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STEEL BEAMS, CONNECTIONS, COLUMNS AND FRAMES

A Preliminary Report and Review of Information on Resistance to Deflection and Collapse

By

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FOREWORD

This preliminary report on "Steel Beams, Connections, Columns, and Frames" was written as part of the work carried out for Sandia Corporation, Albuquerque, New Mexico, under their Purchase Order No. W-592.

Here and there in this preliminary report the reader will find references to final reports that are anticipated. As the present contract terminates (on March 31, 1952), the only final report that is in progress is the one on "Columns" that is being written separately at Lehigh University under the direction of Mr. L. S. Beedle. Aside from this, no other final reports have as yet been authorized. Since Sandia Corporation will no longer sponsor this sort of structural investigation, authorization for the final reports that are referred to herein is subject to the approval of the Armed Forces Special Weapons Project, under whose indirect jurisdiction any further work will be completed.

The authors will be grateful to have typographical errors and other errors called to their attention.

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CHAPTER I

INTRODUCTION

The project for which this is a preliminary report has as its ultimate purpose the evaluation and demonstration of existing means of determining the resistance to collapse of steel tier buildings, bridges, and industrial buildings, with special emphasis on the latter and also on behavior under atomic blast loads. The resistance of a structure is gauged by its complete load-deflection history through both the elastic and non-linear ranges of behavior beyond the ultimate load capacity and up to the point of complete collapse. Collapse will be assumed to be complete when permanent distortion of the main frame has occurred to such a degree that repair for even partial use of the structure is not practicable and complete rebuilding is required to restore the utility of the bridge or building.

Fundamental to all phases of the problem of collapse of a steel structure is the study of the yielding and fracture characteristics of structural steel as a material, which will be reviewed in Chapter II of this report. Bridges and buildings are built up as an aggregation of structural members: the tension member, the beam, and the column. These members, in turn, are joined by connections to form either the truss or frame type of construction that meets the requirements of size, load, etc., for application in a particular building or bridge. Chapters II to X, inclusive, cover these basic steps leading up to the final chapters, which review actual tests and experience with destructive forces, intentional or otherwise, applied to full-size tier buildings, industrial buildings, bridges, and tank structures. Since the project has been carried forward on an unclassified basis, no coverage is included concerning the recent tests of structures under actual or simulated atomic blast. Results of these tests are presumably available in classified reports and reviews.

The analysis of the collapse behavior under dynamic loads requires, of course, a knowledge of the load resistance of structures under dynamic conditions. Nevertheless, most of the available experience regarding the resistance of structures to load is in the form of static test information, and it has been considered desirable to gather together all the available information, whether static or dynamic, in which there exist tests or theories that pertain to the ultimate collapse strength of a structure or one of its component parts.

The project was started April 1, 1951, under authorization by Sandia Corporation, and the principal accomplishment of the first year will be the completion of this "preliminary report." It had been the original intention of the project personnel to make a very thorough literature search, digest, evaluation, and application of each of the subdivisions in turn. At the request of Sandia Corporation, original plans were modified early in the work, and a "preliminary report" covering the whole project was started. In the meantime, Lehigh University, under a separate contract, has started on one phase of the final monograph, to be carried out in collaboration with further work at the University of Michigan.

Although this preliminary report has too broad a coverage to represent a thorough digest of all information available, it has had the value of coordinating all of the thinking concerning the complete project at the outset, and, in addition presents an integrated plan of research covering all phases of the work. These separate projects are proposed as a guide to work that might be carried out experimentally at various universities and laboratories.

This preliminary report is descriptive and nontechnical in nature. It is thus hoped that it will be of use not only in the planning of research projects but as an aid to the understanding of structural problems by those persons interested in its objectives but who may not have had theoretical training in structural analysis. A sampling of the unpublished and published reports that have been catalogued in the more complete card file of the project will be presented as reference lists for each chapter. Tests of actual complete structures, insofar as found, are listed in complete form in connection with Chapters XI, XII, XIII, XIV.

This introductory chapter will include a discussion of the following items:

1) Purpose, scope, and limitations of preliminary report.
2) Preparation of the bibliographic card file.
3) The load-deflection curve or "resistance" function of a structural member or structure.
4) General outline of related fields of research and analysis.
5) Acknowledgement.

1:1 Purpose, Scope, and Limitations of Preliminary Report

The purposes of this report are as follows:

a) to make a preliminary review of readily available pertinent information, published and unpublished, including unfinished current research;
b) to determine those areas in which more information seems to be required, as a guide to the initiation of needed research projects;
c) to furnish new research projects a review of existing information, as an aid to their initial progress; and
d) to form the outline and basis for a more complete series of "final reports"* that will attempt to digest, evaluate, and apply available information so that one can predict the deflection and extent of damage in a steel structure, or one of its component parts, under a prescribed blast-load sequence insofar as this is approximately feasible in the light of existing or currently developed information.

Each section or related group of sections of the final report* (corresponding to chapters of the preliminary report) will be issued separately with preferential order according to the request of the sponsor. Certain sections of the final report will be given a more complete treatment than others, depending on their importance and the extent to which overlap may exist with other investigations of a similar nature. It will also be possible to prepare for distribution the project file cards related to the particular sections of the final report.

The following typical outline will be used for Chapters II, IV, V, VI, VII, VIII, IX, and X of this preliminary report:

1) Introductory Statement
   a) Definitions and notation, assuming a knowledge of elementary strength of materials and test procedures.
   b) General discussion of important factors pertaining to the subject of the chapter.

2) Survey of Theory and Experiment
   This will include references to the selected reference list (part 4, below). The arrangement will vary but will include

* The final report on columns (Chapter 7) is already in progress and is being prepared at Lehigh University, under the supervision of Mr. Lynn Beedle, Assistant to the Director of Fritz Engineering Laboratory.

a discussion of published, unpublished, and current research information. Such discussion as is appropriate to specific projects to be proposed in item 3 will be omitted.

3) Proposed Research Projects
Discussion of current research, reasons for need, and general outline of what is proposed. More complete details would be provided later by the institutions proposing to actually carry out the work.

4) Reference List
Author, subject, and primary references only, as the abstract information will be incorporated into the text of parts 2 and 3.

In general, time available for writing the preliminary report has not permitted detailed study of the references cited at the end of each chapter. In some instances, however, such studies have been made, and in other instances certain reports had been studied prior to the inception of this project. Selection of references has sometimes been based on available abstracts or summaries or upon familiarity with the author or authors, or on the fact that the references themselves offered extensive additional lists of references. In instances where many reports of a similar nature are available, as in the case of tests of walls at the Bureau of Standards, only representative reports are cited herein as references. Approximately four times as many references are available in the project card file as are selected for inclusion in this preliminary report. As mentioned before, no military classified information is included.

1:2 Preparation of the Bibliographic Card File.

Sources for the literature and research survey have included the following:

1) A questionnaire survey to cover current research and unpublished information.
2) A review of existing pertinent abstract services and previous special literature surveys.
3) Visits and on-the-spot searches by project representatives for literature at the principal university research centers in this country and in Western Europe.

To hold the scope of the project within proper limits, the following criteria were set up to guide the inclusion or exclusion of any reference item, whether published or unpublished.

A. References Included in the Card File:

1) Any report of a field or laboratory test carried well into the nonlinear range,* of a complete structure, that has metal elements, regardless of type of metal. In the case of reinforced concrete, there are included tests of foundations of metal buildings, composite structures, and shear walls, or roof panels.
2) Any report of a field or laboratory test, carried well into the inelastic range, of a component part of a steel structure. This would include steel columns, beams, girders, connections, tanks, and miscellaneous elements, and, in addition, wall or roof panels of any material, provided such panels were of a type used in a steel structure.
3) Any report of a theoretical investigation on the ultimate collapse strength of a structural frame, a truss, bridge, tower, or building, provided the theory is applicable to metal structures.
4) Any report of a theoretical investigation on the inelastic behavior of beams, columns, girders, connections, plates or slabs.
5) Any unpublished reports on the yield, ultimate strength, or other mechanical property except fatigue of structural steel, as well as all reports of tests at measured rates of strain, published or unpublished. Very slow rates of strain, such as in creep tests, are not included.
6) Information on actual designs of typical steel buildings, especially in countries other than U.S.A.
7) Information on the dynamical behavior of structures and their component parts, regardless of whether or not the inelastic range is exceeded.
8) All reports having extensive bibliographies or reference lists, if at all related to the project.

B. References Not Included in the Card File.

1) Elastic stress analyses, experimental or analytical.
2) Theoretical plastic stress analyses which have no direct use in predicting the ultimate strength of a steel structure.

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* It should be noted that buckling is a nonlinear phenomenon regardless of whether or not the material is stressed beyond the yield point.

3) Repeated load tests or failure theories.
4) Theoretical analyses (without test results) of plate, column, or beam buckling problems prior to 1940.
5) Theoretical analyses of any type prior to 1920.
6) Published metallurgical theories of plastic flow.

Each item selected for inclusion in the overall bibliography has been recorded on a 5-inch by 8-inch card, a sample card (slightly reduced in size) being illustrated below.

Author	1,4 Subject	1P12 File
Stang, A.H. Jaffe, B.S. Authors	Tests of large welded-steel box girders Title	
Weld J 28:89s-96s (1949) Primary Reference		
App Mech Rev 3:Abs 1105 (June 1950) Secondary References		
Bending tests are reported on four large welded-steel box girders, 30 in. wide, 25 in. deep, and with a span of 22 feet. Each test was run at one of the following temperatures: -40°F, 0°F, 40°F, 80°F. Failures were produced only in the beams tested at -40°F, and 0°F, after the center of the span had deflected 2.45 in. and 8.83 in., respectively. The tests at 40°F and 80°F produced no failure after deflections exceeding 16 in. Tensile tests of the material at these temperatures had indicated the steel was equally ductile at -40°F and 80°F. (Abs from App Mech Rev) Engineering Research Institute University of Michigan		

Referring to the sample card the space in the upper left-hand corner, labeled "author", is for filing purposes in case separate cards are later made for each co-author of a given paper. At the present time there is only one card for each particular paper, and the cards are filed according to the first listed co-author. Continuing across the top of the card in the space labeled "subject" is the number which refers to the chapter or chapters for which the particular card is an appropriate reference. In the upper right-hand corner the notation 1P12 indicates that this card has actually been listed as reference number 12 in the first chapter of the preliminary report. The spaces labeled authors, title, and primary reference require no explanation. In the space labeled secondary reference there is listed the abstract service, if any, where the reference was found and from which the abstract was quoted. In many instances original abstracts have been prepared, but published abstracts have been used wherever available. At the end of the abstract (which is copied in the large space in the lower half of the card) the source of the particular abstract is quoted. These cards are similar to those used by the Pressure Vessel Research Committee of the Welding Research Council in their extensive literature review.

The questionnaire survey to discover as many unpublished reports as possible was started in April, 1951, immediately after initiation of the project on April 1. A form was prepared together with a standard forwarding letter and explanation. Approximately 475 copies were mailed out in late April to persons or organizations in this country and to a limited number of persons in Canada, Great Britain, Switzerland, Norway, Belgium, Japan, and Australia. About one-half of the recipients made replies to the questionnaire. From the statement in the questionnaire the following is quoted: "of special interest are tests that have never been reported in published form. In the case of reports previously published, behavior in the elastic range has usually been emphasized. In such cases, if unpublished information is available on the ultimate collapse strength and corresponding deflection, this, too, would be of special interest. In the case of university testing laboratories, it is thought that many unpublished theses, dissertations, and reports for industrial concerns may be available. In addition, many private companies have conducted tests of their own on complete structure or their parts."

The results of the questionnaire have disclosed a considerable number of reports which add a store of important information to the files of the project.

Visits were made by a project representative to the University of Illinois, Cornell University, Case Institute, Ohio State University, Brooklyn Polytechnic Institute, Lehigh University, Columbia University, M.I.T., Stanford University, University of California, California Institute of Technology, and University of California at Los Angeles. Contacts on the west coast were made by a former graduate student who resides in that area. Mr. Alfons Huber, a former graduate student at Lehigh University, and a resident of Austria, having returned to his home country, was engaged to visit the accessible European university laboratories where structural research was or had been in progress. Although not much unpublished information was disclosed by this trip a very considerable number of reports have been acquired for the project files, including some not listed in current abstract services. In the search for published information the following sources have been utilized:

Am Iron and Steel Inst
Contributions to Metallurgy of Steel 21 and 38
(Surveys of Literature on Plastic Deformation of Metals, covering 1940-49)

Am Soc Civil Engs Trans (1935-1950)

Am Soc Mech Eng J App Mech (1935-51)

Am Soc Metals
Review of Metals Literature (1944-51)

App Mech Reviews (1948-51)

Bautechnik (1928-41) (1947-50)

deJonge, A.E. Richard
Riveted Connections (Abstract)
(Critical survey of information--1837-1944)

Eng Index
(Abstracts) (1910-51)

Int Assn Bridges and Struct Eng
Congress 1 and 2
Publications (1932-51)

Jakkula, A. A. and Stephenson, Herson K.
"Steel Columns - A Survey and Appraisal of Past Works"--Bulletin No. 91, Texas Eng Exp Sta

Library of Congress
Navy Res Sect (abstracted unclassified Government reports at Library of Congress)

Nat Bur of Standards
Letter Circular LC910, publication of Staff of NBS, Eng Mech up to July 27, 1948 (Abstracts)
Bldg Mat and Struct (1938-51)

NACA
Index of NACA Tech Publ (1915-49)
35th and 36th Annual Reports (1949-50)
Reports received in 1951

Pressure Vessel Res Comm
Physical and Mechanical Characteristics of Materials for

Pressure Vessels, Vol 1-4 (Abstracts) (primarily 1937-47, a few prior to 1937)

Soc of Experimental Stress Analysis
Proceedings, Vol 1-9

Steel Structure Res Comm
First, Second, and Final Reports

Structural Engineer (1938-51)

Taylor Model Basin
Rep 507, Catalog of Taylor Model Basin unclassified publications issued up to September 30, 1950

U S Office of Tech Services Bibliography
1948 - June, 1951 (Abstracts)

Zentralblatt für Mechanik (1932-44)
(Abstracts)

The literature search will be extended in the further stages of the investigation by the inclusion of other pertinent periodicals and by the tracking down of references at the end of listed articles that have been included in the survey up to this stage of the work. This latter procedure is probably the most usual and logical means of obtaining complete coverage, but the time available for the preliminary report has not permitted its extensive use.

1:3 The Load-Deflection or "Resistance Function" of a Structural Member or Structure.

Of primary interest in studying any particular structure or structural member is the recording of either the static or dynamic load history by means of the load-deflection curve, wherein a controlling—perhaps maximum—deflection is plotted as a function of load. Consider, for example, the structure in Fig. 1.1, shown in a deflected configuration while being loaded to failure. One might plot the load-deflection curve of such a structure and it conceivably might be as shown in Fig. 1.2.

It is recognized that the resistance function will usually depend to some degree on the rate at which the various parts of the structure are deformed.

Referring to Fig. 1.2, during the initial elastic region, the load increases rapidly with relatively small deflections. If the loads were removed, the structure would return to its original undeflected position. The initial yield of the structure as a unit is that point on the load-deflection curve at which there is a perceptible deviation from a straight line. This frequently is a rather inexact point, depending on the precision of instrumentation and on the scale of the load-deflection plot. Local stress concentrations, residual stresses,* and other effects may cause very localized yielding at much lower loads, but (depending on the scale that is used), these usually will not cause noticeable deviation from linearity until they have accumulated their effects to a certain degree.

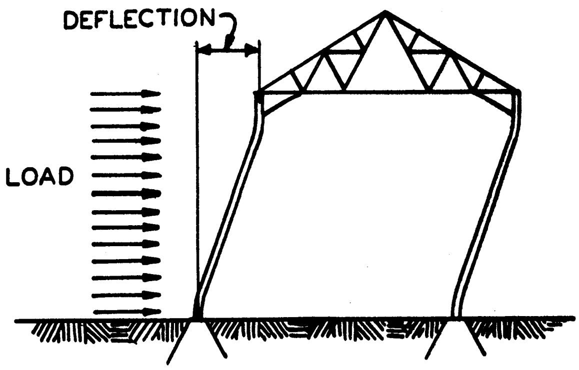

FIG. 1.1

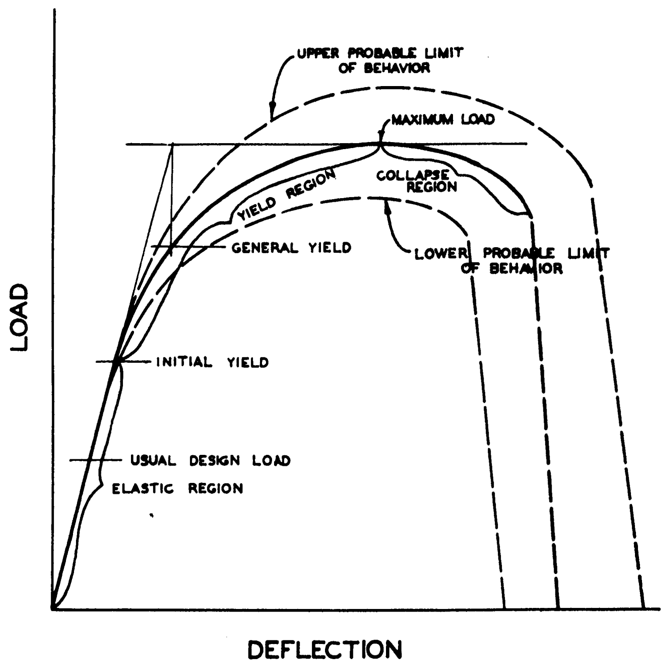

FIG. 1.2 TYPICAL LOAD-DEFLECTION CURVE FOR A STRUCTURE OR STRUCTURAL MEMBER

* In some critical instances, as in the case of intermediate-length steel columns, residual stresses may cause marked lowering of strength and corresponding alteration in the load-deflection curve.

The elastic range and stress analyses pertaining thereto are of little use in this investigation since they usually have little relation to the post-yield behavior of a ductile steel structure.

After the initial yield becomes perceptible, the nonlinear or inelastic range is entered, wherein yielding progresses and deflections usually increase at a progressively increasing rate. In some cases there may be a reversal of this trend, but in any case, if the load is removed, the structure will not return to its original undeflected position, it will have experienced "permanent set." At the maximum load a condition of overall instability is reached. Beyond this point the applied load must be reduced progressively if static equilibrium is to be maintained. Any load greater than that indicated by the curve will, if maintained, cause accelerated motion ending in complete collapse, hence, this region has been termed the collapse region. In general, the maximum load will be determined either by the overall buckling or by fracturing of some key part or parts of the structure, or by a combination of these two factors. Local buckling and local cracks may appear prior to reaching maximum load. Buckling, of course, can occur either in the elastic or plastic range, usually the latter.

The behavior in the yield and collapse regions, as shown in Fig. 1.2, is of primary interest in the present investigation. The probable distribution between upper and lower limits, as indicated qualitatively by the dashed lines in Fig. 1.2, is also of special interest, as is the average behavior of a given type of structure (as might be indicated by the solid line in Fig. 1.2). Scatter is caused by variations in material, fabrication, residual stresses, and the degree to which tolerances of size, straightness, fit, etc., are met.

It is desirable to determine for any given type of structure at least two damage levels, (1) crippling, and (2) collapse. Crippling damage would render the structure unfit for a reasonably long period of time for the use to which it was being put at the time of attack. The time that would be required for repair would be an important consideration. A steel mill distorted to the extent that would make essential crane facilities inoperable so that extensive and time-consuming repairs were required would be considered "crippled". Damage to the extent requiring razing and complete rebuilding will be classified as "collapse". After crippling damage, a structure might be used for some other purpose, such as storage, but it would be unfit for <u>any</u> use after "collapse".

For structures of any given type the proportion of dead to live load stress will increase as span or size increases. In a very large structure, therefore, the dead load may be able to complete the work of collapse after a given duration of blast impulse, whereas in a smaller structure of similar type, the complete collapse would have to be produced by the blast effect alone. Contrariwise, a structure might be loaded beyond its point of maximum load resistance, yet not collapse if the applied blast load dissipated rapidly.

Acknowledgement is made to those who furnished suggestions regarding the "Preliminary Tentative Statement of Objectives," of May 31, 1951. The foregoing statement is in part a revision of the above mentioned tentative statement of objectives.

1:4 General Outline of Related Fields of Research and Analysis.

The primary source of information useful to this survey on the collapse strength of beams, connections, columns, and frames is the store of published and unpublished research reports on structural tests in which complete structures or their component parts have been tested well into the plastic range, in many cases beyond the point of maximum load. Most tests of this sort have been of the static variety, in which the load is applied so slowly that the conditions at any one time are essentially in static equilibrium and do not involve inertia forces. Some dynamic tests have been made and these (if unclassified) are, of course, included. However, the difficulty and cost of loading and instrumentation for this type of test make it a relatively recent source of information. There are, of course, many theoretical papers on dynamic behavior as well as a considerable number of experimental investigations in which the members tested are primarily of academic interest since they are not structural sections but are small-scale rectangular or round-bar beams or columns. While these tests are of value in checking or verifying mathematical solutions they have only a theoretical relationship to the behavior of an actual engineering structure under dynamic loading.

In addition to the reports on structural tests there are other related fields of study which can be correlated to this investigation and to which attention has been given in the search for bibliographical material.

A principal related field is that of the "Theory of Plasticity". When applied to the complete stress analysis of small elements this theory is analogous to the theory of elasticity, with Hooke's law replaced by a generalized inelastic relationship between stress and strain. The theory of plasticity, while attempting a somewhat more realistic approach than the theory of elasticity, is greatly handicapped by the mathematical complexities that arise when Hooke's law is abandoned. Relatively few solutions are available, and these have, in most cases, but little relationship to the strength or overall deformation of a bridge or building comprised of structural beams, columns, and tension members. The branch of plasticity theory which attempts to predict the deformation and strength of steel frames, however, is of direct value to this investigation, from a theoretical point of view. This is a field to which increasing attention has been given in the past twenty years, not so much because of any interest in the forces required to destroy a structure, but rather because of a certain trend of opinion toward what has sometimes been termed "limit design", or "plastic design". The limit designer proposes to calculate the ultimate strength of any structure and divide this calculated load by an arbitrary factor of safety to obtain the allowable design load. In conventional design the allowable usable load is determined on the basis of an average working stress and in some cases this undoubtedly has resulted in over-safe structures. As a result of the interest in the potentialities of limit design many investigations and tests have been carried out, especially in relation to continuous welded steel frames. Baker and his associates in England have been leaders in the theoretical, experimental, and design aspects of this development and have probably carried it further than any other group. Some years ago Van den Broek in this country focused attention on limit design and has since prepared a book which reviews both theory and experiment (1.17).* A very complete review of methods for calculating the ultimate strength of continuous frames has been presented by Symonds (1.13), and the relationship of limit design to dynamic loading has been discussed by White, (1.18). Experimental work in this country closely paralleling that of Baker and his associates in England has been in progress since 1945 at Lehigh University (1.21).

Another field that has some bearing on the ultimate strength of steel structures is that concerned with the transition from ductile to brittle behavior which structural steel experiences at certain critical temperatures under critically associated conditions of stress concentration and stress distribution. The brittle fracture of welded bridges, pressure vessels, and ships has given great impetus to research. Baker (1.3) has reviewed the problem of brittle fracture in ship structures as based on British experience. A U.S. Government publication (1.16) gives a complete record of the brittle fractures in the ships built during the last war together with the changes in design and methods of construction that were instituted to prevent or minimize the possibility of further fractures. The susceptibility to brittle fracture might possibly be taken advantage of during bombing operations. The most favorable conditions for utilizing this factor would be in the bombing of steel bridges or exposed metal tanks, at low temperatures, under which condition the tendency toward brittle behavior is most pronounced.

Although dynamic tests of engineering structures such as bridges or buildings are relatively small in number, there are certain related

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* Numbers in parentheses refer to references at the end of chapter.

fields of investigation that warrant study in connection with this project. The problem of isolating a piece of electronic equipment against shock that might occur during shipboard service or that of protecting a breakable article that is packaged, when the package is dropped, involves the basic problem of response to shock that is similar to that of the response of a building or bridge to a suddenly applied load (1.6). Reports have been widely circulated (1.5, 1.15) on the nature of loads produced by atomic blast. The action of these loads on elastic structures has been described in general by Wilbur (1.20). and numerical, graphical, and analytical methods of analysis are available by means of which the deflections of a building subject to dynamic load can be determined, provided its load deformation characteristics are known in advance (1.4, 1.7, 1.9, 1.18, 1.19). There is a great deal of information available on the stresses induced in highway and railway bridges during the impact of moving loads (1.1, 1.2). Such investigations are in the elastic range, wherein the response of a structure is primarily a function of the natural frequencies of vibration, the load-time function, and the damping of the structure. The numerical methods cited have the advantage that they can be applied in a simple and direct manner to either the elastic or plastic range of dynamic behavior of complex structures. The problem of design against earthquake, as well as the military problems of shock load, has given impetus to the development of many of these methods, and the problem of dynamic analysis during an earthquake is similar to that of analyzing the behavior under sudden pressure loading induced by atomic blast. In the one case the foundation alone is subject to sudden movement, whereas in the case of atomic blast the load is applied to the surface of the building, with the possibility of ground shock also entering the problem. It is not the purpose of this investigation to review dynamic procedures of structural analysis in great detail, but investigations involving such procedures and the study of earthquake phenomena and their effects on actual structures may provide pertinent information that should not be overlooked.

1:5 Acknowledgement.

This is a project of the University of Michigan Civil Engineering Department, which is under the chairmanship of Professor Earnest Boyce. The authorization of the work is the subject of Purchase Order W-592 issued by Sandia Corporation to the Engineering Research Institute of the University of Michigan.

The initial drafts of chapters IV, V, VI, VIII, IX, X, XIII, and XIV were prepared by the first author, L.S. Hu, who has been a full-time research associate. The second author, R. C. Byce, who has been a half-time research assistant, wrote the initial drafts of chapters III, XI, and XII. The third author, Bruce G. Johnston, Professor of Structural Engineering at University of Michigan, wrote chapters I, II, and VII, edited the

other chapters, and has been in general charge of the project.

Special acknowledgment is due Dr. N. M. Newmark, Dr. R. J. Hansen, and Mr. Lynn S. Beedle for their review and criticism of the manuscript, and to many others for helpful suggestions relative to the initial statement of objectives. Acknowledgement is also due to the representatives of Sandia Corporation for their guidance and cooperation in suggesting the trends that the work should take.

