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Summary report, Lehigh University, (1957)

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LEHIGH UNIVERSITY INSTITUTE OF RESEARCH

WELDED CONTINUOUS FRAMES AND THEIR COMPONENTS

Interim Report No. 38

SUMMARY REPORT 1957

by Project Staff

December, 1957

Welded Continuous Frames and Their Components

Interim Report No. 38

SUMMARY REPORT - 1957

by

Project Staff

(Not for publication)

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Project Staff: L. S. Beedle, G. C. Driscoll, J. W. Fisher, T. V. Galambos, R. L. Ketter, T. Kusuda, L. W. Lu, B. Thurlimann

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Fritz Engineering Laboratory
Lehigh University
Bethlehem, Pennsylvania

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WELDED CONTINUOUS FRAMES AND THEIR COMPONENTS

Project Objectives

The original objectives, approved at the March, 1950, meeting of the Lehigh Project Subcommittee are as follows:

1. To determine the behavior of steel beams, columns and continuous welded connections with emphasis on plastic behavior, and to develop theories to predict such behavior.

2. To determine how to proportion various types of welded continuous frames to develop the most balanced resistance in the plastic range so that the greatest possible collapse load will be reached.

3. To determine procedures of analysis that will enable one to calculate the collapse loads of welded continuous frames and to verify the analysis by suitable tests.

4. To determine procedures of analysis that will enable one to calculate the elastic and permanent deformations in welded continuous frames in the range intermediate between elastic limit and collapse load.

5. To explore limitations in the application of plastic range design over and above deformation limitations, namely, fatigue, local buckling, lateral buckling, etc.

6. To develop practical design procedures for the utilization of reserve plastic strength in the design of continuous welded frames.

In brief, then, the program consists of:

1. Column, Beam, and Connection Studies (Frame Components)

2. Frame Studies (Integral Behavior)

3. Practical Applications (Methods of analysis and design with due regard to limitations such as fatigue, deflections, local buckling, etc.).

<u>Welded Continuous Frames and Their Components</u>

Outline of Current Program (1957 - 1958)

PHASE	PROJECT	WORK TO BE DONE (Analysis, Test, Development, Report)	PERSONNEL
I. EVALUATION, ANALYSIS, DESIGN			
205-III	Commentary	Report (Int. 33)	
205-IV	Design Manual and Specifications	Review AISC Manuscript	Beedle Thurlimann Ketter Driscoll
205-V	Design Procedures	Report on "chart" solutions (4 reports involved) Development for multi-story frames (follows 205-VI)	
205-VI	Analysis Procedures	Report on basic methods (Interim Report 27) Development for multi-story frames	
205-IX	Regional Conferences		
II. STUDIES OF STRUCTURES			
205D-I	Portal Frames-Vertical load	Report	Driscoll Lu
205D-II	Portal Frames-Combined load	Report on gable frame tests	
205D-III	Industrial Frames	Report on 2-span frame test. Development and test of 2-span frame of extreme design	

PHASE	PROJECT	WORK TO BE DONE (Analysis, Test, Development, Report)	PERSONNEL
III. STUDIES OF COMPONENTS			
205A-II	Simple columns with thrust & moment	Report (conditions c & d)	Ketter Galambos
		Analysis and Report (condition b)	
		Continue approved tests	
205A-V	Lateral-torsional buckling of columns	Analysis and Report	
205C-II	Corner Connections (Size effect)	Report(P. R. # 23)	Driscoll Fisher
205C-IV	"Tension behavior of corner connections	Report	
205C-VI	Haunched connections	Report on initial study (Smith)	
		Analysis and report (plastic approach)	
		Confirming tests of tapered and curved knees.	
		Final report and design procedure	
248-I	Built-up Members in plastic design Initial studies	Survey, Analysis, test and report of shear-thrust influence	
248-II	Corner Connections with cut-outs	Analysis and report. Test(?)	Thurlimann Kusuda
248-IV	Stiffening Requirements	Analysis	
248-V	Buckling and effective width of deck plating	Analysis	

PHASE	PROJECT	WORK TO BE DONE (Analysis, Test, Development, Report)	PERSONNEL
MODIFICATIONS AND SPECIAL TOPICS			
205B-III	Influence of Shear on plastic moment	Report	Thurlimann
205E-III	Local buckling	Report (P.R.22)	Thurlimann
205E-V	Lateral buckling	Report on White's studies	Thurlimann
		Interim design guide and report	Thurlimann Kusuda
		Analysis	Ketter Galambos
		Confirming tests (coupled with 205H-II)	
205G-III	Repeated Loading Frames	Studies as part of frame Test 205D-III	
205G-IV	Preliminary studies of repetitively-loaded structures	Analysis	
205H-II	Lateral bracing--Fundamentals of bracing requirements	Analysis-effectiveness of various types for Industrial Frames	Ketter Galambos
		Tests	
241	Shear modulus in the plastic range	Report by Haaijer for Publication	Thurlimann
268-I	Rotation Capacity Beams and single-span frames	Report on beams	Driscoll Lu
		Report on single-span frames	
268-II	Multi-span frames	Analysis and report	
268-III	Fixed-base, Industrial frames	Analysis and report	

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205-III

Evaluation, Analysis, and Design, Phase III

"COMMENTARY" ON PLASTIC DESIGN

Statement of Problem:

The evaluation of a considerable amount of research has demonstrated the applicability of plastic analysis to structural design. As a justification for this design method, a report will be prepared in which the results of analysis and tests will be summarized. The results of studies of secondary design considerations will be presented. In some cases the more "exact" solutions will be simplified such that they are appropriate for design use.

Such a report would serve as a background for a practical design manual, a manual that is now being prepared by the American Institute of Steel Construction.

Outline of Work:

1. Preparation of reports stating the design problems, the applicable results of research, the necessary additional research, suggested rules of practice. (completed by Interim Report No. 26 and Progress Report 18)

2. Extension of item (1) and completion of Progress Report to Lehigh Project Committee (WRC)

3. Following approval by WRC Committee, report to be submitted to ASCE Committee on Plasticity Related to Design. The report would then be revised (as needed) by both the WRC and the ASCE Committees and issued as a joint WRC-ASCE report.

4. Revision from time to time as new information becomes available.

Current Program:

Effort is being made to complete item (2) during this period.

Evaluation, Analysis, and Design, Phase V

DESIGN EXAMPLES AND TECHNIQUES

Statement of Problem:

There is a need to illustrate the use of plastic design through the inclusion in reports of specific design examples; the results may be compared with those obtained by the conventional elastic methods.

While the basic theorems of plastic analysis and design are sufficiently general to enable one to obtain the solution to most structural problems, it is felt that further study and development will reveal techniques that will materially reduce the design time.

Outline of Work:

1. A review of the literature on methods of analysis with particular reference to minimum weight design.
2. A study of the basic behavior of typical structures at failure noting the transfer of resistance from one part of the structure to the other.
3. Explore new methods of solution to certain of these problem types (e.g. gable frames, tier buildings, etc.)
4. Formulation of rules for the selection of the more economical (least weight) combination of member sizes in a structure.
5. Preparation of reports on the above.

Current Program:

The current work is aimed at completing a series of reports.

Evaluation, Analysis, and Design, Phase VI

ANALYSIS PROCEDURES

Statement of Problem:

Although some methods of analysis have been rather widely publicized (such as the semi-graphical "equilibrium method") there is need for further explanation and illustration of some of the more recent methods.

Some methods are more suitable to one type of structure than to another. Applicability of the various methods should be studied. Also, certain simplifications may be made to existing methods of analysis, rendering them more suitable for design use.

As more is learned about the plastic behavior of structures and as additional types of structures are encountered, new methods of analysis may be required.

Outline of Work:

1. Summarize existing methods of analyzing structures for ultimate strength.
2. Report on the "Mechanism" method covering the problem of distributed load, use of instantaneous center, checking equilibrium by moment-balancing. Application to industrial building frames.
3. Study practical methods of analysis of tier buildings.
4. Develop analysis procedures for new structural problems encountered.

Current Work:

The current work is to complete Item (2).

Work will then begin on Item (3)

Portal Frames, Phase I

VERTICAL LOADING

Statement of Problem*:

Tests on full-size portal frames are desirable to confirm theories for calculating ultimate loads and deflections and to check the behavior of component parts of frames with isolated tests of connections,columms,and beams.

Two vertical load tests were made on frames fabricated from 8 inch sections to make the indicated comparisons with results of isolated beam, column, and connection tests.

Outline of Work:

This phase of the program was carried out as follows:

1. Design specimens, setups and test procedures.
2. Static load tests on two 14 foot span flat-roof frames with pinned bases.
3. Analysis and report.

Current Program:

A report on the results of the frame tests is being prepared to supplement Progress Report No. 7 which described testing methods.

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* See proposal dated August 25, 1950.

205 D-II

Portal Frames, Phase II

COMBINED LOADING

Statement of Problem:

By testing nearly full-scale portal frames of as-delivered, commercially available sections, the practical limitations that may be encountered in the application of plastic range design may be brought out in full force. This continuation of the frame test program introduces some additional factors such as side loading proportional to blast effects, different boundary conditions, different layout of frame, and larger rolled sections. Thus observations may be made on additional variables as well as gaining further checks on calculations of ultimate load and deflection.

Outline of Work:

A proposal was presented for nine tests of single-bay single-story portal frames with variables in geometry, boundary conditions and loading.*

The work definitely projected is as follows:

1. Design of test set-ups.
2. Test of flat-roof frame with pinned bases under vertical loads and horizontal loads proportional to usual wind load design practice.
3. Test of flat-roof frame with fixed bases under vertical loads and horizontal load proportional to usual wind load design practice. Test to be halted as soon as maximum load is reached.
4. Test of same flat-roof frame with fixed bases under equal vertical and horizontal loads to simulate blast loading.
5. Analysis and report.
6. Test of gabled frame with fixed bases under vertical and horizontal loading.
7. Analysis and report.

Current Program:

The current program is to finish a report on the gabled portal frame test.

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* Proposal dated November 15, 1952.

205D-III

Industrial Frames, Phase III

TWO-SPAN FRAME

Statement of Problem:

Multi-span frames by their greater degree of indeterminacy present more opportunity for increase in carrying capacity due to redistribution of moment. However, this might lead to limitations because of large rotation capacities required to allow the redistribution of moment. A test of a two span frame with a large theoretical reserve due to redistribution of moment is desirable.

Outline of Work:

Work on this phase of the project would be carried out as follows:

1. A test was carried out in the 1956 AISC spring conference, on a two-span gabled frame of reasonably average proportions rather than one requiring a large theoretical reserve due to redistribution of moment. The report on this frame would be completed.
2. Studies of proportions of two-span frames to select a suitable frame with a large effect due to redistribution of moment.
3. Design of test set-up.
4. Fabrication of frame with provisions made to measure "locked in" moments caused by accidental msialinements in fabrication.
5. Test of frame
6. Analysis and report

Current Program:

Complete the report on the 1956 spring conference frame. Work would then be started on item (2).

205A-II

Columns In Continuous Frames, Phase II

SIMPLE COLUMNS WITH THRUST AND MOMENT

Statement of Problem

As a prerequisite to the determination of the strength of members as they are found in building structures, it is first of all necessary to be able to define the behavior of a given member with given end conditions subjected to given conditions of loading. From this information interaction curves of maximum carrying capacity can be obtained. These could then be used in the design of members subjected to comparable loadings as well as form a basis for further developments toward the solution of the more general problems.

Outline of Work

1. A review of the various classical methods of determining the ultimate carrying capacity of beam-columns.

2. For a member subjected to axial thrust and equal end-moments, which result in single curvature deformations; the development of ultimate strength interaction curves for a typical wide-flange shape neglecting the influence of residual stress.

3. An extension of item 2 to include the influence of a symmetrical, cooling type of residual stress pattern.

4. An extension of items 2 and 3 for the following conditions of loading:

   a. Axial thrust plus end-moment applied only at one end of the member,
   b. axial thrust plus equal end moments resulting in double curvature deformation, and
   c. axial thrust plus moment applied at one end of a member while the other end is held fixed.

5. Representation of the interaction curves of 2, 3, and 4 in analytical form for use in design.

6. Carry out a series of tests on as-delivered wide-flange members to confirm the findings of the analytical study.

7. Preparation of reports on same.

Current Program

The current work is aimed at completing parts 4a, 4b, 5, 7 of the above.

Columns in Continuous Frames, Phase V

INELASTIC LATERAL TORSIONAL BUCKLING

Statement of Problem:

It has been observed that as a wide-flange beam-column is bent about is principal axis the strength as predicted from a consideration of the bending stiffness of the member about this axis is never reached. Prior to the attainment of the predicted load the member laterally bends and twists.

Since the most economical placement of material in a cross-section is the one that provides the greater bending stiffness in the direction of the anticipated greater bending moment, there will always be a difference in bending stiffness about each of the principal axes of the member: therefore, a possibility of lateral-torsional buckling.

A solution to this problem is needed to be able to predict the true strength of WF members bent about their major axis. The solution may also prove valuable in the determination of laterally unsupported length for lateral bracing requirements.

Outline of Work:

1. Review of the literature on this type of failure.

2. Develop a method of solution to the problem assuming that the member deforms in single curvature.

3. Establish "bending stiffness-axial load-curvature" relationships (graphical) about each principal axis. Assume annealed WF material.

4. Same as 3 with as-delivered member (idealized cooling residual stress pattern).

5. Develop interaction curves to show the seriousness of this type of failure. Comparison with previous work neglecting this factor.

6. Extension of work to include bi-axial loading problem (see Phase III).

7. Preparation of reports.

Current Program:

The current work is aimed at completing the first 5 of the above. Phase 2 of the above is presently being developed.

205C-II

Corner Connections, Phase II

SIZE EFFECT

Statement of Problem:*

Earlier experimental work done as Phase I consisted of a series of tests of corner connections of one size having different design details.

