التحليل العددى للوصلات الملحومة فى الجمال المعدنية

د٠ فتحى عبد المنعم عبد الفتاح

قسم الهندسة المدنية – جامعة الزقازيق

جمهورية مصر العربية

الملخص:

استخدمت طريقة العناصر المحدودة فى هذا البحث للتمثيل العددى للوصلات الملحومة فى الجمال المعدنية ٠ مثلت كل أعضاء الجمل الذى تمت دراستة و كذلك تفاصيل الوصلات و اللحام لمعرفة سلوك كل عضو منفردا و تأثيرة على بقية الأعضاء الأخرى ٠ أستخدم العنصر المصمت ذو الثمانى أركان (ثمانى نقاط و كل نقطة حرة الحركة فى ثلاث أتجهات) ٠ ألوصلات الملحومة تعمل كزنبرك ذو جساءة للدوران ٠ تنتج هذة الجساءة من الأنفعال المرن للحام والمحدود بقدرة اللحام لمقاومة الأجهادات الناتجة من القوى المؤثرة على الوصلة بالأضافة لتلك الناتجة من مقاومة الدوران للأعضاء الملحومة ٠ تم تحليل النتائج المستخرجة و مناقشة الفرض الشائع الأستخدام فى التحليل العددى للجمال"الوصلات فى الجمال المعدنية مفصلية"وتأثيرة على تصميم اللحام والأعضاء المختلفة بالجمل

عنوان المراسلة:

د. فتحى عبد المنعم عبد الفتاح

١٨ ش هيرودوت – الشاطبى – الإسكندرية

ت: ٤٢٢٣٧٨٦ – ٥٩٧٥٣٩٥

NUMERICAL ANALYSIS OF WELDED CONNECTIONS IN TRUSSES

Fathy Abdelmoniem Abdelfattah
Civil Engineering Department
Zagazig University , Egypt

Abstract:

The finite element method was used to model the behavior of welded connections in trusses. All truss members and connections details were idealized to include their interaction behavior and to know how the truss would behave in the context of welded connections. Solid isoparametric brick elements were used. Welded connections provide rotational stiffness. This is due to welds elastic straining action that is limited by the weld ability to resist the produced stresses. The results obtained were used to discuss the pinned connection assumption, commonly used for the analysis of trusses considering welds design and factor of safety consistency.

Key words

Trusses, welded connections, pined connections

Postal address

Fathy Abdelmoniem Abdelfattah
18, Herodot St., El Shatby, Alexandria, Egypt
(0203) 4223786 - 5975395

1 Introduction:

Studies carried out about the design of steel structures for economy concluded the following.”To obtain optimum price of steel structure, save labour and do not worry about the amount of material”; (1). This is affected to a significant degree by connections design and details. Connections must be simple and cheap to fabricate. Fillet weld is seen to be easy and economic to fabricate for almost all manner of connections. Fillet weld does not need special preparations. In trusses fillet weld is used to connect members to gusset plates at the different joints. This is seen to be easy and cheap, in most cases, in comparison to bolting. In design terms, connections in trusses are assumed to be frictionless hinges and the members are subjected to axial loads. These conditions are not satisfied due to connections details. The literature is full with studies concerning with the behavior of rivets ; bolts and welds as a joining material such as; (2, 3 , 4 , 5, 6) and others .Other studies presented in the literature are concerned with the stresses in gusset plates and their design;(7, 8, 9). In these cases part of the truss was modeled and the interaction behavior of the different elements of the truss was not included in the analysis.

Design methods of connections should be based on the understanding of connections real behavior and not on broad simplifications. Attention has not been paid to the real behavior of connections except at the 1980`s ;(10, 11). Most of this effort is concerned with moment connections. This study investigates the behavior of welded connections in trusses. The truss members; gusset plates and welds were modeled using the finite element method to include their interaction behavior in the analysis and to know how the truss would behave in the context of welded connections. Welds restraining action of the members is discussed. Stresses in the welds are compared to the allowable and yielding stress values of weld material specified in the Egyptian code of practice of steel structures and bridges;(12). The truss considered in this study is of Pratt type as shown in figure 1. This was chosen to keep the number of members and joints to minimum. The study is concerned mainly with connection C that is not a support as joints A and B and not a directly loaded joint as E. Each member is named after the two connections joining between them. The truss is subjected to vertical downward load at joint E equal to 100 KN.