1:6 References

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1.2	ASCE	"Final report of the special committee on impact in highway bridges" Proc ASCE 55:727-49, 1303-4 (1929)
1.3	Baker, J. F.	"The problem of brittle fractures in ship structures" Proc of Conference at Eng Lab, Cambridge Univ (1945)
1.4	Beer, F. P.	"Method for the graphical determination of the motion of a structure with one degree of freedom under blast loading" 1st Nat Cong for App Mech (1951)
1.5	Los Alamos Sci Lab	"The effects of atomic weapons" Superintendent of Documents, U S Gov Printing Off (Sept 1950)
1.6	Mindlin, R. D.	"Dynamics of package cushioning" Monograph B 1369, Tech Publ Bell Telephone System
1.7	Newmark, N. M.	"Methods of analysis for structures subjected to dynamic loading" Report to the Directorate of Intelligence, (1951) U.S. Air Force, Physical Vulnerability Branch
1.8	Prager, W. Hodge, P.G., Jr.	<u>Theory of perfectly plastic solids</u> John Wiley and Sons, New York (1951)

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1.15	U S Atomic Energy Commission	"Civil defense against atomic warfare" Selected reading list, U S Atomic Energy Commission
1.16	U S Navy	"The design and methods of construction of welded steel merchant vessels" U S Gov Printing Off (1947)
1.17	Van den Broek, J.A.	<u>Theory of limit design</u> John Wiley and Sons, New York (1948)
1.18	White, M.P.	"The limit design of structures subjected to impulsive loads—with application for military structures" OSRD Rep 4192, NDRC Armor and Ordnance Rep A-293 (1944)

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1.20	Wilbur, J.B.	"Action of impulsive loads on elastic structures" J Boston Soc Civil Eng 33:163-73 (1946)
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CHAPTER II

STRENGTH, DEFORMATION, AND FRACTURE PROPERTIES OF STRUCTURAL STEEL

2:1 Introductory Statement.

A knowledge of the conditions under which steel will yield or fracture and the stress-strain relations that obtain during yield is a necessary preliminary to the analysis of structural members in their plastic range of behavior. In the elastic range there are accepted average values of the modulus of elasticity with little scatter between various test results. Minimum specification values of the yield point govern the acceptance of structural steel (ASTM A-7) used in bridges or buildings. In the inelastic range, however, there are no such accepted average stress-strain curves for even the commonest types of steel, and the inelastic properties are subject to considerable scatter. Interest is turned from the minimum to the average values of yield point and its expected scatter in a given type of rolled section. Hence, a statistical approach is required to a greater degree than for conventional analysis in the elastic range.

The subject of the inelastic behavior of steel and other metals is one of increasing interest to the structural engineer and has long been under the attention of the physicist, metallurgist, and the mathematician. Although the physicist is interested in all phases of behavior, he is particularly concerned with the basic structure of matter on a submicroscopic scale. The metallurgist's interest lies in the fields of both the engineer and physicist; but he is primarily interested in the chemical constituents of metals, the physical and mechanical behavior of metal crystals and polycrystalline aggregates, and the effects of heat, changes of shape, etc., on the metallic properties of importance to the electrical, mechanical, or structural engineer, as the case may be. The mathematician has developed what is known as a "theory of plasticity" which requires as its basis the formulation of inelastic relationships between stress and strain to replace the linear relation expressed by Hooke's law that is basic to the "theory of elasticity".

Theories of elasticity and plasticity produce, where mathematically possible, general three-dimensional solutions of stress distribution. Fortunately for the structural engineer, the behavior of structural members (columns, beams, and tension members) is governed primarily by uniaxial stress. In other words, the load-carrying part of the stress system is one in which two of the principal stresses are zero or nearly so. (At any point there are always three principal stresses, mutually perpendicular, acting normal to planes in which there are no shear-stress components. The three principal stresses always include the maximum and minimum normal stress components.)

Although the behavior of structural members is primarily determined by uniaxial stress fields, the yielding of a material is primarily a function of shear stresses which, if present, act on planes that make an angle with the principal planes. (The maximum shear stress at a point is equal to one-half of the difference between the maximum and minimum principal stresses and acts on a plane having a normal direction midway between the normal directions of these two principal planes.)

The structural engineer, if he is to predict the strength of a plate in bending or a tank under pressure, must also be concerned in such cases with stress distributions at a point wherein two of the principal stresses may have about the same order of magnitude and only one is near zero.

Studies and experiments to determine the condition or experimental "law" of initial yielding for an isotropic, homogeneous, material in a homogeneous stress field have formed a common meeting ground for the metallurgist, mathematician, and engineer. (An isotropic material has the same properties in every direction. A homogeneous material has the same properties at every point in the given structural part. A homogeneous stress field has the same orientation and magnitude of principal stress at every point.) Real materials, of course, are neither isotropic nor homogeneous, though they may approach this condition on the average. Although much attention has been given to the study of "laws" of initial yielding, and reasonably good theories are available (within the limitations of their assumptions), these theories have little use; nor are they generally required for the prediction of the inelastic behavior of beams, columns, and tension members. As has been said, the behavior of these basic structural members is determined primarily by uniaxial stress. The ductile behavior of material in a uniaxial stress system, in turn, may be based on properties determined by the simple tension or compression test, to which attention is now given.

The initial portion of the typical stress-strain curve for structural steel, in compression or in tension, is shown in Fig. 2.1. The complete curve, plotted to the same scale, would take up a horizontal space of between twenty and thirty times that available on the drawing. Nevertheless, the inelastic strength of beams and columns is very largely determined by stress and strain within the range of Fig. 2.1.

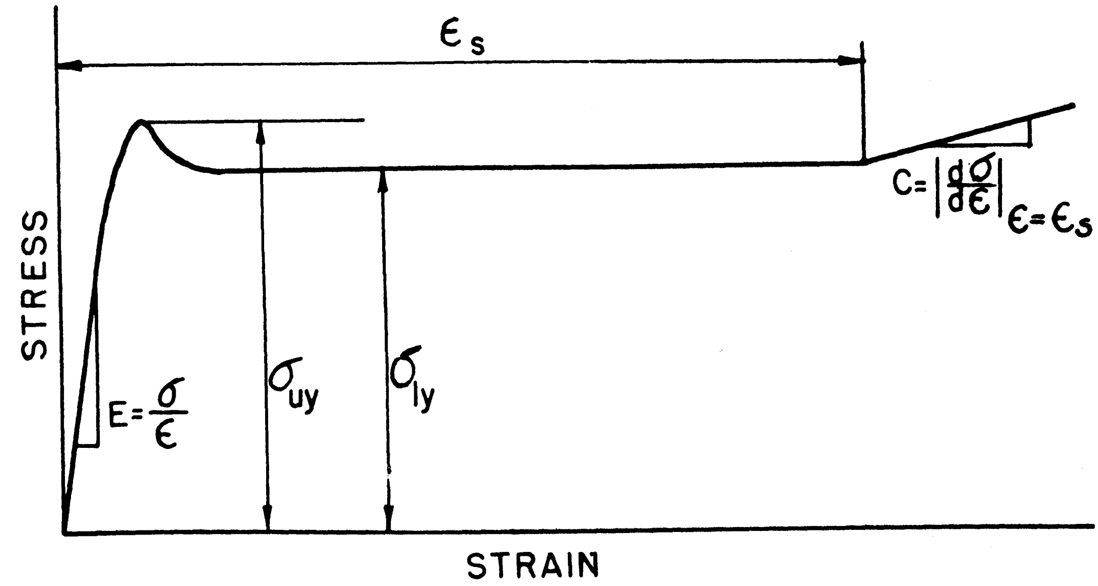


FIG. 2.1

The evaluation of inelastic structural strength can be done only on a "mean" or average, statistical basis, within statistically estimated probable limits. Basic to such evaluation would be the statistical study of stress-strain characteristics over a wide range of samples. To facilitate such a study, the stress-strain curves from each sample might be catalogued by listing the five items shown in Fig. 2.1, the numerical values of which determine the inelastic characteristics of structural steel essential to inelastic strength and deformation calculations. These five items, as shown, include two slopes, two stress levels, and one strain, defined as follows:

E = Young's Modulus = slope of stress-strain curve in elastic range

σ_{uy} = Upper yield point

σ_{ly} = Lower yield level

ϵ_s = strain at initial strain hardening

$C = \left|\frac{d\sigma}{d\epsilon}\right|$ = rate of strain hardening (average values at various strains)

σ_m = Maximum tensile strength (not shown in Fig. 2.1, but of importance in some cases, as, for example, tension member failure.)

The upper yield point is dependent on temperature, rate of strain, and the surface characteristics of the test specimen. The lower yield level may be thought of as the lower limit (for a designated temperature) of a family of "yield levels" dependent on strain-rate and temperature.

The yield level in a tension or compression test may be defined as the level of stress, after initial yield; that, at a given temperature and rate of strain, is sufficient to develop successively new planes of slip in the portions of the test specimen that remain in the elastic state. After initial yielding has proceeded discontinuously from point to point throughout the specimen, "strain-hardening" commences and the stress rises with further increase in average strain. The yield level is frequently of primary significance in determining the ultimate strength of a structure or structural member under either static or dynamic load conditions.

The stress-strain diagram in Fig. 2.1 indicates that in the region just prior to the upper yield point the relationship is apt to be curved. The effect of this curvature is of negligible importance in predicting the ultimate strength of a steel structure or structural member, because of the influence of residual stresses, local yield stress variation, eccentricities of load, curvature of members, etc. These factors cause a much greater "effective" curvature than usually exists in the stress-strain curve for an individual small sample of structural steel.

Fig. 2.2 shows the results of a statistical study, on a very limited scale, of laboratory test data for a number of samples of a certain steel, introduced at this point simply as an indication of the type of information that is desired in order to study the range of behavior. Fig. 2.3 shows a more extensive statistical study of yield point and ultimate strength, based on mill tests.*

Ultimately, in the presence of tensile components of stress, and usually after at least local plastic flow, structural steel fractures. If fracture occurs with but little overall deformation, the material is said to behave in a brittle manner, whereas if considerable plastic flow takes place prior to fracture, the material is said to behave in a ductile manner. The actual behavior depends on a multiplicity of interrelated factors, including temperature, surface characteristics, rate of strain, state of stress (no ductile flow can take place in a state of triaxially equal or "all round" tension or compression),rate of change of stress from

* For a discussion of mill tests, see proposed research project at end of this chapter. The data in Fig. 2-3 were furnished by courtesy of Jackson and Moreland, Engineers, Boston, Mass.

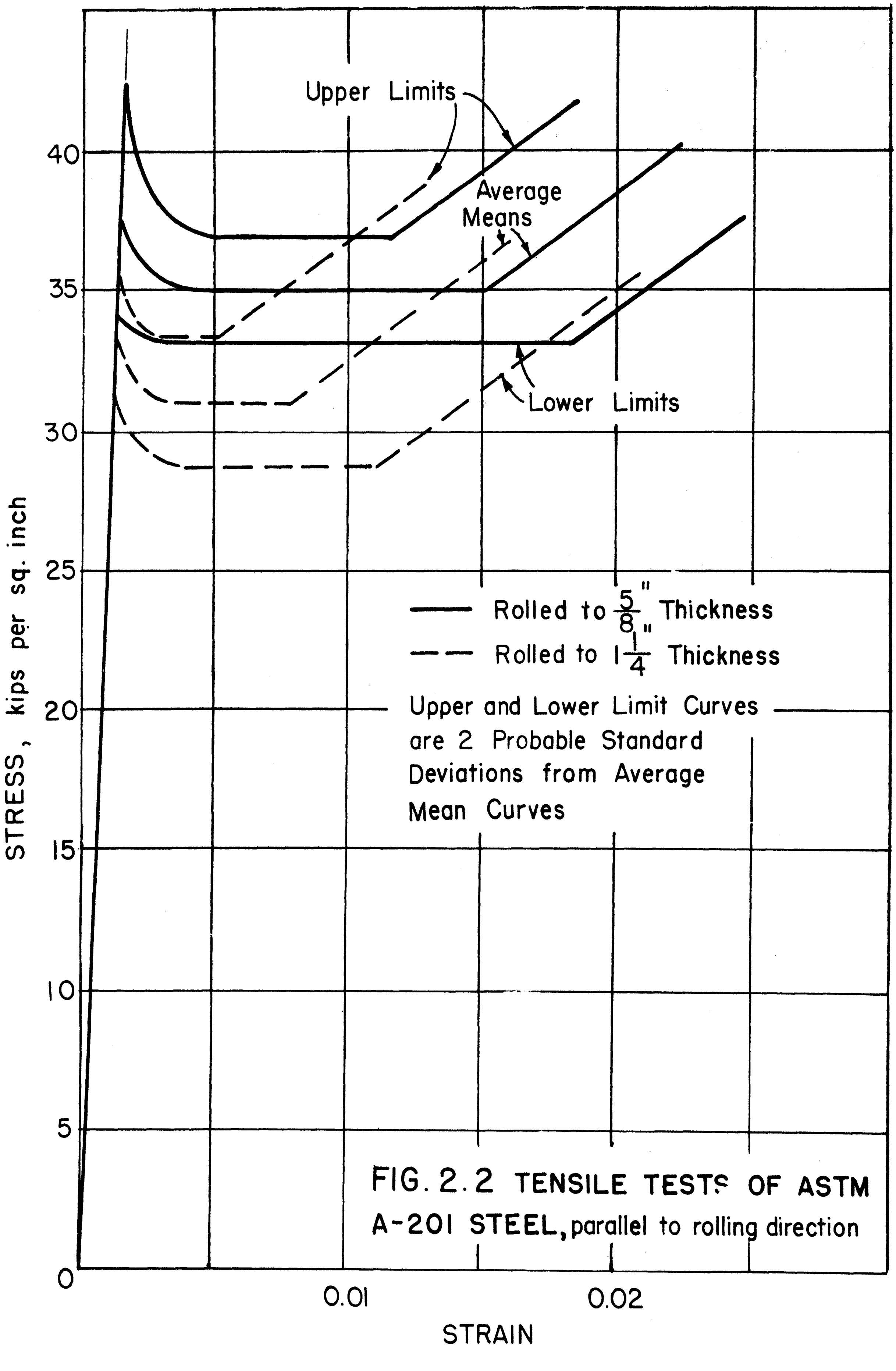
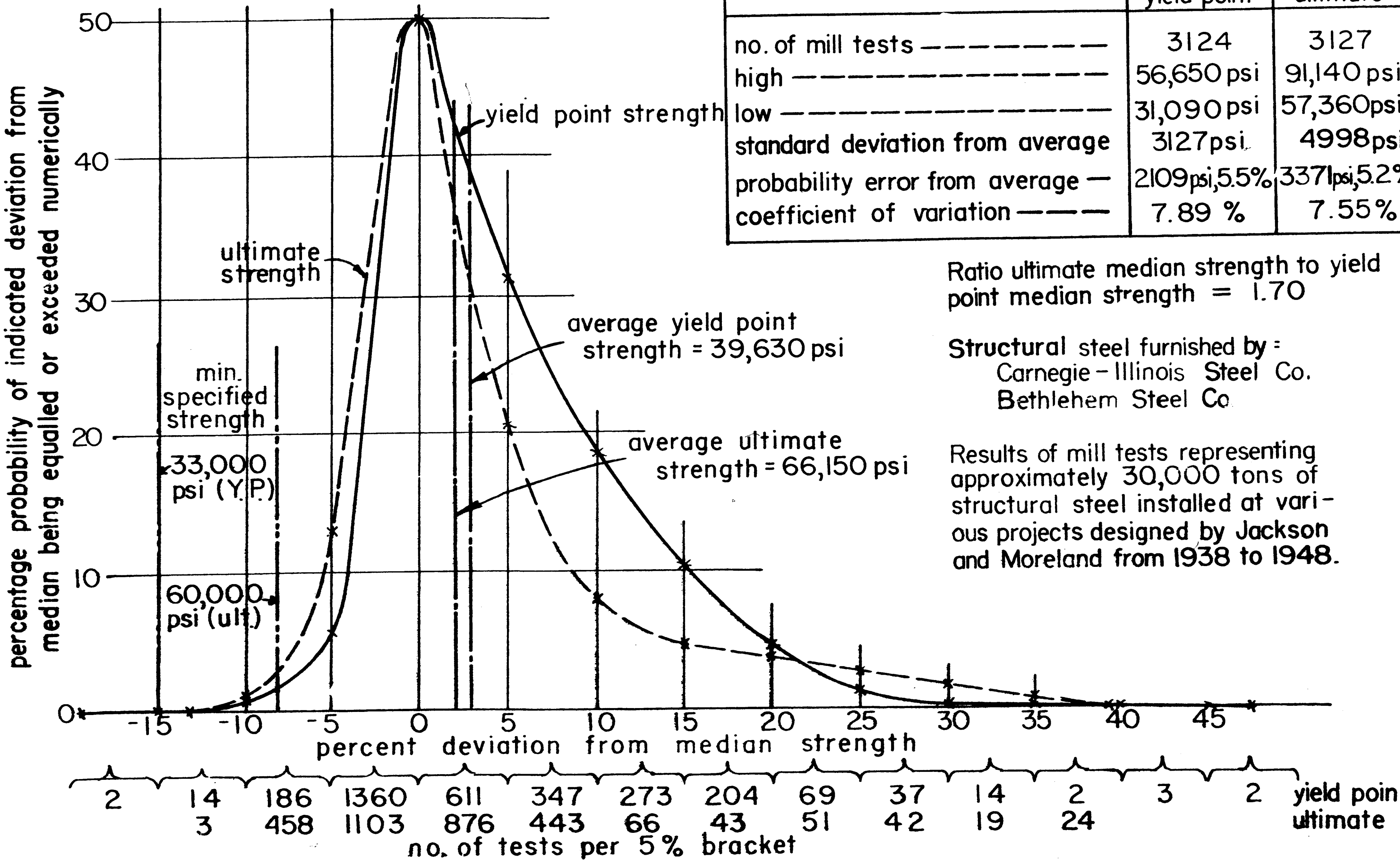


FIG. 2.2 TENSILE TESTS OF ASTM A-201 STEEL, parallel to rolling direction



	yield point	ultimate
no. of mill tests	3124	3127
high	56,650 psi	91,140 psi
low	31,090 psi	57,360psi
standard deviation from average	3127psi	4998psi
probability error from average	2109psi, 5.5%	3371psi, 5.2%
coefficient of variation	7.89 %	7.55%

Ratio ultimate median strength to yield point median strength = 1.70

Structural steel furnished by:
Carnegie-Illinois Steel Co.
Bethlehem Steel Co

Results of mill tests representing approximately 30,000 tons of structural steel installed at various projects designed by Jackson and Moreland from 1938 to 1948.

FIG. 2.3 RESULTS OF MILL TESTS

point to point (stress-gradient), and metallurgical factors involving composition, prior heat treatment, and deformation, including the deformation that may occur during the actual final loading of the material.

In the actual structure such factors as design and workmanship also enter into the problem. In laboratory tests of small specimens of a given steel, conditions can be introduced that will nearly always cause brittle failure, for example, extremely low temperature in the presence of a sharp notch. However, in an actual structure, in spite of the well-known cases of brittle fracture of welded ships, pressure vessels, and bridges, the load and deformation at final fracture will often be subject to rather wide variation. The brittle failure load and corresponding deflection of an actual structure would be difficult to predict, since such failures occur only in borderline cases where a multiplicity of adverse factors occur in critical combination.

2:2 Tension Members.

The behavior of a tension member is directly comparable to that of a small-scale tension test of the material of which it is made. More often than not, failure of such a member in an actual structure will be due to failure of the end connection, to be considered in Chapter VI. The tension member, as compared with the tension test, is apt to be more susceptible to brittle failure, owing to the presence of holes, welds, rivets, or small notches caused by the shearing or flame cutting of edges in shop fabrication. Such notch effects create local zones of triaxial tension stress below the surface, inhibiting ductile behavior, and, at the same time, promote localized regions of large plastic deformation at the surface that exhaust locally the plastic deformability of the metal and lead to the inception of cracks.

2:3 Survey of Theory and Experiment.

Recent authoritative books, abstract services, and literature reviews cover well the phases of this section that are pertinent to this project. About 600 references have been selected for card indexing, but only a selected few of these will be listed in this preliminary report. This abbreviated treatment seems appropriate in view of the afore-mentioned availability of books and surveys, as will be described.

The 23 references listed for this section cover the four viewpoints discussed in the Introductory Statement, i.e , those of the physicist, the metallurgist, the mathematician, and the engineer, insofar as they relate to the interests of this project.

The recent book by Freudenthal (2.13)* is unusual in that it attempts the difficult task of coordinating the approach of the physicist, mathematician, and engineer. Nadai's revision (2.17) of his earlier book, Plasticity, is very complete in describing the flow and fracture properties of metals and criteria for initial inelastic behavior. These books cite many references and represent in themselves thorough literature reviews carried out over periods of many years.

The Subcommittee on Plastic Flow of the American Iron and Steel Institute Research Committee has published two Literature Surveys on this subject, covering the years 1940 to 1949 (2.1, 2.2). These are annotated bibliographies rather than coordinated analyses of the subject matter, but the abstracts are given in some detail. The American Society for Metals in their Review of Metal Literature, available since 1944 (2.3) covers not only all phases of the behavior of metals but metal structures as well. The abstracts in this service are in general more accurate and the coverage more complete than in the Engineering Index. The literature survey conducted by Pennsylvania State College for the Pressure Vessel Research Committee of Welding Research Council (2.19) is a very complete coverage of the period 1937 to 1947 and is based principally on existing abstract services that include Chemical Abstracts, ASM Review of Metal Literature, The Engineering Index, The Industrial Arts Index, The Institute of Metals Metallurgical Abstracts, The Iron and Steel Institute (Great Britian), Metallurgical Abstracts, Metals and Alloys, Science Abstracts, and the U.S. Office of Technical Services Bibliography of Scientific and Industrial Reports. Emphasis in this survey is on the chemical and metallurgical factors affecting flow and fracture properties of metals in general, mechanical properties of metals used in pressure vessels, and test procedure. The material is an annotated bibliography and most of the abstracts are copied verbatim from standard sources, with credit as to their origin. There are four volumes, listing about 5,000 items.

Reference will now be made to the more specialized surveys and articles of general interest. Andrade (2.5) has presented a brief, illustrated and popularly styled presentation on current research of the physicist on the basic behavior of metals, and Orowan (2.18) presents a lengthier review, with emphasis on the fracture problem, from the same viewpoint. There are a number of special reviews of literature and symposia on the fracture problem, not all of which are listed here, as even those listed overlap to some extent (2.4, 2.12, 2.14, 2.15, 2.16). Interest in the fracture problem has been stimulated by the failures of all-welded ships and pressure vessels. Hoggart (2.15) presents the overall problem with brevity and clarity. In a progress summary, that is one of many excellent reports, Williams, et al., (2.23) reports on the investigations of the Ship Structure Committee. In a wartime report Walmsley and Marsh (2.22) present general conclusions based on

* Numbers in parentheses refer to references at end of chapter.

a survey of about 660 references, covering the yield strength, plastic behavior, rupture strength, ductility, energy absorbed to fracture, rate of strain, and other behaviorisms of metals under multi-axial stress systems.

The mathematician (summarizing previous discussion of this topic) takes up two main problems:

1) the formulation of a mathematical statement that will describe the general three-dimensional relationships between stress and strain in the inelastic domain; and

2) by use of the foregoing formulation, applied in the "theory of plasticity", the determination of stress and strain throughout a given member under a given set of loads or imposed deformations.

An excellent and thorough survey of the first problem above, covering literature down to 1950 and including not only an originally annotated bibliography but also a complete historical and thoroughly correlated digest is presented by Drucker (2.11).

The influence of rate of strain on yield strength and other properties is of direct importance in investigations of the dynamical behavior of structures. There is much information in the literature on extremely slow rates at elevated temperature (creep) (2.13, 2.17) and extremely high rates involving shock waves such as are produced by the sudden impact of two bodies moving at relatively greatly different velocities (2.10). There is also some information available in the intermediate range between ordinary test speeds and the somewhat higher rates of strain that might be expected in a flexible structure under the action of a suddenly applied pressure load <u>not involving a forcibly impressed deformation velocity</u>. Nadai (2.17, p. 297) lists references on this intermediate range of strain rate, including some with extensive bibliographies, and provides a brief review of the existing state of information. Of importance is the recently developed information (2.9) regarding the time required to initiate plastic deformation.

Stimulated by the experience with brittle fractures that frequently occurred in welded ship plate, a number of investigations have been devoted to the brittle behavior of large-size tension members, usually in the presence of local stress raisers, and involving the effect of variation in temperature (2.6, 2.7, 2.20, 2.21, 2.24). There are also available in the files of several steel companies results of tests on eyebars, a type of member used in bridges during the early part of the century and still in use to a limited extent. The eyebar comes closest to complete similarity with the small laboratory tension test, and the principal strength problem is not that of the bar but the details of the pinned end connection (see Chapter VI).

2:4 Proposed Research Projects.

RP2.1--Inelastic Properties of Structural Steels.

Referring to Fig. 2.1, the inelastic properties denoted by σ_{uy}, σ_{ly}, ϵ_s , and c should be determined for a great variety of samples taken from all parts of the country. Correlation should be made, if possible, with sample from foreign countries. The results of these tests should be studied statistically to provide an evaluation of mean values and the probable scatter of individual samples. If not studied as a special project under the beam or column project proposals, the question of variation in inelastic properties and the existence, magnitude, and distribution of residual stresses throughout various types of structural sections should be explored on a broad basis. These are the two primary factors that determine the effective stress-strain curve that must be used to describe the bending behavior of a member as a whole, if such behavior is to be computed analytically from the results of tension and compression tests of small samples.

A possible opportunity to amass much information concerning one of the most important plastic items, the yield point, lies in the fact that commercial steel mills have on record the yield points of steel samples taken from an innumerable variety of rolled steel members over a period of many years. The value of this information is limited by at least two factors. Commercial steel-mill tests are made at strain rates (as permitted by ASTM specifications) that cause a very appreciable rise in the yield level as compared with slow strain rates. Secondly, in the case of rolled WF (wide flange) or I-beam shapes, the samples are taken from the webs, which are much thinner than the flanges, hence (owing to their relatively greater cooling rate) the webs have higher yield-point values than the more important flange locations. If the factors previously mentioned can be evaluated within reasonable limits by evaluating steel-mill test speeds and the relation between yield point and thickness of metal in a given shape, the steel-mill records might prove to be a worthwhile field for further study.

Since, under dynamic conditions, reversal of stress may occur in the inelastic range, the great reduction and alteration in inelastic properties under stress reversal, termed the "Bauschinger Effect", should be well defined for commonly used steels. This would also require an extensive test program with a statistical approach.

Of special importance in dynamic studies is the effect of strain rate on the yield level and other inelastic properties of structural steels. Attention should also be given to the recently developed information on the time delay in the initiation of plastic deformation. Considerable information on strain rate is now available and current projects are in progress. Recommendations for a project on this subject will be deferred until the present state of knowledge can be more thoroughly evaluated.

The foregoing recommendations for research might well be broken down into a number of separate projects. However, there is no clear-cut preference now apparent for such breakdown, and it might be just as well to let the wishes of the prospective research institutions govern the distribution of the work and details of project subdivision.

2:5 References.

