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Evaluation of Effective Lengths of Braced Double Angle Diagonals

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Abstract: The effective lengths required for bracing of double angles have been investigated at all fixity ranges. Results indicate that the effective length factors as used in codes and structural analysis are conservative. Except for certain cases as site conditions may allow, it may be predicted as required. Also these effective length factors as may be predicted are only reliable especially when the slenderness ratios of the double angles are high and in the range of 180-250.

Key words: Effective lengths, braced double angles, diogonals, evaluation, Nigeria

INTRODUCTION

The requirement that a structure performs effectively without failure during its design life is fundamental. Failure is implied in the sense of exceeding a certain limit state corresponding, for example, to a measure of un-serviceability or instability. Analysis carried out for steel structures are to obtain an adequate guarantee that during its life, it will satisfy the requirements laid down for it. At the same time, there should be the least possible expenditure of materials and that the structure does not fail at any point during its design life. The method of analysis that neglects load deflection on the lateral displacement of the structure where the axial loads in all members whose flexural stiffness is considered to contribute to the lateral stability of the structure is used herein. It aims at evaluating the effective lengths suitable for bracing of double angles diagonals by theoretical analysis of some sectional parameters in the design code using the BS5950 (2000) and to ensure that the sections evaluated does not exceed the limit states that are recommended in the code. In practice, it must be verified that these limit states are not reached when the design values of the parameters are used in the design models. In particular two verifications have to be made (Kirsch, 1981) which are as follows: the effects of design actions do not exceed the design resistance at the ultimate limit states and the effects of design actions do not exceed the performance criteria for the serviceability limit states.

Generally, specifications provide data upon which a material is supposed to be accepted for the design and construction of a structure. They may often be prepared by various professional associations and governmental agencies. Specifications represent a compromise between theoretical considerations and practical requirements and therefore are not a complete remedy for all design problems; for some structures or loading conditions they may lead to more conservative results than for others. Also, the value of loads and allowable stresses are generally based on past experience and test data and has to be reversed periodically to agree with the latest finding (Nowak and Collins, 2000).

It is on this basis that a theoretical approach to analyze the behaviour of various equal angle sections under varying effective length factors, k, from 0.5-2.0 at 0.1 intervals for braced lengths of 1000-6000 mm, with the intent of determining their safety and economy conditions. The evaluation is based on the specifications laid out in the code of practices, CEN (2003), AISC (1999), AISC (2005) and BS5950 (2000). However, the BS5950 (2000) steel sections are used. Mention is made on the effects of changing values of steel section properties, end conditions and provision of data that can facilitate the estimation of failure in braced double angles diagonals.

DIAGONAL BRACES

Diagonal braces are placed between lateral braces, in the same plane and between chords and webs of frames and trusses. They can also be said to be structural members, which are inclined and are usually carrying axial load which enables a structural frame to behave as a truss to resist horizontal loads. They are also referred to as cross-bracings, because of their appearance within the structure. As seen in a plane, diagonal braces used at the chords of planes act as a sort of 'flat truss' transferring load from the lateral braces to adjacent walls or adjacent struts. Individual chords of adjacent trusses may become part of this diagonal brace flat truss system. If this is the case, these trusses must be redesigned for added bracing loads (Abolhassan, 1998; MacGinley and Ang, 1998).

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Diagonal braces used at the web plane act as a sort of chevron brace for the truss system and transfer loads from the web lateral braces to the roof deck and bottom chord plane. Non-linear time history analyses of towers were performed to investigate the ability of the structure to withstand a major earthquake. The ground shaking was represented by a suite of three acceleration time histories developed for a 1000 years return period. Ground motions were developed to include consideration of near source effects, topographic effects and fault rupture direction (Albermani, 2005). The analysis of the hysteretic behaviour of these elements indicated that the tower remained stable for the 3 grounds motions and that the force and deformation demands on nearly all of the members and connections were acceptable, however, excess demands were predicted for some of the members and connections. He also noted that the experiment were subjected to inadequate compressive capacity due to buckling; lacing braces were inadequate to develop the yield strength in the braces where yielding was predicted to occur, while some double angle braced members had excessive compressive ductility. However, Albermani (2005) offered the solution which included the strengthening of the columns with the addition of cover plates, to provide them with adequate compressive strength so as to avoid buckling. Deficiencies in bracing connections were strengthened by replacement of the gusset plates and welding of the braces to the new or strengthened gussets. According to Nair (2005) the deficient lacing members were more difficult to reinforce, in order to increase compressive capacity. Conventional strengthening of the lacing by replacing them with stronger section members, or reinforcing the section itself would increase both the tensile and compressive load. Consequently, it was desired to find a way to increase the non-linear compressive deformation capacity of the lacing members without increasing their tensile stiffness or strength.