2 Finite element modeling

The geometrical properties of the truss mesh were defined by the truss members, welds and gusset plate a) details; b) dimensions and c) positions. A three dimensional finite element mesh was developed, figure 2. Due to symmetry about the X - Y plane, only half of the truss was modeled. Solid isoparametric brick elements with eight nodes, one at each corner were used . There are three degrees of freedom at each node, displacements, defined with respect to the global cartesian coordinate system X , Y and Z. The members were made to consist of two angles back to back. Angle cross section was modeled using three elements, figure 2. This group of elements was repeated number of times, taken as eighteen in figure 2, to model the member in its length direction. Each member is fillet welded to gusset plate at two positions, I and II . Weld cross section was modeled in the Y - Z plane using one brick element. Four elements were used to model the weld in its length direction , X - Y plane. This was applied similarly for the welds at positions I and II . The maximum dimension of weld leg size was limited to the member angle leg thickness. The gusset plate details depend on the number of members meeting at the joint considered, figure 2. The dimensions were proportioned so that no overlap between members elements was allowed. Due to symmetry,only half the thickness of the gusset plate was modeled using one brick element, in the Y - Z plane and restrained at its back in the Z direction. The gusset plates at joints A and B are assumed to be welded to fixed supports.This was modeled by fully restraining the back of the gusset plates at these joint.

The material of the members, welds and gusset plates was idealized as linear elastic having modulus of elasticity equals 205000 N / mm^2 and Poisson ratio of 0.3. Applied load was idealized as four point loads so that their resultant applies at the center of joint E gusset plate.

2. 1 Data management

It worth mentioning that the author developed computer program using Basic computing language to prepare the input data for the finite element program. By defining the a) the dimensions of the members, welds and gusset plate cross sections; b) the number of elements in the length direction of the members and welds; c) members start and end coordinates and d) the coordinates of each joint and the members meeting at that joint, the program produces the coordinates of each node in the mesh in addition to the formation of all the elements in the mesh. This output data was used as input data for the finite element program. This made it easy to consider different variables in the study. The main advantage; however is to know the position of each node and element in the mesh, even when the mesh is changed; for instance refined. This program may be used for different truss patterns with any number of members and joints. Another program was developed to read and make summation of the forces,at certain nodes of defined elements, from the finite element program results. This facilitated knowing easily the forces in the truss members and overcomes the problem of limited memory available incomparison to the memory required to read the output data files using the conventional programs.

3 - General behavior

Figure 3 shows the displacements of the different elements of the truss. The members were made to consist of two angles back to back of size 75 X 75 X 7. The gusset plate thickness was taken equal to 14 mm. The members were assumed to be welded at two positions I and II from both sides of the gusplates. The weld length was taken as 200 mm at each position; (13) for all the members. The weld was made to have equal legs having a size of 5 mm.The displacements at the center of gusset plates at the different joints were obtained. Their values are found to be larger within a range of 5 - 10 % than those of a similar truss but with frictionless hinges at the different joints. In the latter case, the members rotate freely about the hinges,assumed at the joints. This is not the case when considering the actual behavior of the members. Member C E links between joints C and E. These two joints displaced in the negative direction of the X and Y axes. The member should displace from its both sides with the joints producing a straight member between them. This would be the case when the joints work as hinges. Member C E, however displaced in a different mode,figure 4. The welded portion of the member to the gusset plate displaced nearly in a linear manner. This is limited by gusset plate displacements and deformations in addition to welds elastic strain. The rest of the member rotates about the Z axis linking between the member's ends. The weld in this case restrains the member at its both ends against rotation and the member deforms as shown in figure 4. The undeformed member is drawn in dotted lines.

Figure 5 shows the deformation and displacements of member C D. Joints C and D displaced in the negative direction of the Y axis .The difference between their displacement values refers mainly to the changes in member C D length. Joint D displaced in the positive direction of the X axis while joint C displaced in the opposite direction. Again the member displaced in a mode similar to that of member C E. Similar behavior is observed for all the members. This type of restraining at member's ends is expected to reduce the buckling length of compression members and hence increases the factor of safety value against buckling. Generally, a joint in the truss may be modeled using three springs, one having rotational stiffness to model weld restraining action. The other two springs have transitional stiffness to model members axial stiffness and truss stiffness against displacements.

