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Plastic Design in High Strength Steel

THE EXPERIMENTAL BASES FOR PLASTIC DESIGN — a survey of the literature

by

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Fritz Engineering Laboratory Report No. 297.3

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--A SURVEY OF THE LITERATURE

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I. INTRODUCTION

Plastic design methods possess an elegant theoretical basis[iii,iv]. The theories require the use of a material which is rigid-plastic in the plane of loading and rigid out of the loading plane. An actual structure is not composed of such a material and, in addition, it will contain a number of unknown structural imperfections. To check whether plastic design can be applied to such structures it is necessary to subject the theory to experimental confirmation.

The purpose of this report is to present all those tests which form the basis of the plastic method of design. In the light of these tests it is then possible to draw conclusions as to the correctness and aptness of the design method. Strictly speaking these conclusions can only relate to plastic design when it is applied to the structure under test. It is a matter of judgment and interpretation to relate the test structure and its loading conditions to those structures and conditions which will occur in commercial reality.

The tests to be recorded later will show, basically, the relation between test result and theoretical prediction. Discrepancies between these two quantities can be attributed to three groups of factors:

a) Discrepancies in the basic mechanism theory

b) Differences between the test structure and the mathematical model

c) Structural imperfections in the test structure.

It is not possible for a test to distinguish between these factors. Indeed such a distinction is not necessary in order to verify the method as a design procedure, as the aggregation of these three factors will also exist in any real structure.

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iii Superscripts refer to references which commence on page 5.

In the survey presented below, the aim has been to provide the reader with data from which specific conclusions may be drawn. The report itself is intended as a survey and not as a critical summary. However, it may be stated as a general conclusion that the tests indicate that plastic design is a valid and effective method when used within certain bounds of application. These bounds are very closely those which are defined in the standard texts[i-v].

II. BRIEF HISTORICAL SURVEY

Rolled low carbon steel beams were first introduced in the latter half of the nineteenth century. The usefulness of these members was soon realized and they became a commonly used structural component. As the bases for the elastic theories had recently been established it was natural that these theories were applied to the new members. This resulted in further theoretical developments and the elastic methods became both elaborate and elegant.

Tests were carried out on the beams to verify their elastic behavior, and these tests were usually confirmatory. It is not uncommon to find in technical publications of the time, the statement that a beam had reached its limit of usefulness when the applied moment was M_y, where

$$M_y = S\,\sigma_y \qquad \begin{aligned} S &= \text{section modulus} \\ \sigma_y &= \text{yield stress} \end{aligned}$$

Further, the statement was backed by tests results. Although this interpretation might seem false in the light of present knowledge, it arose from two factors:

a) The experimenters frequently regarded any non-linear behavior of their test beams as unsatisfactory, simply because it was non-linear and therefore outside their design assumptions.

b) The test beams were of practical dimensions but the lateral bracing would often have been considered inadequate by present-day standards. Thus yielding frequently precipitated lateral buckling and a subsequent decrease in the moment capacity. In this respect the moment M_y was in fact, the limit of usefulness of the beam.

The ductile behavior of beams, that is their ability to carry their maximum moment over a considerable range of deformation, was first recorded by Meyer[2.35] in 1908. Utilization of this ductility was the next logical step, but was quite contrary to the elastic concepts of the time. The step was taken first by Kazinczy[3.17] in Hungary in the period between 1910 and 1920, and also by Kist in Holland.

The next problem was the stress distribution in the hinging segment of the beam, and the subject was hotly debated from the 1920's onwards[2.21]. Many workers clung to the modulus of rupture concept, claiming that the plastic moment, M_p, could be calculated from:

$$M_p = S\,\sigma_r \qquad \text{where } \sigma_r > \sigma_y$$

Others claimed that the yielding process across a cross-section was discontinuous and spasmodic. The presently accepted rectangular stress block came to be generally accepted towards the end of the 1930's, however, as late as 1951 a paper was published offering experimental proof that the extreme fiber stress did not exceed σ_y.[2.40]

The period from 1928 to 1938 saw intensive experimental investigation of plastic beam behavior, mainly in Germany. For instance, Maier-Leibnitz, who was one of the leaders in the development, showed that the load capacity of a continuous beam was not affected by settlement of the supports[2.20]. In 1936 Stussi and Kolbrunner published[2.7] their well known paradox experiments aimed at refuting the plastic design method for beams. Although the points raised in that paper can and have been readily countered[iv], it is interesting to note the restrictive effect which the paper had on further German developments in the field of plastic design.

Meanwhile an English research group[v] had been studying the stresses in actual structures and had found little correlation with the elastic predictions. This provided the initial incentive for the development of plastic design methods for framed structures. The success, during World War II, of Baker's[v] plastically designed bomb shelters hastened the investigations and intensive studies were soon implemented.

The major experimental programs took place at Lehigh University in the U. S. A. and at Cambridge University in England. The history of these developments since the War is well documented in publications such as the "Commentary on Plastic Design"[i] and the "Steel Skeleton, Vol. II"[v], and will not be repeated in this report.

III. SURVEY OF THE TESTS

The following section presents a compilation of those tests which provide the experimental bases for the plastic design method. The section is divided into the fourteen Groups shown below, references are not given in those groups for which complete surveys are already available. Accompanying each reference is a summary of the type of test; the material, sections and dimensions used; and any important or unusual implications of the test. An author index is provided at the conclusion of the section.

GROUPS:

1. Stress-Strain Relationships
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9. Instability of Compression Members
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11. Connections
12. Variable and Dynamic Loading
13. Frame Instabiltiy
14. High Strength Steels

GENERAL REFERENCES:

i. Commentary on Plastic Design in Steel
Proc. ASCE (EM Division), July 1959-April 1960

ii. Plastic Design of Steel Frames, by L. S. Beedle
J. Wiley, 1958

iii. Plastic Analysis of Structures, by P. G. Hodge
McGraw-Hill, 1958

iv. The Plastic Methods of Structural Analysis, by
B. G. Neal, 1959, J. Wiley

v. The Steel Skeleton, Vol. II, by J. Baker, M. Horne
and J. Heyman. Cambridge University Press, 1956

ck ASCE Paper 205.64

GROUP I

Stress-Strain Relationships

General reference:

1.0 Beedle, L. S. and Tall, L., Basic Column Strength. Proc. ASCE 86(ST-7), p. 139, July 1960

add

GROUP 2

Simple Beams

2.1 Luxion, W. and Johnston, B. G.
PLASTIC BEHAVIOR OF WIDE-FLANGE BEAMS, Weld Journal 27(11), p. 538s, November 1948

Six tests on 8WF sections, 12' and 14' spans with double point loads. Residual stresses were not measured, however annealed tests indicated that residual stresses could be significant. Curvatures were less than predicted.

2.2 Driscoll, G. C. Jr. and Beedle, L. S.
THE PLASTIC BEHAVIOR OF STRUCTURAL MEMBERS AND FRAMES, Weld Journal 36(6), p. 275s, June 1957

One beam test, 12WF36 with 12' span. Behavior was in agreement with theory. 2 tests with 14WF38 on 15' span, one single and one double point load. Latter test showed no plateau due to inadequate bracing. Single point load test gave an increase in moment above M_p.

2.3 Yang, C., Beedle, L. S. and Johnston, B. G.
RESIDUAL STRESS AND THE YIELD STRENGTH OF STEEL BEAMS, Weld Journal 31(4), p. 205s, April 1952

One simple beam test, 8WF40, 14' span and double point loads. Strength less than predicted by about 10% but long moment plateau was obtained. Residual stresses measured.

2.4 Hall, W. and Newmark, N.
SHEAR DEFLECTIONS OF WIDE-FLANGE STEEL BEAMS IN THE PLASTIC RANGE Trans. ASCE, 122, p. 666, 1957

One test on a simply supported beam, double point load, 9' span with two 4' 6" cantilever spans. 8WF58. Agreement with theory was improved by considering residual stresses.

2.5 Ketter, R., Kaminsky, E. and Beedle, L. S.
PLASTIC DEFORMATION OF WIDE-FLANGE BEAM-COLUMNS Trans. ASCE 120, p. 1028, 1955

Found moment-curvature results from tests on a variety of steel beam-columns. Satisfactory agreement with theory.

2.6 Lee, G. C. and Galambos, T. V.
THE POST-BUCKLING STRENGTH OF WIDE-FLANGE BEAMS, Proc. ASCE, 88(EM1), p. 59, 1962

Five beam tests on 10WF25, spans from 80" to 180". Double point loads. Calculated M_p not always fully attained but tests gave long plastic plateau when adequate bracing was present. Checked predicted deflections.

2.7 Sawyer, H.
POST-ELASTIC BHEAVIOR OF WIDE-FLANGE STEEL BEAMS, Proc. ASCE 87(ST8), p. 43, 1961

Twenty-one beam tests. Sections from 12B14 to 12WF31, spans 3/4" to 128", single point loads. Curves continued to show load increase after reaching test M_p. Tests conducted at rapid rate of loading.

