1978 05 23

TECHNICAL MEMORANDUM - MAY 1978

Re: BOLTED CONNECTIONS - MODES OF FAILURE

DESIGN STANDARDS = EITHER

1) CSA Standard S16.1 - 1974, "Steel Structures for Buildings - Limit States Design".

OR

2) "Specification for Structural Joints Using ASTM A325 or A490 Bolts". Approved by the Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation, (RCRBSJ), February 4, 1976.

ABSTRACT

1. Higher bolt shear and bearing values are permitted in both S16.1 - 1974 and RCRBSJ 1976.

2. Tests show different types of failure, rather than those of more traditional assumptions.

3. Coped beams may require extra attention in detailing, particularly bolt pitch and end distances.

Double Angle Beam-to-Column Connection

Figure 1 shows the typical beam-to-column double angle connection bolted to beam web and column flange with 3/4 inch diameter A325 bolts. Bearing-type connection is assumed.

Beam = W18 x 45; CSA G40.21 - 44W; w = 0.335"

Angles = 3-1/2 x 3-1/2 x 3/8

1. Design by CSA S16.1 - 1974

(a) Factored shear resistance of bolts, (Clause 13.1.1(b), S16.1) (Threads excluded)

$$V_r = 0.60\ \phi\ n\ m\ A_b\ F_u$$

$$= 0.60 \times 0.67 \times 3 \times 2 \times 0.4418 \times 120$$

$$= 128 \text{ kips}$$

(b) (i) In calculating the bearing resistance of the girder web, the designer might assume the end distance to be 3-1/4 in. (see Fig. 1) as this is the distance between the end bolt and the extremity of the section measured parallel to the reaction. The requirements of CSA S16.1 Clause 13.10 (c) cover both tearing out ($B_r = \phi\ t\ n\ e\ F_u$) and excessive deformation due to piling up of material ($B_r \leq 3\ \phi\ t\ d\ n\ F_u$). The presence of the flange material would undoutedly preclude a tearing out type of failure in this case and only the second of the two requirements will be checked, namely:

$$B_r = 3\ \phi\ t\ d\ n\ F_u$$

$$= 3 \times 0.9 \times 0.335 \times 0.75 \times 3 \times 65 = 132 \text{ kips}$$

(ii) Alternatively, a designer might recognize that, although only vertical forces are assumed to be transmitted, the rotation of the beam raises the possibility that the bolts might tear out toward the end of the beam. Using this end distance:

$$B_r = \phi\ t\ n\ e\ F_u$$

$$= 0.90 \times 0.335 \times 3 \times 1.75 \times 65 = 103 \text{ kips}$$

(c) The bearing strength of the angles, using the end distance 1.25" is:

$$B_r = 0.90 \times (2 \times 3/8) \times 3 \times 1.25 \times 65 = 165 \text{ kips}$$

(d) The factored shear resistance of the W18 x 45 (from the LSD Steel Manual, p. 5-123) is

$$V_r = 156 \text{ kips}$$

In order to make a comparison with test results, the unfactored resistances must be computed since different performance factors are used for bolt shear and bearing. Thus:

Bolts - shear $V_r/\phi = 128/0.67 = 191$ kips

Beam - shear $V_r/\phi = 156/0.9 = 173$ kips

Beam - bearing $B_r/\phi = 132/0.9 = 147$ kips or $103/0.9 = 114$ kips

Angles - bearing $B_r/\phi = 165/0.9 = 183$ kips

The design is therefore controlled by bearing in the beam.

2. Design by RCRBSJ Specification.

(a) Bolt shear stress (no threads in shear plane) = 30 ksi

Therefore $V = 3 \times 2 \times 0.4418 \times 30$

$= 80$ kips

(b) Bearing stress.