The question remained as to whether the results of this study on small members would hold true for connections fabricated to connect larger rolled sections.

This phase of the program was set up as a series of connections of varying sizes all of one of the types found to be suitable from the earlier work.

Outline of Work:

The plan of analytical studies and tests is as follows:

1. Select a series of geometrically similar rolled sections.
2. Design connections to join the several rolled shapes.
3. Design test setups and procedures.
4. Calculate predicted behavior of specimens.
5. Test four large size corner connections.
6. Correlate connection behavior with theory and compare results of different size connections.

Current Program:

This phase of study is completed and reported in Progress Report #23.

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* See proposal December 5, 1952.

205C-IV

Corner Connections, Phase IV

TENSION BEHAVIOR

Statement of Problem:

Common loadings on continuous frames require that the corner connections be subjected to combinations of bending and compressive thrust. Certain special cases require that the connections withstand bending and tensile forces. These special cases include buildings subjected to blast loading and the crowns of sharply-peaked gable roofs.

A few tension tests of corner connections are desirable to establish the behavior as a guide to design in the special cases cited, especially since the possibility of weld failure is increased in this type loading.

Outline of Program:

The program includes tension tests of several connections, basically those which have previously been tested in compression in Phases I and II of the corner connection project.

(1)* Test and report on thirteen corner connections joining 8B13 material. Connections previously tested in compression. (Lehigh)

(2)* Tension test and report on two identical corner connections joining 8B13 material. One connection tested in prime condition and one tested in compression prior to tension test. (Texas)

(3) Tension test on three corner connections tested in compression in phase II -- size effect series.

(4) Tension test on two 12WF36 corner connections taken from Frame 3 of project 205D. One corner prime condition, one corner previously strained in compression. (Undergraduate student project)

(5) Report on tests in paragraphs (3) and (4).

Current Program:

Current work is to finish item (5), which will complete this phase of study.

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*Reported Progress Report #15

205C-VI

Corner Connections, Phase VI:

HAUNCHED CONNECTIONS

Statement of Problem:*

Since there are several reasons for the use of haunched welded connections in structures proportioned by the plastic method, there is need for a simple yet accurate method of proportioning such haunches. The method should be such that it would fit into the philosophy of plastic design.

A design procedure should be developed which will assure the lateral stability of the sloping flange when the structure has reached its ultimate load.

Since it was thought necessary to maintain the haunch in an elastic state all previous research has followed that path. However, it is now thought that a fully plastic approach can now be used to proportion the haunch.

Outline of Work:

The project includes analytical studies confirmed by tests as follows:

1. Survey available methods of analysis.
2. Establish practical proportions of haunched connections.
3. Outline a simple method of plastic analysis for bending and shear stresses within haunch.
4. Develop design procedure.
5. Illustrative examples showing analysis and design for connections used in plastically designed frames.
6. Study optimum haunch lengths for plastically-designed frames.
7. Test several connections to correlate with theory.

Current Program:

Items 1 and 2 were completed and reported in Interim Report 37. Current work is aimed at completing items 3, 4, and 5. It is anticipated that Item 7 may be completed during the coming year.

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*See proposal December 1957 (Report No. 205.58)

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Built-Up Members in Plastic Design, Phase I, II, III, IV, V

Statement of Problem*:

Presently available experimental results on the plastic behavior of steel structures are exclusively restricted to structures fabricated with rolled sections. However, built-up members are very often used in practice, especially in ship structures. Typical examples are deck girders with openings in the corners and through the webs. A Vierendeel girder presents a further typical example. The application of "Plastic Design" to such members offers some new problems:

I.
1. Survey on built-up members in actual applications, with special emphasis on ship structures.

2. Theoretical study of the problems in connections with "Plastic Design".

3. Proposal for exploratory testing.

II. Behavior as influenced by cut-outs in corner connections or webs.

III. Stiffening requirements for deep webs as in deck girders.

IV. Local stiffening requirements of flanges and corners.

V. Effective width of deck plate in buckled state after formation of plastic hinge in girder.

Current Program:

Phase I (Outline of work 1-4) and Phase II have been completed and a progress report on these items is now in preparation.

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*See proposal dated 24 January 1955, page 3, Report #205C.18

INFLUENCE OF SHEAR ON PLASTIC MOMENT

Statement of Problem:

In general even in "Plastic Design" shearing forces are relatively small and do not govern a design. Their influence on the plastic moment is simply neglected. In special cases, however, high shearing stresses may lead to a reduction of the plastic moment. Such effects were previously observed in tests and also investigated theoretically. Nevertheless a systematic study was desirable to properly evaluate these effects.

Outline of Work:

Controlled tests on 5 specimens (12WF27) with different ratios of moment to shear were conducted. A parallel theoretical study of the problem was done. These and results available in the literature will be evaluated as to their design implications.

Current Program:

The test program is completed. A report on the work and the findings is pending.

205E-III

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Inelastic Instability, Phase III

LOCAL BUCKLING

Statement of Problem:

The importance of the local buckling problem has been emphasized by premature failure of tested continuous frames because of flange buckling.

The ultimate aim of the theoretical and experimental studies on this subject is to specify the dimensions of WF shapes such that they can safely be used in plastic design.

Outline of Work:

1. Review of available theories.

2. Tests to check the validity of the theory in the plastic range of steel.

3. On the basis of 2 select the valid theory or if possible propose a new theory.

4. Tests on WF shapes subjected to axial compression and pure moment.

5. Report

6. Summarize in a report available results of test on WF shapes subjected to moment gradient.

Current Program:

Parts 1 to 5 are completed. No work on part 6 has yet been started.

205E-V

Inelastic Instability, Phase V

LATERAL BUCKLING

Statement of Problem:

One of the basic assumptions made in designing a structure for ultimate strength by plastic design methods is that a "plastic" hinge can be formed. That is, that the section is capable of undergoing large rotations within a limited region so that the moments may be redistributed to develop the full strength of the structure.

To accomplish these large rotations, provisions must be made to prevent the member from failing prematurely by lateral buckling. The general purpose of this project is to establish a criterion for the lateral buckling of members in the region where part of the member has undergone some inelastic deformation.

Outline of Work:

1. Preliminary tests on fix-ended WF beams under constant moment.

2. Analytical study for comparison with part (1).

3. Extension of analytical study to cases of:

   (a) Varying moment gradient

   (b) Varying end conditions

4. Analytical study on the post-buckling strength of the beams for the different cases.

5. Confirmatory tests under various conditions.

Current Program:

Parts 1 to 3 are completed. An additional 4 tests under varying moment gradients have been conducted. A report on these tests and tentative design recommendations is in progress.

Repeated Loading, Part of Frame Test, Phase III

Statement of Problem:

Methods of "Plastic Analysis" consider in general only proportional loading, i.e. the ratios between all the loads of a given loading configuration stay constant up to the failure load. Actually cases of loading cycles, e.g. live loads in storage house, wind, craneloads etc., are more common than such an idealized loading. Theoretically it can be shown that a limited number of loading cycles can produce successive plastic deformations such that the deflections increase with each cycle. A critical limit called the shake-down load and always smaller than the corresponding ultimate load for proportional loading, can be defined, above which the deflections never cease to increase.

Such theoretical predictions based on simplifying assumptions need some experimental check to evaluate their seriousness with respec t to actual design.

Outline of Work:

Out of the approved program on full-scale frame tests, one frame will be subjected to cyclic loading prior to the failure test under proportional loading. Such a procedure will furnish the necessary results on the behavior of an actual frame under cyclic loading without requiring a special test.

Current Program:

Under consideration in connection with future frame tests.

Lateral Bracing Requirements, Phase II

THE FUNDAMENTALS OF BRACING REQUIREMENTS

Statement of Problem:

The general purpose of this project is an application of the results obtained for the study of inelastic lateral buckling of members to the actual requirements and detailing of structures. Specifically the project is concerned, firstly, with the method of expressing the rotation requirements of a structure in a form from which its lateral buckling strength may be determined. Secondly, it is concerned with the effectiveness of various types of lateral support under various conditions.

Outline of Work:

1. Review of methods for expressing the necessary rotation capacity in the "hinge" region in view of the results obtained under Project 205E-V and 205C-III.
2. Analytical study for purpose of determining necessary design criteria.
3. Analytical study of the effectiveness of various types of lateral bracing.
4. Experimental determination of the effectiveness of various types of lateral bracing.
5. Confirmatory tests of conclusions.

Current Program:

Because of its reliance on other projects, yet to be completed, there is no current program in this project.

Stress-Strain Relationships in the Plastic Range, Phase I

THE TANGENT MODULUS IN SHEAR

Statement of Problem:

Analysis of the local(plate) buckling problem shows that the buckling strength depends on the relationships between the increments of stresses and strains due to the deflection of the plate out of its plane*. For outstanding flanges the buckling strength depends to a large extent on the tangent shear modulus.

Thus, the important problem to be investigated is: The determination of the tangent modulus in shear of structural steel after it has been compressed into the plastic range.

Outline of Program:

1. Design a test set-up which will make it possible to compress and twist a tube successively and simultaneously.

2. Design a strain-gage which will measure the corresponding strains.

3. Perform tests

4. Compare test results with theoretical predictions.

Current Program:

A report is pending.

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* Haaijer, "PLATE BUCKLING IN THE STRAIN HARDENING RANGE", Proc. ASCE, Paper No. 1212, April 1957.
Haaijer, G., Thurlimann, B., "ON INELASTIC BUCKLING IN STEEL", Progress Report No. 22, May 1957.

ROTATION CAPACITY

Statement of Problem:

One of the basic assumptions involved in the computation of ultimate loads in plastic analysis is that the members and connections must have sufficient rotation capacity to allow all plastic hinges to develop. A general method for calculating the necessary rotation capacity is required. The problem is important because lack of adequate rotation capacity would be a serious limitation in the application of plastic analysis to structural design.

Outline of Work:

1. Develop analytical procedure for calculating deflection and rotation at plastic hinges at instant of formation of mechanism.
2. Analyze a number of structures by the method to determine the required rotation capacity of plastic hinges.
3. Compare the amounts of rotation capacity required with that attained in tests of similar structures or components.
4. Propose and conduct necessary tests to correlate with theory.
5. Evaluate problem of rotation capacity in the light of results obtained from analysis and tests.
6. Present practical method of calculating required rotation capacity of plastic hinges.

Current Program:

Work on this problem is the subject of a dissertation which is part of a Ph.D. program.

WELDED CONTINUOUS FRAMES AND THEIR COMPONENTS

List of Reports

I. Progress Reports, Published or for Publication

*1.	Luxion, W. and Johnston, B.G. PLASTIC BEHAVIOR OF WIDE FLANGE BEAMS Welding Journal, 27(11), p. 538-s F. L. 203.3, Reprint No. 63.	November 1948
*2.	Beedle, L. S.; Ready, J.A.; and Johnston, B. G. TESTS OF COLUMNS UNDER COMBINED THRUST AND MOMENT SESA Proceedings, 8(1) p. 109 F. L. 205.2, Reprint No. 72.	December 1950
*3.	Yang, C.H.; Beedle, L. S.; and Johnston B. G. PLASTIC DESIGN AND THE DEFORMATION OF STRUCTURES Welding Journal, 30(7), p. 348-s F. L. 205B.2, Reprint No. 75.	July 1951
*4.	Topractsoglou, A.A.; Beedle, L.S.; and Johnston, B.G. CONNECTIONS FOR WELDED CONTINUOUS PORTAL FRAMES	
	Part I - Test Results and Requirements for Connections, Welding Journal, 30(7), p. 359.s	July 1951
	Part II - Theoretical Analysis of Straight Knees Welding Journal, 30(8), p. 397-s	August 1951
	Part III - Discussion of Test Results and Conclusions, Welding Journal, 31(11), p. 543-s, F.L. 205C.6, Reprint No. 80	November 1952
5.	Yang, C.H.; Beedle, L.S.; and Johnston, B.G. RESIDUAL STRESS AND THE YIELD STRENGTH OF STEEL BEAMS Welding Journal, 31(4), p. 205-s F. L. 205B.8, Reprint No. 78	April 1952
6.	Ketter, R.L.; Beedle, L.S.; and Johnston, B.G. COLUMN STRENGTH UNDER COMBINED BENDING AND THRUST Welding Journal, 31(12), p. 607-s F. L. 205A.6, Reprint No. 81	December 1952
*7.	Ruzek, J.; Knudsen, K.E.; Johnston, E.R.; and Beedle, L.S. WELDED PORTAL FRAMES TESTED TO COLLAPSE Proceedings, SESA, 9(1), p. 159	1952
	Welding Journal, 33(9), p. 469-s (Reprint) F. L. 205D.4, Reprint No. 92	September 1954
-	Beedle, L. S. RESEARCH ON RIGID FRAMES Proceedings, AISC National Engineering Conference, p. 21 F. L. 205.18	April 1952

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*Reprints no longer available.

*8. Johnston, B.G.; Yang, C.H.; and Beedle, L.S. — May 1953
AN EVALUATION OF PLASTIC ANAYLSIS AS APPLIED TO STRUCTURAL DESIGN
Welding Journal, 32(5), p. 224-s
Reprint No. 87, F.L. 205.14

*9. Yang, C.H.; Knudsen, K.E.; Johnston, B.G.; and Beedle, L.S. — May 1953
PLASTIC STRENGTH AND DEFLECTIONS OF CONTINUOUS BEAM
Welding Journal, 32(5), p. 240-s
Reprint No. 86, F.L. 205B.9

10. Ketter, R.L.; Kaminsky, E.L.; and Beedle, L.S. — Oct. 1953
PLASTIC DEFORMATION OF WF BEAM COLUMNS
**ASCE Proceedings Paper 330(V.79)
Reprint No. 91, F.L. 205A.12
Discussion and closure contained in Separates 532, 606 (Reprint Nos. 98, 101)

11. Ketter, R. L.
STABILITY OF BEAM COLUMNS ABOVE THE ELASTIC LIMIT — May 1955
ASCE Proceedings Paper 692 (V.81)
Reprint No. 103, F.L. 205A.14

12. Beedle, L.S. — April 1954
RECENT TESTS OF RIGID FRAMES
Proceedings, AISC National Engineering Conference
p. 13, F.L. 205.23

*13. Ketter, R. L. and Beedle, L. S. — Oct. 1954
Discussion on "STRENGTH OF COLUMNS ELASTICALLY RESTRAINED AND ECCENTRICALLY LOADED" by Fisher, Bijlaard, and Winter.
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WELDED CONTINUOUS FRAMES AND THEIR COMPONENTS

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COLUMN STUDIES
(Progress Report 205A-II)

In July 1957, a report prepared by Theodore V. Galambos and Robert L. Ketter was sent out to the members of the Committee. The title of this interim report was "Further Studies on the Strength of Columns Under Combined Bending and Thrust". The report was Interim Report No. 36 in the project of Welded Continuous Frames and Their Components. The Fritz Laboratory number was 205A.19. A condensed version of this report was presented by Theodore V. Galambos to the ASCE Annual Convention on October 14, 1957 at New York City.