2.8 Popov, E. and Willis, J.
PLASTIC DESIGN OF COVER-PLATED CONTINUOUS BEAMS, Proc. ASCE 84(EM1), paper 1495, January 1958

Two beam tests with simple spans. 8' span, 5I10. Load capacity exceeded M_p (single point load) giving continually rising load-deflection curve.

2.9 Roderick, J. and Pratley, H.
BEHAVIOR OF ROLLED-STEEL JOINTS IN THE PLASTIC RANGE, Brit. Weld. Journal, 1, 1954

Nine tests on 8" by 4" and 10" by 4-1/2" I. 6 tests had single and 3 had double point loads. In all tests the load capacity continued to increase after M_p. Spans were 9' 3".

2.10 Roderick, J. and Heyman, J.
EXTENSION OF SIMPLE PLASTIC THEORY TO TAKE ACCOUNT OF THE STRAIN-HARDENING RANGE, Inst. Mech. Eng., War Emergency Publication, 67, 1951

Tested 12 beams, single point load, 17" span, chose materials in which range between yield and strain hardening became progressively smaller. Agreement between test and theory was good, confirming strain-hardening effect.

2.11 Nelson, H., Wright, D. and Dolphin, J.
DEMONSTRATIONS OF PLASTIC BEHAVIOR OF STEEL FRAMES, Proc. ASCE 83(EM4), paper 1390, October 1957

Tested a 10WF25, 11' span, double point loads. Did not reach M_p, deflections were under-estimated unless residual stresses were considered in calculations.

2.12 Gozum, A.

EXPERIMENTAL "SHAKEDOWN" OF CONTINUOUS STEEL BEAMS, Fritz Engineering Laboratory Report 205G.1, Lehigh University, 1954

Four tests on 4WF13, double point loads, 4' spans. 3 tests showed no moment plateau and in fourth there was a gradual increase in load capacity until predicted M_p was reached.

2.13 Kusuda, T. and Thurlimann, B.

STRENGTH OF WIDE FLANGE BEAMS UNDER THE COMBINED INFLUENCE OF MOMENT, SHEAR AND AXIAL FORCE, Fritz Engineering Laboratory Report 248.1, Lehigh University, 1958

Three tests on structural knees under moment axial force and shear. Curves showed no load plateau. Sections used were 12WF58, 10WF29. Arm lengths for moments from 10" to 33". Final failure by weld fracture.

2.14 Charleton, T.

A TEST ON A TWO-BAY PITCHED ROOF PORTAL STRUCTURE WITH BUTTRESSED OUTER STANCHIONS, Brit. Weld Res. Assoc., March 1959. D1/10/59

Series included 2 beam tests, one single and 2 double point load tests. 7' 6" span, 2" by 3" I. Good agreement with double point load test.

2.15 Massey, C.

LATERAL BRACING FORCE OF STEEL I BEAMS, Proc. ASCE 88(EM6), p. 89, December 1962

Tested I beams 1"-1-2", spans from 10" to 30". 6 tests. M_p reached in tests, moments applied at end of specimens. No data on load-deflection given, however, bracing forces were measured and larger than expected.

2.16 Baker, J. and Heyman, J.

TESTS ON MINIATURE PORTAL FRAMES, Structural Eng., 28(6), 1950

Series included tests on 1/4" by 1/4" annealed rectangular bars. 30 tests but only a few reported. Results as predicted by theory.

2.17 Volterra, E.

RESULTS OF EXPERIMENTS ON METALLIC BEAMS BENT BEYOND THE ELASTIC LIMIT, Journal Inst. Civil Engineers 20(349), 1943

Twenty simple beam tests, not carried to full failure. Spans averaged 38", sections were I, +, and T averaging 3" by 3". Used single and double point loads.

2.18 Yen, Y. C., Lu, L. W. and Driscoll, G. C. Jr.

TESTS ON THE STABILITY OF WELDED STEEL FRAMES, Weld. Research Council Bulletin, 81, September 1962

Series included one beam test with double point load. 30" span, 2-5/8" WF. Obtained standard curve.

2.19 Hendry, A.

AN INVESTIGATION OF THE STRENGTH OF CERTAIN WELDED PORTAL FRAMES IN RELATION TO THE PLASTIC METHOD OF DESIGN, Structural Engineer, 28, 1950

See also 4.21, 6.22. Tested a 3"-1" I on 18" span, single point load. Obtained a gradual strain-hardening effect. Also two (normalized) 1" 1-1/4" I on 20" and 15" spans. Single point load, gradual strain-hardening effect. Also 20 beam tests to check shear--see 6.22.

2.20 Maier-Leibnitz, H.

AUSDEUTUNG UND ANDWENDUNG DER ERGEBNISSE, Preliminary Publication, I. A. B. S. E., 2nd Congress, Berlin, 1936

No new tests but a very useful summary of most simple beam tests prior to 1936.

2.21 Roderick, J. and Phillips, I.

CARRYING CAPACITY OF SIMPLY SUPPORTED MILD STEEL BEAMS, Res. (Engineering Structure Supplement), Colston Papers, 2(9), 1949

Tested 8 beams, 1" square section, (annealed), 15" spans. 4 single and 4 double point loads. Load-deflection curves composed of linear segments. No observed shear effects even with point loads. Also a useful summary of early English work back to turn of century.

2.22 Maier-Leibnitz, H.

VERSUCHE ZUR WEITEREN KLARUNG DER FRAGE DER TATSACHLICHEN TRAGFAHIGKEIT DURCHLAUFENDER TRAGER AUS BAUSTAHL, Stahlbau 9(153), 1936

One test on single point loaded 47" beam, simply supported. 4"-4" I. Gave load-strain and deflection curves. Load capacity continued to gradually increase without plateau in curve.

2.23 Haigh, B.

THE LOWER YIELD POINT IN MILD STEEL, Engineering, 138, November 1934, pgs. 461 and 544.

Two beam tests and a connection test. Beams had single point loads, one beam with vertical plate welded in at midspan. 4" 3" 9.4 I, 48" span. Plastic load exceeded and load-deflection curve continued to increase gradually.

2.24 Maier-Leibnitz, H.

VERSUCHE MIT EINGESPANNTEN UND EINFACHEN BALKEN VON I-FORM AUS ST 37, Bautechnik, 13 p. 264-267, 1935

Two tests on simply supported beams with center point loads. 5' 3" span. Tests on I14.14. Load capacity exceeded and load-deflection curve was continuing to increase gradually at end of test. See also 3.20.

2.25 Robertson, A. and Cook G.
TRANSITION FROM THE ELASTIC TO THE PLASTIC STATE IN MILD STEEL, Proc. Roy. Soc. A88(1913)

Tested a number of mild steel beams 3/8"-3/16" with double point loads 3" apart. Material had definite upper yield point. Load deflection curve was becoming horizontal at end of recorded curve.

2.26 Moore, H.
THE STRENGTH OF I-BEAMS IN FLEXURE, University of Illinois Engineering Experiment Station, Bulletin 68, September 1913

Early U. S. tests on beams. Plastic behavior not always fully recorded but results of 5 tests give confirmation of present theories. Used 8" I 18 lb. beams with 5' and 10' spans, double and single point loads.

2.27 Harrison, H.
THE BEHAVIOR AT COLLAPSE OF MILD STEEL CONTINUOUS BEAMS OF RECTANGULAR SECTION LOADED IN BOTH PRINCIPAL PLANES, Aust. J. Appl. Sci. 13(3), September 1962, p. 207

See also 3.21. Tests on normalized mild steel, 0.6" by 0.3" rectangular sections, with 13" span. Single and double point loads. Even with normalizing single point load capacity began to increase again after some plastic deformation.

2.28 Morrison, J.
THE YIELD OF MILD STEEL WITH PARTICULAR REFERENCE TO THE EFFECT OF SIZE OF SPECIMEN, Proc. Inst. Mech. Engs. 142(3), January 1940, p. 193

Tested 11 small diameter beams under uniform moment to determine yielding process and stress distribution. Plotted moment vs. end rotation, curves showed some moment plateau region. Cause of final failure not given.

2.29 Kazinczy, G.
KRITISCHE BETRACHTUNGEN ZUR PLASTIZATSHEORIE, Second Congress, Int. Assoc. Bridge Struct. Engg, Berlin 1936, Final Report (1939), p. 56 (I5)

One test on I(MP24), 260cm span, 2 point loads 100cm from each support. Test M_p slightly higher than predicted. Moment-curvature curve was standard but still increasing at end of test.

2.30 Bryla, St. and Chmielowiec, A.
EXPERIMENTS ON ROLLED SECTIONS STRENGTHENED BY WELDING, Second Congress, Int. Assoc. Bridge Struct. Engg, Berlin 1936, Final Report (1939), p. 561 (V3)

Tested 25 beams, single span, point loads. No graphical results presented. Final failure frequently due to local buckling. Peak moment adequately given by M_p.

2.31 Cook, G.
THE YIELD POINT AND INITIAL STAGES OF PLASTIC STRAIN IN MILD STEEL SUBJECTED TO UNIFORM AND NON-UNIFORM STRESS DISTRIBUTIONS, Phil. Trans. Roy. Soc. 230(A) 1931, p. 103

Ten tests on 0.20" dia steel beams with double point loads. Tests carried well into plastic range and many points taken in this region. Slight decrease in load capacity in plastic range attributed to upper and lower yield points.