In some cases as noted, this could be achieved by converting guys braced with single diagonals to double angle bracing and in so doing, decreasing the slenderness of the braces without increasing their tensile capacity. However, due to the geometry of the tower and presence of extensive cabling, waves guys and other equipment related to the broadcast transmission use of the tower, it was not possible in all cases, the double angles braces were upgraded with the insertion of new I-section (wide flange or universal beam) members between the back-to-back legs. New wide flange reinforcement was connected to the double angles, using the bolts originally provided in the double angles for intermediate attachment, but with slotted holes in the new I-section beam members, so that the stiffness of the reinforced section matched that of the original members. This resulted (Albermani, 2005; Nair, 2005) in a cross section that had the approximate appearance and slenderness properties of a wide-flange section, while the axial stiffness and tensile strength of the original double angles were increased.

Strengthening techniques for braces: Deficient horizontal steel bracing systems capacity can be improved upon by Nair (1997), increasing the capacity of the existing members or by reducing unbraced lengths; adding new horizontal bracing members to previously unbraced panels (where feasible); adding a steel deck diaphragm to the floor system above the steel bracing and reducing the stresses in the horizontal bracing system by providing supplemental vertical-resisting elements such as braced frames.

There are several reasons for diagonal bracing of steel sections. These include, according to El-Tayem (1986) and White *et al.* (2005), keeping all components upright, straight and in place and transference of loads to other parts of the structure that can better resist these loads. Often, a single member cannot take certain loads over multiple truss systems or to stiffer supports if needed. If the axial load is too high for a given slenderness, weak axis bracing can reduce the effective length and increase member capacity.

Bracing is assumed to be perpendicular to the steel section members to be braced. For inclined or diagonal bracing, the brace strength (force or moment) and stiffness (force per unit displacement or moment per unit rotation) shall be adjusted for the angle of inclination. The evaluation of the stiffness furnished by a brace shall include its member and geometric properties, as well as the effects of connections and anchoring details.

GENERAL PRINCIPLES OF EFFECTIVE LENGTHS OF COMPRESSION MEMBERS

Compressive resistance: The axial load-carrying capacity P_r for simple compression members has been noted (BS5950, 2000; AISC, 1997, 2005; Steel Construction Institute, 2005) to be a function of its slenderness, material strength, cross-sectional shape and method of manufacture. This may be expressed as:

$$P_r = \frac{0.15\pi^2 EI}{kL^2} \quad (1)$$

Using BS5950 (2000) the compression resistance P_c is given in as;

