AN EXPERIMENTAL COMPARISON OF STEEL COLUMN ECCENTRICITIES PRODUCED BY GUSSET AND SEATED BEAM CONNECTIONS

A Thesis

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(Civil)

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by

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Acknowledgment

The writer would like to express his sincere appreciation to Dominion Bridge Company Limited for their donation of the fabricated steel required in the construction of the Test Frame and the Load Frame.

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INTRODUCTION

The original objective of this thesis was to investigate the amount of eccentricity which must be provided for in the design of structural steel columns in buildings. However, after the experimental appartus was designed, fabricated and erected, time was limited. Therefore, the problem to be investigated was limited to an experimental comparison of steel column eccentricities produced by gusset and seated beam connections. It is hoped, however, that the original objective shall be reached, through further experimentation by others at a later date.

As both the gusset and seated beam connections are used in structural design, an experimental comparison as to their ability to transfer moment, caused by eccentric loading, was made.

First, it was necessary to design a Test Frame, on which the experiment could be carried out. In addition, a Load Frame was designed, in which the Test Frame might be loaded. This was done, and the plans turned over to Dominion Bridge Co. Limited, who fabricated the steel. Upon delivery, the steel was erected in the Materials Testing Laboratory.

The loads were applied to the Test Frame by means of Tension Bars which were first calibrated, in order that the amount of applied load might be known. Six gauges were placed at one section on the column leg and a preliminary test run to establish the points where gauges would be most advantageous. Gauges were then placed at six sections, distributed over the length of the column, and the frame was then ready to be tested.

Three tests were run as follows:

Test No. 1. Using the seated beam connections; a load was centrally applied on the lower beam of the Test Frame.

Test No. 2. Using the gusset connections; a load was centrally applied on the lower beam of the Test Frame.

Test No. 3. Using the seated beam connections; a load was centrally applied on the upper beam of the Test Frame.

For these three tests, the bending moment distribution in the column of the Test Frame was calculated, and a discussion of the results made.

A photograph of the Load Frame and the Test Frame, as assembled, immediately follows their design on page 15.

TEST FRAME

In the design of the Test Frame it was attempted to design a model, which would, as far as possible, resemble a bent of a two storey building. The storey heights of the model were made 4 ft. 6 ins. and beam spans 6 ft. 0 ins.

For the columns, it was necessary to use two angles, short legs back to back, in order to obtain an l/r ratio reasonably close to that of actual columns.

In the design of the beams, loading was considered as being applied at the one-third points. Two sets of beams, which were to be made up of channels back to back, were designed. One set was made stiff enough so as to have very little deflection and give a fairly well distributed load on the seat angle. The other set was designed for a stiffness in relation to the column, similar to that for a prototype, which would give results similar to actual field conditions. Due to a shortage of the smaller channels required in the latter set, the Dominion Bridge Co. Limited substituted channels a little larger than those requested, changing the column and beam stiffnesses from the desirable ratio.

FINAL DESIGN OF TEST FRAME

Columns

Using 2 angles $2\frac{1}{2} \times 2 \times \frac{1}{4}$ s.l.b.b.

Unsupported length 4'-6" or 54"

Least $r = 0.59$

$l/r = 54/0.59 = 91.5$

For an unsupported length of 4'-6" the allowable concentric load is 27 kips.

The l/r ratio is suitable, as it is reasonably close to that of actual columns, therefore the 27 kips allowable concentric load will govern.

Apply loads of 2.5 kips at the one-third points.

Eccentricity of load = $1.667 + 0.54 = 2.11"$

Equivalent concentric load = $P + M\,Bx$

$= 2.5 + (2.5 \times 2.11)\ 4.25 = 25.0$ kips.

where $Bx = \dfrac{A}{SM.} = \dfrac{2.12}{0.5} = 4.25$

Therefore, this loading gives an equivalent concentric load approximately equal to the allowable of 27 kips.

Apply loads of 4.0 kips at the one-third points.

Equivalent concentric load = $P + M\,Bx$

$4.0 + (4.0 \times 2.11)\ 4.25 = 39.9$ kips.

This loading is 148 % of the 27 kips allowable.

As this load is well under the failure load, it may be used to exaggerate the column bending conditions.