2.32 Muir, J. and Binnie, D.
THE OVERSTRAINING OF STEEL BY BENDING, Engineering 122, p. 743, 1926

Tested mild steel rectangular beams 0.347"x 0.250" with double point loads. 5" between loads. Obtained typical moment-curvature curves with plateau.

2.33 Bernhult, E.
FLYTGRANSEN VID BOJNING OCH SPANNINGSFORHALLANDET I STANGER OCH ROR UNDER PLASTISK BOJNING, Jernkonterets Annaler Arg 127(10), 1943, p. 491

Tested 18 simply supported, single point load circular beams. Diameter from 6.30 to 35.15mm, span 210mm. Most tests showed increasing load-deflection curve after reaching M_p. Steels from 0.10-0.35% carbon.

2.34 Rinagl, F.
UBER FLIESSGRENZEN UND BIEGEKENNLINIEN, Preliminary Publication, Int. Assoc. Bridge and Struct. Engg. Berlin 1936, p. 1561, Paper I3.

Tested 6 prismatic beams, each with a steel of different upper yield point properties. Double point loads, 470mm span. 4 bars rectangular and flat, 1 in diamond and one circular. Obtained typical plateau type curve, correlated with varying upper yield points.

2.35 Meyer, E.
DIE BERECHNUNG DER DER DURCHBIEGUNG VON STABEN, DEREN MATERIAL DEM HOOKESCHEN GESETZE NICHT FOLGT, Zeitschrift Vereines Deutscher Ingenievre, 52(5), 1908, p. 167

Earliest tests in which plastic beam behavior was recorded. Tested two beams with single point loads. Obtained typical increasing load-deflection curve after M_p. Used 4"x2" rectangles with a 51" span.

2.36 Rianitsyn, A.
CALCUL A LA RUPTURE ET PLASTICITE DES CONSTRUCTIONS (TRANSLATED FROM THE RUSSIAN), Eyrolles, Paris, 1959

Contains a summary of the Russian experiments on the plastic behavior of beams.

2.37 Lazard, A.
THE EFFECT OF PLASTIC YIELD ON THE BENDING OF MILD STEEL PLATE GIRDERS, The Structural Engineer 32(2), February 1954, p. 49

Brief data of tests on 36" deep rolled girders. Insufficient data for conclusions to be drawn, however, curves appear typical.

2.38 Harrison, H.
THE LOAD CAPACITY OF MILD STEEL BEAMS OF CIRCULAR SECTION BENT IN TWO PLANES, Civ. Engg. Trans. Inst. Engs. Aust. CE1(2), September 1959, p. 71

See also 3.23. Control tests included 15" span centrally loaded beams, 14 tested. Result presented indicated that load capacity again increased after M_p was reached.

2.39 Dawance, G.
NOUVELLES RECHERCHES EXPERIMENTALES SUR LA PLASTICITE DES ELEMENTS DE CONSTRUCTION METALLIQUE, Constn. Metallique No. 6, May 1950, Annales de l' Institut Tech du Batiment et des Travaux Publics.

Fifteen tests on beams with double point loads, well reported. Used 8"I, spans from 7' to 15'. Obtained rising plateaus on most moment-curvature curves. Observed brittle behavior at punched holes, drilled holes had no effect on deformation.

2.40 Nishihara, T. and Taira, S.
ON THE YIELDING OF STEEL UNDER BENDING MOMENT, Mem Faculty Engg., Kyoto Uni, XIII(II), March 1951, p. 55

Tested 10 rectangular, 2 circular and 3 crossed sections. Used double point loads 5" apart, section dimensions of order of 1". Obtained moment-curvature curves with very flat plateaus. Measured surface stresses by X-ray.

2.41 Reynolds, G.
ANALYSIS AND DEFLECTIONS OF NONLINEAR STRUCTURES, Civ. Engg. (London), 55(649), p. 1037, August 1960

Tests included beam tests to find inelastic moment-curvature curves. Also a single point load test on 8" span beam. All sections 3/8" square. Checked deflections by complementary energy approach.

2.42 Farnell, K.
STRESS DISTRIBUTION IN OVERSTRAINED MILD STEEL BEAMS, Engineering, 185, p. 788, June 1958

Tests on large and small beams (no dimensions given) with uniform moment. Obtained typical moment-curvature curves. Utilized Leuders line approach in predicting these results.

2.43 Lee, G. C. and Galambos, T. V.
Closure to Discussion of THE POST BUCKLING STRENGTH OF WIDE FLANGE BEAMS, Proc. ASCE 89(EM-1), p. 75, February 1963

Two tests on 10B15 with spans of 160" and 140". Double point loads 96" and 84" apart. Larger plastic hinge region than in Ref. 2.6 caused decrease in the length of the moment plateau.

2.44 Stussi, F.
THEORIE UND PRAXIS IM STAHLBAU, Mitt Schweizer Stahlbauverband, 16, 1956

Included 4 tests on simple beams, 60cm span, section I 60/40, St 44. Single point load. Maximum loads in tests was from 5% below to 8% above predicted maximum load.

2.45 Omerod, A.
BEAMS LOADED IN NON-PRINCIPAL PLANES, Civ. Engg. (London), 56(656), March 1961, p. 336

Tested 1/2" x 1/2" x 1/8" L on simple 24" span with single point load, one leg of angle vertical. Customary load-deflection curve was obtained. See also 3.26.

2.46 Omerod, A.
BEAMS LOADED IN NON-PRINCIPAL PLANES, Civ. Engg. (London), 54(639), October 1959, p. 1173

Tested 3/4" x 3/4" x 1/8" L on simple 30" span with single point load, one leg vertical Customary laod deflection curve obtained, accurately predicted M_p.

2.47 Heyman, J. and Dutton, V.
PLASTIC DESIGN OF PLATE GIRDERS WITH UNSTIFFENED WEBS, Weld. and Metal Fabricn., 22(7), July 1954, p. 268.

See also 6.24. Tests included two simple span beams, 18" and 30" spans, with single point loads, in which shear effect was small. Load-deflection curve showed relatively flat plateau.

2.48 Sparacio, R.
LA RICERA DEL MINIMO COEFFICIENTE DI SICUREZZA A ROTTURA IN PRESENZA DI CARICHI VARIABILI E DISTORSIONI, G. Gen. Civ., 98(10), p. 794-807, October 1960

Tests on shakedown included static tests. Used 2cm x 1cm rectangular sections. Two single span, single point load beams with 80cm span. Load-deflection curve showed steady increase after reaching M_p.

GROUP 3

Continuous Beams

3.1 Hall, W. and Newmark, N.
SHEAR DEFLECTIONS OF WIDE-FLANGE STEEL BEAMS IN THE PLASTIC RANGE, Trans. ASCE, 122, p. 666, 1957

One relevant continuous beam test. Load jack failed before failure. 8WF58, spans 4' 6" - 9' 0" - 4' 6". Deflections checked with calculations by Newmarks method.

3.2 Popov, E. and Willis, J.
PLASTIC DESIGN OF COVER PLATED CONTINUOUS BEAMS, Proc. ASCE 84(EM1), paper 1495, January 1958

Five continuous beam tests. 4 used 6I12.5 with cover plates. 8' spans. Tests stopped at deflection of span/36. At this stage load was still increasing in 4 tests.

3.3 Yang, C., Beedle, L. S. and Johnston, B.
RESIDUAL STRESSES AND THE YIELD STRENGTH OF STEEL BEAMS, Weld. Journal 31(4), p. 205s, April 1952

Five continuous beams. Full load capacity not reached in tests, although mechanisms appeared to form. Used 8Wf40 in 4 tests, 14WF30 in one. 28' lengths include two 7' half side spans. Residual stresses measured.

3.4 Maier-Leibnitz, H.
CONTRIBUTION TO THE PROBLEM OF ULTIMATE CARRYING CAPACITY OF SIMPLE AND CONTINUOUS BEAMS OF STRUCTURAL STEEL AND TIMBER, Die Bautechnik, 1(6), 1927

Three continuous beam tests, 2 with support settlement. Settlement did not affect collapse loads which were in accord with present theories. Spans 8' to 16'. Section made from 2 INP16 I's and 2 cover plates.

3.5 Gozum, A.
EXPERIMENTAL "SHAKEDOWN" OF CONTINUOUS STEEL BEAMS, Fritz Engineering Laboratory Report 205G.1, Lehigh University, 1954

Two 2 span proportionally loaded beams tested. Used 4WF13 single point loads, 4' spans. Obtained larger load capacity than predicted and a continually increasing load-deflection curve.

3.6 Horne, M.
EXPERIMENTAL INVESTIGATION INTO THE BEHAVIOR OF CONTINUOUS AND FIXED ENDED BEAMS, 4th Congress, I. A. B. S. E., Cambridge and London, 1952

Used 1" square as-received steel bars. 11 tests on 16" and 20" spans. 5 tests had single point loads, also support settlement. Obtained good agreement with theory, including deflection prediction.