$$P_c = A_g \rho_c \quad (2)$$

Where, A_g is the gross area and ρ_c is the compression stress. Values of ρ_c in terms of slenderness, λ and material design strength, F_y, are given in table 27 (a-d) of BS5950 (2000). Slenderness is defined (BS5950, 2000; AISC, 1997, 2005) as the ratio of the effective length, L_E, to the least radius of gyration, r_{min}, of a compression member, for example, column. The basis for table 27 in BS5950 (2000) is the set of four column curves from which adequate design parameters can be obtained. The table 27 noted above have resulted from a compressive series of full-scale tests, supported by detailed numerical studies on a representative range of cross-sections (BS5950, 2000; Steel Construction Institute, 2005). Four curves are used in recognition of the fact that for the same slenderness, certain types of cross-sections consistently perform better than others as struts. This is largely due to the arrangement of the materials but is also influenced by the residual stresses that form as a result of different cooling after hot rolling (BS5950, 2000; Steel Construction Institute, 2005). It is catered for in design by using the strut curve given as in table 25 of BS5950 (2000). The first is diagonal bracing. Design is therefore to consult Table 25 of BS5950 (2000) to see which of table 27 (a-d) is appropriate. For example, if the case is being checked in a universal column liable to buckle about its minor axis, table 27(c) should be used. Selection of a trial section fixes r and A_g. The geometrical length will be defined by the application required, so also slenderness ratio and thus permissible compressive stress, p_c, may be obtained. Basic design information relating column stress to slenderness is normally founded on the concept of a pin-ended member. But stated more precisely, this means members whose ends are supported such that they cannot translate relative to one another but are liable to rotate freely. Compression members in actual structures are provided with a variety of different support conditions which are likely to be less resistive in terms of translational restraints giving fixity in position. The usual way of treating this design is to use the concept of an effective column length which may be defined as the length of an equivalent pin-ended column having the same load-carrying capacity as the members under consideration provided with its actual conditions of support. This engineering definition of effective column lengths is also illustrated in the codes BS5950 (1985, 2000) and AISC (1999, 2005) for designs as required.

In determining the slenderness ratio, the geometrical length, L, is replaced by the effective length, L_E. Values of effective length factors, $k = L_E/L$, for a series of standard cases are provided in BS5950 (1985, 2000), BS5400 (1985), AISC (1999, 2005) and other design codes. However, some of the typical values given for structural designs are high, when compared with those values given by elastic stability theory. Hence one can easily see that these values are conservative and higher for those cases in which reliance is replaced on externally provided rotational fixity, which is in recognition of the practical difficulties on externally provided rotational and fixity conditions. This however, is in recognition of the practical difficulties of externally provided rotational condition of full fixity but is in recognition of full fixity. On the other hand, translational restraints of modest stiffness are quite capable of preventing lateral displacements. The distribution of internal member forces has been made on the assumption of pin-joints, some allowance for rotational end restraints when designing the compression members is appropriate. This apparent contradiction of regarding a structure as pin-jointed but using compression member effective lengths that are less than their actual lengths does have a basis founded upon an appropriate version of reality (Steel Construction Institute, 2005).

A typical equivalent for each case in terms of simple braced frames with pinned beam to beam connections is also in the codes. It is necessary to recognize that practical equivalents of pin joints may also be capable of transferring limited moments (Steel Construction Institute, 2005). This point is considered explicitly in BS5950 (2000). The fixity of the joints and the rigidity of adjacent members may be taken into account for the purpose of calculating effective lengths of compression members. BS5950 (2000) limits the maximum slenderness permissible in struts to the following: 180 for members resisting loads other than winds; 250 for members resisting weight and wind loads and 350 for members acting as tie but subject to load reversal under wind action.

In addition, members whose slenderness exceeds 180 must be checked for self weight deflection. If this exceeds the ratio (length/1000) the effect of bending should be taken into account. For the purpose of this work, members for bracing of double angle diagonals, a slenderness ratio of a range 180-250 for members resisting dead weight and wind loads is employed. It is therefore, necessary to know or predict in advance for design purposes, the failure mode of a braced diagonal member.

Failure modes of bracing members: The failure modes of braced members in order of their desirability are Abolhassan (1998):

- Yielding of gross area of member when subjected to tensile force.

- Overall buckling of bracing member when subjected to compression force.
- Bearing failure of bolt holes in bolted built-up bracing members.
- Yielding of stitches and batten plate in built-up bracing members.
- Buckling of individual elements as built-up members.
- Local buckling of bracing member cross-section.
- Slippage of bolts connecting the elements of member in built-up members.
- Fracture of effective net area of member.
- Block shear failure of members.
- Fracture of bolts or welds in built up members.