Beam Channels

The first set was designed for a stiffness in relation to the column, similar to that for a prototype.

Apply loads of 2.5 kips at the one-third points.

Use 2 channels 4 x 1-5/8 at 5.4 lbs.

$I = 2 \times 3.8 = 7.6''^4$

$M = 2500 \times 2 \times 12 = 60{,}000''\#$

$S = \frac{60{,}000 \times 2}{7.6} = 17{,}200$ p.s.i.

The second set was designed stiff enough so as to have very little deflection.

Apply loads of 4.0 kips at the one-third points.

Use 2 channels 6 x 2 at 8.2 lbs.

$I = 2 \times 13.0 = 26.0''^4$

$M = 4000 \times 2 \times 12 = 96{,}000''\#$

$S = \frac{96{,}000 \times 3}{26.0} = 11{,}100$ p.s.i.

The beams substituted for the first set are as follows;

Apply loads of 2.5 kips at the one-third points.

2 channels 5 x $1\frac{3}{4}$ at 6.7 lbs.

$I = 2 \times 7.4 = 14.8''^4$

$M = 2500 \times 2 \times 12 = 60{,}000''\#$

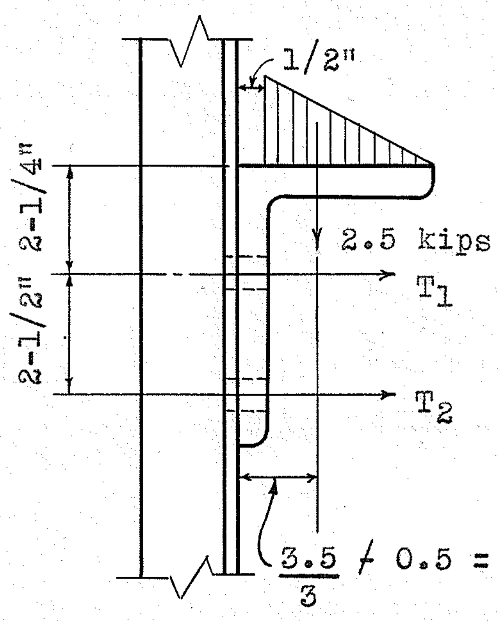
$S = \frac{60{,}000 \times 2.5}{14.8} = 10{,}100$ p.s.i.

Seat Angles

Use four 5/8 inch diameter bolts.

Apply loads of 2.5 kips at the one-third points.

Design for flexure of vertical angle leg at net section on upper rivet line.



Use 6 x 4 x 5/8 angle

$M = 2500(1.667 - 0.312)$

$= 3390"\#$

Length of angle required

$$= \frac{6\ M}{t^2 f}$$

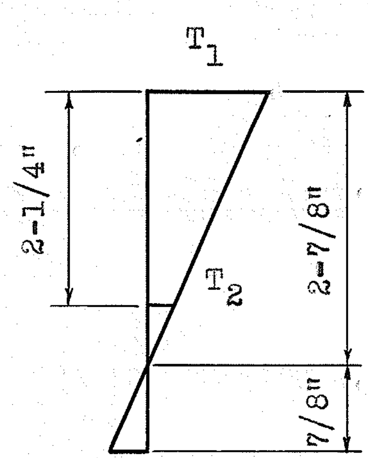
$$= \frac{6 \times 3390}{(0.625)^2 \times 18{,}000} = 2.9"$$

Length of angle available

$2(2.5) + 0.25 - 1.25$

$= 4.0"$ (Satisfactory)

Check for shear and tension in bolts.



$S_s = \frac{2500}{4} = 625$ #/bolt.

Allowable in shear = 3070 #/bolt.

(Satisfactory)

Moment of Inertia of bolt areas and compression area = $1/3 \times 4.75 \times (0.87)^3 + 2 \times 0.307(0.37^2 + 2.87^2)$

$= 6.04"^4$

$$\text{Bolt tension} = \frac{2500 \times 1.667 \times 2.87}{6.04} = 1980 \text{ p.s.i.}$$

Allowable in tension = 18,000 p.s.i. (Satisfactory)

Apply loads of 4.0 kips at the one-third points.

