is changed. Improper detailing of the joining system whether bolts or welds, would increase the size of these areas and the magnitude of the stresses induced therein. At a load less than the critical load, the stress would reach the yield value of the steel material and plastic hinges would initiate. Increasing the applied load would increase the size of the plastic hinges areas. The structure system of the column would change to a chain of hinged axially loaded members. This would lead to early buckling compared to the critical load value obtained from the analytical solution.

The following sections discuss the effects of the different parameters considered.

(i) Welding details

The buckling load of the different segments of the stiffening plates between intermittent welds in C3 is equal to 0.47 of the experimental failure load of the specimen. This value was calculated assuming that the intermittent welds provide fixed supports to the stiffening plates segments. The failure of C3, figure 8, shows no buckling in any of these segments. This indicates that the load transferred to these plates is less than their buckling loads. Specimens C3 and C4 show similar load-deformation relationship up to a load equal to the buckling load of C1, figure 7 . After this load, C3 shows a relation similar to that of C2. The weld lengths in C3 and C4 were designed to sustain the maximum experimental failure load ecpected, i.e. the critical load value. The total weld length in C3 is made less than that of C4. This would limit the areas at which the load transfers from the column to the stiffening elements and vice versa. This is expected to increase the size of stress concentration areas and the magnitude of the stresses induced in it and would lead to the early failure of C3 compared to C4.

(ii) Bolts details and ratio of L_2/L

Specimens C5 and C6 were made typical except in L_2 value. The same number of bolts were used in the two specimens. At the same level of applied load however, the bolts transfer larger part of this load to the stiffening elements in C6 in comparison to C5. This is because the increase of L_2 value in C6 would reduce the axial stiffness of the stiffening elements. Specimen C5 showed stiffer behavior than that of C6 as indicated by their load-deformation relationships, figure 7. Specimen C5 failed by buckling. Specimen C6 failed at the same load but due to the mechanism described before. However, the strengthening of C6 would be more effective if enough number of bolts were used.

(iii) Joining system type, (bolts or welds)

Specimens C7 and C8 were made typical except in the joining system, bolts in the former and welds in the latter. They failed in a mode similar to that of C6 but exhibited higher failure loads. This is refered to the following reasons. Firstly, the area of the flanges through which the load is transferred in C7 and C8 is bigger than the area of the web in C6. Secondly, the number of bolts used in C7 to connect the stiffening elements to the original column is double that used in C6. Specimen C8 exhibited stiffer axial behavior and higher failure load than C7, figure 7. This is due to the areas deduced from the cross section due to the bolts holes in C7.

(iv) ratio of I_2/I_1

Specimens C6 and C7 have different values of I_2/I_1 ratio. They failed at loads less than their critical loads, obtained using the analytical solution, due to the mechanism described before. The comparison in this case would not lead to right conclusions as the joining details in the two specimens are different.

Summary and Conclusions

It is proposed in this study that already existing steel columns can be strengthened by welding or bolting additional stiffening elements to the original cross section over limited lengths of the considered columns. A total number of 8 columns were subjected to concentric axial load until failure. The enhancement gained due to the strengthening technique implemented was between 181% and 318%. They failed by buckling which occurred suddenly in an unsymmetric mode. The results in general show the success of the implemented technique to change and control the buckling loading capacity and the behavior of these specimens. The detailing of the bolts or welds joining the stiffening elements to the original column cross section affects the results significantly. When proprer joining details are used, the columns failed at load values equal to their critical loads. Improper detailing of the joining system whether bolts or welds, would produce areas of stress concentrations at the section where the column cross section is changed. At loads less than the critical loads, the stresses induced in these areas would approach the value of the yield stress of the steel material and plastic hinges would initiate. Increasing the applied load would increase the size of the plastic hinges areas. The structure system of the column would change to a chain of axially loaded hinged members. This would lead to early buckling loads compared to the critical load values obtained from the analytical solution. Recommendations in codes of practice needs to be reviewed for this case. The 24 inches maximum spacing between bolts and between intermittent welds specified in the AISC did not provide proper detailing in four of the tested columns. The Egyptian code of practice requires, for built-up compression members, the use of bolts at pitches not exceeding twelve times the thickness of the thinnest outside plate or six times the bolt diameter. Satifying this requirement along the stiffening elements lengths is relatively expensive and seems to be very consevative when considering the experimental results.

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APPENDIX II - Notations

A cross sectional area of stiffening elements.

E modulus of elasticity

I_1	moment of inertia of column cross section
I_2	moment of inertia of strengthened column cross section.
K_1,K_2	numerical terms.
L	total length of column.
L_1,L_2	length of different portions of column.
m	numerical factor.
P	buckling load
P_{cr}	critical load
P_o	experimental buckling load of specimen C1
P_t	experimental buckling load
P_y	actual yielding load of specimen C1.
P_y'	Yielding load calculated using minimum specified yield stress in code of practice.
s	axial deformation

angle dial gauge specimen

1- Hydraulic jack 2- Roller of 55 mm diameter

3- displacement dial gauge 4- Angle 5- Specimen

Figure 5. Instrumentation

Figure 6. Specimen C2 in test rig

Figure 8. Buckling of specimen C3

Figure 9. Buckling of specimen C6

Figure 10. Buckling of specimen C7

Figure 11. Buckling of specimen C8