The values of the axial forces in the truss members were obtained from the results. They are different to those calculated for a similar truss but with pure hinges at the different joints. In design terms,the magnitudes of these differences are relatively small as shown in table 1 and would not affect the factor of safety to a significant degree. Shear forces are produced. Due to the nature of the element used in the finite element analysis, eight node brick element, moments values were not calculated. Their effect was included in the analysis as will be seen later. Moments are produced due to the induced shear forces and the eccentricity of the axial forces in the members and distributed according to the rotational stiffness of the joints at the ends of each member .

4 - Parametric study

This study was carried out to know the effect of changing the dimensions of the truss details on the induced forces in the members. The results obtained are presented in Table 1 . The percentages of the changes in axial forces values N % in comparison to axial forces obtained of a similar truss but with hinges at the joints are presented for all the members.Shear force value induced in a member is presented as a percentage V % of the axial force induced in that member when considering hinges at the joints. The trusses considered in cases 1; 2; 3 and 4 of table 1 have the same details but different gusset plate thickness t . The maximum changes in the ratios N % and V % are 4.4 % and 2.8 % respectively. The comparison between the results of cases 3; 5 and 6 would indicate the effect of gusset plate dimensions R. The effect of weld size S is considered in cases 3;7 and 8.The change of weld length L is considered in the trusses of cases 3; 9; 10 and 11. The truss considered in case 12 is similar to that of case 3. In case 12; however the members were made to consist of two angles back to back of size 150 X 150 X 15. The results generally show that the changes in the axial forces values induced in the members are marginal except in case 12 at which these changes exceeded 10 %. Member C D is the most affected member with these changes. The maximum shear force value did not exceed 5 % except at case 12.

5 - Load transmission

The member; weld and gusset plate share the same node at the weld root in the finite element mesh described above. The values obtained for the forces in the weld are hence the resultant values and not the actual values. A substructure was developed for the member and fillet weld details as shown in figure 6 to obtain the forces values transmitted by the weld at positions I and II. Again the axial force is distributed between the welds at positions I and II according to their positions from the member cross section center of area. The calculated values were found to be different to those obtained from the finite element analysis. The differences at positions I and II are equal but with different signs. These differences are the components of the moments at member ends, produced due to shear forces and the eccentricity of axial forces in members. Generally, forces are transmitted from gusset plate to member or vice versa through the weld subjecting it to shear forces.

The maximum distortion energy theory (14) was used as a criterion for the yielding at the weld root along the weld length. The principal stresses are obtained from the finite element results and used to calculate the equivalent stress. Connection C of the truss considered in case 3 of table 1 is considered. The equivalent stresses values are presented in figures 7 and 8 for the welds at positions I and II respectively as a ratio to the yield stress of the weld material (12), 244 N / mm^2. Welding is assumed to start at the member end at the center of the joint and goes out ward. This position is named here as weld start while the end of the welding run is named as weld end. Figure 7 shows the ratio of equivalent stress to the yield stress along the weld root at position I. When considering member A C, two regions of stress concentrations are found. The first is at weld start where the equivalent stress exceeded the yield stress value. The stress then reduced in a relatively gradual manner. The second region is at weld end where the equivalent stress value increased dramatically. This is explained as follows. The transmission of forces from the gusset plate to the weld and then from the weld to the member, or vice versa, produces shear stresses in the weld. The member and gusset plate are strained relative to each other in proportion to their stiffness and the induced forces. The gusset plate is strained maximum at the weld start where the transmission of load to or from the gusset plate starts.This depends on stress distribution at that area and limited by the maximum shear strain in the weld and yielding of weld and / or the plate material. The axial and shear forces in the member are maximum at weld end. These forces in addition to the component of the produced moment at the joint cause the maximum strain in the member in an opposite direction to that of the gusset plate. Again,this is limited by the maximum shear strain in the weld and yielding of the weld and / or the member material. This result was found for all the welds at position I in all the joints of the truss but with different magnitude, except the weld of member C D at joint C where slight difference is observed.The ratio of the equivalent stress to the yield stress at the root of the weld along position II is presented in figure 8. Two regions of stress concentrations are found. The first is at weld start similar to the weld at position I but with smaller magnitude. The second region is before the weld end. The strain is expecto be reduced or increased due to the component of the induced moment by the weld at that region.