Also, the failure mode of connection of a bracing member to the gusset plate in the order of desirability are as follows (Abolhassan, 1998) slippage of built-in bolted connections; yielding of gross area of angles and plates used in the connection, bearing failure of bolt holes; local buckling of angles and plates used in the connections; fracture of effective net area of angles and plates in the connection; block shear failure and fracture of bolts and welds.

However, only the criterion of effective lengths of braced double angle diagonals have been evaluated in this work. This is because all the contributory criteria for the end conditions in terms of the practical applications of bolts, rivets and welds or their conditions can only result in the theoretical design value of end conditions and restraints. It is on this basis that theoretical values of the end restraints have been evaluated from 0.5-0.2 at an interval of 0.1 for double angles braced diagonally as used in practical applications for a span of 1-6 m.

RESULTS

These results are obtained by analysing steel sections of equal angles placed back to back and using the method of analysis that neglects load deflection on the lateral displacement of the structure to give values of slenderness ratio and required axial compressive strength. A plot of slenderness ratio against required axial compressive strength is shown in Fig. 1 (a-f). The slenderness ratio obtained from the analysis was used to determine the permissible compressive strength for angle sections 150×150×10 and 100×100×8 mm for grades of 275, 355 and 460N mm^{-2}, respectively. Results indicate that the bracing required for a storey height of 1-4 m, for section 150×150×10 mm, will not be economical since the slenderness ratio does not meet the minimum 180 required as shown in Fig. 1 (b-d). The calculated compressive strength is less than the permissible compressive