The following items were presented in the report:

1) Interaction curves were calculated for pin-ended columns, under axial thrust and bending moment in the strong plane for two equal end-moments causing single curvature, and for an end-moment applied at one end only. The influence of lateral torsional buckling was neglected in the calculations. Interaction curves were prepared for A-7 structural steel, with and without the inclusion of the influence of cooling residual stresses.

2) Approximate design equations were developed.

3) Comparisons of the theoretical results with column test data were made.

Future Work: A revised copy of Interim Report No. 36 will be soon presented to the members of the Committee for approval for publication in the Engineering Mechanics Journal of the ASCE.

The members of the Committee are now invited to make comments and ask questions about this report.

Theodore V. Galambos

Theodore V. Galambos

Lateral-Torsional Buckling of Wide Flange Columns

(Progress Report 205A-V)

Summary of the work currently underway on the the general problem of column buckling due to the combined action of bending and lateral torsional twisting, when the material is strained into the inelastic range.

Introduction An investigation of column test reports shows that columns, loaded by an axial force plus end-moments causing bending in the strong plane, do not fail by pure bending in the plane of the moments, but by a combined bending in a strong plane plus twisting in the direction of the weak plane. This is called lateral-torsional buckling of columns.

The problem: It is desired to find the moment at which lateral torsional buckling sets in for a given column of known length and with a constant axial compressive load. The problem is solved here for the following boundary conditions: A pin ended column (pin-ended in the direction of principal axes for bending and torsion) is subjected to equal end-moments, M_o, in the plane of the web (strong direction) causing a deflection profile of simple curvature. (see Fig 1).

Assumptions

1) Strains are assumed to be proportional to the distance from the neutral axis.

2) The cooling-residual-stress pattern shown in Fig 1 is assumed. This is a fair approximation of cooling-residual stresses present in rolled wide-flange profiles. This then limits the solution to rolled wide-flange profiles of A-7 structural steel.

3) The tensile-compressive stress-strain curve is assumed to be that shown on Fig 1. This assumption indicates that the bending stiffness EI_y and the warping stiffness EI_w of the yielded portions of the cross-section is zero.

4) The torsional stiffness GK_t is assumed to remain undiminished by yielding. This is a fair assumption when only part of one flange is yielded. When one whole flange and part of the web is yielded, G is not 11.5×10^6 psi anymore, but it varies in an as yet unknown pattern. Substitutions of G = 2,500 ksi (the shear modulus at strain hardening as determined by G. Haaijer) gives results which are too low. However, the uncertainty of G does not limit the solution of the problem much, because its influence becomes important only at small values of the slenderness-ratios ($L/r_x < 40$)

5) The whole length of the column is assumed to be equally yielded, i.e. the worst possible condition has been assumed.

6) Deflections in the weak plane are neglected.

The Solution: The three basic homogeneous differential equations of lateral torsional buckling (as given by Bleich in "Buckling Strength of Metal Structures) are reduced to two by the ommission of deflections in the strong plane. By assuming a sinusoidal deflection curve for the twisting and for the strong plane deformations, and by setting the determinant of the differential equations equal to zero, the following equation is detained for the critical buckling load:

$$r_o^2(P_o - P_T)(P_o - P_Y) - P_o^2(e_y - y_o)^2 = 0 \qquad \ldots (1)$$

where

$$r_o^2 = \frac{1}{A}(I_x + I_y) + y_o^2 + \beta_x e_y$$

A = cross-sectional area of unyielded portion of the cross-section
I_x and I_y are the moments of inertia of the unyielded portion about the x and y axis respectively.
y_o = distance from the centroid to the shear center along the y-axis of the unyielded portion of the cross-section.
e_y = eccentricity of load

$$\beta_x = \frac{1}{I_x}\left(\int y^3 dA + \int x^2\, dA\right) - 2y_o;$$

P_o = Applied axial load

$$P_T = \frac{1}{r_o^2}\left(GK_T + \frac{\pi^2 EI_w}{L^2}\right)$$

$$P_Y = \frac{\pi^2 EI_y}{L^2}$$

Solving equation (1) for the critical length for a given axial load and on assumed yield pattern, interaction curves (Fig 1) can be directly computed for any wide-flange section.

Calculations were made for three sections: the 14WF426, the 4WF13, and the 8WF31 column. Of these, the 8WF31 gave the lowest allowable loads, and therefore it was chosen for comparison with test results. (Fig 2) This section can also be used for the determination of design equations.

Future Work:

1) It will be attempted to remove all dubious assumptions from the solution; the variation of GK_T will especially be tried to be determined.

2) Solutions to other boundary conditions and loading cases will be attempted.

3) The ultimate goal is to unify all the solutions into design curves.

Theodore V. Galambos

Fig. 2

Comparison with Testresults for $\frac{ec}{r_x^2} = 1.0$

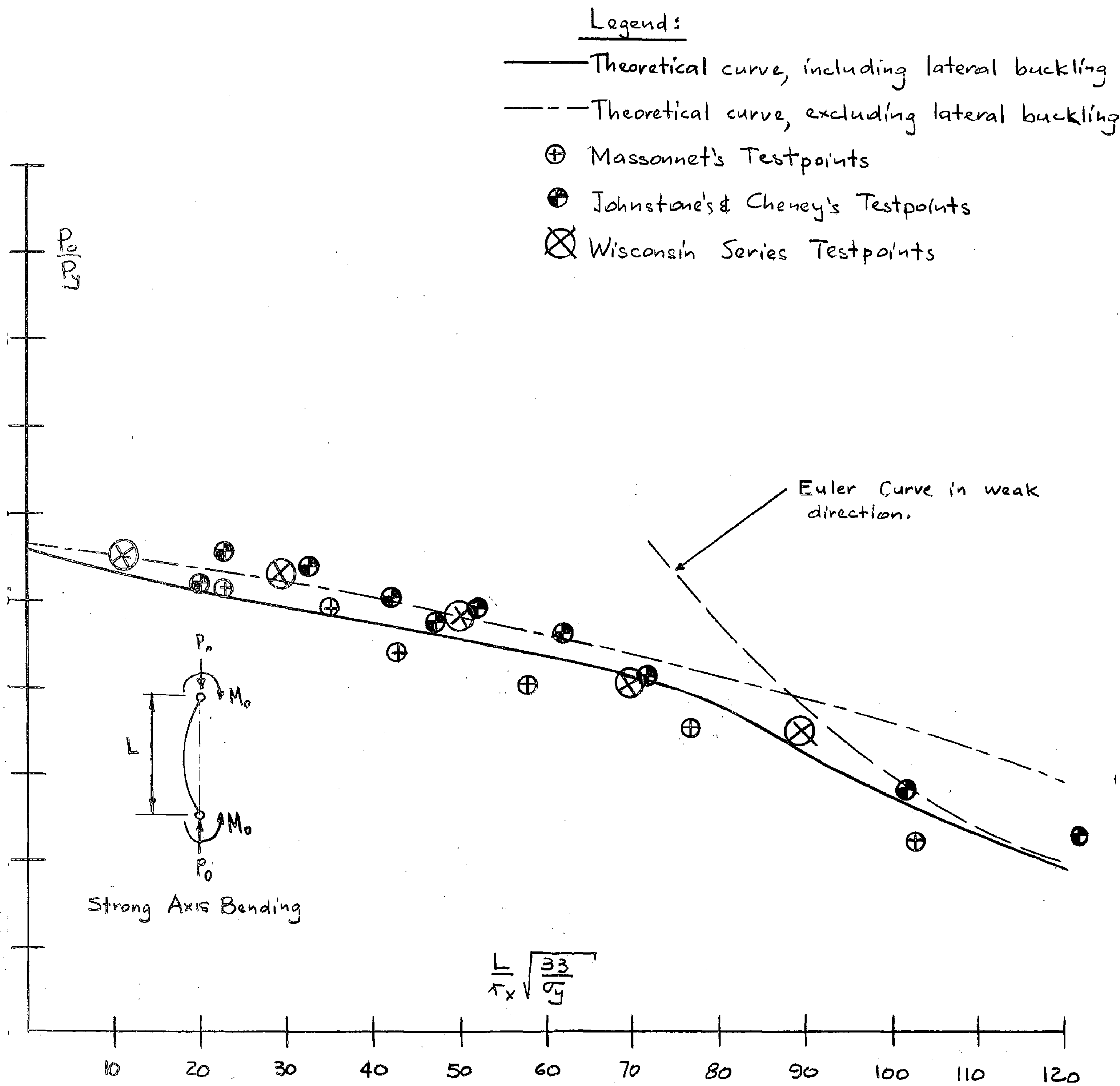
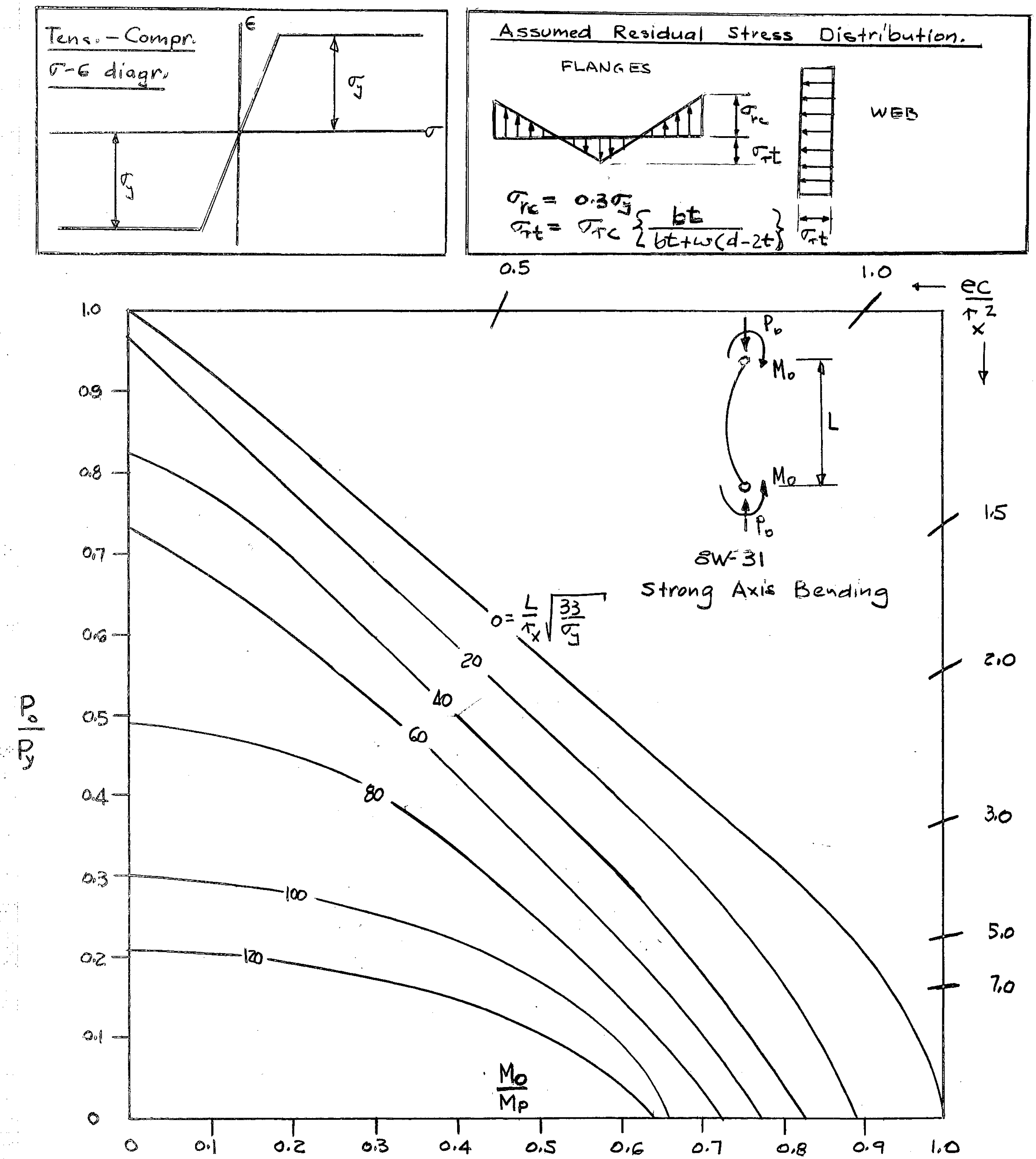


Fig. 1

Interaction Curves for 8WF31 Sections, Including the influence of lateral-torsional buckling and residual stress.



SIZE EFFECT
(205C-II)

The size effect program is now completed and the following is an abstract of the resulting progress report.

"BEHAVIOR OF WELDED CORNER CONNECTIONS"
by
John W. Fisher, George C. Driscoll, and F. W. Schutz

The results of a series of tests on straight corner connections for welded rigid portal frames are reported in this paper. These connections were proportioned using the concepts of plastic analysis. The purpose of this study was to determine the effect of member size on connection behavior. It was desirable to know if connections fabricated of larger size rolled sections would meet the "requirements" for connections which had been established in earlier studies using small specimens.