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2.2	AISI	Plastic deformation of metals--survey of literature 1946-1949, inclusive Contribution to the metallurgy of steel, No. 38 AISI (1950)
2.3	ASM	American Society for Metals--Review of metal literature, Vol 1 (1944) to Vol 7 1950
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2.6	Boodberg, A. Davis, H.E. Parker, E.R. Troxell, G.E.	"Causes of cleavage fracture in ship plate--tests of wide notched plates" Weld J 27:186-99s (1948)
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CHAPTER III

TYPES OF STEEL FRAMEWORKS

Structures will be classified into various groups and those in a particular group may be expected to behave in a similar fashion, dynamically, under the sudden application of blast loads. Reference will be made to drawings of typical examples of each group. The overall classification is as follows:

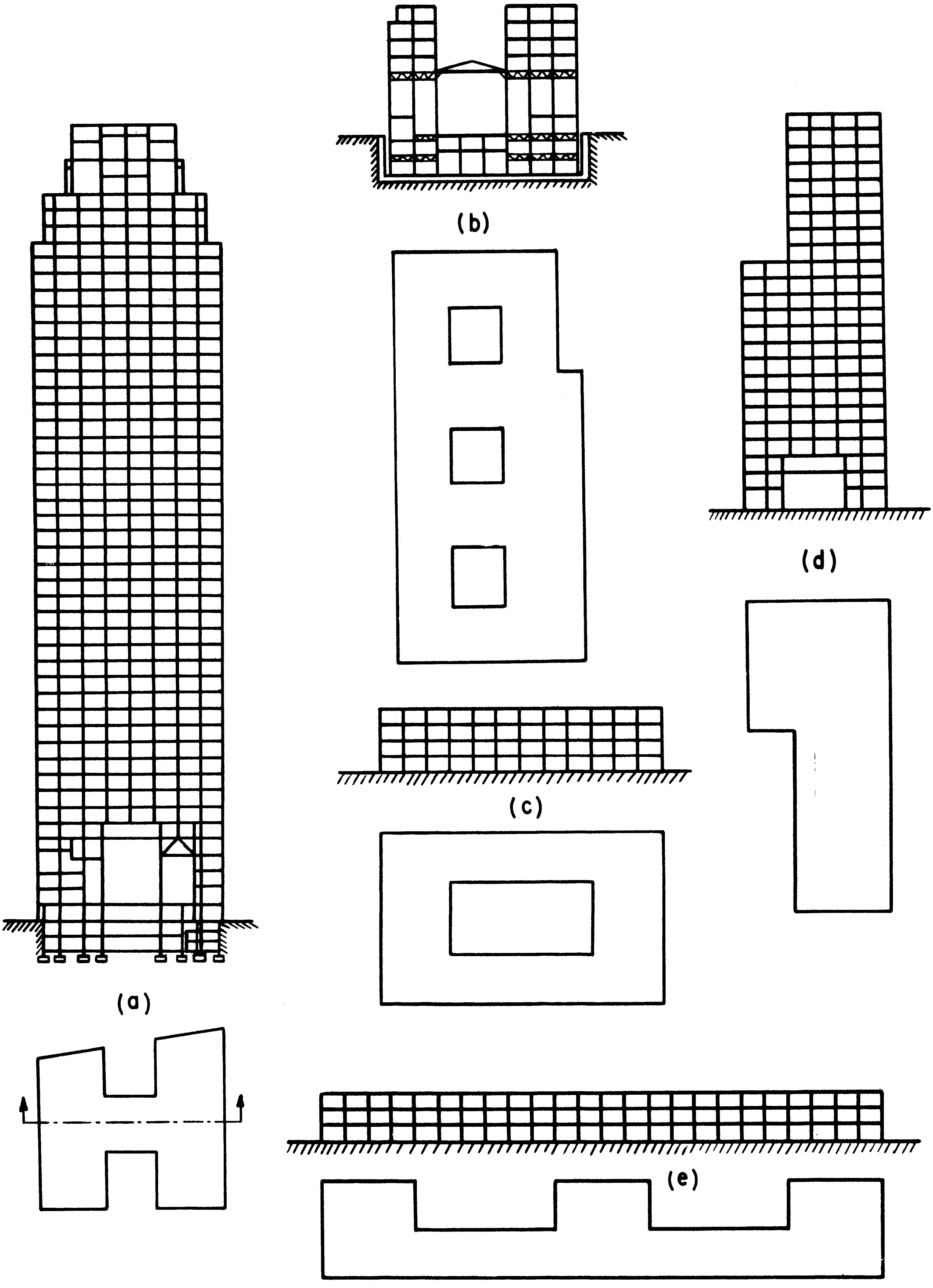
I Tier Buildings (buildings of two or more stories) (Fig. 3.1).

1) Simple framing (flexible or semirigid connections)
2) Some bents wind-braced (either by rigid connections or cross bracing)
3) All bents continuous (all rigid connections)

II Industrial Buildings (Buildings in which there is a large area in which there is no floor between ground level and roof) (Fig. 3.2).

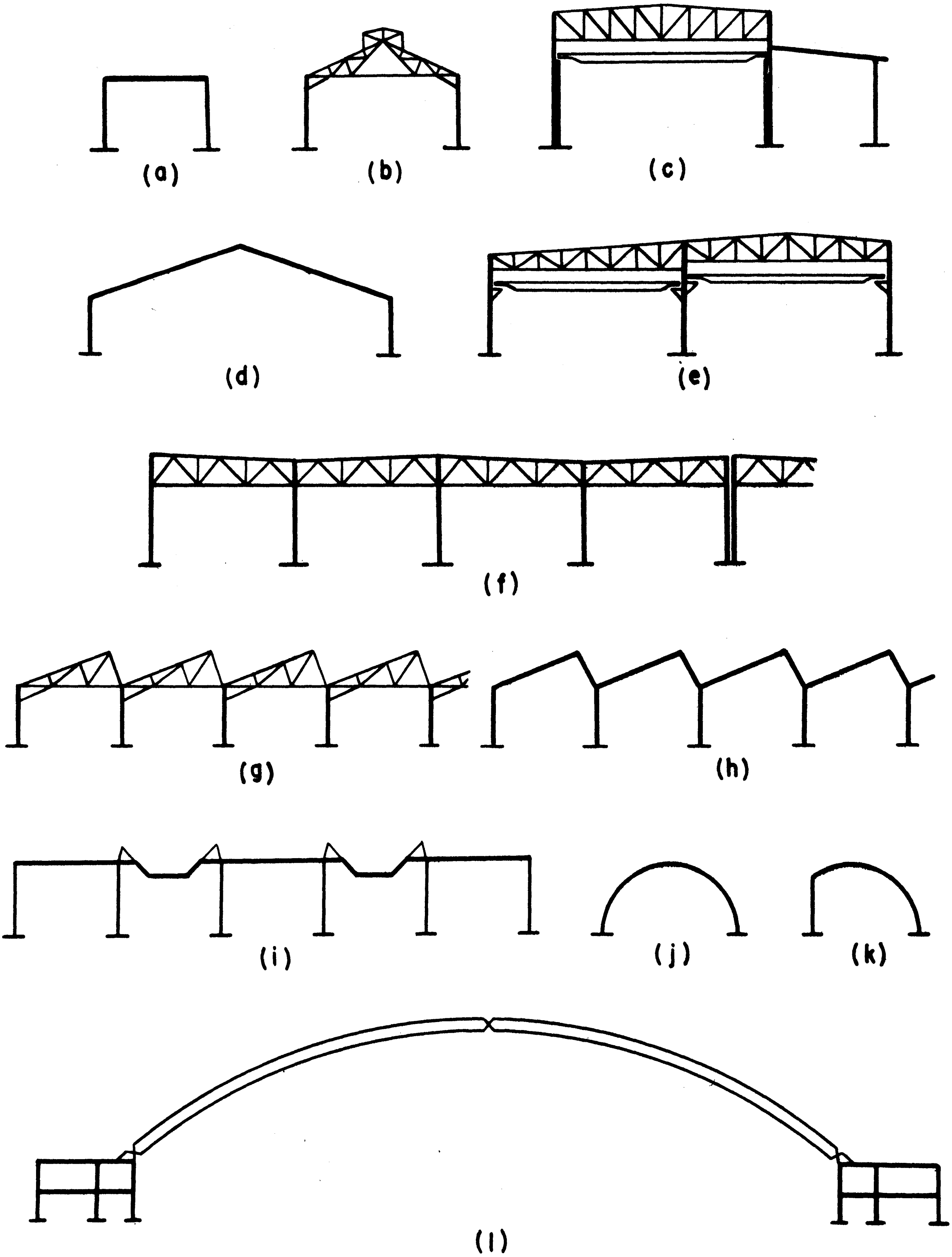
A. One or two bays wide
   1) Simple framing (flexible or semirigid connections)
   2) Continuous
      a) Beam-to-column connection rigid
      b) Saw tooth or arched bents continuous with columns
   3) Roof truss and light columns
   4) Roof truss and heavy columns
   5) Arched roof truss
   6) Arched roof girder

B. More than two bays wide or long low individual or multiple arch spans.
   1) Simple framing (flexible or semirigid connections)
   2) Continuous
      a) Beam-to-column connection rigid
      b) Saw tooth or arched bents continuous with columns



TIER BUILDINGS

FIG. 3.1



INDUSTRIAL BUILDINGS
FIG. 3.2

3) Roof truss and light columns
4) Roof truss and heavy columns
5) Arched roof truss
6) Arched roof girder

III Bridges*
A) Simple girder
B) Simple truss
C) Internally indeterminate truss
D) Continuous girder
E) Continuous truss
F) Cantilever truss
G) Arch girder
H) Arch truss
I) Suspension
J) Pontoon

IV Tanks
A) Elevated tanks
1) Truss tower support (usually cylindrical tank)
2) Frame tower support (usually cylindrical tank)
3) Single tubular support (usually spherical tank)
4) Multiple tubular support (either cylindrical or spherical tank)
5) Combination tubular and truss support
B) Surface tanks
1) Low-pressure tanks
2) Cylindrical pressure vessels
3) Spherical or spheroidal pressure vessels
C) Underground tanks

V Miscellaneous Structures

I Tier Buildings.

This type of structure is characterized both by the tall narrow office building and by the short squat warehouse. These two structures might behave quite differently under blast loading, and for this reason those tier buildings which possess a height-to-width ratio of more than 2 to 1 might be considered as "slender", and those with a height-to-width ratio of less than 2 to 1 as "broad". There is obviously an intermediate

* Since bridges are primarily designed for vertical loads, special attention must be given to behavior under lateral loads and to different types of lateral bracing as used in each of the various bridge types.

range of transition and the suggested division ratio of 2 to 1 is tentative. The following discussion will point out the usual differences in dynamic behavior between "slender" and "broad" tier buildings.

Slender tier buildings with reasonably effective shear resistance deflect laterally in the same way that a cantilever beam deflects transversely, that is, primarily by bending. Broad tier buildings deflect laterally in the same manner that a short stubby beam deflects transversely, that is, the broad building would deflect primarily by shear. In general, the more slender the building the more pronounced are the bending components, and the broader the building the more pronounced are the shearing components. The tier building which has a height-to-width ratio in the region of 2 to 1 deflects laterally by a combination of bending and shear. The general framing used in tier-building construction is one of columns, girders, and beams, all mutually at right angles. The connections of the taller buildings are often designed to take considerable bending stress so that the frame can resist the lateral forces of wind or possibly earthquake. Sometimes, instead of moment-resisting connections, diagonal bracing is used to provide the lateral strength. Very long buildings are usually broken up into several separate buildings in order to allow for displacements due to temperature changes. As long as the separate parts of such a structure are not in contact or do not come in contact during motion, the structure may be considered as several buildings in a row. The foundations of the taller buildings are almost always carried down to bed rock or an equivalent firm and stable base, but the shorter buildings have various footing plans and can therefore have many conditions of foundation restraint with respect either to sliding or vertical motion.

In Fig. 3.1 are line drawings of typical tier-building bents. A slender building and a broad building are shown, along with buildings which are unsymmetrical both in plan and elevation. The tier buildings shown are adopted from actual building designs.

II Industrial Buildings

Buildings which are primarily one story, i.e., those which have a large area in which there is no floor between ground level and roof, are to be considered as industrial buildings, as illustrated in Fig. 3.2. In general, there are two large groups of industrial buildings that might behave dynamically in two very different ways. The one- or two-bay building could be racked by shear under an external lateral load, whereas the building of more than two bays would usually only be "crushed" laterally near the boundary under lateral load and/or vertically in all or part of the remaining area due to the additional "side-on" pressure developed during the passage of the blast wave.

Arches, rigid frames, and the common mill buildings are all included in this group. Although a single-bay long arch building might be racked under combined lateral and vertical load, the bending of the arched members would probably be critical, and "crushing" would therefore be the common mode of failure. For this reason, an exception to the classification scheme should be made in which long low-arch-span structures are considered along with multiple bent industrial buildings of more than 2 bays.

Very long buildings are usually broken up into several separate buildings in order to allow for displacements due to temperature change and/or earthquakes. Each unit has an independent substructure, and, unless they collide due to large deflections, the units may be considered as individual buildings placed one next to another.

The most common industrial building is the mill building, Fig. 3.2, a building with columns and roof trusses. Industrial buildings, in general, have many common construction details. The wall and roof panels are generally very light in weight. There are usually many windows, at least in buildings other than arches. The framework is not fireproofed and is not usually built into the wall or roof. The roof and wall panels merely connect to the outside flange of a purlin or girt, and they in turn connect to the outside flange of the top chord of the roof truss or to the outside flange of the column. Quite often a wall construction is used that eliminates the girts, and in this case the wall is connected directly to the column. The foundations of industrial buildings are usually spread footings but sometimes, because of heavy crane loads and/or poor soil conditions, deep foundations may be required. Columns carrying crane runway brackets are usually fixed at the base; other columns may have base connections equivalent to a hinge.

Fig. 3.2 includes examples of simple and continuous frame construction, common mill buildings with both light and heavy columns, and arched-roof buildings. In all the cases shown, the basic bents may be repeated to make a building many bays wide. Several examples are given to illustrate the possible expansion.

III Bridges.

In the design of bridges there are, in general, three choices as to how the vertical loads may be carried. The girders and trusses, whether simple-span or continuous, carry vertical loads primarily by bending. In arches the loads are carried primarily by compression and depend on horizontal inward thrust developed at arch abutments. In suspension bridges, overhead cable and hanging roadway are the inversion of arch action, producing tension in the cable and depending upon horizontal outward thrust at cable anchorages. In the classification scheme, items A through F under bridges are primarily bending problems. Items G and H are arch problems, and item I is a suspension, or tension, consideration. On the other hand, each bridge type listed in the classification is also a problem in itself; each bridge type may have behavior patterns and possible modes of collapse quite different from any other type.

Although bridges are designed for lateral wind loads, their primary function is to carry the vertical loads imposed by rail or highway traffic; hence, it may be expected that bridges will be particularly vulnerable to lateral blast loads. Differences of structure with regard to lateral bracing should be considered in connection with each bridge type. On the one extreme, the laterally strong through-truss highway bridge with lateral truss bracing in the plane of the top chords together with the great lateral rigidity of a concrete road slab in or near the plane of the lower chords, may have quite different behavior under lateral loads than an ore bridge with rather minimum lateral bracing for wind and braking forces alone.

IV Tanks.

Structures that are used as receptacles for gases, liquids, or possibly bulk solids, such as grain or carbon filter material, will be called tanks. In general, there are three possible locations for tanks, viz., elevated, on the surface, and underground, and each location produces a different dynamic behavior problem. The elevated tank's dynamic action is dependent on the supports for the tank, that is, whether there is a truss tower, frame tower, single pipe, or multiple pipe support; and the classification is subdivided to take this into account. The design pressure and the shape of surface tanks are important variables in determining the strength of that type of tank. The classification lists low pressure tanks, cylindrical pressure vessels, and spherical or spheroidal pressure vessels in order to separate this group of tanks into their various subgroups.

V Miscellaneous Structures.

Any steel structure that might not easily fit into one of the other categories might be considered as a miscellaneous structure. Large derricks, steel dams, and conveyor-belt structures, to list a few, would be included under this heading.

References.

The references for this chapter are not listed; they are a combination of generally available books primarily concerned with construction, together with periodicals of construction news. Some of the examples were taken from actual designs with the knowledge that many existing structures are similar to those depicted.

CHAPTER IV

BEAMS AND GIRDERS

4:1 Introductory Statement.

Beams are defined herein as straight members transmitting moment and shear caused by loads in the transverse direction only. A girder is a large beam, often the beam that supports other beams, and is usually built up of several plates and shapes connected by rivets or welds. The term "plate girder" is applied to large girders with an I-shaped section, usually built up of angles and plates. A "box girder" is an elongated hollow box built up of structural elements such as angles, plates, channels, or even I-sections. The terms "stringer", "girt", and "purlin" are names for small beams that usually support a floorslab, wall, or roof, respectively. In a plate-girder bridge, for example, the floorslab is supported directly on the stringers, the stringers by the floor-beams, and finally the floor-beams by the main girders. As far as their structural action is concerned, stringers, beams, and girders are the same. For the remainder of this section, the term "beam" will be used as a general term for stringers, beams, and girders.

Beams are sometimes termed "fixed-end", "simply supported", "cantilever", etc., depending on the condition of restraint at the ends. The condition of restraint at the end of a beam may be defined with reference to the restraint against (a) rotation about the x, y, or z axis (see Fig. 4.1), and (b) displacement in the x, y, or z direction. Some commonly assumed conditions of restraint are shown in Fig. 4.2. A free end (4.2a) is one with neither displacement nor rotation restraint. A simple support (4.2b) provides no restraint against rotation but is fixed laterally in position. A hinged end (4.2c) is free to rotate about the x axis but is restrained against <u>any</u> displacement and against any rotation about the y axis. A deflecting support (4.2d) is restrained against displacement and (in the case illustrated) is not restrained with respect to rotation. Fig. 4.2e shows a case where the beam end is laterally position-fixed but with respect to rotation is restrained to a degree intermediate between a pinned end and full fixity. A beam-to-column connection may introduce a local region of reduced rotational resistance, in which case it is termed a "semi-fixed" or "semi-rigid" connection. Finally, Fig. 4.2f indicates an end that is fixed in every respect--a condition that can be approached closely but never quite realized in actual practice.

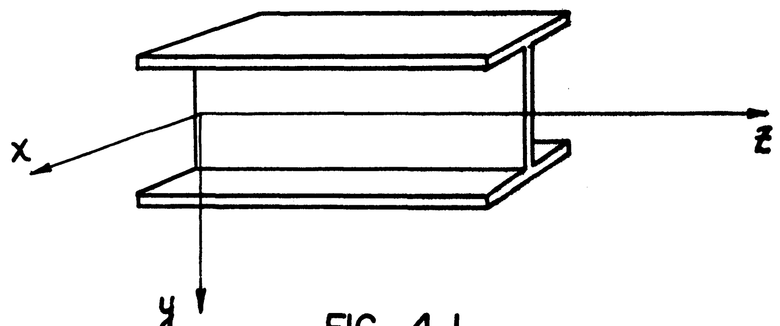


FIG. 4.1

Fig. 4.2 does not present all the possible combinations of restraining condition by any means, and it should also be kept in mind that one type of rotation restraint may exist with respect to the y axis and another with respect to the x axis. Furthermore, there are various types of torsional restraint possible with respect to rotation about the z axis, or parallel axes, but these will not be considered in detail in this preliminary report. Another aspect of restraint is the fact that it may change in degree as the beam or connection yields.

Continuous beams of two or more spans may be thought of as a series of end-connected individual beams, transmitting moment from one beam to another, and providing mutual elastic end restraint with respect to rotation. Interior supports may be simple, hinged, and elastic with respect to either displacement or rotation. In special instances, interior hinges (at points away from the supports) may be used in a series of continuous beam spans; if provided in sufficient number, these may render the structure statically determinate. Such hinges transmit shear, but do not transmit moment, from one beam segment to another.

The strength of beams will be considered under three classifications, namely, bending, torsion, and buckling. Shear failure of short beams will be considered under the bending classification. The problem of beam behavior under shock loads will also be considered.

Certain factors are common to all problems concerned with the strength of a beam. The shape of the stress-strain curves in tension and compression is of fundamental importance. Simplifications and approximations have been widely used in analyzing the plastic behavior of steel beams. The theory based on uniformly distributed yielding at a constant yield-stress level (Fig. 2.1), neglecting strain-hardening, is sometimes referred to as the simple plastic theory. Although this theory has proved reasonably satisfactory for the prediction of the strength of simple spans, the greater and more localized rotation occurring over the supports of continuous beams requires the consideration of strain-hardening in such cases.

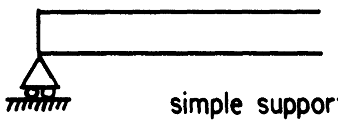
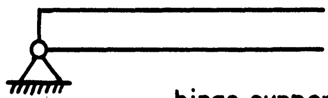
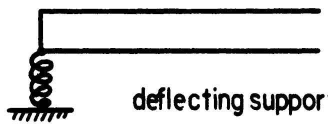
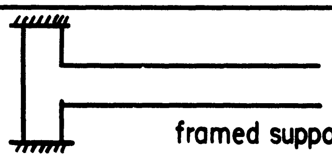
		rotation	lateral (x or y) displacement	longitudinal (z) displacement
a	 free end	free	free	free
b	 simple support	free	fixed	free
c	 hinge support	free	fixed	fixed
d	 deflecting support	free	partial restraint	partial restraint
e	 framed support	partial restraint	fixed	partial restraint
f	 fixed end	fixed	fixed	fixed

FIG. 4.2

The cross-sectional shape of the beam is one of several factors influencing its ultimate strength. On the basis of the simple plastic theory, which neglects strain-hardening, a beam will continue to take increasing moment after initial yielding occurs in the extreme fibers, approaching a limiting value, sometimes termed the "hinge-moment", when yielding that started at the extreme fibers penetrates to the neutral axis of the beam. The ratio of the hinge-moment to the moment at initial yield is sometimes termed the "shape-factor" of the beam. A wide-flange structural section usually has a shape-factor between 1.10 and 1.20. More compact sections have larger shape-factors; for example, the rectangular section has a factor of 1.50. Local and/or lateral buckling frequently will occur before the additional plastic reserve strength is realized.

The cross-sectional shape also determines the sectional properties: the principal moments of inertia, the orientation of the principal axes, the location of the shear center, which is the axis through which the plane of the loads must act if bending is to take place without twist, etc.

The width-thickness ratios and arrangement of the component parts of a section also affect the elastic or plastic local buckling characteristics --an important factor influencing ultimate strength. Local buckling problems will be discussed in Chapter VIII.

The shapes commonly used for structural steel beams are rolled WF (wide flange) sections and built-up I-sections. Less frequently, rolled channel sections, angle sections, Z-sections, and built-up double-angle sections may be used as beams.

The load type and distribution is another factor influencing the strength of a beam. If the plane of the loads is at an angle to the principal axes of the section, the beam will not bend in the same plane as it is loaded. Such bending is termed "unsymmetrical" or "complex". If the loads are not applied through the "shear center", which may or may not be the same as the centroidal axis, depending on the shape of the section, twist will occur in combination with bending. In general, there may occur a combination of both unsymmetrical bending and twist. The distribution of loads along the beam, considered together with the change in shape of the section along the beam, as in the case of a plate girder with variable-length cover plates, determines the relative amount of material in the plastic range after the yield point is exceeded. This is an influencing factor on the reserve plastic strength of the beam above the loads that cause initial yielding.

The condition of end restraint, with respect to rotation or displacement as discussed earlier in this section, may have a marked effect on the ultimate strength of a beam. In a fixed-end beam, or any individual span of a continuous beam, for example, three plastic hinges must form before collapse occurs. Local and/or lateral buckling may appreciably reduce the magnitude of the plastic hinge moments. Nevertheless, overall reserve strength of the structure in the plastic range is usually much greater in continuous beams than in the case of simple spans. Longitudinal end restraint may also become a factor after the deflections become quite large, in which case the beam may function in part as a suspended cable, resisting greatly increased loads, dependent on the end connection strength.

Residual stress is another factor which may change the load-deflection curve for a beam. Structural shapes, after rolling, cool unevenly. After the temperature of the whole beam drops to room temperature, "residual" stresses remain in the beam. These may have a magnitude greater than half the yield point of the material. Residual stress may also be introduced by cold-straightening after rolling, by cutting, or by welding.

Built-up beams may behave differently from beams of rolled sections. Built-up sections consist usually of several smaller sections, connected by rivets or welds. The size, spacing, and arrangement of the rivets or welds may affect the behavior of the beam.

Encasement is also an important factor which may change the behavior of the beam. For fireproofing, corrosion resistance, or other reasons, steel members are sometimes encased in concrete. The concrete encasement usually increases the strength of the member, especially in the initial stages of loading when cracks have not yet formed in the concrete.

Structural beams are not commonly designed to carry torsional moments, but torsion, if present, may be important because of the inherent torsional weakness of open sections.

Buckling is a very important type of structural behavior, especially in the investigation of the ultimate strength of structures. Buckling may occur in a slender structure without exceeding the yield point of the material, or, as is most frequently the case (since structures are usually designed so as not to buckle elastically), the buckling may occur after the proportional limit is passed. In a structure wherein all or part of the primary load-carrying stress is compression, the resistance to deflection in a direction normal to that of the principal compressive stress may decrease to zero, thereby making possible different deflected configurations at the same load. In the case of a beam, buckling may occur if the beam is loaded so as to bend about its axis of <u>greatest</u> bending strength. A critical load may be reached wherein the stiffness of the beam with respect to lateral bending about its axis of <u>least</u> bending strength, combined with torsion, may be reduced to zero. Lateral buckling, as this is called, may also be combined with local buckling of the compression flange. Local buckling in the web of a beam may be induced by flange buckling or may be caused by shear (diagonal compression) in the web, or by compression under concentrated loads, or above points of support. As a buckling load is approached, deflections normal to the direction of the compressive stress appear and rapidly increase with an increasing rate as the buckling load is approached.

Lateral-torsional buckling of beams will be included in this section, while the local buckling of plate elements of beams will be discussed in Chapter VIII.

Under shock loading, the strength of a beam may be modified because of the strain-rate as the yield range is entered. Since primary interest in this review is on loads that cause complete failure, elastic response to shock and the requisite accompanying study of beam vibrations, while not of great importance, is of some interest in the study of behavior up to the transition to the plastic range.

4:2 Survey of Theory and Experiment.

Analytical and experimental investigations on the strength of beams are plentiful, but few reviews or surveys can be found. About 300 references have been selected for further study in writing the final report, or for use by any research project that is set up. Of these, 70 references have been selected for this section of the preliminary report, covering the four important divisions of the subject matter of this section: bending, torsion, lateral buckling, and dynamic effects.

The recently revised book by Nadai (4.47), in Chapter 22, discusses the plastic bending of a beam. In Chapter 21, plastic torsion of a round solid bar is discussed; and in Chapter 35, experimental representation of plastic torsion is discussed for idealized stress-strain diagrams. Baker (4.5) reviewed the recent investigations of beams and frames in the plastic range, including the studies of beam strength in bending and lateral buckling; plastic torsion has not been included. Yang, Beedle, and Johnston (4.70) have reviewed the various factors affecting the plastic strength of wide-flange rolled beams. A bibliography on box beams has been prepared by Madsen (4.39), including information on ultimate strength, torsion, dynamic properties, buckling, and tests. The beam tests at Cornell University (4.11) included almost all kinds of failure possible for beams tested under static loads. This investigation is still in progress. The current investigations of various American universities and colleges are covered by the Review and Directory of the American Society for Engineering Education (4.2). At the University of Adelaide, South Australia, there are unpublished reports on their current research on steel beams (4.59).