(i) $\sigma_b = 1.5\ F_u = 1.5 \times 65 = 97.5$ ksi

$B = 3 \times 0.335 \times 0.75 \times 97.5 = 73.5$ kips

(ii) or, taking L = 3-1/4"

$$\sigma_b = \frac{LF_u}{2d} = \frac{3.25 \times 65}{2 \times 0.75} = 105.6 \text{ ksi}$$

(iii) or taking L = 1-3/4", as was done in 1(b) (ii) above,

$$\sigma_b = \frac{1.75 \times 65}{2 \times 0.75} = 75.8 \text{ ksi}$$

Therefore $B = 3 \times 0.335 \times 0.75 \times 75.8 = 57.1$ kips

Although this is an allowable stress type of formulation, the factor of safety used is the same (2.0) for both bolt shear and bearing. Hence, bearing controls the design. To determine the ultimate strength, twice that value would be taken. (Fisher and Struik, "Guide to Design Criteria for Bolted and Riveted Joints", John Wiley, 1974), that is $57.1 \times 2 = 114.3$ kips for an end distance of 1-3/4" or, for $\sigma_b = 97.5$ ksi, $= 73.5 \times 2 = 147$ kips.

TEST RESULTS

1. Uncoped Beam Test

The test assembly shown in Figure 1 was tested in high shear and low moment to closely duplicate an actual girder connected with bolted double angles with an *uncoped* top flange.

Test load obtained in connection = 147 kips

This test load is 10% less than that predicted when adjusted for actual F_u and web thickness of beam.

At failure there was a tear from the top of the bottom bolt hole generally horizontally to the edge of the beam web, that is, the failure was not consistent with the assumption that the reaction is only vertical.

In view of this above observation, it is recommended that the check of bearing resistance include the investigation of possible failure between the bolts and the *end* of the beam.

2. Coped Beam Tests

A second series of tests modelled a top flange cope of 6 inches in length and of a depth sufficient to leave an end distance, e, (See Figure 2) of 3 bolt diameter (2.25 inches) thus permitting use of the highest bearing stress or resistance as appropriate to the particular design standard.

For three tests the following failure loads for the connection were observed 112^K, 113^K, and 108^K for an average value of 111 kips.

Referring to Figure 2, failure was by tension across line B-B and shear along line A-A. The researcher has termed this type of failure a "block shear" failure.

COMMENTS

A mode of failure, termed "*block shear*", now may control the ultimate strength of bolted connections when the flange is coped because of the use of higher bearing and shear strengths. Of special concern are joints where, referring to Figure 2, the edge distance, e_o; the end distance, e and the pitch p are all small. Increasing these dimensions should improve the ultimate behaviour to the point where bearing or bolt shear may again control the ultimate strength of the connection.

1. How to Calculate "*Block Shear*" ?

The researcher has suggested the following rule to check for block shear.

Ultimate resistance = Sum of shear resistance along A-A plus the tensile resistance across B-B (See Figure 2).

$$= w \left[L - (n-1/2)\, d\right] 0.53\, F_u + w \left(e_o - \frac{1}{2} d\right) F_u \; \ldots \; (1)$$

where

w = thickness of beam web

L = length of shear plane A-A

n = number of bolts

d = bolt hole diameter

F_u = specified minimum tensile strength

e_o = edge distance from centre of bolt hole to edge of web taken horizontally

Modified for use with S16.1 - 1974 this might become:

$$\text{Factored Resistance} = w \left[L - (n-1/2)\, d\right] 0.53\, \phi\, F_u + w \left(e_o - \frac{1}{2} d\right) \phi\, F_u \; --- \; (2)$$

where ϕ = 0.90

Modified for use with RCRBSJ Specification this might become:

$$\text{Allowable} = w\left[L - (n - 1/2)\ d\right]\ 0.53 \times 0.60\ F_u + w\left(e_o - \frac{1}{2}\ d\right)\ 0.60\ F_u \quad \ldots (3)$$

Using equation (2) above for the *design* of this connection would result in a limiting factored resistance of:

$$0.335\left[8.25 - (3 - 1/2)\ 0.8125\right]\ 0.53 \times 0.90 \times 65$$

$$+\ 0.335\ (1.75 - 1/2 \times 0.8125)\ 0.9 \times 65$$

$$= 65 + 26$$

$= 91$ kips $< 128^{K}$ for bolt shear

Alternately, *designing* using the RCRBSJ specification (Eq n (3)) would result in an allowable load of:

$$0.335\left[8.25 - (3 - 1/2)\ 0.8125\right]\ 0.53 \times 0.60 \times 65$$

$$+\ 0.335\ (1.75 - 1/2 \times 0.8125)\ 0.60 \times 65$$

$$= 43 + 18$$

$= 61$ kips > 57.1 kips for bearing (If $\sigma_b = 1.5\ F_u$, had been used, then

