1953

The behavior of welded portal frames, notes for talk, Jan. 1953

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Notes for Talk
on
THE BEHAVIOR OF WELDED PORTAL FRAMES

AWS, Philadelphia
Oct. 21, 1952

I. INTRODUCTION:

As part of a project being sponsored jointly by Welding Research Council and the Navy Department, two single-span welded rigid frames were tested through the elastic and plastic ranges. These tests had been preceded by several years of theoretical and experiemental study of the component parts of frames - the columns, beams, and connections. Although such studies of isolated components had by no means been completed, it was considered worthwhile by the Lehigh Project Subcommittee to proceed with the testing of actual steel frames using commercially-available rolled shapes and welded in a fully continuous manner. Having studied the behavior of beams and columns as separate elements, is there anything that happens to alter their fundamental behavior when these parts are joined together by welding to achieve continuity? The objectives of the tests and of this talk today are consequently as follows:

(1) How do the behavior of isolated structural elements compare with the same components when tested as part of a complete frame.

(2) Are frames as strong as predicted by available theory based on the behavior of beams, columns and connections.

(3) What further can we learn with regard to the application of plastic design methods.

II. TEST METHODS AND DESCRIPTION

The test technique was described in a paper presented before the Society for Experimental Stress Analysis last May, and that material will not be repeated here except to point out the arrangement of the frames and of the loading on them.

SLIDE 1

This slide shows the dimensions of the frame in which a column height of 7-feet was used with a girder span of 14-feet. Concentrated loads were applied at the three-eighths points. The frames were tested with the column bases in the pin-end condition and rollers were provided at one end in order that the horizontal reactions could be measured. Sway to the side was prevented in this first series of simple tests.

This slide also shows the advantages of continuous constructinn by welding. At the bottom is shown the simple beam bending moment diagram; when the same loads P are applied to the complete frame, there is a resultant reduction in the magnitude of maximum bending moment.

SLIDE 2

This figure is a photograph of the set-up. Loads were applied by hydraulic jack and measured with tube dynamometers. This is frame 1 (8WF40), the second frame consisting of uniform section 8B13 ~~xxxxxxx~~ shape. This picture was taken at the end of the test when the center deflection was about 10% of the girder span, most of the deflection occuring in the plastic range of stress.

III COMPONENTS

In order to ~~xxxxxx~~ evaluate the behavior of a structural component it is necessary to examine the moment-rotation characteristics exhibited by it.

SLIDE 3

In this slide in which moment is plotted as a function of rotation per unit length, curve A is a representation of the Simple Plastic Theory applied to a beam under pure bending. Component B is too flexible in the elastic range and develops insufficient strength in the plastic range. Member D, while possessing adequate elastic stiffness and strength in the plastic range, has too little Rotation Capacity . . . the ability to deform plastically at the plastic hinge moment. Member C would be adequate; i.e. it has sufficient elastic stiffness, plastic strength, and rotation capacity.

The behavior of the beams will now be considered.

SLIDE 4

The beam is of course the top portion of the frame. The solid line represents the nominal bending moment diagram. Presence of horizontal thrust at the column bases affects the experimental moments since this same thrust is applied as ~~xxxx~~ an axial force at the ends of the girder. Thus, when the frame deflects, there is an additional increment of moment equal to the thrust multiplied by the deflection at the cross-section. The dotted line in the figure shows the bending moment diagram when the load on Frame 1 was 50-kips at each loadppoint, the denter deflection being $7\frac{1}{2}$ inches. The increase in center-line moment was more than 10%.

SLIDE 5

In this slide moment is plotted against ϕ. The result for test B1 (a simple beam) is shown in comparison with the M-ϕ curve at the center of the portal frame girder. This slide also illustrates the influence of axial thrust. In the inset are shown stress-distributions for the elastic-limit case and for the plastic limit. Zero axial thrust is compared with a case in which axial thrust is present. It is seen that the influence of axial load is to decrease the bending moment at which initial yielding and plastic collapse ~~shown~~ should occur.

This is indicated also in the theoretical curves that have been plotted. The upper is for zero axial thrust; the lower is for the frame test (using the maximum thrust present at the end of the test). The influence for this test is seen to be rather small. It is noted that nearly the same difference is observed in the experimental curves.

When the straining is continued sufficiently, as was done in the case of the frame test, then the section carries a moment greater than that corresponding to a plastic hinge neglecting axial thrust.

The next slide shows the center portion of the two frames in the region under pure bending.