The knees were tested in a universal testing machine in a manner that simulated forces and moments tending to close the connection. Measurements were made to determine the rotation in the vicinity of the plastic hinge, the relative deflection between ends of the legs, the magnitude of the lateral support forces, and commencement of local web and flange buckling.

Theoretical expressions used in the calculation of the forces and deformations are described. From these expressions it is possible to correlate theory with the test results.

The factors leading to the design of the stiffening which includes the end plate, diagonal stiffener, and vertical stiffener are discussed. The welds were proportioned to carry plastic moments and forces. The limiting stress for butt welds was taken as the tensile or compressive resistance of the base metal of weld metal. For fillet welds the critical stress is the shear stress on the minimum throat area. The same overload factor of 33/20 which was applied to the normal stresses was also applied to the shear stress.

A comparison between welds proportioned in this manner and those proportioned by present day weld procedures is given. This showed the only weld which is larger by "plastic" design is the column web fillet weld to the beam flange.

The sequences used for welding the test specimens in order that induced stresses and distortion could be minimized are outlined.

The results of test carried out on connections using 14WF30, 24WF100, 30WF108 and 36WF230 rolled sections are presented. These welded connections were of the same general proportions as may be found in present day construction. The results of earlier experimental work on the same type connection fabricated from 8B13 sections is included for comparison.

The tests showed that changing the size of member has no ill effect on the performance of the type connection tested. It was found that connections of large size members are able to absorb a sufficient amount of rotation after reaching maximum moment provided adequate lateral support is furnished.

There were no weld cracks or failures, even after straining the knee much more than the amount necessary to merely reach maximum load. Hence, welds designed plastically with the same load factor for shear that is used for normal stresses have sufficient strength to allow the connection to reach the plastic hinge moment. Also, proper welding procedure planned to minimize distortion of weldments along with careful inspection of the welding will assure the development of plastic hinges without premature fracture of the welds in A-7 steel.

It can be concluded that the results of earlier theoretical and experimental work on small members can be applied to the larger rolled sections.

Equations and sample calculations for use in design are presented in an Appendix.

John W. Fisher

John W. Fisher

HAUNCHED CONNECTIONS
(Progress Report 205C-VI)

"A Plastic Analysis of Haunched Connections"
(A Theoretical Study) (205C.22)

A short summary of the analytical work on haunched connections is presented as follows:

If the influence of shear and axial force is temporarily neglected, the stress distribution within the haunch is assumed as indicated. (Fig 1)

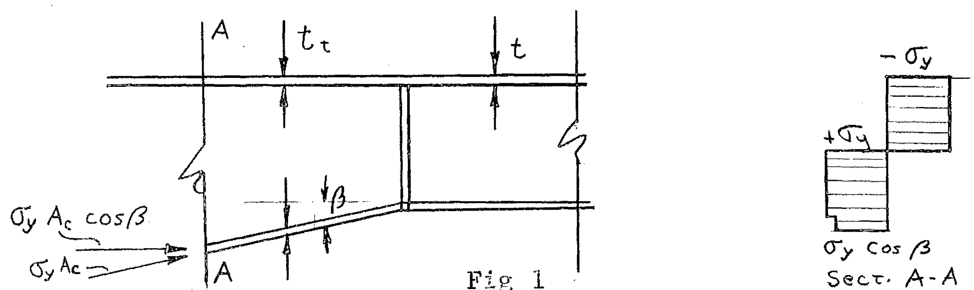


Fig 1

Therefore, $M_p = \sigma_y Z$

If the sloping flange is increased by $1/\cos\beta$ a condition of symmetry exists and the required thickness of the flanges can be obtained. Generally the geometry is fixed and the web thickness and flange width are generally maintained equal to those of the adjacent rolled section.

Obviously the effects of axial force and shear are less within the haunch than in the adjacent rolled section and can be neglected if neglected in the rolled section. The critical section within the haunch when a plastic hinge forms at the intersection of rolled section and haunch is then located. It is found that when the haunch angle, β, is approximate 12° the haunch is critical along its entire length. As β is increased the critical section occurs only at the haunch-rolled section intersection.

Since lateral buckling is inseparable from a plastic analysis, the critical buckling length of the compression flange was obtained, assumming simply supported end conditions and uniform strain hardening between points A & B. Since at the critical angle of β the hinges form at those points, this is a approximation of the behavior.

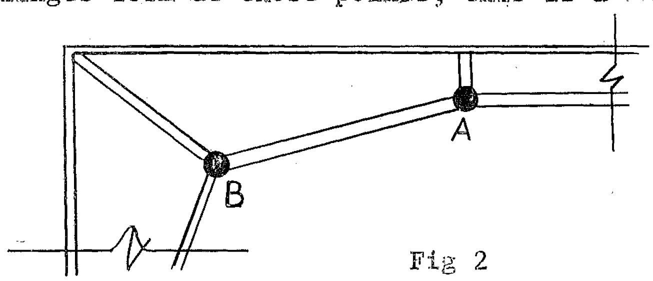


Fig 2

Hence we obtain

$$(L/r)_{cr} = \pi \sqrt{\frac{E_{sh}}{\sigma_y}} = 16.5$$

when compression flange is fabricated from plate material

$$L_{cr} = 16.5\, r_x = 16.5 \frac{b}{\sqrt{12}} = 4.8b$$

As β is increased the strain-hardened zone decreases. This allows an increase in the critical buckling length (Fig. 6 and 7) If it is undesirable to increase the haunch angle above the critical angle, an increase in L/r can be obtained by increasing the haunch flange thickness such that the strains are controlled. We obtain the required percent increase in haunch flange thickness as

$$\Delta t = 0.1 \, (L/_b - 4.8)$$

Adding increments for strain-hardening (10%) and for deviation in Z (5%) we obtain

$$t_c = (1 + \Delta t) \; (1.15) \; t/\cos\beta$$

where t is the thickness of the adjacent rolled section.

Reinforcement in the knee is designed to carry the unbalance in flange forces due to their sudden change in direction. This will prevent web buckling and shearing deformation from occurring.

A similar analysis can be made of curved haunched connections. Again neglecting temporarily the influence of axial force and shear, we obtain the following assumed stress distribution. (Fig 3)

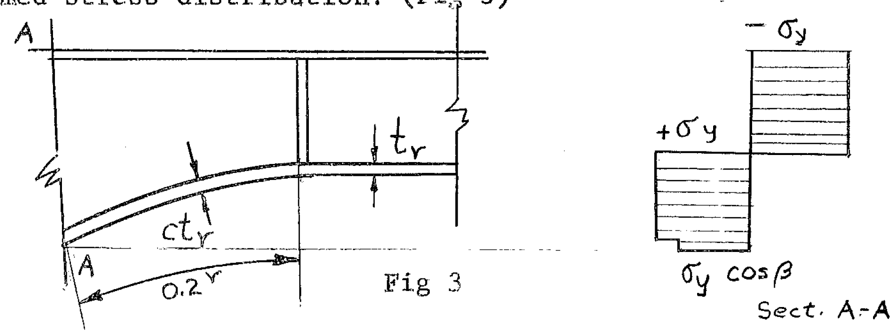


Fig 3

Therefore, $M_p = \sigma_y Z$

If at the most critical section we increase the haunch flange thickness by $1/\cos\beta$, from the symmetry and the assumed geometry we can obtain the required haunch flange thickness. The most critical section is obtained by equating the the applied moment to the plastic moment value. Maximizing this we obtain (Fig 4)

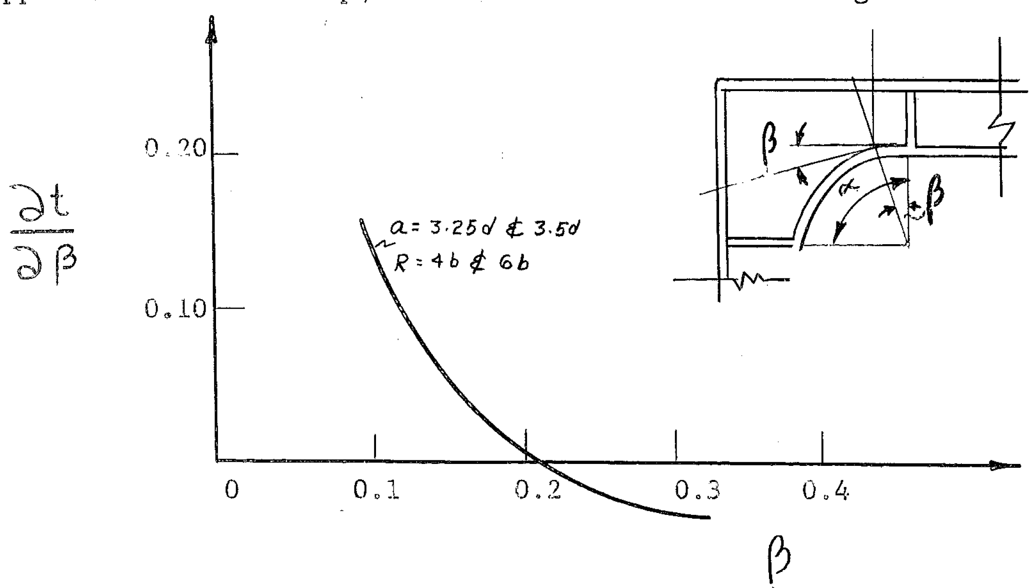


Fig 4

Hence the most critical section within the haunch occurs 0.2^r from the point of tangency when the plastic hinge value is also realized at the point of tangency. This also further substantiates our assumed stress distribution.

The axial force and shear can again be neglected since it has less effect on the haunch than on the adjacent rolled section.

Since the plastic hinges form at the points of tangency and at 0.2^r from it. A simply supported curved member is assumed, which is restrained from buckling in its weak direction and is forced to buckle laterally. By the method of Work we obtain the critical buckling length as

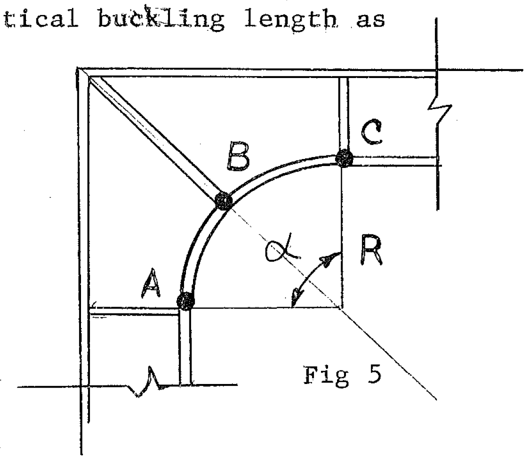


Fig 5

$(R\alpha)cr = 16.5\ r_x$

If lateral support is provided at points A, B, & C. The maximum allowance radius becomes

$$R_{cr} \cong 6b$$

In gable frames $\alpha < 90^{\circ}$ therefore a greater allowable radius is obtained.

As was done in straight haunches, the thickness of the flanges could be increased to control the strain in order that a greater allowable radius could be obtained in square knees. Therefore $t = 0.1\ (R/b\ .\pi/4 - 4.8)$, $\underline{R > 6b}$ or $t_c = (1 + \Delta t)\ (1.15)\ t$. Where t is the required flange thickness 0.2 from the tangent point as obtained by simple plastic analysis.

Since the stress in the curved flange produces a radial component which causes cross-bending it was necessary to consider its effect on the plastic moment and the geometry of the section. If $b^2/2Rt \leq 1$ then the selected flange plate will be adequate. Radial stiffeners can be designed conservatively by the same procedure outlined for haunched connections assuming further that they must resist a plastic flange force which passes through the mid point of the haunch and the points of tangency.

All of the expressions presented here are theoretical values and need to be confirmed by tests. Previous tests of specimens designed elastically have to a limited degree confirmed several of the analytical results. The location of the most critical section in curved knees is substantial in tests reported in P. R. #4.

Premature lateral buckling has been observed in several connections tested previously.