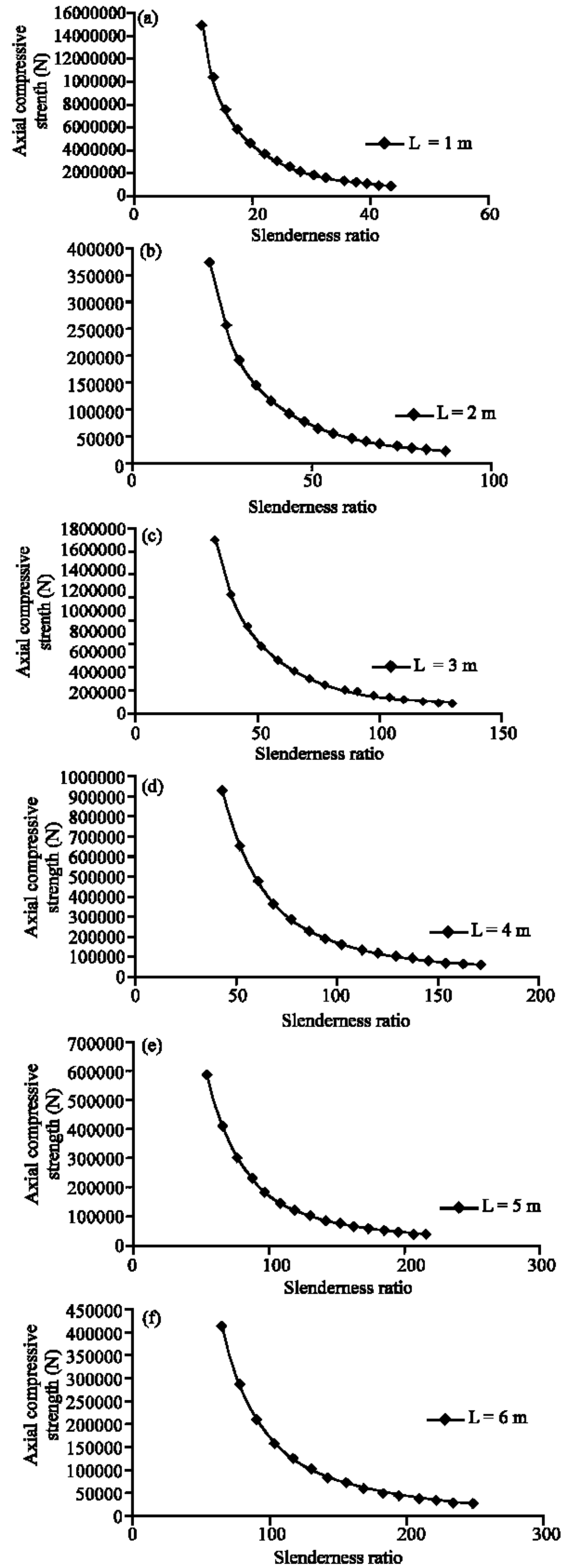

Fig. 1: Slenderness Ratio to Axial compressive strength (N) for L 150×150×10 mm

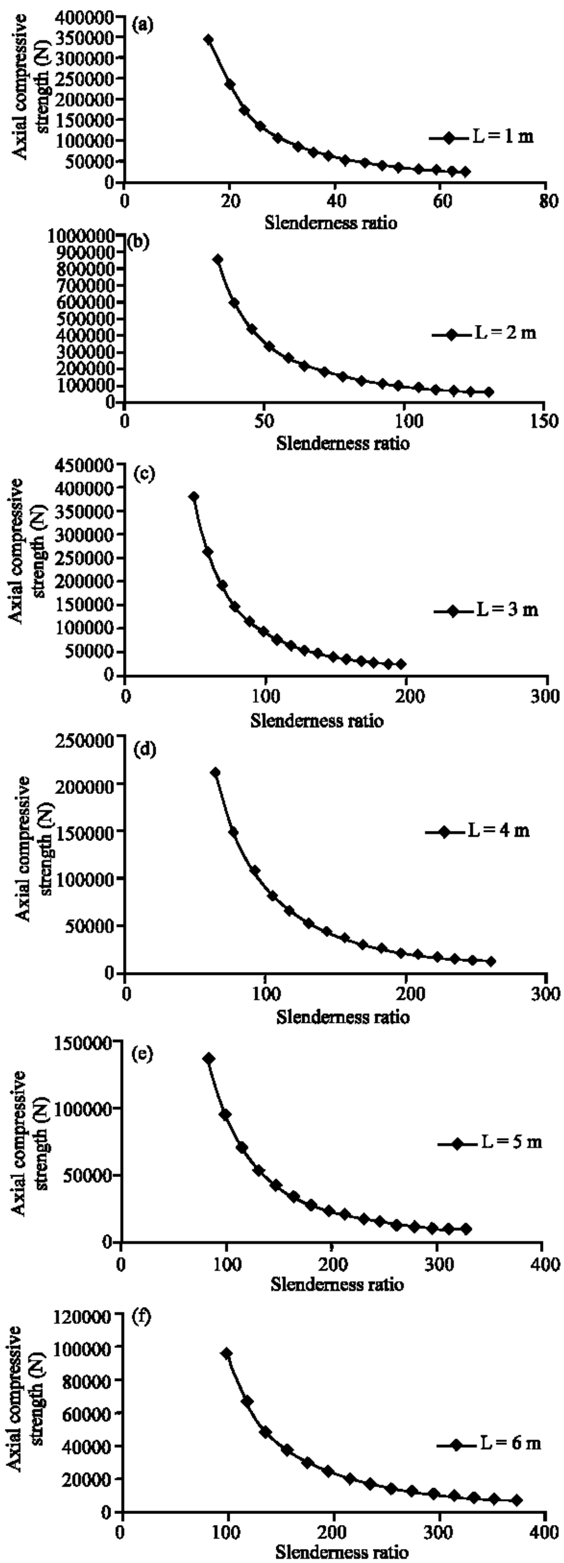

Fig. 2: Slenderness Ratio to Axial compressive strength (N) for L100×100×8 mm

strengths for a storey height of 5-6 m and adequate for steel grade 275N mm^{-2}. While for grades of 355 and 460N mm^{-2}, the calculated compressive strength is less than the permissible compressive strength for all the storey heights within the range of slenderness ratio considered; thus the section is adequate. Suitable effective length factors of storey heights of 2, 3 and 4-6 m are 1.2-2.0, 0.8-1.9 and 0.6-1.9, respectively. Using steel sections 100×10×8 mm for storey height of 1-2 m will not be economical, since none of the slenderness ratio meets the 180 minimum requirements, except at a storey height of 3 m as shown in Fig. 2 (a-c) with an effective length factor of 1.9. When the section is at a storey height of 4, 5 and 6 m, then the suitable effective length factors would be in the range 1.4-1.8, 1.1-1.6 and 0.95-1.34, respectively also shown in Fig. 2 (d-f). The calculated compressive stress is less than permissible for storey heights of 3-6 m for grades of 275 N mm^{-2} and greater than the permissible for all other grades.