SLIDE 6

Here the difference in behavior of the 8WF40 shape (upper) differs from the lighter 8B13 (lower) in that the former deforms considerably more under bending than the latter. The light section collapsed due to local buckling of flange and web shortly after the plastic range was reached.

This slide also shows the influence of axial thrust, since the neutral axis (revealed by the flaking of mill scale) has shifted away from the compression side and into the zone originally in tension.

SLIDE 7

This slide shows one each of the corner connections used in Frames 1 and 2. To the left is the 8B13, the 8WF40 being to the right. These connections were formed by welding the column to the lower flange of the beam, installing the vertical and diagonal stiffeners on each side of the girder web, and completing the joint with the welded end plate.

Evident is the additional deformation of T1 beyond that exhibited by T2. Again, the latter was limited by local buckling of flange and web shortly after the elastic limit was reached. Evidence of yielding in the girder web, in the connection web and in the column is seen. (In the latter, once again we can see how the neutral axis has shifted due to the presence of axial thrust. In this case, the shift is greater than in the ~~case of the~~ girder because "P" is greater than "H". There is an obvious differenc in the degree of local buckling.

SLIDE 8

This slide shows the Moment-rotation characteristics of the two connections seen in the previous slide. In each case, the theoretical curves are shown by dashed lines, the solid curves representing the experiments. T1 carries greater load than predicted by the simple plastic theory due to strain-hardening. T2 collapsed after reaching the "hinge" moment, a collapse that was due to local buckling shown in a previous slide. Even so, the rotation capacity was good.

SLIDE 9

Here are compared two identical connections, one a part of a frame, the other having been tested separately as shown in the inset. The theoretical moment-rotation curve (dotted) is approached in the plastic range by both connections. The agreement of Connection L with T2 Frame is excellent in the early range. The small discrepancy in the plastic range is possibly due to difference in efficiency of lateral support.

Having examined the behavior of beams and connections, the behavior of the columns will now be studied.

SLIDE 10

In the skethc on this M-Θ curve the frame is shown in its deformed condition. The loading on the columns is also shown in an inset. Four theoretical values here are (1) the plastic hinge moment, (2) the plastic moment taking into account the influence of direct stress, (3) the yield moment neglecting axial thrust, and (4) the theoretical yield moment considering this factor. The column carried moment greater than that predicted by the theory that neglects direct stress.

SLIDE 11

Although no exact duplicate of the column in the frame was available in test, this M-Θ curve is of a similar isolated column. The tendency here is the same as that in the frame. The column carries more bending moment than predicted.

Finally, we will consider the behavior of the frame as a whole. The over-all behavior is indicated by the load-deflection curve of the next slide.

SLIDE 12

T1 showed a strength considerably greater than the plastic load and this is due to strain-hardening. T2 came within 1% of the predicted plastic load and then collapsed due to the local instability that has been seen in the earlier slides. Even so, this behavior is heartening since the 8B13 has very pood local buckling characteristics. The major difference, of course, is in energy absorption; this is shown in the next slide.

SLIDE 13

On a non-dimensional basis it is seen here that there is a considerable difference in the energy absorption of the two frames. This is of importance when considering the design of buildings subjected to blast loading.

SLIDE 14

This table summarizes the behavior of of the two frames with respect to ~~twex~~ the yield strength and the maximum strength. For each frame, observed and computed values are compared with reference to various criteria. The sketch at the bottom illustrates the comparisons made. ~~Eixxtxzxithzxexpxz~~ The first yield line was observed in frame 1 at 56% of the computed yield load. The ratio for Frame 2 was 0.42. The General Yield strength, defined by the graphical construction shown was almost identical with the predicted yield strength. (1.05 and 1.01). The load reduction at the initial yield deflection is 16.5% ~~and~~ for Frame 1 and 11.5% for Frame 2.

Directing our attention to the Maximum strength, the frames carried 33% and 37% more load than the predicted yield load. ~~Summarizex~~ Frame 1 carries 10% more load than the plastic load, and frame 2 reaches to within 1% of the computed value.

SUMMARY

1. The behavior of isolated structural components agrees well with the results of frame tests. This includes the elastic and plastic region, but it must be remembered that the axial thrust was relatively low.
2. Concerning the strength of the frames, they were as strong or stronger than predicted by the Simple Plastic Theory that neglects direct stress. This confirms that when the axial thrust is relatively low, it may be neglected without serious error. These tests, of course, do not establish the range.
3. At working loads determined according to present rules, both frames had exceeded the yield-point. By use of plastic desigh methods, the working load would have been raised more than 10%, and in this case the extent of yielding would not have increased significantly....and the deflection would have been within allowable limits.
4. The importance of local and lateral instability in the plastic range must not be overlooked. Nevertheless, we can continue to give serious consideration to procedures of plastic analysis as applied to strudtural design.