John W. Fisher

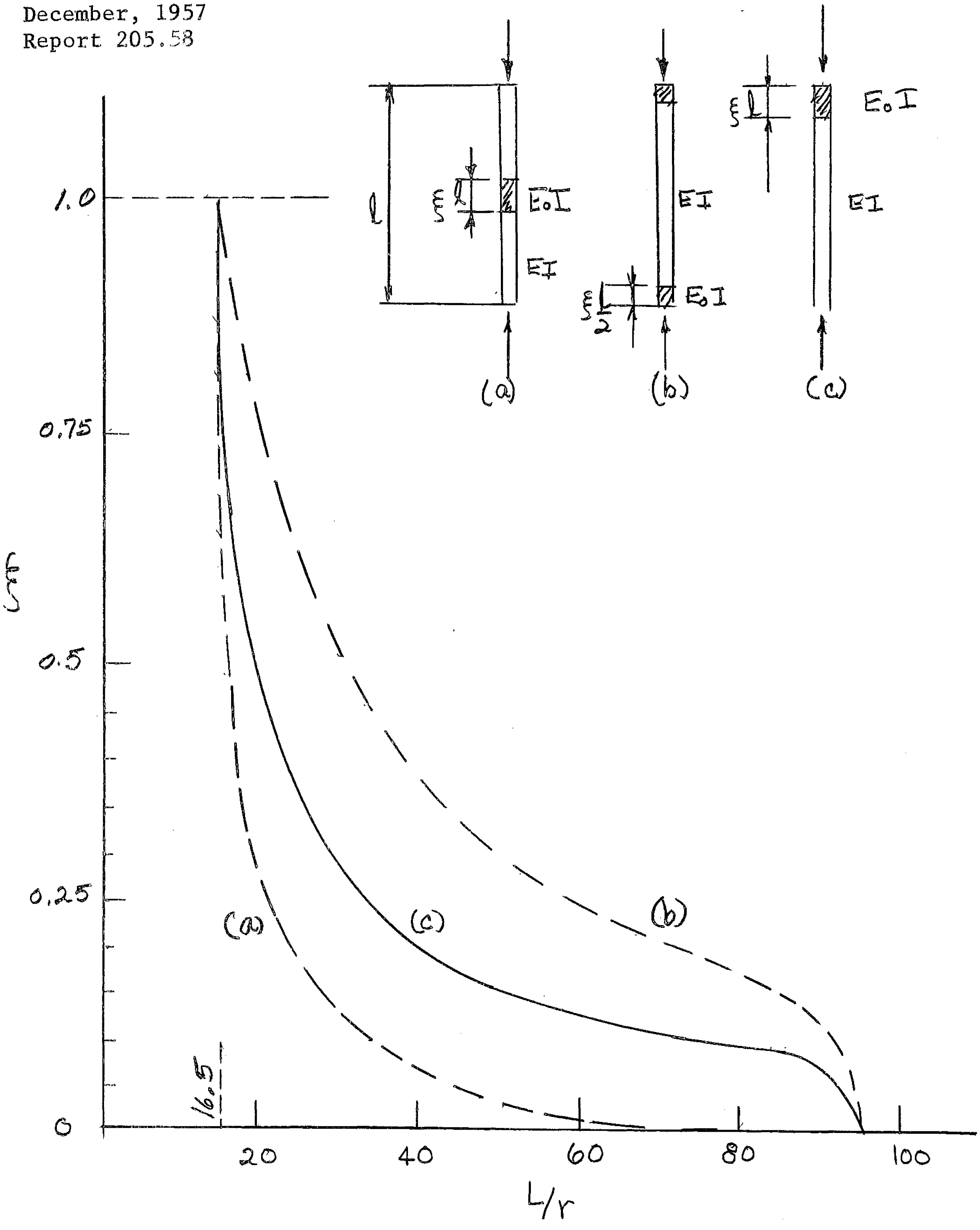

1.0
0.75
0.5
0.25
0
$\frac{\xi}{l}$
16.5
20
40
60
80
100
L/r
(a)
(b)
(c)
l
ξ
E_0I
EI
$\frac{\xi}{2}$

FIGURE 6

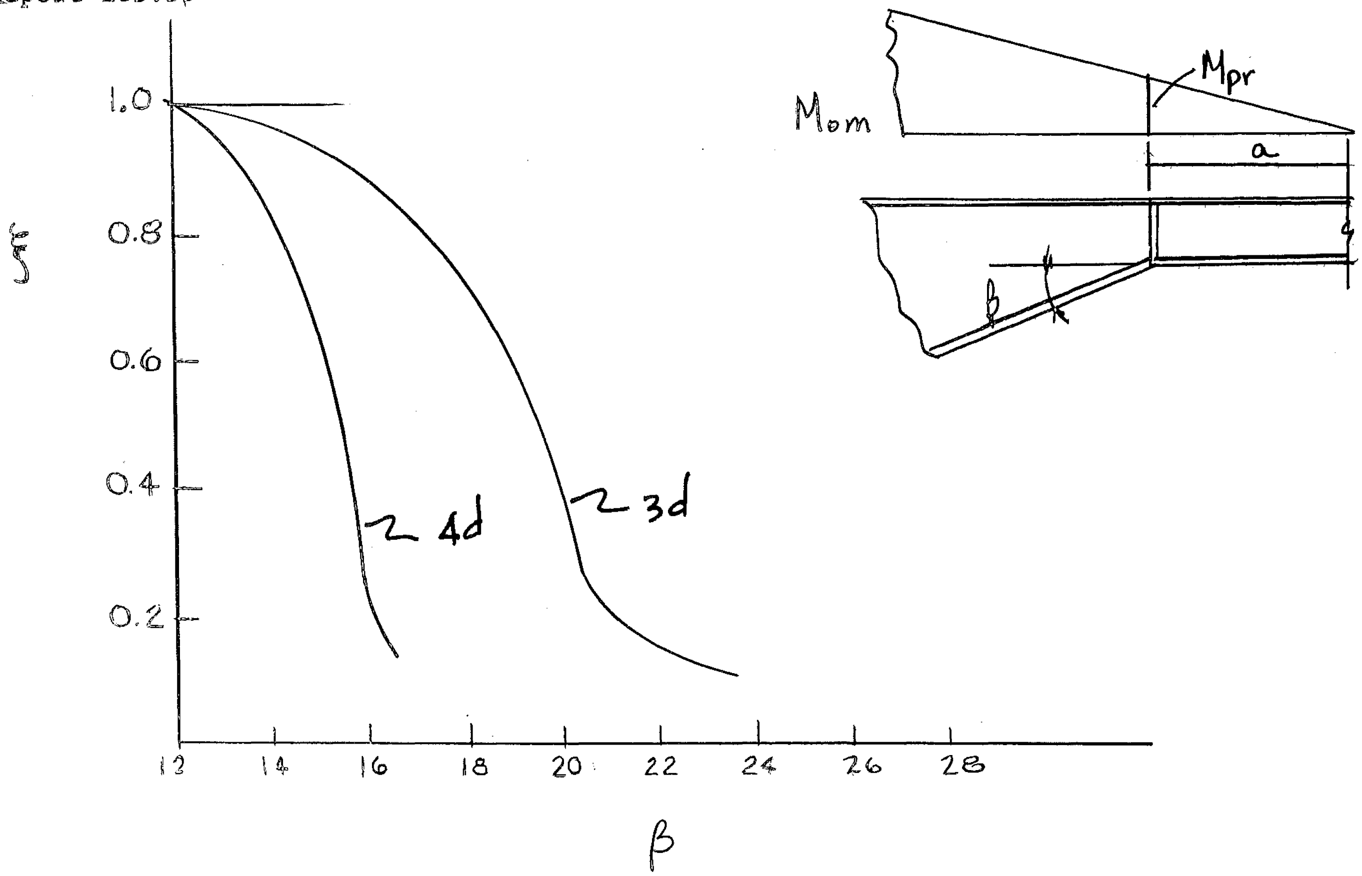


FIGURE 7

WELDED CONTINUOUS FRAMES AND THEIR COMPONENTS

Proposal for Further Work on
Haunched Connections

by: John W. Fisher

Since there are several reasons for the use of haunched welded connections in plastic design, there is a need for a simple yet accurate method of proportioning such haunches. Due to the poor rotation capacity that was generally observed in a haunched member, it was thought that a design procedure should be developed which would assure elastic behavior of the haunched connection even when the structure as a whole had reached the ultimate load condition. However, it was noted that the primary cause of failure for most haunched connections was lateral buckling of the compression flange. Observation of connections designed elastically showed the flanges were yielded. Therefore, an accurate analysis of the critical buckling length was not possible. Hence, it was thought that a method of analysis could be developed which would include the effect of lateral buckling, using the concepts of plastic analysis.

The use of haunched connections may be necessary in order that rolled sections may be used for the prismatic beam sections of the frame. This condition may easily be encountered for long span portal frames. Baker has shown that causing hinges to form in the prismatic beam section adjacent to the haunch at ultimate load results in a savings of as much as 10% in weight of the main frame. This 10% savings can be achieved when the haunch has a length equal to 1/20 of the span when compared to the same frame having no haunch.

There are presently available several methods of elastic analysis. However, the majority of these have been so complex that their use has not been widespread. For the range of practical haunch proportions, the plastic approach now being developed presents a simple and usable procedure which fits into the philosophy of plastic design. This analysis is much simpler than any known elastic solution, and greatly simplifies the design of haunched connections.

In order to substantiate the theoretical analysis a modest test program covering the points outlined is necessary, and a suggested test program for consideration by the committee is indicated in Fig. 1 as a check on the theoretical solutions.

Outline of Proposed Work

Analytical:

(1) Complete the theoretical plastic analysis now being made for haunched connections.
(2) Develop a design procedure.
(3) Give several illustrative problems showing the analysis and design for haunched connections from plastically designed frames.

Experimental:

(1) Design and fabricate a straight haunched connection according to the rutes developed in the analytical phase with the angle β at the critical angle found from the analytical studies, such that compressive flange is uniformly strain-hardened and $(L/r_x)_{cr} = 16.5$ (Fig. 1-A).

(2) Design and fabricate a straight haunched connection with $\beta > \beta_{cr}$ according to design rules developed, such that $(L/r)_{cr} > 16.5$ (Fig. 1-B).

(3) Design and fabricate a straight haunched connection with the angle β critical and $(L/r) > (L/r)_{cr}$ between haunch and rolled section, but using stiffeners tied to purlins as intermediate points of support. (Fig. 1-C).

(4) Design and fabricate a straight haunched connection with angle β critical and $(L/r) > (L/r) = 16.5$ but with the flanges increased in thickness by the rules developed in analytical phase. (Fig. 1-D).

(5) Design and fabricate a straight haunched connection with angle β critical but with a channel type compression flange such that r_{xx} is increased. (Fig. 1-E).

(6) Design and fabricate a curved haunched connection for a square corner frame with the radius $> (R)_{cr}$ but with the thickness of the flanges increased by the rules developed in the analytical phase. (Fig. 1-F).

(7) Design and fabricate a curved knee with radius = 6b and thickness of haunch flange such that a hinge forms 0.2 r in from tangent point as well as in rolled section. (Fig. 1-G).

(8) Test the connections listed in items 1 thru 7. These tests should demonstrate the ability to proportion haunches by the method developed in the theoretical analysis, and enable us to check the assumptions and rules developed in the theoretical phase, i.e., the

critical buckling length of the compression flange and different angles β , location of most critical section, stiffener design, and the adequacy of bracing to prevent premature lateral buckling.

The above information can be obtained from connections fabricated from rolled sections indicated in Table 1. The approximate geometrical proportions are indicated in Fig. 1 and Table 2.

The lateral support system used in previous tests has allowed rotation to occur about a horizontal axis. It is thought that the proposed system will approach the actual lateral support that is provided in a portal frame knee. Fig. 2 indicates several details showing the proposed system.

Several different test set-ups are being considered. One is to place the connections in a test frame such that the connection is lying on its side. The outside corner of the knee could be rigidly fixed by using the detail shown in Fig. 2A or 2B. The extremities of the legs could then be pulled together with jacks. The resulting deflection would not effect the lateral support system. Another test set-up would be to place the knee in the position indicated in Fig. 3. The outside corner would be rigidly fixed at the base.

The instrumentation, gage locations, and measurements to be taken would be similar to those described in previous connection tests.

When final design of connections and test set-up is made, copies would be sent to the committee for their approval.

Cost Estimate:

No additional funds are required to perform these tests.

TABLE 1

Member	b/t	d/w	f	d/b	w	t
10B17	12.19	42.2	1.15	2.52	0.240	0.329
10B19	10.20	41.0	1.15	2.55	0.250	0.394
12B22	9.50	47.3	1.16	3.06	0.260	0.424
12WF27	16.25	49.8	1.11	1.84	0.240	0.400
10WF25	13.40	40.0	1.12	1.75	0.252	0.430

TABLE 2
HAUNCH CONNECTION TESTS

Fig. 1	Test No.	Flange Thick-ness "t"	Length of Compres-sion Flange	Knee Type	Lateral Support	β	Notes
A	44	Eq. 1	5b	Tapered	Yes	12°	Critical angle, critical length
B	45	t_r	5b	Tapered	Yes	12°	Critical length obtained from Fig. 7
C	46	Eq. 1	5b	Tapered	Yes	12°	Intermediate point of support used so that critical length is not exceeded
D	47	Eq. 2	5b	Tapered	Yes	12°	Haunch flange thickness increased to obtain greater critical buckling length
E	48	--	$L = 16.5r_x$	Tapered	Yes	12°	Critical angle, critical length with channel used for compression flange
F	49	Eq. 3	(R) 4.8b	Curved	Yes	-	Flange thickness increased to obtain greater critical buckling length
G	50	Eq. 1	(R)=4.8b	Curved	Yes	-	Critical length, critical section