Referring now to more specific subjects, plastic bending has been thoroughly investigated. Roderick and Phillips (4.54) present a review, including test results, on the behavior of beams in plastic bending. Roderick

(4.53) recently published test results of rolled steel joists in an investigation of their plastic behavior. Hrennikoff (4.31) discussed the plastic bending of structural I-beams, using tension stress-strain curves for both compression and tension, and compared the results with the analysis according to simple plastic theory. Horowicz (4.30) proposed a numerical method for predicting the resisting moment of any cross section with any stress-strain curve. He also suggested that, for unsymmetrical sections and for materials having different stress-strain curves in tension and compression, good approximations may be obtained by assuming that the neutral axis passes through the centroid of the cross section. The extensive test results of simple and continuous beams by Maier-Leibnitz are typified by two papers (4.40, 4.41). The effects of residual stresses, holes, welding, imperfections, etc. on the strength and behavior of I-beams in pure bending have been investigated experimentally by Dawance (4.12) in France. Experimental investigations of commercial lightweight sections under bending have been made by Winter (4.68, 4.11), by Maugh (4.22), and by Mayrose (4.43). The plastic behavior of the wide-flange beams has been studied experimentally, with consideration of the effect of residual stress, by Luxion and Johnston (4.37). Plastic bending has been studied by Fritsche (4.21) theoretically and by Kollbrunner (4.34). Complex bending in the plastic range ,i.e., bending not restricted to a principal plane, has been studied recently by Aghabian and Popov (4.1) with respect to beams of rectangular cross section, in pure bending, and in comparison with test results of steel beams. Williams (4.66) has analyzed beams of rectangular, I, and angle section in complex bending, using an average of the idealized tension and compression stress-strain curves, including the effect of upper yielding point. Williams also presents some experimental results on plastic bending tests of aluminum beams. The effect of shear on the plastic behavior of beams of rectangular cross section in bending has been studied by Horne (4.29) in England. Horne also investigated both experimentally and theoretically (4.27) the effect of strain-hardening on the equalization of moments in the simple plastic theory for beams of rectangular and I-section. The effect of residual stresses on the plastic behavior of beams has been investigated, both theoretically and experimentally, by Baker and Horne (4.6) in England and by Yang, Beedle, and Johnston (4.70) in America. Lazard reported recently the results of a series of tests of I-girders built up by welding (4.36). Beams of I-section with reduced web area have been investigated experimentally by Mount and Welch (4.46) for plugged holes with concentrated load at the center of the span, and by Wendt and Withey (4.65) for truss-shaped web with a uniform load or two concentrated loads symmetrically placed about the center of the span. Behavior of composite I-beams has been studied theoretically as well as experimentally by Siess (4.55) and by Viest (4.60). Ashton (4.4) presents test results to show that prestretching increases the strength of steel beams. Richards has reported (4.51) on an experimental investigation of plastic bending of rectangular beams on elastic foundations. Symonds has reviewed methods for the plastic analysis of continuous beams or frames in recent reports (4.56, 4.57). The fundamental assumptions on which the simple plastic theory is based have been studied by Yang (4.69) theoretically and experimentally, including the effect of end restraint on the structural behavior in plastic range. The ultimate strength of the continuous beam has also been studied in other countries: in Germany by Vogt (4.62), Gruening and Kohl (4.22), and Eisenmann (4.17), who considered fractural failure; and in England, experimentally by Horne (4.26). Effect of encasement on the strength of structural beams has been investigated experimentally in England, Canada, and in this country at Ohio State University (4.44). Current research is being carried on at several universities (4.2). Research on unsymmetrical bending of beams in the plastic range is being carried on at the Civil Engineering Department of the University of California (p. 14, 4.2), and at the Department of Theoretical and Applied Mechanics of the University of Illinois (p. 62, 4.2). Research on inelastic bending of metal beams having various cross sections, including the effects of time and temperature, and research on the influence of inelastic deformation on the load-carrying capacity of metal members are under way at the T.A.M. department of the University of Illinois (p. 62, 4.2). Research on the properties of structural members in the inelastic range has also been started at the Department of Engineering Mechanics of the University of Texas (p. 178, 4.2).

There is relatively less information on plastic torsion than on plastic bending. Plastic torsion of thin-walled closed cylinders has been discussed in a recent paper by Handelman (4.23), theoretically for any shear stress-strain relations. De Silva (4.14) studied the plastic torsion of a square section by membrane analogy. Torsinal strength of a stiffened D-section has been studied experimentally by Kavanaugh (4.32). Lyse and Johnston (4.38) have studied the elastic torsion of structural I- and wide-flange sections with tests which were carried into the plastic range. Christopherson (4.10) made a theoretical investigation of plastic torsion in an I-beam. Torsion of plate girders has been thoroughly studied (analytically and experimentally) by Chang and Johnston (4.8), who considered the stress distribution, stiffness, and the strength of bolted, riveted, and welded plate girders under uniform twisting moment. An extension of this project to consider combined bending and torsion is under way at Lehigh and Swarthmore.

In 1932, Proctor published a paper on laterally unsupported beams (4.50), in which he reviewed both the theoretical and experimental investigations on lateral buckling of beams. Recently Bleich made a critical review of the buckling strength of metal structures (4.7), in which Chapter 4 on the lateral buckling of beams covers most of the important investigations before 1948. The theory of lateral buckling of beams in the elastic range has been discussed for various loading and restraint conditions in Chapter 5 of Timoshenko's book on the theory of elastic stability (4.58). The ultimate strength of slender beams has been investigated by Winter (4.67) for various loading and end restraints. The strength of rolled beams and plate girders in lateral buckling has been investigated by deVries (4.15), with special attention to design specifications. Engel (4.18) reported a theory of elastic lateral buckling of bars of thin-walled open cross section having a single axis of symmetry, with results of tests on cantilever angle sections. Flint (4.20) considered the stability of I-beams without lateral support, and later (4.19) the effect on the buckling strength of various restraints at the end or in the span. In both papers Flint compared theoretical results with the tests. The lateral buckling of channel and Z-beams subjected to pure bending has been studied by Hill (4.24) with experimental results on aluminum alloy sections. Moore (4.45) made several series of tests on I-beams which failed by lateral buckling, including the effect of end restraints. The lateral buckling of beams in the plastic range has been investigated both theoretically and experimentally for rectangular sections by Neal (4.48) and theoretically for I-sections by Horne (4.28). Compression flanges of large plate girders have been studied theoretically by Kindler (4.33). The lateral buckling of beams in the plastic range has also been studied theoretically by Wang (4.63), and theoretically and experimentally by Yang (4.69). Current research on lateral buckling of beams is under way at the Civil Engineering Department of the University of Washington (p. 196, 4.2). At the University of Illinois the numerical method is being applied to the analysis of various buckling problems of beams (p. 60, 4.2).

Investigations on impact prior to 1937 have been well reviewed by Arnold (4.3), considering only the information of impact on simply supported beams. Shock transmission in beams has been discussed theoretically by Robinson (4.52) with special attention to aircraft design. Recently Duwez, Clark, and Bohnenblust (4.16) studied impact behavior of long beams in the plastic range and compared the computed deflections with tests results on steel and copper beams. An experimental study of lateral motions of a beam under controlled impact conditions has been made by Vigness (4.61), and his results compared with those of infinite-beam and finite-beam theories. Impact effect on a continuous beam has been studied theoretically by Hoppmann (4.25). Chenoweth (4.9) has made a series of tests to compare the effects of impact and explosive loads upon a simply supported beam of I-section stressed beyond the elastic limit. The behavior of structural steel beams under impulse loads is being studied experimentally at Massachusetts Institute of Technology (4.49 and p. 85, 4.2). The effect of rotary inertia and shear deformation on impact of long beams has been investigated theoretically by Dengler and Goland (4.13), and further numerical evaluations with experimental verification indicated. Welter and Bukalski (4.64) discuss the influence of the rate of deformation upon the bending strength of metal beams. Static and impact tests resulting in both brittle and ductile failure of plain and welded steel beams at room and subzero temperatures have been made by Krefeld and others (4.35).

4:3 Recommendations for Research.

On the basis of the foregoing review of existing information and current investigations, research projects are herein recommended. In each of these, emphasis should be placed on the study of behavior at the ultimate load and the determination of complete load-deflection curves throughout the test history. The first proposal is for a specific study of special effects resulting from large deflections, but the same effects should be studied in all the other projects, insofar as possible. To differentiate the project numbers from the subsection numbers, each number will be preceded by the letters RP, signifying "research proposal". Usually, in connection with each subject, a certain amount of work has already been carried out, as cited in the review, and is available in the card file at the University of Michigan. It is assumed that a study of the literature will precede other work on each project.

RP 4.1--<u>Restraint and participation effects of adjacent framing on the strength of beams undergoing large plastic deflections</u>.

1) A study of the strengthening effect of standard types of roof and wall construction on the supporting purlins and girts. Tests of complete panels will be required.
2) A study of possible cable or "catenary" action of a beam that is one of a series of contiguous beams framing end-on-end in successive panels. The additional resistance to lateral loads that may result from catenary action will depend on the strength of the end connections in tension, the degree to which adjacent framing is position-fixed, and the ability of the beam to withstand large deflection without fracture.
3) A study of the effectiveness of lateral bracing of various types, including lateral bracing provided by floor, roof, or wall units, in preventing twisting, lateral buckling, or bending, in unsymmetrical bending or bending where the plane of the loads and/or supports does not coincide with the shear center. Some of the work under this proposal could be correlated or superceded by work under other projects as proposed herein, RP 4.2 and RP 4.4.

RP 4.2--<u>Strength and behavior of structural beams in complex bending under concentrated and uniform loads</u>. When a structure is subjected to heavy lateral (as distinguished from vertical) loads, some of the lateral forces are transmitted to the beams. The amount of force transmitted to the beam depends upon the relative rigidity of the columns and floor members (beams and slabs), and also on the location of the beam. These lateral forces, usually in the direction perpendicular to the strong direction of the beam, in combination with the vertical forces, cause "complex" or "unsymmetrical" bending. Complex bending of beams in the plastic range has been studied theoretically and experimentally to a very limited extent, for pure bending, but little information has been obtained for concentrated and uniform loads. The following outline is suggested:

1) Theoretical and experimental study of the behavior in complex bending of structural beams (I-, wide-flange, channel- and angle-beams) emphasizing the plastic range.
2) Effect of various end restraints simulating actual design practice.
3) Effect of various types of lateral support, provided by various typical roof and wall framing arrangements. This phase should be coordinated with RP 4.1.

RP 4.3--Experimental investigation of lateral buckling of wide-flange-beams in plastic range. Considerable information has been obtained on elastic lateral buckling and plastic local buckling of beams, but little information is available on plastic lateral buckling of beams. In the usual design range, failure by lateral buckling is most apt to occur after the yield stress has been reached at the most stressed point of the beam. Horne (4.28) has published a theoretical method of analysis recently but little experimental confirmation has been given. The following outline is suggested:

1) Tests of standard I-beams, wide-flange beams, and plate girders, designed to fail of lateral buckling in the plastic range.
2) Study of the post-buckling behavior.
3) Effect of loading conditions - pure bending, concentrated load, and uniform load.
4) Effect of end restraint.
5) Effect of lower-flange restraint.
6) Effect of residual stress.
7) Examination of Horne's theory and other theories that may be available with modification or new methods if necessary.

RP 4.4--Strength and behavior of steel structural beams under impulsive loading. In reviewing the information on the dynamic behavior of beams, it is discovered that limited information is available which can be used for practical cases. The arbitrary increase of yield level applied to static test results is of questionable validity. A systematic investigation should be made in relation to those beams and framing arrangements which are actually used in construction. It is suggested that the following items be included in such a project or projects.

1) Theoretical study of the rate of deformation in structural beams under impulsive load with consideration of various end conditions and various load-time curves consistent with the load-transmission-time curves expected from usual wall and roof construction.
2) Theoretical and experimental investigation of strength and behavior of beams with various rates of deformation.
3) Tests of structural steel beams under controlled load-time pulses.
4) Correlation of this investigation with RP 4.1, RP 4.2, and RP 4.3, with spot checks, at least, to study complex bending and local and lateral buckling, under impulsive loadings.

RP 4.5--Strength and behavior of plate girder beams with variable moment of inertia. In larger beams, such as riveted or welded plate girders, or rolled sections with added welded cover plates, the conditions of end and lateral restraint as found in practice will be somewhat different from the case of shorter beams. In addition, the behavior will be modified because of the variable moment of inertia and more uniform distribution of highly stressed material. The differences to be expected in the larger girders, as compared with the analyses and tests of the smaller beams, as outlined in the four preceding proposals, should be reviewed. Additional tests and theoretical studies, with emphasis on numerical methods of analysis, should be carried out subsequent to the completion of the major part of the first four projects that have been proposed. These plate girder tests will be costly and should be limited insofar as possible to a verification--if such exists--of extrapolated predictions resulting from the investigations on smaller beams.

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CHAPTER V

ROOF, WALL, AND FLOOR PANELS

5:1 Introductory Statement.

In framed building construction the roof, wall, and floor panels are incidental elements that transmit loads to the structural load-carrying frame of the building. Temporarily, at least, the failure of such panels will reduce the blast load transmitted to the frame and, at the same time, may lower the resistance level. It is therefore appropriate in this report to consider the load-deflection characteristics of roof, wall, and floor panels.

In ordinary construction, roof and floor panels are supported on purlins or stringers, which in turn are supported by columns. For small industrial buildings, roofs are sometimes directly supported on walls; such construction being referred to as "wall-bearing", in which case wall panels become primary strength elements that transmit loads to the foundation. Failure of the wall panels will cause failure of a wall-bearing building.

The strength of steel structures is the principal consideration in this report. However, other materials are commonly used for panels in steel structures such as buildings and highway bridges. Therefore, roof and floor panels made of reinforced concrete and walls made of plain and reinforced concrete, brick, tile, or other nonmetallic material will be discussed in this chapter as well as those made of steel.

Nonmetallic building materials such as plain gypsum, concrete, tile, or brick are rather brittle and, in bending, failure will occur when the deflections are relatively small. Panels made of steel sheeting, on the other hand, may not fail until the deflections are so large that the tension in the middle plane of the panels due to the stretching that accompanies bending is no longer negligible. Reinforced concrete panels, if properly designed, may deflect to a considerable degree before failure is complete; such behavior is determined by the ductility of the steel. These tension forces tend to reduce the deflections and increase the strength of the panel under transverse loading, but they may have an adverse effect on edge-restraining beams due to unbalanced lateral pull that is developed.

Windows or other openings may reduce the strength of a panel and alter load-transmitting characteristics under blast. The location, type shape, and reinforcement, if any, at openings also should be considered.

Walls are connected to girts or columns; roofs and floors are supported on purlins or floor beams. Panels may be continuous over supports. The degree of edge restraint against rotation of a panel varies from zero (simple edge support) to full (fixed edge support) for different edge connections. A panel supported by an edge beam may have a certain rotational restraint that is a function of the torsional characteristics of the supporting beam. In addition, such a panel will be supported elastically with respect to position. In some cases when panels are fixed at one edge and free at the others, they are called "cantilever panels".

It has been mentioned above that tension stresses may amount to a considerable magnitude if there are large deflections due to bending. On the other hand, compression or shear may be introduced in panels that function as structural diaphragms. In the case of wall-bearing construction, when the external vertical load is extraordinarily heavy, the axial compression or shear transmitted to the wall panels may be great enough to cause failure of the wall, by crushing, buckling, or a combination of crushing and buckling. In framed construction, vertical loads are transmitted to the foundation principally through columns; thus wall panels take very little compression in their own plane. For a structure under heavy lateral load, especially for tier buildings in which no wind bracing is provided, axial compression may be introduced to roof and floor panels and shear into the wall panels. Wind bracing may consist of special moment-resisting beam-column connections and/or truss or "x-braced" panels. In buildings without x-bracing, columns are the main members resisting lateral loads, and walls and floor members (beams, girders, roof, floor, and wall panels) act as diaphragms which make the building act as a unit. The distribution of lateral loads among these diaphragms and the columns varies with their relative stiffness. The stiffer the member, the larger the amount of load that is transmitted through it.

In some construction, especially of steel, the thickness of panels is sometimes reduced to such a degree that they have to be "reinforced" by stiffeners to give the required strength. Beams connected to the roof and floor panels and corrugations in sheet-metal roof panels may be considered as special types of reinforcing stiffeners.

Strength of panels, with or without stiffeners, which are made of materials capable of resisting tension, such as metals and reinforced concrete, and which are subjected primarily to compression or shear might properly be discussed in Chapter VIII. This chapter, however, will include reference to all wall, roof, and floor panels in bending, compression or shear. The panels referred to above are all rectangular. Circular panels are rather rare in ordinary buildings and when used for the top or bottom of circular tanks they are constructed of steel or reinforced concrete. Curved tops or bottoms of tanks are to be considered in the chapter on tanks.

Roofs of the shape of a portion of cylinder, dome, or cone are commonly used in some buildings such as the commercial Quonset buildings. These are of a cylindrical shape, having a semicircular cross section; their strength will be discussed in the chapter on industrial buildings.

All the statements outlined above are for static loading. If the rate of loading is very high, the dynamic effect on the structure should be taken into consideration. It is worth while to point out that it is the strain rate, not the rate of loading, that is the primary modifying factor affecting the strength of dynamically loaded structures. The relation between rate of strain and rate of loading varies for different weights and sizes of structure, for different parts of a given structure, and for different load-time pressure relationship.

Dynamic effects in structural members in general have been discussed in Chapter II; therefore, in this chapter, only dynamic behavior of roof, wall, and floor panels is considered.

5:2 Survey of Theory and Experiment.

Considerable work has been done on the subject of this chapter, especially in the theoretical field About 200 references have been selected for further study in writing the final report, or for use by any research project that is set up, but only 48 selected from these will be listed in this preliminary report.

1) Bending strength of panels, rectangular and circular, with the effects of tension in middle plane due to large deflection, openings, various edge restraint, and stiffeners (including the corrugated sheet deck).
2) Compression and shearing strength of panels made of various building materials.
3) Dynamic effect on the strength of panels in 1) and 2).

In the following reviews, the published references will be mentioned first and then the unpublished and current references presented in order.

Timoshenko's book (5.41) gives a good review of the theory of the bent plate, including large deflections in the elastic range. Plastic bending has been discussed. Small-deflection theory gives good results when the deflection is less than the thickness of the plate; large-deflection theory should be used for larger deflections. When the deflection becomes large enough, the stretching of the middle plane of the plate causes tension stress, which is sometimes referred to as "membrane stress". When the deflection further increases, the membrane action becomes the primary resistance to the applied load and the resistance due to bending becomes relatively negligible. Gleyzal has published two papers on steel thin circular plates under pressure (5.10, 5.11). Theoretical methods based on the theory of plasticity have been given for membranes in which the bending resistance is neglected. A test on a mild-steel plate shows that the calculated stresses are quite close to the measured values at the beginning of loading, but about 10 per cent higher when the plate is near failure. Test results on small-scale thin circular plates made of aluminum alloy, stainless steel, or magnesium alloy are also available for clamped edges (5.23).

For reinforced-concrete slabs in bending, a fracture-line theory has been introduced by Ingerslev (5.16), and reviewed recently by Johansen (5.17), for estimating the strength of rectangular reinforced-concrete slabs. Test results on reinforced-concrete rectangular slabs were given by Ingerslev (5.16) and others (5.12, 5.34). Tests of reinforced-concrete floor-slabs on steel girders have been reported by Blirot (5.4) and by Newmark, Siess, and Peckman (5.27). In the Illinois paper (5.27), tests on slabs for skew bridges were also included. Unpublished test results on reinforced-concrete slabs of small-scale I-beam highway bridges are also available at the University of Illinois (5.29). This program involved three types of load between slab and beam. The test slabs were continuous over several beams.

The effect of openings on the bending strength of flat plates has seldom been considered. One paper published by the Taylor Model Basin (5.39) and one current research study at the Civil Engineering Department of the University of Washington are on the effect of openings on the tensile strength of flat plates. These might have some correlation with the problem of openings in bent plates.

A great number of aluminum-alloy rectangular plates have been tested in bending by NACA (5.30). The plates in these tests are usually very thin; therefore, it is possible that the roof and floor panels in building or bridge structures will behave differently. The Taylor Model Basin also has published some test results on small-scale thin rectangular plates, with or without stiffeners (5.7), under hydrostatic pressure. There is considerable information on the strength of corrugated or stiffened flat panels. Notable are the unpublished reports at Cornell University (5.5)*, which provide a wealth of material on studies in which commercial steel roof decks made of sheet metals from several manufacturers have been tested to failure under two concentrated loads. The Aluminum Company of America has published some bending test results of stiffened flat aluminum panels (5.14). Mayrose has a store of unpublished commercial test results on structural corrugated sheets and wall panels (5.21)*. Test results on various types of ship floors have been published by the Bureau of Ships, Navy Department (5.26). Transverse test results on some commercial steel floors have been published by Frankland and Whittemore (5.8) and by Whittemore, et al. (5.47). All these tests of corrugated or stiffened flat panels were taken to failure, or nearly so.

Test results on plain-concrete walls under axial loading, with or without eccentricity, have been published at the University of Illinois (5.24), and similar results are available for ribbed or ordinary plain concrete walls under axial loading and flexure (5.31). Several papers have been published by Stang, et al. (5.35, 5.36, 5.37) and Whittemore, et al. (5.45, 5.46, 5 47, 5.48) on the test results of walls made of bricks or hollow tiles under longitudinal compression, shear or transverse flexure loadings. The references cited are samplings of many similar investigations on all types of wall and panel construction. Buckling tests of walls under compression have also been carried out by Haller on various eccentricities and slenderness ratios (5.13) and by Sedden on thin concrete walls (5.33). There is very little information on wall footings. Early tests were made by Talbot on wall footings of brick, unreinforced concrete and I-beams encased in concrete.

There are two reports (5.28, 5.9) from M.I.T. which give the test results on reinforced-concrete one-way slabs subjected to impulsive load and on the plain- and reinforced-concrete shear walls under static loads. Programs are under way or planned at M.I.T. for further tests on concrete shear walls (5.9). Stiffened panels, tested transversely under side-on noncontact underwater-explosive load, have been reported (5.25), including a theoretical analysis. Ball (5.3) has analyzed a reinforced-concrete slab struck by a bomb at the center. Effects of aerial attacks on roofs have been discussed by McCawley (5.22), by Rizzetto (5.32), and theoretically by Viesser (5.43). A theoretical study of dynamic effects on structures has been made by L'Hermit (5.19) together with results of tests on circular slabs of plain and reinforced concrete.

Current M.I.T. studies on shear walls (5.9) and dynamic tests on masonry wall structures at UCLA (5.42) are also in progress.

* These references, in particular, are examples of many similar reports on file.

5:3 Recommendations for Research.

Early and extensive work at the Bureau of Standards has involved compressive, transverse, impact, and racking strength of building material panels of a great variety of materials. Many tests in bending and compression of lightweight metal roof and wall panels have been conducted in recent years and research projects are now in progress on dynamic loading of panels. In view of these facts it seems obvious that further detailed evaluation and correlation of existing information is desirable prior to embarking on extensive new projects in this field. Such an evaluation will be made as one of the final reports of this project. A test program may be proposed at a later date to fill in gaps in existing information. The further work for the final report of this project will include analyses of existing data to reveal or confirm rules for calculating the ultimate strength and load-deflection characteristics of panels. For example, the experimental information on corrugated roof decks is rather complete at Cornell University. The following will be included:

1) A systematic consideration of panels of various materials such as are commercially used in roof, floor, and wall panels. Circular panels of steel and reinforced concrete as designed for tank covers should be included.

2) Analysis of strength properties affected by various methods of fastening panels to frames and interconnecting successive panels.

3) Effect of large deflections and plastic or brittle behavior.

4) Static test program proposed to fill out information not already on hand. It should be noted that roof panels are in many cases mass-produced and that the field is highly competitive, involving many patents. Existing information, including certain of the reports herein cited, is in some cases available only upon permission of the commercial sponsors of the programs in question.

5) Proposal for dynamic tests and analyses to determine degree of applicability of static test information. The distribution and load-time character of panel reaction should be studied, as this determines the load-time relation for the structural frame supporting the panels.

The following research projects are also suggested at this time to be carried out in parallel with the further study of existing information.

RP 5.1--Steel-framed wall, floor, and roof panels, with and without openings, as shear, compression or bending elements. This project should be coordinated with RP 4.1, which proposes studies of the strengthening effect of wall and roof panels adjacent to steel beams in bending. Special emphasis is to be given to the effect of window or door openings and to the partition or distribution of load between the steel frame and paneling material. Approximate analytical procedures for predicting strength and deformation should be developed, but because of the variation of panel types and the multiplicity of factors affecting the load deformation characteristics it seems probable that an empirical-experimental approach should largely govern the work.

The following factors should be considered:

1) Special attention to load-deflection characteristics at and beyond maximum loads, and to the partition of load between steel frame and panel material at various stages of test.

2) Panels to include adjacent steel framing and framing connections, to be made of various typical materials, both metal and nonmetal, as are used for industrial or tier building roof, wall, and floor construction. Panels with and without window and door openings are to be included, with various size, location, and numbers of such openings.

3) Bending tests under distributed lateral load, shear wall or racking tests, and compressive tests are to be included.

4) Dynamic load tests should be included, sufficient in number to establish correlation with static test results.

RP 5.2--Wall and column footings. Wall and column footings for wall-bearing or framed buildings are usually designed primarily for vertical loads. During lateral blast load, the forces transmitted by the columns and walls to the footings will cause vertical, sliding, and moment force components at the footing. Rotation of footings, especially, will have an important bearing on the ultimate strength of the steel frame of a building. Tests and analyses are proposed with consideration of the following factors:

1) Various combinations and sequences of moment, horizontal, and vertical loading, to simulate expected sequences during blast.

2) Various foundation media.

3) Various depths, sizes, and types of footing.

4) Dynamic load as well as static load tests. Evaluation of variation in behavior as a function of rate of loading.

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CHAPTER VI

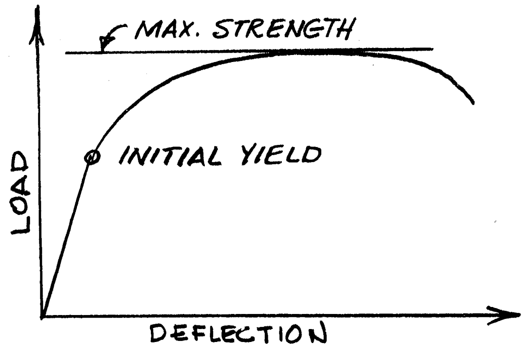
CONNECTIONS

6:1 Introductory Statement.