D3

~~23~~

~~2~~

~~24~~

~~30~~

~~11~~

Slide List

No.	Old Slide No.	P.R. Fig No.	Photo No.	Description
1.	205D.3-12	D.3-23 *		T1,T2 - Dimensions and loading. Moment diagram
2.	205D.3-1	D.3-2	*	T1 - Photo of set-up. large Defl.
3.	205.18-	.18-3		Rotation Capacity
4.	205D.3-13	D.3-24		Moment diagram on frame. Influence of axial thrust
5.	--	--	--	Comparative M-ø curves, beam B1 and F1
6.	205A.5-6	205.13- -*	*	Photo of T1, T2 center portion under bending.
7.	205A.5-20		*	T1, T2 connections
8.	---	--	--	M-θ comparative curves for connections from T1, T2
9.	--	--	--	M-θ comparison T2 and Conn L
10.	--	--	--	M-θ of column in frame T1
11.	205.18-	205.18-14	--	M-θ of column test
12.	205D.3-20	D.3-30	--	Load-deflection T1, T2
13.	~~205~~ --	205D.3 27	--	Load-Deflection (non-dimensional)
14.	--	--	--	Table of results

DISCUSSION

No.	Old Slide No.	P.R. Fig No.	Photo No.	Description
A.	*	B8-13	--	M-ø of continuous beams near suppt.
B.	--	BSB Diss p.117a	--	M-θ for connection in elastic region
C.	--	PR-4(I) 54	--	Equivalent Length
D.	*	205.18-15	*	Column (local buckling)
E.	205D.2-	--	--	Load-deflection: Comparison of Frame and continuous beam
F.	*	205D.3-11	*-	Photo of lateral support T2

PH B.bl. Series #2 Mental Health Pamphlets & Reprints

205D.2

NOTES FOR TALK

Purpose of work & talk

(a.) Coordinated study
(b.) ~~Compare behavior of comp. with parts in an actual frame.~~
(c.) ~~Is the frame as strong as predicted by available theory.~~ Are deformations predictable on basis of theories?
(d.) ~~are there any new limitations~~

Theme: Bring in the welding angle

BIG **IF** (1)

Description of Tests

described in SCSA paper

C2
C16

Results

(a) Components [compare (a) Theory (b) Frame result {T1, T2} (c) Isolated component result]

Theme: (a) Do plastic hinges develop (b) are they maintained?

0. Importance of M-ϕ
1. Beams ✓ — Infl. of axial thrust
2. Connection
3. Column

(b) Overall behavior (deflection & strength)

1. Elastic
2. Yield Strength
3. Plastic behavior (a) bending (b) shear
4. Plastic Instability (a) The types (b) Local (c) Lateral

C19
C22
C24
C25 C26
C28

Summary

Possible ~~lim~~ further limitations?
Suitability of welding

C2, 16, 6, 18, 6a, 8a 9, 10, 11, 11a, 12, 12a, 13, 19, 22, 24, 25, 26, 28, 30

(1) 205.18

M_p P_1
M_{pc} P_2

card No.

$\frac{M_p}{h} = H_p$

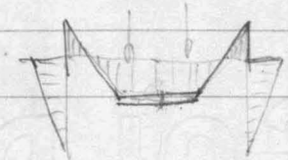
$\frac{M_{pc}}{h} = H_{pc}$

Beam comparison

Photo of center part $\begin{cases} T1 \\ 205B - 205.18\text{-}18b \text{ ?? but better.} \\ 203 \end{cases}$? C26

Axial thrust influence

M-ϕ relationship $\begin{cases} C18? \\ C6 \\ C6a \end{cases}$



7.5"

Connection Comparisons

Photo of Conn M (205C) C —

" " T1 T2 C28

Comparison of M-θ

Theory vs Test & Conn.(L) vs. Frame T1 C9

Equiv. length $\begin{cases} C9 \\ C10 \end{cases}$

Complete M-θ (8B13) C11

8B13 & 8WF40 C11a

Column comparison

Photo of cols T1 : # 127 T2: 216 C12 C12a

M-θ C13

ε_s