3.7 Stussi, F. and Kolbrunner, C.
BEITRAG ZUM TRAGLASTVERFAHREN, Bautechnik, 13 p. 264-267, 1935

Fourteen tests on 3 span beams with a single point load in center span. 46mm x 35mm I. Center span 60cms. Outer span varied. Results show a decrease of the load capacity as outer span increases in length.

3.8 Nelson, H., Wright, D. and Dolphin, J.
DEMONSTRATION OF PLASTIC BEHAVIOR OF STEEL FRAMES, Proc. ASCE 83(EM4), paper 1390, October 1957

Test on propped cantilever. High shear and moment at support caused reduction in plastic moment. 8B13 12' 0 span.

3.9 Van den Broek, J.
THEORY OF LIMIT DESIGN, DISCUSSION BY PETERSON, Trans. ASCE, Vol. 105, 1940

One test on three span continuous beam, two 3" channels, 6' 0" spans, single point loads. Showed that mechanism approach could predict loads.

3.10 Volterra, E.
RESULTS OF EXPERIMENTS ON METALLIC BEAMS BENT BEYOND THE ELASTIC LIMIT, J. Inst. C. E., 20, 349, 1943

Nine tests. Results not fully presented although a mechanism appeared to form. 128" total length. Tests on I and T. Not carried to failure. See also 2.17. Span 38" average.

3.11 Hartmann, F.
DIE FORMÄNDERUNGEN EINFACHER UND DURCHLAUFENDER STAHLTRAGER MIT EINEM VERSUCHE, Schweiz. Bautzg. 101(75), 1933

One test. 2 span beam with double unsymmetrical point loads on one span. 2/10'. I NP 12. Deflections calculated by curvature methods were overestimated. Collapse load was reached.

3.12 Blessey, W.
PRIVATE COMMUNICATION TO L. BEEDLE, 7 FEB. 1958, Tulane University

One test. 2 span beam. 12' spans. 12WF36. Load capacity significantly exceeded. Single point loads in each span.

3.13 Driscoll, G. C., Jr. and Beedle, L. S.
THE PLASTIC BEHAVIOR OF STRUCTURAL MEMBERS AND FRAMES, Weld. Journal 36(6), p. 275s, June 1957 Cross ref?

One continuous beam test. 2 x 10' spans with double point loads. 12WF36. Collapse load was attained eventually but shear was critical over the center support.

3.14 Maier-Leibnitz, H.
AUSDEUTUNG UND ANWENDUNG DER ERGEBNISSE, Preliminary Publication, I. A. B. S. E., 2nd Congress, Berling, 1936

No new tests but a very useful summary of most continuous beam tests prior to 1936.

3.15 Highway Research Board
THE AASHO ROAD TEST. REPORT NO. 4. BRIDGE TEST, 61D. Publication 953. National Academy of Science

Eight tests on non-composite steel road bridges. Simple supports and spans. Yielding occurred earlier in tests than expected and caused significant plastic behavior. Cover plates appeared to extend hinges to end of cover plates. 5 of tests had cover plates (18WF55, 18WF50, 18WF60, 18WF96, 21WF62, 50' spans, A 7)

3.16 Kazinczy, C.
KISERLETEK BEFALAZOTT TARTOKKAL, Betonszemle, 2(4), p. 68, 1914, 2(5), p. 83, 1914, 2(6), p. 101, 1914

Copies of these tests are now unavailable, however, work is mentioned here as it provides first experiments performed on plastic design methods. See also 3.17.

3.17 Hoff, N.
Discussion of ARTICLE IN WELDING JOURNAL, Weld. Jnl. 33(1), p. 14-s, 1954

A resume and discussion of the early historically important work of Kazinczy, C. No actual data given. See 3.16.

3.18 Schaim, J.
DER DURCHLAUFENDE TRAGER UNTER BERUCKSICHTIGUNG DER PLASTIZITAT, Stahlbau, 3(13), 1930

Six tests to failure of 4 two span and 2 single span beams of I16 and I14 section, 26' full span. Beams tested in braced pairs, with final failure by lateral buckling.

3.19 Maier Leibnitz, H.
VERSUCHE ZUR WEITEREN KLARUNG DER FRAGE DER TATSACHLICHEN DURCHLAUFENDER TRAGER AUS BAUSTAHL, Stahlbau 9(153), 1936

See also 2.22. One test of beam over 4 supports. 4"/4" I, spans 94"-47"-94". One single point center load. Test oriented towards Ref. 3.7. Failure by lateral buckling. Results were conservatively estimated.

3.20 Maier Leibnitz, H.
VERSUCHE MIT EINGESPANNTEN UND EINFACHEN BALKEN VON I-FORM AUS ST37, Bautechnik, 7(313), 1929

See also 2.24. 2 tests on beams with 1 and 2 half spans, 13' total length. 2 loads at ends and 2 loads on middle span. Plastic load was reached in tests but only after considerable deflections. I14.14 and BurbachI 152/127.

3.21 Harrison, H.
THE BEHAVIOR AT COLLAPSE OF MILD STEEL CONTINUOUS BEAMS OF RECTANGULAR SECTION LOADED IN BOTH PRINCIPAL PLANES, Aust. J. Appl. Sci. 13(3), September 1962, p. 207

Tests on 7 model steel beams, 2 span with far ends fixed. Spans 23" & 13", rectangular 0.6" by 0.3". Normalized. Varied two mutually perpandicular loads. Strain-hardening prevented well defined collapse loads.

3.22 Rianitsyn, A.
CALCUL A LA RUPTURE ET PLASTICITE DES CONSTRUCTIONS (TRANSLATED FROM THE RUSSIAN), Eyrolles, Paris, 1959

Contains a summary of Russian tests on the plastic behavior of continuous beams.

3.23 Harrison, H.
THE LOAD CARRYING CAPACITY OF MILD STEEL BEAMS OF CIRCULAR SECTION BENT IN TWO PLANES, Civ. Engg. Trans. Inst. Engs. Aust., CE1(2), September 1959.

Tested 3/8" dia rods loaded in two mutually perpandicular directions. Simply supported and loaded at third points, also propped cantilevers and built-up beams. Also single load two span beams. Modes of failure were as predicted, failure loads somewhat higher.

3.24 Reynolds, C.
ANALYSIS AND DEFLECTIONS OF NON-LINEAR STRUCTURES, Civ. Engg. (London), 55(649), p. 1037, August 1960

Tested one continuous beam with 10" and 5" spans, section 3/8" square. Single point load on 10" span. Results checked well with authors complementary energy approach.

3.25 Stussi, F.
THEORIE UND PRAXIS IM STAHLBAU, Mitt Schewizer Stahlbauverband, 16, 1956

Tests similar to those reported in Ref. 3.7. Tested 5 pairs of beams. Dimensions as in 3.7 except for two tests with shorter outer spans, also used a different section (I 60/40). Results similar to those in 3.7. Greatest variation within test pair was 9.5%.

GROUP 4

Frames

4.1 Knudsen, K., Ruzek, J., Johnston, E. and Beedle, L. S.
WELDED PORTAL FRAMES TESTED TO COLLAPSE, Weld. Jnl. 33(9), p. 469s, September 1954

Tested 2 rectangular frames of uniform cross-section, 8WF40 or 8B13. Pin base, sway prevented, double point loads. Collapse load reached in both frames. With 8WF40 test load capacity kept increasing, but fell off in other test.

4.2 Schilling, C., Schutz, F. and Beedle, L. S.
BEHAVIOR OF WELDED SINGLE-SPAN FRAMES UNDER COMBINED LOADING, Weld. Jnl. 35(5), p. 234-s, May 1956

Two rectangular portals, 30' by 10', 12WF36. Bases pinned and fixed, double point loads. Lateral supports proved critical but maximum loads checked with theory. Distirbution of moments in plastic range verified.

4.3 Driscoll, G. C., Jr.
TEST OF TWO SPAN PORTAL FRAME, Proc. AISC Natl. Engg. Conf., p. 74, 1956

One test using 10B17 and 8B13. Spans 28' and 20', height 13'. Maximum load underestimated due to malfunction of bracing system. Deflections underestimated. Maximum load at predicted mechanism deflection.

4.4 Popov, E. and McCarthy, R.
DEFLECTION STABILITY OF FRAMES UNDER REPEATED LOADS, Proc. ASCE 86(EM-1), January 1960, p. 61

Tests included one frame under proportional loading. Rectangular portal 6' by 6', 4WF13, pinned base, horizontal and vertical loads. Results showed plastic load capacity exceeded in test.

4.5 Nelson, H., Wright, D. and Dolphin, J.
DEMONSTRATIONS OF PLASTIC BEHAVIOR OF STEEL FRAMES, Proc. ASCE, paper 1390, October 1957

Six portal tests, 4B13, 8' by 4'. Various combinations of single point loads. Collapse loads were exceeded with steadily increasing load-deflection curves. Imperfect base fixity did not affect loads.