By "connection" is meant the juncture either between adjoining structural members, or between the ends of a structural member and its supports. In the simplest structure, such as a single beam of rolled section, the ends of the beams are connected by some means to the supports. In the modern all-welded continuous frame the connection is a vital link in the transmission of thrust, shear, and moment from one member to another. The strength of connections may have a very important bearing on the strength of any structure.

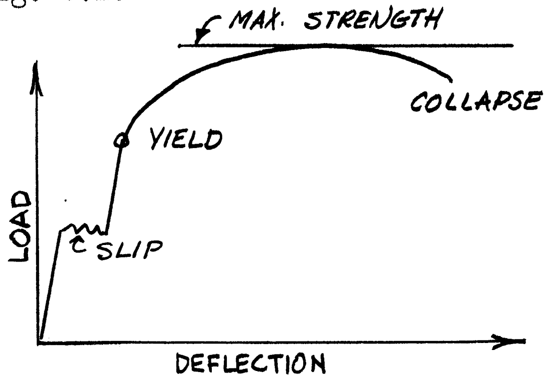
The strength of a structure may be influenced not only by the ultimate strength of a connection, but by the complete pattern of its load-deformation history. In the case of connection behavior to resist moment, the terms "load" and "deformation" should be interpreted as meaning "moment" and "relative rotation between connected parts", respectively. In the elastic range of connection behavior, one may define the rotational stiffness of a connection as the slope of the moment—angle-change curve. The stiffness will be variable in the plastic range. In a beam, if the end connections are flexible, as in a pinned connection, the stiffness is nearly zero, and the beam is "simply supported" at the ends. If the end connections hold the ends of the beam rigidly, the stiffness is effectively complete, and the beam is "fixed" at its ends. As discussed in Chapter IV of this report, both the ordinary strength (based on load when the yield stress is reached at extreme fibers) and the maximum strength of the beam with fixed ends are greater than those of the same beam with simply supported ends. For intermediate connection stiffness, in the case of a beam-column connection, the strength of the beam is between that of a fixed-ended beam and a simple beam. Therefore it is always desirable to know the load-deflection curve for a connection, thus describing its complete behavior.

Fig. 6.1 shows a typical load deflection curve for some connections, which is quite similar in characteristics to the load-deflection curve of a structure. However, for some connections, such as riveted ones, the load-deflection curves are of a different type. Fig. 6.2 shows a typical curve of this type. At the beginning of loading the resistance is due to the friction between members, while the bolts are not yet in bearing. When the friction is overcome, the deflection of the connection suddenly increases until the bolts come into bearing. After this "slip", the behavior of the connection is similar to that shown in Fig. 6.1.



Deflection
Fig. 6.1

The foregoing comments apply to ductile behavior such as occurs in well designed steel structures, at normal temperatures, involving first-class materials and workmanship. Under adverse conditions, brittle fracture may occur when the load is still increasing, prior to reaching point B of the load-deflection curve.



Deflection
Fig. 6.2

In this chapter the strength and behavior of connections will be discussed. Different classifications may be made from different points of view. In discussing strength and behavior, a classification according to their structural action (thrust, shear, bending, or combination thereof) and the connecting medium (bolt, pin, rivet, or weld) seems best suited. Reference may be made to Fig. 6.3 for illustrations of some typical connections.

1. Connections transmitting thrust or shear.
   - A. Pin joints: friction is negligible and usually there is only one pin for one joint.
   - B. Lap joints: bolted, riveted, welded.
   - C. Butt joints: bolted, riveted, welded.

2. Connections subjected primarily to shear, or shear combined with bending, either riveted, welded, or bolted, except as noted.
   - A. Beam-column connections (Beams may be connected to either the flange or the web of the columns.)

(1) Seat angle
(2) Seat angle with top reinforcement (plate, angle, bracket, etc.)
(3) Web clip angle (or plate, if welded)
(4) Split T
(5) Direct butt connection (welded only)

B. Beam-girder connections:-riveted, welded, or bolted.

C. Rigid frame knees, riveted or welded (Rigid frame knees are frequently reinforced by adding stiffeners to the web.)
(1) Straight knees
(2) Haunched knees
(3) Curved knees
(4) Beam splices
(5) Column base anchorages
(6) Special connections, such as threaded ends of round-bar tension members.

Some of these connections are shown in Fig. 6.3. These figures are intended to indicate the general types of this classification rather than to give a complete list for all possible connections. Ref. 6.50, for example, summarizes various types of continuous-frame connections.

The strength and ductility of the connecting media (rivets, bolts, pins, welds) must be known under the various conditions of loading before the strength of connections can be predicted. The usual cases of loading are shear, tension, bending, bearing, and their combinations. For example, in bolted butt joints, the bolts are subjected to shear, bearing, and bending; in riveted beam-column connections, the rivets in the vertical leg of the top connecting angle are subjected to tension and shear.

Roller supports will be considered as in the same category as pins, although their behavior is not precisely the same.

If the connecting medium does not fail, the failure may occur in one of the members adjacent to the connection. Some of the ways in which the connected member may fail are as follows:

1. Net section failure, that is, tensile failure through the minimum effective net area that passes through the center of the hole, or holes. For pinned, bolted, or riveted connections of tension members, the net section is one of the weak points. The net section area provides less material available to resist the load, and the presence of the holes causes the stress to be concentrated in the net section near the holes. These factors increase any tendency toward brittle failure.

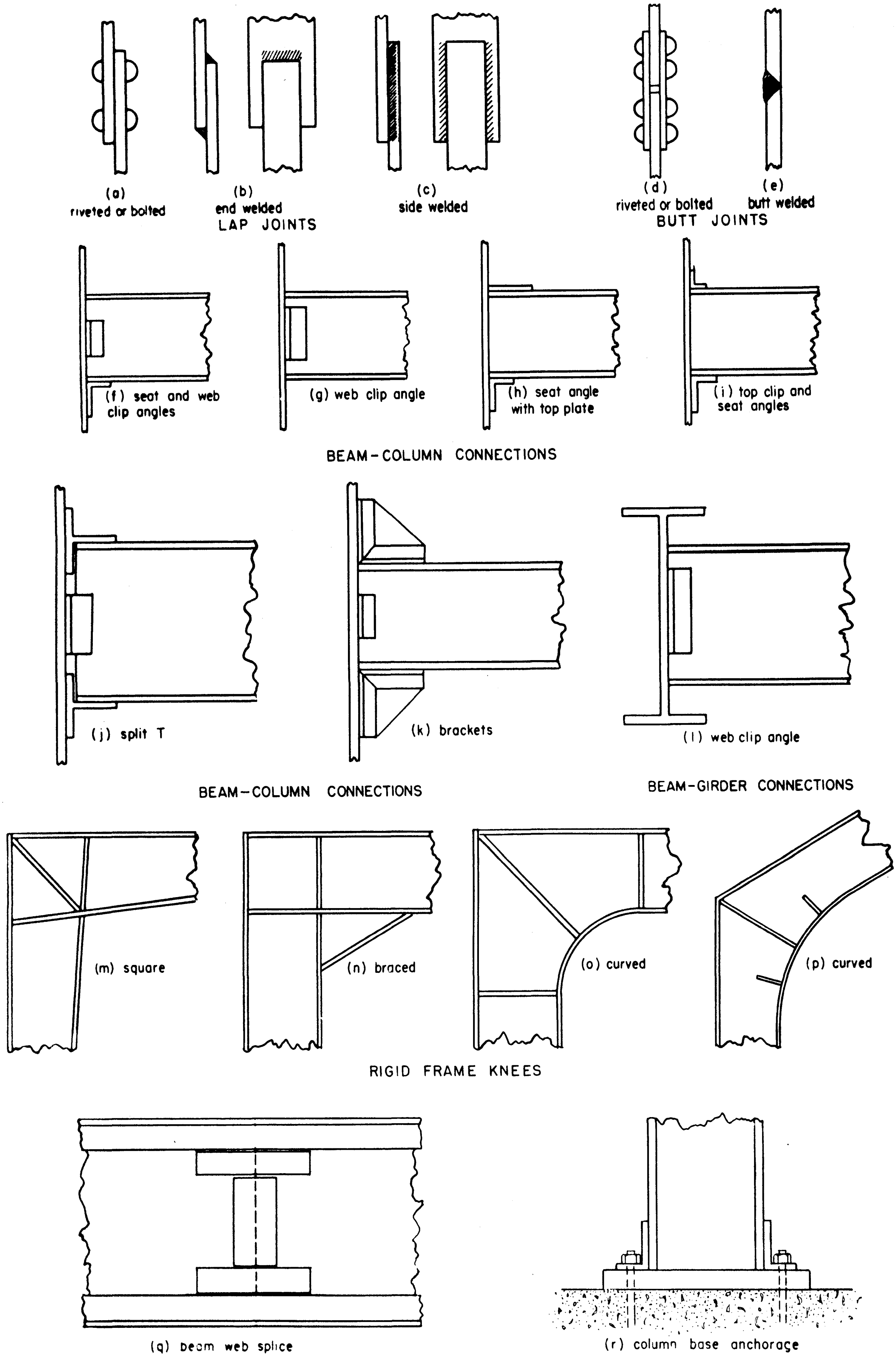


FIG. 6.3 TYPES OF CONNECTIONS

2. Bending failure. As shown in Fig. 6.4, if the distance cd is not large enough, bending stress (the shaded area may be considered as a deep beam, see Fig. 6.4b) may be great enough to cause cracks on the tension side at d. This kind of failure is also obtained in tests of pinned connections.

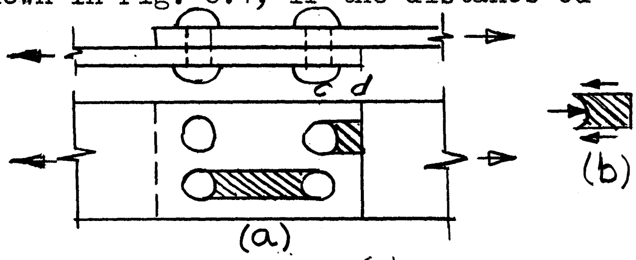


Fig. 6.4

3. Dishing failure. For the same region (c-d of Fig. 6.4), in the case of relatively thin plates, failure may occur by buckling or dishing as shown in Fig. 6.5. Consideration of this type of failure is considered essential in the design of pin-connected plates.

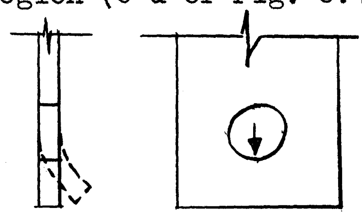


Fig. 6.5

4. Shearing failure. The shaded areas shown in Fig. 6.4 may be sheared off along two planes as shown in Fig. 6.4b.

5. Failure of top and seat angles. In this type of beam-column connection, the top and seat angles will yield as deformation increases, and fracture may ultimately occur.

6. Buckling failure. (a) Wind bracing beam-column connections: As shown in Fig. 6.3k when the connection is subject to a moment, one of the brackets is in compression. Although these brackets may be reinforced by stiffeners, the compression may be large enough to cause the bracket to buckle laterally.

(b) Flange of the continuous-frame knees: For continuous-frame knees as shown in Fig. 6.3m to p, very similar to the lower bracket of the beam-column connections, the lower flange may buckle locally and/or laterally when subjected to compression.

(c) Web of the continuous-frame knees.

7. Yield in web due to shear force. In square connection (Fig. 6.3m), shear yield in the knee may cause large relative rotations to develop between the ends of the framing members.

8. Reversal of the loading. Heavy lateral loads may cause the reversal of loading on some connections. The behavior of the connections under reversal of loading, in general, is not the same, especially if prior yield has occurred in one particular sense.

9. Column base anchorages. When the column bases are subjected to bending, the anchor bolts on the tension side may fail in tension or load, permitting column base rotation, and decreasing the effectiveness of the column with respect to lateral load. Under large lateral load there is also the possibility of direct shear failure of anchor bolts.

The yielding or fracture of some part of a connection does not necessarily coincide with the overall failure of the connection. In such case, a redistribution of stress in the connection and in adjacent structural members may occur. For example, the failure of one rivet in a connection of many rivets, or the yielding of the top or seat angle in a beam-column connection does not mean that there will be a sudden decrease in load-carrying capacity.

6.2 Survey of Theory and Experiment.

A large number of papers have been written on steel connections and most of these include test results. About 400 references have been selected for further study in writing the final report, but only 55 will be listed in this preliminary report. These selected references cover both theoretical and experimental work. Up to the present time, most of the theoretical investigations have considered only elastic behavior even though the tests frequently have been carried to collapse.

The published information on riveted connections prior to 1944 has been well covered in de Jonge's review and bibliography (6.15). Although it does not emphasize the failure of the connections, this book provides an excellent review of both theoretical and experimental investigations of riveted joints, as well as information on combined riveted and welded joints. The recently developed Research Council on Riveted and Bolted Structural Joints has issued a progress report (6.42) which discusses current test programs and results of tests on the static strength of riveted and bolted joints subjected primarily to axial forces, the strength of rivets in shear and combined shear and tension, and their fatigue strength. Current investigations on steel connections are being carried on at the University of Adelaide, South Australia (6.52); details of the investigations are not known.

A review of welding and welded joints (6.8) was published in 1931 with special attention to tests, but Rappolt has made a more complete review (6.41) (unpublished) which covered both theoretical and experimental works prior to 1937. Girkman, in 1940, gave a survey of the theory of welded connections (6.22), with emphasis on elastic behavior. Frankl has published recently a paper on tests of welded connections (6.19) in which a critical examination of the literature has been included. Literature on impact tests of welded joints prior to 1936 has been reviewed by Spraragen and Claussen (6.44) and literature on the behavior of welded joints at low temperature prior to 1943 has been studied by Spraragen and Cordovi (6.45).

Reference will now be made to more specialized articles, according to the order outlined in the preliminary statement of this chapter. The strength of connecting media seems well studied. Steinman has derived two empirical formulas for the ultimate strength of bridge pins under shear (6.47) from some very old test results. The strength of bridge pins in bending has been evaluated in tests by the California Department of Public Works (6.10). At the University of Illinois two master's theses (6.25, 6.28) have described experimental investigations of the bearing strength of steel rollers. Tests of rivets are well covered in de Jonge's review (6.15). For example, two papers (abstracts 897 and 971 in 6.15) have been published on the tensile tests of rivets and another (abstract 955 in reference 6.15) on tests of rivets under pure tension and combined tension, shear, and flexure. Lehigh Structural Steel Company has tested bolts in shear or tension (6.37). The effect of clamping force on the strength of bolts has been investigated experimentally by Lenzen (6.38). Large bolts have been tested by Jones (6.34), illustrating the little known type of failure wherein lateral contraction induced stripping of the threads at loads much below their normal ultimate strength. Effect of impact on the tensile strength of bolts was reported in 1938 (6.54).

Structural weld tests under various conditions of loading and for various kinds of welds have been reviewed by Priest (6.40) and Welter and Poly (6.53). The extensive early tests of the American Bureau of Welding (6.58) should be mentioned. Plug and slot welds have been tested by Gibson (6.21). A recent paper by Zschokke and Montandon presents results of an experimental investigation on spot-welds (6.55), which have also been extensively investigated in this country at Rennselaer Polytechnic Institute. Brittle failure of welds has been discussed by Kennedy (6.35).

Dell has made some tests on pin connections (6.16). Many tests in the early part of the century were made on pinned eyebars and may be found in the files of the principal steel companies. An extensive test series to evaluate the strength of pinned plates in pin connections is due to Johnston (6.31), who classified the failure of pinned plates into three types, namely failure at side (net section), failure at bottom (bending), and failure by dishing (buckling). Empirical formulas based on test results permit a prediction of maximum load accompanying each type of failure.

Riveted connections subjected primarily to axial forces are well covered in de Jonge's review (6.15) prior to 1945. McKibben (abstract 401, 6.15) gives test results of steel angle tension joints for nine different types of connection. Gayhart (abstract 844, 6.15) presents results of an extensive experimental study on this subject which included details as to the ultimate strength. The effect of rivet pattern on the strength of riveted joints has been investigated by Cozzone (6.14) for lap joints and by Badir (6.2) for butt joints. At the University of Illinois large model tests have been made on riveted and bolted lap joints (6.11) and on riveted butt joints (6.7). Currently, at the same school, investigations by Schutz provide correlation of various test results. Bolted connections in light gage steel members have been tested at the University of Michigan (6.12) and empirical formulas derived for their strength. Dynamic effects in plate connections have been investigated by Jeppesen (6.30) for both riveted and welded joints.

Small-scale welded connections have been tested under tension recently by the Taylor Model Basin (6.51) for various types of joints.

The action of combined riveted and welded connections has been thoroughly studied in Germany. Bierett (abstract 975, 6.15) tested lap joints and Kayser has published several papers on this subject (abstracts 965, 985, 1010, 6.15). In 1933 Hohn (abstract 1036, 6.15) considered the distribution of load between rivets and welds and also the effect of eccentricity.

The connections subjected primarily to bending, or bending combined with shear, present more complicated problems than those transmitting primarily thrust forces. The connecting media in these connections are subjected to complex states of stresses.

Many tests have been made on beam-column connections. Early tests were conducted by the American Bridge Co. (6.56, 6.57). Unstiffened seat-angle connections have been investigated by Columbia University (6.13) and at Lehigh University (6.39) for both riveted and welded angles. This problem has been reviewed by AISC (6.1). Investigators at Lehigh University have tested almost all types of beam-column connections (6.9, 6.24, 6.32, 6.33), with load-deflection curves to complete failure usually determined. Seat-angle connections reinforced by welded top plate were tested by Brandes and Mains (6.9). A very extensive experimental investigation on riveted beam-column connections was carried out by Hechtman and Johnston (6.24), in which various types of connections (as shown in Fig. 6.3g,i,j) were studied, and also by Johnston and Deits (6.32). Johnston and Godfrey (6.33) tested certain typical beam connections with failure due primarily to vertical shear. Various types of riveted wind connections for beam-column have been studied by Wilson and Moore (abstract 662, 6.15). Batho contributed valuable information on the beam-column connection (6.4, 6.5, 6.6), considering both the load-deflection curves of beam-column connections (6.6, 6.5) and the effect of concrete encasement (6.5, 6.4). Batho also originated the "beam-line" method of test interpretation in relating results to frame behavior. Riveted plate girder splices have been studied by Garrelts and Madsen (6,20) and dynamic effects in butt-welded splices by Krefeld and others (6.36). Column splices under transverse loading have been investigated by Edwards and others (6.18). The dynamic behavior of welded T-shape connections has been studied by Hoppmann (6.29).

Another important type of connection subjected primarily to bending is the continuous or "rigid" frame knee. The National Bureau of Standards has made several tests on both square and curved welded knees (6.46). The David Taylor Model Basin tests on various types of rigid frame knees have been reported (6.43) with special attention to the type of failure. In England, Hendry has made tests on welded rigid frame knees of various types (6.26, 6.27) with attention to the effect of different knee details. Another study (6.8) by Hendry has been completed at King's College, but no report has been published. Lehigh University also has made a series of tests on welded rigid frame knees of various types and knee details (6.49, 6.50, 6.60) with theoretical analysis of square knees. Most of the rigid frame knees were tested in compression, i.e., the inner flange in compression, which is the usual case for a portal frame under vertical loading. Under heavy lateral loads, some knees may have tension in the inner flange and compression in the outer. Some of the Lehigh knee connections previously tested in compression were later tested in tension (6.48), providing information applicable when reversed loading occurs.

Riveted column bases have been tested to failure in compression (6.17). Strength of steel anchors in concrete has been investigated experimentally by Graham (6.23), providing information that may be used for predicting the ultimate strength of column base anchorages under bending.

6:3 Recommendations for Research.

As the review in part 2 of this chapter indicates, a great number of tests to ultimate failure have been made on various types of beam-column and continuous-frame connections, but few theoretical investigations consider the behavior in the range of the load-deflection curve near maximum load. Emphasis in these studies has been on design rather than ultimate strength. A systematic theoretical study with the help of existing test results may be useful in coordinating information that can be used to predict the complete behavior of the connections, and this is expected to be an immediate objective now that this preliminary report is completed. Certain projects, nevertheless, appear desirable at this time.

RP 6.1--Theoretical and experimental investigation of the complete behavior of beam-girder, beam-column, and continuous-frame connections.

(1) Tests to date have covered a limited range of connection types. A survey* should be made of types used in this country and in other countries for both industrial and tier buildings, including interior beam-to-column framing and exterior framing in the plane of the walls. Test specimens should be made to cover a spot check of all different types, including types designed for vertical load only as well as moment-resisting connections, somewhat as was done on a more limited scale in the Lehigh investigations (6.32). In applications to tier buildings, riveted, bolted, and welded beam-column connections should be included. Lateral support of walls and floors should be simulated in appropriate cases.

(2) The tests should include both normal and reversed loading, separately to failure, and in sequence. Normal loading is here defined as that producing a negative moment at the connected end of the beam. In sequential loading, the first loading should be carried into the inelastic range to varying degrees, to be followed by reversed loading to failure. The initial load in the sequence should include both normal and reverse cases.

(3) Tests at various deformation rates should be made in sufficient number over a sufficient range of types to establish a relationship with the static test results.

RP 6.2--Strength of column base anchorage under moment and/or lateral load. Few investigations, either theoretical or experimental, have been done on the strength of column base anchorages subjected to shear and bending. Under ordinary loading an adequately designed structure will probably not fail at the column base anchorage; but for very heavy lateral loading, such failure is quite possible.

A research program on this type of connection should include consideration of the following.

(1) Bond of various types of anchors in various qualities of concrete.
(2) Effect of compressive, bending, and shear loads.
(3) Various details of column-base welded or riveted assemblies, including cases where bases are pinned or on rocker shoes.

A proposal to test resistance of wall and column footings as a whole was made as part of Chapter V.

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* A survey of this type is planned as a part of the continuation of the present investigation.

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CHAPTER VII

COLUMNS

7:1 Introductory Statement.

Most commonly the ductile failure of a steel structure is brought about by the collapse of a compression element, of which the best known example is the simple column. A column is a slender structural member which has for its primary purpose the transmission of compression force. It may also carry lateral loads, be eccentrically loaded at the ends, or in some other manner be called upon to carry bending moment and shear in addition to the direct compression force.

The term "buckling"* as used herein refers to the general behavior of a compression member or element when it undergoes marked deflection laterally out of the line or plane of the applied compressive load or loads. Buckling, then, in a general sense, involves the waving or wrinkling of a compression element. A critical buckling load, on the other hand, will refer to a theoretical load in which an idealized** column or compression element reaches a state wherein its resistance to lateral deflection out of the plane or line of the applied compressive loads or load reduces to zero, in which case more than one configuration will represent an equilibrium condition.

The buckling of a column may be primarily an "integral" behavior, in which case the shape of every cross section will remain approximately the same during the failure process. Such buckling will occur, for example, if a solid round bar or compact rectangular bar fails as a column. Column buckling may also be influenced by "local buckling", especially in the case of thin-walled structural shapes. Columns built up of structural shapes often will fail or begin to fail by integral buckling, but plastic local

* See also Section 4:1.

**An idealization as used herein is a set of assumptions defining an imaginary concept of a structural member in a manner that is intended to permit mathematical analysis.

buckling will usually develop as failure progresses. In this chapter attention will be directed to integral buckling problems, and local buckling will be considered in Chapter VIII. Attention here will also be confined to the column as an individual element, with the overall problem of failure of the structure, as brought about by column action, being considered in later Chapters IX, X, XI, XII, XIII, XIV. In considering the overall failure of a structure, special consideration must also be given to the contributing strength (to the column) of adjacent walls, fireproofing, etc., normally not counted upon in the design.

At Lehigh University, where the complete report on this section is now being prepared, a chart* of column modes of failure and influencing factors has been prepared, as follows:

I. MODE OF FAILURE (elastic and inelastic buckling)
- A. Bending
- B. Bending and Torsion
- C. Bending and Local Buckling
- D. Bending, Torsion, and Local Buckling

II. INFLUENCING FACTORS
- A. Type of Member
    1. Cross-sectional form (prismatic members)
    2. Slenderness ratio
    3. Longitudinal variation in cross section
    4. Initial eccentricities and initial curvature
    5. Action of splices
    6. Size of cross section
- B. Type of Structure (columns in trusses and rigid frames)
- C. Boundary Conditions (loads and restraint)
    1. Type and location of loads
        - a. End forces and moments (3 coordinate axes)
        - b. Intermediate loads: lateral and longitudinal
    2. End restraints (3 coordinate axes) and supports
    3. Lateral support
    4. Encasement
    5. Action of end connections
    6. Column footings

*Taken from "Interim Report No. 1, Classification of Column Problems" by Lynn S. Beedle. Fritz Laboratory Report 226.2, Dec, 1951.

D. Mechanical Properties of Material
   1. Type of material (steel)
   2. Variation in material properties throughout member
   3. Effect of fabrication processes on material properties
   4. Residual stress due to fabrication process

E. Time effects
   1. Impact
      a. Blast loading

Jakkula and Stephenson, in their column survey (7.26)* indicate 30 different shapes of cross section, of which the first 25 are types used in steel construction and the last 5 are more common in thin-guage metal construction or aircraft. There has been a trend in recent years to the use of single rolled sections (wide-flange shapes), wherever possible, because of low cost, ease of framing, and general availability. These are used both in bridges and buildings. Additional cross-sectional area may be obtained by the use of welded cover plates that are added to the flanges, a practice common in building construction. The usual shapes of column sections may be summarized as follows:

(1) The single angle.
(2) The tee shape, rolled as such, or cut from a wide-flange beam, or built up out of two angles.
(3) The channel shape, rolled, or built up of plates and angles.
(4) The H-shaped section, rolled as such (wide-flange shape), or built up out of various combinations of plates, channels, angles, or zee bars. These are adaptable to "single-plane" truss construction.
(5) The box or closed section, similarly built up out of plate and rolled section combinations. This section is adaptable to "double-plane" truss construction.
(6) The hollow cylindrical, or pipe cross section, a very effective shape, but not common because of end connection difficulties and higher unit cost as compared with rolled shapes.

With regard to longitudinal details one should consider the use of lacing bars, batten plates, or perforated cover plates as alternate possibilities. Columns may be riveted, welded, or fabricated by a combination of both processes. The spacing of rivets or welds may also affect the strength. The foregoing factors of this paragraph will be considered as a part of Chapter VIII on "Plate Elements of Structural Members."

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*Numbers refer to references at the end of this chapter.

The end conditions of restraint and load in a given column may depend on the type of structure in which it is used. The Column Research Council list* of category groups is an indication of the variety of structures in which the compression member may well be the primary strength-determining element. The list includes, in part, the following: railway bridges, highway bridges, tier buildings, industrial buildings, hangars, derricks, hydraulic structures, shore structures, mobile equipment, and towers.