61 kips $<$ 73.5 kips)

Comparing this suggested equation with the real test beam using the actual measured properties and dimensions gives:

$$0.305\left[8.25 - (3 - 1/3)\ 0.8125\right]\ 0.53 \times 79 + 0.305\ (1.75 - 1/2 \times 0.8125)\ 79$$

$$= 79.4 + 32.4$$

$= 112$ kips vs 111 kips average of 3 tests

Influence of Increased Pitch

If the same 3 bolts were installed with a pitch of 6 inches then Equation (2) would yield a factored resistance of:

$$0.335\left[12.25 - (3 - 1/2)\ 0.8125\right]\ 0.53 \times 0.90 \times 65$$

$$+\ 0.335\ (1.75 - 1/2 \times 0.8125)\ 0.9 \times 65$$

$$= 106 + 26$$

$= 132$ kips $>$ 128 kips for shear.

SUMMARY

The use of higher bearing and bolt shear values permitted in S16.1 - 1974 or RCRBSJ - 1976 can cause the mode of failure in the connection to become one of "block shear" when the top flange is coped and the connection is one where the pitch, end distance and/or, edge distances are small.

Both the "block shear" mode and the tear-out between the bolt and the end of the beam should be considered when dealing with coped members.

The proposed equations for "block shear" should be used only for connections of normal proportions, since only a limited number of tests have been performed.