t obtained from

Eq. 1 , $t = d_x/2 - \sqrt{d_x^2/4 \cdot (b/b-w) - Z/b-w}$ $(t \geqq t_r)$

Eq. 2 , $t = (1 + \Delta t)(1.15)t$ where $\Delta t = 0.1(L/b - 4.8)$

Eq. 3 , $t = (1 + \Delta t)(1.15)t$ where $\Delta t = 0.1(R\pi/4b - 4.8)$

$t = t_r$ t_r = flange thickness of rolled section

NOTE: Rolled section to be selected from Table 1

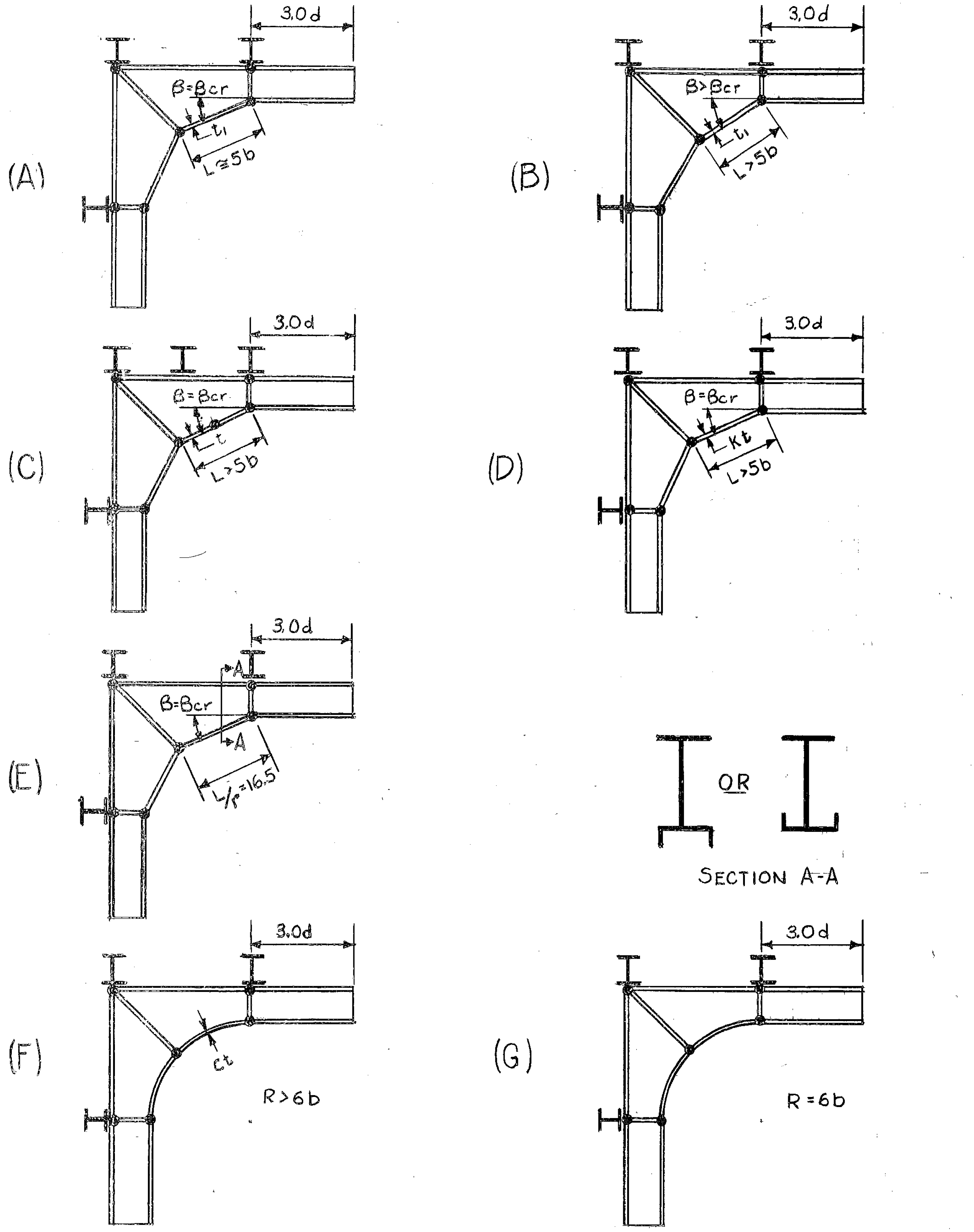

(A)
3.0d
β=βcr
t1
L≅5b
(B)
3.0d
β>βcr
t1
L>5b
(C)
3.0d
β=βcr
t
L>5b
(D)
3.0d
β=βcr
Kt
L>5b
(E)
3.0d
β=βcr
A
A
L/r=16.5
OR
SECTION A-A
(F)
3.0d
ct
R>6b
(G)
3.0d
R=6b

FIGURE 1

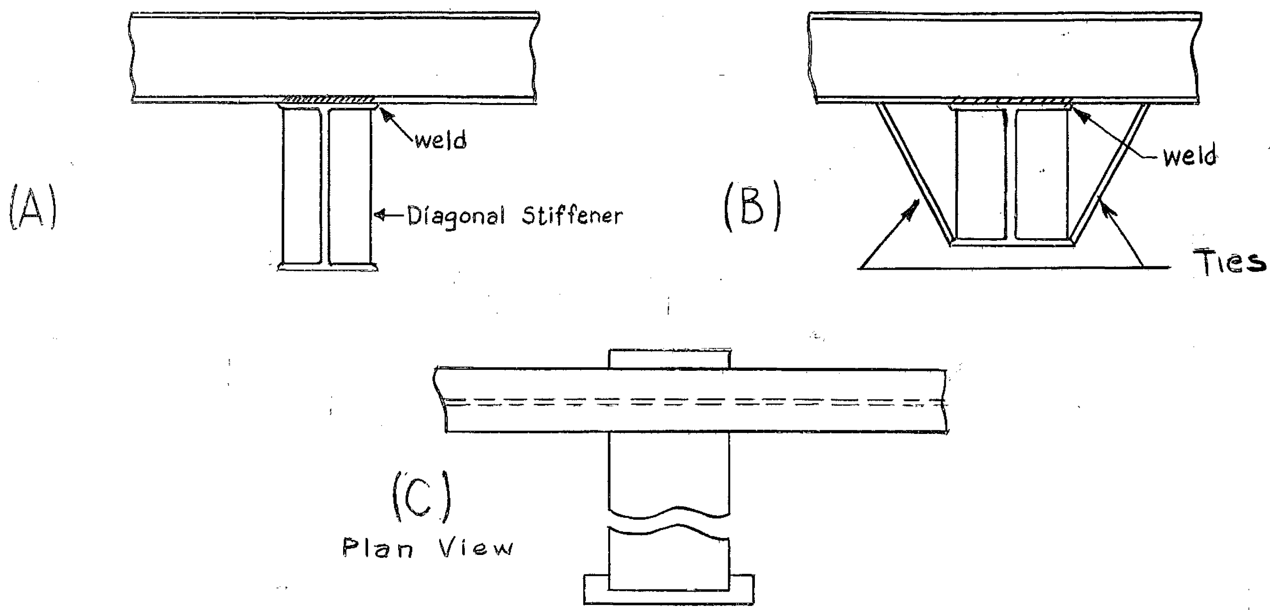


FIGURE 2

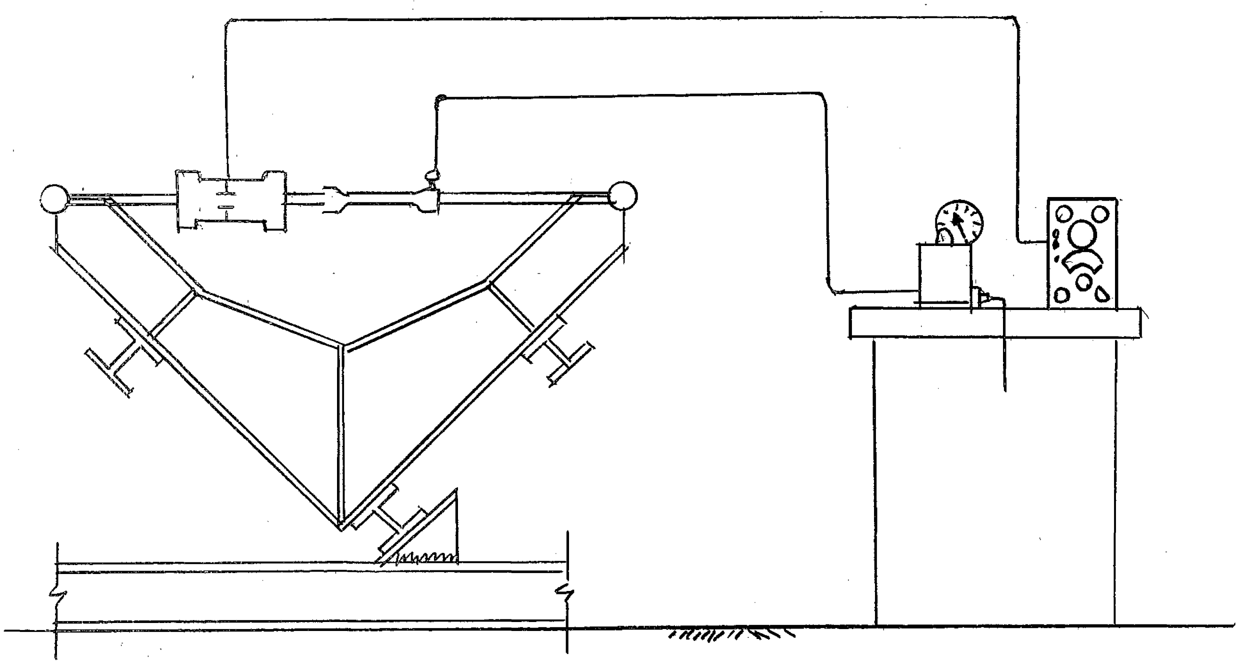


FIGURE 3

248-I

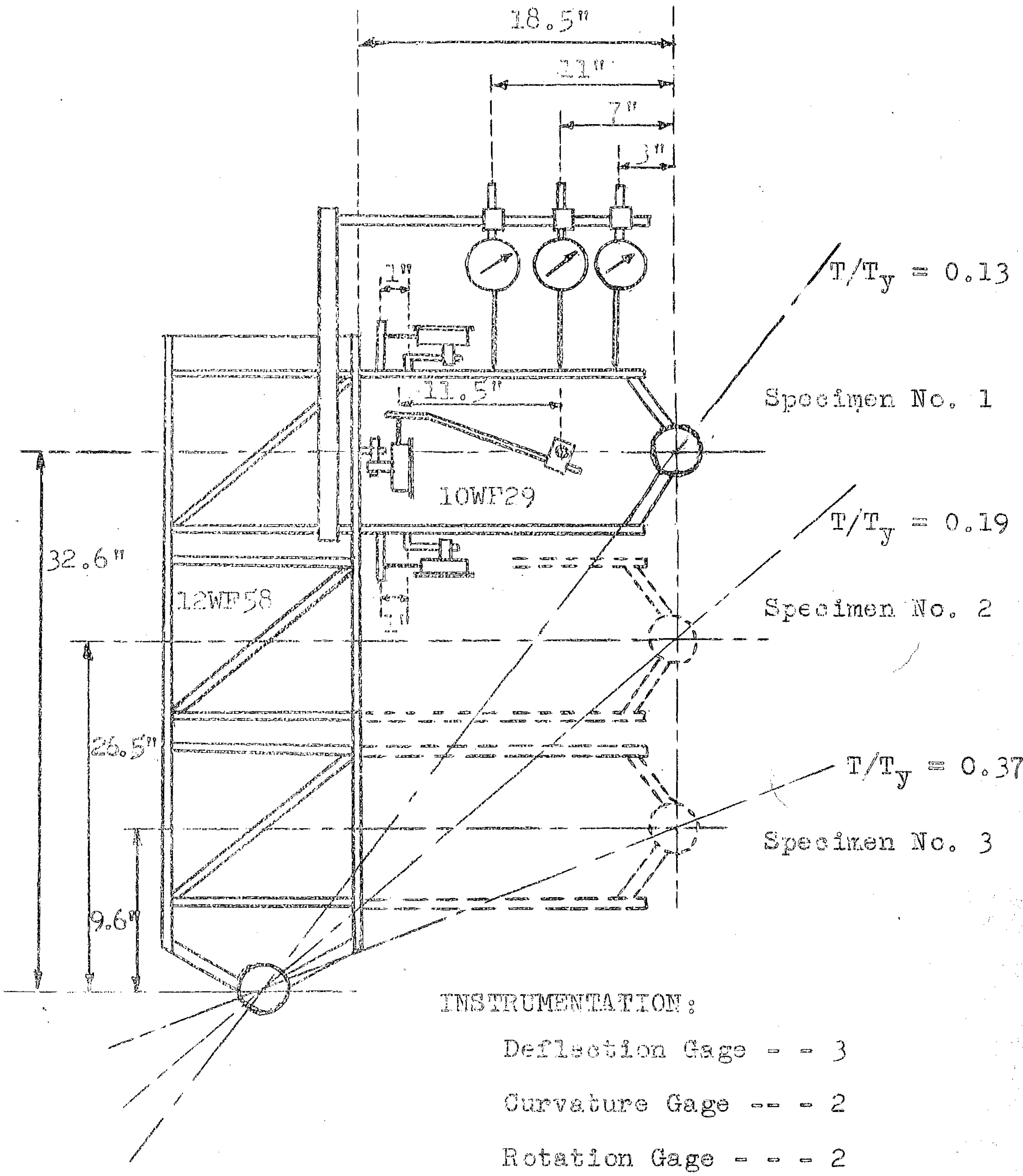
BUILT-UP MEMBERS IN PLASTIC DESIGN

The enclosed sheets present a short summary of the test results conducted to substantiate the theoretical analysis on the reduction of the fully plastic moment due to shear and axial forces.

On page 2 the test specimens and instrumentation are illustrated as shown in Fig. 1. The ultimate load Pu which is related to the modified fully plastic moment M_{pm} is determined from load-deflection curve, moment-curvature curve and load-hinge rotation curve for each of three specimens. Fig. 2 shows the test result of load-deflection curve at 7" from end. The correlation between theoretical and experimental result in Pu is given as follows:

T/T_y	Theory		Test				
	(A) kips	(B) kips	Defl.(1) kips	Defl.(2) kips	Defl.(3) kips	M-Ø kips	Hinge Rotation kips
0.13	73	70	70	70	70	70	70
0.19	82	82	82	82	82	85	82
0.37	79	124	126	126	126	126	120

The relation between prediction and test results on load-deflection curve at 3" from end is shown in Fig. 3. The analysis of deflection in strain-hardening range gives fair correlation with test result. The ratio of the reduced fully plastic moment M_{pm} to the fully plastic moment M_p is plotted on Fig. 4 and the approach B shows good prediction of the test result. Fig. 5 shows one example of this analysis on the reduced fully plastic moment M_{pm} to the design of a beam with cutout.