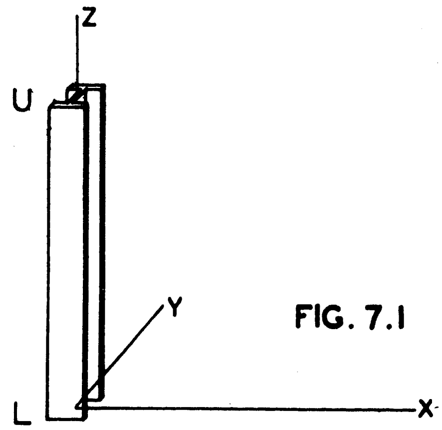

FIG. 7.1

The end conditions of restraint may be specified in reference to the x, y, and z coordinate directions at each end of a column as indicated in Fig. 7.1. The connecting structure may limit the rotation to a very small quantity, not greatly different from zero, at either end and about one or more of these axes. In such a case the column is "rotation-fixed" with respect to the given axis. The end of a column may be free to rotate, which is the case of a free end and is approximately true for the pinned end. If framed to other elastic members, as is usually the case, there will be elastic restraint against rotation, intermediate** between rotation fixation and rotation freedom. The amount of elastic restraint may vary with load if the adjacent framed members are under variable axial load. Independent of rotation freedom or restraint, the column ends may be "position-fixed" or "fixed against side sway" at one or both ends, in either the x or y directions, or both. Alternatively the ends may be relatively free to move in the x or y direction. Prior to buckling, the column itself provides relative position fixation in the longitudinal or z direction between the two ends. If the structure also provides restraint in this direction, the column, in buckling, may continue to carry a constant or somewhat decreased load with disproportionate load increases carried by other parts of the structure. There are thus, at each end, three possibilities of rotational freedom or fixation and two possibilities of position freedom or fixation. In the failure of a large structure, it will usually not be possible for the ends of a column to rotate appreciably about the z axis; hence the column behavior, in the absence of applied end moments or lateral loads, usually may be considered as a problem in a single plane (the xz plane or the yz plane of Fig. 7.1) provided failure does not include variable twist within the column length.

Various types of elastic buckling configurations in a single plane are summarized in Fig. 7.2. The shape of the deflected curve in the plastic

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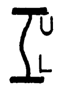
* Given on p. 3 of Sept. 25, 1951, revision "Column Research Council - By-Laws and Rules of Procedure".

** Not true if the tendency toward buckling in the adjacent member causes its restraint to become negative.

range will be similar but with the curvature occurring more or less locally, where "plastic hinges" are developed, while the connecting portions remain relatively straight. (See Fig. 1.1)

type of restraint	buckled form	condition at U		condition at L		Euler length factor "K"
		rotation	lateral position	rotation	lateral position	
1		free	*fixed	free	*fixed	1.0
2		free	*fixed	*fixed	*fixed	0.7
3		*fixed	*fixed	*fixed	*fixed	0.5
4		*fixed	free	*fixed	fixed	1.0
5		free	free	*fixed	*fixed	2.0

* May also be elastically restrained, or, in laboratory tests may be a "flat-end" contact capable of transmitting only compression.

FIG. 7.2

During buckling the columns shown in Fig. 7.2 develop bending moment and (in cases 2, 4, and 5) lateral force components in the lateral directions at one or both ends. Prior to buckling, in the perfectly straight and centrally loaded "ideal" column, these moments and shears would not (ideally) exist. They would exist after buckling, however, because of the restraining action of the adjacent parts of the structure. If end moments and lateral force components are applied, with the "z" force component less than the buckling load the column cannot then reach its theoretical buckling load in the elastic range but may buckle inelastically. End moments and/or lateral (x,y) force components, either at the end or at intermediate locations, might be assumed in an infinite variety of combinations; hence no attempt will be made here to classify such cases, and important special cases will be considered in appropriate later chapters.

7:2 Survey of Theory and Experiment.

Excellent surveys of the voluminous theoretical and experimental work on columns are currently available. Therefore, except in the case of unpublished reports not found in these surveys, only a few published reports are listed as references for this section.

Salmon's great book (7.45) was published thirty years ago, listing 376 references relating to work carried out between 1729 and 1921. These are reviewed and discussed in detail in Part III. A number of new ideas and a great deal of experimental work have been added since this book appeared, but it is a good beginning for any careful study of the problem. Jakkula and Stephenson (7.26) have reviewed the literature to a limited extent between the years of 1921 and 1947 and have summarized the more important experimental work during the same period. The work of ASCE committees in reviewing and conducting tests on columns between 1910 and 1931 (7.1, 7.2, 7.3, 7.4, 7.5) has had a great influence on column design formulas. The Column Research Council recently conducted a world-wide survey (7.8) of both published and unpublished recent theoretical and experimental investigations, summarized through early 1950. Timoshenko (7.55) has provided the principal advanced text covering the whole field, and his review (7.54) of both bending and buckling of thin-walled members of open sections is also of general interest. The Bureau of Ships, through the David Taylor Model Basin, and Frankland and Lienhard, Consulting Engineers of New York, have sponsored a book (7.11) published in January of 1952, wherein the author, the late Dr. Friedrich Bleich,* in words quoted from the foreword, "has assembled all available data on this subject, has appraised it and selected the most useful and applicable methods of analysis for inclusion..to provide this up-to-date treatise on the buckling strength of metal structures."

Considering the column as an individual structural member, the estimation of maximum strength in the light of existing theory is primarily a matter of the probability or statistical analysis of the effect of the uncertainties that are met in materials and fabricated structural members. The theoretical strength formulas for integral buckling of "ideal columns" have been available for many years, but a column, more than any other type of member, is markedly sensitive to minor variations of material properties and imperfections of workmanship. Although Euler** the originator of the first and greatest column formula, did not intend to restrict his formula to elastic behavior, it is generally construed to be so restricted, in which case it is based on the following set of assumptions:

Materials:

(1) Perfect elasticity.
(2) Homogeneity of mechanical properties.
(3) Isotropy of mechanical properties.
(4) No initial internal stresses.***

* Completed and edited by Dr. Hans Bleich, son of the author.

** Euler, Leonard, "Sur la Force des colonnes", Academie des Sciences de Berlin, Mem. 13:252 (1757). English translation by J.A. van den Broek, Am J Phy 15:315 (1947).

***Not required for Euler's theory, but included for later discussion.

Member, and conditions of restraint:

(5) Perfectly straight, and of uniform section.

(6) Load resultant and centroidal axis are coincident prior to buckling.

(7) Ends, if pinned, are frictionless, or if fixed, are perfectly fixed.

Buckling behavior;

(8) Ordinary bending theory for small deflection holds.

(9) Shear may be neglected

(10) Cross-sectional shape same before and after buckling.

Because of assumption (8) the Euler formula does not give a true maximum load, but it is a satisfactory approximation if very large deflections of very slender columns are omitted from consideration. It may be written in terms of buckling load P_e and cross sectional area A:

$$\frac{P_e}{A} = \frac{\pi^2 E}{\left[\frac{(KL)}{r}\right]^2} \qquad (7.1)$$

where E is Young's Modulus, L is the column length, r is the radius of gyration of the cross section in the plane of buckling, and K is a length coefficient as indicated in Fig. 7.2.

Starting from this equation, by use of the theoretical and/or statistical analyses, as may be required, the complete survey of the ten listed assumptions will permit an evaluation of the probable strength range of a column of a given type. This is much easier to say than to do. The assumption of a perfectly elastic material led to divergences from actual experience that unjustly discredited Euler's work for many years. Engesser* in 1889 and von Karman* in 1910 contributed greatly to the adaptation of the Euler Theory to inelastic behavior, with further illumination provided by F. R. Shanley (7.48) in 1947. Utilizing the experimentally determined stress-strain curve of the material, and retaining the remaining nine assumptions, (with minor revisions on account of inelastic behavior), Eq. 7.1 is modified by substituting the slope of the stress strain curve (E_t), in place of E, for the stress corresponding to P/A in the column. As brought out by Shanley, the Euler formula, so modified, defines the axial "tangent modulus load" at which the column may start to bend. For structural sections, this load will in some cases be very near the buckling load and may be used as a basis for design formulas. The actual maximum strength will be somewhere between the tangent modulus load and the "double-modulus" load.

*See Ref. (7.47) for Shanley's account of this early development.

The early development of the "double modulus" theory also has been described by Shanley (7.47). The double-modulus theory is based on the concept that during buckling the strain on the convex side of the column is positive (tensile) and hence is related to stress by the elastic modulus. This would be the case if the column remained perfectly straight up to the "double modulus" buckling load. A number of recent investigations, that of Duberg and Wilder (7.17), largely inspired by Shanley's ideas, for example, have explored more deeply the problem of inelastic column strength, and these studies will need to be considered, especially in those cases where the buckling strength may be appreciably greater than the tangent-modulus load.

Aluminum alloys, stainless steels, heat treated steels, and certain other materials have decidedly nonlinear characteristics in the vicinity of the yield strength level. Structural steel, on the other hand, has nearly linear stress strain characteristics up to the yield point, when strains ten to twenty times the elastic strain develop suddenly with no increase in stress level. Although, for structural steel column strength evaluation, a statistical study of the yield stress level of the material seems desirable, (Fig. 2.3 for example), the effect of the relatively minor deviations in linearity of the stress-strain curve near the yield point are overshadowed by the possible effects of residual, or initial stresses, induced either by welding or by cooling after rolling. Although Salmon (Ref. 7.45, p. 155) mentions this factor and presents tests as supporting evidence, recent investigators often have not listed this item at all. Measurement of residual stress magnitude, distribution, and their effect on maximum bending and buckling strength of standard wide-flange shapes were reported recently by Yang, Beedle, and Johnston (7.61), and recent Lehigh tests have indicated procedures for experimentally predicting column strengths when residual stress is present. The effect on a few columns of large residual stress due to welding was investigated by Wilson and Brown (7.59), who found no appreciable reduction in column strength. The majority of the Wilson-Brown column tests had a slenderness (L/r) ratio of 65, with none greater than 75. The tests were in the flat end condition; hence, the Euler length factor K was probably between 0.50 and 0.60, giving an "equivalent pinned end" slenderness ratio of not more than about 40. For typical residual stress patterns, in the case of such short columns, the only effect of residual stress would be in connection with local buckling.

Long columns that buckle elastically should theoretically develop their buckling load in the presence of residual stress, provided the maximum stress remains below the yield point. A general treatment of the residual stress problem in its relation to buckling has been presented by W.R. Osgood (7.41).

It was well known before 1921 (Ref. 7.45, p. 155) that the past history of the material has a great influence on the ultimate strength of columns. Early tests by Baker and others showed that the resistance to flexure of mild-steel column was increased by previous compression and decreased by previous tension or cold-straightening. By Bauschinger's effect* on the stress-strain relation, Salmon explained these phenomena (Ref. 7.45, p. 156) on the assumption that it is the elastic limit which is the most important factor in determining the ultimate strength of columns. If a column is bent to one side by some forces which bring the maximum stresses in the column into plastic range, there will be residual stresses left in the column when the forces are removed. Therefore if a reversal of the loading is possible, the effect of these residual stresses as well as that of the Bauschinger effect on the strength of the column should be investigated. This may be of particular importance in connection with the behavior of buildings that experience negative drag immediately after the positive phase of an atomic blast wave.

The literature on the behavior of the column as an integral part of a truss or continuous frame will be discussed in Chapters IX and X. The current work in progress under the sponsorship of the Column Research Council on this and other special phases of the column problem is summarized in a recent report by Johnston, et al., (7.27). The following paragraphs will review current research on several special topics concerning column strength, and in considerable part the review of these topics will make use of the aforementioned summary of Column Research Council activities (7.27).

On the subject of lateral-torsional buckling of columns, the similar and closely related phenomenon that occurs in beams has been reviewed in Chapter IV. A program on eccentrically loaded columns has been under way for a number of years at the Research Laboratories of the Aluminum Company of America. This work includes both theoretical and experimental investigations (7.15, 7.20, and 7.21). The research work on columns now in progress at Lehigh University has included tests under combined moment and axial load, which necessarily involve the problem of lateral torsional buckling (7.9, 7.29).

The general problem of the column under combined axial load and bending moment is being reviewed at Brown University under a Column Research Council project and an initial report by Zickel and Drucker (7.63) has been released. Lateral-torsional buckling was not included in the initial considerations of their review but will be covered by studies now in progress at Brown University. The plastic behavior of a column under combined bending and direct stress, without involving the problem of lateral-torsional buckling, also has been under extensive investigation at Cambridge University (7.6, 7.29, 7.42, 7.43). Work on this subject at Cornell University will be discussed in the chapters on trusses and frames. Van den Broek (7.56, 7.57) also has studied the behavior of the columns under inelastic conditions.

*See Chapter II.

The competitive development of light gage steel sections in various patented forms by certain steel companies has led to a considerable number of test programs which include all phases of column behavior. Most of this work is unpublished. The work at Cornell University has been done for the American Iron and Steel Institute and has formed the basis for specifications for lightweight steel construction (7.16). Similar work on light gauge steel construction has been in progress at the University of Michigan (7.31, 7.32) and the University of Detroit (7.33, 7.34), and parallel developments in Great Britian have been reviewed by W S. Smith (7.49) and by Baker and Roderick (7.7).

On the subject of the stability of bridge chords without lateral bracing, an investigation under Column Research Council sponsorship is under way at Pennsylvania State College. A preliminary report (7.25) reviews the literature on this subject. The work will include tests of models as well as a theoretical investigation.

Test information on the strengthing effect of gunite or concrete encasement of steel columns is provided by Morris and Shank (7.37), and the strength of columns in brick walls has been tested by Harris, Stang, and McBurney (7.19). Stang, Whittemore, and Parsons (7.51) tested steel columns encased in concrete, and other similar Bureau of Standards investigations have been made. In the modified type of column tests proposed later in this chapter, the effect of encasement should be included as a variable.

The cross sectional shape and the method of joining the various parts of the column have an important bearing on its strength. Benjamin (7.10) has been studying the behavior of built-up columns, both from an experimental and an analytical standpoint. Large built-up columns have recently been tested at the University of Illinois (7.14, 7.38). Tests of columns with perforated cover plates have been carried on at the Bureau of Standards (7.50).

The column under dynamic loading is under investigation in Japan (7.29) and has been studied extensively by Hoff (7.22, 7.23, 7.24), who, as chairman of the Column Research Council committee on this topic, has prepared the following statement that is included in its entirety because of its close relationship to this project. Since the statement is intended as the basis for further work to be sponsored by Sandia Corporation on the subject of dynamically loaded columns, this will be considered as research in progress, and no proposal for dynamic tests of columns will be made in this report.

> "During a strong blast the peak pressure on a structure generally builds up practically instantaneously, then reduces to zero within one second; but the stress history in the members of the structure depends upon the geometry and inertia of the structure and the stress-strain relationship of its material of construction. The question to which an answer is urgently needed can be stated like this: In what structures

subjected to strong blasts does one have to consider dynamics in analyzing the failure of compression members? If in all or most cases it would be possible to neglect the dynamics of buckling, then the problem of the failure of columns would be considerably simplified.

"The objective of the investigation is to establish the conditions of failure of columns in buildings, and for this reason consideration of the inelastic properties of the material will be of paramount importance. Similarly, the dead weight of the structure and the soil properties may have a significant effect. The interaction between the various structural elements must be considered because it manifests itself in the end conditions to which the column is subjected. In particular, the amount of sidesway influences the failing load materially.

"It is realized that a complete answer to the problem can be had only through a lengthy investigation, including both analytical and experimental work, especially if the refraction of shock waves, the yielding of the connections and the soil, and the dynamic stress-strain relationship of the material are to be taken into account. It appears likely, however, that a qualified answer can be obtained in a much shorter time purely analytically, if the work of the investigation is limited to finding out whether the static failing load of the column represents a satisfactory engineering approximation to the actual failing load in a building subjected to a strong blast. The load history for the individual column as well as the time dependent boundary conditions must be established approximately on the basis of available experimental data and the analytical methods of applied mechanics."

7:3 Recommendations for Research.

Column Research Council, organized especially for investigating the strength and behavior of columns, has offered its services, in the form of technical advice, to the present project.

RP 7.1--Investigation of column behavior in the region of the maximum load. Most column tests are conducted in the laboratory with pinned or flat ends, with sidesway not permitted, are without lateral load, and are usually stopped at or near maximum load. In the failure of an industrial building under lateral blast load, the columns have moments and/or rotations applied at their ends, experience large sidesway, may be loaded laterally by the wall or floors due to transmitted blast pressure, and, if failure is complete, are probably deflected far beyond an amount corresponding to their maximum load capacity. An extensive column investigation is here proposed that will consider those factors, as outlined above, that correspond to actual conditions pertaining during lateral destructive loads.

The items listed below follow the pattern indicated by the chart of column problems presented earlier in the chapter. The proposals are grouped together under one project, since close coordination between the various phases is desirable. However, the overall project is one of considerable magnitude and might well be subdivided into several parts under a single coordinating head, or research committee.

1. Tests should be designed so that sidesway is permitted, and so that bending, torsion, and local buckling may be strength-determining factors, separately, or in combination.
2. The program should include various shapes of column cross section, such as may be used in industrial or tier buildings, including both solid wide-flange shapes and built-up members.
3. In order to simulate the sidesway and end conditions that are desired, it may be necessary to test columns in pairs in a guiding frame.
4. Various load sequences and combinations, such as forced sidesway with and without reversal of direction, axial load, end rotation, and lateral load, should be included; these would simulate the variety of conditions that might obtain in both industrial and tier buildings.
5. The effect of fireproofing or other column encasement should be included, without duplicating the work proposed in Chapter V for test of combination steel frame and wall units.
6. Test results in the program should be coordinated with the dynamic studies for which research proposals are currently being solicited and which would presumably be followed by dynamic tests.

A study of the resistance of various standard types of column footings under lateral and rotational movement has already been suggested in RP 5.2 on "Wall and column footings".

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CHAPTER VIII

PLATE ELEMENTS OF STRUCTURAL MEMBERS

8:1 Introductory Statement.

Structural elements with thicknesses rather small in comparison with their lengths and widths, if flat, will be referred to as plates, and if curved, will be referred to as shells or, if the curvature is not very large, as curved plates. Most structural steel members are composed of plate elements, and may be rolled as an integral unit or built up by welding, bolting or riveting. For example, the rolled wide-flange beam may be considered as an assemblage of five plate elements -- one web and four outstanding flange elements. Truly "built-up" members, on the other hand, are formed by riveting or welding together plates, channels, angles, etc.

The initial part of Chapter VII defines "buckling" and differentiates between general and local buckling. When a structural member is in action, except in pure tension, some element of the member is usually under compression and/or shear which, if increased to a critical level, will cause the buckling of that element. For example, when a wide-flange simply supported beam is in bending, the top flange elements are under compression, and may buckle while the other portions remain undamaged. Such "local buckling" in one element or in a local region of a structural member (as distinguished from integral buckling) is one of the principal subjects of this chapter. Since most steel members are designed to reach the yield point before local buckling commences, such buckling will usually be a plastic-range type of behavior and its evaluation will be especially important in the study of ultimate strength of structures. The local buckling of the compression flange or web of a beam has been mentioned in Chapter IV on "Beams and Girders", the local buckling of plate elements of a column in Chapter VII on "Columns", and the general buckling of the wall panels in Chapter V on "Roof, Wall, and Floor Panels". These and other buckling problems, such as the strength of lacing bars in a built-up column, will be reviewed further in this chapter. Local buckling may also precipitate general collapse if it induces or accompanies integral buckling of other parts of the column.

Another subject included in this chapter is that of the buckling strength and behavior of a plate curved in one direction only, under edge loads applied in a direction tangent to the middle surface of the plate.

The buckling strength of a plate is markedly influenced by conditions of edge restraint. A plate element of any part of a structural member may have various edge-restraint conditions that are analogous to those at the end of a beam, as discussed in Chapter IV. In the case of a plate these restraint conditions are distributed along a boundary, and the free edge, simply supported edge, hinged edge, and fixed edge may be considered as exactly similar to those cases illustrated in Fig. 4.2. In the case of elastic support the problem is more complicated, especially if the plate is built into an edge or interior supporting beam or stiffener which is premitted to deflect laterally and twist. In this case the interrelation between the torsional and bending behavior of the beam or stiffener and the boundary moments and reactions of the plate element must be considered. The mathematical statement of these and other boundary conditions in the elastic range may be found in standard references on plate bending and buckling theory (8.62). When plates in different planes are joined together as at the juncture of the flange and web of a wide-flange beam, the plate with the greatest tendency to buckle is restrained elastically with respect to boundary rotation by the plate or plates with the least tendency toward buckling. The restraint value varies with stress in the plane of the plate. Approximate solution of the interrelation between various plate elements of typical structural sections is well reviewed by Bleich (8.8). Plate elements, after initial buckling, often can withstand a considerable additional load because of the redistribution of stress that accompanies large deflections. Although the behavior of plate elements after buckling is a very important consideration in the investigation of the ultimate strength of structures, it is neglected in the usual civil engineering structural design. Special emphasis therefore, should be placed on the complete buckling behavior and the ultimate strength of plate elements.

8:2 Survey of Theory and Experiment.

Several surveys of the subject matter of this section are currently available. Therefore, in general, only these surveys and references on problems which have not been covered by these surveys are listed as references for this section.

Timoshenko's book (8.62), published in 1936, was the first text in English to summarize, in detail, almost all the important problems of elastic buckling of thin plates. It includes consideration of various edge and loading conditions, with or without stiffeners (longitudinal and transverse). Practical applications of the theoretical results to compression members and plate girders have also been discussed. The book also contains a detailed summary of the theoretical and experimental work prior to 1936. In buckling theory, an "idealized" plate is defined by the following set of assumptions analogous to those used in developing column buckling theory:

Material:
(1) Perfect elasticity.
(2) Homogeneity and isotropy of mechanical properties.
(3) No initial internal stresses.

Member:
(4) Perfectly flat, and of uniform section.
(5) Load resultant and middle plane of the plate are coincident.
(6) Edge restraint is constant.

Buckling behavior:
(7) Deflection is small (less than 1/2 the plate thickness)
(8) Plate is thin, i.e., the transverse shearing effect on the deflection of plate is negligible.

Plastic buckling (a modification of assumption (1)above) and the large deflection theory (modification of assumption (7) above) have also been discussed briefly for a few special cases. Moisseiff and Lienhard (8.46) simplified many of the results in Timoshenko's book (8.62) by approximations into simple forms, primarily for design applications, and discussed the effect of longitudinal and transverse stiffeners.

Winter (8.74), reviewed the recent investigations on the behavior of compression plate elements, including approximate solutions, plastic buckling, post-buckling behavior, small deviations from flatness, and some experimental results. In the book by F. Bleich (8.8), there is a very detailed and critical review on this subject. In Chapter 12 Bleich takes up the problem of plate elements (without stiffeners) under edge compression and in Chapter 15 he discusses the problem of buckling of web plates of girders (with or without stiffeners). All results are presented by means of simplified formulas and graphs.

In a recent paper Stowell, Heimerl, Libove, and Lundquist (8.58) summarized the investigations on the buckling of flat plates as studied in aeronautical research during and since World War II, with special emphasis on plastic buckling and the ultimate (or maximum) strength in compression. The Column Research Council recently conducted a world-wide survey (8.3) on both published and unpublished recent theoretical and experimental investigations, summarized through early 1950. Jakkula and Stephenson (8.26) have reviewed the literature to a limited extent between the years 1921 and 1947 and summarized the important experimental work during the same period. Massachusetts Institute of Technology prepared a bibliography on elastic stability (8.44), in which references 203 to 351 are for the buckling of plates.

Approximate and arbitrary modification to include the effects of inelastic stress-strain relations have been attempted by F. Bleich (8.8) and others. Many efforts have been made to solve this problem more exactly using the mathematical theory of plasticity. Ilyushin obtained certain results and Stowell (8.8, Chapter 12) improved and simplified Ilyushin's theory. Many papers on this subject have been published in the past few years (8.6, 8.22, 8.28, 8.61). Attempts (8.5, 8.12, 8.23, 8.33, 8.76) have also been made to include the effects of large deflections which is essential to the evaluation of maximum strength. Stowell (8.58 - ref 43) considered both large deflections and plasticity of the material to solve this problem for a relatively simple element, the hinged flange. Results of tests to determine the ultimate strength of plate elements or sections have been available for various cases (8.60, 8.67, 8.59, 8.63). Tests and analyses of wide-flange shapes are currently under way at Lehigh University.

The effect of perforations on the buckling strength of plates has been theoretically investigated by Kroll (8.29) and by Levy, Woolley, and Kroll (8.35). Tests of perforated columns were made by Stang, and Greenspan (8.57), and welded built-up columns, laced or stay-plated, were tested by Haider (8.21) early in 1931. The behavior of built-up columns is also being studied by Benjamin (8.4). At M.I.T., Norris and others (8.50) have under investigation the effect of intermittent edge welds on the strength of built-up welded columns. The buckling phenomena of corrugated and reinforced corrugated plates has been studied theoretically and experimentally by Seydel (8.56). The diaphragm action of steel floors has been investigated experimentally by H.H. Robertson Co. (8.1, 8.11). The influence of local buckling on the behavior of welded frame connections has been studied recently at Lehigh (8.79).

The buckling of cylindrical shells under axial compression has been studied by von Karman and Tsien (8.68), whose method has been used recently by Donnell (8.14) in an investigation of the effect of imperfections. Donnell (8.14) includes a review of test results. A historical survey has been made by Hoff (8.25) on thin-walled monocoques. Marquerre (8.39) discussed the initial elastic buckling of cylindrical shells with variable curvature in a recent paper, and the later range of buckling behavior has been theoretically investigated by Schunck (8.54).

8:3 Recommendations for Research.

RP 8.1--Ultimate plastic buckling strength of flat plate elements of structural members. Compared with the number of investigations on the initial elastic buckling strength of the plates, relatively few papers (8.58, 8.30, 8.55) are available on the ultimate strength of plates in which large deflections and plasticity of the material are both considered. After a careful study of existing literature, gaps in available information should be closed by experimental and theoretical studies in which the following factors are included:

(1) Various edge restraint and loading conditions, utilizing short lengths of various types of column or girder section.
(2) Tests to include various longitudinal and/or lateral stiffener arrangements.
(3) Tests to be designed for plastic range buckling, with test studies through the complete range of behavior to and beyond maximum load.

RP 8.2--Ultimate plastic buckling strength of curved plates and cylindrical elements. The following would be included:

(1) Tests and analyses of ultimate strength of steel cylinders and cylindrical segments under axial compression.
(2) and (3) Similar to RP 8.1.

RP 8.3--Effect of local buckling on complete behavior of structural members. Local buckling of a structural element is not necessarily followed by immediate failure of the whole member. The effect of local buckling on the overall strength and behavior of the member is a function of the cross-sectional shape, continuity, load arrangement, lateral support location, and the location of the buckled element. For instance, local buckling of the flange of a plate girder may be more serious than the local buckling of the web, especially if vertical stiffeners are present There is a need for systematic research to study the effect of local buckling on the behavior of the member from the initiation of local buckling to the final failure of the whole member. However, there would be some difficulty in setting up such a project without duplicating to a considerable extent some of the work previously proposed under Chapters IV and VII on beams and columns. The following project, therefore, should be carried out in coordination with other projects on beams and columns, to study specifically related local buckling problems, and to include additional experimental work that may later seem to be required. The following factors should be considered:

(1) Various shapes of cross section, dimensions, and end conditions of the members.
(2) Various members (column, beam, etc.).
(3) Various loading conditions.
(4) The interrelation between local buckling and the general

buckling of beams (lateral-torsional buckling) and columns (bending and/or torsion).