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TEST NO. 1 -- SERIES I

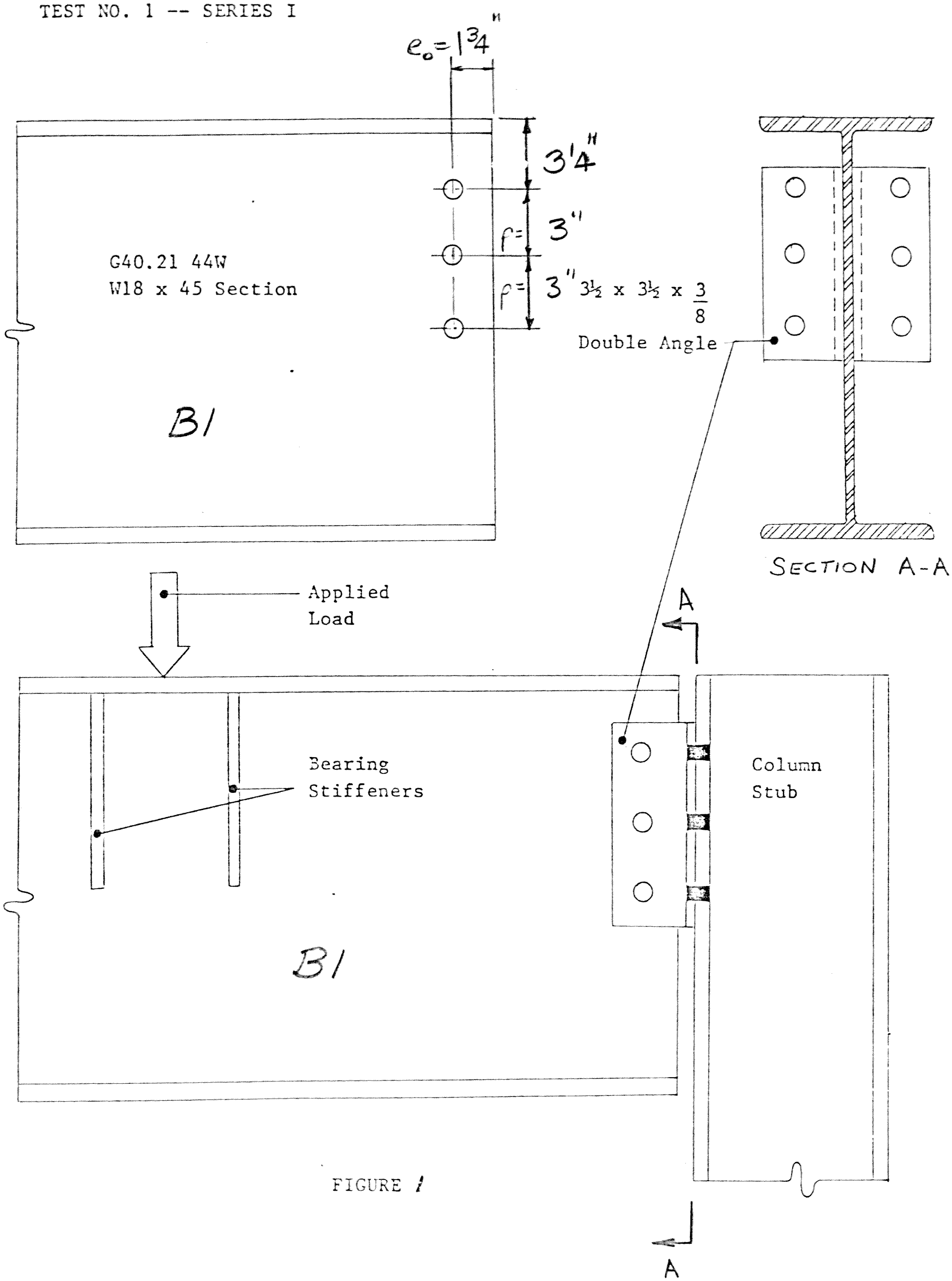

FIGURE 1

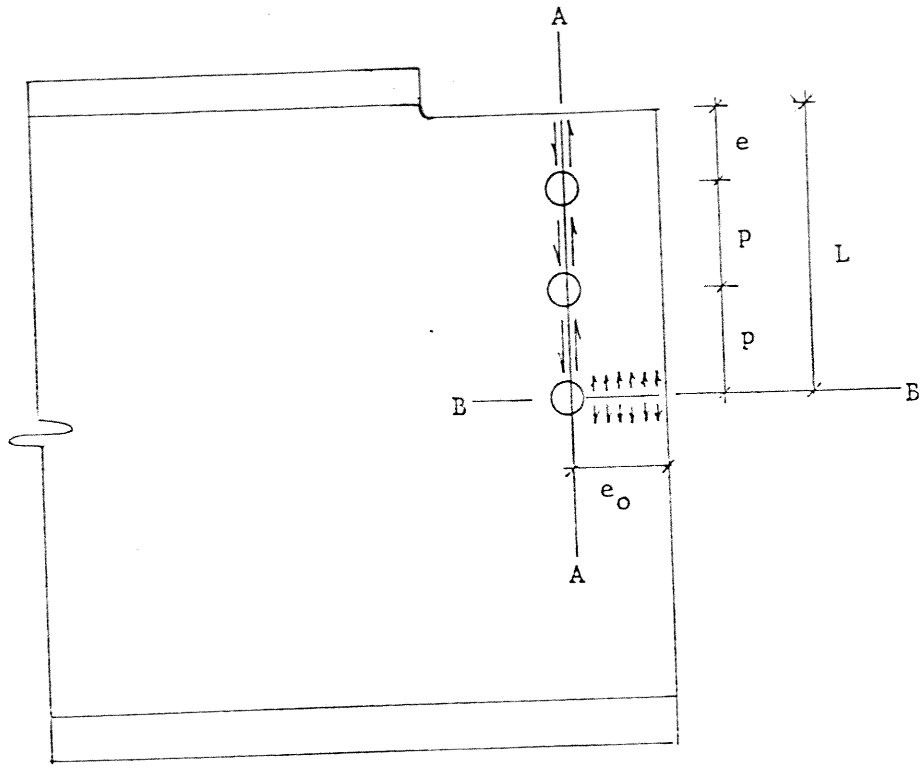

FIGURE 2