Design for flexure of vertical angle leg at net section on upper rivet line.

Consider same 6 x 4 x 5/8 angle

$M = 4000\ (1.667 - 0.312) = 5420''\#$

$$\text{Length of angle required} = \frac{6 \times 5420}{(0.625)^2 \times 18{,}000} = 4.62''$$

Length of angle available = 4.0"

This will be satisfactory since at this loading the columns are at 148 % of their allowable.

Check for shear and tension in bolts.

$$S_s = \frac{4000}{4} = 1000\ \#/\text{bolt.}$$

Allowable in shear = 3070 #/bolt. (Satisfactory)

Moment of Inertia of bolt areas and compression area = $6.04''^4$

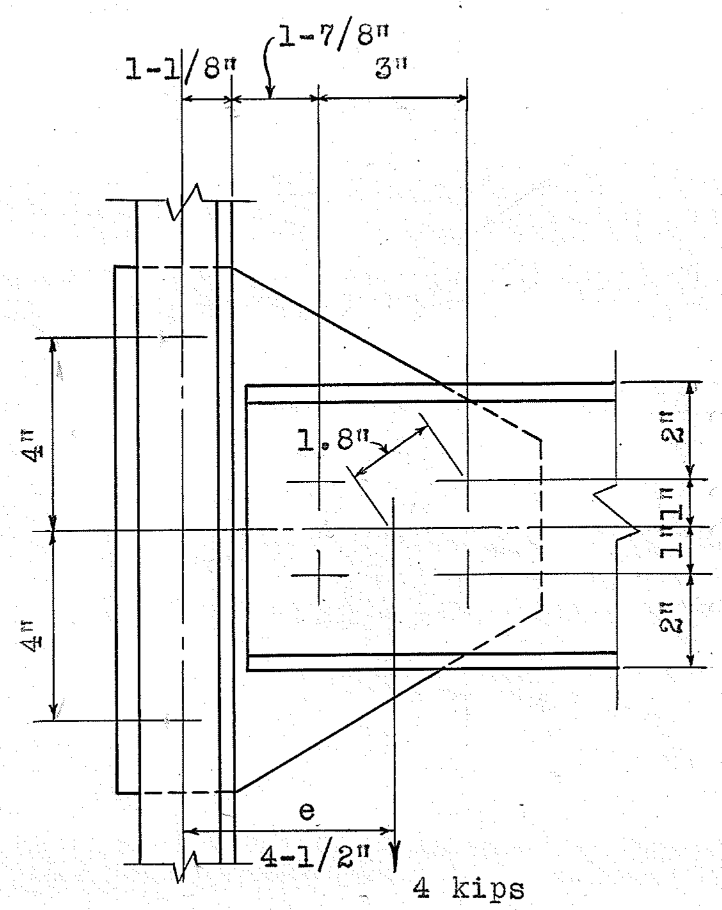
$$\text{Bolt tension} = \frac{4000 \times 1.667 \times 2.87}{6.04} = 3{,}170 \text{ p.s.i.}$$

Allowable in tension = 18,000 p.s.i. (Satisfactory)

Gussets

Use 5/8 inch diameter bolts.

Apply loads of 4.0 kips at the one-third points.



Use 3 bolts in the column.

$$S_s = \sqrt{\left(\frac{P}{n}\right)^2 + \left(\frac{Pec}{\Sigma y^2}\right)^2} = \sqrt{\left(\frac{4}{3}\right)^2 + \left(\frac{4 \times 4.5 \times 4}{2 \times (4)^2}\right)^2}$$

$$= \sqrt{1.78 + 5.06} = 2.62 \text{ kips/bolt.}$$

Allowable in shear = 2.93 kips/bolt.

(Satisfactory)

Use 4 bolts in the beam.

$$S_s = \sqrt{\left(\frac{4}{4}\right)^2 + \left(\frac{4 \times 4.5 \times 1.8}{4 \times (1.8)^2}\right)^2}$$

$$= \sqrt{1.0 + 6.25} = 2.69 \text{ kips/bolt.}$$

Allowable in shear = 2.93 kips/bolt.

(Satisfactory)

A detailed drawing of the Test Frame is shown on page 8.