CONCLUSION

In conclusion, the following observations were made from the study. Recommendations of BS5950 (2000) which is also as given in AISC (1999) suggests that an effective length factor of 1.4 would be suitable for section 100×100×8 mm for all storey heights, except for a storey height of 1-2 m which will be conservative. For steel sections 15×150×10 mm angles, an effective length factor of 1.7 would be suitable for design for all storey heights except for a storey height of 1-4 m which would be conservative or not economical. But for all other sections, the effective length factors can be predicted when faced with challenges on site as observed from results. It is also observed that, for high column lengths or storey heights say 4 and 5 m, an effective length factor of 1.65 will be satisfactory for bracing of these angles, which will result in the calculated axial compressive strengths not exceeding the permissible compressive strengths for 355 and 460 N mm^{-2} grades of steel.

Notations: The following notations were used in this work and are in general conformance to that used in the references:

k = Effective length factor.
E = Modulus of Elasticity.
r_{min} = Minimum radius of gyration.
I = Moment of inertia.
P_r = Required axial compressive strength.
ρ_c = Compressive strength.
λ = Slenderness ratio.
A_g = Gross area.
L = Storey height.
L_E = Effective length.

REFERENCES

Abolhassan, A., 1998. Seismic Behaviour and Design of Gusset Plates. Department of Civil and Environmental Engineering University of California, Berkeley. U.S.A.

AISC, 1999. Load and Resistance Factor Design Specification for Structural Steel Buildings. American Institute of Steel Construction, Chicago. U.S.A.

AISC, 2005. Specification for Structural Steel Buildings. American Institute of Steel Construction, Chicago. U.S.A.

Albermani, F., 2005. Upgrading of Towers Using a Diaphragm Bracing System. Seminar, Department of Civil Engineering, University of Queensland, Brisbane Australia, pp: 1-15.

British Standards Institution, 2000. BS5950: Structural Use of Steelwork in Buildings. BSI, London. UK.

CEN, 2003. Eurocode 3: Design of Steel Structures. Part 1.1: General Rules and Rules for Buildings. Final Draft prEV 1993-1-1. European Committee for Standardization.

El-Tayem, S., 1986. Effective Length Factor for the Design of X-bracing Systems. Eng. J., 23: 350-358.

Galambos, T.V., 1998. Guide to Stability Design Criteria for Metal Structures. Structural Stability Research Council. 5th Edn. John Wiley and Sons, New York, U.S.A.

Kirsch, U., 1981. Optimum Structural Design. McGraw-Hill, Inc., New York, U.S.A.

MacGinley, T.J. and T.C. Ang, 1998. Structural Steelwork: Design to Limit State Theory. 2nd Edn. Butterworth Heinemann, Oxford. UK.

Nair, R.S., 1997. Practical Application of Energy Methods to Structural Stability Problems. Eng. J. AISC., 34: 358-365.

Nair, R.S., 2005. Stability and Analysis. Modern Steel Construction, pp: 29-30.

Nowak, A.S. and K.R. Collins, 2000. Reliability of Structures. McGraw-Hill Inc., New York, U.S.A.

Steel Construction Institute, 2005. Steel Designers Manual. 6th Edn. Davison, B. and G.W. Owens (Eds.). Blackwell Publishing, Oxford. Britain, pp: 380-386.

White, D.W., A.E. Surovek, B.N. Alemdar, C. Chang, Y.D. Kim and G.H. Kuchenbecker, 2005. Stability Analysis and Design of Steel Building Frames: The AISC (2005) Specification and Beyond. Proceedings, Structures Congress, XXV: 1-49.