INSTRUMENTATION:

Deflection Gage - - 3

Curvature Gage -- - 2

Rotation Gage - - - 2

Fig. 1 - TEST SPECIMENS AND INSTRUMENTATION

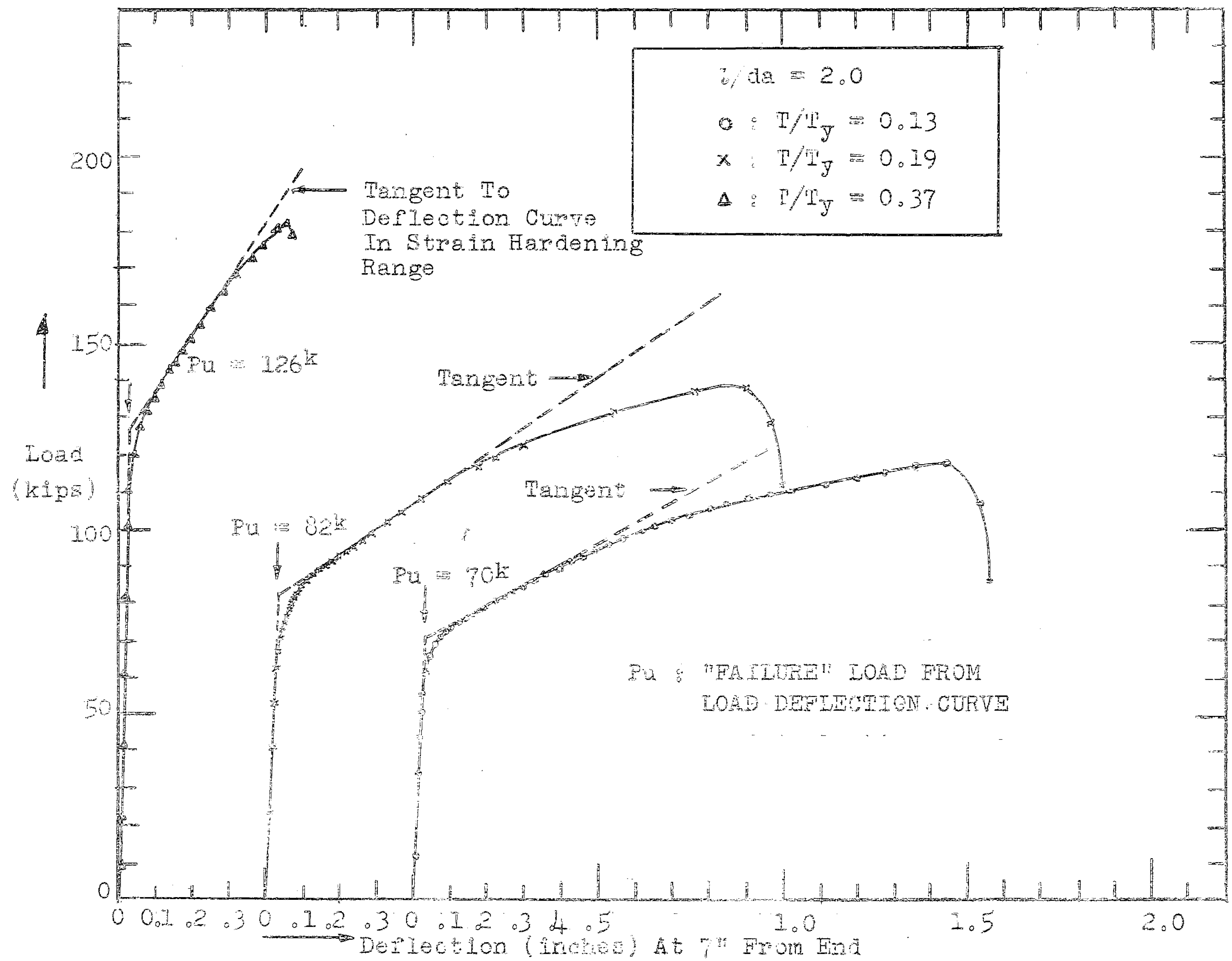


Fig. 2 - LOAD-DEFLECTION CURVES (II)

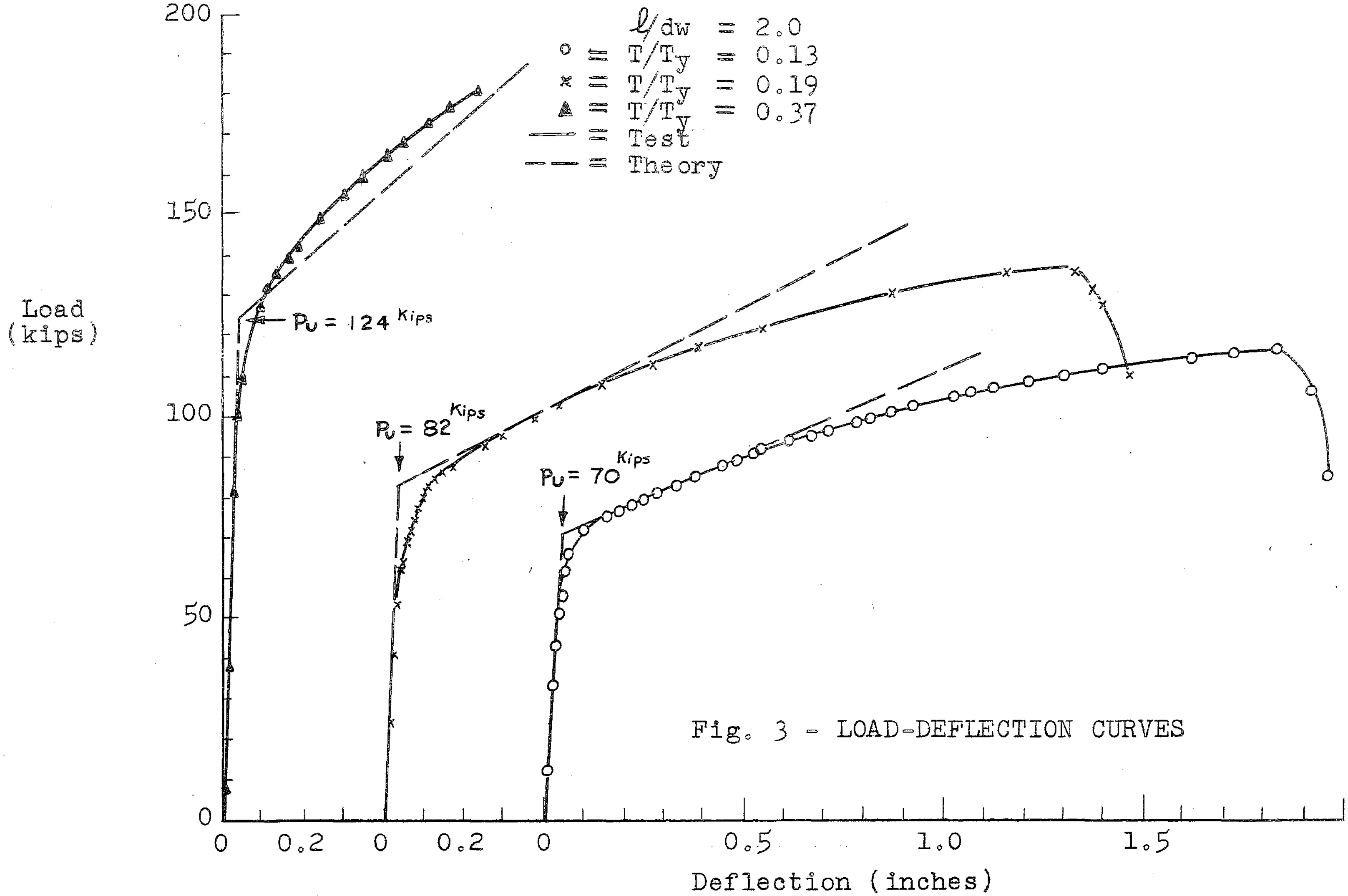


Fig. 3 - LOAD-DEFLECTION CURVES

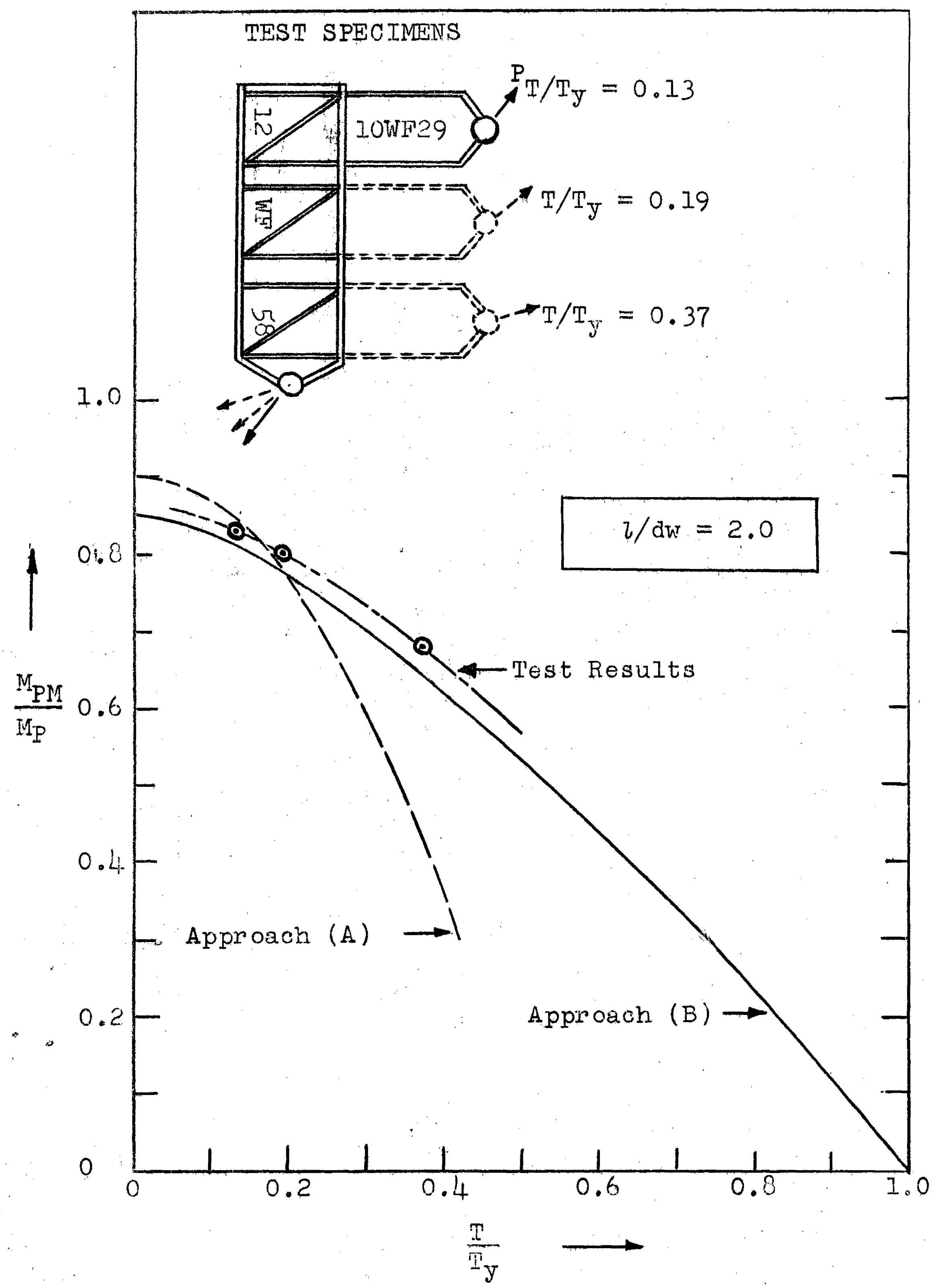


Fig. 4 - REDUCTION OF PLASTIC MOMENT
DUE TO SHEAR AND AXIAL FORCE

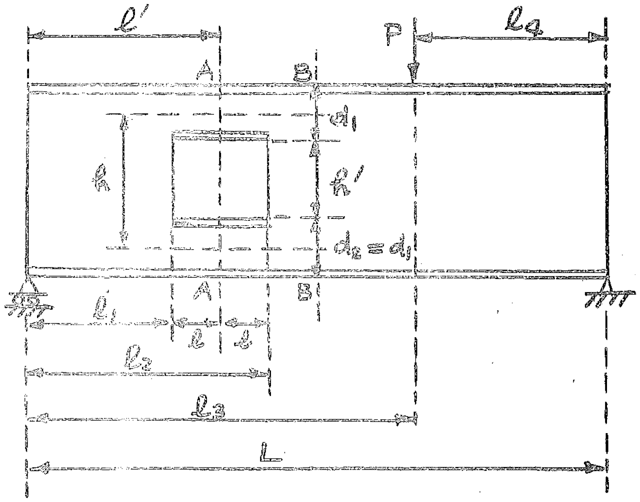


Fig. 5(a) - BEAM WITH CUTOUT

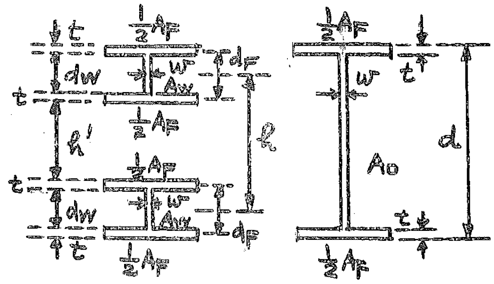


Fig. 5(d) - CROSS SECTION AT A-A AND B-B

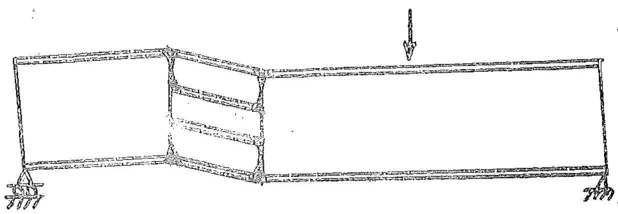


Fig. 5(b) - MECHANISM (I)

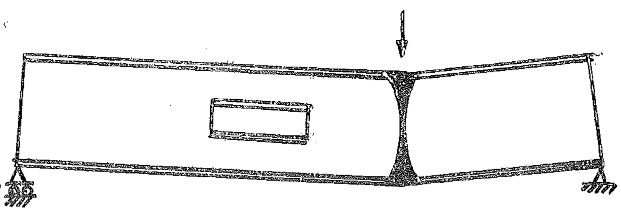


Fig. 5(c) MECHANISM (II)