4.6 Baker, J. and Roderick, J.
TESTS ON FULL SCALE PORTAL FRAMES, Proc. Inst. Civ. Engrs., January 1952

Six rectangular portal frames. 16' by 8', 8" by 4" I. Bases were pinned, fixed and with one run of weld. Single point loads, horizontal and vertical. Predicted loads were attained, maximum load carried for three days. Lightly welded base just performed adequately.

4.7 Baker, J. and Eicknoff, K.
THE BEHAVIOR OF SAWTOOTH PORTAL FRAMES, Conf. on the Correlation between Calculated and Observed Stresses and Displacements in Structures. Inst. Civil Engs., p. 107, 1955

Two tests on saw tooth. 5"/3"I. 16'/14'. Members horizontally braced. Behavior as predicted. Observed that hinges form away from joint itself. Member with practical foundation behaved normally.

4.8 Baker, J. and Eickhoff, K.
A TEST ON A PITCHED ROOF PORTAL, Prel, Publicn., IABSE, 5th Congress, Lisbon, 1956

One pitched roof portal. 16'/11'. 7"/4" I. Collapse load well predicted but deflections underestimated. A load at ridge vertically and one at column top horizontally.

4.9 Driscoll, G. C., Jr. and Beedle, L. S.
THE PLASTIC BEHAVIOR OF STRUCTURAL MEMBERS AND FRAMES, Weld Jnl. 36(6), p. 275-s, June 1957

One pitched portal 40'/16'. 12WF36. Collapse load well predicted but deflection underestimated. Double point loads on each roof beam and on one column.

4.10 Baker, J. and Roderick, J.

AN EXPERIMENTAL INVESTIGATION OF THE STRENGTH OF SEVEN PORTAL FRAMES, Trans. Inst. Weld., 1(4), 1938

Six rectangular portals-7th repeated later as feet spread. Failure load well estimated. Used single and 4 point loading vertically. 1-1/2" by 1-1/4" I. All fixed bases. 20" by 10".

4.11 Baker, J. and Roderick, J.

FURTHER TESTS ON BEAMS AND PORTALS, Trans. Inst. Weld. 3(2), 1940, p. 83

Repeat test from 4.10. 4 point vertical loading on beam. Discrepancy blamed on effect of shear. 20" x 10". 1-1/2" x 1-1/4" I. Collapse load not attained and moment distribution did not agree with calculations.

4.12 Vickery, B.

THE INFLUENCE OF DEFORMATIONSAND STRAIN-HARDENING ON THE COLLAPSE LOAD OF RIGID FRAME STEEL STRUCTURES, Inst. Eng. Aust., CE Trans., Vol. CE 3, No. 2, September 1961

Four pitched frames. 30"/24". Pin base. 0.3"/0.6" I. 4 point vertical loads on two roof beams and horiz. on 1 column. Load-deflection seemed to give no plateau. Simple theory load was overestimated. Better agreement considering deformation and strain-hardening.

4.13 Roderick, J. and Harrison, H.

SOME ASPECTS OF PLASTIC BEHAVIOR, to be published, Inst. Eng. Aust. CE Trams. Vol. CE 3, No. 2. September 1961

At least 2 pitched portals. 20'/9'. 5"/2-1/2" I. Double point load on each beam, vertical load at column top. Excellent agreement between test and theory for loads and deflections.

4.14 Baker, J. and Charleton, T.

A TEST ON A TWO STORY SINGLE BAY PORTAL STRUCTURE, Brit. Weld. Jnl., May 1958

Tests on a two story frame, 12' wide with 6' stories. Columns 4" by 3" I, beams 5" by 3" I. Single point vertical load on each floor, horizontal loads at column tops. Close prediction of observed maximum moment.

4.15 Charleton, T.

A TEST ON A TWO BAY PITCHED ROOF PORTAL STRUCTURE WITH BUTTRESSED OUTER STANCHIONS, Brit. Weld. Res. Assoc., D1/10/59, March 1959

Spans 32', height 8', section 3" by 3" I. Single point loads on two roof beams. Some weld failure and instability during test, causing discrepancy between test and theory.

4.16 Blessey, W.
PRIVATE COMMUNICATION TO L. S. BEEDLE, February 7, 1958

Tested a rectangular portal, 12' by 6', section 12WF36. Single point load. Results did not agree very well with theory.

4.17 Baker, J. Heyman, J.
TESTS ON MINIATURE PORTAL FRAMES, Struct. Engr. 28(6), 1950

Fifteen tests on rectangular fixed base portals, 4" span by 2" height. Members mainly 1/4" by 1/4". Single point vertical load on beam and horizontal load at column top. Maximum loads were generally underestimated by the theory.

4.18 Vickery, J.
THE BEHAVIOR AT COLLAPSE OF SIMPLE STEEL FRAMES WITH TAPERED MEMBERS, J. Inst. Strt. Eng. XL(11), November 1962, p. 365

Tested 4 tapered model frames, 0.6/0.3" I and 30"/24" size. Also 3 frames of 5/2-1/2" I and 20'/9' external-1 fixed base, 1 pinned and 1 pinned and tapered. Found deflections caused simple plastic theory to overestimate strength and tapers made instability more critical.

4.19 Yen, Y. C., Lu, L. W. and Driscoll, G. C., Jr.
TESTS ON THE STABILITY OF STEEL FRAMES, Weld. Research Council Bulletin 81, September 1962

Three tests on pinned base rectangular portal frames. 87" span, heights 44", 66" and 88". Section 2.625" WF. Triple points loads on beams and two vertical loads on column tops. Buckling reduced load capacity about 16%

4.20 Girkmann, K.
UBER DIE AUSWIRKUNG DER "SELBSTHILFE" DES BAUSTAHLS IN RAHMENARTIGEN STABWERKEN, Stahlbau 5(121), 1932

One test on 59"/23" rectangular frame. Pin base, double angle tie rod. Members 28cm channels, back to back. Test aimed at checking (plastic) moment distribution. Results checked with normal load-deflection curve. Photos show hinges but rivetted gussets very large.

4.21 Hendry, A.
AN INVESTIGATION OF THE STRENGTH OF CERTAIN WELDED PORTAL FRAMES IN RELATION TO THE PLASTIC METHOD OF DESIGN, Struct. Eng. 28, (1950)

Tested 12 rectangular portal frames. Spans 20" to 36". Sections 1"/1-1/4", 3"/1", 3"/1-1/2" I. Single and double point loads. Tests indicate collapse behavior and also the effects of axial and shear forces on the fully plastic moment.

4.22 Andrews, E.
TRIALS WITH A SPECIMEN FRAME IN STEEL, Preliminary Publication, International Assoc. Bridge and Struct. Engg., 2nd Congress, 1936, p. 859

Tested one frame, 55"/38" rectangular. 4"/1-3/4" I. Photographs and description show clear mechanism failure mode.

4.23 Baker, J.
PLASTICITY AS A FACTOR IN THE DESIGN OF WAR TIME STRUCTURES, The Civil Engineer in War, 3, 1948, (Inst. Civil Engs.)

Describes the design and performance of the war-time Morrison shelters. Cubes of 6"/6"/3-8" and 3"/2-1/2"/1/4" angles. When loaded collapse behavior indicated mechanism mode. 2'6"/4'/6'6" structures showed 6 1-4" midspan deflection.

4.24 Baker, J., Williams, E. and Lax, D.
THE DESIGN OF FRAMED BUILDINGS AGAINST HIGH-EXPLOSIVE BOMBS, The Civil Engineer in War, 3, 1948, (Inst. Civil Engs.)

Contains photographs and commentary on bombed steel frame buildings. Clear illustration of load redistribution in structures, of reserves of strength, and of failure modes.

GROUP 5

Deflections and Rotations

5.1 Knudsen, K., Yang, C., Johnston, B., and Beedle, L. S.
PLASTIC STRENGTH AND DEFLECTION OF CONTINUOUS BEAMS, Weld. Jnl., 32(5), p. 240-s, 1953

Compares previously obtained experimental results with deflection calculations. (results from 2.3, etc.) Deflections larger than predicted, especially with welded regions and constant moment lengths.

5.2 Driscoll, G. C., Jr.
TEST OF TWO-SPAN GABLED PORTAL FRAME, Proc. AISC Natl. Engg. Conf., p. 74, 1956

See also 4.3. Deflections were underestimated. Columns were welded to base plates which were then bolted to the floor so would be slightly less than rigid.

5.3 Lu, L. W., Chapman, B. and Driscoll, G. C., Jr.
PLASTIC STRENGTH AND DEFLECTION OF A GABLE FRAME, Proc. ASCE, paper 1390, October 1957

See also 4.9. Deflections and rotations carefully calculated for critical points in the frame. Found general shapes predicted well for load deflection but that there were non-conservative knees in curves.

5.4 Stussi, F. and Kollbrunner, C.
BEITRAG ZUM TRAGLASTVERFAHREN, Bautechnik, 13, p. 246-267, 1935

See also 3.7. As outer spans of three span beams were increased so collapse load decreased. Plastic design showed no decrease in collapse load. Counter arguments in iii and iv.

5.5 Baker, J. and Heyman, J.
TESTS ON MINATURE PORTAL FRAMES, Proc. ASCE, paper 1390, October 1957

See also 2.16. Deflections did not become large as strain-hardeinig seemed to take strength above predicted collapse.