RP 8.4--Local buckling of plate elements of structural members under dynamic loads. There is little experimental or theoretical information available on this subject. However, as in the case of the previous project proposal (RP 8.3), duplication with projects already proposed under the chapters on beams and columns should be avoided. Correlation with these other projects should be planned and additional experimental and analytical work programmed at a later date. The effect of rate of deformation on the form of both elastic and plastic plate buckling under various conditions of edge restraint and type of loading should be studied.

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CHAPTER IX

TRUSSES

9:1 Introductory Statement.

A truss is a frame of straight members, joined at their ends, that form a pattern of triangles. The unique feature of a truss is the fact that its members sustain the applied loads primarily by means of axial force rather than by shear or moment. The joints between truss members may be hinged, semi-rigid, or rigid. A plane truss is one in which all the members of the structure are in one plane. Actual structures are space frames, but in many cases the main load-carrying trusses act more or less independently in their plane and secondary bracing members hold these main trusses in position. As an example, the main trusses of a bridge are usually designed as plane trusses, held in position by lateral top and/or bottom trusses, in horizontal planes, with cross-bracing in vertical planes normal to the planes of the main trusses. In a highway bridge, the floor slab, floor beams, etc., are primarily load-carrying elements but also act as bottom bracing between the two trusses.

The members of a truss may be contrasted to those of a continuous frame that transmit bending moment, shear and axial force as primary or "load-carrying" stresses. Continuous frames will be discussed in Chapter X of this preliminary report.

The individual members of frames have been discussed in Chapter II for tension, in Chapter VII for compression and in Chapter IV for bending. Connections have been discussed in Chapter VI. It is the object of this chapter to discuss both the local and over all behavior of a truss, assuming that the complete load-deformation relations of members and connections are already known.

The members of a truss may be joined by pinned, riveted, or welded connections. The members at a pin connection may experience relative rotation without developing appreciable bending moment, but this is not the case for riveted or welded connections. Hinged connections, nevertheless, are usually assumed as a basis for the design of trusses, irrespective of type of connection. This simplification usually is satisfactory, because bending moments in truss members are of secondary effect and do not usually impair the safety of the structure. In the ordinary design, the axial stress computed by assuming hinged connections is called the primary stress and the stress due to bending (computed from the deflections of the truss as caused by the primary stresses) is called the secondary stress. In the ordinary design, it is assumed that the secondary stresses have no effect in changing either the axial stresses in members or the deflections of the truss. The effect of secondary stress, or more exactly, the effect of the moment resistance of the connections, on the general yield and ultimate strength of a truss is still an unsettled problem.

Essentially, a truss is an open web beam and the end conditions of a truss can be described in the same way as for beams (see Chapter IV). A bridge truss is usually pin-ended or supported on rollers, while a building truss is usually connected to columns and hence is supported at the ends with rotational elastic restraint. A truss tower structure is free at the top and fixed at base. The strength of trusses, like that of a beam, will be affected appreciably by the end conditions. It is assumed initially herein that each connection between truss members is hinged. The effect of connection rigidity will be considered separately later in this chapter.

Trusses with pinned connections, real or assumed, may be classified as either statically determinate or statically indeterminate. Trusses may be statically indeterminate externally, with redundant supports, or internally, with redundant members.

A truss may be reduced by local failure to behave as a mechanism, or it may fail as a whole by lateral buckling. The second type of failure has been discussed briefly in Chapter VII on "columns", and this chapter will cover failure as a mechanism.

A mechanism may be defined as an assemblage of members in which there are enough hinges to permit the members of the structure to take various positions in space without change of stress and without increased resistance to external load. Several examples of such failures are shown in Fig. 9.1, which depicts the formation of effective hinges in a truss as a result of yielding or buckling of a particular member. A "moment" hinge can take no appreciable additional moment above the yield value. A "shear" hinge is one that may be able to transmit moment but can take no increment of shear force after yielding commences.

The development of a moment hinge is illustrated in Fig. 9.1 (a), wherein a top chord member is presumed to have buckled. If buckling occurs in the elastic range the member would continue to resist the buckling load without reduction and the plastic hinge moment in the truss would be the product of this force and the depth of the truss, plus the moment resistance of the lower joint alone. A "shear hinge" would be produced by the buckling of a web member as shown in Fig. 9.1 (b), which, if relatively weaker than the adjacent chord members, would permit shear displacements of the truss with the type of mechanism as shown. If the buckling of a member occurs in the plastic range, as is usually the case, the hinge moment, as deformation continues, will decrease rapidly after reaching the maximum value.

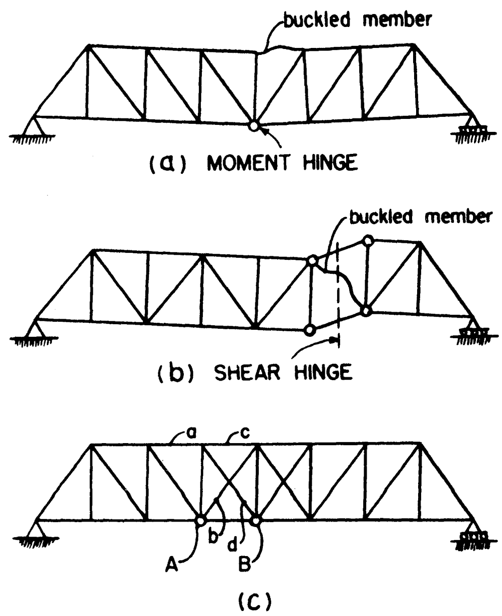

FIG. 9.1

Fig. 9.1 (c) illustrates for a simple case different types of behavior possible for a truss that is internally indeterminate. If member "a" buckles, a moment hinge will be formed at A, and collapse will occur in the same manner as indicated in Fig. 9.1 (a). On the other hand, if member "b" starts to buckle, collapse cannot occur immediately if there is still reserve strength in other members. Additional load will be carried until another member in the panel, such as "c" also reaches its maximum load, or alternatively, until some member outside the indeterminate panel such as "a" buckles.

The foregoing discussion applies to a truss in which all members are joined by frictionless pins, which, of course, is not the case in practice, since even when pins are used there is considerable rotational friction present.

Compression members connected by gusset plates in a truss frame may be considered as columns with restrained ends. The degree of the end restraint of a member in a truss is a function of the bending stiffnesses of the adjacent members. Thus, the end restraint provided to any member in a truss is a function of all the other members of the truss. Prior to yield, tension members always add positive restraint to the ends of the adjacent members. Compression members also add positive restraint to the ends of the adjacent members when the axial compression load is low, but their contribution to restraint progressively reduces and may become negative when their tendency toward buckling reaches a critical value.

The separation for purposes of design of one plane of a frame from the whole structure requires the neglect of bracing, which may contribute an appreciable increment of strength to the whole structure. Therefore, in an investigation of ultimate strength of trusses, the additional strength due to bracing participation should be considered.

Variation in load-deflection characteristics as a function of rapid rate of loading should be considered. This has been discussed in reference to separate members and connections in the pertinent earlier chapters. In a large truss, the rate of transmission of load from one part to another may be an important influencing factor in dynamic behavior.

9.2 Survey of Theory and Experiment.

The analysis and design of trusses has been a development covering many centuries. Most attention has been given to the elastic range, and comparatively little effort has been applied to the prediction of complete load-deflection relations.

The recently published book by the late F. Bleich (9.7) provides a good critical review of all buckling problems; in it, Chapter 6 is devoted to both trusses and rigid frames. Kavanagh recently made extensive historical reviews (9.13, 9.14). The buckling of trusses also has been reviewed in the Column Research Council Survey (9.5). Ratzersdorfer's book (9.22) gives general methods for the buckling analysis of hinged and rigidly connected trusses. The buckling of hinged trusses has been investigated by von Mises and Ratzersdorfer (9.30, 9.27) for both plane and space trusses.

The method of moment distribution has been applied to the buckling analysis of rigidly connected trusses by Lundquist (9.18, 9.5) and later by Hoff and others (9.11). Hoff included inelastic buckling and gusset-plate effects. Wessman and Kavanagh (9.33) have studied application to practical design with several important conclusions. Kavanagh (9.15) has applied the moment distribution method to frames with semi-rigid connections, including nonlinear joint rigidity and plastic buckling. Winter, Hsu, Koo, and Loh (9.35) have developed a method of end restraint calculation for the elastic buckling analysis of plane frames.

Test results, as well as analytical studies, were reported by Ballhaus and Niles (9.4) for pin-jointed trusses. Hoff and others (9.11) have tested small model welded and riveted trusses as cantilevers under the combined action of shear and moment. Some model structures of the Warren type with rigid connections have been made in Tasmania (9.28) for the purpose of developing and checking a simplification of the Lundquist-Hoff methods (9.18, 9.11). Thern (9.26), reported on test results of three small model trusses which were (a) riveted with conventional gusset plates, (b) welded with gusset plates, and (c) welded without gusset plates. In all cases, failure occurred by buckling of the compression member. Secondary stresses and deflections were measured. Two full-scale welded Pratt trusses (one of

H-section members and the other of pipe-section members) were tested by the Austin Company (9.3). In each test, two identical trusses were braced in the plane of the top chords to form a unit and connected at ends to columns. Loads were applied at top panel points. Failure was caused by buckling of the end verticals. Inglis has made some tests on simple welded triangular trusses (9.12), in which the test measurements were made somewhat beyond the buckling load.

The yielding failure of indeterminate hinged trusses has been discussed theoretically by Steinbacher, Gaylord, and Rey (9.25), including comparison with some test results. Fundamental questions important in estimating load-carrying capacity have been discussed by Kohl (9.16). Samuelson (9.23) made a test on a hinged steel truss with one degree of internal redundancy. One tension member was made much weaker than the other members and the test was carried only far enough to cause the weak member to yield. Gruening and Kohl tested a continuous truss of 3 spans beyond the elastic limit (9.10).

The transmission tower is probably the most common example of the true space truss. Many transmission towers have been tested to failure under lateral load by the Canadian Bridge Co. (9.8, 9.29) and the Stone and Webster Engineering Corp. (9.34) has made similar tests. Van den Broek (9.29) has applied the "limit design" procedure to the transmission tower problem.

Impact effects in bridge trusses have been studied by AREA and the results have been published in two papers (9.1, 9.2). This work has been in the elastic design load range. Fraenkel (9.9) has discussed both elastic and plastic behavior of statically determinate steel truss and girder bridges under the loads resulting from the explosion of a "nominal" atom bomb 2000 ft above the bridge.

9.3 Recommendations for Research.

Section 9.2 of this chapter has indicated that considerable information is available regarding the strength of determinate trusses. Less is known concerning indeterminate structures. Past and current research will be evaluated and methods for strength calculation of trusses will be summarized in the final reports of this investigation. Tests of complete existing structures that may be condemned for further use will be proposed in the later chapters on bridges and industrial buildings. Apart from such tests, the following projects are outlined as desirable. These may be subdivided if too large for any one organization.

RP 9.1--Ultimate strength of trusses as applied to bridge and building construction.

(1) Determinate trusses. Emphasis would be on behavior after

initial yielding of a tension member or buckling of a compression member. Analyses would be supplemented by tests of models and the following factors would be considered as variables:

(a) Effect of size and type of gusset plate or other connection detail, together with a comparison of welded and riveted connections.

(b) Effect of highway floor and bracing or railway lateral bracing in adding to strength of highway or railway bridges, respectively.

(c) Effect of strength of roof trusses in industrial buildings.

(d) The function of wind and sway bracing in industrial buildings in providing unified action of all parts of the roof structure.

(e) Effect of walls in adding to strength of trusses in tier buildings and other special features that may pertain to this application of the truss.

(2) Internally indeterminate trusses. Buckling or yielding of successive redundant members as the ultimate capacity of internally redundant trusses is approached and reached.

(a) to (e) same as preceding, under Item (1), and coordinate with studies pertaining to determinate trusses.

(3) Load conditions to be considered in connection with items (1) and (2) of this project.

(a) Vertical loads to be considered in all cases.

(b) Strength under horizontally applied loads to be given special emphasis. Such loads should be considered both in the plane of the truss and normal to it, including the effect of cross bracing and the roof or floor systems. When horizontal loads are assumed in the plane of the truss, as applied to industrial buildings, the resistance of adjacent bays at the far end may put the entire truss under direct compression, and its strength in this respect is of importance in considering the transmission of loads into the adjacent parts of the building. When a building has only one bay, the attachment of the chords to the building columns may induce racking loads involving horizontal shearing weakness. These and other special problems relating to truss behavior as part of an industrial building frame should be given special consideration.

<u>RP 9.2--Ultimate strength of long-span bridges</u>. Very little information is available on this subject. In long-span structures the dynamic analysis approach is particularly essential in view of the greater relative period of vibration and greater importance of elastic range behavior, as compared with smaller structures.

Numerical analysis procedures should be utilized, and dynamic tests of small models might be used. The project should include cantilever, suspension, and continuous truss bridges.

RP 9.3--Bridge supports. Roller supports are commonly used for bridge trusses. The traveling distance of the rollers is limited to a certain range. The movement of a single roller beyond the traveling range is stopped by a sudden increase in elevation (a steel bar welded to the base plate, for instance); for a group of rollers, the motion is stopped by the prevention of the rolling due to the contact between the flat vertical faces of the segmental rollers. Near the ultimate strength of the truss the deflections are usually large and a change in the behavior of roller supports may be expected.

In view of the possible lateral displacement of the complete bridge, the strength of various bridge supports under lateral load should be considered.

(1) Behavior of roller (single or group) supports beyond the traveling range.
(2) Effect of the strength of the stopper or connection between rollers.
(3) Effect of change of behavior of the roller supports on the strength and behavior of the truss frames.
(4) Lateral load-deflection characteristics of various bridge supports.

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CHAPTER X

FRAMES AND ARCHES

10:1 Introductory Statement.

Although a truss is usually considered as a type of frame, for the purpose of this report the following differences between frames and trusses are assumed:

Frames	Trusses
1. Loads may be applied at any location.	1. Loads are usually applied at joints.
2. Bending moment and shear are relatively as important as axial force.	2. Axial force is of primary importance.
3. Unit structure is polygon of four or more sides.	3. Unit structure is triangle.
4. In conventional elastic design, end moments of all members are computed by assuming that there are no axial forces in the members.	4. In conventional elastic design, forces in all members are computed by assuming that all connections are hinged.

Frames may be classified as "rigid" (continuous), "semi-rigid", or "simple", according to the rigidity of connections. The following definitions of these three types of frame are quoted from Section 1 of the American Institute of Steel Construction "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings":

> Type 1, commonly designated as "rigid-frame" (continuous, restrained frame), assumes that the end connections of all members in the frame have sufficient rigidity to hold virtually unchanged the original angles between such members and the members to which they connect.

Type 2, commonly designated as "conventional" or "simple" framing (unrestrained, free-ended), assumes that the ends of beams and girders are connected for shear only, and are free to rotate under load.

Type 3, commonly designated as "semi-rigid framing" (partially restrained), assumes that the connections of beams and girders possess a dependable and known moment capacity intermediate in degree between the complete rigidity of type 1 and the complete flexibility of type 2.

A truss sometimes acts as a member of a rigid frame (as in a mill-building), and this type of structure will be included in the following review of continuous frames.

An arch may be defined as a curved structure, concave downward, which is subjected primarily to forces in its plane and which makes use of horizontal reactive thrust at its abutments to reduce the bending moment in the structure. Arches include the truss arch and the girder section arch. Truss arches are usually made up of individual straight members; girder arches utilize a curved plate girder section in which both bending moment and axial force are of primary importance in the design. Truss arches were included in the classification of trusses, discussed in Chapter IX.

This chapter will consider only the over all behavior and ultimate strength of frames and arches. Individual members and connections are covered in Chapters IV, VI, VII, IX, and local failures are referred to in Chapter VIII.

Applications of frames and arches in steel structures include the following:

Frames are used in tier buildings, industrial buildings, rigid-frame bridges, and Vierendeel "trusses" (a Veirendeel "truss" is not a truss according to the terminology used in this report).

Arches find use in auditoriums, gymnasiums, industrial buildings such as Quonset buildings, and arch bridges.

Failure of a frame or arch may be influenced by a number of factors which are discussed in the following paragraphs:

(1) Plastic yield failure. When a frame or arch is loaded above the design allowable load, some section may approach complete yield, due primarily to bending moment, and this section is then said to be a "plastic hinge" as discussed in Chapter IV. Localized "plastic" hinges occur usually at connections or points of concentrated load. In locations of more or less

uniform bending moment, the plastic region may be distributed and less localized than is usually assumed when speaking of a "plastic hinge". When the number and location of hinges produce a mechanism in a vital part of a structure, failure will occur. The term "plastic" hinge is used in a qualitative sense and is not intended to imply that the moment resistance will remain constant as deformation proceeds. Local buckling may cause a reduction in moment resistance, whereas strain hardening may cause an increase. The frame shown in Fig. 10.1 (a) can fail as a mechanism with 6 plastic hinges, as shown in Fig. 10.1 (b), or, under different conditions, with 7 hinges, as in Fig. 10.1 (c). In an idealized sense, assuming little effect of walls or floors, the tier building frame in Fig. 10.2 may

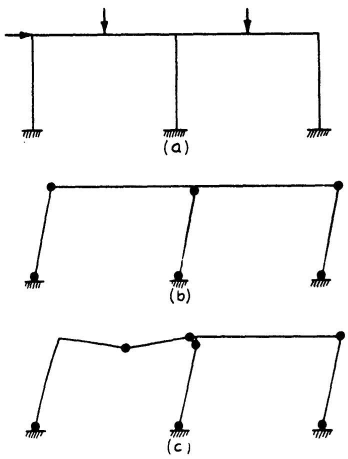

FIG. 10.1

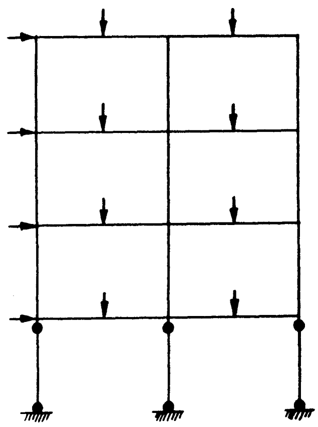
FIG. 10.2

also experience failure with the formation of only 6 hinges, as shown, provided the load that causes such hinge formation is maintained without reduction. This latter requirement is an important consideration in all cases of this type, since under dynamic conditions of load, with a rapid decrease and possible reversal of over-all drag, a structure does not necessarily fail after becoming a "mechanism". It is also seen that the degree of structural indeterminacy may determine the upper limit of the number of hinges required to produce a mechanism, but the exact number that might actually form cannot be predetermined by any general rule. Each problem in plastic-dynamic structural analysis must be handled as an individual case.

(2) Instability in the plane of the arch or frame. A frame may also fail due to instability when a sufficient number of members lose their shear or bending stiffness because of axial load. Every compression member (including the arch) may be considered as a single member with varying end restraints and/or loads, which are determined by the behavior of other parts of the structure. The buckling of one member in a frame does not necessarily mean the failure of the frame. In general, if a frame fails by deformation within its own plane it is due

to a combination of instability and plastic yield, i.e., a combination of (1) and (2).

(3) Lateral instability. When a plane frame or an arch is not sufficiently braced in the direction normal to its plane, it is liable to lateral buckling failure. Such failure is often preceded or accompanied by local buckling of plate elements.

(4) Sidesway. When sidesway is prevented, the strength of a frame is usually much greater than that when sidesway is permitted, provided the loading condition is similar in each case. For a symmetrical structure with horizontal beams, no sidesway is present under symmetrical loading. In the case of similarly loaded arched or gabled frames, over-all sidesway will not occur, but the joints will translate with similar effect since the ends of individual members will experience differential lateral movement.

(5) Effect of shock loading. Under suddenly applied load the general mode of failure may be different than under static load. In addition, as discussed in earlier chapters, the rate of strain will affect the general strength level of the members of a frame. It is also possible for a frame to become a mechanism without failure if the dynamic load reduces rapidly and if the subsequent resistance of the structure is sufficient to stop its motion.

10:2 Survey of Theory and Experiment.

A considerable amount of work has been done on frames and arches, as exemplified by the 53 references that have been listed out of about 200 that have been card-indexed on this subject.

Recent reviews are available on the strength of frames with straight members. The buckling strength of such frames is well covered by two very recent books, one by F. Bleich (10.10) and the other a review by the Committee of Column Research of Japan (10.16). The former (10.10) reviews theoretical methods of attack and presents a variety of results in detail; the latter (10.16) tabulates and records the sources of many solutions, including 60 elastic frame buckling problems. Baker (10.3) has summarized the extensive work in Great Britain, especially at Cambridge, on the plastic yielding strength of frames with straight members. Symonds (10.46) has recently made a review of the various methods of plastic-hinge analysis of the collapse strength of rigid frames. In general, those references that are covered in detail by Symonds will be omitted in this report.

Buckling of the columns in simple frames has been well covered by the two reviews (10.10, 10.16) mentioned above. When joint translation is prevented, the buckling of any frame composed of straight members is similar to that of a truss with rigid connections, as discussed in Chapter IX.

Buckling of any member of a frame of straight members, with sidesway permitted, has been investigated by Kavanagh (10.29) using the moment-distribution method modified for joint translation, and by Chwalla and Jokisch (10.13) using the slope-deflection method modified for axial forces. The end-restraint method developed by Winter, Hsu, and others (10.53) can be applied to frames with sidesway. Semi-rigid connections have been discussed by Kavanagh and Moore (10.30).

Failure of frames due to plastic-hinge formation has been studied at Brown University for idealized structures (10.46, 10.55). Recently three papers have been written on some new techniques (10.21, 10.24, 10.48) for estimating the ultimate strength of rigid frames, based on the assumption of simple plastic theory (i.e., strain-hardening effect is neglected). The effect of strain-hardening on the ultimate strength has been investigated by Horne (10.26). Semi-rigid frames have been analyzed by Schenker (10.44) by the moment-distribution method, using the assumption of simple plastic theory. A large number of test results have been published in England on rigid frames (10.4, 10.5, 10.22) for the purpose of checking the simple plastic theory. Current research at Lehigh (10.55) includes tests on nearly full-size welded portal frames.

Dynamic load effects on the strength of frames have been investigated by Pan (10.40) for one-story frames with consideration of plasticity under earthquake motion. Penzien and Williams (10.41) have studied theoretically a frame subjected to atomic explosion, including the effect of wall slabs. Newmark (10.37) has applied numerical procedure to the analysis of dynamic behavior of structures for both elastic and plastic ranges.

Elastic buckling of arches has been reviewed by the Committee of Column Research of Japan (10.16), and by Timoshenko (10.50). Results of tests on buckling of arches may be found in several papers (10.8, 10.12, 10.14, 10.15, 10.17, 10.18, 10.19, 10.31, 10.32). Plastic analysis of arches has been discussed recently by Swida (10.45) and by Vreedenburgh (10.51). Dynamic effects in steel arches have been studied by Bryla (10.11). Current work is in progress at Illinois Institute of Technology (p. 51 of 10.1).

10:3 Recommendations for Research.

A great deal of theoretical study has been given to the ultimate limit capacity of continuous steel frames. In addition, there is a limited amount of experimental information on simple portal frames. Most of the existing information is based on simplified idealizations that avoid the practical difficulties met in real structures, such as the effect of local and/or lateral buckling of frame elements, the stiffening effects of walls and floors, the effect of residual stress, and other modifying factors.

Emphasis in proposed projects is to be on applications in actual structures with the following projects suggested:

RP 10.1--Steel framing in tier building construction. Plane and/or three-dimensional unit segments of typical tier-building construction should be tested under racking or shear loads to determine the partition of load between the various composite elements of the unit structure. Similar tests were proposed as project RP 5.1, in which case variations in the wall or floor material were to be the variable and consideration was also to be given to door and window openings. In this project (RP 10.1), the principal variable is to be the steel frame construction and one or more typical wall and floor constructions are to be used. If desired, projects RP 5.1 and RP 10.1 can be carried out together. The following items should receive consideration:

(1) Comparison of riveted, welded, and bolted construction utilizing various typical details in both simple and continuous framing.

(2) A study of small model tests to determine the feasibility of predicting relative wall and floor participation on a small scale.

RP 10.2--Steel framing in industrial buildings. Unit segments of various types of industrial buildings utilizing frame construction are to be tested as three-dimensional models approaching full size as nearly as feasible. A similar test program has been outlined for truss frames as project RP 9.1. The project should include various simple and continuous frame types as outlined in Chapter III under II-B. Special attention in the project should be given to the following items:

(1) Tests should involve various combinations of horizontal and vertical loads applied in the plane of the frame.

(2) Tests of two or more parallel frame units connected by usual types of roof and wall bracing, and including the roof and wall panels, with loads applied normal to the plane of the frames.

(3) A special study of local and/or lateral buckling should be made, together with a study of the efficacy of usual lateral bracing to prevent it. The modifications necessary in usual assumptions of plastic hinge formation occasioned by buckling should be considered.

(4) A special study should be made on the subject of transmission of horizontal loads from bay to bay in multi-bay

construction. This phase of the project, especially, should also include dynamic tests and analyses.

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CHAPTER XI

TIER BUILDINGS

11:1 Introductory Statement.

The fundamental problems concerning the behavior of the component parts of a steel structure and their interrelation have been discussed in the preceding chapters, IV to X inclusive. This and the three succeeding chapters, XII, XIII, and XIV, will be concerned with complete structures. The research proposals in this and succeeding chapters are intended to supplement the proposals of earlier chapters and should of course be coordinated with them.

Buildings which are two or more stories high will be called tier buildings. This classification includes not only the tall, narrow office building but also the short, squat warehouse. In Chapter III a division was made between the two types of tier buildings just mentioned: tier buildings which had a height-to-width ratio greater than 2 to 1 were termed "slender" and those buildings which had a height-to-width ratio of less than 2 to 1 were called "broad".

A building under lateral load is, on the whole, a cantilever beam, and as such must provide internal resistance to the resultant shear and moment at any level. The internal distribution of this resistance depends to a large extent on the stiffness of the beam-column connections, on the contribution of walls to the over-all stiffness of the structure, and on the relative slenderness of the complete structure. Under blast, very broad buildings might be subject to relatively large forces downward due to pressure on the roof. The load thereby added to the existing gravity load would be dependent on whether or not the walls failed quickly enough to provide relief due to internal pressure development.

The type of wind bracing used in slender buildings may be a factor affecting dynamic behavior. Rigid connections frequently are used throughout a structure in order to produce lateral resistance to wind and earthquake forces; but in some cases only certain "bents" are "wind braced", these having either rigid connections or cross bracing with the other bents having flexible or semi-rigid connections. More flexible connections are used in broad tier buildings, as these structures are not usually "wind braced". However, if construction is welded, such buildings may also have continuous or rigid connections.