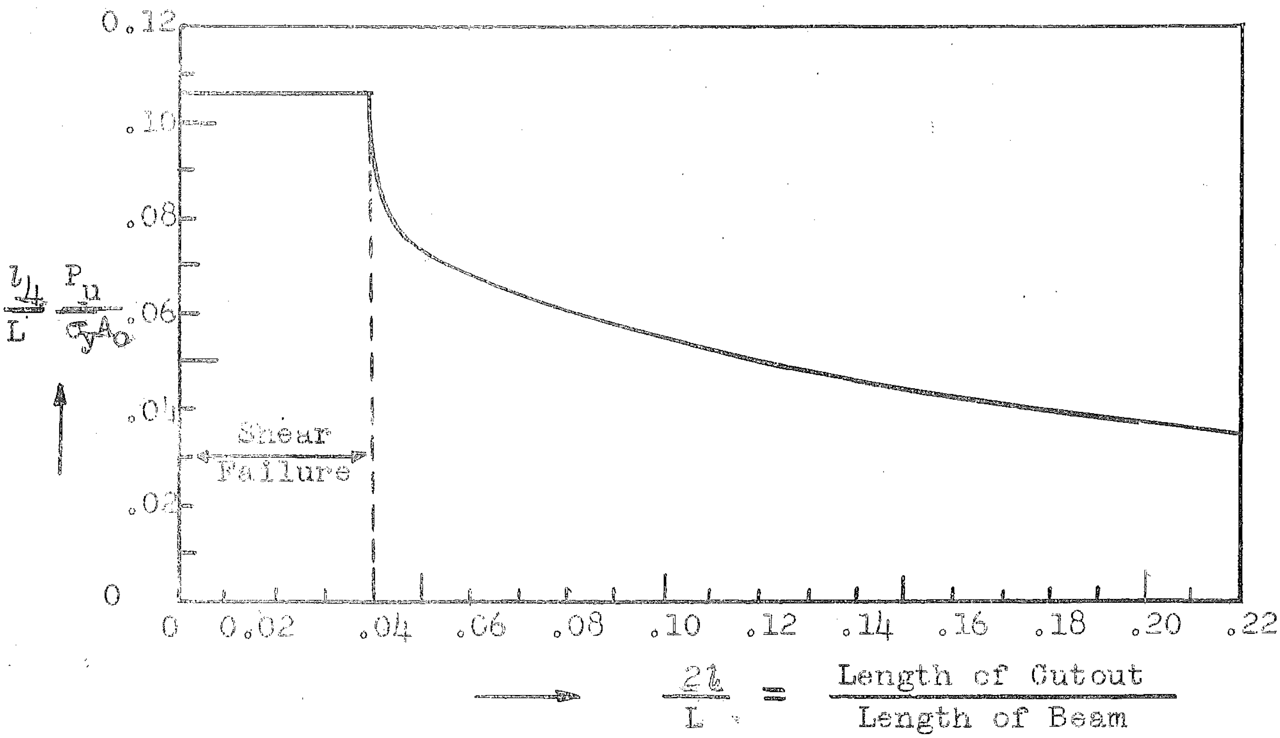
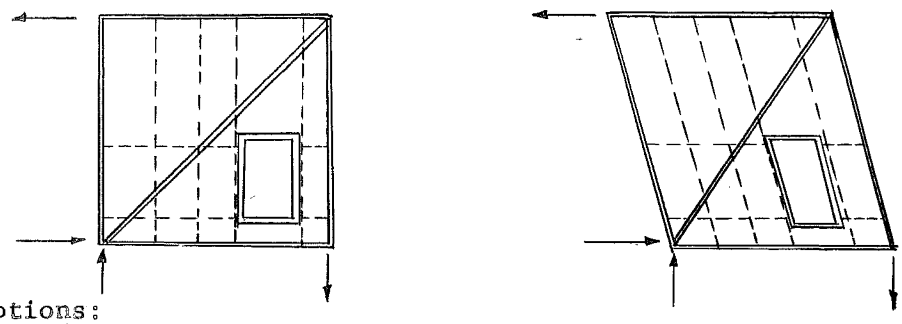


Fig. 5(e) - ULTIMATE LOAD OF BUILT-UP BEAM WITH CUTOUT

CORNER CONNECTIONS WITH CUTOUT
248-II

Analysis of corner connection with cutout (upper bound)



Assumptions:

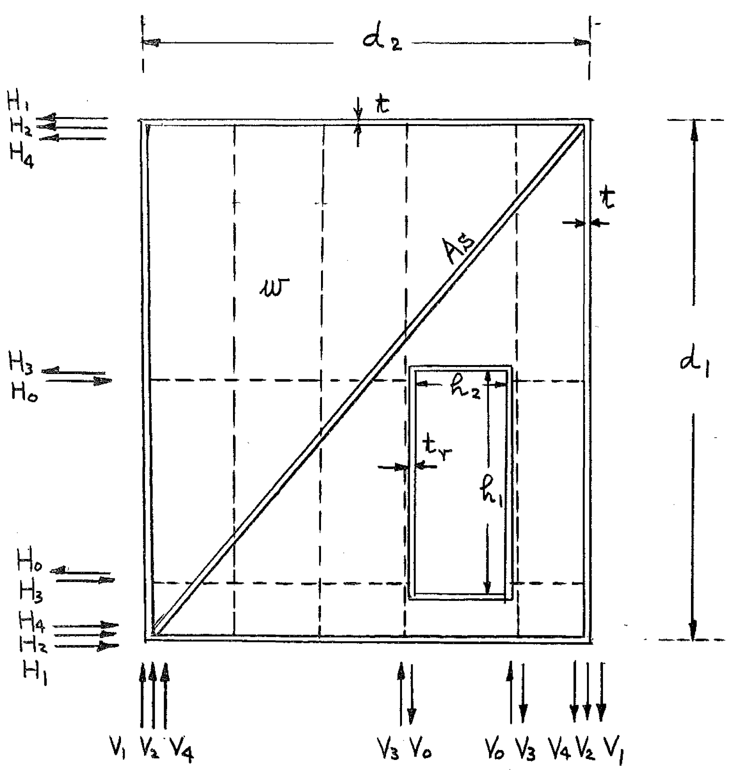
1. Shear failure of web plate. (no buckling)

2. Panel mechanism of frame work outside of web plate. (with certain amount of effective widths.)

3. Panel mechanism of reinforcement around cutout.

4. Diagonal stiffener fails in compression.

Decomposition of load carrying capacity of panel:



$$H_1 = \tau_{yh}\, wd_2$$

$$H_2 = \tau_{ys}\, \frac{2bt^2}{d_1}$$

$$H_3 = \tau_{ys}\, \frac{2b_y t_y^2}{h_1}$$

$$H_4 = \tau_{yh}\, \frac{2\,A_s}{1+(d_1/d_2)^2}$$

$$H_0 = \tau_{yh}\, wh_2$$

$$V_1 = \tau_{yh}\, wd_1$$

$$V_2 = \tau_{ys}\, \frac{2bt^2}{d_2}$$

$$V_3 = \tau_{ys}\, \frac{2brtr^2}{h_2}$$

$$V_4 = \tau_{yh}\, \frac{a\,As}{1+(d_2/d_1)^2}$$

$$V_0 = \tau_{yh}\, wh_1$$

Design condition,

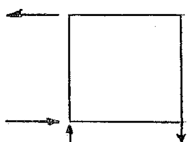
M haunch $= M_p$

Effective width of deck plate = 30 t

$\tau_{yh} = 25$ ksi

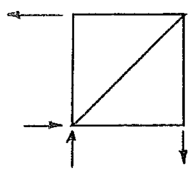
$\tau_{ys} = 30$ ksi

1) Required web thickness of corner connection with neither diagonal stiffener nor cutout.



$w_r = 0.415''$

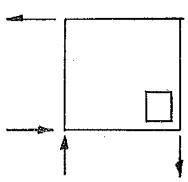
2) Required diagonal stiffener. (with no cutout)



$A_{so} = 3.2''^2$

(where $w = 0.375'' = 3/8''$)

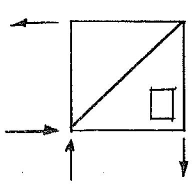
3) Required web thickness without diagonal stiffener.



(with cutout)

$w = 0.463''$ (Designed plate thickness $= 3/8''$)
$0.375''$

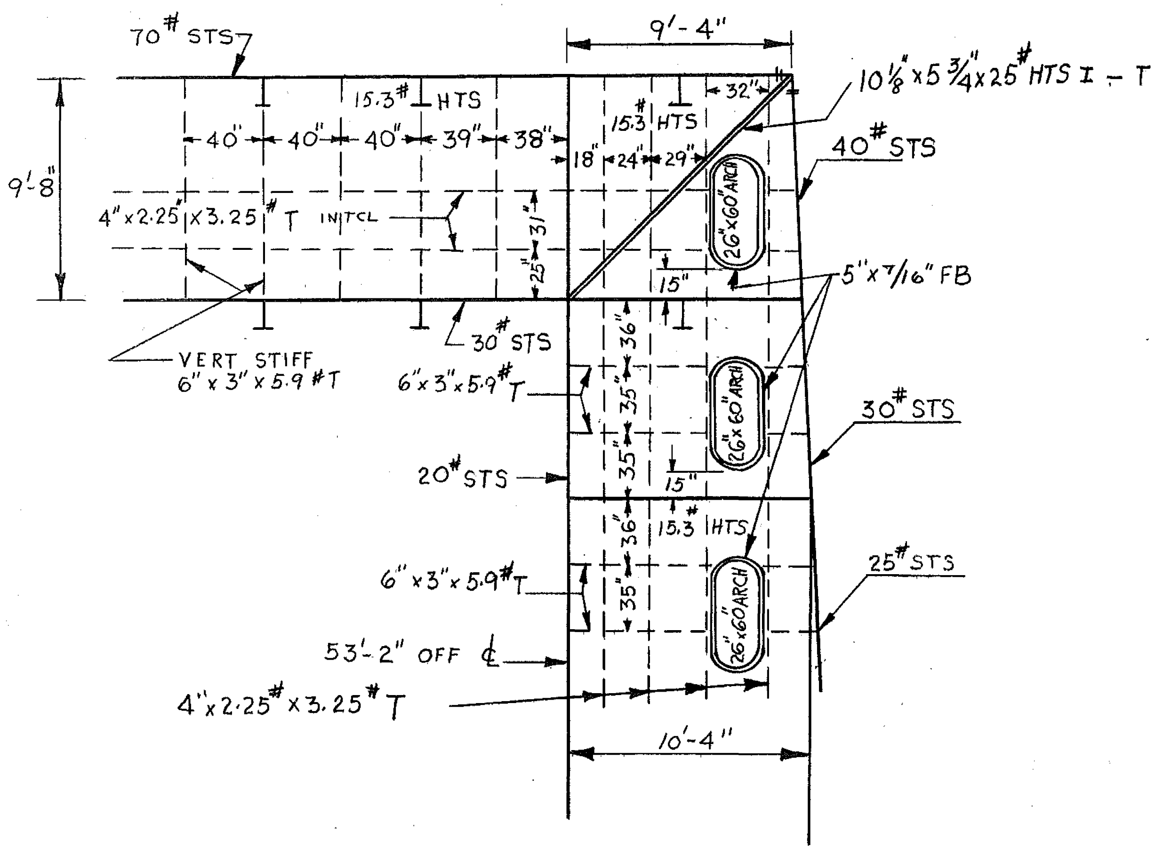
4) Required diagonal stiffener (with cutout)



$A_s = 6.22''^2$

(where $w = 0.375'' = 3/8''$)

Designed stiffener $= 7.35''^2$ (OK)



DETAILS OF KNEE OF BENT

BUCKLING AND EFFECTIVE WIDTH OF DECK PLATING
248-V
- Proposed Future Work -

Presently used specifications for ship structures, for example, Rules of American Bureau of Shipping, Rules of Lloyds' Register of Shipping, Great Britain, and Rules of Japanese Marine Corporation, Japan, are based partly on conventional elastic design and mostly on experiences obtained by trial and error. The theoretical studies on ship structure have been carried out especially on the buckling strength of a stiffened plate within the elastic range. Consequently the specifications for such a stiffened plate like deck or bottom construction of the hull can safely be applied for structures in which the design is based upon theoretical first yield as the limiting condition. However, the wide application of welding to the hull caused for instance, the corrugation of the bottom shell plating, a type of failure which has become extremely common since the war in ships having transversely framed bottom of welded construction. This type of failure has been noticed in almost every country of the world and requires some change in their specifications on welded panel structures. This is a problem which is related to the plastic instability of a stiffened plate together with the deformation and the residual stress due to welding.

There are many experiments carried out in England and Japan, however, it is not yet solved completely, since the theoretical analysis on plastic buckling of stiffened plate have never been solved yet.

This new problem on ship structure insists that plastic design based upon ultimate strength should be applied to ship structures and presently used specifications must be reinvestigated on the basis of plastic design. In general, plastic design imposes more severe requirements on the sections with regard to plastic instability. The proposed investigation will deal, therefore, with the general instability of stiffened plates. The approach is planned as follows:

1) Required bending rigidity of stiffeners with effective width of plate in plastic range.

2) Effect of torsional resistance of flanged stiffeners which consists of St. Venant's torsion and warping torsion.

3) Buckling strength of stiffened plates having transverse stiffeners and longitudinal stiffeners.

4) Optimum design of compressed panel having flanged integral stiffeners.

Lateral Buckling (205E-V)
(Progress Report)

The four tests on 10WF21 sections proposed and adopted at the last annual meeting (Dec. 17, 1956) have been completed. A report containing a description of the tests, the results and analytical considerations is in preparation.

In the derivation of a simplified design procedure to determine the maximum permissible unbraced length of WF beams, considerable difficulties have been encountered. Presently, a second draft of a procedure is under study to determine its reliability.

The future work will be concentrated on completing the above mentioned report and finalizing a design procedure for directing the lateral bracing of WF beams.

Tadao Kusuda

Tadao Kusuda

Bruno Thurlimann

Bruno Thurlimann