5.6 Yang, C., Beedle, L. S. and Johnston, B.
RESIDUAL STRESSES AND THE YIELD STRENGTH OF STEEL BEAMS, Weld. Jnl. 31(4), p. 205s, April 1952

See also 2.3. 6 beam tests indicated that estimated deflections are under-estimated-residual stresses not considered in these deflection estimates--see 5.1.

5.7 Van den Broek, J.
THEORY OF LIMIT DESIGN, Theory of Limit Design, J. Wiley, 1948

Shows three tests on cantilevers of rectangular, round and WF section. Elastic deflections were predicted only. Tests were not carried far into plastic range.

5.8 Schilling, C., Schultz, F. and Beedle, L. S.
BEHAVIOR OF WELDED SINGLE-SPAN FRAMES UNDER COMBINED LOADING, Weld. Jnl. 35(5), p. 234-s, May 1956

See also 4.2. Deflections of two portals checked out in predicted form but were underestimated by analysis which neglected residual stresses.

5.9 Popov, E. and Willis, J.
PLASTIC DESIGN OF COVER-PLATED CONTINUOUS BEAMS, Proc. ASCE 84(EM1), paper 1495, January 1958

See also 3.2. Of interest here as it was decided to stop tests at a deflection of 1/36 of span. Only one test had stopped showing a load increase at this deflection.

5.10 Roderick, J. and Heyman, J.
EXTENSION OF SIMPLE PLASTIC THEORY TO TAKE ACCOUNT OF THE STRAIN-HARDENING RANGE, Inst. Eng., War Emergency Publication, 67, 1951

See also 2.10. Deflection agreement was good for materials with no plateau in the stress-strain diagram. Justified use of two piece stress-strain diagram for calculating deflections.

5.11 Roderick, J. and Pratley, H.
BEHAVIOR OF ROLLED STEEL JOINTS IN THE PLASTIC RANGE, Civ. Engg. (London), 55(649), p. 1037, August 1960

See also 2.9. Deflection curves showed typical non-conservative knee at hinge formation points.

5.12 Nelson, H., Wright, D. and Dolphin, J.
DEMONSTRATIONS OF PLASTIC BEHAVIOR OF STEEL FRAMES, Proc. ASCE, paper 1390, October 1957

See also 4.5. Portal test deflections were compared with theory and deflections were underestimated.

5.13 Knudsen, K., Ruzek, J., Johnston, E. and Beedle, L. S.
WELDED PORTAL FRAMES TESTED TO COLLAPSE, Weld. Jnl. 33(9), p. 469s, September 1954

See also 4.1. Deviation from elastic predictions not significant, however, no inelastic deflections calculated.

5.14 Baker, J. and Roderick, J.
TESTS ON FULL SCALE PORTALS, Proc. Inst. Civ. Engrs., January 1952

See also 4.6. There was an early departure from the elastic load-deflection curve and the inelastic curve was curved rather than composed of linear segments.

5.15 Baker, J. and Heyman, J.
TEST ON MINIATURE PORTAL FRAMES, Struct. Engr. 28(6), 1950

See also 4.17. Deflection curve presented on small scale however plot appears curved rather than linear.

5.16 Baker, J. and Eickhoff, K.
THE BEHAVIOR OF SAW-TOOTH PORTAL FRAMES, Conf. on the Correlation between Calculated and Observed Stresses and Displacements in Structures. Inst. Civil Engs., p. 107, 1955

See also 4.17. Deflections did not deviate much from elastic predictions and showed tendency to be linear in inelastic range.

5.17 Baker, J. and Roderick, J.
AN EXPERIMENTAL INVESTIGATION OF THE STRENGTH OF SEVEN PORTAL FRAMES, Trans, Inst. Weld., 1(4), 1938

See also 4.10. Curves for load-deflection are of normal form, slightly segmental.

5.18 Baker, J. and Roderick, J.
FURTHER TESTS ON BEAMS AND PORTALS, Trans. Inst. Weld. 3(2), 1940, p. 83

Portal tests showed a slightly segmental load-deflection curve. In beam tests there was immediate strain-hardening when the shear forces were high. See also 4.11.

5.19 Little, D. and Smith, A.
SOME STEEL STRUCTURES DESIGNED BY PLASTIC THEORY, Proc. Inst. Civ. Engs., Part III, 4, 1955

Test of deflections in a full size commericial plastically designed structure. With all bays loaded (to avoid distribution of loads) deflections were 90% of calculated values (cladding present).

5.20 Vickery, B.
THE INFLUENCE OF DEFORMATIONS AND STRAIN-HARDENING ON THE COLLAPSE LOAD OF RIGID FRAME STEEL STRUCTURES, Inst. Eng. Aust., CE Trans., Vol. CE 3, No. 2, September 1961

See also 4.12. No plateau in load-deflection curves. Considering strain-hardening gave a much improved estimate of deflections.

5.21 Baker, J. and Charleton, T.
A TEST ON A TWO STORY SINGLE BAY PORTAL STRUCTURE, Brit. Weld. Jnl., May 1958

See also 4.14. Deflections were predicted with quite good accuracy.

5.22 Roderick, J.
THE ELASTO-PLASTIC ANALYSIS OF TWO EXPERIMENTAL PORTAL FRAMES, Struct. Eng., August 1960

Checked deflections of portals described in 4.6 and 5.14. Considered strain-hardening but agreement was still not good so also considered effect of movement of column tops. Agreement then satisfactory. Used curvature methods.

5.23 Blessey, W.
TESTS ON BEAMS AND FRAMES TO FAILURE, February 7, 1958

See also 4.16. Calculated deflections were under estimated.

5.24 Roderick, J. and Phillips, I.
CARRYING CAPACITY OF SIMPLY SUPPORTED MILD STEEL BEAMS, Res. (Engineering Structures Supplement), Colston Papers, 2(9), 1949

Compares results of 8 tests with various deflection calculations (tests in 2.21). Current methods gave good agreement. Load-deflection curves showed only a slight knee effect.

5.25 Yen, Y. C., Lu, L. W. and Driscoll, G. C., Jr.
TESTS ON THE STABILITY OF STEEL FRAMES, Weld. Research Council Bulletin 81, September 1962

See also 4.19. Horizontal deflections were of buckling type. Vertical deflection not recorded. Deflected shape plotted and was curved rather than hinged indicating non-mechanism collapse.

5.26 Hartman, F.
DIE FORMANDERUNGEN EINFACHER UND DURCHLAUFENDER STAHLTRAGER MIT EINEM VERSUCH, Schweiz, Bauztg. 101(75), 1933

See 3.11. Deflections calculated by curvature methods were overestimated in this case.

5.27 Highway Research Board
THE AASHO ROAD TEST. REPORT NO. 4. BRIDGE TEST, 51D., Publication 953. National Academy of Science

See also 3.15. Deflections in bridges (non-composite) as failure approached were of order of 14"--span/43.

5.28 Popov, E. and McCarthy, R.
DEFLECTION STABILITY OF FRAMES UNDER REPEATED LOADS, Proc. ASCE 86(EM1), January 1960, p. 61

See also 4.4. Test indicated frame stiffer than predicted in elastic range but as no peak load capacity was reached deflection criterion was used in inelastic zone. Large secondary moments produced in columns by deflection.

5.29 Reynolds, C.
ANALYSIS AND DEFLECTIONS ON NON-LINEAR STRUCTURES, Civ, Engg. (London), 55(649), p. 1037, August 1960

Tests described in 2.41 and 3.24. Obtained good agreement between test deflections and predictions using complementary energy theory.

GROUP 6

Shear

6.1 Yang, C. and Beedle, L. S.
THE BEHAVIOR OF I AND WF BEAMS IN SHEAR, Fritz Engineering Laboratory Report 205B.21, Lehigh University, 1951

Nine tests were performed with double point loading on simply supported beams. Spans from 12" to 36". Section 4I7.7. Noticed a marked reduction in plastic moment due to shear.

6.2 Driscoll, G. C., Jr. and Beedle, L. S.
THE PLASTIC BEHAVIOR OF STRUCTURAL MEMBERS AND FRAMES, Weld. Journal 36(6), p. 275s, June 1957

See also 3.13. Shear had an influence over center support in continuous beam test. Shear deflections were of same magnitude as flexural. Principal stresses caused early shear yielding in web. Ult. load still reached.

6.3 Fujita, Y.
THE INFLUENCE OF SHEAR ON THE FULLY PLASTIC MOMENT OF BEAMS, Fritz Engineering Laboratory Report 205.23, Lehigh University, 1955

Tested 5 12WF27 beams. Test results exceeded theory loadwise and conclusion for these tests was that shear was not critical.

6.4 Yang, C., Beedle, L. S. and Johnston, B.
RESIDUAL STRESSES AND THE YIELD STRENGTH OF STEEL BEAMS, Weld. Journal 31(4), p. 205s, April 1952

Described in 2.3. Continuous beam tests, as loads were moved towards supports the effects of shear became more evident.