The effect of walls, floors, and encasement of the members is to strengthen the structural framework. The amount of increase in strength that can be attributed to such effects cannot be readily determined but in general it can be said that at least in the initial stages of deformation the walls and floors must greatly stiffen the structure. The fireproofing around the members also introduces a degree of "composite" behavior.

Component parts of a tier building have been discussed in Chapters I-X. Of particular interest are Chapters IV through VII, and Chapter IX, on beams and girders; roof, wall, and floor panels; connections; columns; and frames and arches, respectively.

No records have been found reporting tests to collapse on actual steel tier buildings. Apparently only a very few tests have been attempted even within the elastic range of the structure. Some experimental work has been done on models, usually stressed below the elastic limit, to determine natural frequencies or the distribution of forces in the frame of the building, but these tests are of little value in predicting ultimate collapse strength because they do not include the effect of the walls, floors, actual connections, dead-load distribution, damping, etc. The consideration of damage caused by wind and earthquake may help determine the more important factors to be considered in predicting the damage caused by a blast. General factors influencing the extent of damage due to a blast may be found in a review of the damage at Nagasaki and Hiroshima.

The load deflection record of a structure under a known blast impulse can be approximated if information is available on the resistance of that structure to lateral deflection, the damping, and the distribution of mass. The resistance and damping must be that of the whole structure and not just the framework alone, because walls, floors, partitions, and fireproofing around the members play a major role during relative motion of the various parts. All the factors mentioned may vary with rate and degree of deformation.

In the case of tall buildings, the natural period of vibration is apt to be considerably longer than the duration of the impulse and such a structure is relatively less responsive to short-duration earthquake and blast loads as compared with structures of greater natural frequency. It has been shown that the peak elastic response may be further reduced by ductile yielding of the material. When all the factors are considered, steel tier buildings appear to be relatively resistant to dynamic loading and a great deal stronger than an analysis of only the framework would indicate.

11:2 Survey of Theory and Experiment.

Reports that have some connection with the prediction of damage due to blast will be considered first. Although relatively few unclassified papers treat specifically of blast, it should be remembered that the many articles on analysis of buildings subjected to earthquakes are equally applicable to blast problems. Baker and his associates (11.2, 11.4) have treated the problem of the high-explosive bomb quite thoroughly. Jacobsen and Wells (11.2) measured the natural frequencies of full-size test structures before and after a blast to determine the loss in stiffness resulting from the blast. Newmark (11.18) has presented an improved numerical procedure for determining the dynamic behavior of structures both in the elastic and plastic ranges. Penzien and Williams (11.24) considered the action of tier buildings subjected to an atomic blast, comparing several methods of analysis and investigating the effect of wall slabs.

For large structures composed of many members the yield point is not well defined. Local yielding or even collapse of individual walls may have little effect on the total resistance of the structure, and for this reason, elastic behavior may at least give some clues related to collapse strength. Tests of the action of tier buildings in the plastic range being unknown, some tests wholly in the elastic range will be included in the references for this chapter.

Baker's early work (11.3) describes full-scale building tests that, although primarily within the elastic region, suggest the increase in strength and rigidity that can be realized from walls, floors, and encasement of beams and columns.

The elastic vibration frequencies of many buildings were determined by the U S Coast and Geodetic Survey (11.33) and by Suyehiro and Ishimoto (11.30). These data are important in themselves; they could also be used to estimate the increased stiffness of structures in the elastic range by comparing the actual period of vibration to the period computed considering only the bare steel framework.

The reduction of peak response and peak stresses due to ductility of the material is studied for buildings in a report by Pan, Goodman, and Newmark (11.23) and a doctoral dissertation by Pan (11.22).

Tests and theoretical investigations involving earthquake and wind damage, while concerned with the general problem of dynamic behavior, provide little information on the behavior of buildings at maximum resistance and in the final stages of collapse. The investigators in the fields of wind and earthquakes are interested in obtaining a design procedure that will result in a building that will resist such forces without structural damage. Nevertheless, something may be learned from the results of the wind and earthquake research.

The report of Biot (11.5) and the discussions that follow it give a fairly complete review of the major investigations in earthquake research. The lateral force code (11.14) proposed by the Joint Committee of the ASCE San Francisco Section and the Structural Engineering Association of Northern California includes a statement and discussion of the lateral-force problem and gives a very complete bibliography of theroetical and experimental reports. Morris's paper (11.16) on the design of wind bracing cites several cases of damage to tier buildings caused by high winds and discusses briefly the significance of the damage. The ASCE report (11.1) on the Florida hurricane describes damage to two tier buildings that had inadequate beam-column connections.

A few of the more important earthquake and wind investigations with models will be listed. The report of Takabeya and Sakai (11.31) describes tests of special plastic models to determine the weakest point in a tier building under earthquake loads. The models ranged in size from one-story, one-bent frames to six- or seven-story bents and up to five bays wide. The models were tested on a shaking table to simulate an earthquake and tests were carried to complete collapse.

Studies of tier building models in forced vibration were made by several investigators. Jacobsen and Ayre (11.11) considered a three-dimensional model of a 16-story building and measured the shears between floors when the model was near resonance. Varying degrees of flexibility in the first story were used. Ockleston (11.20) also tested three-dimensional models, including the effects of damping and distribution of the mass. Ockleston's report gives a very good description of the dynamic action of a structure and includes an extensive reference list.

Morris, Large, and Carpenter (11.17) tested the model of part of the wind bent of a 55-story building. The loads were applied statically and were limited so that the stresses were in the elastic range. The cross-section of the members was H-shaped, and the shape of the building was varied to determine the effect of stepped bents and of openings in the framework. Consideration of these factors makes this series of reports of special value, as these factors are not considered elsewhere (11.17)*

Other miscellaneous reports are listed. Some describe earthquake damage; some propose procedures to analyze dynamic systems; others consider the effect of short duration blasts on buildings and theoretical methods to determine natural frequencies and shear distribution to each floor; and some

*Only one of the series of reports is included in the list of references.

give design recommendations based on experience with wind and earthquake damage. Although most of these studies do not make any definite contribution to the study of blast damage, they do give considerable insight into the overall problem of dynamic behavior and this is important in the blast problem.

11:3 Recommendations for Research.

From this limited survey of the unrestricted, published and unpublished reports bearing on tier buildings, it is quite obvious that the necessary information is not available to determine collapse, assuming such to be possible. To build tier buildings for test purposes is obviously very uneconomical.

RP 11.1--Destructive tests of actual tier buildings scheduled for razing. Actual structures, both of recent and old construction, sometimes must be destroyed in order to provide right-of-way for an express highway or other new construction. It is proposed that certain of these condemned buildings be tested to destruction, recognizing that careful study would be required in each individual case to ascertain whether such tests were feasible and warranted. In some cases actual collapse loads could be obtained. The city governments throughout the country could notify a project headquarters of proposed condemnations and the project, after the preliminary feasibility studies, could then move in and make the necessary tests at the site. The city engineer of Detroit, Michigan, was asked his opinion of this plan and he stated that it was definitely feasible and that he would be very willing to participate in such a program. He stated that he often had knowledge of the condemnation of buildings far in advance of the time of actual razing and that he therefore believed there would often be sufficient time for testing without delay in new construction.

RP 11.2--Destructive tests of models of tier buildings including effects of walls and floors. Models which included the effects of damping, walls, and floors might be tested in such a manner that the dynamic behavior of their prototype could be determined. The test results of the models would have to be well correlated with full-size tests by cooperation with project RP 11.1.

RP 11.3--Utility and repair of tier buildings after partial failure. A study of various degrees of failure in relation to usability of tier buildings should be conducted. The time required for a certain structural building type to be returned to full or partial service after a certain type and extent of damage might be a more rational consideration than actual collapse, which in the case of at least some tier buildings may be unlikely to occur.

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CHAPTER XII

INDUSTRIAL BUILDINGS

12:1 Introductory Statement.

Buildings in which there is a considerable area with no intermediate floor between ground level and roof will be defined as "industrial". There are many types of industrial buildings, including frames with either semi-rigid or rigid connections, buildings with roof trusses and either light or heavy columns, and buildings with arch trusses or arch girders. Examples were described in Chapter III and illustrated in Fig. 3.2. The classification given in Chapter III also distinguishes between "narrow" industrial buildings of one or two bays and "broad" industrial buildings of more than two bays. Building trusses were considered in Chapter IX, and frames and arches in Chapter X. This chapter considers the industrial building as a whole.

The construction of industrial buildings is generally much lighter than that of tier buildings; therefore, the strengthening effects of walls, floors, roof, and other secondary structural elements probably are not as great. However, under blast the load on the frame depends on the dynamic behavior of the roof and wall panels, for if these elements collapse in the early stages of the blast load there may be a great increase in the chance for frame survival.

Columns are the critical members during building sidesway under lateral load, and a study of their behavior in industrial buildings is of special importance. On broad industrial buildings the effect of the downward forces resulting from roof pressures would be relatively greater than the lateral forces, but both types may induce column collapse involving sidesway. In addition, the downward forces may be largely relieved by internal pressure in cases where wall panels collapse quickly.

12:2 Survey of Theory and Experiment.

A review of the damage at Hiroshima and Nagasaki has, of course, given considerable information about modes of failure of industrial buildings. The U.S. Strategic Bombing Survey report (12.28) is noteworthy, and Bowman (12.8) has presented a concise summary. Baker's review (12.3) of high-explosive bomb damage is valuable as a means of comparing damage caused by the two different means. Newmark (12.19) and Penzien and Williams (12.23) considered the dynamical problem of structures with particular attention to atomic blast. One of the many reports by Bleakney (12.6) is cited as an example of shock-tube tests that have provided quantitative information on pressure-time relations pertaining to blast loads on industrial buildings.

Baker (12.4) and Roderick (12.24) describe full-size steel frames tested under static load conditions. Other tests by the same authors and their associates are reviewed in Chapter 1X on frames. Johnson (12.14) gives a description of a nondestructive loading test on an all-welded industrial building (similar to type "a" in Fig. 3.2) in order to determine the resistance of the structure to wind and earthquake.

Cissel and Legatski (12.9) give results typical of tests to failure of various light gage metal arch structures (Fig. 3.2j and k). Wessman (12.30) reports on the destructive tests on a portable hangar with an arch span of 194 feet and a rise of 45 feet. Alt and Harris (12.1) have studied snow load distributions as a cause of the failure of similar-type structures.

Due to the similarity between the blast problem and the wind and earthquake problem, the results of investigations on buildings subjected to wind and earthquake forces may be of interest. Ayre (12.2) reports model tests in the elastic range on a one-story building in which damping and resonance were considered. The tests were entirely within the elastic range. Biot (12.5) presents theoretical and experimental methods for obtaining the action of the simple frames during earthquakes, and the discussions that follow this report present an over-all description of the dynamical problem with a long list of references on the subject. Jennison (12.13) compares model bent test results with theoretical predictions.

The joint committee of the San Francisco Section of ASCE and the Structural Engineers Association of California (12.15) included, in their recommendations for a lateral force code for earthquakes and wind, a list of important papers on this subject. Ockleston (12.20) has reported on theoretical and experimental investigations to determine the effect of earthquakes on framed buildings with various degrees of damping and mass distribution.

Pan (12.21) and Pan, Goodman, and Newmark (12.22) showed how the peak elastic response and the peak stresses might be reduced due to the ductility of the material. Sezawa and Kanai (12.25) give a theoretical mathematical analysis of buildings, considering the effect of different conditions for the column-base connection. A comparison of continuous and trussed industrial steel frame building constructions under blast load is being made at the University of Michigan (12.15).

Suyehiro and Ishimoto (12.26) and the U.S. Coast and Geodetic Survey (12.27) measured the periods of vibration of existing structures. Correlation of these results with the calculated natural period of the bare framework would determine the increase in stiffness and damping due to walls, floors, roof, and other nonstructural elements of the building. Several papers (12.10, 12.12, 12.18, and 12.31) discuss and/or promote various lateral-force analyses for earthquake and wind.

Baker, Williams, and Lax (12.3), Bondy (12.7), and Wahl (12.29) report on the effects of short-duration blasts on buildings. Because this type of blast is usually local, the over-all effect on the building is not important to the problem at hand, but the local effects on walls, for instance, may be very pertinent. The spreading of failure from its point of inception by successive interaction of adjacent frames, as described by Baker et al. (12.3), is of general interest.

Dryden and Hill (12.11) describe the results of low-velocity wind-tunnel tests conducted on models of a mill building (Fig. 3.2b) both with and without a monitor. Pressure maps are given showing high suctions on certain parts of the model's exterior. This paper is included as one example of a considerable number in the same category, which presumably have at best only an indirect relation to pressure distribution during atomic blast.

12:3 Research Proposals for Industrial Buildings.

RP 12.1--Destructive tests of actual industrial buildings scheduled for razing. The testing of condemned industrial buildings should be combined with RP 11.1, the testing of tier buildings. A statement of the proposal is given under tier buildings.

RP 12.2--Destructive tests of models of industrial buildings including effects of roof and walls. Model testing of industrial buildings to collapse, if correlated with full-sized tests and including the effects of walls, etc., would probably provide valuable information at relatively small expense. Investigations could be made of the many various types, sizes, and combinations of industrial buildings that exist in practice.

RP 12.3--Failure levels, operability, and repair of industrial buildings after partial failure. An independent or combined project could determine the failure levels, as is proposed in RP 11.3 for tier buildings. The operability and repair of crane facilities would be an important added consideration in the case of industrial buildings.

RP 12.4--An investigation of actual failures of industrial buildings under snow or wind load. Failure of flat-roof and flat-arch-roof structures is not uncommon during particularly heavy snowfalls. The load in such cases may be known within reasonable limits. The prompt investigation of such failures would add to the knowledge of the ultimate strength of structures. Since the realization of conditions leading to such failures would be largely accidental and sporadic, this suggestion might well be combined with project RP 12.1. In the case of failures due to wind, the load at failure would be more uncertain than in the case of snow, and the occurrence of failures at least equally sporadic. The possibility of reviewing records of past failures should also be exploited.

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CHAPTER XIII

BRIDGES

13:1 Introductory Statement.

This chapter should be considered as supplementary to earlier chapters, IV, VI, VII, IX, and X, in which the behavior of members, trusses, and frames has been reviewed. The present chapter will be limited to a brief review of recorded instances of actual bridge failures or tests of specific bridges or bridge models.

13:2 Survey of Theory and Experiment.

The failure of a bridge is usually due either to (1) brittle fracture or (2) plastic yield and/or buckling of a primary member. Brittle failure has been observed most frequently in welded structures under severe low-temperature conditions combined with other adverse factors. Bridges, unlike building frames which are covered with walls and roofs, are usually exposed to extremes of temperature and thus are more susceptible to brittle failure. A brittle failure of a welded Vierendeel bridge at Hasselt, Belgium, has been reported and the causes of failure discussed (13.1). Recently Louis (13.15) has reported on 14 cases of brittle failure in bridges, with a discussion of probable causes of failure. Bijlaard in a recent paper (13.4) explains in general the brittle failure of welded bridges. Brittle failure may also result from repeated loading, but this is not a concern of this report.

Although brittle failures have occurred, most steel structures, including bridges, fail due to yield and/or buckling of a primary member. The failure of certain members of a bridge does not necessarily result in the collapse of the whole bridge. A few interesting cases of bridge accident have been recorded by the California Department of Public Works (13.7, 13.8), in which it is reported that main members of the trusses were severely damaged by trucks, but the bridges still stood or even carried light traffic.

Many bridges, most of which are scale models, have been tested to study their static or dynamic behaviors. Chai, in his doctoral dissertation (13.9), tested a 1/20-scale model of a single-track railroad bridge to study the load distribution from the floor system through the main trusses, including the upper, lower, and lateral bracings. Investigations on the behavior of floor beam hangers under static and dynamic loadings have been reported by the AREA (13.21, 13.25). The tests on army Bailey bridges have been discussed first by Firmage (13.12) and more recently by Stegmaier (13.22). The Bailey bridges tested were the half-through type; hence, they failed by lateral buckling of the top chords (13.12). Saller has discussed a test of a Russian bridge (13.20) in which two diagonals were stressed beyond the elastic limit and then buckled extensively.

Highway bridges have been tested by the British Building Research Station (13.10). The first part of this research was the testing of actual cast-iron girder bridges and masonry arch bridges, some of which were tested to failure. The second part involved laboratory tests on scale models of bridge decks. Tests were carried up to and beyond cracking of the reinforced-concrete slabs and composite beam action was also studied.

Several series of highway bridge models have been tested at the University of Illinois (13.16, 13.18, 13.24). Penman (13.18) has tested 12 1/4-scale models of simple-span multiple-stringer highway bridges. The tests were carried to final failure of the concrete slab after the yield point of the beams had been reached. Ward (13.24) has tested a two-span continuous I-beam bridge with the primary failure of the bridge due to the yielding of the interior beams. Newmark, Siess, and Peckham (13.16) have reported on test results of five 1/4-scale skew I-beam bridges in which the slabs were tested to failure. Many tests have been made on the impact in railway bridges, most of them by AREA, in which the effects of ballast, composite beam action, and variation between electric and steam trains have been considered. Only two of the AREA reports are listed here (13.2, 13.3), since these tests in the elastic range have only an indirect bearing on the ultimate strength of bridges. Looney (13.14) has reviewed the impact behavior of railway bridges, including tests on 45 bridges.

Fraenkel (13.13) has discussed in detail the ultimate strength of truss and girder bridges under overhead atomic explosion with application to seven specific cases. The loadings on the bridge were first determined for the explosion of a nominal atomic bomb 2000 feet above the bridge, considering the reflection of the shock wave from the water below the bridge. The behavior is investigated in both the elastic and plastic ranges. Patterson (13.17) has reported on the destruction of a three-span railway bridge during the war. The aerodynamic failure of the Tacoma-Narrows Suspension Bridge has given impetus to studies involving the dynamic behavior of such structures, as, for example, the work of Farquharson (13.11), Steinman (13.23), and F. Bleich et al. (13.5).

13:3 Proposed Research Projects.

RP 13.1--Destructive tests of actual bridges scheduled for razing. Specific research projects relating to the ultimate strength of bridge components and the basic behavior of girder and truss bridges have been suggested in connection with earlier chapters.

It is suggested that a project similar to that proposed in connection with buildings (RP 11.1) be conducted--possibly in combination with the building project. State highway engineers and chief engineers of railways would be contacted so that information regarding any bridges about to be scrapped or condemned would be transmitted to the research agency in charge of the project. The American Railway Engineering Association would probably be willing to cooperate. Wherever feasible, these bridges would then be field-tested to destruction under static or dynamic conditions. One procedure that might be followed would be to jack up the supports of a particular bridge and provide for the simultaneous sudden drop of the complete bridge from its elevated position onto the supporting piers.

RP 13.2--Destructive tests of models of bridges, including effect of floor and bracing systems. As in proposals RP 11.2 and 12.2, on tier buildings and industrial buildings, respectively, tests of models of bridges, if shown to be representative of the behavior of larger structures, would permit evaluation and comparison of many different types at relatively small expense as compared to tests of full-size structures.

RP 13.3--Failure levels, operability, and repair of bridges after partial failure. The effect of partial failure in reducing bridge capacity and the time requirements for making repairs to bridges suffering partial failure should be evaluated.

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CHAPTER XIV

TANKS

14:1 Introductory Statement.

Steel tanks represent a special class of structure, but they are of particular importance in industry because of their use in storing vital materials such as water, oil, and gas, and as functioning units in chemical process industries.

Tanks should be considered with regard to their elevation: (1) elevated tanks, such as are supported on tubular columns, rigid frames, or space truss frames; (2) surface tanks, in which the support is at or slightly below ground elevation; and (3) subsurface tanks, which have been buried for protection against air attack.

Two types of failure should be considered: (1) collapse failure of the tank proper, due to the increase in all-around external pressure; and (2) failure of the supporting structure (applying only to elevated structures and already reviewed in part in Chapters VII, IX, and X on trusses and frames, respectively).

Tanks have been classified further according to their external shape, including short cylinders with flat ends, such as are used in surface storage applications, long cylinders with hemispherical or other types of outwardly convex end, spherical, and other special shapes.

14:2 Survey of Theory and Experiment.

Steel tanks are made of thin steel sheets, plain or stiffened. Books by Flügge (14.6) and Timoshenko (14.23, 14.24) are listed as references for bending and buckling of thin shells of various shapes. Buckling of thin circular cylinders and compression, bending, and shear of thick-walled round tubes are studied theoretically by Plantema (14.14) with reference to available test results.

Early theories and tests for the collapse strength against buckling of thin cylindrical shells under external pressure were well reviewed and criticized by Windenberg and Trilling (14.27) in 1934. (See also Reference 14.1 for other more recent work at the Model Basin.) Thin-walled monocoques have been covered by Hoff (14.7), who includes an historical survey. In 1941 Sturm (14.20) also studied the collapse strength of thin-walled cylinders, considering special factors such as plastic behavior and out-of-roundness, with comparison between theoretical and test results. Cornell (14.4) has reported the test results on 258 specimens of long cylindrical shells under external pressure. Wilson and Olson (14.26) have tested cylindrical shells under axial compression and transverse loads. Cylindrical shells with transverse stiffeners under hydrostatic pressure have been studied by Kempner and Salerno (14.8) analytically for plastic behavior, and by Salerno, Levine, and Pulos (14.18) for buckling behavior. The later has been simplified by Salerno and Levine (14.17) by using a hydrostatic buckling-pressure parameter.

Theory of buckling of thin spherical shells has been discussed recently by Mustari and Surkin (14.12) and by Müller (14.11). Chicago Bridge and Iron Company (14.28) has tested spherical shells under external pressure to failure. Spherical shells subjected to suddenly applied external pressure have been studied by Kito in Japan (14.9).

Elevated tanks under sudden earth shock have been studied both theoretically and experimentally by Brown (14.3), Ruge (14.16), and Williams (14.25). In Japan, this problem has been studied recently by Takahasi and Suzuki (14.21) for horizontal vibration and by Takahasi and Tazima (14.22) for rotational vibration.

14:3 Further Research.

In certain special instances the testing of full-size tank structures might be feasible, but the desirability of sponsoring any specific project on this special subject is questionable. The more complete review and correlation of existing information contemplated for the final reports of this project may reveal needed areas of investigation, but no proposals for research will be made at the present time.

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APPENDIX A

SUMMARY CHART OF PROPOSED RESEARCH PROJECTS

The following chart, Fig. A.1, shows all the proposed research projects grouped according to their mutual interrelationship. Projects concerning the fundamental behavior of individual structural members and connections are at the ~~left~~ top side of the chart, leading in logical sequence to tests of complete bridges and buildings and studies of repair possibilities at the ~~extreme right~~ bottom. The first number assigned to each project refers to the chapter wherein a more detailed description has been given.

It is recognized that the over-all research program as proposed herein is of considerable magnitude and would entail expenditure of several million dollars. Many different organizations would need to be called upon to take various assignments within the over-all program. It would not be desirable to start all the projects at once, and in view of the over-all cost and greater importance of certain projects it is recommended that a committee be set up to assign project priorities.

It is also recognized that there may be projects already under way which have been classified by military agencies. Each project would need to be evaluated carefully by the cognizant committee. This committee would also receive the detailed proposals forthcoming as a result of invitations sent to various university research institutes, etc., and would recommend final allocation according to demonstrated interest, adequate facilities, ability of personnel, and satisfactory budget.

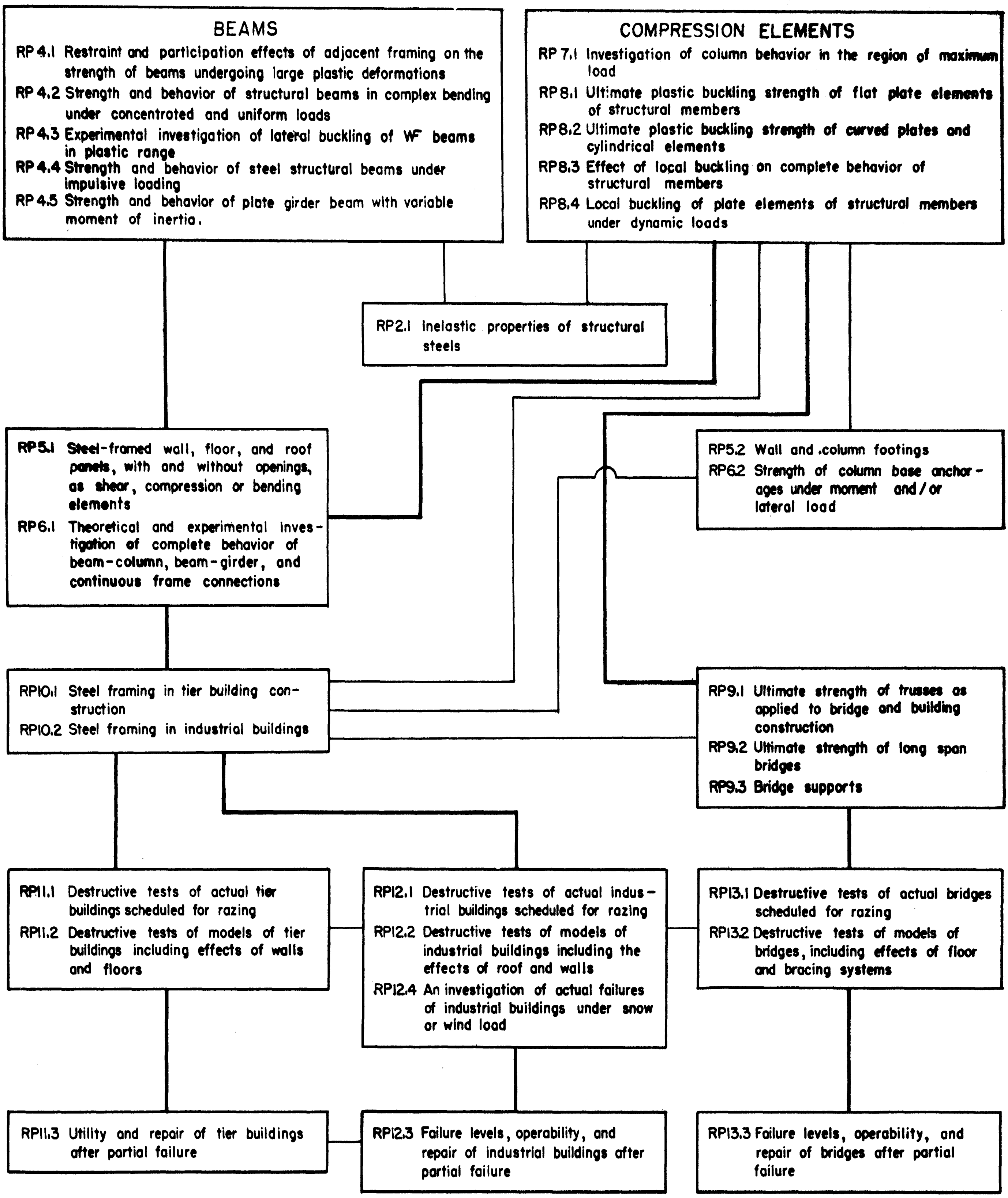

FIG. A.1

SUMMARY CHART

OF

PROPOSED RESEARCH PROJECTS