The increase of weld length as in case 11 of table 1 showed the same results but with different magnitude. The equivalent stress value at weld start reduced nearly by 10 % but increased significantly at weld end.This may refer to the reduction in member flexibility due to the reduction in its unrestrained length. The weld leg size was increased from 5 mm in case 3 to 7 mm in case 8 of table 1. The equivalent stress values showed significant reductions of the order of 75 % in comparison to those of case 3, discussed before. The increase of weld leg size in case 8 would increase the weld allowable strength by 40 %; (12) but the volume of the weld metal is increased by 96 %. Generally, the use of excessively large welds is not recommended in codes of practice as the weld may crack due to the high contraction stresses induced. A common practice is to make the welding around the member to resist corrosion. The amount of weld and continuity in this case would reduce stress concentrations at weld start. This is not the case at weld end as the space between the two angles which is equal to gusset plate thickness does not allow welding around the member.

The results in general show that the stresses values at the weld root exceeded the allowable stress value specified in codes of practice (12) and even exceeded the yield stress of the weld material in some cases. Yielding at weld root would cause distributing of the stresses along the weld length.In design terms, slight local yielding at weld root and limited area of the gusset plate may be considered of minimal significance when failure criterion is initial ductile yielding. However, when this condition is combined with impact and / or repeated loading cracks are expected to initiate at weld root; propagate and finally causing fracture.

6 - Conclusions

Welded connections in trusses provide rotational stiffness at the joints. This is limited by the welds maximum shear strain and yielding of the weld and / or the gusset plate and the member material. The welds restrains the members against rotation. This produces axial and shear forces in the members in addition to moments. The assumption commonly used for the analysis of trusses"frictionless hinges at truss joints"is found to predict the axial forces in the members within a range of 10 %. The shear forces and moments produced is expected to change the factor of safety of members design not to a significant degree. Two regions of stress concentrations was found at the start and end of the weld. The stresses exceeded the allowable stress values and even the yield stress at the weld root in some cases This would reduce the factor of safety of the welds design. The use of large weld leg size is found to reduce the stresses at the weld root to a significant degree.

7 - References

1) Van Douwen, A. A., Design for economy in bolted and welded connections, Joints in structural steel work - Proc. Int. Conf. held at Teesside Polytechnic, Cleveland, U. K , (1981)

2) Fisher, J. W. and Struik, J. H. A.,Guide to design criteria for bolted and riveted joints, John Wiley & sons, Inc., New York, U. S. A., (1974).

3) Godley, M. H. R. and Needham, F. H., Comparative tests on 8.8 and HSFG bolts in tension and shear, The Structure Engineer, 60 A, U. K., (1982).

4) Wallaert, J. J. and Fisher, J. W., Shear strength of high strength bolts, J. Struct. Div., Proc. ASCE, 91, (1965).

5) Bresler, B., Lin, T. Y. and Scalzi, J. B., Design of steel structures, John Wiley & Sons, Inc., New York, U. S. A., (1960).

6) Holmes, M. and Martin, L. H., Analysis and design of structural connections - reinforced concrete and steel, Ellis Horwood Limited, England, U. K., (1983).

7) Desi D. Vasarhelyi, Tests of gusset plates models, J. Struct. Div., Proc. ASCE , 97, (1971)

8) Davis, C. S., Computer analysis of the stresses in a gusset plate, Thesis presented to the University of Washington, at Seattle, U. S. A., (1967).

9) Whitmore, R. E., Experimental investigations of stresses in gusset plates, University of Tennessee, Engineering Experiment Station, Bulletin No. 16,

U. S.A. ,(1952).

10) Joints in structural steel work, Proc. Int. Conf. held at Teesside Polytechnic, Cleveland, U. K., (1981).

11) Conference on joints in structures, Institute of Structure Engineers, University of Sheffield, (1976).

12) Research Center for Housing, Building and Physical Planning, Egyptian Code of Practice, Steel Structures and Bridges,Cairo, Egypt, (1989).

13) American Institute of Steel Construction, Inc., AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings, U. S. A., (1969).

14) Robert C. Juvinall, Engineering considerations of stress; strain and strength, McGRAW-HILL book company, New York, U. S. A., (1967).