6.5 Hall, W. and Newmark, N.
SHEAR DEFLECTIONS OF WIDE FLANGE BEAMS IN THE PLASTIC RANGE, Trans. ASCE, 122, p. 666, 1957

See also 2.4. Two tests in which shear deflections were critical. However load capacity was not significantly affected. 8WF58. 9' center span, 2 4' 6" half outer spans. Varied position of two single point loads in center span.

6.6 Baker, J. and Roderick, J.
FURTHER TESTS ON BEAMS AND PORTALS, Trans. Inst. Weld. 3(2), 1940, p. 83

See also 4.11. 14 tests on simply supported beams with double point loads kept 5" apart. Span increased from 7" to 13.5". 1 1-4"/1 1-4" I. With loads near supports there was no plateau in load deflection curves. No load capacity decrease.

6.7 Kusuda, T. and Thurlimann, B.
STRENGTH OF WIDE FLANGE BEAMS UNDER THE COMBINED INFLUENCE OF MOMENT, SHEAR AND AXIAL FORCE, Fritz Engineering Laboratory Report 248.1, Lehigh University, 1958

See also 2.13. Strain-hardening appeared almost immediately after yielding. Failure was by weld fracture. Results fitted calculations.

6.8 Haaijer, G.
THE TANGENT MODULUS IN SHEAR IN UNIAXIALLY PLASTICIZED STEEL, Fritz Engineering Laboratory Report 241.1, Lehigh University

Three tests to find G_t in the strain-hardening region. Found a levelling off of G_t as shearing progressed.

6.9 Nelson, H., Wright, D. and Dolphin, J.
DEMONSTRATIONS OF PLASTIC BEHAVIOR OF STEEL FRAMES, Proc. ASCE 83(EM4), paper 1390, October 1957

See also 2.11, 3.8, 4.5, and 5.12. One test on 12WF25. 8' span. Two point loads 21" from supports. No strain-hardening, reduction due to shear was only 5%. Flaking of web whitewash indicated shear hinge.

6.10 Johnston, B. and Kubo, G.
WEB CRIPPLING AT SEAT ANGLE SUPPORTS, Fritz Engineering Laboratory Report 192A2, 1941

Four tests on 12WF50 over a 5' span. Contains useful measurements of deflected shape of web. Tests not carried to failure.

6.11 Gozum, A.
EXPERIMENTAL STUDIES OF SHAPEDOWN OF CONTINUOUS BEAMS, Fritz Engineering Laboratory Report 205G.1, Lehigh University, 1954

See also 2.12 adn 3.5. In two continuous beam test there was no plateau in deflection curves under single point loads.

6.22 Hendry, A.
AN INVESTIGATION OF THE STRENGTH OF CERTAIN WELDED PORTAL FRAMES IN RELATION TO THE PLASTIC METHOD OF DESIGN, Struct. Eng. 28, (1950)

See also 4.21. Portal test with load near column tops showed marked shear effect. Also tested 20 beams of 3" to 30" span with single and double point loads, 4"/3", 3"/1 1-2", 3"/1" and 1"/1 1-4" I beams. As shear increased so did strain-hardening effect although plastic moment was reduced. Also tested were two 36"/9" portals, 3"/1 1-2" I. Capacity decreased but strain-hardening increased for single point load compared with double point load.

6.23 Roderick, J. and Phillips, I.
CARRYING CAPACITY OF SIMPLY SUPPORTED MILD STEEL BEAMS, Res. (Engineering Structures Supplement), Colston Papers, 2(9), 1949

See also 2.21, 5.24. Single point load tests showed plateau in load deflection curves. Effect of shear on load capacity not observed.

GROUP 7

Compression Plastic Modulus

7.1 Ketter, R., Beedle, L. S. and Johnston, B.
COLUMN STRENGTH UNDER COMBINED BENDING AND THRUST, Weld. Journal 31(12), p. 607-s, 1952

Compares results from early T series column tests (F. L. Reports 205.A30, A35) with modulus predictions. Agreement reasonable, but factors such as lateral buckling caused fluctuations. Variety of test conditions.

7.2 Driscoll, G. C., Jr. and Beedle, L. S.
THE PLASTIC BEHAVIOR OF STRUCTURAL MEMBERS AND FRAMES, Weld. Journal 36(6), p. 275s, June 1957

See also 2.2. Two tests on eccentric stub columns. 12 WF36, 36" long. Both results fell slightly above interaction curve however yielding occured earlier than predicted.

7.3 Kusuda, T. and Thurlimann, B.
STRENGTH OF WIDE FLANGE BEAMS UNDER COMBINED INFLUENCE OF MOMENT, SHEAR AND AXIAL FORCE, Fritz Engineering Laboratory Report 248.1, Lehigh University, 1958

See also 6.7. The results of three tests well predicted by theory presented.

7.4 Hendry, A.
AN INVESTIGATION OF THE STRENGTH OF CERTAIN WELDED PORTAL FRAMES IN RELATION TO THE PLASTIC METHOD OF DESIGN, Struct. Eng. 28, (1950)

See also 4.21, 6.22. Did 4 portal tests in which column forces were high and observed a reduction in strength over simple theory (however strain-hardening complicated effect). Also tested C frames with 3"/1 1-2" I, 18"/20" to check reduction in modulus-agreement good.

GROUP 8

Local Buckling

8.1 Fisher, J., Driscoll, G. C., Jr. and Schutz, F.
BEHAVIOR OF WELDED CORNER CONNECTIONS, Weld. Journal, 37(5), p. 216-s, May 1958

Corner connection tests in which local buckling occurred. Web and flange buckling shapes measured and advent noted. Local buckling not always catastrophic. Used 14WF30, 8B13, 24WF100, 30WF108, 36WF230.

8.2 Haaijer, G. and Thurlimann, B.
ON INELASTIC BUCKLING OF STEEL, Proc. ASCE 84(EM-2), p. 1581, April 1958

Tests on short rectangular columns to illustrate buckling above yield stress (0.74"/0.54"). Tests on angles to illustrate plate buckling (2.3"/4.9") equal legs. Tests on WF section to check application of theory (10WF33, 8WF24, 10WF39, 12WF35, 10WF21). Results obtained compared with presented theory.

8.3 Topractsoglu, A., Beedle, L. S. and Johnston, B.
CONNECTIONS FOR WELDED CONTINUOUS PORTAL FRAMES, Weld. Journal 30(7), 30(8), 31(11), 1951-52

Fourteen connection tests, local buckling observed in all. Occurrence well documented. 8WF31, 14WF30, 8B13. Photographs give good illustration of buckling process.

8.4 Fisher, J. W. and Driscoll, G. C., Jr.
CORNER CONNECTIONS LOADED IN TENSION, Fritz Engineering Laboratory Report 205C.23, Lehigh University, 1958

Tests on specimens of Ref. 8.3 in tension, plus three new specimens. Local buckling was observed in a number of tests-although effect not always catastrophic, as was the case in Ref. 8.3.

8.5 Toprac, A.
AN INVESTIGATION OF WELDED RIGID CONNECTIONS FOR PORTAL FRAMES, Weld. Journal, January 1954, p. 40-s

Tests on 11 joints in which local buckling was frequently observed with sometimes catastrophic effects. 8B13, 6I12.5, 6I17.25.

8.6 Driscoll, G. C., Jr. and Beedle, L. S.
THE PLASTIC BEHAVIOR OF STRUCTURAL MEMBERS AND FRAMES, Weld. Journal 36(6), p. 275-s, June 1957

See also 3.13, 6.2. Local buckling occurred in stub column tests (12WF36) and in corner connection tests on 30WF108. Not reported in gable or beam tests. Unloading began to occur in connection test immediately after local buckling.

8.7 Yang, C., Beedle, L. S. and Johnston, B.
RESIDUAL STRESSES AND THE YIELD STRENGTH OF STEEL BEAMS, Weld. Journal 31(4), p. 205s, April 1952

See also 2.3. Local buckling occurred in simple beam tests with overhanging ends. 14WF30. Caused immediate falling off of load. Occurred over support.

8.8 Lee, G. and Ketter, R.
THE INFLUENCE OF RESIDUAL STRESSES ON THE STRENGTH OF MEMBERS OF HIGH STRENGTH STEEL, Fritz Laboratory Report 269.1A, 1958

In stub column test on 8WF31 local buckling occurred early before yield load was reached. A242 steel.

8.9 Nelson, H., Wright, D. and Dolphin, J.
DEMONSTRATIONS OF PLASTIC BEHAVIOR OF STEEL FRAMES, Proc. ASCE 83(EM4), paper 1390, October 1957

See also 2.11, 3.8 and 4.5. In propped cantilever test (8B13) local buckling occurred at support. However in test on 6WF15.5 as a beam with constant moment no local buckling occurred although b/t=22. Local buckling was not catastrophic in propped cantilever.

8.10 Fujita, Y.
THE INFLUENCE OF SHEAR ON THE FULL PLASTIC MOMENT OF BEAMS, Fritz Engineering Laboratory Report 205B.23, Lehigh University, 1955

See also 6.3. In tests on five 12WF27 simple beams local buckling was prime cause of failure in each.

8.11 Tamaro, G.
COLUMN CURVE FOR LOW SLENDERNESS RATIOS, Fritz Engineering Laboratory Report 354.146, 1961 wrong place

Tested 5 stub columns in which local buckling occurred although not theoretically predicted, section-6WF25. There was some post-buckling strength.

8.12 Graham, J., Sherbourne, A., Khabbaz, R. and Jensen, C.
WELDED INTERIOR BEAM-TO-COLUMN CONNECTIONS, A. I. S. C. 1959, Proc. National Engineerg. Conference

Thirteen tests in which local buckling occurred. 8WF31, 8WF48, 8WF58, 8WF67, 12WF40, 12WF65, 12WF99, and others. Tests were on X shapes and on similar 3-dimensional shapes, as well as direct web buckling tests. Local buckling was generally not catastrophic.

8.13 Sawyer, H.
POST-ELASTIC BEHAVIOR OF WIDE-FLANGE STEEL BEAMS, Proc. ASCE 87(ST8), p. 43, 1961

See also 2.7. Local buckling occurred in the 18 tests, however point of occurrence was not noted.

8.14 Baker, J. and Charleton, T.
A TEST ON A TWO STORY SINGLE BAY PORTAL STRUCTURE, Brit. Weld. Jnl., May 1958

See also 4.14. Flange local buckling was observed but appeared to affect bracing more than frame itself. 4"/3"/10" I.

8.15 Lee, G. and Ketter, R.
THE EFFECT OF RESIDUAL STRESSES ON THE STRENGTH OF MEMBERS OF HIGH STRENGTH STEEL, Frit Engineering Laboratory Report 269.1A, 1958

See also 14.1. Flange local buckling occurred in the stub column tests. 8WF31. Buckling was before attainment of yield load.

8.16 Lee, G. and Galambos, T. V.
THE POST-BUCKLING STRENGTH OF WIDE FLANGE BEAMS, Proc. ASCE, 88(EM1), 1962

See also 2.6. Tests are well documented and record point of occurrence of local buckling (visual criterion). It appears to occur at strains corresponding approximately to the Haaijer theory for the sections and steel (A7) tested.

GROUP 9

Instability of Compression Members

General reference:

9.0 Austin, W.
STRENGTH AND DESIGN OF METAL BEAM COLUMNS, Proc. ASCE, 87(ST-4), April 1961

GROUP 10

Lateral Buckling

General reference:

10.0 Lee, G. C.
A SURVEY OF THE LITERATURE ON THE LATERAL INSTABILITY OF BEAMS, Weld. Res. Council Bulletin 63, August 1960

GROUP 11

Connections

General reference:

11.0 Fisher, J. W., (Ch. 9) and Rumpf, J. (Ch. 10)
STRUCTURAL STEEL DESIGN, Fritz Engineering Laboratory Report 354.3, Lehigh University, 1962.

GROUP 12

Variable Loading

General reference:

12.0 Reference i, Chapter 6.4

GROUP 13

Frame Instability

General reference:

13.0 Lu, L. W.
A SURVEY OF THE LITERATURE ON THE INSTABILITY OF FRAMES, Fritz Engineering Laboratory Report 276.2, December 1961

GROUP 14

High Strength Steel

14.1 Lee, G. and Ketter, R.
THE EFFECT OF RESIDUAL STRESSES ON THE STRENGTH OF MEMBERS OF HIGH STRENGTH STEEL, Fritz Engineering Laboratory Report 269.1A, 1958

Used A242 steel, 8WF31. Did 9 tension tests, 2 compression tests, 1 residual stress measurement, and 2 columns loaded axially about their weak axis, $1/r_y$=54,72. Residual stress effect seemed less than for A7.

14.2 Feder, D. and Lee, G.
RESIDUAL STRESSES IN HIGH STRENGTH STEEL Fritz Engineering Laboratory Report 269.2, 1959

Used A242 steel and did 15 tension tests, 2 compression tests, 3 residual stress tests, 3 stub columns and 4 axially loaded columns ($1/r_y$=62,75). Used 12WF50, 12WF65. Conclusions as for 14.1.

14.3 Nitta, A., Ketter, R. and Thurlimann, B.
STRENGTH OF ROUND COLUMNS OF USS "T1" STEEL, Fritz Engineering Laboratory Report 272.1, 1959

Ten tension tests, 2 poissons ratio tests, 11 residual stress tests, 5 stub column tests. Residual stress levels were of order of half the yield stress.

14.4 Nitta, A. and Thurlimann, B.
EFFECT OF COLD BENDING ON COLUMN STRENGTH, Fritz Engineering Laboratory Report 272.2, 1960

Used T-1 steel. 6 residual stress tests, 10 stub column tests, 5 columns and 17 tension tests, (plus some A-7 tests). Tests were directed towards column behavior.

14.5 Ueda, Y. and Galambos, T. V.
COLUMN TESTS ON 7 1-2" ROUND SOLID BARS, Proc. ASCE 88(ST-4), August 1962

Used round T-1 steel. 5 tension tests on annealed and 5 on as-used steel. Also 1 stub column test and 1 axially loaded column from each of these two groups. Found eccentricity to be more important than for lower strength steels.

14.6 Fujita, Y. and Driscoll, G. C., Jr.
STRENGTH OF ROUND COLUMNS, Proc. ASCE 88(ST-2), April 1962

Used round T-1 steel. Included 3 stub column tests as well as 8 column and 2 beam-column tests.

ADDENDA

3.26 Omerod, A.
BEAMS LOADED IN NON-PRINCIPAL PLANES, Civ. Engg. (London), 56(656), March 1961, p. 336

See also 2.45. Tested section on 27" span with one end fixed. Single point load at third point nearer fixed end. Load to form mechanism was only slightly above predicted load.

3.27 Heyman, J. and Dutton, V.
PLASTIC DESIGN OF PLATE GIRDERS WITH UNSTIFFENED WEBS, Weld. and Metal Fabricn., 22(7), July 1954, p. 268

See also 6.24. Two continuous beam tests. Shear had a significant effect. Mechanisms formed consecutively rather than simultaneously.

3.28 Sparacio, R.
LA RICERA DEL MINIMO COEFFICIENTE DI SICUREZZA A ROTTURA IN PRESENZA DI CARICHI VARIABILI E DISTORSIONI, G. Gen. Civ. 98(10), p. 794-807, October 1960

See also 2.48. Two tests with two 80cm spans, two single point loads. Predicted load capacity was attained with load capacity continuing to increase after M_p.

6.24 Heyman, J. and Dutton, V.
PLASTIC DESIGN OF PLATE GIRDERS WITH UNSTIFFENED WEBS, Weld. and Metal Fabricn., 22(7), July 1954, p. 268

Six beam tests to check shear effect. Used 2-7/8" x 7/8" beams, single point loads on single and double spans. Single spans from 6" to 30", double spans each 13-1/2". Shear modification presented was confirmed by tests.

6.25 Green, A. and Hundy, B.
PLASTIC YIELDING OF I-BEAMS, Engg., 184, (4767), July 1957, p. 74 and 184(4768), July 1957, p. 112

Tested two types of 1" deep beams. Introduced section changes to emphasize shear effect. Single point, non-central loads. 23 beams tested on various span lengths. Test results were compared with various theories predicting effect of shear on M_p.

AUTHOR LIST

Authors quoted on addenda pages:

IV. ACKNOWLEDGMENTS

This study is part of a general investigation "Plastic Design in High Strength Steel" currently being carried out at Fritz Engineering Laboratory of the Civil Engineering Department of Lehigh University under the general direction of Lynn S. Beedle. The investigation is sponsored by the American Institute of Steel Construction.

Fix. TVG is Proj. Dir

The author expresses his thanks to Professor T. V. Galambos who provided the incentive for the study, Miss P. Orsagh of Lehigh Library who helped in the location of many references and Miss Marilyn Courtright who typed this report.

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DEPARTMENT OF CIVIL ENGINEERING
FRITZ ENGINEERING LABORATORY

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April 10, 1963

MEMBERS OF LEHIGH PROJECT SUBCOMMITTEE (WRC)

Higgins, T. R.	Fox, G.	Kreidler, C.
Amirikian, A.	Grover, L.	Newmark, N. M.
Beedle, L. S.	Jameson, W. H.	Pisetzner, E.
Crowley, J. M.	Johnston, B. G.	Stuchell, R. M.
Dill, F. H.	Kavanagh, T. C.	Vasta, J.
Epstein, S.	Ketter, R. L.	

Gentlemen:

RE: THE EXPERIMENTAL BASES FOR PLASTIC DESIGN - a survey of the literature. Fritz Laboratory Report No. 297.3.

Please find enclosed a copy of Fritz Laboratory Report 297.3. This report originated as part of a study to determine the most desirable testing program for the project "Plastic Design in High Strength Steel". As the information collected is not available elsewhere in any single annotated source, it is planned to submit the report for publication as a WRC Bulletin.

We would appreciate your approval of this course of action. A post-card has been enclosed for your convenience in replying.

Sincerely yours

Theodore V. Galambos

Theodore V. Galambos

TVG/va
Encl: