報告番号 ※甲 乙 第 2759 号

主論文の要旨

題名

ULTIMATE STRENGTH VARIATIONS OF STRUCTURAL STEEL MEMBERS

（鋼構造部材の極限強度の変動特性に関する研究）

氏名 伊藤義人

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鋼構造部材の耐荷力問題は，古くから多くの実験的，理論的研究が行われており，各種の設計示方書にその成果が取り入れられている．近年，設計法の合理性を高めるため極限強度を基準にして，そのばらつきをも信頼性理論をもとにして取り入れられようとしている．この時，鋼構造部材の座屈現象に関する耐荷力問題を設計に十分精度よく取り入れるためには，基礎的な実験データが重要な役割を果たす．

確率論的見地から行った実験的，理論的研究は，中心軸圧縮柱については，すでに2，3行われているが，鋼はりについてはこれまでほとんど行われていない．本論文では，鋼はりについて強度のばらつきに着目した3種の実験的研究と内外の実験データを収集評価したデータベースアプローチを行っている．さらに，鋼柱，鋼板については，内外の実験データを収集評価したデータベースを作成し，それをもとにして，ばらつきに着目した強度特性についての検討がなされている．

本論文は，2編からなり，第1編は4章からなり鋼はりの横ねじれ座屈強度に関しての変動特性に着目した実験的研究を取り扱っており，第2編では，全体の結語を含めて5章からなり，内外の鋼構造部材に関する実験情報の数値データベースの作成と，これを利用した鋼構造部材の強度特性についての研究を取り扱っている．

まず，第1章の緒論では，鋼はり，鋼柱，鋼板の耐荷力に関しての諸問題について，既往の研究について述べるとともに，本論文の目的と構成について示している．

第2章では，圧延はりの横ねじれ座屈強度特性，とくにその変動性に与える要因を検討するための実験的研究を行った．25本の7m長の圧延H形鋼（H－200×100×5.5×8mm）の原材から，スパン長2.6m，2.0m，および1.5mの横ねじれ実験はりをそれぞれ切り出し，合計75本のはりについて両端単純支持，スパン中央集中載荷実験を行った．同時に，各はりの初期変形，材料強度および残留応力を測定し，横ねじれ座屈強度の変動との関連を検討した．また，既往の実験結果および設計基準強度曲線との比較を行った．ここで得られた結果は，次のようである．

1) 各々25本の3種の長さをもつ鋼はりの強度の変動係数は，スパンの長い方

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からそれぞれ0.74，0.048，0.045であり，スパンの長いものほどばらつきが大きい．

2) 鋼はりの横ねじれ座屈強度の変動性に最も大きな影響を与える要因は，実測の降伏点応力と断面寸法から計算した全塑性モーメントである．全塑性モーメントの影響は，細長比パラメータが大きくなると小さくなる．

3) 今回の実験はりにおいて，実測の全塑性モーメントで無次元化した最大モーメントは，無次元化していない最大モーメントの変動係数より24～45％小さくなり，その度合は，短い鋼はり程小さい．

第3章では，溶接はりの横ねじれ座屈強度について，第2章で用いた実験装置と同じものを使って，実験的研究を行った．断面は，H－250×100×6×8㎜であり，5.46mの34本の原材を溶接組み立てし，そこより2.6mと1.8mの実験はりを合計68体切り出した．同時に，圧延はりと同じように各はりの初期変形，材料強度および残留応力を測定し，横ねじれ座屈強度の変動との関連を検討し，圧延はりの結果と比較した．ここで得られた結果は，次のようである．

1) 溶接はりの横ねじれ座屈強度の変動性に最も大きな影響を与える要因は圧延はりと同様，実測の全塑性モーメントである．

2) 実測の全塑性モーメントで無次元化した最大モーメントの変動係数は，無次元化していない最大モーメントの変動係数より1.8～22.6％小さい．

3) 同一の荷重・境界条件の下では，溶接はりのほうが圧延はりに比べて平均強度が低く，また，ばらつきは大きい．

第4章では，21体の2スパン連続ばりの横ねじれ座屈強度についての実験的研究を行い，第2章，第3章の単純ばりの変形特性，強度特性と比較している．実験に用いた断面は，第3章の溶接はりと同一のものを用い，荷重は，片側スパンの中央に集中載荷した．ここで，得られた結果は，次のようである．

1) 2スパン連続ばりの横ねじれ座屈強度は，隣接スパンの拘束のため，単純ば

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りと1端単純他端固定のはりの強度の中間に位置している.

2) 連続ばりの強度を各種の有効長さを用いて評価を行い，実験から得られた圧縮フランジの水平曲率の変曲点が連続ばりの強度を評価する際に有効であることを明らかにした.

3) 連続ばりと単純ばりの強度の変動性は，本実験から見る限り差異はない.

第5章では，鋼構造部材の実験情報のデータベースNDSSの作成とその構成について述べている. 実験情報をデータベース化することによって，内外で行われた実験結果を統一的に扱うことができ，一般に膨大な労力と資源を必要とする実験の重複を避けることができるとともに実験の必要な範囲を明らかにすることができる. また，設計示方書の改定の基礎的資料および理論研究者のための基礎データともなる. 本章では，データベースの作成について述べたあと，引張試験から得られる鋼構造用鋼材の材料強度に関する実験情報の数値データベースを利用して種々の統計処理を行い，鋼構造用鋼材の材料強度特性を明らかにした.

ここで得られた結果は，次のようである.

1) 鋼柱，鋼はり，鋼板の耐荷力および鋼構造用鋼材の材料強度に関する実験情報のデータベースを確立した. すでに，一部の実験情報に関する加工データは，内外の研究者に提供した.

2) 鋼構造用鋼材の材料強度に関する変動特性および適合分布曲線を実験データをもとに明らかにした. 材料特性値の中で最も重要な定数である降伏点応力の実測値と公称値の比の平均値と変動係数は，それぞれ1.168と0.111である. また，その分布形は正に偏心している.

第6章では，中心軸圧縮柱の強度特性について，1665体の実験データをもとに，種々の処理を行い，断面形状，製作方法，鋼種などにより，強度特性がどのように変わるかを明らかにした. また，ヨーロッパ，米国で提案されている複数設計曲線と実験データを比較し，その妥当性を検討した. その後，鋼柱を断面形状，製

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作方法，鋼種などにより，3つに分類し，各々に平均値曲線と下限を示す曲線を提案した．ここで得られた結果は，次のようである．

1) 各種の断面形状の鋼柱の中心軸圧縮強度を比較した結果，一般に，溶接のH形断面柱は，圧延のH形断面柱に比較して低めの平均強度と大きめのばらつきを有している．

2) 鋼柱の設計における複数設計曲線の適用の合理性を明らかにした．

3) 新しい分類の複数設計曲線を実験データにもとずき提案した．

第7章では，鋼はりの横ねじれ座屈強度に関する検討を，544体の内外の実験データをもとに行った．鋼はりは，鋼柱と異なり一義的に強度を表わすことができる形状パラメータがなく，多くのパラメータが強度に影響を与える．製作方法，断面形状，荷重・境界条件によって強度がどのように影響を受けるかを検討した．

ここで得られた結果は，次のようである．

1) 鋼はりの耐荷力強度を評価する時，圧延はり，溶接はりに対しては，全塑性モーメント，プレートガーダーに対しては，降伏モーメントで無次元化するとばらつきを小さく抑えることができる．

2) 溶接はりの横ねじれ座屈強度は，圧延はりに比べて低めの平均強度と大きなばらつきを有している。

3) 鋼はりを圧延はり，溶接はり，プレートガーダーの3つに分類し，各々について，実験データをもとに強度の平均値曲線と下限値曲線を求めた．

第8章では，一様圧縮を受ける無補剛鋼板の座屈強度特性を793体の実験データをもとに求めた．無補剛鋼板の境界条件，残留応力および初期変形による強度特性の違いを明らかにした．ここで得られた結果は，次のようである．

1) 非載荷辺が単純支持の単一板と正方形箱形断面の板の平均強度に差はない．

2) 初期変形による鋼板の強度の低減を実験データから明らかにした．

3) 鋼板を残留応力のあるなしによって2つに分類し，各々について平均強度曲

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線と下限強度曲線を求めた．

第9章は，結語であり，全体の要約と各章で得られた結論のうち重要なものを再記述し，その後，本論文の総括を行っている．最後に，本論文に関連した分野において今後の研究が必要な問題について述べている．

図・本館

ULTIMATE STRENGTH VARIATIONS OF STRUCTURAL STEEL MEMBERS

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NOV. 1984

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ULTIMATE STRENGTH VARIATIONS OF STRUCTURAL STEEL MEMBERS

by

Yoshito ITOH

ABSTRACT

This dissertation presents a study of ultimate strength variations of structural steel members, i.e., columns, beams and unstiffened plates. In Part I, statistical buckling tests were carefully carried out for 75 rolled beams and 68 welded beams together with measurements of initial imperfections. The influences of the variation of various parameters on the scattered test points of the lateral-torsional buckling were studied. The buckling behavior, buckling strength and the strength variation of rolled and welded beams were compared.

Twenty-one two-span continuous beams were also tested to compare the test results with those of simple beams.

In Part II, a data base approach of structural steel members was described. The numerical data base regarding steel columns, steel beams and steel plates was established on a world-wide basis. Material properties from tensile tests in Japan were also stored in the data base. This data base provides fundamental information for code writers and researchers.

Variations and fitted distributions of material properties were obtained from the 2154 test data in the data base.

Statistical assessments of columns, beams and unstiffened plates were made to compare available formulas. The effects of initial imperfections such as residual stresses and initial crookednesses were reviewed

considering their variations. New multiple design formulas for columns, beams and unstiffened plates were presented to explain the test data from a statistical viewpoint more accurately.

Part I Experimental Study on Laterally Unsupported Beams

Chapter 1. INTRODUCTION

1-1 STATEMENT OF THE PROBLEM

Columns, beams and plates are fundamental types of members in a steel structure and their instability is a major case of failure.

These structural elements always have a number of imperfections, such as i) variations in cross-sectional properties, ii) initial out-of-straightness, iii) residual stresses, iv) variation of material properties, and v) load eccentricity. Many design specifications are now in the process of introducing new editions based on modern limit states principles based on the reliability concept (1-12). However, it is difficult to take account of all the above imperfections in the formulation of design rules. This fact has increased the need for a large volume of good quality tests since such information is essential for the satisfactory calibration which insures an appropriate level of structural reliability. The larger data base can justify a more reliable resistance factor.

Experiments are generally expensive and require much effort and good equipment to obtain accurate data. During the past thirty years a large volume of structural tests have been conducted in many countries. However, most tests have been performed for various purposes using the deterministic approach, and reports of these works are fragmented and often published papers do not provide sufficient data.

Collecting detailed test data and establishing this data base on a world-wide basis is required.

Some experiments on steel columns have been carried out from a statistical viewpoint and a comparatively large volume of tests for pin-ended, centrally loaded columns is available. Computer simulations on

steel columns were also performed. The European Convention for Constructional Steelwork (ECCS) (11) and the Structural Stability Research Council (SSRC) (12) have both independently recommended the multiple column curves to accurately formulate the column strength.

On the other hand, no beam experiment had been performed from a statistical viewpoint. Careful tests were needed to obtain reliable statistical data on initial imperfections such as residual stresses in beam-type members and initial out-of-crookednesses, and also to provide statistical data for laterally buckling strength of beams under the nominally identical conditions of loading, supporting and member size.

The problems studied in this dissertation include strength characteristics, especially variations, of structural steel members. Two method for the lateral-torsional buckling of steel beams, i.e., experimental and data base approaches, were used. A data base approach was performed for centrally loaded steel columns and steel plates which subjected to in-plane compression.

1-2 REVIEW OF PREVIOUS RESEARCH

In this section, previous investigations of structural steel members from statistical or probablistic approaches are mainly reviewed. The historical review of deterministic investigations of steel beams (12,13,14,15,16), steel columns (17,18,19,20) and unstiffened plates (21,22,23) have already been made by many researchers, and some of them are mentioned in subsequently related chapters.

Lateral-Torsional Buckling of Beams --- The lateral-torsional buckling behavior of narrow rectangular beams was first treated independently by Prandle and by Michell in 1899. Timoshenko presented the fundamental differential equations for doubly symmetric I-beams and solved them by an energy approach in 1905. Since then, a number of investigations of the elastic and inelastic buckling behavior of steel beams have been made theoretically and experimentally on various conditions, because the lateral-buckling strength depends on a number of parameters such as support and loading conditions, types of cross-sections, web deformations, material and geometrical imperfections. The different slenderness parameters for steel beams are used in the design codes.

In 1977 Fukumoto and Kubo (24,25) surveyed lateral-torsional buckling tests conducted at various institutions. A summary of the results was presented in tables and figures for 159 rolled beams and 119 welded beams and girders. From the statistical evaluation, two different formulas were suggested for rolled and welded beams, respectively, in opposition to the use of the single curve employed by the European Convention for Constructional Steelwork (ECCS).

Figure 1-1 shows a bar diagram of the number of beam tests of full-scale steel beams included in this dissertation, in which the abscissa

is the year of the original reference papers published (26-53). From 1964 to 1977, the experimental investigations of about 20 sources using the deterministic approach were mainly reported. These experiments were performed for various purposes, including the problem of the rotation capacity and the adequate spacing of lateral support, which were investigated in Ref. (41-46). Detailed support and loading conditions are described in Chapter 8.

Recently, statistical experiments on rolled and welded beams were carried out by Fukumoto, Kubo and the author (48,49,50), and are described in this dissertation.

Centrally Loaded Columns --- The centrally loaded, hinged column is probably the most exhaustively investigated case among structural stability problems from deterministic and probablistic approaches. The problem of column stability in structural members was first treated by Leonard Euler. He derived a rigorous solution for the elastic column in 1744.

More than one hundred years later, the tangent modulus theory was established for the inelastic buckling strength of columns by Engesser. In 1885, he presented the reduced modulus theory in reply to Jasinsky's criticism. The reduced modulus theory was considered to be a theoretically correct solution for column strength. However, test results of most columns were closer to the load suggested by the tangent modulus theory.

In 1947, Shanley (54) gave an explanation for this paradox using his famous model, which gave the bifurcation point of a centrally loaded column as the tangent modulus load. It was realized that the tangent modulus load gives a lower bound and the reduced modulus load an upper bound to column strength.

Since then, theoretical and experimental studies of column strength

have taken into account geometrical and material imperfections such as initial curvatures and residual stresses. A summary of researches done up until the early 1960's is described in Refs. (17) and (18).

Figure 1-2 shows a bar diagram of the centrally loaded column tests (55-84) included in this dissertation. From 1950 to 1970, many tests were performed to determine the influence of the material and geometrical imperfections at Lehigh University.

Beginning in 1958 a comprehensive series of tests of centrally loaded steel columns was carried out by Technical Committee 8 (stability) of the European Convention for Constructional Steelwork (ECCS) which had been founded in 1955 (55). Seven European countries and U.S.A. took part in the testing of more than 1,000 steel columns under similar conditions. The ECCS tests account for about 60 percent of the data in Fig. 1-2.

In 1972, Schulz (85) studied column strength by using a deterministic approach which relied on computer analysis of input data based on a statistical evaluation of the ECCS test results such as geometrical imperfections, yield stresses and residual stresses. Strating and Vos (86) investigated column strength employing a probablistic approach in 1972. Column strength of an IPE 160 section was calculated by using a Monte-Carlo simulation method, in which information concerning the column variables was also obtained from the above ECCS tests.

Based on the test results and theoretical studies, ECCS (11) published (1975) design recommendations for axially loaded members, which proposed multiple column curves depending on shapes, steel grades and fabrication procedures. ECCS believes that accurate and economical design requires the use of a multiple column curve concept.

In 1972, Bjorhovde (17) presented computer simulations of both

deterministic and probablistic approaches to arrive at more rational and adequate means of assessing the strength of centrally loaded, pinned-end, initially curved, prismatic steel columns. The deterministic column strengths of 112 columns, which represent rolled and welded wide-flange and box shapes of a variety of steel grades, sizes, manufacturing methods, etc., were analyzed. The initial out-of-straightness was assumed to be 0.001 ℓ (ℓ = column length). The uncertainties as to column strength could be reduced by defining three subgroups, each of which was represented by a single average curve.

The Structural Stability Research Council (SSRC) (12), formerly the Column Research Council (CRC) founded in 1944, presented a group of three column curves (Curves 1, 2 and 3) in the Third Edition of the SSRC Guide from the above study (17). Bjorhovde also presented the multiple column curves (Curves 1P, 2P and 3P) which were obtained from the probablistic investigation.

Since 1972 a series of experimental studies from the statistical approach has been performed by Aoki and Fukumoto (71,72). 60 rolled H-section columns and welded 108 H-section columns were tested and the test results were analyzed from a probablistic viewpoint. In 1979, Fukumoto and Kajita (73) investigated experimentally the strength variation of 20 welded box columns from the statistical viewpoint. In 1973, Lenz (70) collected 137 column tests at Lehigh University and 269 European column tests, and compared several methods of prediction of column strength where based on probablistic analyses using these column data. In 1981, Hall (87) proposed a steel column strength criteria from the statistical manipulation of 393 test results.

Rectangular Plates in Compression --- The stability of steel plates

which are subjected to in-plane compression is a problem that has continued to attracted considerable attention. In 1891, Bryan first calculated the elastic critical stress for a simply supported plate. The behavior of rectangular thin-plate elements subjected to in-plane compression has been investigated for the aircraft industry in early studies. The buckling strength of rectangular plates in a number of different cases of loading and edge restraints has been studied by using the elastic small deflection theory (88,89). The von Kármán equations provide the plate behavior under conditions of large deflection. Since these equations are complicated, Timoshenko (90) gave a simple example of energy methods to solve the equations.

In 1932, von Kármán suggested an approximate formula for the "effective width" concept of which is still in use in practical design. Since then, Winter (91,92), Möller (93), Dwight (94), Usami and Fukumoto (95), etc. modified the effective width formula. Lind (96) gathered experimental data for 336 light gage steel plates and proposed the modified effective width formula. In 1975, Faulkner (22) made an extensive review of effective plating as well as a list of important effective width formulas, and approximated the coefficient of variation for his mean curve.

Figure 1-3 shows a bar diagram of the number of the 793 unstiffened steel plate tests (95,97-123), compressed uniformly, which are dealt with in this dissertation. No statistical experiment was performed until now. From 1962 to 1979, more than 20 % of the specimens in Fig. 1-3 were tested in the Cambridge University of United Kingdom (97,101,106,109,110,111). The 53 % of the specimens were tested on single plates and 13 % on cruciform columns. The remaining specimens tested on box columns.

1-3 OBJECTIVES AND SCOPE

The aim of this dissertation is to investigate the characteristics of ultimate strengths of structural steel members, i.e., columns, beams and plates. Three experimental programs for the lateral-torsional buckling behavior of steel beams and the data-base approach for ultimate strengths of steel columns, steel beams and steel plates are presented.

All the work in this dissertation is claimed as the author's original.

In Chapter 2, an experimental study of laterally unsupported beams is described from the statistical consideration. Nominally identical twenty-five rolled beams for each group of three different lengths are tested under a concentrated load applied vertically at the midspan of the compression flange. The variation of the geometrical and material imperfections are measured and the influence of the variation of various parameters on the scattered test points of the lateral-torsional buckling are reviewed. The obtained test results are compared with the existing test data and design formulas.

An experimental investigation of laterally unsupported welded beams from the statistical consideration is described in Chapter 3. Thirty-four nominally identical welded beams for each group of two different lengths, are tested under a concentrated load applied vertically at the midspan of the compression flange. In order to compare the buckling behavior between welded and rolled beams directly, geometrically similar cross-sections are used. These test results are compared with those of the test data mentioned in Chapter 2.

INTRODUCTION

Chapter 4 describes an experimental study of lateral-torsional buckling tests on welded continuous beams. Twenty-one two-span continuous welded beams are tested under a concentrated load. In order to compare the buckling behavior of simple and continuous welded beams directly, the same cross-sections adopted in Chapter 3 are used. These test results are compared with the calculated results based on the effective length concept and on the existing design method. The test results are also compared with those of rolled and welded beams obtained in Chapter 2 and 3.

In Chapter 5, a basic consideration is described for the data-base NDSS (Numerical Data-Base for Steel Structures) regarding the ultimate strength of steel structures and structural components. A computer program management strategy is prepared together with statistical and graphical packages for the processing of the data. Statistical results of the mechanical properties of structural steels are discussed in Chapter 5.

An evaluation of column strength by the data-base approach is presented in Chapter 6. A total of 1665 individual column test results have been surveyed on a world-wide basis and stored on the NDSS system. Chapter 6 summarizes the test data editing, searching, retrieval and management that have been carried out on steel columns. After various computational and statistical manipulations, the information on steel column tests are evaluated and comparisons are made statistically between the surveyed test data and the ECCS and SSRC multiple column curves. Proposals for new multiple column curves are also made possible from the experimental data-base correlations.

Chapter 7 describes an evaluation of beam strengths on the lateral-torsional buckling from the data-base approach. A total of 544 individual beam test results are stored on the NDSS system. The statistical results are compared with available beam strength formulas. New multiple beam formulas which more accurately explain the test data from a statistical point of view are also proposed.

In Chapter 8, the ultimate strength of uniformly compressed plates is discussed from the data-base approach. A total of 793 individual plate test results are surveyed and stored on the NDSS system. Comparisons are made statistically between the reviewed test data and existing theoretical and empirical formulas for the ultimate strength of plates. New multiple plate formulas are also proposed.

Finally, Chapter 9 consists of a summary of this dissertation, conclusions for each chapter and a special section which reiterates some of the more important conclusions, and a proposal for future research subjects which relate to the topic of this dissertation.

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Fig. 1-1 Histogram of Beam Data in NDSS

Fig. 1-2 Histogram of Column Data in NDSS

Fig. 1-3 Histogram of Plate Data in NDSS

Chapter 2. STATISTICAL STUDY OF EXPERIMENTS ON ROLLED BEAMS

2-1 INTRODUCTION

Structural beams always include a number of imperfections. These include geometrical imperfections such as variations in cross-sectional dimensions, initial crookednesses in deflection and twist, and load eccentricities and material imperfections such as variations in yield stresses and residual stresses.

The effect of residual stresses upon the inelastic lateral-torsional buckling strength has been studied intensively for rolled and welded beams by several investigators (2,3,4,8,14). In recent studies biaxial bending analyses were extended to include the inelastic behavior of beams with geometrical and material imperfections (1,13).

In Refs. 5 and 6, an extensive survey of literature was carried out on the lateral-torsional buckling tests that had been conducted at various institutions. A summary of the surveyed results was presented in tables and figures for 159 rolled beams and for 119 welded beams. The scattered test points were compared with the design formulas of lateral-torsional buckling strength.

The design formula recommended by European Convention for Constructional Steelwork (ECCS) for laterally unsupported beams is given (12) by

$$M_u/M_p = (1 + \bar{\lambda}^{2n})^{-1/n} \qquad \text{----(2-1)}$$

in which M_u, M_p = ultimate and full plastic moments, respectively; $\bar{\lambda}$ = $\sqrt{M_p/M_E}$, M_E = elastic critical moment; and n = system factor and is given

by the simple value of 2.5 to ensure that the resulting design curve corresponds to a mean value for rolled beams.

In Ref. 7, Fukumoto and Kubo recommended from the test survey that the system factors become n = 2.5 and 1.5, respectively, for the mean M and M-2S (S = standard deviation) for rolled beams and n = 2.0 and 1.0 for welded beams.

After the review of the existing lateral-torsional buckling test data, the writer feels that careful tests are needed to have reliable statistical data on initial imperfections such as residual stresses in beam-type members and initial crookednesses, and also to provide statistical data for lateral-torsional buckling strength of beams under the nominally identical conditions of loading, supporting, and member sizes.

This chapter presents an experimental investigation of laterally unsupported rolled beams from a statistical consideration. Nominally identical 25 rolled beams for each group of three different lengths are tested under a concentrated load applied vertically at the midspan of the compression flange. The variations of the geometrical and material imperfections are measured and the influences of the variations of the various parameters on the scattered test points of the lateral-torsional buckling are reviewed. The obtained test results are also compared with the existing test data and design formulas.

2-2 TEST PROGRAM

Specimens and Testing --- Twenty-five 7 m long members having the nominally identical cross section of rolled I-200 mm x 100 mm x 5.5 mm x 8 mm have been prepared. As shown in Fig. 2-1(a), from each 7 m member, beam having three different span lengths of 2.6 m (A), 2.0 m (B), and 1.5 m (C),

tensile coupons, and a short beam (D) for residual stress measurements have been cut out. Figure 2-1(b) shows the allotment of specimens in 7 m long members. The nominal cross-sectional properties and beam parameters are shown in Table 2-1, in which the fillets (r=11 mm) are included for calculation. The Saint-Venant torsion constant J is calculated by the approximate formula (10) as follow:

$$J = \frac{2}{3}bt^3 + \frac{1}{3}(h-2t)w^3 + 2\alpha D^4 - 0.420t^4$$

$$\alpha = -0.0420 + 0.2204\frac{w}{t} + 0.135\frac{r}{t} - 0.0865\frac{wr}{t^2} - 0.0725\left(\frac{w}{t}\right)^2 \quad \text{----(2-2)}$$

$$D = \left[(t+r)^2 + w\left(r+\frac{t_w}{4}\right)\right] / (2r+t)$$

in which b = flange width, h = beam height, t = flange thickness, w = web thickness, and r = fillet radius.

Beams are restrained at their supports against torsions but not against warping. Details of the fork supports are shown in Fig. 2-2(a), 2-2(b), and Photo. 2-1. A test beam rests on the bed which swings about the horizontal axis (x-x axis) through four self-aligning roller bearings (point 1, in Fig. 2-2(b)), and the arms with knife-edge guides hold the beam web from both side, which provides a torsionally simply-supported condition. The whole fixture rotates about the vertical axis (y-y axis) through thrust ball bearings (point 3, in Fig. 2-2(b)), which transit the end reaction, and sealed bearing (point 4, in Fig. 2-2(b)), and move in the longitudinal direction through slide needle bearings (point 5, in Fig. 2-2(b)).

From the strain reading along the axis of the test beam a preliminary test shows the laterally simply supported end condition has been obtained successfully. A concentrated load is applied vertically at the mid span of the compression flange through the Lehigh-type gravitational load simulator (15) and the loading yoke. The general arrangement of the tests is seen in Fig. 2-3 and Photo. 2-2. Photo. 2-3 shows the gravitational load simulator which could ensure that the applied load remains vertical during test. The loading yoke shown in Fig. 2-4 and Photo. 2-4 could eliminate any restraining effects of the applied load.

Tensile Coupon Test --- Four tensile coupons, i.e., two from upper and lower flange and two from web are cut from each of the 25 members. As shown in Photo. 2-5, two dial gages are attached with the coupon to control the strain rate.

Residual Stress Measurement --- Longitudinal residual stresses are measured by the sectioning method. The measuring equipments are shown in Photo. 2-6. Gage points for measuring the residual strains with a mechanical extensiometer of 100 mm gage length and sawing positions are shown in Fig. 2-5. The optimum mesh sizes of sectioning are determined empirically so that the magnitude and distribution of stresses do not change even with the increase of the number of mesh.

Initial Crookednesses --- Initial measurements of the crookedness of 75 beams are taken at the equally divided five points about minor axes and in angle of rotation. The example of measurements is shown in Photo. 2-7.

2-3 TEST RESULTS FOR INITIAL IMPERFECTIONS

Tensile Coupon Test Results --- A typical stress-strain relationship of plate (t=8 mm) for a flange is shown in Fig. 2-6. Symbol o is a

measurement point when the rate of strain increase is nearly zero. Figures 2-7(a), 2-7(b), and 2-7(c), respectively, show the mechanical properties of static yield stress σ_y, modulus of elasticity E, and maximum stress σ_u in histograms in which N = number of coupons; M = mean value; S = standard deviation and ω = coefficient of variation. The values M and M± 2S are also shown in the abscissa. Dotted curves indicate the corresponding Gaussian distribution curves. In Fig. 2-7(a), the yield stress distributions in flange and in web are clearly separated, having two peaks in the histograms. The corresponding Gaussian distributions are shown for flange and web plates. Mean values of σ_y are greater by 7 % in flange and 22.3 % in web than the minimum specified value of σ_y = 235 MN/m^2 of ordinary mild steel. On the contrary, there are no such separate distributions in flange and web for E and σ_u values as shown in Figs. 2-7(b) and 2-7(c). The yield stresses of flange and web in each test beam are listed in Table 2-2 and a part of tensile coupons after test is shown in Photo. 2-8.

Initial Crookedness --- Figure 2-8 describes the initial measurements of the crookedness of 75 beams, taken at 5 points about the minor axis and the angle of rotation. The measured initial crookedness of 75 beams are listed in Table 2-3. Figure 2-9 shows the nondimensionalized initial crookedness of 75 beams at the midspan about minor axis and angle of rotation. Mean values of f_0/L in the minor axis is 0.080 x 10^{-3} and M+2S value is 0.186 x 10^{-3} . The foregoing values are about one-fifth of the initial value of 1/1,000 generally accepted for column design.

Residual Stresses --- The measured values of released stress after crosswise cut are compared with those after sectioning more in Fig. 2-10. The crosswise cut have released by about 80 % in flanges and by about 100 % in web. The measured residual stresses of 25 members are listed in Table

2-4. Figure 2-11(a) shows the measured residual stress distribution for 25 members and a summary of the measured results is shown in Fig. 2-11(b) for M and M± S at the measured gage points. Note that the upper flange in Figs. 2-11 always indicates higher tensile and compressive residual stresses than those in the lower flange. The lower flange is always in contact with the cooling bed in a steel factory. The different residual stresses in the upper and lower flange may be explained by the cooling conditions in the bed where the lower flange has less temperature gradient compared with the upper flange. The test beams are set up in such a way so that the upper flange is always in compression under bending.

Figures 2-12(a), 2-12(b), and 2-12(c) show the variations of the residual stresses at the upper flange center, at the end of upper flange, and at the web center, respectively. The mean values of the residual stress are 0.69 σ_{yf}, 0.08 σ_{yf}, and 0.62 σ_{yw} at the upper flange center, flange end, and web center, respectively. The values σ_{yf} and σ_{yw} are the actual mean values of yield stresses in flange and web, respectively. The values M± 2S are also given in the abscissa. The considerable large value of coefficient of variation ω= 0.438 at the flange end results from the low mean value of the compressive residual stresses. Gaussian distribution curves are also given in the figures for comparison.

Cross-Sectional Dimensions and Area --- The measured cross-sectional dimensions of 75 beams are listed in Table 2-5. Figure 2-13 shows a measured histogram of the cross-sectional variation of the 25 members. The actual mean area is 2 % less than the nominal area. The values M± 2S and the corresponding Gaussian distribution curve are also given in Fig. 2-13.

2-4 LATERAL-TORSIONAL BUCKLING TEST RESULTS

Load-Deflection Curves --- Figure 2-14 shows an example of load versus horizontal and vertical deflection curves for a beam chosen from 25 members and cut out into pieces A, B, and C as in Fig. 2-1(a). From the elastic theory, the equation of the straight lines in Fig. 2-14 may be deduced as follow: $\delta_v = PL^3/48EI(1+12kr^2E/L^2G)$. The second term in the parenthesis is for shear effect. Due to initial crookednesses, the value of δ_H exists from the early stage of loading and the P $\sim$ δ_H curve gradually decreases after passing the peak load, in which δ_H is the horizontal displacement of the corresponding flange. The P $\sim$ δ_v curves, however, drop rapidly in the unloading range. Photo. 2-9 shows a laterally buckled beam (A-Series) and no local buckling was observed.

Strain-Distribution --- Figure 2-15 shows an example of measured strain distributions on a compression flange for one of the test beams (A-7). This figure describes the following two strain distributions on each stage of loading: (1) vertical strain distribution, and (2) lateral strain distribution.

Ultimate Strength --- Figure 2-16 summarizes the buckling test results showing histograms for A, B, and C series corresponding to the maximum load P_{max}. The corresponding Gaussian distribution is shown in each series and the measured M and M$\pm$ 2S values are also given in the abscissa. Contrary to the cases in Figs. 2-7(a), 2-7(b), and 2-7(c), the histogram in Fig. 2-16 does not fit well with the Gaussian curves.

The coefficient of variation of the ultimate strength increases with the beam length from C to A. Table 2-6 lists the ultimate strength P_{max} of the 25 nominally identical beams in A, B, and C series,respectively.

Variation of the ultimate strength of the test beams is shown in Fig. 2-17 in which M_u/M_p is in the ordinate and the modified slenderness

parameter $\bar{\lambda}$ is in the abscissa. The ultimate moment $M_u = P_{max}$ L/4 is at the midspan. The figure gives the buckling strength for a wider range in the inelastic zone of M_u/M_p = 0.7 ~ 1.0 and $\bar{\lambda}$ = 0.7 ~ 1.2. The dropping curve is for $M_u/M_p = 1/\bar{\lambda}^2$ which represents an elastic buckling curve. The lower part of the figure also shows the corresponding coefficient of variation, ω.

In order to present the test data in the nondimensional form, the following three different M_p values are used for comparison: (1) Use the measured σ_y of tensile coupons cut out from the same number of member with A, B, and C beams (the different mean σ_y values in flange and in web and the nominal cross-sectional dimensions are used to determine the plastic moment $(M_p)_{an}$; (2) use the measured σ_y as previously shown and the measured cross-sectional dimensions, $(M_p)_{aa}$; and (3) use the nominal σ_y = 235 MN/m^2 and the nominal cross-sectional dimension that lead to $(M_p)_{nn}$ = 49.30 kNm.

It may be seen from Fig. 2-17 that the considerable variation of the ultimate strength still exists after the test data are nondimensionalized by $(M_p)_{nn}$. However, by using $(M_p)_{aa}$, the variation of the test data becomes small. By using $(M_p)_{an}$, the variation of the test data is close to the results obtained by using $(M_p)_{aa}$. Influence of the variation of M_p due to the actual yield stresses is comparatively profound on the variation of the ultimate strength for medium and short beams. The values M-2S from the test data are still lower than the observed minimum ultimate strength.

Table 2-7 lists a summary of the test results of the 25 beams in the A, B, and C series. The test data of members 1-25 are nondimensionalized by using $(M_p)_{aa}$. The values of M, S, and ω are also given for each series.

2-5 STATISTICAL EVALUATION

Statistical Test Data --- Statistical data of the geometrical and material properties and of the ultimate strength are summarized for the mean, standard deviation and coefficient of variation in Table 2-8. In this table it is seen that the coefficient of variation of initial crookednesses is much higher than that of other properties.

Influence of Beam Parameters on Variation of Ultimate Strength --- In Fig. 2-18 P_{max} , $(M_p)_{aa}$,$\sigma_{r,tip}$ and f_0/L about the minor axis are plotted against the beam number, where the beam is ranked in order according to beam strength in the A series.

Figure 2-18 shows that the P_{max} variate curves for the C and B series are remarkably similar. On the contrary, the A series is somewhat different in details from the other two curves, even though all three curves are roughly in a gradual ascent with the abscissa. There may be a significant positive correlation between M_p and P_{max} curves for both the C and B series.

The more detailed analysis on the sensitivity of the parameters upon the variation of the ultimate strength is made herein by comparing the coefficients of rank correlation.

Table 2-9 shows the obtained rank correlation coefficients for any given pair of rankings between rows and columns. The coefficients measure the degree of correspondence between the ranks of the beam parameters such as yield stress, area, compressive residual stress at the flange tip and f_0/L about the minor axis, and the ultimate strength P_{max} and M_u/M_p where M_p is either $(M_p)_{aa}$ or $(M_p)_{an}$.

The agreement between the rankings is perfect when a coefficient is +1.0 and if the disagreement is perfect, a coefficient is -1.0. Increasing values from -1 to +1 should correspond to increasing agreements between the

ranks (11).

Ultimate Strength Versus Yield Stresses and M_p --- As shown in Table 2-9, the correlation between the ranking of P_{max} and σ_{yf}, σ_{yw}, or $(M_p)_{aa}$ is positive and the agreements as substantial. The agreement is also higher when the beams are in the short range like the B and C series. However, the correlation of σ_y or $(M_p)_{aa}$ with the variations of M_u/M_p nondimensionalized by $(M_p)_{aa}$ or $(M_p)_{an}$ is poor compared with that of P_{max} .

Ultimate Strength Versus Residual Stresses --- As shown in Table 2-9, the coefficient between the ranking of $\sigma_{r,tip}$ and $M_u/(M_p)_{aa}$ or $M_u/(M_p)_{an}$ is negative and small in every series indicating negative correlation between the two rankings. This means that $\sigma_{r,tip}$ may reduce the ultimate strength M_u/M_p according to the amount of $\sigma_{r,tip}$.

Ultimate Strength Versus Initial Crookedness --- The coefficient of rank correlation between initial crookedness and P_{max} and M_u/M_p is small and nearly equal to zero, indicating very little correlation between the foregoing rankings. This may also be seen from Fig. 2-18.

Correlation of Ultimate Strengths for A, B, and C Series --- Since the beam of the same specimen number in each series is cut out from a 7 m long member, the cross-sectional properties and residual distributions are assumed to be identical in each series when the specimens are of the same number. Then the coefficient of rank correlation between the rankings may be perfect positive correlation if the initial geometrical imperfections are disregarded.

The degree of correspondence between two rankings of P_{max} in the A, B, and C series is significantly high as seen in Table 4 and rewritten as follows: coefficient (A and B) = 0.74; coefficient (B and C) = 0.86; and coefficient (C and A) = 0.68. This indicates that the B and C series agree

more than A and B or A and C. These results may also be observed in Fig. 2-18 where the rankings of P_{max} are very similar between the B and C series. The less correspondence between two rankings of the A and C series may be explained by the different effect of initial deflection patterns on long (A) and short (C) beams.

2-6 COMPARISON WITH SPECIFIED FORMULA AND AVAILABLE TEST RESULTS

Comparison with the Specification formula --- In Fig. 2-19 the comparison of the test results with the specification formula of JRA (Japan Highway Bridge Specification). The following two different yield stress σ_y are used for comparison: (1) Use the measured σ_y of tensile coupons (flange), and (2) use the nominal $\sigma_y = 235\ N/mm^2$. The test results of Lindner (15) are also shown in Fig. 2-19. The test points are comparatively higher than the specific formula.

Comparison with Available Test Results --- In Fig. 2-20 the present test results for 75 beams are compared with the past 128 test points (14) for the same H-200 mm x 100 mm x 5.5 mm x 8 mm rolled sections. Various supporting and loading conditions of the past test are modified to make those results comparable with the present test results. The test points are nondimensionalized by the actual yield stresses and cross-sectional dimensions. The values M and M-2S and coefficient of variation are also given in Fig. 2-20.

The present test has been conducted carefully to provided statistical data of lateral-torsional buckling strength under the nominally identical supporting and loading conditions, and the coefficients of variation of the strength increase with beam length and are 0.0245, 0.0291, and 0.0564 for the C, B, A series, respectively. As shown in Fig. 2-20 these values are

considerably low as compared with the past scattered test points. Dashed curved correspond to the ECCS formula for n = 1.5 and 2.5. From the present and past test results in Fig. 2-20 the following general formula which is the similar function of JRA proposed:

$$M_u/M_p = 1 - 0.412(\bar{\lambda} - 0.2) \qquad \text{----(2-3)}$$

Eq. (2-3) ensures empirically the lower bound for the scattered test points.

2-7 CONCLUSIONS

Nominally identical 25 rolled beams for each group of three different lengths are tested carefully under a concentrated load at the mid span of the compressive flange and the variations of the geometrical and material imperfections are obtained from the statistical consideration. Degree of correspondence between the various beam parameters and the scattered ultimate strength is investigated. The main beam parameter that highly influences the variation of the ultimate strength is the variable value of the actual full plastic moment that contains actual yield stress and cross-sectional dimensions. The degree of correspondence between the ultimate strength and M_p becomes small with an increasing slenderness functions. Compressive residual stresses at the flange tip always reduce the ultimate strength showing the negative correlation between the two variates. However, the relationship between the initial crookedness and the ultimate strength is little known since the observed initial crookedness is less than 1/5,000 of the span length.

The coefficients of variation of M_u/M_p that are nondimensionalized by

the actual $(M_p)_{aa}$ values become less and these are 24 %, 40 %, and 45 % less than those of P_{max} (or M_u) for the A, B, and C series, respectively. When the ultimate strengths are nondimensionalized by the full plastic moment $(M_p)_{an}$, coefficients of variation of M_u/M_p are 15 %, 32 %, and 32 % less than those of P_{max} (or M_u), respectively, for the foregoing three series.

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NOTATION

The following symbols are used in this chapter:

A = area of cross section;
E = modulus of elasticity;
f = frequency;
G = modulus of elasticity in shear;
k = shear coefficient;
L = span length of member;
M = bending moment;
M_E = elastic critical moment;
M_p = plastic moment;
$(M_p)_{aa}$ = plastic moment for actual yield stress and cross section;
$(M_p)_{an}$ = plastic moment for actual yield stress and nominal cross section;
$(M_p)_{nn}$ = nominal plastic moment;
M_u = ultimate bending moment;
P_{max} = maximum load;
r = radius of gyration of area;
S = standard deviation;
u_o = initial deflection in lateral direction;
δ_H = lateral deflection;
δ_V = vertical deflection;
$\bar{\lambda}$ = slenderness function;
σ_r = residual stress;
$\sigma_{r,tip}$ = residual compressive stress at flange end;
σ_y = yield stress;
σ_{yf} = yield stress in flange;
σ_{yw} = yield stress in web;
φ_o = initial angle of rotation; and
ω = coefficient of variation.

Table 2-1 Nominal Cross-Sectional Properties

	A-Series	B-Series	C-Series
L (cm)	260	200	150
M_E (kNm)	49.00	68.72	104.88
$\overline{\lambda} = \sqrt{M_p/M_E}$	1.003	0.847	0.686
$A = 27.16\ cm^2$, $I_x = 1840\ cm^4$, $I_y = 134\ cm^4$			
$j = 5.765\ cm^4$, $I_\omega = 12288\ cm^6$, $f = 1.138$			

L = Span Length, A = Sectional Area,

I_x = Moment of Inertia about the Strong Axis,

I_y = Moment of Inertia about the Weak Axis,

j = Torsional Constant, I_ω = Warping Moment of Inertia,

f = Shape Factor, M_p = Full plastic Moment,

M_E = Elastic Critical Moment

Table 2-2 Yield Stresses of Flange and Web in Each Test Beam

Beam No.	σ_{yf} (N/mm^2)	σ_{yw} (N/mm^2)
1	260.0	296.0
2	259.6	301.2
3	261.5	300.9
4	263.1	307.1
5	254.1	290.1
6	254.0	288.4
7	245.3	272.9
8	244.2	278.3
9	254.4	283.5
10	243.8	289.9
11	239.6	271.4
12	247.1	281.2
13	239.2	270.8
14	257.2	295.9
15	254.7	292.1
16	262.5	291.3
17	254.4	289.1
18	258.1	294.1
19	258.4	299.0
20	244.2	291.4
21	253.4	286.3
22	249.4	282.7
23	250.4	287.2
24	250.1	277.9
25	242.2	271.8

Table 2-3 Initial Crookednesses of 75 Beams (x 10^{-5})

Beam No.	u_o			φ_o		
	0.25L	0.50L	0.75L	0.25L	0.5L	0.75L
A- 1	3.1	5.0	6.2	4.3	5.3	4.8
A- 2	0.4	0.8	10.4	1.6	1.1	1.6
A- 3	1.2	1.9	4.6	5.3	18.7	13.9
A- 4	11.2	6.5	6.5	34.7	21.9	21.9
A- 5	6.5	1.2	1.5	11.8	20.8	10.1
A- 6	9.2	9.2	5.0	1.1	2.7	0.5
A- 7	8.1	12.3	3.5	5.3	6.9	7.5
A- 8	10.8	5.8	8.5	8.5	19.8	17.1
A- 9	10.0	11.2	8.5	4.3	8.0	10.7
A-10	1.5	4.2	0.8	1.1	8.5	5.9
A-11	2.3	5.4	5.4	8.5	10.1	10.1
A-12	2.7	13.1	3.1	6.9	2.7	3.7
A-13	6.5	10.8	5.0	3.7	0.5	2.1
A-14	3.5	8.1	7.7	0.0	11.2	9.1
A-15	4.6	13.1	9.6	7.5	16.0	9.1
A-16	20.4	20.0	13.8	5.9	1.6	9.6
A-17	8.1	16.9	18.8	1.1	1.6	2.7
A-18	14.2	12.7	14.2	19.8	9.6	5.3
A-19	17.7	8.5	14.2	9.1	4.3	4.8
A-20	4.2	7.3	0.4	10.1	8.5	2.7
A-21	3.1	2.3	1.9	4.8	3.2	6.4
A-22	7.7	1.5	7.7	13.9	12.8	5.3
A-23	4.6	5.0	10.4	3.2	4.3	5.3
A-24	5.8	0.8	1.5	6.4	5.9	1.6
A-25	5.8	16.2	6.2	3.7	15.5	11.2
B- 1	2.5	2.0	3.5	9.7	10.4	0.7
B- 2	6.5	16.0	9.0	12.5	21.5	18.8
B- 3	4.0	10.5	4.0	2.8	0.7	5.6
B- 4	6.5	8.5	3.5	0.7	2.8	3.5
B- 5	7.5	3.5	3.5	6.3	13.9	45.8
B- 6	5.0	4.0	1.0	11.8	17.4	17.4
B- 7	2.0	9.0	6.0	39.6	66.0	95.8
B- 8	5.5	2.0	5.0	4.9	18.1	36.1
B- 9	6.0	6.0	12.5	2.1	8.3	16.7
B-10	11.0	4.5	7.0	8.3	6.3	11.8
B-11	0.0	4.5	17.0	4.2	8.3	6.9
B-12	3.0	4.5	2.0	9.0	15.3	8.3
B-13	9.5	8.0	6.0	3.5	2.8	6.3
B-14	12.5	7.0	1.5	13.9	27.1	52.8
B-15	0.5	9.0	11.5	7.6	38.9	66.7
B-16	7.0	14.0	20.5	11.8	16.7	9.0
B-17	9.5	9.0	3.5	4.9	1.4	3.5
B-18	3.5	3.5	4.0	14.6	27.8	33.3
B-19	8.5	4.0	2.5	5.6	0.0	9.7
B-20	13.0	4.5	0.5	5.6	17.4	32.6
B-21	4.5	4.0	1.0	13.2	33.3	68.8
B-22	2.5	2.5	4.0	3.5	3.5	7.6
B-23	3.5	11.5	1.5	10.4	22.2	38.2
B-24	4.0	3.0	4.5	6.3	0.7	4.9
B-25	2.0	9.5	6.5	37.5	0.0	35.4

Table 2-3 Continued

Beam No.	u_o			φ_o		
	0.25L	0.50L	0.75L	0.25L	0.5L	0.75L
C- 1	3.3	3.3	0.7	108.3	62.0	34.3
C- 2	4.0	4.0	10.0	6.5	2.8	2.8
C- 3	3.3	0.7	0.7	4.6	12.0	3.7
C- 4	12.0	8.7	0.0	3.7	7.4	8.3
C- 5	2.7	4.7	2.0	4.6	0.9	1.9
C- 6	9.3	23.3	12.0	1.9	5.6	8.3
C- 7	3.3	3.3	0.0	14.8	7.4	15.7
C- 8	4.0	20.7	8.0	2.8	11.1	0.0
C- 9	0.7	16.7	6.0	0.0	11.1	3.7
C-10	1.3	8.7	5.3	13.0	2.8	13.9
C-11	4.0	4.7	4.0	21.3	34.3	53.7
C-12	10.7	7.3	2.0	2.8	4.6	0.9
C-13	8.7	1.3	2.0	13.9	13.9	9.3
C-14	16.7	17.3	11.3	18.5	24.1	52.8
C-15	22.7	19.3	7.3	18.5	30.6	21.3
C-16	3.3	4.7	7.3	10.2	4.6	3.7
C-17	18.7	9.3	4.7	6.5	0.0	0.9
C-18	5.3	0.0	6.0	43.5	81.5	123.1
C-19	2.7	6.7	2.0	9.3	1.9	6.5
C-20	10.0	14.0	12.7	38.0	61.1	72.2
C-21	7.3	14.0	6.7	14.8	17.6	14.8
C-22	8.0	15.3	12.0	6.5	15.7	8.3
C-23	2.7	7.3	4.7	1.9	9.3	18.5
C-24	0.0	5.3	1.3	3.7	1.9	3.7
C-25	6.0	7.3	1.3	5.6	8.3	10.2

Table 2-4 Residual Stresses of 25 Members, in N/mm²

Point No.	Beam No. 1 Upper Flange	Beam No. 1 Lower Flange	Beam No. 1 Web	Beam No. 2 Upper Flange	Beam No. 2 Lower Flange	Beam No. 2 Web	Beam No. 3 Upper Flange	Beam No. 3 Lower Flange	Beam No. 3 Web
1	-22.7	-10.3	87.5	-26.8	-6.2	77.2	-16.5	4.1	89.5
2	-18.5	-14.4	-15.5	-29.9	6.2	-28.8	-16.5	-1.1	-12.4
3	-27.9	-4.1	-15.5	-33.0	-4.1	-59.7	-23.7	-10.3	-25.8
4	26.8	42.2	-71.1	31.9	44.2	-121.5	44.2	33.9	-107.1
5	98.9	99.8	-139.1	103.0	91.6	-153.5	118.4	86.5	-146.2
6	175.0	138.0	-170.9	177.1	152.4	-173.0	185.3	144.2	-168.9
7	101.9	86.5	-172.0	94.7	84.4	-166.8	106.0	92.7	-170.9
8	33.0	42.2	-153.5	35.0	37.1	-153.5	40.1	44.2	-150.3
9	-22.7	3.0	-120.5	-9.3	-2.1	-111.2	-14.4	3.0	-97.9
10	-13.4	2.1	-46.4	-13.4	-3.1	-19.6	-15.5	-9.3	-26.8
11	-26.8	-10.3	16.5	-39.1	-33.0	9.2	-26.8	-16.5	12.4
12			98.9			81.3			93.7

Point No.	Beam No. 4 Upper Flange	Beam No. 4 Lower Flange	Beam No. 4 Web	Beam No. 5 Upper Flange	Beam No. 5 Lower Flange	Beam No. 5 Web	Beam No. 6 Upper Flange	Beam No. 6 Lower Flange	Beam No. 6 Web
1	-20.6	-4.1	95.7	-16.5	8.2	89.5	-10.3	6.2	85.4
2	-20.6	3.0	-15.5	-12.4	2.1	39.1	-16.5	-6.2	-9.3
3	-26.8	3.0	-21.7	-17.6	2.1	-24.7	-22.7	-20.6	-16.5
4	30.9	36.0	-97.9	40.1	49.4	-105.0	20.6	42.2	-100.9
5	107.1	80.3	-148.3	107.1	94.7	-138.0	92.7	85.4	-150.3
6	181.2	129.7	-182.3	197.7	150.3	-170.9	152.4	148.3	-179.2
7	103.0	105.0	-184.4	103.0	88.6	-176.1	113.3	88.6	-183.3
8	33.9	45.3	-161.7	37.1	46.3	-163.8	40.1	54.5	-162.7
9	-17.6	-6.2	-119.4	-14.4	13.3	-116.4	-12.4	2.1	-129.7
10	-35.0	-6.2	-40.2	-2.1	8.2	-46.4	-8.2	-6.2	-65.9
11	-43.2	-18.5	11.3	-20.6	-16.5	8.2	-18.5	-14.4	-26.8
12			79.2			80.3			73.1

Point No.	Beam No. 7 Upper Flange	Beam No. 7 Lower Flange	Beam No. 7 Web	Beam No. 8 Upper Flange	Beam No. 8 Lower Flange	Beam No. 8 Web	Beam No. 9 Upper Flange	Beam No. 9 Lower Flange	Beam No. 9 Web
1	12.4	4.1	213.1	-30.9	-16.5	106.0	-14.4	-4.1	112.2
2	-51.5	1.0	-16.5	-25.8	-8.2	-9.3	-32.0	9.2	-9.3
3	-28.8	-2.1	-35.0	-23.7	-1.1	-10.3	-19.6	-8.2	-19.6
4	29.8	53.5	-117.4	40.1	49.4	-114.3	43.2	41.2	-97.9
5	125.6	88.6	-164.8	119.4	108.1	-155.5	110.1	98.9	-146.2
6	179.2	111.2	-192.6	187.4	135.9	-173.0	173.0	133.9	-172.0
7	105.0	88.6	-201.8	99.8	106.0	-177.1	132.8	96.8	-173.0
8	19.5	45.3	-187.4	30.9	52.5	-166.8	36.0	50.4	-153.5
9	-12.4	23.6	-113.3	-17.6	6.2	-115.3	-19.6	0.0	-104.0
10	-6.2	-4.1	-39.1	-13.4	-10.3	-29.9	-7.3	7.2	-34.0
11	-20.6	12.4	6.2	-16.5	-26.8	8.2	-16.5	-12.4	8.2
12			110.1			115.3			81.3

Table 2-4 Continued

Point No.	Beam No. 10 Upper Flange	Beam No. 10 Lower Flange	Beam No. 10 Web	Beam No. 11 Upper Flange	Beam No. 11 Lower Flange	Beam No. 11 Web	Beam No. 12 Upper Flange	Beam No. 12 Lower Flange	Beam No. 12 Web
1	-28.8	-8.2	113.3	-20.6	-22.7	88.6	-20.6	-4.1	107.1
2	-21.7	4.1	-51.5	-33.0	-12.4	-23.7	-28.8	10.3	-25.8
3	-17.6	-2.1	-32.0	-23.7	-6.2	-30.9	-27.9	13.3	-18.5
4	39.1	42.2	-94.7	41.2	35.0	-96.8	36.0	36.0	-95.8
5	111.2	84.4	-142.1	90.6	92.7	-149.4	107.1	78.3	-142.1
6	146.2	131.8	-160.6	140.0	127.7	-175.0	177.1	127.7	-169.9
7	94.7	106.0	-166.8	84.4	98.9	-173.0	86.5	96.8	-170.9
8	33.0	49.4	-149.4	31.9	49.4	-146.2	30.9	50.4	-154.5
9	-16.5	5.1	-115.3	-25.8	-1.1	-120.5	-20.6	7.2	-113.3
10	-2.1	0.0	-47.4	-18.5	-2.1	-44.3	-20.6	4.1	-43.2
11	-20.6	-22.7	-1.1	-30.9	-33.0	-5.2	-24.7	-12.4	14.4
12			97.8			81.3			72.1

Point No.	Beam No. 13 Upper Flange	Beam No. 13 Lower Flange	Beam No. 13 Web	Beam No. 14 Upper Flange	Beam No. 14 Lower Flange	Beam No. 14 Web	Beam No. 15 Upper Flange	Beam No. 15 Lower Flange	Beam No. 15 Web
1	10.3	-2.1	83.4	-20.6	-20.6	107.1	-8.2	-4.1	83.4
2	-35.0	3.0	-3.1	-22.7	-24.7	12.4	-12.4	-14.4	14.4
3	-17.6	1.0	4.1	-25.8	0.0	-16.5	-11.4	-4.1	-23.7
4	39.1	39.1	-98.9	36.0	44.2	-94.7	56.6	44.2	-103.0
5	109.1	101.9	-158.6	113.3	92.7	-148.3	117.4	97.8	-147.3
6	179.2	133.9	-170.9	177.1	142.1	-185.3	162.7	146.2	-174.1
7	109.1	94.7	-177.1	107.1	106.0	-182.3	109.1	105.0	-170.9
8	38.0	41.2	-160.6	36.0	45.3	-157.6	44.2	45.3	-154.5
9	-8.2	-5.2	-115.3	-23.7	2.1	-115.3	-8.2	-8.2	-108.2
10	2.1	4.1	-55.6	-9.3	5.1	-41.2	-9.3	1.0	-41.2
11	-26.8	-12.4	-1.1	-20.6	-14.4	5.1	-14.4	-6.2	5.1
12			81.3			79.2			82.4

Point No.	Beam No. 16 Upper Flange	Beam No. 16 Lower Flange	Beam No. 16 Web	Beam No. 17 Upper Flange	Beam No. 17 Lower Flange	Beam No. 17 Web	Beam No. 18 Upper Flange	Beam No. 18 Lower Flange	Beam No. 18 Web
1	-33.0	-30.9	148.3	-4.1	-18.5	92.7	-18.5	-14.4	99.8
2	-27.9	-4.1	-52.6	-8.2	-19.6	-24.7	-24.7	0.0	-11.4
3	-9.3	14.4	-21.7	-15.5	-17.6	-36.1	-26.8	2.1	-24.7
4	29.8	36.0	-45.3	50.4	47.4	-112.3	41.2	43.2	-110.2
5	84.4	90.6	-147.3	116.3	111.2	-157.6	105.0	109.1	-162.7
6	170.9	129.7	-186.4	158.6	148.3	-181.2	152.4	135.9	-174.1
7	109.1	123.6	-188.5	109.1	97.8	-189.5	114.2	108.1	-192.6
8	40.1	55.6	-168.9	48.3	46.3	-161.7	47.4	44.2	-159.7
9	-26.8	-4.1	-128.8	-10.3	-6.2	-122.6	-9.3	-6.2	-112.3
10	-27.9	4.1	-52.6	-10.3	-4.1	-43.2	-35.0	-2.1	-35.0
11	-59.7	-61.8	-14.4	-20.6	-12.4	1.0	-33.0	-14.4	6.2
12			81.3			92.7			88.6

Table 2-4 Continued

Point No.	Beam No. 19			Beam No. 20			Beam No. 21		
	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web
1	-16.5	4.1	90.6	-18.5	4.1	96.8	-2.1	-4.1	76.2
2	-24.7	0.0	-16.5	-23.7	6.2	-9.3	-22.7	2.1	-49.4
3	-22.7	4.1	-23.7	-30.9	6.2	-23.7	-18.5	-6.2	-18.5
4	46.3	41.2	-94.7	45.3	49.4	-91.7	20.6	42.2	-90.6
5	112.2	88.6	-144.2	117.4	89.5	-145.2	71.0	96.8	-128.8
6	183.3	129.7	-177.1	185.3	140.0	-170.9	201.8	144.2	-162.7
7	118.4	100.9	-181.2	117.4	95.7	-177.1	106.0	98.9	-170.9
8	46.3	45.3	-158.6	47.4	47.4	-152.4	33.9	47.4	-144.2
9	-11.4	-3.1	-109.1	-19.6	2.1	-106.1	-20.6	12.4	-103.0
10	-22.7	1.0	-40.2	-20.6	7.2	-35.0	-7.3	0.0	-59.7
11	-26.8	-30.9	3.0	-35.0	-12.4	23.6	-12.4	-16.5	10.3
12			90.6			86.5			76.2

Point No.	Beam No. 22			Beam No. 23			Beam No. 24		
	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web
1	-16.5	2.1	38.0	-14.4	-4.1	129.7	-14.4	-12.4	108.1
2	-30.9	-14.4	-25.8	-12.4	-20.6	-55.6	-26.8	-5.2	-12.4
3	-26.8	-3.1	-25.8	-8.2	3.0	-23.7	-23.7	-5.2	-38.1
4	31.9	35.0	-97.9	14.4	27.8	-84.4	29.8	41.2	-107.1
5	93.7	74.1	-141.1	95.7	92.7	-152.4	75.1	94.7	-149.4
6	189.5	133.9	-166.8	177.1	148.3	-177.1	166.8	129.7	-169.9
7	99.8	85.4	-170.9	98.9	106.0	-177.1	92.7	97.8	-167.9
8	28.8	38.0	-149.4	25.7	38.0	-156.5	35.0	47.4	-151.4
9	-19.6	7.2	-106.1	-17.6	-8.2	-100.9	-14.4	-1.1	-106.1
10	-16.5	-3.1	-30.9	-22.7	0.0	-33.0	-4.1	9.2	-23.7
11	-22.7	-16.5	-3.1	-30.9	-10.3	35.0	-12.4	-16.5	3.0
12			73.1			91.6			84.4

Point No.	Beam No. 25		
	Upper Flange	Lower Flange	Web
1	-10.3	8.2	93.7
2	15.4	10.3	2.1
3	-33.0	10.3	-9.3
4	16.5	56.6	-76.2
5	83.4	88.6	-131.8
6	193.6	127.7	-162.7
7	100.9	83.4	-166.8
8	29.8	40.1	-146.2
9	-25.8	11.3	-109.1
10	-7.3	12.4	-33.0
11	-20.6	-2.1	5.1
12			82.4

Table 2-5 Measured Cross-Sectional Dimensions, in mm

Beam No.	A-Series				B-Series				C-Series			
	b	h	t	w	b	h	t	w	b	h	t	w
1	99.4	198.5	7.89	5.32	99.1	198.4	7.86	5.25	99.0	198.1	7.51	5.28
2	99.2	198.4	7.91	5.35	99.1	198.4	7.91	5.26	99.2	198.5	7.86	5.36
3	99.3	198.4	7.84	5.38	99.4	198.5	7.88	5.40	99.4	198.5	7.85	5.42
4	99.6	198.5	8.03	5.44	99.4	198.5	7.92	5.34	99.4	198.5	7.84	5.42
5	99.4	198.5	7.89	5.35	99.2	198.5	7.88	5.25	99.2	198.4	7.81	5.31
6	99.7	198.8	7.94	5.47	99.7	198.8	7.91	5.38	99.6	198.7	7.83	5.41
7	99.3	198.8	7.77	5.29	99.4	198.7	7.84	5.18	99.4	198.7	7.77	5.31
8	99.4	198.6	7.90	5.26	99.3	198.5	7.90	5.23	99.3	198.4	7.77	5.33
9	99.8	198.9	8.00	5.45	99.5	198.6	7.97	5.34	99.5	198.6	7.86	5.49
10	99.8	198.7	7.89	5.44	99.6	198.6	7.96	5.34	99.6	198.6	7.86	5.42
11	99.2	198.6	7.95	5.35	99.4	198.5	7.91	5.27	99.3	198.5	7.78	5.36
12	99.7	198.7	8.01	5.46	99.8	198.7	7.97	5.39	99.5	198.6	7.83	5.44
13	99.4	198.6	7.89	5.29	99.3	198.5	7.94	5.23	99.3	198.5	7.77	5.34
14	99.8	198.7	7.89	5.47	99.9	198.6	7.97	5.37	99.7	198.6	7.86	5.53
15	100.1	198.8	7.99	5.52	100.0	198.7	7.98	5.47	99.9	198.6	7.89	5.55
16	99.6	198.7	7.97	5.31	99.4	198.5	7.95	5.34	99.4	198.5	7.84	5.41
17	99.8	198.6	7.92	5.40	99.7	198.6	7.95	5.42	99.5	198.6	7.85	5.45
18	99.9	198.7	7.94	5.45	99.8	198.6	8.00	5.41	99.7	199.6	7.89	5.51
19	100.0	198.7	8.06	5.56	100.0	198.7	7.99	5.47	99.9	198.6	7.90	5.53
20	99.6	198.7	7.95	5.41	99.5	198.6	7.92	5.28	99.6	198.6	7.84	5.49
21	100.0	199.5	8.17	5.64	100.1	199.3	8.10	5.51	99.9	199.2	8.05	5.61
22	99.7	198.8	7.91	5.35	99.6	198.8	7.94	5.35	99.5	198.5	7.79	5.42
23	99.7	198.7	7.95	5.42	99.7	198.7	7.93	5.33	99.6	198.7	7.81	5.46
24	99.8	198.6	7.89	5.40	99.6	198.6	7.94	5.35	99.6	198.5	7.80	5.42
25	99.9	198.6	7.93	5.48	99.5	198.6	7.86	5.45	99.6	198.6	7.81	5.47

Table 2-6 Maximum Load P_{max} , in kilonewtons

(unit in kN)

Beam No.	A-Series	B-Series	C-Series
1	70.5	91.5	119.2
2	68.6	87.3	122.6
3	64.4	88.1	120.6
4	72.5	91.9	122.7
5	69.9	92.3	122.1
6	66.0	84.8	121.0
7	61.5	81.1	113.3
8	63.9	83.4	116.2
9	69.4	87.2	121.3
10	71.4	86.1	115.9
11	59.7	83.1	110.0
12	62.0	84.6	114.0
13	62.9	83.1	113.0
14	64.2	91.2	125.7
15	76.4	96.6	130.9
16	70.9	87.8	121.6
17	75.2	90.2	126.3
18	74.8	92.6	126.6
19	76.8	92.8	127.2
20	64.3	84.7	122.3
21	76.9	92.2	124.6
22	68.8	83.0	116.2
23	65.5	86.4	121.6
24	66.4	84.1	114.0
25	69.1	81.7	113.5
M	68.5	87.5	120.0
S	5.07	4.22	5.39
ω	0.0740	0.0482	0.0449

M = Mean Value , S = Standard Deviation

ω = Coefficient of Variation

Table 2-7 M_u/M_p – λ Values (For Actual σ_y and Area)

Beam No.	A-Series		B-Series		C-Series	
	M_u/M_p	$\sqrt{M_p/M_E}$	M_u/M_p	$\sqrt{M_p/M_E}$	M_u/M_p	$\sqrt{M_p/M_E}$
1	0.843	1.079	0.847	0.912	0.853	0.748
2	0.817	1.090	0.803	0.921	0.844	0.746
3	0.765	1.085	0.800	0.914	0.824	0.741
4	0.836	1.075	0.827	0.913	0.830	0.742
5	0.853	1.092	0.872	0.924	0.869	0.750
6	0.796	1.071	0.792	0.905	0.852	0.735
7	0.791	1.069	0.802	0.898	0.840	0.728
8	0.814	1.057	0.818	0.892	0.862	0.726
9	0.835	1.058	0.816	0.895	0.854	0.729
10	0.892	1.054	0.827	0.888	0.839	0.722
11	0.771	1.047	0.830	0.884	0.830	0.720
12	0.764	1.049	0.807	0.888	0.824	0.724
13	0.818	1.046	0.832	0.882	0.856	0.718
14	0.765	1.080	0.834	0.909	0.865	0.740
15	0.908	1.073	0.887	0.907	0.906	0.738
16	0.837	1.075	0.799	0.910	0.835	0.739
17	0.910	1.083	0.837	0.914	0.886	0.744
18	0.887	1.076	0.842	0.907	0.863	0.739
19	0.893	1.075	0.837	0.911	0.866	0.741
20	0.800	1.066	0.818	0.902	0.881	0.732
21	0.901	1.057	0.840	0.895	0.854	0.729
22	0.852	1.059	0.788	0.893	0.838	0.727
23	0.801	1.061	0.817	0.897	0.867	0.730
24	0.823	1.055	0.803	0.890	0.823	0.724
25	0.877	1.051	0.804	0.892	0.841	0.723
M	0.834	1.067	0.823	0.902	0.852	0.733
S	0.0470		0.0239		0.0209	
ω	0.0564		0.0291		0.0245	

M_u = Ultimate Moment
M_p = Full Plastic Moment
M_E = Elastic Critical Moment

M = Mean Value
S = Standard Deviation
ω = Coefficient of Variation

Table 2-8 Statistical Data

			M	S	ω
P_{max} (kN)	A-Series		68.5	5.07	0.0740
	B-Series		87.5	4.22	0.0482
	C-Series		120.0	5.39	0.0449
Material Properties (N/cm^2)	σ_{yf}		25200	726	0.0288
	σ_{yw}		28760	1000	0.0347
	$E \times 10^6$		20.17	0.517	0.0256
	σ_u		43640	1410	0.0324
Sectional Area (cm^2)	A		26.63	0.300	0.0113
Plastic Moment (kN·m)	M_p		53.11	1.755	0.0331
Residual Stress (N/cm^2)	Flange Tip		2070	902	0.438
	Flange Center		17490	1608	0.092
	Web Center		17720	912	0.052
Initial Crookedness ($\times 10^{-3}$)	A Series	u_0	0.080	0.054	0.67
		v_0	0.128	0.093	0.73
		ϕ_0	0.107	0.085	0.80
	B Series	u_0	0.068	0.037	0.55
		v_0	0.065	0.045	0.69
		ϕ_0	0.158	0.150	0.95
	C Series	u_0	0.091	0.066	0.72
		v_0	0.058	0.045	0.78
		ϕ_0	0.130	0.160	1.23

Table 2-9 Coefficient of Rank Correlation for Test Results

		P_{max}			$M_u/M_p)_{aa}$			$M_u/M_p)_{an}$		
		A	B	C	A	B	C	A	B	C
	σ_{yf}	0.52	0.75	0.67	0.18	0.10	0.02	0.15	0.15	0.01
	σ_{yw}	0.49	0.77	0.74	0.16	0.23	0.18	0.12	0.25	0.17
	$M_p)_{aa}$	0.67	0.83	0.82	0.32	0.19	0.11	0.34	0.32	0.28
	σ_{rtip}	-0.02	0.10	0.09	-0.19	-0.11	-0.22	-0.21	-0.13	-0.26
u_0/L	A	0.08			0.12			0.14		
	B		-0.03			-0.27			-0.23	
	C			0.29			0.30			0.46
ϕ_0	A	0.02			0.00			0.00		
	B		0.16			0.15			0.09	
	C			0.02			0.16			0.09
P_{max}	A	1.00	0.74	0.68	0.90			0.89		
	B		1.00	0.86		0.64			0.72	
	C			1.00			0.61			0.71

Fig. 2-1 (a) Allotment of Test Specimens

Fig. 2-1 (b) Allotment of Test Specimens

Fig. 2-2 (a) A Scheme of the End Fixture

Fig. 2-2 (b) A Cross Sectional Scheme of the End Fixture

Fig. 2-3 General Arrangement of Apparatus

Fig. 2-4 Loading Yoke

Fig. 2-5 Residual Stress Measurement

Fig. 2-6 Stress-Strain Relationship by Tensile Coupon Test (No. 16 flange)

(a) Yield Stress

(b) Young's Modulus

Fig. 2-7 Histograms of Material Properties

(c) Ultimate Stress

Fig. 2-7 Histograms of Material Properties

Fig. 2-8 Initial Crookedness of Test Beams

Fig. 2-9 Histograms of Initial Crookedness

Fig. 2-10 Residual Stress Change during Sectioning Process

Fig. 2-11 (a) Residual Stress Distribution

Fig. 2-11 (b) Variation of Residual Stress Patterns

Fig. 2-12 Histograms of Residual Stress

Fig. 2-13 Histogram of Cross-Sectional Area

Fig. 2-14 Load-Deflection Curves

Fig. 2-15 Strain Distribution of Compression Flange

Fig. 2-16 Histograms of Maximum Load

Fig. 2-17 Buckling Test Results

Fig. 2-18 Variations of P_{max} and Testing Parameters

Fig. 2-19 Comparison of Test Results with the Existing Design Formula

Fig. 2-20 Scatter of Test Results and Comparison with Recommended Formula

Photo. 2-1 End Support

Photo. 2-2 Test Set-Up

Photo. 2-3 Gravitational Load Simulator

Photo. 2-4 Loading Yoke

Photo. 2-5 Tensile Coupon Test

Photo. 2-6 Equipments for Residual Stress Measurement

Photo. 2-8 Tensile Coupons after Test

Photo. 2-7 Measurement of Initial Crookedness

Photo. 2-9 Test Beam during Test

Photo. 2-10 75 Test Beams after Failure

Chapter 3. STATISTICAL STUDY OF EXPERIMENTS ON WELDED BEAMS

3-1 INTRODUCTION

Theoretical studies on inelastic lateral-torsional buckling of welded beams (2,9,10) have indicated that residual stresses presented in welded beams which are significantly different from those in rolled beams have a considerably different influence on inelastic buckling strength. It has been a very important question concerning the behavior and design of inelastic beams that welded beams indeed have a lower lateral-torsional buckling strength than rolled beams (1).

Refs. 3 and 4 contain a summary of the surveyed results on the lateral-torsional buckling tests that had been conducted at various institutions. A statistical comparison of the lateral-torsional buckling strength between rolled and welded beams indicates that the welded beams give lower mean values and larger coefficient of variation as compared with the rolled beams.

In Chapter 2, the bending strength of rolled wide-flange beams subjected to a concentrated load applied to the midspan has been examined for 75 specimens with the nominally identical cross section. The variation of the geometrical and material imperfections was measured and the influences of the various parameters on the scattered test points of the lateral-torsional buckling were analyzed from a statistical point of views.

Unfortunately, not many tests (6) exist for welded beams failing in the inelastic range where residual stresses due to the welding process may reduce the buckling strength differently as compared with rolled residual stresses.

This chapter presents an experimental investigation of laterally

unsupported welded beams from a statistical consideration. Thirty-four, nominally identical welded beams for each group of two different lengths, are tested under a concentrated load applied vertically at the midspan of the compression flange. The loading and supporting conditions are the same for the rolled beams of Chapter 2. In order to be able to compare directly the buckling behavior of welded and rolled beams (5), geometrically similar sections are used in the test series; the influences of the variations of the various parameters on the scattered test points of the lateral-torsional buckling are examined for the welded beams. The obtained test results are also compared with the obtained test data in Chapter 2.

3-2 TEST PROGRAM

Specimen and Testing --- Twenty-four 5.02 m long members, having nominally identical cross section of welded I-250 mm x 100 mm x 6 mm x 8 mm (ordinary mild steel) have been prepared. As shown in Fig. 3-1 from 5.02 m long member, beams having two different span length of 1.8 m (D series) and 2.6 m (E series), and a short beam for residual stress measurement, have been cut out. And tensile coupons were cut out from each plate before fabrication. Beams are restrained at both ends against torsion but not warping (5).

The welded built-up sections are made from flame-cut plates. Single-run weld by manual with weld size 6 mm is used as shown in Photo. 3-1.

The nominal cross-sectional properties and beam parameters are shown in Table 3-1, in which the fillet welds are included for calculation.

Tensile Coupon Test --- Four tensile coupons, i.e., two from flange plates and two from web plates, are cut out from each of the 34 members,

making 136 in all.

As shown in Photo. 3-2, a potentiometer is attached with the coupon to control the strain rate.

Residual Stress Measurement --- Longitudinal residual stresses are measured by the sectioning method. The optimum mesh sizes of sectioning are shown in Fig. 3-2, where the mesh sizes have to be even smaller than those for the rolled beams.

Initial Imperfections --- Initial measurements of the crookednesses of 68 beams are taken at the equally-divided five points about the minor axis and in angle of rotation.

3-3 TEST RESULTS FOR INITIAL IMPERFECTIONS

Tensile Coupon Test Results --- Stress strain relationship of 136 plates (flange: t=8 mm, web: t=6 mm) for flange and web is shown in Fig. 3-3. Figures 3-4(a) and 3-4(b), respectively, show mechanical properties of static yield stress σ_y and modulus of elasticity, E, in histograms, in which N = number of coupons. The values of M and M± 2S (M = mean value, S=standard deviation) are also shown by the arrows in the abscissa. Curves fitted to the histograms indicate the corresponding Gaussian distribution curves. The shaded histograms are for flange (t=8 mm) and the plain ones for web (t=6 mm). The yield stress distributions in flange and in web are separated, having two peaks in the histograms. Mean values of σ_y are greater by 24 % in flange, and 43.5 % in web, than the minimum specified value of σ_y = 235 MN/m^2 of ordinary mild steel. On the contrary, there are no such separated distributions in flange and web for E value, as shown in Fig. 3-4(b), given M = 209.2 GN/m^2 and ω = 0.0104.

The yield stresses of flange and web in each test beam are listed in

Table 3-2 and 136 tensile coupons after test are shown in Photo. 3-3.

Initial Crookedness --- Figure 3-5 describes the initial measurements of the crookedness of 68 beams, taken at 5 points about the minor axis and the angle of rotation. The measured initial crookednesses of 68 beams are listed in Table 3-3.

Figures 3-6(a) and 3-6(b), respectively, show nondimensionalized initial crookednesses at the midspan of the 68 beams about the minor axis and angle of rotation [$\bar{\varphi}_0 = \varphi_0(h/\ell) \times 10^{-3}$ in which φ_0 = the initial angle of rotation in radian; and h = the beam height] . Mean values of f_0/ℓ in the minor axis is 0.296 (0.08) x 10^{-3} , and the value of the coefficient of variation is 0.922 (0.66); the value in the parenthesis indicates the rolled beams (5). The mean value of $\bar{\varphi}_0$ is 0.504 (0.138) x 10^{-3} . The goodness-of-fit significant tests by χ^2 and Kolmogorov-Simrnov methods indicate that the frequency distributions of the measured values are close fit the Weibull models.

Residual Stress --- An example of measurement is shown in Photo. 3-4(a) and cut pieces of specimens are shown in Photo. 3-4(b). The measured values of released stress after crosswise cut is compared that after sectioning more in Fig. 3-7.

The measured residual stresses of 34 members are listed in Table 3-4. Figure 3-8 shows the measured residual stress distributions for 34 members, and a summary of the measured results is shown in Fig. 3-9 for M and M± S at the measured gage points. The large magnitudes of the tensile residual stresses approaching the yield point at the weld are in the more than half area of flange and less compressive residual stresses are at the flange tips. The residual stress patterns in the present welded beam-type sections are somewhat different in the flange, as compared to the patterns for the

column-type sections (7,8). The measured residual stress distributions of 25 rolled beams (5) are also shown for comparison in Fig. 3-8.

The modified residual stresses $\bar{\sigma}_{rc}$ may be defined to represent the average compression residual stress nondimensionalized by the actual σ_y in the compression flange, as

$$\bar{\sigma}_{rc} = \int_{A_c} \sigma_{rc} \quad dA_c \ / \ A_c \cdot \sigma_y \qquad \text{----(3-1)}$$

in which σ_{rc} = compressive residual stresses in the compressive flange; and A_c = compressive residual stress area in the compression flange. Fig. 3-10 shows the variation of $\bar{\sigma}_{rc}$ for the welded and rolled sections, respectively. The mean value and the coefficient of variation of $\bar{\sigma}_{rc}$ are 0.120 and 0.466 for the welded beams, respectively. For the rolled beams, the mean value and the coefficient of variation of $\bar{\sigma}_{rc}$ are calculated as 0.066 and 0.241 from the measured residual stress distributions (5).

Cross-Sectional Area --- The measured cross-sectional dimensions of 68 beams are listed in Table 3-5. Figure 3-11 shows a measured histogram of the cross-sectional variation of the 34 members, taking the area and the nondimensionalized area A/A_{nom} in the abscissa, in which A_{nom} = the nominal area. The mean area is equal to the nominal area. The values M± 2S, and the corresponding Gaussian distribution curve, are also given in Fig. 3-11.

3-4 LATERAL-TORSIONAL BUCKLING TEST RESULTS

Ultimate Strength --- Photo. 3-5 shows a test beam (E-6) during test. Table 3-6 lists the maximum load P_{max} of the 34 nominally identical beams (1-34), and the resulting M, S, and ω values for the D and E series. The P_{max} values are shown in histograms in Fig. 3-12 for D and E series. The

corresponding Gaussian curve, and the measured M and M± 2S values, are also given in Fig. 3-12.

A variation of the ultimate strength of the test beams is shown in Fig. 3-13, in which M_u/M_p is in the ordinate; and $\bar{\lambda}=\sqrt{M_p/M_E}$ is in the abscissa, in which M_p = the full plastic moment; and M_E = the elastic critical moment. The $\bar{\lambda}$ values are listed in Table 3-6. The ultimate moment $M_u = P_{max} \cdot 1/4$, is at the midspan. The lower part of the figure shows the corresponding coefficient of variation ω.

In order to present the test data in the nondimensionalized form, the following two M_p values are used for comparison: (1) The measured σ_y and the measured cross-sectional dimensions [the different σ_y values in flange and in web are used to determine the plastic moment $(M_p)_a$] ; and (2) the nominal σ_y = 235 MN/m^2 and the nominal cross-sectional dimensions that lead to $(M_p)_n$ = 66.85 kNm. Table 3-6 lists a summary of the test results of the 34 beams in D and E series. The test data of beams 1-34 are nondimensionalized by using $(M_p)_a$. The values of M, S, and ω are also given in each series.

It may be seen from Table 3-6 and Figs. 3-12 and 3-13 that the considerable variation of the ultimate strength still exists after the test data are nondimensionalized by $(M_p)_n$. However, by using $(M_p)_a$, the variation of the test data becomes small. Influence of the variation of $(M_p)_a$, due to the actual yield stresses is comparatively profound on the variation of the ultimate strength for the D series.

3-5 STATISTICAL EVALUATION

Statistical Test Data --- Statistical data of the geometrical and material properties and of the ultimate strength are summarized for the

mean, standard deviation, and coefficient of variation in Table 3-7. In this table, large values of coefficients of variation of initial crookednesses are obtained among other imperfection parameters and this will be explained that the frequency distribution of the initial crookednesses may be approximated by the Weibull model while the other imperfections may be approximation by the Gaussian models.

Influence of Imperfection Parameters on Variation of Ultimate Strength --- The following three parameters will be used to investigate their influence on the strength variation of welded beams: (1) The full plastic moment, M_p, which was reflected by the variations of the yield stress and of the cross-sectional dimensions; (2) the modified residual stresses $\bar{\sigma}_{rc}$, in Eq. (3-1); and (3) initial crookednesses, u_o/ℓ, in the lateral direction.

M_u Versus M_p --- Figures 3-14(a) and 3-14(b) show scattergrams of 34 pairs of M_p and M_u in welded series and the data of 25 pairs for rolled beams A, B, and C series are also plotted, respectively. The estimate lines for the A~E series are also shown, based on the simple linear model between the values of M_p and M_u.

The sample correlation coefficients $r(M_p, M_u)$ for the pairs of M_p and M_u are summarized in Table 3-8, in which the values of $r(M_p, M_u)$ are 0.70 and 0.37 in the D and E series, respectively, and they are less than those for the rolled A, B, and C series. The degree of correlation between M_p and M_u becomes small with an increasing slenderness function of both the rolled and welded beams.

$M_u/(M_p)_a$ Versus Residual Stress --- Figure 3-15 compares the scattergram of 34 pairs of $\bar{\sigma}_{rc}$ and $M_u/(M_p)_a$ in welded D series, and 25 pairs in rolled B series. Both the D and B series are nearly the same

nominal slenderness function: $\bar{\lambda}_D$ = 0.848 and $\bar{\lambda}_B$ = 0.847. This figure shows that the mean value of $\bar{\sigma}_{rc}$ in welded beams, which is 1.82 times larger than those in rolled beams may be a lower $M_u/(M_p)_a$ on the mean base. The small values of the sample correlation coefficient $r[\bar{\sigma}_{rc}, M_u/(M_p)_a]$ between the modified residual stresses and $M_u/(M_p)_a$ which are listed in Tables 3-8, may indicate that there exists very little correlation between the variation of $\bar{\sigma}_{rc}$ and $M_u/(M_p)_a$ for welded D, as well as E series. However, the values of $r[\bar{\sigma}_{rc}, M_u/(M_p)_a]$ being large in negative for rolled B, as well as the A and C series, indicate that the variation of $\bar{\sigma}_{rc}$ may reduce the $M_u/(M_p)_a$ values.

The difference of the mean value of $\bar{\sigma}_{rc}$ being 0.054 between the D and B series may be a potential factor in reducing the $M_u/(M_p)_a$ value from 0.823 to 0.790.

$M_u/(M_p)_a$ Versus Initial Crookedness --- Figure 3-16 shows a scattergram of u_o/ℓ and $M_u/(M_p)_a$ in both the D and B series. The figure and the simple correlation coefficient in Table 3-8, between u_o/ℓ and $M_u/(M_p)_a$ which is small and nearly equal to zero, indicates very little correlation between the two variates. However, the u_o/ℓ values in welded beams where the mean u_o/ℓ value is 4.38 times larger than that in rolled beams, may slightly reduce the $M_u/(M_p)_a$ value on the mean base.

3-6 COMPARISON WITH ULTIMATE STRENGTH OF ROLLED BEAMS

In Fig. 3-17, the present results for 68 welded beams are compared with the previous 75 rolled test points (5). The test points are nondimensionalized by the actual yield stresses and cross-sectional dimensions. The coefficients of variation are also given in Fig. 3-17. Under the same supporting and loading conditions for the welded and rolled

beams, the welded beams have a lower ultimate strength than the rolled beams in the mean values, and the coefficients of variation of ω_D = 0.079 and ω_E = 0.110 are considerably large,as compared with the values of ω_A =0.0564, ω_B =0.0291, ω_C = 0.0245 in the rolled beams.

3-7 CONCLUSIONS

Statistical buckling tests are carried out for the 34, nominally identical welded beams in each D and E series. The beams are tested under the same loading and supported conditions used for the rolled beams (5). The variation of the geometrical and material imperfections are obtained and the degree of correlations between the scattered ultimate strength and the imperfection parameters are investigated.

The main imperfection parameter that highly influences the variation of the ultimate strength, is the variable value of the actual full plastic moment that contains actual yield stress and cross-sectional dimensions. The degree of correlation between the ultimate strength and $(M_p)_a$ becomes small with an increasing slenderness function.

The coefficients of variation of $M_u/(M_p)_a$ that are nondimensionalized by the actual $(M_p)_a$ values become less, and these are 22.6 % and 1.8 % less than those of P_{max} [or $M_u/(M_p)_n$] for the D and E series, respectively.

Measurements of compressive residual stresses and initial crookednesses in the welded beams have shown that the mean and the coefficient of variation are significantly large in the welded beams, as compared with those in the rolled beams. The welded beams have a lower ultimate strength, $M_u/(M_p)_a$ in the mean value than the rolled beams. Large variation of the ultimate strength $M_u/(M_p)_a$ in the welded beams may be due to the large variation in the compressive residual stresses and the initial

lateral crookednesses.

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NOTATION

The following symbols are used in this chapter:

A = area of cross section;
A_c = compressive residual stress area;
A_{nom} = nominal area;
E = modulus of elasticity;
f = frequency;
h = beam height;
I_x = moment of inertia about the major axis;
I_y = moment of inertia about the minor axis;
I_ω = warping moment of inertia;
J = torsional constant;
ℓ = span length of member;
M = mean value;
M_E = elastic critical moment;
M_p = plastic moment;
$(M_p)_a$ = plastic moment for actual yield stress and cross section;
$(M_p)_n$ = nominal plastic moment;
M_u = ultimate bending moment;
N = number of data;
P_{max} = maximum load;
r = sample correlation coefficient;
S = standard deviation;
t = plate thickness;
u_o = initial deflection in lateral direction;
$\bar{\lambda}$ = slenderness function;
σ_{rc} = compressive residual stress in compression flange;
$\bar{\sigma}_{rc}$ = nondimensional average residual compressive stress;
σ_y = yield stress;
σ_{yf} = yield stress in flange;
σ_{yw} = yield stress in web;
φ_o = initial angle of rotation; and
ω = coefficient of variation.

Table 3-1 Nominal Cross-Sectional Properties

Series	Span Length (mm)	$\bar{\lambda}$	M_E (kNm)	No. of Beams
D	1800	0.837	95.42	34
E	2600	1.095	55.75	34
$A = 30.76\ cm^2$		$I_x = 3079\ cm^4$		$I_y = 134\ cm^4$
$J = 6.118\ cm^4$		$I_\omega = 19521\ cm^6$		$M_p = 66.85$ kNm
$f = 1.153$		$r_y = 2.087$ cm		

A = Area of Cross Section

Ix= Moment of Inertia about the Strong Axis

Iy= Moment of Inertia about the Weak Axis

J = Torsional Constant

Iω= Warping Moment of Inertia

f = Shape Factor

Mp= Full Plastic Moment

M_E= Elastic Critical Moment

$\bar{\lambda}$ = Slenderness Function $\sqrt{M_p/M_E}$

Table 3-2 Yield Stresses of Flange and Web in Each Test Beam

Beam No.	σ_{yf} (N/mm^2)	σ_{yw} (N/mm^2)
1	306.2	359.9
2	311.4	363.7
3	313.0	346.3
4	311.9	347.8
5	313.8	355.0
6	280.6	371.3
7	283.6	330.4
8	282.0	346.4
9	277.7	347.9
10	273.6	349.9
11	276.5	356.1
12	280.1	338.3
13	277.2	354.4
14	275.2	364.7
15	270.8	364.2
16	288.8	365.6
17	293.1	372.3
18	299.0	353.7
19	304.0	339.4
20	302.9	341.5
21	275.1	343.8
22	275.1	345.6
23	272.9	346.7
24	271.9	337.2
25	266.6	294.1
26	271.9	279.5
27	277.7	298.8
28	279.0	291.1
29	266.4	298.5
30	266.2	298.5
31	267.2	299.5
32	277.7	321.4
33	277.4	325.0
34	273.4	334.6

Table 3-3 Initial Crookednesses of 68 Beams (x 10^{-5})

Beam No.	u_o			φ_o		
	0.25L	0.50L	0.75L	0.25L	0.5L	0.75L
D- 1	56.1	65.0	52.2	2.0	15.0	54.5
D- 2	16.7	17.2	0.0	94.6	86.5	89.2
D- 3	30.0	50.6	55.6	27.2	119.8	27.2
D- 4	1.1	18.9	58.9	28.6	17.7	9.5
D- 5	2.2	16.1	1.7	96.7	53.8	19.7
D- 6	52.2	40.0	20.6	44.9	1.4	9.5
D- 7	5.6	12.8	64.4	34.0	121.9	94.0
D- 8	30.0	10.6	38.9	135.5	82.4	23.8
D- 9	60.0	16.1	20.6	40.2	8.2	80.3
D-10	27.2	17.8	45.0	22.5	3.4	13.6
D-11	47.8	79.4	28.9	155.9	163.4	122.5
D-12	16.1	6.7	47.8	86.5	55.1	3.4
D-13	84.4	41.7	60.0	32.0	62.6	32.7
D-14	3.9	8.3	27.2	18.4	8.9	40.2
D-15	6.7	18.3	70.6	27.9	51.1	4.8
D-16	36.1	33.9	37.2	23.8	12.9	18.4
D-17	29.4	52.8	6.1	60.6	63.3	66.7
D-18	105.0	23.3	38.3	92.6	48.3	87.8
D-19	7.8	30.6	6.7	41.5	7.5	23.1
D-20	13.9	14.4	40.6	31.3	128.7	67.4
D-21	33.9	36.1	35.0	78.3	113.0	63.3
D-22	8.3	2.2	48.9	68.1	53.8	42.9
D-23	16.7	2.2	18.9	0.7	51.1	22.5
D-24	62.2	19.4	47.8	8.2	47.0	62.0
D-25	11.1	15.0	7.8	13.6	2.7	25.9
D-26	24.4	28.3	5.0	96.0	70.1	75.6
D-27	39.4	17.8	0.0	21.8	50.4	49.0
D-28	17.2	48.3	12.8	36.1	35.4	20.4
D-29	17.8	16.7	3.9	19.1	23.8	72.8
D-30	15.6	5.6	46.7	51.7	60.6	59.2
D-31	32.2	23.3	35.6	36.8	19.1	6.1
D-32	108.9	80.0	26.7	6.1	7.5	27.2
D-33	2.8	3.9	23.3	31.3	70.8	16.3
D-34	15.0	109.4	38.3	9.5	25.2	138.2

Table 3-3 Continued

Beam No.	u_o			φ_o		
	0.25L	0.50L	0.75L	0.25L	0.5L	0.75L
E- 1	28.1	23.5	1.2	24.5	14.1	35.4
E- 2	14.6	2.3	51.5	58.0	33.9	72.1
E- 3	5.4	34.2	28.8	62.2	112.7	63.2
E- 4	7.3	28.8	19.6	50.0	20.3	7.1
E- 5	26.2	16.2	26.9	28.8	60.3	34.4
E- 6	25.8	53.8	30.4	0.5	76.8	111.7
E- 7	18.8	38.5	26.2	24.5	94.3	15.1
E- 8	58.1	10.4	14.6	42.9	16.0	40.5
E- 9	9.2	1.2	22.3	126.3	121.6	87.2
E-10	8.5	7.3	2.3	5.7	14.1	52.8
E-11	16.5	27.3	23.1	99.0	113.6	72.6
E-12	24.2	11.2	10.8	6.1	64.6	60.3
E-13	21.5	16.2	7.7	64.6	83.4	63.2
E-14	55.8	26.9	53.8	42.9	74.9	51.4
E-15	25.8	11.9	63.1	44.3	10.4	28.3
E-16	22.7	15.8	5.0	66.0	66.5	4.7
E-17	8.8	0.0	7.3	24.0	40.1	31.6
E-18	6.2	50.4	33.8	24.0	21.2	50.4
E-19	19.2	52.7	36.2	5.2	16.0	55.6
E-20	43.1	13.8	51.5	29.2	136.7	140.9
E-21	36.9	36.9	20.0	55.6	25.5	73.1
E-22	17.7	3.5	3.1	9.0	62.2	80.1
E-23	27.7	18.5	26.9	15.6	49.5	4.7
E-24	19.6	13.1	64.6	24.0	15.1	66.0
E-25	35.4	1.2	11.5	7.1	33.0	8.5
E-26	49.2	14.2	6.9	84.8	9.9	7.1
E-27	24.2	16.5	16.5	64.1	28.3	42.4
E-28	1.5	27.3	18.1	28.3	66.5	96.2
E-29	57.3	63.8	61.2	17.9	9.4	2.4
E-30	10.4	52.7	20.4	34.4	42.4	6.1
E-31	15.4	20.8	13.5	49.0	36.3	7.1
E-32	51.2	75.4	37.3	13.2	5.7	13.2
E-33	83.1	132.7	80.8	17.0	19.8	22.6
E-34	90.8	108.5	71.9	41.5	87.7	107.9

Table 3-4 Residual Stresses of 34 Members, in N/mm^2

Point No.	Beam No. 1 Upper Flange	Beam No. 1 Lower Flange	Beam No. 1 Web	Beam No. 2 Upper Flange	Beam No. 2 Lower Flange	Beam No. 2 Web	Beam No. 3 Upper Flange	Beam No. 3 Lower Flange	Beam No. 3 Web
1	-80.3	-51.5	213.2	-39.1	-84.4	228.6	-20.6	-76.2	353.2
2	-33.0	-45.3	151.4	-83.5	-78.3	113.3	-17.6	-74.1	162.7
3	-16.5	-27.9	-79.3	-67.0	-77.5	-110.8	-20.6	-16.5	110.2
4	-154.5	-167.9	-163.8	-52.6	-77.3	-173.0	-131.8	-52.6	-166.8
5	-4.1	-22.7	-163.8	10.8	-72.1	-183.2	-130.8	-117.4	-150.3
6	161.7	160.6	-158.6	230.7	212.1	-164.8	116.4	125.6	-159.7
7	318.2	339.8	-155.5	343.9	329.5	-175.0	358.3	249.0	-151.4
8	358.3	352.2	-146.2	387.2	348.0	-160.6	327.4	362.5	-216.2
9	281.2	256.4	-144.2	328.5	321.3	-157.6	331.6	327.4	-164.8
10	138.0	37.1	-144.2	253.3	140.0	-146.2	107.1	311.0	-151.4
11	2.1	-13.4	-139.1	-4.1	-78.3	-150.3	-25.8	-98.9	-150.3
12	-122.6	-68.0	-123.6	-108.2	-75.2	-133.9	-61.8	-100.9	-146.2
13	-18.5	-20.6	-49.4	-35.0	-35.0	-108.2	20.6	-43.2	-59.7
14	-37.1	-45.3	143.2	-61.8	-61.8	117.4	-30.9	-54.6	208.0
15	-45.3	-51.5	155.5	-90.6	-86.5	156.7	-37.1	-86.5	195.6

Point No.	Beam No. 4 Upper Flange	Beam No. 4 Lower Flange	Beam No. 4 Web	Beam No. 5 Upper Flange	Beam No. 5 Lower Flange	Beam No. 5 Web	Beam No. 6 Upper Flange	Beam No. 6 Lower Flange	Beam No. 6 Web
1	-57.7	135.9	265.7	-47.4	-37.1	243.0	-68.0	-47.4	315.1
2	-63.8	39.1	-32.0	-45.3	-37.1	201.8	-63.8	-41.2	125.6
3	-35.0	-69.0	-103.0	-26.8	-22.7	-59.7	-41.2	-45.3	-158.6
4	-38.1	-82.4	-170.9	-53.5	-105.0	-162.7	0.0	-33.0	-179.2
5	-125.6	-105.0	-165.8	-90.6	-115.3	-158.6	45.3	-86.5	-146.2
6	147.3	353.1	-173.0	160.6	234.8	-158.6	181.2	183.3	-168.9
7	358.3	444.8	-158.6	356.3	354.2	-150.3	339.8	311.0	-150.3
8	366.6	442.8	-152.4	422.2	395.4	-278.0	358.3	333.6	-129.7
9	293.5	300.7	-170.9	335.7	321.3	-156.5	350.1	321.3	-142.1
10	216.2	221.4	-153.5	222.4	150.3	-138.0	230.7	360.4	-129.7
11	-90.6	-84.4	-142.1	-92.7	-65.9	-133.9	-33.0	-68.0	-123.6
12	-76.0	-133.9	-151.4	-103.0	-57.7	-129.7	-63.8	-39.1	-115.3
13	-57.7	-65.9	-93.8	-84.4	-68.0	-63.8	-28.8	-63.8	-100.9
14	-87.6	-109.1	119.4	-16.5	-74.1	154.5	-51.5	-80.3	158.6
15	-14.4	-96.8	193.6	-18.5	-80.3	218.3	-53.5	-94.7	333.6

Table 3-4 Continued

Point No.	Beam No. 7			Beam No. 8			Beam No. 9		
	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web
1	-16.5	-49.4	300.7	-59.7	-68.0	282.1	-22.7	-70.0	212.1
2	0.0	-72.1	185.3	-51.5	-98.9	138.0	-10.3	-72.1	152.4
3	14.4	-18.5	-109.1	-41.2	-84.4	-103.0	4.1	-35.0	-72.1
4	-20.6	-14.4	-203.9	-22.7	-49.4	-177.1	-10.3	-45.3	-185.3
5	-47.4	-63.8	-179.2	-57.7	-6.2	-175.0	30.9	26.8	-191.5
6	140.0	201.8	-156.5	156.5	226.5	-158.6	144.2	226.5	-168.9
7	321.3	356.3	-154.5	329.5	323.3	-164.8	263.6	313.0	-175.0
8	327.4	337.7	-148.3	346.0	337.7	-152.4	265.7	358.3	-164.8
9	327.4	348.0	-138.0	308.9	313.0	-138.0	220.4	317.1	-164.8
10	243.0	154.5	-140.0	208.0	119.4	-129.7	185.3	183.3	-156.5
11	43.2	-45.3	-148.3	-53.5	-88.6	-138.0	78.3	57.7	-150.3
12	-41.2	14.4	-135.9	-90.6	6.2	-133.9	-26.8	-47.4	-135.9
13	-49.4	-26.8	-68.0	-26.8	-10.3	-113.3	-20.6	-45.3	-72.1
14	-41.2	-33.0	144.2	-84.4	-47.4	142.1	-33.0	-65.9	135.9
15	-61.8	-28.8	339.8	-70.0	-51.5	162.7	-43.2	-80.3	278.0

Point No.	Beam No. 10			Beam No. 11			Beam No. 12		
	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web
1	20.6	-39.1	160.6	-51.5	-51.5	284.2	-14.4	-35.0	385.1
2	26.8	-35.0	113.3	-18.5	-45.3	129.7	-4.1	-16.5	179.2
3	41.2	-18.5	-55.6	-6.2	-47.4	-94.7	-18.5	-8.2	-76.2
4	-2.1	6.2	-166.8	-49.4	-61.8	-183.3	-16.5	-30.9	-183.3
5	-80.3	-70.0	-175.0	146.2	-10.3	-175.0	8.2	68.0	-185.3
6	63.8	199.8	-168.9	249.2	183.3	-170.9	199.8	201.8	-148.3
7	259.5	296.6	-154.5	329.5	319.2	-179.2	319.2	333.6	-160.6
8	327.4	378.9	-152.4	356.3	311.0	-164.8	385.1	350.1	-179.2
9	280.1	319.2	-148.3	331.6	284.2	-164.8	315.1	298.6	-197.7
10	173.0	189.5	-142.1	175.0	193.6	-166.8	247.1	78.3	-216.2
11	37.1	39.1	-135.9	-30.9	4.1	-160.6	2.1	-45.3	-222.4
12	-16.5	-14.4	-123.6	-22.7	-55.6	-154.5	-24.7	-12.4	-236.8
13	-12.4	-37.1	-61.8	-16.5	-76.2	-43.2	10.3	61.8	-59.7
14	-39.1	-39.1	127.7	-24.7	-90.6	140.0	-8.2	28.8	55.6
15	-53.5	-65.9	144.2	-41.2	-76.2	300.7	-22.7	26.8	304.8

Table 3-4 Continued

Point No.	Beam No. 13			Beam No. 14			Beam No. 15		
	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web
1	-30.9	-45.3	158.6	-86.5	-45.3	337.7	-68.0	-63.8	333.6
2	-20.6	-43.2	144.2	-80.3	-35.0	119.4	-59.7	-65.9	187.4
3	-10.3	-30.9	-146.2	-59.7	-18.5	-82.4	-43.2	-47.4	-70.0
4	-20.6	-35.0	-179.2	-45.3	-49.4	-123.6	-47.4	-53.5	-187.4
5	59.7	51.5	-183.3	-37.1	-6.2	-135.9	-12.4	30.9	-175.0
6	208.0	247.1	-183.3	152.4	129.7	-138.0	189.5	224.5	-166.8
7	348.0	339.8	-183.3	327.4	329.5	-154.5	329.5	321.3	-148.3
8	350.1	341.9	-173.0	337.7	329.5	-150.3	313.0	311.0	-177.1
9	313.0	319.2	-181.2	327.4	362.5	-156.5	300.7	323.3	-166.8
10	228.6	138.0	-179.2	203.9	218.3	-164.8	220.4	166.8	-166.8
11	53.5	-24.7	-166.8	-35.0	-8.2	-170.9	22.7	-26.8	-138.0
12	-39.1	-22.7	-160.6	-65.9	-47.4	-173.0	-51.5	-45.3	-152.4
13	-43.2	-14.4	111.2	-55.6	-18.5	-123.6	-65.9	-37.1	-107.1
14	-45.3	-14.4	98.9	-57.7	-37.1	115.3	-92.7	-45.3	251.2
15	-49.4	-20.6	308.9	-78.3	-45.3	191.5	-88.6	-53.5	333.6

Point No.	Beam No. 16			Beam No. 17			Beam No. 18		
	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web
1	-88.6	-96.8	343.9	-80.3	-127.7	222.4	-51.5	-63.8	175.0
2	-59.7	-70.0	146.2	-63.8	-94.7	177.1	-41.2	-53.5	125.6
3	-33.0	-45.3	-26.8	-51.5	-4.1	-16.5	-16.5	-57.7	-63.8
4	-59.7	-47.4	-135.9	-68.0	39.1	-160.6	2.1	-88.6	-123.6
5	-57.7	-18.5	-142.1	16.5	10.3	-162.7	-65.9	-65.9	-135.9
6	142.1	195.6	-133.9	187.4	232.7	-158.6	156.5	160.6	-133.9
7	333.6	333.6	-156.5	358.3	290.4	-160.6	292.4	337.7	-133.9
8	335.7	335.7	-160.6	358.3	348.0	-160.6	331.6	358.3	-129.7
9	339.8	337.7	-142.1	306.9	138.0	-142.1	288.3	325.4	-125.6
10	88.6	146.2	-138.0	212.1	177.1	-138.0	238.9	152.4	-131.8
11	-76.2	-78.3	-131.8	26.8	-111.2	-146.2	-113.3	-22.7	-133.9
12	-68.0	-57.7	-121.5	-86.5	-90.6	-150.3	-53.5	-35.0	-158.6
13	-37.1	-76.2	-28.8	-39.1	-24.7	-37.1	2.1	-45.3	-65.9
14	-37.1	-96.8	179.2	-49.4	-51.5	170.9	-12.4	-70.0	131.8
15	-59.7	-107.1	339.8	-61.8	-74.1	271.8	-10.3	-88.6	300.7

Table 3-4 Continued

Point No.	Beam No. 19			Beam No. 20			Beam No. 21		
	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web
1	-18.5	-45.3	300.7	-70.0	-70.0	308.9	-28.8	-30.9	226.5
2	-18.5	-30.9	119.4	-55.6	-22.7	177.1	-35.0	-24.7	140.0
3	10.3	-10.3	-98.9	-49.4	-10.3	20.6	-10.3	-20.6	-49.4
4	-59.7	-26.8	-166.8	-41.2	-51.5	-121.5	-43.2	-22.7	-164.8
5	-55.6	-26.8	-166.8	-30.9	20.6	-179.2	-45.3	-53.5	-156.5
6	166.8	181.2	-164.8	226.5	193.6	-177.1	117.4	121.5	-154.5
7	321.3	278.0	-158.6	333.6	343.9	-183.3	286.3	304.8	-146.2
8	331.6	306.9	-164.8	352.2	339.8	-179.2	331.6	350.1	-129.7
9	311.0	288.3	-156.5	313.0	331.6	-175.0	294.5	308.9	-131.8
10	224.5	160.6	-146.2	43.2	181.2	-168.9	216.2	210.1	-142.1
11	16.5	-26.8	-164.8	-88.6	20.6	-181.2	33.0	8.2	-127.7
12	-88.6	-55.6	-160.6	-111.2	4.1	-179.2	-53.5	70.0	-113.3
13	-43.2	-28.8	51.5	-45.3	-8.2	-117.4	-28.8	-30.9	-72.1
14	-57.7	-47.4	150.3	-70.0	-33.0	117.4	-41.2	-92.7	138.0
15	-61.8	-72.1	257.4	-70.0	-57.7	383.0	-47.4	-100.9	234.8

Point No.	Beam No. 22			Beam No. 23			Beam No. 24		
	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web
1	-39.1	-30.9	346.0	-35.0	-72.1	273.9	103.0	-2.1	109.1
2	-41.2	-28.8	150.3	-16.5	-37.1	131.8	35.0	-6.2	84.4
3	-28.8	-14.4	-10.3	-6.2	-16.5	-4.1	63.8	12.4	24.7
4	-22.7	-45.3	-150.3	-37.1	-41.2	-195.6	33.0	-20.6	-148.3
5	-55.6	14.4	-156.5	18.5	49.4	-208.0	-22.7	103.0	-150.3
6	105.0	168.9	-146.2	183.3	224.5	-193.6	107.1	175.0	-146.2
7	311.0	249.2	-142.1	292.4	337.7	-185.3	249.2	210.1	-146.2
8	304.8	263.6	-146.2	271.8	337.7	-166.8	319.2	267.7	-142.1
9	331.6	269.8	-135.9	271.8	317.1	-166.8	230.7	181.2	-150.3
10	168.9	191.5	-152.4	193.6	146.2	-173.0	236.8	115.3	-135.9
11	-26.8	16.5	-138.0	76.2	-24.7	-144.2	47.4	76.2	-140.0
12	-41.2	-28.8	-138.0	-33.0	-6.2	-158.6	-30.9	39.1	-131.8
13	-16.5	-16.5	-107.1	-26.8	-14.4	-22.7	-22.7	92.7	-33.0
14	-20.6	-41.2	135.9	-41.2	-14.4	131.8	-51.5	65.9	63.8
15	-28.8	-55.6	243.0	-51.5	-18.5	311.0	-45.3	37.1	90.6

Table 3-4 Continued

Point No.	Beam No. 25			Beam No. 26			Beam No. 27		
	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web
1	-55.6	-68.0	259.5	-65.9	-80.3	273.9	-35.0	-22.7	199.8
2	-43.2	-68.0	140.0	-61.8	-78.3	140.0	-24.7	-10.3	127.7
3	-35.0	-43.2	-82.4	-49.4	-55.6	-65.9	-2.1	14.4	-88.6
4	-47.4	-43.2	-146.2	-65.9	-53.5	-216.2	-43.2	10.3	-197.7
5	37.1	-57.7	-152.4	-6.2	-24.7	-220.4	6.2	-59.7	-183.3
6	158.6	220.4	-140.0	195.6	142.1	-205.9	177.1	88.6	-168.9
7	321.3	300.7	-135.9	315.1	290.4	-191.5	304.8	288.3	-168.9
8	331.6	311.0	-138.0	346.0	387.2	-173.0	372.8	323.3	-162.7
9	321.3	304.8	-138.0	313.0	311.0	-185.3	304.8	296.6	-152.4
10	199.8	119.4	-138.0	177.1	208.0	-205.9	160.6	175.0	-162.7
11	10.3	-41.2	-133.9	14.4	131.8	-125.6	-30.9	6.2	-164.8
12	-37.1	-57.7	-82.4	-61.8	39.1	-107.1	-10.3	-53.5	-170.9
13	-6.2	-53.5	68.0	-41.2	-24.7	-47.4	26.8	-35.0	-26.8
14	-8.2	-53.5	220.4	-63.8	-37.1	94.7	4.1	-47.4	168.9
15	-30.9	-55.6	331.6	-72.1	-70.0	135.9	-2.1	-47.4	253.3

Point No.	Beam No. 28			Beam No. 29			Beam No. 30		
	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web
1	-70.0	-86.5	150.3	-61.8	-45.3	234.8	12.4	-16.5	251.2
2	-33.0	-55.6	140.0	-37.1	-20.6	109.1	41.2	8.2	140.0
3	-14.4	-26.8	-37.1	-10.3	-8.2	10.3	59.7	10.3	-30.9
4	-72.1	-26.8	-146.2	-14.4	-4.1	-150.3	37.1	24.7	-170.9
5	-33.0	45.3	-146.2	55.6	51.5	-156.5	-22.7	43.2	-177.1
6	142.1	222.4	-144.2	177.1	173.0	-152.4	80.3	191.5	-175.0
7	249.2	288.3	-142.1	160.6	304.8	-160.6	253.3	308.9	-166.8
8	284.2	346.0	-138.0	321.3	296.6	-164.8	259.5	304.8	-170.9
9	189.5	288.3	-146.2	261.5	302.7	-156.5	271.8	315.1	-164.8
10	173.0	201.8	-135.9	39.1	80.3	-164.8	154.5	158.6	-175.0
11	26.8	-39.1	-138.0	-53.5	-8.2	-164.8	47.4	2.1	-168.9
12	-28.8	-86.5	-135.9	6.2	61.8	-164.8	-22.7	24.7	-179.2
13	-8.2	-14.4	-6.2	35.0	72.1	-30.9	-30.9	2.1	-90.6
14	30.9	-57.7	150.3	-2.1	37.1	164.8	-57.7	-12.4	115.3
15	-78.3	-78.3	197.7	-12.4	22.7	296.6	-121.5	-20.6	304.8

Table 3-4 Continued

Point No.	Beam No. 31			Beam No. 32			Beam No. 33		
	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web	Upper Flange	Lower Flange	Web
1	-78.3	-39.1	306.9	-51.5	-47.4	158.6	-8.2	-37.1	162.7
2	-68.0	-16.5	208.0	-72.1	-20.6	142.1	2.1	-12.4	144.2
3	-37.1	2.1	-51.5	-55.6	-2.1	-113.3	26.8	16.5	-16.5
4	-51.5	55.6	-131.8	-80.3	-41.2	-164.8	8.2	8.2	-187.4
5	76.2	26.8	-129.7	-45.3	-16.5	-156.5	70.0	45.3	-170.9
6	226.5	170.9	-148.3	168.9	131.8	-146.2	193.6	175.0	-152.4
7	346.0	300.7	-144.2	317.1	224.5	-152.4	290.4	333.6	-152.4
8	337.7	278.0	-158.6	321.3	220.4	-144.2	348.0	315.1	-154.5
9	321.3	325.4	-160.6	313.0	354.2	-150.3	308.9	253.3	-156.5
10	230.7	47.4	-168.9	142.1	109.1	-146.2	109.1	158.6	-158.6
11	-2.1	-33.0	-168.9	-76.2	-26.8	-170.9	26.8	-20.6	-156.5
12	-51.5	41.2	-201.8	0.0	-49.4	-181.2	22.7	-26.8	-160.6
13	-59.7	78.3	-41.2	-12.4	8.2	16.5	68.0	53.5	-24.7
14	-84.4	53.5	170.9	-28.8	10.3	80.3	63.8	49.4	173.0
15	-90.6	37.1	306.9	-59.7	-10.3	164.8	45.3	12.4	187.4

Point No.	Beam No. 34		
	Upper Flange	Lower Flange	Web
1	-30.9	-78.3	164.8
2	-12.4	-59.7	133.9
3	14.4	-39.1	-92.7
4	33.0	-22.7	-185.3
5	55.6	-4.1	-185.3
6	193.6	203.9	-170.9
7	269.8	306.9	-168.9
8	339.8	317.1	-164.8
9	282.1	228.6	-152.4
10	199.8	168.9	-150.3
11	43.2	-59.7	-133.9
12	-8.2	-22.7	-96.8
13	2.1	4.1	-57.7
14	-35.0	-8.2	107.1
15	-45.3	-109.1	133.9

Table 3-5 Measured Cross-Sectional Dimensions, in mm

Beam No.	E-Series b	h	t	w	f	D-Series b	h	t	w	f
1	101.2	251.9	8.26	6.23	6.63	101.1	252.3	8.29	6.26	6.12
2	100.8	252.2	8.38	6.17	6.13	100.7	252.5	8.33	6.20	6.06
3	101.2	252.7	8.39	6.16	6.00	101.2	252.6	8.38	6.19	5.75
4	100.9	251.4	8.44	6.08	5.81	100.8	252.2	8.36	6.14	5.75
5	101.2	252.2	8.45	6.00	6.08	101.2	251.5	8.36	6.00	5.75
6	100.6	250.9	7.90	5.95	5.25	100.8	250.4	7.93	5.93	5.32
7	100.4	250.7	7.80	5.79	5.90	100.3	251.4	7.92	5.78	5.88
8	100.6	250.6	7.85	5.83	5.60	100.5	251.4	7.94	5.80	5.00
9	100.6	250.7	7.90	5.86	6.20	100.4	251.3	7.92	5.79	6.31
10	99.8	250.4	7.90	5.85	5.81	99.9	250.8	7.93	5.79	5.75
11	100.9	250.1	7.77	5.84	5.50	100.9	250.5	7.86	5.79	5.81
12	101.1	249.9	7.86	5.80	6.31	101.0	250.5	7.83	5.75	6.13
13	100.9	250.2	7.73	5.81	6.13	100.9	250.8	7.84	5.80	6.31
14	99.8	249.6	7.75	5.85	5.94	99.8	250.0	7.80	5.85	5.31
15	101.0	250.4	7.70	5.86	6.19	100.9	250.2	7.72	5.85	6.44
16	100.8	250.5	8.13	5.85	5.69	100.8	251.1	8.15	5.84	5.50
17	100.9	251.2	8.20	5.86	6.25	100.9	251.2	8.21	5.84	6.06
18	100.8	251.7	8.27	5.82	5.38	100.8	251.6	8.33	5.81	5.50
19	100.6	251.2	8.27	5.72	6.44	100.5	251.4	8.45	5.70	6.50
20	101.0	252.2	8.34	5.76	6.13	100.9	252.0	8.40	5.75	6.19
21	101.0	250.2	7.84	5.76	5.81	101.0	249.8	7.78	5.78	5.88
22	101.1	250.1	7.65	5.77	5.75	101.0	250.0	7.74	5.76	5.63
23	101.0	249.6	7.81	5.77	5.88	101.0	249.7	7.73	5.74	5.88
24	99.7	249.4	7.70	5.72	5.70	99.7	249.1	7.70	5.72	6.25
25	101.1	249.9	7.62	5.87	5.70	101.0	249.8	7.64	5.91	5.94
26	99.5	251.1	8.04	5.87	6.21	99.4	250.9	7.96	5.86	6.00
27	99.0	250.0	7.98	5.68	6.40	99.3	251.0	7.90	5.67	6.00
28	99.4	251.3	8.08	5.93	6.44	99.4	251.4	8.12	5.90	5.88
29	100.2	251.3	8.09	6.08	6.00	100.4	250.2	8.14	6.08	5.94
30	100.3	249.9	8.08	6.01	5.81	99.9	250.4	8.10	5.98	5.63
31	100.4	250.2	8.10	5.87	5.75	100.6	250.3	8.05	5.84	5.88
32	100.9	250.6	8.12	6.01	6.00	100.9	250.2	8.13	5.98	6.31
33	100.1	251.0	8.13	6.05	6.31	100.1	250.9	8.13	6.03	6.31
34	100.7	250.5	8.11	6.05	6.88	100.8	250.3	8.15	6.02	6.50

b = flange width, h = beam height,
t = flange thickness, w = web thickness,
f = fillet length

Table 3-6 Maximum Load P_{max} and M_u/M_p – λ Values (For Actual σ_y and Area)

Beam Nnmber	D-Series			E-Series		
	Pmax (kN)	Mu/Mp)a	$\sqrt{Mp)a/M_E}$	Pmax (kN)	Mu/Mp)a	$\sqrt{Mp)a/M_E}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	176.0	0.802	0.961	98.6	0.664	1.257
2	164.8	0.758	0.967	91.9	0.610	1.259
3	168.0	0.779	0.953	83.8	0.560	1.246
4	170.9	0.800	0.954	94.8	0.642	1.240
5	186.9	0.873	0.954	108.1	0.720	1.246
6	157.0	0.817	0.942	90.9	0.684	1.240
7	124.6	0.671	0.936	78.6	0.617	1.234
8	140.9	0.752	0.939	101.6	0.787	1.239
9	151.3	0.809	0.937	84.9	0.656	1.228
10	129.3	0.704	0.934	91.2	0.718	1.228
11	131.0	0.703	0.940	84.0	0.657	1.238
12	150.0	0.813	0.928	90.0	0.702	1.218
13	123.1	0.658	0.933	69.3	0.542	1.231
14	141.3	0.767	0.952	82.1	0.643	1.254
15	153.6	0.832	0.937	100.1	0.785	1.231
16	143.1	0.723	0.939	90.5	0.662	1.231
17	161.9	0.797	0.945	97.0	0.688	1.240
18	165.1	0.812	0.940	85.1	0.607	1.234
19	152.8	0.745	0.948	103.1	0.735	1.248
20	166.9	0.813	0.937	82.7	0.583	1.231
21	154.1	0.848	0.925	85.1	0.673	1.212
22	129.3	0.714	0.926	73.3	0.587	1.223
23	139.1	0.772	0.929	73.2	0.583	1.218
24	128.2	0.728	0.938	89.2	0.735	1.230
25	139.2	0.825	0.907	81.6	0.702	1.192
26	160.3	0.935	0.904	67.5	0.563	1.181
27	150.5	0.861	0.918	93.0	0.765	1.203
28	143.3	0.799	0.908	87.6	0.702	1.191
29	150.7	0.851	0.890	100.5	0.818	1.170
30	152.1	0.871	0.891	99.5	0.825	1.163
31	147.4	0.844	0.889	78.9	0.652	1.164
32	147.3	0.789	0.906	95.1	0.763	1.185
33	136.0	0.725	0.913	83.9	0.646	1.205
34	156.9	0.834	0.913	81.0	0.620	1.200
M	149.8	0.790	0.930	88.2	0.673	1.221
S	15.25	0.0624		9.89	0.0740	
ω	0.102	0.079		0.112	0.110	

Table 3-7 Statistical Values of Test Results

		Dimension	M	S	ω
Pmax	D-Series	(kN)	149.8	15.25	0.102
	E-Series		88.2	9.89	0.112
Mp)a		(kNm)	85.4	6.30	0.0739
Material Properties	σ_y Flange	(MN/m²)	283.5	14.98	0.0528
	σ_y Web		337.7	25.48	0.0755
	σ_u Flange		437.1	9.52	0.0218
	σ_u Web		454.6	15.23	0.0335
	E Flange	(GN/m²)	208.7	1.81	0.0087
	E Web		209.7	2.40	0.0114
Sectional Area	A	(cm²)	30.69	0.780	0.0254
Nondimensionalized Average Residual Compressive Stress	$\overline{\sigma}_{rc}$		0.121	0.0561	0.464
Nondimensionalized Initial Crookedness	u_0/l	$\times 10^{-3}$	0.296	0.273	0.922
	v_0/l		0.125	0.101	0.808
	$\overline{\phi}_0$		0.504	0.390	0.774

Table 3-8 Sample Correlation Coefficients for Test Results

N=34	D-Series			E-Series		
	Mp	$\bar{\sigma}_{rc}$	u_0/ℓ	Mp	$\bar{\sigma}_{rc}$	u_0/ℓ
Mu	0.70	0.55	0.15	0.37	0.21	0.05
Mu/Mp	-0.06	0.17	0.02	-0.28	-0.18	0.06

N=25	A-Series			B-Series			C-Series		
	Mp	$\bar{\sigma}_{rc}$	u_0/ℓ	Mp	$\bar{\sigma}_{rc}$	u_0/ℓ	Mp	$\bar{\sigma}_{rc}$	u_0/ℓ
Mu	0.68	-0.31	0.14	0.80	-0.24	0.007	0.84	-0.24	0.26
Mu/Mp	0.31	-0.42	0.16	0.21	-0.40	-0.20	0.17	-0.45	0.39

Type	Beam Length(mm)	Span Length(mm)	$\bar{\lambda}$	Number
D	1860	1800	0.848	34
E	2660	2600	1.209	34

Fig. 3-1 Allotment of Test Specimens

Fig. 3-2 Residual Stress Measurement

Fig. 3-3 Stress-Strain Relationship by Tensile Coupon Test of 136 Specimens

(a) Yield Stress

(b) Young's Modulus

Fig. 3-4 Histograms of Material Properties

Fig. 3-5 Initial Crookedness of Test Beams

Fig. 3-6 Histograms of Initial Crookedness

Fig. 3-7 Residual Stress Change during Sectioning Process

Fig. 3-8 Residual Stress Distribution

Fig. 3-9 Variation of Residual Stress Patterns

Fig. 3-10 Histograms of Compressive Residual Stress $\overline{\sigma}_{rc}$

Fig. 3-11 Histogram of Cross-Sectional Area

Fig. 3-12 Histograms of Maximum Load

Fig. 3-13 Buckling Test Results

Fig. 3-14 M_u - M_p Scattergram

Fig. 3-15 M_u/M_p – $\overline{\sigma}_{rc}$ Scattergram

Fig. 3-16 M_u/M_p – u_o/ℓ Scattergram

Fig. 3-17 Summary of Test Results for Rolled A, B and C and Welded D and E Series

Photo. 3-1 Fabrication of Welded Beams

Photo. 3-2 Tensile Coupon Test

Photo. 3-3 Tensile Coupons after Test

Photo. 3-4(a) Residual Stress Measurement

Photo. 3-4(b) Residual Stress Specimens after Sectioning

Photo. 3-5 Test Beam during Test

Chapter 4. LATERAL-TORSIONAL BUCKLING TESTS ON WELDED CONTINUOUS BEAMS

4-1 INTRODUCTION

Basic beam strength of laterally unsupported simple beams has been proposed in some design codes for in-plane uniform bending, and the moment modification factors and effective length factors have been employed to allow for the in-plane bending moment distribution and for the effects of lateral continuity between segments and of buckling restraints, respectively. Salvadori (13) presented an approximate method of analyzing the lateral-torsional buckling of laterally continuous beams. He proposed that the effects of lateral continuity between adjacent segments should be ignored so that each segment could be regarded as being simply supported laterally. Thus, the elastic buckling of each segment is analyzed for its in-plane moment distribution by using the moment modification factors and an effective length equal to the segment length. This approximate method always yields lower bound solutions, unless all the segments are critical simultaneously.

Nethercot and Trahair (10,11) developed a modified form of Salvadori's method which leads to much closer approximate solutions. The accuracy of Salvadori's lower bound was improved by allowing for the approximate lateral bending and warping interactions between the critical segment and its adjacent segments. The effective length of the critical segment was determined in the same way as the critical length of an end-restrained compression member. Later, Poowannachaikul and Trahair (12) considered the influence of elastic end-restraints, produced by the continuity of the beam, on its buckling strength. They evaluated the restraints for a three-span continuous beam in which the interior span was loaded and the

adjacent side spans were unloaded. Trahair (14) conducted some tests on models of continuous aluminum I-beams and found good correspondence between the tests and his own theoretical predictions. Hartmann (9) calculated the influence of different kinds of restraints at the interior supports on the buckling strength. In an experimental study, he verified his theoretical predictions with tests on models of thin rectangular cross sections. Yoshida, et al. (15) presented the inelastic buckling analysis of two-span continuous beams by extending their theoretical investigation to more stocky beams in which segment in-plane moment redistribution takes place before failure. The theoretical results were consistently higher than the experimental test results. Bansal (1) tested 34 continuous steel beams in relatively short range to study the influence of various parameters on the lateral instability of three-span beams. The test results of beam strength and deformability are compared for loading conditions, varieties of cross sections, and yield strengths. Dux and Kitipornchai (3) conducted a series of experiments on the buckling of simply supported laterally continuous I-beams in the inelastic range. Points of load application were prevented from moving laterally and from twisting. The test results supported the theoretical curves of Nethercot and Trahair (10,11), and the current trend to multiple design curves based on moment gradient (4,6).

Recently, Galambos (8) and Bjorhovde (2) have indicated that relatively little data were available for use in developing the design criteria for laterally unsupported continuous beams, and they suggested that further research of ultimate behavior on laterally unsupported continuous beams is needed theoretically and experimentally.

The purpose of the present experimental study is to determine the effect of the adjacent nonloaded span, L_2, on the lateral buckling strength

and deformation of the critical loaded span, L_1. Twenty-one two-span continuous welded beams were tested under a concentrated load. Ratio of the spans, L_2/L_1, varies from 0.287 ~ 0.909.

In order to compare directly the buckling behavior of simple (5,7) and continuous welded beams, geometrically the same sections were used in the test series. The test results are compared with the calculated results based on the effective length concept and also the existing design method.

4-2 TEST PROGRAM

Specimen and Testing --- Twenty-one two-span continuous beams with nominally identical cross sections of welded I-250 x 100 x 6 x 8 mm (ordinary mild steel), as shown in Fig. 4-1, have been prepared and were tested in seven groups of three, each group having different span lengths. The welded built-up sections are made from flame-cut plates. Single-run weld by manual size 6 mm is used. The ratio of the spans, L_2/L_1, varies from 0.287 ~ 0.909 (Table 4-1). The line diagram in Fig. 4-2 identifies the components of the apparatus used to load and support the test beams. A concentrated load is always applied vertically at midspan, AB, through the Lehigh-type gravitational load simulator, which should eliminate any restraining effects of the applied load (Photo. 4-1). A plate welded at the end of the test beam is bolted directly to a fixed support, C, by 18 bolts. Continuous beams are restrained at the supports, A and B, against twisting about the longitudinal axis and moving laterally, but not against warping at support A. Longitudinal moving is free at both A and B. Support C provides the fixed supported condition vertically, laterally, and torsionally. The detailed description of the loading apparatus and of the supports, A and B, was given in Chapter 2.

The critical loaded span, L_1, is varied from 2.2~3.5 m, and its adjacent nonloaded span, L_2, takes on only two values, i.e., 2.0 m and 1.0 m. Measured points of deflections and strains are shown in Fig. 4-3. Strain gages are placed to separate the measured strains into in-plane and out-plane flange deformation and to obtain inflection points experimentally.

4-3 SUBSIDIARY TESTS

Tensile Coupon Tests --- Flanges and web of continuous test beams are each made from three original plates by flame cutting. Five tensile coupons are cut out from each original plate, making 30 in all. Photo. 4-2 shows a coupon attached to the grips.

A typical stress-strain relationship of a plate (t=8.0 mm) for a flange is shown in Fig. 4-4. Symbol o is a measurement point when the rate of strain increase is nearly zero, and the corresponding line is also given in Fig. 4-4 which is calculated by a least square method. Its initial tangent indicates modulus of elasticity, E, of the plate and the continuing plateau, a flow of yielding. The last tangent denotes modulus of initial strain-hardening, E_{st} .

Mechanical properties of six original plates are summarized in Table 4-2 for the mean values and the standard deviations obtained from five test coupons, respectively. The following eight mechanical properties are thus obtained: (1) Yield stress, σ_y; (2) maximum stress, σ_u; (3) modulus of elasticity, E; (4) modulus of initial strain-hardening, E_{st} ; (5) Poisson's ratio ν; (6) yield strain ε_y; (7) initial hardening strain, ε_{st} ; and (8) elongation, $\Delta\ell$. The mean value of σ_y is greater by 18.7 % in the flange plate, and 51.7 % in the web plate, than the minimum specified value, σ_y =

235 MN/m^2, of ordinary mild steel. There are comparatively large differences between the flange plate and web plate for σ_y, σ_u, E_{st}, and $\Delta\ell$.

The yield stresses of flange and web in each test beam are listed in Table 4-3.

Geometrical Imperfection --- Figure 4-5 describes initial measurements of the crookedness of 21 continuous test beams, taken at 18 points about the minor axis and the angle of rotation when beams and apparatus are satisfactorily arranged. Photo. 4-3 shows the equipment for the initial crookedness measurement. Initial crookednesses of seven groups of three, each group having different span lengths, are drawn as if each group had the same length. Measured initial crookednesses of 21 continuous beams are listed in Table 4-4.

Figure 4-6 shows nondimensionalized initial crookednesses at each midspan of 21 continuous beams. The corresponding Weibull frequency distribution curves are given in each histogram. Mean values of u_o/L_1 and u_o/L_2 in the minor axis are 0.57 x 10^{-3} and 0.40 x 10^{-3}, respectively, and the mean values of φ_o (H/L_1) and φ_o/(H/L_2) are 0.40 x 10^{-3} and 0.52 x 10^{-3}, respectively, in which H = beam height. The values of u_o/L and φ_o (H/L) on welded simple beams (5) were 0.296 x 10^{-3} and 0.504 x 10^{-3}, respectively. The mean nondimensionalized initial bows of each span on continuous beams are a little greater than those of welded simple beams, while the mean nondimensionalized initial twists are similar to those of welded simple beams.

Residual Stress Measurement --- Longitudinal residual stresses are measured by the sectioning method. The mesh sizes of sectioning are equal to those of the welded simple test beam described in Chapter 3. Three test beams are prepared and their flanges are made from different original

plates. The measured residual stresses of 21 members are listed in Table 4-5. Figure 4-7 shows measured residual stress distributions for three members, and the average measured residual stress distribution of the simple welded beams in Chapter 3 is also shown in Fig. 4-7 by symbol o. The residual stress distributions for the three members are remarkably similar to those of the welded simple beams in Chapter 3.

Cross-Sectional Area --- The measured cross-sectional dimensions of 21 continuous beams are listed in Table 4-6.

4-4 LATERAL-TORSIONAL BUCKLING TEST RESULTS

Load-Strain Curves --- Figure 4-8 shows load versus horizontal and vertical strain curves at the midloaded span for four arbitrary test beams. The thin dark lines in the figure correspond to the computed values based on the elasticity theory. The test values of strain due to vertical deflection have a good agreement with those of the elasticity theory and, thus, ensure satisfactory loading and support apparatus. Photo. 4-4 shows the twenty-one test specimens after failure.

Load-Deflection Curves and Effective Values of Deflection --- Figures 4-9(a) and 4-9(b) show four examples of load versus horizontal deflection and twisting, respectively, of the upper flange at the midloaded span for continuous test beams. The two examples of load versus horizontal deflection and twisting, respectively, of the upper flange at the midspan for beams chosen from 68 welded simple beams in Chapter 3 and also shown in Figs. 4-9(a) and 4-9(b) compared with the deflection behavior of continuous beams. Compression flanges were not buckled locally before due to lateral buckling.

The following two parameters will be used to investigate the effect of

lateral deflection on the continuous beams: (1) The lateral deflection, $\delta_{HP_{max}}$, at the maximum load in Fig. 4-9(a); and (2) the tangent value of the load-horizontal deflection curve when unloading occurs, θ. The latter denotes the decrease in the load corresponding to an increase of 1 mm in the lateral deflection. The relationships between P_{max}, $\delta_{HP_{max}}$, and θ are shown in Figs. 4-10(a) and 4-10(b), respectively, for the 21 continuous beams.

Figures 4-11(a) and 4-11(b) show similar relationships for 68 welded simple beams. On longer beams, the sample correlation between P_{max} and $\delta_{HP_{max}}$ is little. However, on stocky beams, the sample correlation between P_{max} and $\delta_{HP_{max}}$ has small negative values. The sample correlations between P_{max} and θ have apparently positive values on both continuous beams and welded simple beams, as in Fig. 4-10(b) and Fig. 4-11(b).

Strain Distributions --- Figure 4-12 shows an example of measured strain distributions on an upper and a lower flange for one of the continuous test beams (B-a-2) with a loaded span of 3.0 m and a nonloaded span of 2.0 m. This figure describes the following four strain distributions on each stage of loading: (1) vertical strain distribution of the upper flange; (2) vertical strain distribution of the lower flange; (3) lateral strain distribution of the upper flange; and (4) lateral strain distribution of the lower flange. These distributions were interpolated between the measured strain values by straight line approximations. The in-plane strain distribution of the upper flange, although it differs in sign, has the same shape and magnitude as that of the lower flange.

The zero strain point on the loaded span, which was the inflection point of the vertical curvature, did not change with the increase of load. The zero point of lateral strain distribution of the upper flange was

nearer to the concentrated load point than that of the in-plane strain distribution. The lateral strain distribution of the lower flange did not increase grossly according to the increase of load and it is apparent that the lateral deflection of the lower flange was small.

Ultimate Strength --- The ultimate strength of the continuous test beams is described in Fig. 4-13(a), with a plot of P_u/P_y versus $\lambda = \sqrt{P_y/P_E}$, in which P_y = the yield strength when M_A reaches yield moment, M_y (Fig. 4-1), and P_E = the elastic critical load of the continuous test beam calculated by the FEM. The minimum value and the maximum value of P_p/P_y in the test beams, in which P_p = the simple plastic load which causes failure mechanism, is also shown in Fig. 4-13(a). The dropping curve is for $P_u/P_y = 1/\lambda^2$, which represents an elastic buckling curve.

Figure 4-13(b) also describes the ultimate strength of the continuous test beams, using the P_p instead of the P_y, in the ordinate and the abscissa of Fig. 4-13(a). The dashed curves correspond to the following formula for n = 1.0, 1.5, and 2.0:

$$P_u/P_p = [1 \ / \ (1 + \sqrt{P_p/P_E}^{2n})]^{1/n} \qquad \text{---(4-1)}$$

Note that Eq. (4-1) is a modified version of the ECCS formula expressed in bending moments for determinate beams.

Table 4-7 lists the maximum strength, P_{max}, of 21 continuous test beams and the reference values of Fig. 4-13(a) and Fig. 4-13(b). In Fig. 4-13(a), low differences in λ values between the beams with the nonloaded spans of 2.0 m and 1.0 m can be noticed. However, in Fig. 4-13(b), the $\lambda = \sqrt{P_p/P_E}$ value of the beam with the nonloaded span 1.0 m was lower than that of the beam with the nonloaded span of 2.0 m, as the P_p value was

independent of the length of the nonloaded span. In Fig. 4-13(a), the average ratios of $(P_u/P_y)_b$ and $(P_u/P_y)_a$ corresponding to each of the series, A, B, and C, were 0.992, 0.934, and 0.881, respectively. Thus, it is observed that a beam with a shorter length of loaded span is strongly affected with the length of the nonloaded span.

4-5 COMPARISON WITH AVAILABLE TEST DATA AND CALCULATED RESULTS FOR DIFFERENT EFFECTIVE LENGTHS

Comparison with ultimate strength of simple beams --- Figure 4-14 compares the current results for 21 continuous beams with 75 rolled beams in Chapter 2 and 68 welded simple beams in Chapter 3, in which $P_u/(P_p)_{simple}$ is in the ordinate; $\lambda = \sqrt{(P_p)_{simple}/(P_E)_{simple}}$ is in the abscissa (in which $(P_p)_{simple}$ = the simple plastic failure load, assuming that the loaded span was a simple beam); and also $(P_E)_{simple}$ = the elastic critical load of that simple beam. Symbols △ and o are used for rolled simple beams and welded simple beams, respectively, and the other symbols are used for the 21 continuous beams. The buckling strength curves of simple beam and propped cantilever beam were also shown in Fig. 4-14.

As shown in Fig. 4-14, the ultimate strengths of continuous test beams were higher than those of simple beams and were scattered between the elastic buckling strength curves of simple beam and propped cantilever beam which was considered to have a nonloaded span with infinite rigidity. It is obvious that the nonloaded span restrained the loaded span.

Comparison with Ultimate Strength of Other Continuous Beams and Simple Beams --- The present test results for 21 continuous beams are compared with Trahair's eight continuous beams in Fig. 4-15. The results of previous 75 rolled beams and 68 welded beams are also shown in Fig. 4-15. The

ordinate and the abscissa are described by P_u/P_y and $\lambda = \sqrt{P_y/P_E}$, respectively, in which P_y = the first yield load, and P_E = the elastic critical load of the beam considered. The values M and M-2S in each group are shown by the arrow in Fig. 4-15. The corresponding coefficients of variation, ω, are also given in Fig. 4-15.

On two of Trahair's test beams a concentrated load was applied vertically at the midspan of one side span, and on the other beams two concentrated loads were applied vertically at the midspans of each side span. The present results of series B and C have a good agreement with Trahair's results. However, the results of series D, with a loaded span of 2.2 m and a nonloaded span of 2.0 m, show comparatively higher strengths than those of Trahair's results. It may be explained by the difference in loading and boundary conditions. In Table 4-8, the present results of series B and C are compared with simple beams which are scattered among the similar values. The average ultimate strength nondimensionalized by P_y was more than 10 % higher than that of simple beams. The test results of present continuous beams and of Trahair's continuous beams are scattered among the lower part of the test results of simple beams.

In Fig. 4-16, the ultimate strength for the 21 continuous test beams is compared with that of 75 rolled simple beams and 68 welded simple beams, in which P_u/P_p = the ordinate; and $\lambda = \sqrt{P_p/P_E}$ = the abscissa, in which P_p = the simple plastic failure load, and P_E =the elastic critical load. The dashed curves correspond to Eq. (4-1) for n=1.0, 1.5, and 2.0. Each of the mean results of the ultimate strength of the present seven series is a little above the curve, n=2.0, and each lowest value and M-2S value of the present seven series are also a little above the curve of n=1.5. The coefficients of variation of ultimate strength on continuous test beams are

close to the values of simple beams. It may be concluded from Fig. 4-16 that the plotted ultimate strength points in P_u/P_p versus $\lambda = \sqrt{P_p/P_E}$ ordinates are almost similar in variation for both simple and continuous beams and that Eq. (4-1) is also applicable to the indeterminate beams.

Application of Effective Length Concept -- The concept of effective length is applied to predict the elastic critical load . The effective length concept has been applied to allow for the effects of lateral continuity between beam segments and of buckling restraints. In Fig. 4-17, the following four effective lengths were used to evaluate the ultimate strength of the 21 continuous test beams: (1) The length of loaded span, $L_e = L_1$; (2) the length between the left-end support and the in-plane inflection point of vertical deflection, $L_e = L_{ev}$, in which L_{ev} was determined by the measured vertical strain distribution; (3) the length between the left-end support and the lateral inflection point of upper flange, $L_e = L_{eH}$, in which L_{eH} was determined by the measured lateral strain distribution of upper flange; and (4) the length between the left-end support and the inflection point of the continuous column with the constant force on loaded span, when the continuous beam was treated as the continuous column, $L_e = L_{eC}$.

The actual in-plane moments between the left-end support and each inflection point, which were obtained by the measured vertical strain distribution, are used in the calculation of P_E. The effective lengths of (1), (2), (3), and (4) are shown in Table 4-9. The values in the parentheses denote the factors of the effective length nondimensionalized by each loaded span length, L_1.

The values obtained by the elastic analysis are also given in Table 4-9. The theoretical inflection points of lateral deflection and twisting

were obtained as different values, and the measured inflection point was between the preceding two theoretical inflection points. The effective length of a beam with a nonloaded span of 1.0 m was shorter by 1.5 %, 3.3 %, and 3.3 %, respectively, in series A, B, and C than that of a beam with a nonloaded span 2.0 m. It appears that the degree of the restraint by the nonloaded span is longer on beams with shorter nonloaded span.

The effective length of (4) in Table 4-9 was obtained as that of a continuous column subjected to the constant axial force on a loaded span. When the continuous column was under the constant axial force applied at both loaded span and nonloaded span, its effective length is longer by 5 % than that of the previous value.

As shown in Fig. 4-17, the effective length, using the measured lateral inflection, was on the nonconservative side, while Salvadori's effective length (1) was considerably on the conservative side. The test results, using the effective length of (4) scattered above the ECCS formula of n=1.5, and these values seem to have a good agreement with simple beams.

4-6 CONCLUSIONS

Twenty-one two-span continuous beams with nominally identical cross sections of welded I-250 x 100 x 6 x 8 mm were tested in seven groups of three, each group having different span lengths. These continuous beams were tested under a concentrated load on the midspan of the one-side span. Geometrical and material imperfections were also measured. The test results were compared directly with the buckling behavior of simple beams and also with the calculated results based on the effective length concept.

As the loading span was restrained by the nonloaded span, ultimate strengths of continuous beams were higher than those of simple beams and

were scattered between the elastic buckling strength curves of simple beams and of propped cantilever beams which were considered to have a nonloaded span with infinite rigidity.

In the plot of (P_u/P_y) versus $(\lambda = \sqrt{P_y/P_E})$, the test results of continuous beams were scattered on the lower part of the results of simple beams. However, the coefficients of variation were close to the values of simple beams.

The parameter (P_p/P_E) versus $(\lambda = \sqrt{P_p/P_E})$ is found to be effective for evaluating the ultimate strength of both simple and continuous beams. Eq. (4-1) may be applicable to the indeterminate beams and the determinate ones as far as the reference loads, P_p and P_E, should be evaluated for the actual supported and load conditions.

Based on the present experimental investigations, the effective length concept using the inflection points is recommended for evaluating the ultimate strength of continuous beams.

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NOTATION

The following symbols are used in this chapter:

E = modulus of elasticity;
E_{st} = strain hardening modulus;
f = frequency;
H = beam height;
L = span length of member;
L_e = effective length;
L_{eC} = effective length as continuous column;
L_{eH} = effective length using lateral inflection;
L_{eV} = effective length using in-plane inflection;
L_1 = length of loaded span;
L_2 = length of nonloaded span;
M = mean value;
M_E = elastic critical moment;
M_p = plastic moment;
M_u = ultimate bending moment;
N = number of data;
P = vertical load;
P_E = elastic critical load;
P_{max} = maximum load;
P_p = simple plastic failure load;
P_u = ultimate load;
P_y = yield load;
S = standard deviation;
u = lateral displacement;
u_o = initial deflection in lateral direction;
$\Delta\ell$ = elongation;
$\delta_{HP_{max}}$ = lateral deflection at maximum load;
ε_V = vertical strain;
ε_y = yield strain;
ε_{st} = strain onset of strain-hardening;
θ = angle of unloading curve;
λ = slenderness function;
σ_y = yield stress;
σ_{yf} = yield stress in flange;
σ_{yw} = yield stress in web;
ν = Poisson's ratio;
φ = torsional displacement;
φ_o = initial angle of rotation; and
ω = coefficient of variation.

Table 4-1 Span Length of Test Specimens

Series	Span Length		Span Ratio	Number of Beams
	L_1 (mm)	L_2 (mm)	L_2/L_1	
A-a	3500	2000	0.571	3
A-b	3500	1000	0.286	3
B-a	3000	2000	0.667	3
B-b	3000	1000	0.333	3
C-a	2500	2000	0.800	3
C-b	2500	1000	0.400	3
D-a	2200	2000	0.909	3

Table 4-2 Tensile Test Results

			F_y MPa	F_u MPa	E $\times 10^5$ MPa	E_{st} $\times 10^5$ MPa	ν	ε $\times 10^{-6}$	ε_{st} $\times 10^{-6}$	$\Delta\ell$ %
Flange Plates N=15	1	M	273.6	442.5	2.104	0.0547	0.286	1300	17780	29.9
		S	3.5	2.1	0.0083	0.0035	0.0026	19	988	1.02
	2	M	278.9	414.4	2.105	0.0529	0.287	1330	19300	30.1
		S	6.9	3.1	0.0090	0.0079	0.0028	29	1575	2.05
	3	M	284.3	444.9	2.102	0.0515	0.285	1360	19750	30.2
		S	3.0	2.1	0.0022	0.0110	0.0021	13	2020	1.68
Web Plates N=15	1	M	370.0	466.0	2.089	0.0486	0.277	1780	22200	22.4
		S	5.1	2.5	0.0058	0.0096	0.0019	29	1463	0.54
	2	M	360.0	462.8	2.085	0.0437	0.279	1730	19590	22.9
		S	1.8	2.5	0.0088	0.0067	0.0013	11	873	0.60
	3	M	339.2	453.2	2.096	0.0440	0.277	1620	23130	27.3
		S	4.2	2.1	0.0167	0.0036	0.0018	16	792	1.21

N = number of specimens, M = mean value, S = standard deviation
F_y = yield stress, F_u = maximum stress, E = modulus of elasticity
E_{st} = modulus of initial strain-hardening, ν = Poisson's ratio
ε_y = yield strain, ε_{st} = initial hardening strain, $\Delta\ell$ = elongation

Table 4-3 Yield stresses of Beams

Beam No.	σ_{yf} (N/mm^2)	σ_{yw} (N/mm^2)
A-a-1	273.6	370.0
A-a-2	273.6	370.0
A-a-3	273.6	370.0
A-b-1	278.9	360.0
A-b-2	278.9	370.0
A-b-3	273.6	370.0
B-a-1	278.9	360.0
B-a-2	273.6	370.0
B-a-3	273.6	370.0
B-b-1	284.3	339.2
B-b-2	284.3	360.0
B-b-3	284.3	360.0
C-a-1	273.6	370.0
C-a-2	278.9	360.0
C-a-3	278.9	360.0
C-b-1	284.3	339.2
C-b-2	284.3	360.0
C-b-3	284.3	360.0
D-a-1	284.3	339.2
D-a-2	278.9	360.0
D-a-3	273.6	339.2

Table 4-4 Initial Crookedness of 21 Members ($\times 10^{-2}$)

Beam No.	Initial Lateral Deflection of Loaded Span								
	$0.1L_1$	$0.2L_1$	$0.3L_1$	$0.4L_1$	$0.5L_1$	$0.6L_1$	$0.7L_1$	$0.8L_1$	$0.9L_1$
A-a-1	43	152	187	193	281	304	245	169	43
A-a-2	54	100	56	22	58	84	45	48	15
A-a-3	-48	-62	-61	-63	-6	25	68	70	3
A-b-1	-42	-119	-248	-315	-341	-314	-303	-271	-96
A-b-2	-120	-131	-229	-324	-322	-248	-263	-220	-142
A-b-3	-53	-81	-83	-83	-104	-121	-197	-195	-88
B-a-1	104	80	79	167	250	251	184	156	62
B-a-2	-21	-45	-121	-93	-59	-5	-19	0	24
B-a-3	-133	-147	-44	-112	-61	-54	-101	-78	-24
B-b-1	-65	-70	-74	-48	-88	-72	-75	-31	-33
B-b-2	-68	-112	-147	-135	-97	-123	-93	-115	-25
B-b-3	-129	-218	-212	-174	-175	-180	-141	-124	-31
C-a-1	-70	-100	-79	-97	-115	-92	-47	-38	15
C-a-2	24	-40	-21	-87	-135	-79	-107	-98	-33
C-a-3	14	-33	-40	21	34	40	85	14	-23
C-b-1	-115	-250	-345	-282	-200	-177	-168	-141	-30
C-b-2	-95	-205	-321	-300	-318	-279	-321	-193	-106
C-b-3	-198	-307	-464	-487	-552	-551	-444	-322	-169
D-a-1	10	65	116	58	75	24	-21	65	55
D-a-2	-18	-44	-136	-142	-137	-116	-96	-135	-92
D-a-3	-51	-136	-150	-84	-126	-124	-92	-100	-56

Beam No.	Initial Lateral Deflection of Nonloaded Span								
	$0.1L_2$	$0.2L_2$	$0.3L_2$	$0.4L_2$	$0.5L_2$	$0.6L_2$	$0.7L_2$	$0.8L_2$	$0.9L_2$
A-a-1	8	42	58	55	5	-25	-42	-29	-3
A-a-2	-31	-24	-58	-100	-59	-97	-92	-71	-10
A-a-3	-42	-55	-50	-75	-107	3	10	37	36
A-b-1	19	16	15	34	48	34	28	27	10
A-b-2	27	13	-9	-15	-10	-1	8	13	10
A-b-3	20	40	47	45	48	61	64	43	19
B-a-1	5	2	1	25	51	35	0	14	22
B-a-2	-48	12	-19	-13	54	72	96	118	75
B-a-3	-40	-77	-61	-37	14	2	25	-31	-27
B-b-1	0	18	-14	-82	-99	-70	-36	-10	0
B-b-2	12	7	-33	-60	-66	-68	-67	-52	-38
B-b-3	-45	-54	-57	-66	-63	-52	-42	-12	16
C-a-1	42	19	-9	35	25	13	20	23	-37
C-a-2	42	63	27	2	37	-3	-33	-8	-4
C-a-3	-18	-21	-6	18	13	-15	-78	-36	-70
C-b-1	-22	-67	-82	-68	-83	-120	-118	-85	-49
C-b-2	-1	5	28	52	55	46	54	50	24
C-b-3	10	22	1	-24	-19	10	15	-4	-11
D-a-1	-63	-144	-209	-184	-140	-107	-144	-143	-119
D-a-2	-71	-139	-134	-177	-167	-168	-162	-134	-111
D-a-3	-39	-28	-32	-44	-19	-51	-43	-83	-28

Table 4-4 Continued

Beam No.	Initial Bow of Loaded Span								
	$0.1L_1$	$0.2L_1$	$0.3L_1$	$0.4L_1$	$0.5L_1$	$0.6L_1$	$0.7L_1$	$0.8L_1$	$0.9L_1$
A-a-1	-11	14	53	58	57	61	61	11	-16
A-a-2	13	-1	-49	-74	-87	-80	-58	-21	-22
A-a-3	21	36	67	25	31	42	8	31	23
A-b-1	-30	-57	-3	-29	-49	-65	-60	-61	-52
A-b-2	15	16	32	61	67	55	22	-24	-19
A-b-3	-9	-42	-50	-76	-95	-64	0	26	17
B-a-1	45	56	74	27	-12	37	62	29	18
B-a-2	-1	2	17	30	38	20	20	29	17
B-a-3	67	74	3	-5	5	22	27	8	-15
B-b-1	-10	-4	24	31	1	19	43	65	66
B-b-2	20	30	49	37	22	22	19	11	-7
B-b-3	38	71	94	129	125	119	96	56	46
C-a-1	-59	-95	-63	-98	-78	-79	-68	-43	-27
C-a-2	-12	-31	-24	9	41	36	21	44	2
C-a-3	43	25	-4	-23	-51	-63	-50	-29	-29
C-b-1	-64	-56	-45	-44	-18	-5	0	0	25
C-b-2	-13	-23	6	33	57	2	2	7	29
C-b-3	-3	-17	-24	-45	-43	-11	14	-3	2
D-a-1	-25	-29	-32	-20	-6	8	-24	-34	-35
D-a-2	-44	-43	-28	-19	0	-2	-20	-10	0
D-a-3	-21	-44	-57	-67	-74	-101	-104	-76	-63

Beam No.	Initial Bow of Nonloaded Span								
	$0.1L_2$	$0.2L_2$	$0.3L_2$	$0.4L_2$	$0.5L_2$	$0.6L_2$	$0.7L_2$	$0.8L_2$	$0.9L_2$
A-a-1	-2	5	22	32	35	47	49	30	36
A-a-2	14	20	49	83	105	87	90	39	32
A-a-3	33	58	96	99	92	70	41	13	35
A-b-1	0	3	-3	-15	-14	-27	-30	-10	0
A-b-2	-4	-14	-24	-16	-8	-12	-25	4	20
A-b-3	-24	-17	1	1	-1	2	18	23	24
B-a-1	-6	19	34	10	11	14	-30	-4	6
B-a-2	15	15	5	18	-2	-17	-49	-41	-20
B-a-3	20	35	52	48	51	65	96	87	53
B-b-1	-5	-6	6	22	8	-3	5	13	1
B-b-2	11	20	31	35	35	33	37	40	21
B-b-3	10	10	6	12	11	-5	-7	11	4
C-a-1	0	31	28	69	41	62	65	30	16
C-a-2	-1	26	-10	-21	-16	3	23	-2	12
C-a-3	-4	-9	-41	-22	-29	-3	-12	0	0
C-b-1	-6	-18	-41	-52	-31	-23	-19	6	20
C-b-2	2	15	40	41	37	28	15	21	15
C-b-3	-10	-22	-16	-4	-12	-13	0	0	-5
D-a-1	13	7	-10	-2	-30	-41	-51	-35	-4
D-a-2	-35	-37	-53	-91	-103	-87	-62	-53	-28
D-a-3	-4	-29	-29	-61	-47	-35	-58	-39	-8

Table 4-5 Residual Stress, in N/mm^2

Point No.	Beam No. 1 Upper Flange	Beam No. 1 Lower Flange	Beam No. 1 Web	Beam No. 2 Upper Flange	Beam No. 2 Lower Flange	Beam No. 2 Web	Beam No. 3 Upper Flange	Beam No. 3 Lower Flange	Beam No. 3 Web
1	43.1	53.9	384.4	53.9	-57.9	408.0	-2.0	74.5	349.1
2	26.5	19.6	137.3	45.1	-65.7	74.5	-13.7	58.8	144.2
3	0.0	-23.5	-122.6	36.3	-68.6	-193.2	-32.4	35.3	-136.3
4	-24.5	-119.6	-169.7	-38.2	-52.0	-219.7	-47.1	6.9	-149.1
5	-89.2	-127.5	-166.7	-102.0	-101.0	-218.7	-109.8	70.6	-147.1
6	189.3	154.9	-185.3	60.8	196.1	-196.1	171.6	103.0	-143.2
7	302.0	314.8	-169.7	311.9	337.3	-186.3	309.9	184.4	-134.4
8	263.8	271.6	-163.8	282.4	321.7	-177.5	271.6	150.0	-133.4
9	286.4	313.8	-189.3	328.5	340.3	-169.7	287.3	177.5	-125.5
10	89.2	48.1	-131.4	182.4	117.7	-154.9	66.7	108.9	-123.6
11	-152.0	-95.1	-127.5	-5.9	-65.7	-146.1	-132.4	-10.8	-133.4
12	-7.8	-1.0	-140.2	-12.7	-24.5	-141.2	-5.9	52.0	-128.5
13	-7.8	-26.5	-108.9	-36.3	-24.5	-121.6	-37.3	21.6	-86.3
14	8.8	11.8	91.2	-29.4	16.7	67.7	-15.7	23.5	156.9
15	41.2	26.5	417.8	-22.6	18.6	382.5	-5.9	22.6	269.7

Point No.	Average of Ref. 5 Upper Flange	Average of Ref. 5 Lower Flange	Average of Ref. 5 Web
1	-41.1	-49.6	251.2
2	-33.2	-41.9	139.6
3	-16.2	-25.5	-58.9
4	-37.0	-36.8	-166.8
5	-11.5	-12.3	-167.0
6	164.9	190.7	-160.4
7	309.1	311.6	-159.2
8	336.1	332.2	-161.6
9	300.6	300.4	-155.3
10	184.2	163.8	-154.7
11	-6.7	-22.8	-150.1
12	-49.2	-29.1	-147.0
13	-19.9	-16.6	-49.6
14	-39.3	-35.8	140.5
15	-49.2	-51.4	246.0

Table 4-6 Measured Cross-Sectional Dimensions, in mm

Beam No.	b	h	t	w	f
A-a-1	100.5	250.8	8.01	6.07	5.60
A-a-2	100.9	250.8	8.06	6.11	5.90
A-a-3	100.7	250.7	8.02	6.06	5.70
A-b-1	100.7	252.1	8.18	6.06	6.30
A-b-2	100.5	251.2	8.05	6.06	6.40
A-b-3	100.4	251.3	8.28	6.24	6.70
B-a-1	100.4	251.3	8.26	6.20	6.80
B-a-2	100.5	250.7	8.04	6.08	5.40
B-a-3	100.5	250.3	8.03	6.07	5.60
B-b-1	100.7	250.3	8.25	6.26	6.30
B-b-2	100.7	250.1	8.21	6.06	5.80
B-b-3	100.8	250.6	8.10	6.03	5.90
C-a-1	100.7	251.3	8.05	6.14	5.50
C-a-2	100.9	251.3	8.21	6.05	6.00
C-a-3	100.7	250.3	8.12	6.06	5.80
C-b-1	100.6	251.5	8.33	6.27	5.90
C-b-2	100.7	249.9	8.13	6.08	6.30
C-b-3	100.8	250.3	8.05	6.06	6.10
D-a-1	100.7	251.8	8.22	6.09	5.60
D-a-2	101.2	251.7	8.23	6.08	5.90
D-a-3	100.9	251.6	8.30	6.25	6.60

Table 4-7 Test Results and Reference Loads

Beam number	P_{max}, in kilonewtons	P_u/P_y	$\sqrt{P_y/P_E}$	P_u/P_p	$\sqrt{P_p/P_E}$
A-a-1	65.7	0.624	1.230	0.440	1.465
2	68.8	0.647	1.225	0.456	1.459
3	58.4	0.554	1.228	0.391	1.463
A-b-1	67.3	0.571	1.229	0.438	1.403
2	73.0	0.629	1.236	0.476	1.421
3	71.3	0.610	1.213	0.457	1.401
B-a-1	109.1	0.853	1.118	0.601	1.333
2	82.6	0.684	1.109	0.474	1.332
3	85.5	0.709	1.110	0.491	1.333
B-b-1	94.4	0.681	1.113	0.529	1.263
2	100.6	0.737	1.113	0.562	1.274
3	92.4	0.680	1.119	0.519	1.281
C-a-1	142.6	0.996	0.976	0.676	1.185
2	126.7	0.854	0.980	0.592	1.178
3	134.8	0.924	0.983	0.639	1.182
C-b-1	153.4	0.929	0.976	0.710	1.117
2	117.3	0.729	0.983	0.547	1.136
3	125.7	0.787	0.984	0.589	1.137
D-a-1	161.8	0.956	0.883	0.671	1.063
2	159.5	0.954	0.889	0.652	1.076
3	186.7	1.121	0.901	0.774	1.076

Table 4-8 Comparison with Simple Beam Test Results

	λ	$P_u/P_y)_M$	$P_u/P_y)_{M-2S}$	ω
① Continuous Beams N = 6	1.114	0.724	0.590	0.0926
② Welded Simple Beams N = 34	1.108	0.817	0.634	0.1114
①/②		0.886	0.931	0.831
③ Continuous Beams N = 6	0.981	0.870	0.671	0.1142
④ Rolled Simple Beams N = 25	0.968	0.976	0.864	0.0572
③/④		0.891	0.777	1.997

Table 4-9 Effective Length

(in mm)

Beam	(1) $L_e = L_1$	(2) $L_e = L_{eV}$		(3) $L_e = L_{eH}$			(4) $L_e = L_{eC}$
		Test	Theory	Test	Theory		
					u	ϕ	
A-a	3500 (1.00)	2811 (0.803)	2772 (0.792)	2640 (0.754)	2457 (0.702)	2758 (0.788)	2734 (0.781)
A-b	3500 (1.00)	2729 (0.780)	2674 (0.764)	2601 (0.740)	2387 (0.682)	2692 (0.769)	2608 (0.745)
B-a	3000 (1.00)	2457 (0.819)	2400 (0.800)	2304 (0.768)	2121 (0.707)	2382 (0.794)	2374 (0.791)
B-b	3000 (1.00)	2386 (0.795)	2307 (0.769)	2230 (0.743)	2058 (0.686)	2316 (0.772)	2254 (0.751)
C-a	2500 (1.00)	2052 (0.821)	2025 (0.810)	1887 (0.754)	1770 (0.708)	2118 (0.847)	2010 (0.804)
C-b	2500 (1.00)	2000 (0.800)	1940 (0.776)	1827 (0.731)	1725 (0.690)	1943 (0.777)	1902 (0.761)
D-a	2200 (1.00)	1815 (0.825)	1800 (0.818)	1624 (0.738)	1577 (0.717)	1778 (0.808)	1788 (0.813)

L_e = effective length
L_{eC} = effective length as continuous column
L_{eH} = effective length using lateral inflection
L_{eV} = effective length using in-plane inflection

Fig. 4-1 Test Specimen of Two-Span Continuous Test Beams

Fig. 4-2 General Arrangement of Apparatus

Fig. 4-3 Measured Points of Strains and Deflections

Fig. 4-4 Stress Strain Relationship of Coupon

Fig. 4-5 Initial Crookedness of Continuous Test Beams

Fig. 4-6 Histograms of Initial Crookedness

Fig. 4-7 Residual Stress Distribution

Fig. 4-8 Load-Strain Curves of Continuous Test Beams

Fig. 4-9(a) Load-Lateral Deflection Curves of Continuous Test Beams and Simple Beams

Fig. 4-9(b) Load-Twisting Curves of Continuous Test Beams and Simple Beams

Fig. 4-10 P_{max} -$\delta_{Hp_{max}}$ and P_{max} -θ Scattergrams of Continuous Test Beams

Fig. 4-11 P_{max} -$\delta_{Hp_{max}}$ and P_{max} -θ Scattergrams of Simple Beams

Fig. 4-12 Strain Distributions of Continuous Test Beams

Fig. 4-13(a) Buckling Test Results in P_u/P_y versus $\lambda=\sqrt{P_y/P_E}$

Fig. 4-13(b) Buckling Test Results in P_u/P_p versus $\lambda=\sqrt{P_p/P_E}$

Fig. 4-14 Comparison with Ultimate Strength of Simple beams

Fig. 4-15 Comparison with Available Test Data

Fig. 4-16 Comparison with Ultimate Strength of Simple Beams

Fig. 4-17 Test Results by Using Effective Lengths

Photo. 4-1 Testing

Photo. 4-2 Tensile Coupon Test

Photo. 4-3 Equipment for Initial Crookedness Measurement

Photo. 4-4 Twenty-one Test Specimens after Failure

Part II Ultimate Strength of Structural Steel Members from Data-Base Approach

Chapter 5. ESTABLISHMENT OF NUMERICAL DATA BASE OF STRUCTURAL STEEL MEMBERS

5-1 INTRODUCTION

(1) General

A data base may be defined as a collection of interrelated data stored together without harmful or unnecessary redundancy to serve multiple applications. The computer industry is improving its capability, and the cost of data-storage hardware is dropping more rapidly than other costs in data processing. It becomes cheaper to store data on computer files than to store them on paper. The technology in computer system software for the data base management is also being developed (1).

Under the above circumstance, many data bases have been built on a number of fields since 1950's. Most of them are for bibliography, in which the Chemical Abstract (CA) is the most famous data base on the scientific field (2). Data base management systems (DBMS) for bibliographic data bases have been developed by computer corporations, which have many facilities and multi-purpose abilities.

Some numerical data bases such as structures of proteinic molecule and cosmic rays have been built (3,4). On the civil engineering field, the data base of geological columnar sections is reported in Ref. (5). The data base management system for each numerical data base should be generally developed by its data base administrator.

(2) Requirement of Numerical Data Base of Steel Structures

Test data relating to the behavior of steel structures and structural components throughout the entire range of loading up to ultimate load are an essential requirement both for the preparation of design codes and for

the verification of new theoretical approaches for predicting the resistance of structures.

Many countries are now in the process of introducing new editions of their design specifications based on modern limit states principles (6,7). This fact has increased the need for a large volume of good quality test data since such information is essential for the satisfactory calibration of these methods so as to insure the appropriate level of the structural reliability.

The requirement for good quality test data is also expected to increase with the continuing advancement of powerful, computerized analytical methods since these require very careful checking against well documented test results if they are subsequently to be used with confidence.

During the past thirty years a large volume of structural tests has been conducted in laboratories both in Japan and overseas. However, reporting of this work is fragmented and often published papers do not provide sufficient data for the purpose outlined above. It is felt that the solution to this would be the establishment on a world wide basis of a computerized data base containing detailed results of these tests.

The specific benefits that would derive from this include:

i) Prevention of experimental data from being scattered and lost.

ii) Effective use of data through the identical evaluation

iii) Rapid retrieval, easy processing and graphic presentation of needed data.

iv) Availability to specification writing bodies of all relevant test data.

v) Avoidance of expensive duplication of testing effort.

vi) Availability to researchers engaged on theoretical work of well documented test histories.

vii) Encouragement to researchers engaged on experimental work to present their results in a consistent manner.

This chapter aims at providing a basic framework for the data base NDSS (Numerical Data-Base for Steel Structures) regarding the ultimate strength of steel structures and structural components. The computer management strategy is prepared, together with some statistical and graphical packages for the processing of data before output. Results of some of the more readily available test series on mechanical properties of steels, columns, beams, and unstiffened plates are deposited in the system (8-15).

5-2 INSTRUMENTATION FOR THE NDSS SYSTEM

(1) Instrumentation

Instrumentation for the NDSS system is divided into three blocks as follows:

i) Data sources from laboratories and available documents.

ii) Instrumentation for data conversion and for input and output of data installed in the Civil Engineering Department.

iii) A large computer system at the University Computation Center for source data filing, searching, retrieval, and processing of data.

These three blocks are shown in Fig. 5-1. Data editing, searching, retrieval and management can be done on the university computer system. A part of processing of original data can be performed by using

mini-computers and personal computers.

(2) **Data Conversion**

Data sources may be obtained in the digital form on paper tapes, floppy diskettes, and magnetic tapes, and in the analog form by a data recorder and XY-recorder. Analog form data are converted into machine readable forms by the AD-converter and XY-coordinates reading machine (degitiser) connected with a computer.

Recent developments on measuring devices used in structural laboratories have enabled a large volume of test data to be obtained in the digital form stored on floppy diskettes, magnetic tapes and paper tapes. Machine readable data can be fed directly into computers.

Since measuring devices recently tend to have digital interfaces such as the GP-IB and RS-232C, floppy diskettes with a personal computer are often used to install experimental data. The trouble is no compatibility of floppy diskettes between personal computers and the university computer. The floppy diskette of the university computer depends on the IBM standard format and its character code is the EBCDIC code, while floppy diskettes of personal computers depend on original formats and their code is usually the ASCII code. Tables 5-1(a),(b) and 5-2 show track and directory allocations of floppy diskettes of the university computer and the personal computers, respectively.

In order to solve this problem the author developed some conversion programs by using the FORTRAN77 and machine language, which enable that floppy diskettes of personal computers can be read and written on the university computer (16). The floppy diskettes which are managed by four operating systems for personal computers are available to convert; (i) MS-DOS (Microsoft Corporation) (17), (ii) CP/M-86 (Digital Research

Corporation) (18), (iii) N88-basic (Nippon Electric Corporation) (19) and (iv) F-basic (Fujitsu Corporation) (20).

Figure 5-2 shows an example of a data file transfer from a data-set of the university computer to a diskette of the personal computer which is controlled by the MS-DOS operating system.

(3) **Data Processing Flowchart of the NDSS System**

Figure 5-3 shows a flowchart of the data processing on the NDSS system. Data collection for original data files may require direct contact with researchers to obtain original test data (first material). Machine readable data can be fed directly into the computer and the data are then stored in original data files through editing programs. Data which may be found in published documents (second material) can be converted into machine readable data and fed into the computer for original data files.

Data base files for the NDSS system consist of (1) main data files for individual bank items, i.e., mechanical properties of steels, columns, beams, plates, and (2) a group of branch files under each of bank items. Detailed contents of data base files will be explained later. In the numerical data base, users may often wish to view test data not only in its original format but also in a prescribed format so that users can compare test data with their own theoretical predictions or with proposed design formulas.

Various computational and statistical manipulation programs must be available and these programs may be required for use with any documents in the system that contain numerical data. These include a graphics package that permits the presentation of certain data in the form of plots either from a plotter or on a screen.

Program packages for output data files presently available in the NDSS

system are as follows:

i) Statistical Analysis (21,22,23)

a) statistical assessments of data: mean, standard deviation, coefficient of variation, etc.,

b) construction of histograms,

c) estimation of probability distribution functions and goodness-of-fit tests (χ^2-test, Kolmogorov-Smirnov test),

d) regression analysis, and

e) occurrence of random variables.

ii) Analytical Interpretation

a) calculation of cross sectional properties of test specimens,

b) data conversion to specified parameters such as buckling parameter, nondimensional buckling strength,

c) structural analysis, and

d) reference loads calculated by design formulas.

iii) Graphics

a) histograms, cumulative distribution curves and fitted probability distribution curves,

b) scattergrams,

c) data plot on various probability papers,

d) graphical presentation of load-displacement relationships, initial crookednesses, residual stress distributions, etc., and

e) plot of test data on various specified buckling strength curves.

Information retrieval system of the NDSS system is developed primarily

for on-line processing. Off-line batch processing is also possible in the system. Through time-shearing, the on-line system can be used for searches in which users need data information immediately at a terminal in a conversational style.

5-3 INPUT DATA BASE FILES AND OUTPUT INFORMATION

(1) Input Data

In this section, the data base of material properties is mainly explained. Another items, i.e., columns, beams and plates, will be dealt in following chapters.

Following mechanical properties obtained from tensile coupon tests are stored in the main data base file.

1) upper yield stress σ_{yu},
2) lower yield stress or static yield stress σ_{yl},
3) ultimate tensile stress σ_u,
4) yield strain ε_y,
5) modulus of elasticity E,
6) initial strain hardening modulus E_{st},
7) initial strain hardening strain ε_{st},
8) Poisson's ratio ν, and
9) elongation $\Delta\ell$.

These nine items are allocated in the 80 byte card image as shown in Fig. 5-4. Nominal values specified for the same steel grade are also given. Sizes of tensile coupons, locations of the coupons in the rolled or welded section and the reference source in which the data appeared are also allocated in the record.

Available stress-strain curve of each coupon is stored in the branch file. Typical stress-strain curves of mild steel are shown in Fig. 5-5.

(2) Example of Data Retrieval

Figure 5-6 is an example of data retrieval on the NDSS system. Capital letters with underline are the input of the user, while small letters are the output list of the system.

Firstly, the NDSS is started by the command "EXEC NDSS" as shown in Fig. 5-6. The system asks which data base item is required. In this example, the steel column is selected.

The NDSS system has two main processing procedures. One of them is a direct mode of TSS and another is a local batch mode. After the selection of the direct procedure, available programs are listed. In this example the general data retrieval program is selected, which enables needed data to be copied to the user's data set and to be displayed on a graphic equipment. A list of bibliographic sources for retrieved data can be also obtained, if necessary.

In this example, box section and H-section columns with welded forming are retrieved. 406 test data retrieved are drawn to the Xerographic printer as shown in Fig. 5-7.

(3) Examples of Output Information

Some examples of the output with on-line information retrieval will be shown here. Figure 5-8 shows test results of beams from a source reference. Test points nondimensionalized by actual plastic moments are compared with the ECCS beam formulas. The strength difference between source references can be easily compared like this.

Figure 5-9 shows detailed test data of a test beam. Material properties, initial crookednesses, residual stresses, load-deflection

curves and the nondimensionalized strength of the test beam are shown by using both the main data file and the branch data files.

Other examples of output information will be shown in Chapters 5, 6, 7 and 8.

5-4 MATERIAL PROPERTIES OF STEELS

Material properties are very important parameters to evaluate the strength variation of steel structures. Alpsten (24) studied mechanical properties in detail by using about 57,000 tensile test results in Sweden. Galambos et al. (25) presented the background and the rationale for selecting material properties statistics of structural steels for use in the development of Load and Resistance Factor Design (LRFD) criteria. Nishimura (26) checked the difference of material properties between steel manufacturing companies and compared between test results and values in Mill Sheets.

In this section the data base of material properties in Japan is described.

(1) Number of Data

Original data currently stored in the data base are listed in Table 5-3. 2137 test data are categorized by steel grades, plate thickness ($t \leq 10$ mm and $t > 10$ mm) and locations of coupons.

Figure 5-10 shows a histogram of plate thicknesses of test coupons. Since test data were gathered from tensile coupon tests in structural laboratories, a number of test results of $t = 6$ mm and $t = 8$ mm are available. The mean value shown by the dotted line is about 10.5 mm.

(2) Influence Parameters on Variation of Material Properties

Three factors are considered here as influence parameters on the

variation of material properties, i.e., i) steel grade, ii) plate thickness and iii) location of coupon.

Steel Grade --- Steels can be roughly divided into three groups from the Japanese Industrial Standard (JIS) (29), i.e., i) SS (structural steels), ii) SM (weldable steels) and iii) HT (high strength steels). Figures 5-11(a), 5-11(b), and 5-11(c) show histograms of test data for lower yield stress σ_{yl}, ultimate stress σ_u, and modulus of elasticity E and they are nondimensionalized by nominal values guaranteed for each steel grade. Histograms are for three different steels. In Figs. 5-11 vertical dotted lines represent mean values and fitted normal distribution curves are also given for all the test data.

Table 5-4 shows statistical data of nine mechanical properties for three groups. Differences between means and between variances are tested by t test and F test, respectively. For lower yield stress, ultimate stress and Poisson's ratio there are little differences between SS and SM steels. However, between HT and other steels there are some differences. For modulus of elasticity there is no significant difference between steel grades, and its mean value is 2.1×10^6 kgf/cm^2 (206 kN/mm^2) which is usually specified in design specifications.

Plate Thickness --- Generally, there is a tendancy that the thinner the plate, the higher the yield stress is due to the cooling condition. The minimum specified value of yield stress in the JIS changes for three groups according to the plate thickness.

Figures 5-12(a) and 5-12(b) show histograms of lower yield stresses and ultimate stresses, respectively. These histograms are divided into two groups by the plate thickness $t \leq 10$ mm and $t > 10$ mm. Statistical data of nine mechanical properties for $t \leq 10$ mm and $t > 10$ mm are shown in Table

5-5.

There are significant differences between both means and variances for lower yield stress, modulus of elasticity and Poisson's ratio. The mean value of the lower yield stress of t ≦ 10 mm is higher by 6.1 % than that of t > 10 mm. However, it is not obvious that the correlation between lower yield stress and plate thickness even in the same steel grade exists.

Location --- In Chapter 2, it was shown that there was significant difference between yield stresses of flanges and webs of rolled H-section. Here, the difference between material properties of rolled plates and rolled shapes is described.

Figures 5-13(a) and 5-13(b) show histograms of lower yield stress and ultimate stress. These histograms are divided into two groups by locations of coupons. Statistical data of nine mechanical properties for each location are shown in Table 5-6. There are little significant differences under the significant level of 5 % between both means and variances.

(3) **Variations and Fitted Distribution Functions**

Five probability distributions are used to fit the sample frequency. The probability density functions are

Normal distribution

$$PDF = \frac{1}{\sigma\sqrt{2\pi}} e^{-\frac{(x-\mu)^2}{2\sigma^2}} \qquad -\infty < x < \infty \qquad \text{---(5-1)}$$

Log-normal distribution

$$PDF = \frac{1}{\zeta x\sqrt{2\pi}} e^{-\frac{1}{2}\left(\frac{\ln x-\lambda}{\zeta}\right)^2} \qquad 0 \leq x < \infty \qquad \text{---(5-2)}$$

Beta distribution

$$PDF = \begin{cases} \dfrac{1}{B(\alpha,\beta)(b-a)^{\alpha+\beta-1}}(x-a)^{\alpha-1}(b-x)^{\beta-1} & a \leqq x \leqq b \\ 0 & \end{cases} \qquad \text{---(5-3)}$$

Gumbel distribution

$$PDF = \alpha \exp\left[-\alpha(x-u)-e^{-\alpha(x-u)}\right] \qquad -\infty < x < \infty \qquad \text{---(5-4)}$$

Weibull distribution

$$PDF = \begin{cases} \dfrac{m}{\alpha}(x-\gamma)^{m-1}\exp\left\{-\dfrac{(x-\gamma)^m}{\alpha}\right\} & x \geqq \gamma \\ 0 & x < \gamma \end{cases} \qquad \text{---(5-5)}$$

Figures 5-14(a) and 5-14(b) show σ_{yl}/F_y and σ_u/F_u as both cumulative distribution curves and frequency histograms for all the test data. A normal distribution together with other fitted distribution curves, e.g., Weibull and Gumbel curves, tested by the Kolmogorov-Smirnov method, are also shown. Vertical arrows on the abscissa of the bottom histograms indicate the M± 2S values (M = mean, S = standard deviation), and vertical arrows on the top cumulative distribution curves indicate locations of the maximum difference D_n on the cumulative distribution curves between the

test data and the above three distribution curves in the Kolmogorov-Smirnov test.

Table 5-7 summarizes the statistical information on material properties of all the test data and goodness-of-fit distribution curves in descending order as tested by the Kolmogorov-Smirnov method. Table 5-8 shows moment coefficients of skewness and kurtosis for mechanical properties (see Appendix 5-I).

Yield Stress --- The statistical distribution of σ_{yl}, which is very close to the static yield stress σ_{ys}, may be best fitted to 1) Gumbel, 2) Weibull and then 3) Log-normal distributions. The σ_{yl} histogram is in good agreement with Gumbel's in the peak range and with Weibull's at both tails.

Figures 5-15(a) and 5-15(b) show nondimensionalized yield stresses of SS41 steels plotted on the normal and log-normal probability papers. As shown in Figs. 5-15, it is obvious that the distribution of yield stresses is not symmetrical but has a positive skewness. This may be due to the existence of the guaranteed minimum value.

The mean value of lower yield stresses $(\sigma_{yl})_m$ is;

$$(\sigma_{yl})_m = 1.168F_y \text{ and } COV\ \omega = 0.111 \qquad \text{---(5-6)}$$

for all the test data.

There are 7 % of the specimens which have lower values than the minimum specified value. This may be due to the difference of test methods. Since the data in the NDSS were gathered from structural laboratories, the strain rate of loading may be almost zero. Minimum specified values in the JIS may guarantee upper yield stresses.

Modulus of Elasticity --- The mean value of modulus of elasticity is almost 2.1×10^6 kgf/cm^2 (206 kN/mm^2) which is specified in the JRA (30). The coefficient of variation of modulus of elasticity of 4.5 % is the smallest among material properties

Ultimate Stress --- The coefficient of variation of nondimensionalized ultimate stresses is as small as 6.27 %. The sample frequency distribution of ultimate stresses may be fitted to a normal distribution, but the kurtosis is a little bit Leptokurtic.

Strain-Hardening Properties --- Strain-hardening properties i.e., initial strain hardening modulus and hardening strain are essential in the application of the plastic analysis. Coefficients of variation of these properties are large than those of other mechanical properties. However, since there are comparatively small numbers of data and the hardening properties may not be well defined due to the complex behavior, the reliability of these data may be low.

(4) Comparison between Test Results and Mill Sheet Values

Mill Sheets of steel manufacturing companies contain the yield stress, ultimate stress and elongation as well as the chemical composition. Material properties in Mill Sheets are usually determined by the JIS procedure. Since the incremental stress rate of loading is specified in the range of 1 to 3 kgf/mm^2/s (9.8 to 29 N/mm^2/s) in the JIS, yield stresses in Mill Sheets may correspond to upper yield stresses.

Figures 5-16(a) to 5-16(d) show histograms of upper yield stresses, lower yield stresses, ultimate stresses and elongations divided by values in Mill Sheets, respectively. Hatched area is of test results of rolled shapes. Statistical data are shown in Table 5-9. There is little difference between ultimate stresses of test results and Mill Sheet values, and its

coefficient of variation is as small as 5.5 %. This may be reasoned that the strain effect is almost zero upon the ultimate stress.

(5) Relationship between Mechanical Properties

Figures 5-17(a) and 5-17(b) show scattergrams between σ_{yu} and σ_{yl} for N = 174 and σ_{yl} and σ_u for N = 1612, respectively. Regression analysis, which is a program package included in branch files, gives the following linear relations between two variables.

$$\sigma_{yu} = 0.952\sigma_{yl} + 52.5 \quad N/mm^2$$

$$\sigma_{yu} = 0.952\sigma_{yl} + 535 \quad kgf/cm^2 \qquad \text{--- (5-7)}$$

with the correlation coefficient R = 0.920.

$$\sigma_{yl} = 1.205\sigma_u - 245.8 \quad N/mm^2$$

$$\sigma_{yl} = 1.205\sigma_u - 2506 \quad kgf/cm^2 \qquad \text{--- (5-8)}$$

with the correlation coefficient R = 0.963.

5-5 CONCLUSIONS

This chapter contains the basic framework for the data base NDSS (Numerical Data Base for Steel Structures) regarding the ultimate strength of steel structures and structural components. Instrumentation, processing diagrams, input data base files and output information of the NDSS system are described. Some of the more readily available test series on mechanical properties of steels, columns, beams, and plate strengths are stored in the

data base. Statistical programs are prepared which permit the analysis of test data and which enable probability models to be verified. Graphics package is also prepared to allow the presentation of certain data in the form of plots on paper or at a terminal. Most of the figures contained in Chapters 5, 6, 7 and 8 have been produced automatically in hard forms by the graphics package.

Comparison and calibration of test data with any proposed buckling design formulas may be arranged with a view to insuring appropriate levels of structural reliability in the limit states design method.

The following main conclusions are obtained

(1) A numerical data base system was built for material properties of steels, steel columns, steel beams and unstiffened steel plates. A small data base management system of the relational data base was also developed.

(2) Variations and fitted distribution functions of material properties are made clear from the data base approach. The mean value and the coefficient of variation of yield stresses nondimensionalized by nominal values are 1.168 and 0.111, respectively. The frequency distribution of yield stresses has a positive skewness.

Appendix 5-I Two Measures of Departure from Normality

Skewness is the degree of asymmetry, or departure from symmetry of a distribution, and kurtosis is the degree of peakness of a distribution. Skewness and kurtosis are the two simplest measures of departure from normality. They can be defined by using the statistics of the 3rd and 4th degree.

$$g_1 = k_3/k_2^{3/2}, \quad g_2 = k_4/k_2^2 \qquad \text{---(5-A-1)}$$

$$k_2 = \frac{1}{n-1}\sum(x-\bar{x})^2, \qquad k_3 = \frac{n}{(n-1)(n-2)}\sum(x-\bar{x})^3$$

$$k_4 = \frac{n}{(n-1)(n-2)(n-3)}\left\{(n+1)\sum(x-\bar{x})^4 - 3\frac{n-1}{n}\left[\sum(x-\bar{x})^2\right]\right\}$$

in which n = number of sample data, x = value of sample data and $\bar{x}$ = mean value. When the sample data obey the normal distribution, the standard deviations of these statistical values are

$$\begin{cases} S_1 = \sqrt{\dfrac{6n(n-1)}{(n-2)(n+1)(n+3)}} \\ S_2 = \sqrt{\dfrac{24n(n-1)^2}{(n-3)(n-2)(n+3)(n+5)}} \end{cases} \qquad \text{---(5-A-2)}$$

The departure from normality can be obtained from V_1 and V_2

$$\begin{cases} V_1 = g_1/S_1 \\ V_2 = g_2/S_2 \end{cases} \qquad \text{---(5-A-3)}$$

V_1 and V_2 are equal to zero when the sample data obey the normal distribution absolutely. If V_1 is positive, the distribution is said to be skewed to the right or to have positive skewness as shown in Fig. 5-A. If the reverse is true, it is said to be skewed to the left or to have negative skewness. V_2 is positive for a leptokurtic distribution, negative for a platykurtic distribution as shown in Fig. 5-B.

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NOTATION

The following symbols are used in this chapter:

E	= modulus of elasticity;
E_n	= specified modulus of elasticity;
E_{st}	= initial strain hardening modulus;
F	= frequency;
F_u	= nominal ultimate stress;
F_y	= nominal yield stress;
M	= mean value;
R	= correlation coefficient;
S	= standard deviation;
t	= plate thickness;
$\Delta\ell$	= elongation;
ε_{st}	= initial strain hardening strain;
ε_y	= yield strain;
σ_u	= ultimate stress;
σ_y	= yield stress;
σ_{yl}	= lower yield stress;
σ_{ys}	= static yield stress;
σ_{yu}	= upper yield stress;
ν	= Poisson's ratio;
ω	= coefficient of variation.

Table 5-1(a) Comparison of Track Allocations on Double Side Floppy

Track No.	Surface No.	Sector No.	N88-Basic	F-Basic	University Computer (IBM Compatible)
0	0	1-4	IPL	IPL	reserved for system
		5-6	reserved	reserved	error map
		7			volume label
		8-26			directory
	1	1		FAT	
		2		ID	
		3		reserved	
		4-26		directory	
1-34	0,1	1-26	data area	data area	data area
35	0	1-22	directory		
		23	ID		
		24-26	FAT		
	1	1-26	data area		
36-74	0,1	1-26			
75-76	0,1	1-26			alternate track

Table 5-1(b) Comparison of Track Allocations on Single Side Floppy

Track No.	Sector No.	MS-DOS (V.1.25)	University Computer (IBM Compatible)
0	1	IPL	reserved for system
	2-4	FAT	
	5-6		error map
	7		volume label
	8-14		directory
	14-26	directory	
1	1-4		data area
	5-26	data area	
2-73	1-26		
74-76	1-26		alternate track

Table 5-2 Comparison of Directories

Location (Byte)	N88-Basic	F-Basic	MS-DOS	CP/M and CP/M-86
0	file name	file name	file name	entry type
1-5				file name
6-7	extension			
8		unused	extension	
9	attribute			extension
10	cluster no.			
11	unused	attribute	attribute	
12		ASCII flag	unused	file extent
13		random access flag		unused
14		cluster no.		
15		unused		current extent size
16-21	---			directory allocation table
22-23	---		time of creation	
24-25	---		date of creation	
26-27	---		cluster no.	
28-31	---		area size	

Table 5-3 Number of data in Tensile Coupon Test

	Steel Grade			Plate Thickness		Location		Total No.
	SS	SM	HT	$t \leq 10$	$t > 10$	Plates	Shapes	
σ_{yu}	169	5	0	173	1	158	16	174
σ_{yl}	1337	508	209	1474	580	1330	724	2054
σ_u	1112	328	179	1192	427	1027	592	1619
ε_y	239	9	5	240	13	172	81	253
E	775	161	88	885	139	661	363	1024
E_{st}	242	19	4	243	22	172	93	265
ε_{st}	261	19	1	258	23	180	101	281
ν	412	117	59	521	67	506	82	588
Δl	526	100	41	509	158	416	251	667

σ_{yu} = Upper yield stress σ_{yl} = Lower yield stress

σ_u = Ultimate stress ε_y = Yield strain

E = Modulus of elasticity E_{st} = Strain-hardening modulus

ε_{st} = Strain at onset of strain-hardening

ν = Poisson's ratio Δl = Elongation

Table 5-4 Statistical Data in Three Steel Grades

	SS (SS34 - SS50)				SM (SM41 - SM58)				HT (HT60 - HT80)			
	N	M	S	ω	N	M	S	ω	N	M	S	ω
σ_{yu}/F_y	169	1.401	0.156	0.111	5	1.174	0.175	0.149	0	-	-	-
σ_{yl}/F_y	1337	1.179	0.132	0.112	508	1.162	0.135	0.116	209	1.112	0.082	0.074
σ_u/F_u	1112	1.096	0.069	0.063	328	1.099	0.066	0.060	179	1.054	0.059	0.056
$\varepsilon_y/\varepsilon_{yn}$	239	1.322	0.171	0.129	9	1.253	0.211	0.168	5	1.173	1.099	0.084
E/E_n	775	0.998	0.044	0.044	161	1.006	0.045	0.045	88	0.999	0.054	0.054
E_{st}	242	0.039	0.011	0.283	19	0.033	0.010	0.295	4	0.032	0.013	0.403
ε_{st}	261	20170	4304	0.213	19	18290	6327	0.346	1	-	-	-
ν/ν_n	412	0.936	0.075	0.080	117	0.952	0.114	0.120	59	0.912	0.076	0.083
$\Delta l/\Delta l_n$	526	1.586	0.285	0.180	100	1.393	0.265	0.190	41	1.207	0.290	0.240

F_y = Nominal yield stress

F_u = Nominal ultimate stress

ε_{yn} = Nominal yield strain

E_n = 2.1 kg/cm^2

ν_n = 0.3

Δl_n = Nominal elongation

N = Number of data

M = Mean value

S = Standard deviation

ω = Coefficient of variation

Table 5-5 Statistical Data in Two Groups of Plate Thickness

	$t \leq 10$				$t > 10$			
	N	M	S	ω	N	M	S	ω
σ_{yu}/F_y	173	1.396	0.161	0.115	0	-	-	-
σ_{yl}/F_y	1474	1.187	0.132	0.111	580	1.119	0.110	0.098
σ_u/F_u	1192	1.091	0.070	0.064	427	1.094	0.065	0.060
$\varepsilon_y/\varepsilon_{yn}$	240	1.328	0.168	0.127	13	1.104	0.127	0.115
E/E_n	885	1.000	0.044	0.044	139	0.991	0.053	0.053
E_{st}	243	0.038	0.011	0.284	22	0.043	0.013	0.305
ε_{st}	258	20496	4198	0.205	23	14560	4243	0.291
ν/ν_n	521	0.940	0.086	0.092	67	0.909	0.068	0.075
$\Delta l/\Delta l_n$	509	1.545	0.287	0.186	158	1.496	0.344	0.230

Table 5-6 Statistical Data in Two Locations

	Plates				Shapes			
	N	M	S	ω	N	M	S	ω
σ_{yu}/F_y	158	1.414	0.154	0.109	16	1.201	0.077	0.064
σ_{yl}/F_y	1330	1.166	0.134	0.115	724	1.172	0.123	0.105
σ_u/F_u	1027	1.088	0.069	0.064	592	1.098	0.066	0.060
$\varepsilon_y/\varepsilon_{yn}$	172	1.357	0.170	0.125	81	1.231	0.145	0.118
E/E_n	661	1.001	0.046	0.046	363	0.995	0.042	0.042
E_{st}	172	0.039	0.011	0.290	93	0.038	0.011	0.287
ε_{st}	180	21300	4100	0.193	101	17800	4300	0.244
ν/ν_n	506	0.938	0.078	0.084	82	0.928	0.119	0.128
ℓ/ℓ_n	416	1.556	0.281	0.181	251	1.497	0.331	0.221

Table 5-7 Statistical Data of Mechanical Properties

	N	M	S	ω	Good-fit Distributions (1)	(2)	(3)
σ_{yu}/F_y	174	1.394	0.161	0.115	Gumb,	Log	
σ_{yl}/F_y	2054	1.168	0.130	0.111	Gumb,	Weib,	Log
σ_u/F_u	1619	1.092	0.068	0.063	Nor,	Beta,	Weib
$\varepsilon_y/\varepsilon_{yn}$	253	1.317	0.173	0.131	Log,	Gumb	
E/E_n	1024	0.999	0.045	0.045	Log,	Nor	
E_{st}	265	0.038	0.011	0.289	Nor,	Weib	
ε_{st}	281	20010	4500	0.225	Weib,	Nor	
ν/ν_n	588	0.937	0.085	0.091	Gumb,	Log	
$\Delta l/\Delta l_n$	667	1.534	0.302	0.197	Nor,	Beta	

Nor = Normal distribution

Log = Log-normal distribution

Beta = Beta distribution

Gumb = Gumbel distribution

Weib = Weibull distribution

Table 5-8 Moment Coefficients of Skewness and Kurtosis

			σ_{yu}/F_y	σ_{yl}/F_y	σ_u/F_u	$\varepsilon_y/\varepsilon_{yn}$	E/E_n	E_{st}	ε_{st}	ν/ν_n	$\Delta l/\Delta l_n$
Steel Grade	SS	V_1	0.85	7.85	-4.91	1.17	1.16	0.99	-2.74	11.82	5.62
		V_2	-1.69	1.02	2.46	-1.27	8.97	0.52	-2.09	36.67	3.71
	SM	V_1	0.40	5.83	3.21	-0.71	-0.46	0.24	0.98	4.55	-3.42
		V_2	-1.00	4.00	5.03	0.05	3.76	0.25	-0.51	4.23	1.33
	HT	V_1	-	-1.41	-4.38	-1.95	0.86	-0.64	-	2.04	-1.69
		V_2	-	4.04	10.93	1.50	1.19	-0.40	-	1.30	-0.02
Plates Thickness	$t \leq 10$	V_1	0.33	9.03	-4.02	1.25	1.82	1.14	-2.94	12.96	4.40
		V_2	-1.31	3.10	22.50	-1.23	9.86	1.18	-1.19	27.46	5.35
	$t > 10$	V_1	-	4.40	0.88	-0.58	0.16	-0.69	4.69	-0.29	-0.91
		V_2	-	1.75	10.48	-0.62	1.03	-0.87	7.66	2.45	1.85
Location	Plates	V_1	-0.08	8.21	-1.43	0.30	-0.60	0.01	-3.43	10.04	0.49
		V_2	-0.63	3.22	1.58	-1.01	7.12	0.68	0.45	18.85	5.38
	Shapes	V_1	2.14	8.08	-3.51	0.31	3.12	1.84	0.58	6.54	3.30
		V_2	0.71	3.50	22.94	-0.55	6.46	0.14	-1.75	12.13	2.57
Total		V_1	0.38	11.11	-3.25	0.99	1.22	1.04	-2.31	13.14	2.30
		V_2	-1.35	4.69	25.05	-1.07	9.17	0.49	-2.12	28.99	6.05

Table 5-9 Comparison between Test Results and Mill Sheet Values

	M	S	ω
σ_{yu}/σ_{uM}	1.035	0.148	0.143
σ_{yl}/σ_{yM}	0.947	0.111	0.117
σ_u/σ_{uM}	0.999	0.036	0.036
$\Delta\ell/\Delta\ell_M$	0.927	0.137	0.148

Fig. 5-1 Instrumentation in NDSS

```
MSDOS
 ****************************
 *   NDSS DATA-BASE SUBSYSTEM  *
 * MS-DOS(V 1.25) FLOPPY       *
 * CENTER DSN ----> FLOPPY     *
 * FLOPPY     ----> CENTER DSN *
 *      VERSION 1.3            *
 ****************************
                    DATA 06/13/84
  *** DEAD READ START ***

 DID YOU SET A FLOPPY TO THE FLOPPY DEVICE (Y OR N OR SKIP)=? Y
 *** MS-DOS FLOPPY(1S) DEAD READ ***

 *** FPU1Y001   M FPU.DUMMY1 ***
 *** FPU1Y002   K FPU.DUMMY1 ***
  NUMBER OF RECORD IS  1924

 *** MS-DOS FLOPPY(1S) DEAD READ END ***
   *** DATOFP AND FPDRDA START ***

 DO YOU WANT DATOFP OR FPTODA (Y OR N)=? Y
 *** MS-DOS FLOPPY START (DATOFP FPTODA) ***
 *** COMMAND TABLE ***

1  : AREA      --- UNUSED AREA OF FLOPPY
2  : DATOFP    --- COPY FROM CENTER DSN TO FLOPPY
3  : DELETE    --- DELETE A FLOPPY FILE
4  : FILES     --- FILE NAME LIST OF FLOPPY
5  : FPTODA    --- COPY FROM FLOPPY TO CENTER DSN
6  : INIT      --- INITIALIZE AS MS-DOS FLOPPY
7  : LIST      --- SOURCE LIST OF A FLOPPY FILE
8  : LISTC     --- CENTER DSN LIST
9  : LISTD     --- DCB LIST OF A CENTER DSN
10 : LISTF     --- LIST OF FAT TABLE
11 : END       --- END
  INPUT COMMAND     =? 2
  UNUSED CLUSTER NUMBER =  432
  INPUT FULL NAME OF CENTER DSN =? ZZ9999.NDSS.DATA
  INPUT ELM NAME. IF NECESSARY =? _
      1 : BASIC PROGRAM    --- LRECL=<254  BYTE
      2 : DATA (RANDOM)    --- LRECL=<128  BYTE
      3 : DATA (SEQUENTIAL)-- LRECL=<2560 BYTE
      4 : FORTRAN PROGRAM --- LRECL=<80   BYTE
SELECT FILE TYPE NO=? 3
 RECORD NO. OF CENTER DSN =  286
DSN= ZZ9999.NDSS.DATA ELM=
RECORD LENGTH OF CENTER DSN    =    80
RECORD NUMBER OF CENTER DSN    =   286
RECORD NUMBER IN A SECTOR      =     1
NECESSARY SECTORS  OF FLOPPY   =    55
NECESSARY CLUSTERS OF FLOPPY   =    14
NO. OF SECTORS IN LAST CLUSTER=      3
*** PUSH ENTER KEY ***   =?
 *** COMMAND TABLE ***
          } command table
 INPUT COMMAND     =? END

*** MS-DOS FLOPPY END   (DATOFP FPTODA) ***
 *** DEAD WRITE START ***

DID YOU SET A FLOPPY TO THE FLOPPY DEVICE (Y OR N OR SKIP)=? Y
*** MS-DOS FLOPPY DEAD WRITE ***

*** FPU1Y001   M FPU.DUMMY1 ***
*** FPU1Y002   K FPU.DUMMY1 ***
 NUMBER OF RECORD IS  1924

*** MS-DOS FLOPPY DEAD WRITE END ***
*** MS-DOS FLOPPY END ***
```

Fig. 5-2 An Example of Data Transfer

Fig. 5-3 Flowchart of Data Processing in NDSS

Fig. 5-4 Data Format for Mechanical Properties in NDSS

Fig. 5-5 Typical Stress-Strain Curves of Mild Steel

```
EXEC NDSS
  *** welcome to ndss application system 12/08/83 ***
  ****************************************************
  *        *** welcome to ndss data base ***         *
  *                                                  *
  * if you have any trouble, please telephone to ext 4632*
  ****************************************************
                                          date 12/08/83

please select the data type
'material','column','beam','plate','end'in which ? COLUMN
column data bank start

please select procedure
' d'  tss demand,  l  local batch ? D
** tss demand system of column data start **

                program table for column data in ndss

          elm name type contents
            pcolsect   s  drawing the cross sections in tests
            psymbol    s  drawing the available symbols for plotting
                |          |                |
            lsearch3   l  general searching program
                  jpl table ended
 end of data
select the program name and type in the above table ? LSEARCH3.L
xyout file name input ? PREV1
if necessary, input dsn and dd name ?
** select the searching parameter **
*   1 type of section
*   2 method of fabrication
*   3 cutting method of plate
*   4 after fabrication
*   5 thichness of plate
*   6 hight and flange width
*   7 test type of material
*   8 bending axis
*   9 type of failure mode
*  10 test country
*  11 sectional area
*  12 detailed type of section
*  13 reference
*  14 values of yield stress
*  15 values of slenderness ratio
*  16 basic design curve
** number of parameter and parameter no.?
00250 ? 2,1,2
2 1 2
* 1      type of section
b    box section
c    composite
h    h section
o    circular round section
r    solid round section
s    tube section (square)
t    t section
   ** select the parameter ( ex  a+b+c ) ?
   02590 ? B+H
   b+h
   number of data in a group is 1084
   * 2      method of fabrication
   h    hybrid welded
   k    make the section by cutting
   p    rolled with welded cover plate
   q    quenched after rolled
   r    rolled
   v    riveted
   w    welded
        unknown
   ** select the parameter ( ex  a+b c ) ?
   02590 ? W
   w
   number of data in a group is 711

   number of total data is 408

   ** output reference no  ? ** y or n
   00570 ? Y
   y
   ** the searched data were in the next references **

   13  mcfalls,r k , and tall,l
       a study of welded columns manufactured from flame cut plates
       welding journal, vol  48,april,1969
                                  |
   29  aoki,t , and fukumoto,y
       experimental study of welded h 200x200 columns
       preprint of annual meeting,jsce,october,1982
** output data to file  / ** y or n
00770 ? Y
y
** next step ? ** y or n
00150 ? N
n
   table of output machine for xy file
         n   don't output
         e   electro graphic (versatec)
         r   lazer printer
         t   tektronix and xy plotter
   where output xy  ? E
        versaplot x07 0
iev090i   &plot lblk=4000 &end
if xyout file not necessary, input 'n' ? N
entry (a)        prev1 deleted
*** xy successfully ended ***
** tss demand system of column ended **
column data bank ended

please select the data type
'material','column','beam','plate','end'in which ? END
ndss successfully ended, see again
```

Fig. 5-6 An Example of Data Retrieval in NDSS

Fig. 5-7 An Example of Graphic Output

Fig. 5-8 Test Results of Beams from a Reference

D-SERIES NO 2

	flange	web
σ_{yu}	3584	4182
σ_{yl}	3175	3695
σ_u	4518	4756
E	2.12	2.13
E_{st}	0.0496	0.0426
ε_y	1678	1917
ε_{st}	22819	25917
ν	0.276	0.276
Δl	24.2	22.5

(a) Material properties

(d) Load - Deflection Curves

(b) Residual Stress (c) Initial Crookedness

(e) Buckling Test Point

Fig. 5-9 Detailed Test Results of a Beam

Fig. 5-10 Histogram of Plate Thickness

Fig. 5-11(a) Histograms of Lower Yield Stress

Fig. 5-11(b) Histograms of Ultimate Stress

Fig. 5-11(c) Histograms of Modulus of Elasticity

Fig. 5-12(a) Histograms of Lower Yield Stress

Fig. 5-12(b) Histograms of Ultimate Stress

Fig. 5-13(a) Histograms of Lower Yield Stress

Fig. 5-13(b) Histograms of Ultimate Stress

Fig. 5-14(a) Histogram and Cumulative Curve of Yield Stress

Fig. 5-14(b) Histogram and Cumulative Curve of Ultimate Stress

Fig. 5-15 Yield stresses on Normal and Log-normal Probability Papers

Fig. 5-16 Comparisons between Test Results and Mill Sheet Values

Fig. 5-17(a) Scattergram between Upper and Lower Yield Stresses

Fig. 5-17(b) Scattergram between Lower Yield Stresses and Ultimate Stresses

Fig. 5-A Skewness

Fig. 5-B Kurtosis

Chapter 6. EVALUATION OF COMPRESSIVE STRENGTH OF CENTRALLY LOADED STEEL COLUMNS

6-1 INTRODUCTION

Extensive researches have been made since 1950 investigating the ultimate strength of centrally loaded steel columns. This has involved studies of residual stress distributions and initial crookednesses (1-34). The CRC Column Strength Curve has been generally accepted and used as the basis for steel column design formula in a number of specifications. Some European countries have adopted the multiple column curves (35,36) recommended by the European Convention for Constructional Steelwork (ECCS). These curves are designated as (a_0), (a), (b), (c) and (d), and the Structural Stability Research Council (SSRC) multiple column curves (37) are designated 1,2 and 3, depending on the different fabrication process, cross-sectional shape, buckling axis, steel grade, etc.

The multiple column curves of the ECCS and SSRC were obtained numerically by computer simulations using deterministic and probabilistic approaches with experimental geometrical imperfections and residual stresses and yield stresses (38,39,40).

The author has been developing a numerical data-base for the ultimate strength of steel structures (41,44,45) under the guidance of Professor Fukumoto. The author initiated an extensive survey of column tests and stored the test data on a Numerical Data-Base for Steel Structures (NDSS). This chapter summarizes the test data editing, searching, retrieval and management that have been carried out on steel columns. After various computational and statistical manipulations, the information on steel column tests has been evaluated and comparisons made statistically between

the surveyed test data and the ECCS and SSRC multiple column curves. Proposals for new multiple column curves are made from the experimental data-base correlations.

6-2 COLUMN TEST DATA STORED ON THE NUMERICAL DATA-BASE

(1) Number of Column Tests Surveyed

During the past thirty years numerous series of tests have been conducted in European countries, North America and Japan. Test results from various sources (1-30) have so far been assembled and stored in the NDSS. A total of 1665 individual test results is categorized by cross-section type and country of origin as given in Table 6-1. In Table 6-1, 1018 test results relate to Europe, 137 to the U.S.A. and 510 to Japan. The test columns are of rolled H, welded H, welded box, square tube, circular tube, circular solid, T, and composite shapes. The original papers, in which the data were presented, are listed in Refs.(1)-(30).

The columns, except the circular solid shapes (3,4), were tested under pin-ended conditions. Since the effective length factors of the circular solid columns were determined experimentally in Refs.(3) and (4), the test results have been modified to the pin-ended condition herein.

For the ECCS column data, copies of the original laboratory data sheets have been supplied by J. Brrozzetti of CTICM with information on measured yield stress, cross-sectional dimensions and initial deformation. A preliminary survey of the American column data are carried out in Ref.(16) but much of the data have been re-checked from the original papers.

(2) Explanation of Surveyed Column Data

The test data on each specimen consist of the following items: a) maximum load, P_{max}, b) measured cross sectional area, $(A)_a$, c) nominal cross sectional area, $(A)_n$, d) measured yield stress, $(\sigma_y)_a$, e) nominal yield stress, $(\sigma_y)_n$, and f) slenderness ratio about the test axis. The following test information which belongs to the individual column is also stored in the data-base files with items a) to e) above:

1) Type of profile : I-shape, box shape, circular tube shape, square tube shape, solid round shape, T-shape
name of profile --- IAP150, 8WF31, etc. (71 names).

2) Fabrication process : before assembly --- flame-cut, shear-cut, universal mill;
: during assembly --- rolled, welded, drawn, cut-out forming;
: after assembly --- as-delivered, annealed, cold-straightened.

3) Geometry : ordinal section, heavy section;
: height h / width b;
: cross sectional dimensions.

4) Yield stress : steel grade;
: testing method --- stub column test, tensile coupon test.

5) Buckling axis : major axis, minor axis.

6) Failure mode : flexural buckling, torsional buckling, local buckling.

7) Source reference : original paper;
: country in which testing was conducted.

8) Initial crookedness.

9) Column curve according to
: ECCS column curves (a_0), (a), (b), (c) and (d);
SSRC column curves (1), (2) and (3).

10) Identification : column test index;
: sequential data number.

In the following correlations columns which failed by torsional and

local modes and columns formed by the cut-out process are omitted. Since the measured modulus of elasticity E of steel has been reported in very few of the surveyed papers, the value of E = 206 kN/mm^2 has been used for processing the data.

(3) Cross-Sectional Area, Yield Stresses and Initial Crookednesses

Cross-Sectional Area --- Table 6-2 lists nominal cross-sectional dimensions and cross-sectional areas of the columns. The columns vary from a maximum cross-sectional area of 60.7 cm^2 for rolled 14WF320, to a minimum cross-sectional area of 5.25 cm^2 for circular tubes with D = 60.5 mm and t = 2.9 mm. A total of 1535 test columns have had their cross-sectional areas $(A)_a$ measured. The mean value M of $(A)_a/(A)_n$ for these columns is 1.01, and the standard deviation S and the coefficient of variation ω are 0.0365 and 0.034, respectively.

Whenever possible the measured cross-sectional area has been used to obtain the actual column strength in the nondimensional form, while the nominal cross-sectional area has been used when the measured cross-sectional area was not available.

Yield Stress --- It appears that a yield stress value at a slow strain rate may be the most indicative with respect to true column behavior. Yield stresses obtained by tensile coupon tests, $(\sigma_y)_t$ and by stub column tests, $(\sigma_y)_{stub}$ are both given when they are available. The yield stress from a stub column test is determined by dividing the axial yield load P_y by the actual cross-sectional area $(A)_a$.

The mean value M of $(\sigma_y)_{stub}/(\sigma_y)_t$ is 1.046 for 641 of the ECCS test columns for which yield stresses have been obtained by both methods. The standard deviation S and the coefficient of variation ω are 0.0907 and 0.0867, respectively. Whenever possible the yield stress obtained from a

stub column test has been used to calculate the actual nondimensional maximum stress. However, since this information is only available for about 85 % of the column tests, tensile coupon results have had to be used for the remainder.

Figure 6-1(a) represents a histogram of the ratio of measured yield stress $(\sigma_y)_{stub}$ for 1217 of the test columns to nominal yield stress $(\sigma_y)_n$ and Fig. 6-1(b) for $(\sigma_y)_t$ of 222 columns. The mean measured value, standard deviation and coefficient of variation of $(\sigma_y)_{stub}$ are M = 1.27, S = 0.127 and ω = 0.010, respectively. The values of $(\sigma_y)_t$ are M = 1.24, S = 0.169 and ω = 0.0136, respectively. Quite substantial variations are present with the mean measured value, being 27 and 24 percent in excess of the nominal figures.

Initial Crookedness --- The initial crookedness of a column is one of the important factors that influence the column strength. An initial crookedness of 0.001ℓ at mid-height in the plane of bending has been generally accepted as the tolerance appropriate for the column curves. Figure 6-1(c) shows a histogram of the $(f_o/\ell)_{max}$ value for 208 of the IPE160 test columns of the ECCS, in which $(f_o)_{max}$ is the maximum initial crookedness and ℓ is the length of column. The mean value and standard deviation of $(f_o/\ell)_{max}$ are M = 0.79/1000 and S = 0.326/1000, respectively. Furthermore, the $(f_o/\ell)_{max}$ values for 437 of the I-section columns in NDSS are M= 0.501/1000 and S = 0.433/1000, respectively. Some 11 percent of I-section columns are in excess of the $\ell/1000$ value.

6-3 COLUMN STRENGTHS FOR VARIOUS PROFILES

(1) H-Section

Rolled H-Section --- Cross-sectional dimensions of rolled H-sections

vary between height h = 100 mm to 365 mm, flange width b = 75 mm to 371 mm, flange thickness t_f = 7.4 mm to 32.7 mm and web thickness t_w = 4.5 mm to 23.0 mm. Table 6-2 lists nominal cross-sectional dimensions of the columns. DIR20, 12WF61 and 12WF120 are with flange thickness in excess of 25.4 mm (1 inch).

To compare column strengths between different types of section, the ratio of the test points compared with a column curve is calculated for the same cross-sectional profile. The ECCS column curves (b) is specified for all types of profile. Table 6-3 summarizes the statistical results, that is, the mean and the mean minus two standard deviations of the ratio for each type of section. The following two yield loads P_y have been used to non-dimensionalize the column strength : (i) the actual $(P_y)_a$ which is obtained by using the measured yield stress $(\sigma_y)_a$ and measured cross-sectional area $(A)_a$, and $(Exp)_a = P_{max}/(P_y)_a$; and (ii) the nominal $(P_y)_n$ which is obtained by using the nominal yield stress $(\sigma_y)_n$ and nominal cross-sectional area $(A)_n$, and $(Exp)_n = P_{max}/(P_y)_n$. In Table 6-3 the test columns are further categorized into sub-groups with respect to the buckling axis, the steel grade, and the fabrication process after hot rolling such as as-delivered (D), annealed (A), and cold-straightened (C).

The strength characteristics of the rolled H-sections may be summarized as follows from the results of Table 6-3:

(1) The mean strength of the annealed columns is 10 % - 30 % higher than that of the as-delivered columns.

(2) No clear difference in column strengths between major- and minor-axis buckling may be found from the test data. This may be partly due to the limited number of test points (only 16) for major-axis buckling.

(3) The mean strength for high strength steel with σ_y > 250 N/mm^2 is 10 %

- 20 % higher than that for structural carbon steel for all types of profile. This may be because the magnitude and distribution of the residual stresses are mainly influenced by the geometry of the cross section and the amount of heat input and not by the yield stress.

(4) Heavy rolled H-sections with flange thickness in excess of 25.4 mm (1 inch) possess strengths 20 % - 30 % less than those of normal H-sections. 14WF111 sections with a flange thickness of 22.2 mm tend to be of even lower strength.

(5) H-columns with a depth-to-width ratio of more than 1.2, such as IAP160, IPE200, IPN160, RSJ5"x3" and 12WF50, tend to have higher strengths but these are subject to greater variation.

(6) The largest coefficient of variation for column strength over each 0.1 interval of the nondimensional slenderness ratio $\bar{\lambda}$ occurs for $\bar{\lambda} \fallingdotseq 1.25$.

Welded H-Section --- Ten different welded H-profiles as summarized in Table 6-2 have been stored in the NDSS. Their dimensions are similar to those of rolled H-sections. It should be mentioned that the cross-sectional dimensions of some test columns may be smaller than those in the practical use.

Table 6-4 summarizes the statistical results for each type of welded H-sections. The columns are categorized into sub-groups with respect to the buckling axis, the steel grade and the fabrication process after assembly. Fabrication processes such as flame-cutting (F), universal mill (U), shear-cutting (S) or plate annealed (E) are also named for plate elements.

The strength characteristics of the welded H-sections may be summarized as follows from the results of Table 6-4:

(1) Columns with universal mill plates possess strengths between 20 % and

35 % less than those with flame-cut plates. This may be explained by the fact that the flame-cutting of plates yields a favorable compressive residual stress along the plate edges. Flame-cut columns may be lower in strength than annealed columns.

(2) Annealed columns may possess the highest strength.

(3) High-strength welded steel columns possess relatively high strength compared with rolled H-section of high strength steel.

(4) H-100x100x6x8 columns in Ref.(18) are appreciably weaker and it may be concluded that unfavorable residual stresses were locked-in in these small columns although flange components are flame-cut. The statistical results shown bracketed in Table 6-4 are those for all the columns excluding those given in Ref.(18).

Comparison between Rolled and Welded H-Sections --- From a comparison of Tables 6-3 and 6-4, column strength may be less in welded columns than in rolled columns. This result may be due to the combined effect of unfavorable residual stresses and a larger initial out-of-straightness. Furthermore, the strength variation of welded H-sections is larger than that of rolled H-sections. For welded H-sections, the differentiation between flame-cut and universal mill plates may be an important factor which influences column strength.

Other Shapes --- Riveted columns and composite columns are omitted as well as columns with cover plates because of the small number of results available.

(2) Welded Box Section

Table 6-5 summarizes the statistical results for welded sections. The strength characteristics of welded box sections may be summarized as follows:

(1) Columns with universal mill plates possess strengths less than those with flame-cut plates.

(2) Annealed columns may possess strengths between 20 % and 30 % higher than as-delivered columns for structural carbon steel and about 10 % higher for high strength steel.

(3) High strength steel columns possess relatively high strength as would be expected for H-sections. B-100x6 columns with structural carbon steel may possess exceptionally high strength.

(3) Tubular Section

Circular Tubes --- Table 6-6 summarizes the statistical results for circular tubes. Test columns are either seamless pipes or one-seamed tubes. One-seamed tubes were fabricated by butt welding. Steel grades of columns have been categorized into two groups with yield stresses of 235 N/mm^2 (24 kgf/mm^2) and of 324 N/mm^2 (33 kgf/mm^2). The maximum and minimum outer radii are 139.8 mm (A = 19.13 cm^2) and 60.5 mm (A = 5.25 cm^2) respectively for eleven of the tubes.

The strength characteristics of the circular tubes are as follows:

(1) The mean strength ratio of annealed : seamless : one-seamed is 1.05 : 1.02 : 1.0 . The exact fabrication process of the tubes, i.e., welded or seamless, appears to have had little effect on column strength.

(2) Tubes possess the highest column strength compared with other sections.

Square Tubes --- Table 6-7 summarizes the statistical results for square tubes. Test columns are either seamless square tubes or seamed square tubes. The steel yield stress is 235 N/mm^2 (24 kgf/mm^2).

The strength characteristics of the square tubes are as follows:

(1) No clear difference between column strength of seamless square tubes

and seamed square tubes may be found from the test data.

(2) Cross-sectional size appears to have little effect on column strength.

(3) The lower bound of the test results is a little higher than the ECCS column curve (a) except for $\bar{\lambda}$ = 0.6 to 0.7 .

6-4 COMPARISON BETWEEN THE COLUMN TEST RESULTS AND THE AVAILABLE MULTIPLE COLUMN CURVES

(1) The ECCS Multiple Column Curves

The ECCS multiple column curves have been categorized into five groups designated as (a_0), (a), (b), (c) and (d). A selection table for the column curves is shown in Table 6-8. Heavy or thick-walled sections have been defined by the ECCS with a flange thickness in excess of 40 mm, and high strength steel has been defined as that having a yield stress exceeding 430 N/mm^2.

Nine of the annealed welded box columns with high strength steel would be allocated to the ECCS column curve (a_0). The test points non-dimensionalized using both actual yield load $(P_y)_a$ and nominal yield load $(P_y)_n$ are a little above the ECCS column curve (a_0) except one test point. No column tests have been available which could be allocated to the ECCS column curve (d).

The following are the results of comparisons made between the tests and the ECCS column curves.

<u>ECCS Column Curve (a)</u> --- Figures 6-2(a) and 6-2(b) show 509 of the tests that would be compared with the ECCS column curve (a) using the selection table (Table 6-8). Figures 6-2(a) and 6-2(b) show the test results non-dimensionalized using nominal $(P_y)_n$ and actual $(P_y)_a$,

respectively. The crosses and arrows indicate M and (M-2S) of all the test points which fall into each 0.1 interval of nondimensional slenderness $\bar{\lambda} = 1/\pi \sqrt{\sigma_y/E}\ \ell/r$ (r = radius of gyration). The lower part of each figure indicates the coefficient of variation ω of the test points over each 0.1 interval of the abscissa. Symbols used for the test points are given in Table 6-9. The statistical results of the ratio of the test results to the ECCS column curve (a) are also given in Table 6-9. The statistical results are categorized into four groups according to cross sectional shape.

As shown in Fig. 6-2(a) and Table 6-9, the ECCS column curve (a) forms a lower envelope for all the test points when non-dimensionalized using $(P_y)_n$. The test results for circular tubes would increase the scatter because of large variations in the actual yield stress. Some 21.2 percent of the test points are less than the ECCS column curve (a) when non-dimensionalized using actual $(P_y)_a$.

ECCS Column Curve (b) --- Figures 6-3(a) and 6-3(b) show 689 of the tests that would be allocated to the ECCS column curve (b) non-dimensionalized using nominal and actual yield load, respectively. From a comparison between Fig. 6-3(a) and Table 6-9, only three test points from welded H-section columns are a little less than the ECCS column curve (b). The lower part in the test points may correspond to the welded H-sections and the welded box sections, and the upper part to the rolled H-sections. Some 10.9 percent of the test points are less than the ECCS column curve (b) when non-dimensionalized using actual $(P_y)_a$.

ECCS Column Curve (c) --- Figures 6-4(a) and 6-4(b) show 218 of the tests that would be allocated to the ECCS column curve (c). From a comparison between Fig. 6-4(a) and Table 6-9, eight test points for heavy rolled H sections and welded H-sections with universal mill plates are

considerably less than the ECCS column curve (c), while the other test points are above the ECCS column curve (c). Seventeen of the tests are less than the ECCS column curve (c) when non-dimensionalized using actual $(P_y)_a$.

It is obvious from the above comparison that the test points tend to fall largely above the relevant ECCS column curves when non-dimensionalized using the nominal properties. When the test results are non-dimensionalized using actual properties, they fall still higher with an increase in scatter results.

(M-2S) values of all the test points in a specified category over each 0.1 interval of $\bar{\lambda}$ may sometimes be less than the allocated ECCS column curve.

(2) The SSRC Multiple Column Curves

The SSRC multiple column curves have been categorized into three groups designated as (1),(2) and (3) in the third edition of the Guide (1976).

The following are results of comparisons between the tests and the appropriate SSRC column curves.

SSRC Column Curve (1) --- Figure 6-5 shows 274 of the tests that would be allocated to the SSRC column curve (1). The test results are non-dimensionalized using actual yield load. The statistical results of the ratio of the test results to the SSRC column curve (1) are summarized in Table 6-10. From a comparison between Fig. 6-5 and Table 6-10, the SSRC column curve (1) clearly represents the mean value of the test points for much of the range of $\bar{\lambda}$. Table 6-10 indicates that variation in cross-sectional profile appears to have little effect on the mean column strength. (M-2S) values of all the test points over each 0.1 interval of $\bar{\lambda}$

almost coincide with the minimum envelope of the test points.

SSRC Column Curve (2) --- Figure 6-6 shows 852 of the tests that would be allocated to the SSRC column curve (2). From a comparison between Fig. 6-6 and Table 6-10, the test results tend to lie above the SSRC column curve (2). The results from rolled H-sections and circular tubes are the highest of all the test points.

SSRC Column Curve (3) --- Figure 6-7 shows 127 of the tests that would be allocated to the SSRC column curve (3). From a comparison between Fig. 6-7 and Table 6-10, the results tend to lie above the SSRC column curve (3). Eight tests of heavy rolled H-section and welded H-section with universal mill plates tend to be lower than the SSRC column curve (3) for $\bar{\lambda}$ = 0.4 to 0.7 .

It is obvious from the above comparisons that the SSRC column curve (1) is close to the mean of the test data allocated to curve (1), and SSRC column curves (2) and (3) are ,however, close to the lower bound of the test data allocated to curves (2) and (3), respectively.

(3) The CRC Column Curve

Figure 6-8 shows the results of 1251 tests compared with the CRC basic column curve (48). The test results have been non-dimensionalized using actual $(P_y)_a$. The statistical results of the ratio of the test results to the CRC column curve is summarized in Table 6-11 for seven different types of cross section.

Figure 6-8 suggests that the CRC column curve provides a reasonable mean column strength curve over the whole slenderness range. The mean curve for the test points, which has been obtained from the means over each 0.1 interval of non-dimensional slenderness, shown by the crosses falls

remarkably close to the CRC column curve. Although the largest standard deviation of the ratio of the test results to the CRC column curve occurs for $\bar{\lambda} \fallingdotseq 1.0$, it is only slightly greater than the values for much of the range.

Taking results for different types of cross section, it is of interest to note from Table 6-11 that the lowest values for the ratio of the test results to the CRC column curve are obtained for profiles which in any application of the multiple-column-curve concept would be associated with the lower curves, that is, welded H-sections, welded boxes and tees.

6-5 PROPOSED MULTIPLE COLUMN CURVES FROM EXPERIMENTAL DATA-BASE APPROACH

In this section, the NDSS column data are used to see whether a statistically derived set of column curves would form a more appropriate basis for design. Test data non-dimensionalized by the actual yield stress has been used to produce design curves in terms of nominal yield stress; the mean ratio of $(\sigma_y)_a/(\sigma_y)_n$ is therefore a factor to be incorporated separately into the process of determining column resistance.

(1) New Category

Three column curves, groups (1), (2) and (3), are proposed with the definition of profile type as shown in the selection table, Table 6-8.

Group (1) --- Rolled H-sections, tubes and other sections which have been annealed or made of high strength steel are allocated to Group (1). Figure 6-9 shows 958 of the tests that would be allocated to this group.

In Fig. 6-9, the results in this group are plotted on normal probability paper to compare a normal model and the test data over each 0.1

interval of $\bar{\lambda}$. Since the slope of the test points is proportional to the standard deviation for each specified $\bar{\lambda}$, the variable conditional variance has to be defined against $\bar{\lambda}$-value.

A nonlinear regression analysis (42,43) with an assumed variable conditional variance is performed to obtain the mean curve for the test points. The Group (1) curve is divided into two parts with respect to $\bar{\lambda}$.

The mean curve of the test points is obtained by the least squares method as,

$$E(P/P_y \mid \bar{\lambda}) = 0.994+0.0928\bar{\lambda}-0.325\bar{\lambda}^2 \qquad (0.2\leq\bar{\lambda}\leq1.25)$$

$$E(P/P_y \mid \bar{\lambda}) = 0.942/\bar{\lambda}^2 \qquad (\bar{\lambda}\geq1.25) \qquad \text{--- (6-1)}$$

and the standard deviation function, which is a root of conditional variance, is :

$$S = \sqrt{Var(P/P_y \mid \bar{\lambda})} = 0.0789-0.00316/\bar{\lambda}^2 \qquad (0.2\leq\bar{\lambda}\leq1.25)$$

$$S = \sqrt{Var(P/P_y \mid \bar{\lambda})} = 0.120/\bar{\lambda}^2 \qquad (\bar{\lambda}\geq1.25) \qquad \text{--- (6-2)}$$

and $E(P/P_y \mid \bar{\lambda}) = 1.0$ and $Var(P/P_y \mid \bar{\lambda}) = 0$ are defined for the test points for $\bar{\lambda}\leq0.2$.

Figure 6-10(a) shows the test points compared with the Proposed formulas. Figure 6-10(b) shows both the mean curve and the curve corresponding to the mean minus two standard deviations compared with histograms of test data and the fitted normal distribution curves for 0.1 intervals of $\bar{\lambda}$. Cumulative probability of p = 10^{-8} to 10^{-2} is scaled on the normal distribution curves. The proposed column formulas give a good fit to the test points for $\bar{\lambda}\geq0.2$. $E(P/P_y \mid \bar{\lambda}) - 2\sqrt{Var(P/P_y \mid \bar{\lambda})}$ curve has a probability of 2.275 % of points falling below it assuming column strength obeys a normal distribution.

Table 6-12 summarizes the statistical results of the ratio of the test points to the mean function (Eq.(6-1)). The mean values are nearly equal to 1.0 even for different sections, and the variation of (M-2S) becomes small.

The variation of the test points over each 0.1 interval of $\bar{\lambda}$ is well fitted first to a Weibull and then to a normal distribution from the Kolmogorov-Smirnov fitness test.

Group (2) --- Welded H-sections, welded boxes and tees are allocated to this group. Figure 6-11 shows 250 of the tests that would be allocated to this group. A regression analysis is not performed for $\bar{\lambda} \geqq 1.25$ because of the lack of sufficient test data, and the SSRC column curve (2) is used in this region.

The mean curve and the standard deviation are obtained as

$$E(P/P_y \mid \bar{\lambda}) = 1.031 - 0.0969\bar{\lambda} - 0.292\bar{\lambda}^2 \qquad (0.2 \leqq \bar{\lambda} \leqq 1.25)$$

$$E(P/P_y \mid \bar{\lambda}) = SSRC\ column\ curve\ (2) \qquad (\bar{\lambda} \geqq 1.25) \qquad \text{--- (6-3)}$$

$$S = \sqrt{Var(P/P_y \mid \bar{\lambda})} = 0.0747 - 0.00299/\bar{\lambda}^2 \qquad (0.2 \leqq \bar{\lambda} \leqq 1.25)$$

$$S = \sqrt{Var(P/P_y \mid \bar{\lambda})^2} = 0.114/\bar{\lambda}^2 \qquad (\bar{\lambda} \geqq 1.25) \qquad \text{--- (6-4)}$$

Figure 6-11 shows the tests compared with the proposed formulas. The proposed formula agrees quite well with the statistical results. From Table 6-12, the differences between M and (M-2S) values for each type of section in the Group (2) are seen to be very small.

Group (3) --- Heavy rolled H-sections and welded H-sections with universal mill plates are allocated to this group. Figure 6-12 shows 41 of the associated tests. A regression analysis has not been performed due to insufficient test data, especially for $\bar{\lambda} \geqq 1.25$. Only the mean function was,

thus, estimated for $\bar{\lambda} \leq 1.25$. The SSRC column curve (3) is used for $\bar{\lambda} \geq 1.25$.

$$E(P/P_y \mid \bar{\lambda}) = 1.084 - 0.3898\bar{\lambda} - 0.144\bar{\lambda}^2 \qquad (0.2 \leq \bar{\lambda} \leq 1.25)$$

--- (6-5)

$$E(P/P_y \mid \bar{\lambda}) = SSRC\ column\ curve\ (3) \qquad (\bar{\lambda} \geq 1.25)$$

Eq. (6-5) are shown in Fig. 6-12 with the associated test results.

The Group (1), (2) and (3) curves are also expressed in terms of a unified column formula which was proposed for the ECCS column curves (49). Parameter α in Eq. (6-6) is selected through a regression analysis to obtain the mean curve of the test points assigned to each curve.

$$P/P_y = \{ [1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2] - \sqrt{[1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2]^2 - 4\bar{\lambda}^2} \} / 2\bar{\lambda}^2$$

--- (6-6)

where α denotes a statistically obtained parameter equalling 0.089, 0.224 and 0.432 for Group (1), (2) and (3), respectively. These curves are very close to Eqs. (6-1), (6-3) and (6-5).

6-6 CONCLUSIONS

The following main conclusions have been drawn from the results of the experimental data-base approach.

(1) A numerical data-base system has been developed for experiments on steel columns. Details of cross-sectional shapes, fabrication process, material properties, buckling axis, failure mode and others have been stored in the data-base for a total of 1665 column tests conducted in Europe, U.S.A. (mainly at Lehigh University) and Japan. Various computational and statistical manipulation programs are available to evaluate the test data.

(2) Welded H-sections possess less strength with increased scatter compared with rolled H-sections. For welded H-sections, the fabrication process (flame-cut, shear-cut or universal mill) along the flange tips appears to affect column strength.

(3) The ECCS and SSRC multiple column strength curves have been compared with the test data allocated to the individual curves, and statistical manipulations have been carried out.

(4) New multiple column strength formulas which explain more accurately the test data are proposed from the statistical results based on the experimental data-base.

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NOTATION

The following symbols are used in this chapter:

$(A)_a$	= measured area of cross section;
$(A)_n$	= nominal area of cross section;
b	= flange width;
E	= modulus of elasticity;
$(Exp)_a$	= $P/(P_y)_a$;
$(Exp)_n$	= $P/(P_y)_n$;
F	= frequency;
$(f_o)_{max}$	= maximum initial crookedness;
h	= depth of a section;
ℓ	= column length;
M	= mean value;
P	= maximum load;
P_y	= yield load;
$(P_y)_a$	= measured yield load;
$(P_y)_n$	= nominal yield load;
r	= radius of gyration of area;
S	= standard deviation;
t_f	= flange thickness;
t_w	= web thickness;
α	= parameter of the unified column formula;
$\bar{\lambda}$	= nondimensional slenderness;
σ_y	= yield stress;
$(\sigma_y)_{stub}$	= yield stress obtained from stub column test;
$(\sigma_y)_t$	= yield stress obtained from tensile coupon test; and
ω	= coefficient of variation.

Table 6-1 Number of Column Data

Type of Profile		ECCS	USA	JAPAN	TOTAL
	Rolled	502	55	87	644
H or I	Welded	22	31	229	282
	Other	–	5	4	9
Box	Welded	74	14	41	129
Square	Rolled	67	–	–	67
Tube	Welded	120	–	–	120
Circular	Rolled	99	–	4	103
Tube	Welded	40	–	145	185
Circular Solid		–	26	–	26
T Shape	Rolled	80	–	–	80
	Riveted	14	–	–	14
Composite		–	6	–	6
Total		1018	137	510	1665

Table 6-2 Nominal Cross-Sectional Dimensions in mm

H- and I-Shape Section					
Name of Section	Area	h	b	t_f	t_w
Rolled					
IAP 150	2042	150	75	8.50	5.30
IPE 160	2010	160	82	7.40	5.00
IPE 200	2850	200	100	8.50	5.60
IPN 160	2280	160	74	9.50	6.30
DIE 20	5703	190	197	11.00	7.00
DIR 20	13594	220	206	26.00	16.00
RSJ 5''X3''	1706	127	76.2	7.60	4.50
4WF13	2465	106	103	8.76	7.11
5WF18.5	3503	130	128	10.67	6.73
6WF15.5	2942	152	152	6.83	5.97
6WF25	4742	162	154	11.58	8.13
8WF24	4555	201	165	10.11	6.22
8WF31	5884	203	203	11.00	7.32
8WF67	12710	229	210	23.70	14.60
10WF66	12516	264	257	19.00	11.61
12WF50	9484	310	205	16.28	9.42
12WF65	12321	308	305	15.39	9.91
12WF92	17484	321	309	21.74	13.84
12WF120	22774	333	313	28.09	18.03
12WF161	30581	353	318	37.74	22.99
12WF190	36064	365	322	44.09	26.92
14WF78	14774	357	305	18.24	10.87
14WF111	21097	365	371	22.17	13.72
14WF142	26968	375	394	27.00	17.27
14WF228	43290	406	403	42.88	26.54
14WF320	60710	427	424	53.16	48.01
16WF88	16710	410	292	20.19	12.80
18WF105	19935	465	299	23.14	14.07
H-100X100X6X8	2190	100	100	8.00	6.00
H-125X125X6X9	3031	125	125	9.00	6.00
Welded					
I RECONSTITUE	5590	190	197	11.00	7.00
BUILT-UP WELDED	11613	254	229	19.05	12.70
6H27 (6''X7''H)	5323	178	152	12.70	9.52
6''X9''H	6677	229	152	0.0	0.0
10H61 (9''X10''H)	11613	267	229	19.05	12.70
12H79	15226	306	305	19.05	12.70
14H202	38064	394	394	38.10	25.40
H-15X15X3X3	117	15	15	3.00	3.00
H-100X100X6X8	2190	100	100	8.00	6.00
H-125X125X6X9	3031	125	125	9.00	6.00
H-200X200X8X10	5440	200	200	10.00	8.00
Other					
HYBRID	5323	178	152	12.70	9.52
8WF31 WITH COVER PLATE	9271	203	203	11.00	7.32
BUILT-UP RIVETED	12742	254	263	12.70	9.52

(unit in mm)

Table 6-2 Continued

Box Section

Name of Section	Area	b	t
Welded			
2 CORNIERES SOUDEES	982	50	5.00
CAISSON	5600	150	10.00
BOX 6''X6''	3710	152	6.35
BOX 10''X10''	12258	254	12.70
B-100X6	2256	100	6.00
B-250X10	9600	250	10.00
B-250X11	10516	250	11.00
Seamless Tube			
BS 15 SANS SOUDURE	2370	127	4.76
Seamed Tube			
BS 15 SOUDE 1329	1329	76.2	4.88
BS 15 SOUDE 1990	1990	88.9	6.35

Circular Tube Section

Name of Section	Area	D	t
Seamless			
TUBE SANS SOUDURE 1320	1320	88.90	5.00
TUBE SANS SOUDURE 2028	2028	88.90	8.00
TUBE SANS SOUDURE 1996	1996	121.00	5.50
O-101.6X3.5	1079	101.60	3.50
O-114.3X4.5	1552	114.30	4.50
Seamed			
BS 15 SOUDE 2154	2154	114.30	6.35
O-60.5X2.9	525	60.50	2.90
O-60.5X3.2	576	60.50	3.20
O-76.2X3.2	734	76.20	3.20
O-101.6X2.9	899	101.60	2.90
O-114.3X4.5	1552	114.30	4.50
O-139.8X4.5	1913	139.80	4.50

T-Shape Section

Name of Section	Area	h	b	t_f	t_w
TB 60	1700	60	120	10.00	10.00
1/2 IPN 200	1872	100	90	11.30	7.50
T 4''X3''	2102	76.2	102	12.70	12.70
2 CORNIERES RIVEES	1872	70.0	140	7.00	14.00

Circular Solid Shape

Name of Section	Area	D
DIA. 2 3/4''	3897	69.9
DIA. 7 1/2''	28503	191

Table 6-3 Comparison of Test Results for Rolled H-Columns and ECCS Column Curve

Type of Profile	Fab. Cond.	$Exp)_n$/ ECCS (b) N	M	M-2S	$Exp)_a$/ ECCS (b) N	M	M-2S
DIE20	DYL	8	1.230	1.066	8	1.136	0.995
	AYL	2	1.344	1.332	2	1.285	1.261
DIR20	DYL	8	1.139	1.032	8	1.137	1.013
	AYL	3	1.417	1.143	2	1.515	1.343
H-100x100	DXL	5	1.225	0.972	5	1.053	0.835
x6x8	DYL	72	1.700	1.256	72	1.383	1.087
H-125x125	DXL	5	1.682	1.370	5	1.272	1.046
x6x9	DYL	5	1.686	1.489	5	1.193	1.063
IAP150	DYL	215	1.591	1.211	215	1.212	0.909
IAP160	DYL	209	1.557	1.092	204	1.163	0.938
IPE200	DYL	8	1.444	1.014	8	1.261	1.003
	AYL	3	1.420	0.828	3	1.223	0.718
IPN160	DYL	36	1.384	1.050	36	1.238	0.954
RSJ5''x3''	DYL	10	1.652	1.371	10	1.238	1.023
10WF66	DXL	1	1.664	–	1	1.278	–
12WF50	DYL	1	1.505	–	1	1.150	–
	DYM	1	1.360	–	1	1.146	–
12WF65	DYL	1	1.245	–	1	1.123	–
	DYM	1	1.293	–	1	1.138	–
12WF161	DYL	7	0.645	0.507	7	0.843	0.665
12WF120	DYH	2	1.134	0.830	2	1.140	0.834
14WF111	DYL	2	1.051	0.811	2	0.977	0.754
4WF13	DXL	2	1.576	1.337	2	1.307	1.108
5WF18.5	DYL	2	1.308	1.268	2	1.176	1.140
	CYL	1	1.548	–	1	1.392	–
6WF15.5	DYL	1	1.294	–	1	1.163	–
8WF31	DXL	3	1.182	1.082	3	1.053	0.932
	DYL	3	1.202	1.007	3	1.062	0.922
	DYM	2	1.286	0.959	2	1.140	0.850
	DYH	1	1.241	–	1	1.248	–
	AYL	2	1.181	0.972	2	1.241	1.021
8WF24	DYL	1	1.112	–	1	1.000	–
8WF67	DYL	1	1.136	–	1	1.193	–
Total		618	1.540	1.037	618	1.210	0.900

Refs. 1),2),7),8),9),12),14),17),20),21) and 25)
N = number of data, M = mean, S = standard deviation
D = as-delivered, A = annealed, C = cold-straightened
Y = minor axis, X = major axis
L = σ_y < 250 N/mm^2, M = 250 N/mm^2 < σ_y < 400 N/mm^2, H = σ_y > 400 N/mm^2

Table 6-4 Comparison of Test Results for Welded H-Columns and ECCS Column Curve

Type of Profile	Fab. Cond.	$Exp)_n$/ ECCS (b) N	M	M-2S	$Exp)_e$/ ECCS (b) N	M	M-2S
Built-up	UDXL	1	0.881	–	1	0.847	–
Welded	UDYL	2	0.780	0.685	2	0.800	0.667
H-100x100	FDYL	168	1.100	0.748	168	0.860	0.585
x6x8	FEYL	1	1.253	–	1	1.138	–
H-125x125	FDYL	5	1.102	0.856	5	0.987	0.786
x6x9	FDXL	5	1.172	1.119	5	1.065	1.039
	UAYL	5	1.554	1.127	5	1.232	0.880
	UAXL	4	1.457	1.161	4	1.182	0.962
	UEYL	5	1.229	0.890	5	0.954	0.688
	UEXL	5	1.225	0.993	5	0.970	0.804
H-200x200	FDYL	30	1.853	1.480	30	1.150	0.918
x8x10							
I-Recon-	FDYL	16	1.119	0.922	16	0.962	0.783
stitue	FAYL	6	1.199	1.075	6	1.284	1.048
10H61	FDYH	2	1.481	1.010	2	1.264	0.862
	FDYL	3	0.833	0.717	0	–	–
12H79	FDYL	3	1.163	0.904	3	1.057	0.822
14H202	FDYL	3	1.018	0.980	3	1.032	0.994
6H27	FDYH	2	1.349	1.079	2	1.224	0.979
	FDXH	1	1.079	–	1	0.980	–
	SDYH	2	1.346	1.211	2	1.209	1.088
	SDXH	1	1.077	–	1	0.997	–
	UDYL	2	1.291	1.110	2	0.835	0.718
6''x9''H	FEYM	3	0.940	0.732	0	–	–
Total		275	1.207	0.613	269	0.944	0.581
		(101	1.374	0.644)	(101	1.083	0.767)

Refs. 1),8),10),13),14),18),21),26) and 29)
F = flame-cut, U = universal mill, S = shear-cut
E = plate annealed

Table 6-5 Comparison of Test Results for Welded Box Columns and ECCS Column Curve

Type of Profile	Fab. Cond.	$Exp)_n$/ ECCS (b) N	M	M-2S	$Exp)_a$/ ECCS (b) N	M	M-2S
B-100x6	FDL	20	1.918	1.639	20	1.292	1.190
Caisson	FDL	16	1.252	1.061	16	1.060	0.903
150x10	FAL	6	1.357	1.274	6	1.358	1.166
2 Cornieres Soudees 50x5	FDL	52	1.328	1.090	52	1.059	0.864
B-250x11	FDH	6	1.297	1.092	6	1.187	1.057
	FAH	6	1.400	1.205	6	1.273	1.055
B-250x10	FDH	6	1.272	0.933	6	1.129	0.910
	FAH	3	1.213	0.825	3	1.229	0.949
B-10''x10''	FDH	2	1.305	0.917	2	1.222	0.858
254x10	UDL	8	1.090	0.998	8	0.979	0.910
B-6''x6''	FDH	2	1.465	1.257	2	1.276	1.095
152x6.4	UDL	2	1.632	1.461	2	0.979	0.877
Total		129	1.400	0.877	129	1.147	0.817

Refs. 1),10),14),19) and 27)

Table 6-6 Comparison of Test Results for Circular Tube Columns and ECCS Column Curve

Type of Profile	Fab. Cond.	$Exp)_n$/ ECCS (b) N	M	M-2S	$Exp)_a$/ ECCS (b) N	M	M-2S
Tube Sans Soudure 88.9x8	RDL	10	1.706	1.370	10	1.316	1.065
Tube Sans Soudure 121x5.5	RDL	29	1.516	1.266	0	–	–
Tube Sans Soudure 88.9x5	RDL	60	1.248	1.114	0	–	–
O-101.6x3.5	RDM	2	1.992	1.336	2	1.059	0.681
O-114.3x4.5	RDM	2	1.477	1.314	2	0.983	0.859
	WDL	90	1.655	1.318	90	1.202	0.957
BS 15 Soude 114.3x3.5	WDL	40	1.459	0.926	35	1.165	0.897
O-76.2x3.2	WDL	4	1.487	1.368	4	1.060	0.975
O-139.8x4.5	WDH	6	1.352	1.274	6	1.031	0.971
O-101.6x2.9	WDM	13	1.394	1.246	13	1.015	0.906
	WAM	4	1.733	1.345	4	1.269	0.993
O-60.5x3.2	WDM	13	1.823	1.159	13	1.366	0.868
O-60.5x2.9	WDM	15	2.603	2.169	15	1.455	1.226
Total		288	1.569	0.879	194	1.208	0.871

Refs. 1),22),24),25),28) and 30)
R = rolled, W = welded

Table 6-7 Comparison of Test Results for Square Tube Columns and ECCS Column Curve

Type of Profile	Fab. Cond.	$Exp)_n$/ ECCS (b) N	M	M-2S	$Exp)_a$/ ECCS (b) N	M	M-2S
Tube Sans Soudure 127x4.76	RDL	67	1.461	1.092	22	1.176	0.955
BS 15 Soude 88.9x6.35	WDL	40	1.434	1.065	34	1.179	0.862
BS 15 Soude 76.2x4.88	WDL	80	1.359	1.143	28	1.199	0.992
Total		187	1.412	1.092	84	1.185	0.927

Ref. 1)

Table 6-8 Selection Table for Column Design

Selection Parameter				ECCS DL	ECCS DH	ECCS AL	ECCS AH	SSRC DL*	SSRC DH**	SSRC AL*	SSRC AH**	Proposed DL	Proposed DH	Proposed AL	Proposed AH
Rolled H	Light	major	h/b > 1.2	a	a	a	a_0	2	1	1	1	1	1	1	1
			h/b ≤ 1.2	b											
		minor	h/b > 1.2	b	b	b	a	2	1	1	1	1	1	1	1
			h/b ≤ 1.2	c											
	Heavy			d				3	1	1	1	3	2	1	1
Welded H	Light	flame-cut	major	b	a	a	a_0	2	1	1	1	2	2	1	1
			minor						2						
		universal mill	major	b	a	b	a	3	1	1	1	3	3	1	1
			minor	c	b				2						
	Heavy	flame-cut	major	c				2	1	1	1	2	2	1	1
			minor						2						
		universal mill	major	c				3	1	1	1	3	2	1	1
			minor	d					2						
Circular and Square Tubes	Massive Solid							2	1	1	1	1	1	1	1
		rolled		a	a	a	a	1	1	1	1	1	1	1	1
		welded		a				2	1	1	1				
T Shape				c								2			
Welded Box		flame-cut		b	a	a	a_0	2	1	1	1	2	1	1	1
		universal mill						2				3			

D = as-delivered, A = annealed, L = σ_y < 400 N/mm^2, H = σ_y > 400 N/mm^2

L* = A7, A36 steel, H** = A514 steel

Table 6-9 Comparison of Test Results and ECCS Multiple Column Curves

Type of Profile	Symb.	Exp)$_n$/ ECCS (a) in Subgroup ECCS (a)					Exp)$_n$/ ECCS (a) in Subgroup ECCS (a)				
		N	M	S	ω	M-2S	N	M	S	ω	M-2S
Welded H	▲	12	1.242	0.138	0.111	0.965	12	1.068	0.0919	0.0861	0.884
Welded Box	■	22	1.196	0.0842	0.0704	1.028	22	1.112	0.0984	0.0885	0.915
Circular Box	○	288	1.431	0.302	0.211	0.828	194	1.104	0.135	0.122	0.834
Square Tube	□	187	1.283	0.134	0.105	1.014	84	1.076	0.107	0.0990	0.863
Total		509	1.362	0.256	0.188	0.851	312	1.104	0.135	0.122	0.834
Type of Profile	**Symb.**	**Exp)$_n$/ ECCS (b) in Subgroup ECCS (b)**					**Exp)$_n$/ ECCS (b) in Subgroup ECCS (b)**				
		N	M	S	ω	M-2S	N	M	S	ω	M-2S
Rolled H	△	510	1.547	0.220	0.142	1.106	504	1.195	0.138	0.116	0.919
Welded H	▲	81	1.433	0.370	0.258	0.693	81	1.091	0.147	0.135	0.797
Welded Box	■	98	1.423	0.288	0.202	0.847	98	1.119	0.167	0.149	0.785
Total		689	1.516	0.257	0.170	1.001	683	1.172	0.149	0.127	0.874
Type of Profile	**Symb.**	**Exp)$_n$/ ECCS (c) in Subgroup ECCS (c)**					**Exp)$_n$/ ECCS (c) in Subgroup ECCS (c)**				
		N	M	S	ω	M-2S	N	M	S	ω	M-2S
Rolled H	△	114	1.655	0.440	0.241	0.856	114	1.399	0.231	0.165	0.937
Welded H	▲	10	1.295	0.316	0.244	0.663	10	1.018	0.215	0.211	0.588
T Shape	T	94	1.523	0.331	0.218	0.861	94	1.236	0.189	0.153	0.858
Total		218	1.582	0.377	0.239	0.827	218	1.311	0.236	0.180	0.840

Table 6-10 Comparison of Test Results and SSRC Multiple Column Curves

Type of Profile	Symbol	$Exp)_a$/ SSRC (1) in Subgroup SSRC (1)		
		N	M	M-2S
Rolled H	△	13	1.079	0.806
Welded H	▲	20	1.054	0.833
Welded Box	■	31	1.037	0.888
Circular Box	○	100	1.028	0.716
Square Tube	□	84	0.998	0.806
Circular Solid	●	26	0.997	0.818
Total		274	1.021	0.775

Type of Profile	Symbol	$Exp)_a$/ SSRC (2) in Subgroup SSRC (2)		
		N	M	M-2S
Rolled H	△	590	1.198	0.870
Welded H	▲	68	1.084	0.820
Welded Box	■	98	1.121	0.782
Circular Tube	○	94	1.192	0.972
Total		850	1.179	0.857

Type of Profile	Symbol	$Exp)_a$/ SSRC (3) in Subgroup SSRC (3)		
		N	M	M-2S
Rolled H	△	15	1.206	0.682
Welded H	▲	16	1.132	0.724
T Shape	T	94	1.379	0.943
Total		125	1.327	0.848

Table 6-11 Comparison of Test Results and CRC Column Curve

Type of Profile	Symbol	$Exp)_a$/ CRC		
		N	M	ω
Rolled H	△	618	0.988	0.114
Welded H	▲	101	0.910	0.130
Welded Box	■	129	0.953	0.106
Circular Box	○	194	1.020	0.104
Square Tube	□	84	0.990	0.102
Circular Solid	●	26	0.992	0.083
T Shape	T	94	0.920	0.120
Total		1251	0.978	0.117

Table 6-12 Comparison of Test Results and Proposed Multiple Column Curves

Type of Profile	Symbol	Exp)$_a$/ Proposed (1) in Subgroup Proposed (1)		
		N	M	M-2S
Rolled H	△	603	0.998	0.791
Welded H	▲	20	1.033	0.834
Welded Box	■	31	1.015	0.868
Circular Box	○	194	1.014	0.780
Square Tube	□	84	0.975	0.784
Circular Solid	●	26	0.980	0.804
Total		958	1.000	0.890

Type of Profile	Symbol	Exp)$_a$/ Proposed (2) in Subgroup Proposed (2)		
		N	M	M-2S
Welded H	▲	68	1.043	0.764
Welded Box	■	88	1.086	0.760
T Shape	T	94	1.090	0.731
Total		250	1.076	0.747

Type of Profile	Symbol	Exp)$_a$/ Proposed (3) in Subgroup Proposed (3)		
		N	M	M-2S
Rolled H	△	15	1.065	0.664
Welded H	▲	16	0.997	0.683
Welded Box	■	10	1.046	0.906
Total		41	1.034	0.716

Fig. 6-1(a) Histogram of Yield Stress by Stub Column Test

Fig. 6-1(b) Histogram of Yield Stress by Tensile Coupon Test

Fig. 6-1(c) Histogram of Initial Crookedness of IPE160 Column

Fig. 6-2(a) Test Results Compared with ECCS Curve (a)

(non-dimensionalized using $(P_y)_n$)

Fig. 6-2(b) Test Results Compared with ECCS Curve (a)

(non-dimensionalized using $(P_y)_a$)

Fig. 6-3(a) Test Results Compared with ECCS Curve (b)

(non-dimensionalized using $(P_y)_n$)

Fig. 6-3(b) Test Results Compared with ECCS Curve (b)

(non-dimensionalized using $(P_y)_a$)

Fig. 6-4(a) Test Results Compared with ECCS Curve (c)

(non-dimensionalized using $(P_y)_n$)

Fig. 6-4(b) Test Results Compared with ECCS Curve (c)

(non-dimensionalized using $(P_y)_a$)

Fig. 6-5 Test Results Compared with SSRC Curve (1)

Fig. 6-6 Test Results Compared with SSRC Curve (2)

Fig. 6-7 Test Results Compared with SSRC Curve (3)

Fig. 6-8 Test Results Compared with CRC Curve

Fig. 6-9 Experimental Column Data on Normal Probability Paper

Fig. 6-10(a) Test Data and Group (1)

Fig. 6-10(b) Summary of Test Data and Group (1)

Fig. 6-11 Test Data and Group (2)

Fig. 6-12 Test Data and Group (3)

Chapter 7. EVALUATION OF LATERAL-TORSIONAL BUCKLING STRENGTH OF STEEL BEAMS

7-1 INTRODUCTION

In Chapter 6, a total of 1665 individual column test results was surveyed and stored on the Numerical Data-Base for the Ultimate Strength of the Steel Structures (NDSS). Data editing, searching, retrieval and management have been carried out on steel columns, and after various computational and statistical manipulations, evaluation of the information on steel column tests were summarized.

In Refs. (39) and (40), Fukumoto and Kubo have made statistical assessments of early lateral-torsional buckling tests and compared them with the ECCS strength formula

$$M_u/M_p = [1 + (M_p/M_E)^{2n}]^{-1/n} \qquad \text{---(7-1)}$$

in which M_p is the fully plastic moment for the cross section and M_E the elastic critical moment for the beam. They suggested that the system factor n as follows:

Rolled Beams : n=2.5 for mean strengths (M)

n=1.5 for mean minus two standard deviations (M-2S)

Welded Beams : n=2.0 for M

n=1.0 for M-2S

It was also shown that the (M-2S) values are quite close to the lower bound for the test results surveyed.

Recent experiments of N=335 of Refs. (29)-(31) have been added to Ref. (39) and then a total of 544 individual test results were stored in the

NDSS system. This chapter describes some examples of the use of the experimental data-base on the lateral torsional buckling of beams. Statistical results are compared with available beam strength formulas. A unified formula which was proposed for the ECCS multiple column curves (43) is obtained empirically for the beam strengths from the results of the beam tests surveyed.

The author also proposes new multiple beam formulas which explain more accurately the test data from a statistical point of view.

7-2 BEAM TEST DATA STORED ON THE NUMERICAL DATA-BASE

(1) Number of Beam Tests Surveyed

A total of 544 individual test results is categorized by fabrication process, cross-sectional type and country of origin as given in Table 7-1. In Table 7-1, 421 test results relate to Japan, 49 to U.K., 33 to U.S.A., 27 to West Germany and 15 to Australia. The original papers, in which the data were presented, are listed in references (1) to (28). Tables 7-2(a), 7-2(b) and 7-2(c) summarized the test programs stored on the NDSS system for rolled H, welded H and welded plate girders, respectively. In this chapter plate girders indicate welded beams with vertical stiffeners or both vertical and horizontal stiffeners on the intermediate of span. Sixty-four percent of tests surveyed here were performed on rolled beams which were comparatively small sections.

In Tables 7-2, (1) to (6) and (11) to (14), (15) in the column the Boundary and Loading Condition are of laterally continuous beams with intermediate braced points (marked ×), (7), (9) and (10) are of unbraced simply supported beams, (8) of braced beams with heavy box section loading beams.

From a deterministic approach, a small number of beams was tested in each institute except the author's tests in Chapters 2 and 3. A hundred and forty-three beams were tested from a standpoint of statistical considerations by the author. These studies were already explained in Chapters 2 and 3.

The reference papers for which it was impossible to pick up test results from small graphs, or the papers where the interpretation of tests was not easy because of unknown bracing conditions, are not listed in these tables.

(2) **Number of Beam Tests Surveyed**

The test data on each specimen consist of the following items: (a) maximum moment, M_u, (b) fully plastic moment, M_p, (c) yield moment, M_y, and (d) elastic critical moment, M_E. The following test information which belongs to a individual beam is also stored in data-base files with items (a) to (d) above.

1) Type of profile : monosymmetric cross section, double symmetric cross section, with or without vertical stiffener, with or without horizontal stiffener;

   name of profile --- 250UB37.2, IPE200, etc. (58 names).

2) Geometry : measured and nominal cross-sectional dimensions such as beam height h, flange width b, flange thickness t_f and web thickness t_w.

3) Fabrication process : during assembly --- rolled and welded;

   after assembly --- as-delivered, annealed.

4) Yield stress : measured and nominal yield stress.

5) Loading and boundary conditions.

6) Initial imperfections : residual stresses and initial crookednesses.

7) Source reference : original paper,country in which testing was conducted.

8) Identification : beam test index, sequential number.

The elastic critical moment, M_E, of the laterally supported beam is calculated by the method of Ref. (46) when exact M_E is not available. Load-deflection curves, load-strain curves, material properties and initial imperfections which were obtained by the author's tests (N = 164) are completely stored in data-base files.

(3) Cross-Sectional Dimensions and Yield Stress

Cross-Sectional Dimensions --- Nominally cross-sectional dimensions are shown in Table 7-2(a), (b) and (c).

Statistical data of $(h/b)\sqrt{\sigma_y/235}$, $(L/b)\sqrt{\sigma_y/235}$, $(h_w/t_w)\sqrt{\sigma_y/235}$ and $(b/t_f)\sqrt{\sigma_y/235}$ are shown in Table 7-3 in which σ_y = yield stress of individual test specimens.

Figures 7-1(a) and (b) show histograms of $(b/t_f)\sqrt{\sigma_y/235}$ and $(h_w/t_w)\sqrt{\sigma_y/235}$ of the rolled beams, respectively. Histograms for the welded beams and the plate girders are also shown in Figs. 7-1(c) to (f). The minimum value of $(h_w/t_w)\sqrt{\sigma_y/235}$ of the welded beams is about 86 except two test beams which had high strength steel of 687 N/mm^2 on yield stress, while the minimum value of plate girders is about 77. The welded beams and plate girders are roughly divided into two groups by the value of $(h_w/t_w)\sqrt{\sigma_y/235}$.

Yield Stress --- Figure 7-2(a) represents a histogram of measured yield stresses for 544 test beams. As shown in Fig. 7-2(a), 88 % of the test data are of ordinary steel and 12 % of high strength steel with $\sigma_y \leq$ 430 N/mm^2. σ_y = 690 N/mm^2 is the highest nominal yield stress among the test results. Figure 7-2(b) shows measured yield stresses

nondimensionalized by nominal values.

(4) Initial Imperfections

Initial Crookednesses --- Measured values of initial crookednesses were available in references for about 57 % of the test specimens. Histograms of initial out-of-crookednesses about the minor axis on 75 rolled beams and 68 welded beams are shown in Fig. 7-3(a), which are measured results of the author's test beams.

Residual Stress --- In a few test beams residual stresses were available in references. Residual stress distributions of the author's test beams are shown in Fig. 7-3(b).

7-3 STATISTICAL ASSESSMENTS OF BEAM TEST RESULTS

(1) Rolled Beams

In order to present test data in the nondimensional from, the following two beam moments are used for comparisons: (i) the plastic moment, M_p and (ii) the yield moment, M_y. M_y and M_p are calculated by using measured yield stresses and measured cross-sectional dimensions when they are available in reference sources.

Figure 7-4(a) shows results of 348 rolled beam tests when nondimensionalized using actual M_p. Crosses (+) and arrows (↓) indicate the mean M and the mean minus two standard deviations (M-2S) of the test points over 0.1 interval of $\bar{\lambda}$. Symbols (●) of lower part in the figure indicate the coefficient of variation ω of the test points over each 0.1 interval of $\bar{\lambda}$. The test results are compared with the ECCS beam curves, Eq. (7-1), for n = 2.5, 2.0 and 1.5. Nine test results of annealed beams which are plotted by the symbol (●), are above the mean strength values shown by crosses.

As shown in Fig. 7-4(a), the ECCS curve for n = 2.5 provides a reasonable mean strength shown by crosses over short and medium slenderness ranges. The curve for n = 1.5 provides the (M-2S) strength curve shown by arrows and also forms a lower bound for the test points.

Similarly, Fig. 7-4(b) show results of 348 rolled beam tests when nondimensionalized using actual M_y. The test results are compared with the following equation for n = 2.5, 2.0 and 1.5.

$$M_u/M_y = \{ 1 + (\sqrt{M_y/M_E})^2 \}^{-1/n} \qquad \text{---(7-2)}$$

Eq. (7-2) is used M_y instead of M_p in the ECCS beam curve. The test results are above M_u/M_y = 1.0 over small $\bar{\lambda}$ values. This indicates that surveyed rolled sections can deliver the fully plastic moment in-plane due to bending.

Statistical results, that is, the mean and standard deviation of the ratio of the test results to Eq. (7-1) and Eq. (7-2), $(Exp)_a$/ECCS(n), for n= 2.5 and 2.0 are summarized in Table 7-4.

For each group of the Boundary and Loading Condition the ratios of the test results to Eq. (7-1), $(Exp)_a$/ECCS(n), for n=2.5 are summarized in Table 7-5. There are comparatively large difference between the ratios of each group. This may be partly due to the limit number and the local scatterness of test points for a categorized Boundary and Loading Conditions shown in Table 7-5.

(2) Welded Beams

Figures 7-5(a) and 7-5(b) show results of 154 welded beam tests when nondimensionalized using actual M_p and M_y, respectively. Twelve test results of annealed welded beams which are plotted by symbol (▲) in Fig. 7-5(a) possess relatively high strength except one point. The test results

are compared with the ECCS beam curves for n = 2.5, 2.0, 1.5 and 1.0 in Fig. 7-5(a). The ECCS curve for n = 2.0 provided a reasonable mean strength points shown by crosses over short and medium slenderness ranges. The curve for n = 1.0 provides the (M-2S) strength curve shown by arrows and also forms a lower bound for the test points. Similarly, in Fig. 7-5(b) the test results are compared with Eq. (7-2) for n = 2.5, 2.0 and 1.5. The welded beams over the short slenderness range have also enough strength of M_p as rolled beams.

Coefficients of variation shown in lower parts of Fig. 7-5(a) and 7-5(b) are from 0.056 to 0.140 and from 0.027 to 0.170, respectively. The largest coefficient of variation for beam strength over 0.1 interval of the nondimensional slenderness ratio $\bar{\lambda}$ occurs for $\bar{\lambda}$ = 1.0 ~ 1.1. Beam strengths nondimensionalized by M_p tend to have lower variation than that nondimensionalized by M_y. Statistical results of the ratio of the test results for welded beams to Eq. (7-1) and Eq. (7-2) are also summarized in Table 7-4. Mean strengths of welded beams are a little lower than those of rolled beams. The coefficient of variation is large in welded beams than that in rolled beams. Large variation of the ultimate strength M_u/M_p or M_u/M_y in the welded beams may be due to the larger variation in compressive residual stresses and initial lateral crookednesses (24).

(3) **Plate Girders**

Figures 7-6(a) and 7-6(b) show results of 42 plate girders when nondimensionalized using M_p and M_y, respectively. Eight plate girders with vertical and horizontal stiffener and 18 plate girders with monosymmetric cross section are contained in these results. The width-thickness of plate girder webs varies from 77 to 371 and the girder height-compression flange width ratio from 3.4 to 22.8. These ratios indicate a possibility of web

buckling due to bending and that would cause the early failure of the compression flange lateral instability.

From a comparison of Figs. 7-6(a) and 7-6(b), the strength variation of plate girders becomes less when using the yield moment M_y instead of M_p. From Fig. 7-6(b), the ECCS curve for n = 1.5 using M_y instead of M_p provides a reasonable mean strength shown by crosses in the medium slenderness range. The curve for n = 1.0 provides in the (M-2S) strength shown by arrows.

Basler and Thürlimann (48) proposed the ultimate strength of plate girder in bending without flange instability.

$$\frac{M_u}{M_y} = 1-0.0005\frac{A_w}{A_f}\left(\frac{h}{t_w}-5.7\sqrt{\frac{E}{\sigma_y}}\right) \qquad \text{---(7-3)}$$

where A_w = area of web and A_f = area of compression flange.

In the test plate girders without horizontal stiffener (N=34) the 14 plate girders of Refs. (1), (14) and (15) have strengths of $M_u/M_y < 1.0$ from Eq. (7-3). Especially, the three plate girders of Refs. (1) and (10) have strengths of $M_u/M_y < 0.9$.

Six test results scattered in the range of $0.6 < \bar{\lambda} < 0.9$ have lower strengths. These test results were picked up from Refs. (6), (8) and (28).

Further investigation would be needed for the interaction problem of the post-buckling web plate behavior and over all lateral instability of girders.

7-4 COMPARISON BETWEEN THE BEAM TEST RESULTS AND SOME AVAILABLE BEAM CURVES

(1) ECCS Curves

Figure 7-7(a) shows the ECCS curves of Eq. (7-1) from n = 1.0 to n =

3.5. The strength curve changes larger in the range of small system factor.

Figures 7-4(a), 7-5(a) and 7-6(a) show results of comparisons between the test results and the ECCS beam curves. Statistical results of the ratio of the test results to the ECCS beam curves were shown in Table 7-4. According to these figures and Table 7-4, the ECCS beam curve for n = 2.5 provides a reasonable mean strength of rolled and welded beams for the range $\bar{\lambda} < 1.0$. However, for $\bar{\lambda} > 1.0$, the ECCS beam curve for n = 2.5 underestimates the mean strengths of the test data.

The ECCS beam function, Eq. (7-1), was applied to express the mean strength of the test beams. Results of regression analysis using the function of Eq. (7-1) are shown in Figs. 7-7(b), (c) and (d). The system factors of n = 2.892 and 2.539 are obtained for rolled and welded beams, respectively, when the test points are nondimensionalized by the actual plastic moment. n = 1.71 is also obtained for the plate girders when the test points are nondimensionalized by actual yield moments.

(2) **Beam Curve Specified JRA**

In Fig. 7-8, the test results for rolled and welded beams including plate girders are plotted in the σ_u/σ_y versus α, Eq. (7-5), coordinates. The test points are compared with design formulas, Eq. (7-4), specified in Japanese Highway Bridge Specification (JRA).

$$\sigma_u/\sigma_y = \begin{cases} 1.0 & (\alpha \leq 0.2) \\ \beta\{1.0-0.412(\alpha-0.2)\} \leq 1.0 & (\alpha > 0.2) \end{cases} \qquad \text{---(7-4)}$$

in which

$$K = \begin{cases} 2 & (A_w/A_c \leq 2) \\ \sqrt{3+A_w/2A_c} & (A_w/A_c > 2) \end{cases}$$

$$\beta = \frac{1}{0.6+0.4k} \leq 2.5, \qquad \text{---(7-5)}$$

$$\alpha = \frac{2}{\pi} K \left(\frac{L}{b}\right) \sqrt{\frac{\sigma_y}{E}}$$

where A_w and A_c = areas of web and compressive flange, respectively, L = distance between points of bracing of compression flange, b = flange width in compression, σ_y = yield stress, σ_u = maximum stress, E = modulus of elasticity, k = M_2/M_1 , M_2 and M_1 = end-moments at bracing points.

In Fig. 7-8, test points of beams with K ≠ 1 and $L/b\sqrt{\sigma_y/235} > 3.0$ are picked off. Coefficients of variation, ω, are also shown at the lower part of Fig. 7-8.

(3) ECCS Unified Column Function

The ECCS multiple column curves have been categorized into five group designated as (α_0), (a), (b), (c) and (d). These curves are also expressed in terms of a following unified column formula (43).

$$\sigma_{cr}/\sigma_y = [1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2-\sqrt{\{1+\alpha(\bar{\lambda}-0.2)+\bar{\lambda}^2\}^2-4\bar{\lambda}}\,]/2\bar{\lambda}^2 \qquad \text{---(7-6)}$$

Figure 7-9(a) shows the ECCS unified column functions of Eq. (7-6) from α = 0.05 to 0.550

Parameter α in Eq. (7-6) has been selected through a regression analysis to obtain fitted curves of the numerically obtained discrete points. The above function, Eq. (7-6), was applied here to express mean strengths of the test beams. Statistically obtained parameters α are equal to 0.077 and 0.104 for the rolled and welded beams, as shown in Figs. 7-9(b) and (c), respectively, when the test points are nondimensionalized by actual plastic moments. α = 0.373 is also obtained for the plate girders

as shown in Fig. 7-9(d) when the test points are nondimensionalized by actual plastic and yield moments.

From Figs. 7-9(b) and 7-9(c), mean test points can be well expressed by a unified column formula, Eq. (7-6), when parameters α are properly selected for the rolled and welded beams, respectively.

7-5 PROPOSED BEAM CURVES FROM EXPERIMENTAL DATA-BASE APPROACH

In this section, the NDSS beam data are used to see whether a statistically derived set of beam curves would form a more appropriate basis for design. Test data nondimensionalized by actual yield stresses have been used to produce design curves in terms of nominal yield stress, the mean ratio of $(\sigma_y)_a/(\sigma_y)_n$ is therefore a factor to be incorporated separately into the process of determining beam resistance.

(1) Rolled Beams

As shown in the previous section, rolled beams have enough strength of M_p in the short slenderness range and it is clear that M_p may be suitable as the nondimensional factor. The $M_u/M_p \sim \sqrt{M_p/M_E}$ coordinates are adopted for rolled beams in this section. The strength of rolled beams on the very long slenderness range considering residual stresses may have the elastic critical strength, M_E. Assuming that the ultimate beam strength M_u in the inelastic range is equal to the plastic moment M_p for $\bar{\lambda} \leq C_1$ and for longer slenderness range M_u beams M_E for $\bar{\lambda} \geq C_2$ on the $M_u/M_p \sim \bar{\lambda}(=\sqrt{M_p/M_E})$ coordinates, a parabola function was used to represent the inelastic mean strength curve as follow:

$$E[M_u/M_p \mid \bar{\lambda}]$$

$$= \begin{cases} 1.0 & (\bar{\lambda} \leq C_1) \\ C_R \{ (\bar{\lambda}-C_1)^2 - (C_2-C_1)(\bar{\lambda}-C_1) \} + \dfrac{1-C_2^2}{C_2^2(C_2-C_1)}(\bar{\lambda}-C_1)+1 & (C_1 < \bar{\lambda} \leq C_2) \\ \dfrac{1}{\bar{\lambda}^2} & (\bar{\lambda} > C_2) \end{cases} \quad \text{---(7-7)}$$

Two hyperbola functions for the variable conditional variance are assumed

$$\sqrt{Var\,[M_u/M_p \mid \bar{\lambda}]}$$

$$= \begin{cases} 0 & (\bar{\lambda} \leq C_3) \\ S_R(-\dfrac{1}{\bar{\lambda}^2}+\dfrac{1}{C_3^2}) & (C_3 < \bar{\lambda} \leq C_4) \\ S_R\{-C_4+(\dfrac{C_4}{C_3})^2/\bar{\lambda}^2\} & (\bar{\lambda} > C_4) \end{cases} \quad \text{---(7-8)}$$

Values of terminal points C_1=0.4, $C_2=\sqrt{2}$, C_3=0.2 and C_4=1.05 where the curve is divided are chosen based on the test data. From a nonlinear regression analysis with an assumed variable conditional variance, the parameters C_R= -0.306 and S_R= 0.00345 are obtained, respectively. A mean value as a representative value for the statistical tests under the same test conditions.

Curves of the mean and mean minus two standard deviations are determined from Eqs. (7-7) and (7-8) using obtained coefficients $C_1 \sim C_4$, C_R and S_R as,

<u>mean curve:</u>

$$M_u/M_p = \begin{cases} 1.0 & (\bar{\lambda} \leq 0.4) \\ -0.30\bar{\lambda}^2+0.0513\bar{\lambda}+1.028 & (0.4 < \bar{\lambda} \leq \sqrt{2}) \\ \dfrac{1}{\bar{\lambda}^2} & (\bar{\lambda} > \sqrt{2}) \end{cases} \quad \text{---(7-9)}$$

Eq. (7-10) approximates the M-2S curve smoothly by a polynomial function of $\bar{\lambda}^4$.

(M-2S) curve:

$$M_u/M_p = 0.0592\bar{\lambda}^4 - 0.281\bar{\lambda}^3 + 0.491\bar{\lambda}^2 - 0.806\bar{\lambda} + 1.144 \quad (0.2<\bar{\lambda}) \qquad \text{---(7-10)}$$

Figure 7-10(a) shows both the mean curve and the M-2S curve, compared with the test data.

Table 7-6 summarizes statistical results of the ratio of the test points to the mean function (Eq. (7-9)). Mean values for the rolled beams are nearly equal to 1.0 even for different sections and annealing conditions. The test points of the high strength steel are plotted very closely to the mean curve.

The lower part of the figure indicates the standard deviation of the test points over each 0.1 interval of $\bar{\lambda}$ and the standard deviation function Eq. (7-8). Eq. (7-8) can explain quite well the experimental standard deviation. The (M-2S) function, Eq. (7-10), of all the test points over each 0.1 interval of $\bar{\lambda}$ almost coincide with the minimum envelope of the test points.

(2) Welded Beams

The same terminal values of C_1=0.4, $C_2=\sqrt{2}$, C_3=0.2 and C_4=1.05 are used for the welded beams. Through a nonlinear regression analysis, the values of C_R = -0.0076 and S_R= 0.00377 are obtained.

Eqs. 7-11 and 7-12, respectively, represent the mean and (M-2S) curves for the welded beams. The mean curve can be expressed statistically by a simple straight line in the inelastic range.

mean curve:

$$M_u/M_p = \begin{cases} 1.0 & (\bar{\lambda} \leq 0.4) \\ -0.493\bar{\lambda}+1.197 & (0.4 < \bar{\lambda} \leq \sqrt{2}) \\ \dfrac{1}{\bar{\lambda}^2} & (\bar{\lambda} > \sqrt{2}) \end{cases} \qquad \text{---(7-11)}$$

(M-2S) curve:

$$M_u/M_p = -0.0235\bar{\lambda}^4+0.119\bar{\lambda}^3-0.0621\bar{\lambda}^2-0.623\bar{\lambda}+1.126 \quad (0.2<\bar{\lambda}) \qquad \text{---(7-12)}$$

Figure 7-10(b) shows results of 154 tests compared with Eqs. (7-11) and (7-12). The (M-2S) curve almost coincides with the minimum envelope of the test points. Table 7-6 shows statistical results of the ratio of the test points to the mean function, Eq. (7-11).

(3) Plate Girders

The available number of plate girders is limited to 42 in this study. It is not so accurate enough to estimate the mean strength and the lower bound of girder strength. The mean function was roughly calculated by the nonlinear regression analysis with constant conditional variance, in which M_y was used for the nondimensional factor instead of M_p. The obtained value of C_R is 0.154. Results of the analysis and the test points are shown in Fig. 7-10(c). The (M-2S) function obtained for welded beams is also shown in Fig. 7-10(c), in which M_y is used instead of M_p. Generally, the design philosophy of plate girders is primarily to prevent the lateral-torsional buckling by sufficiently braced members and to deliver the in-plane bending strength efficiently. Further investigation is necessary to determine the strength of plate girders considering the combined web and flange buckling.

7-6 CONCLUSIONS

The following main conclusions have been drawn from the results of the

experimental data-base approach .

(1) A numerical data-base system has been developed for experiments on steel beams. Details of cross-sectional shape, load and boundary conditions, fabrication process, material properties and initial imperfections, etc. have been stored in the data-base for a total of 544 beam tests conducted in Japan, U.K., U.S.A., West Germany and Australia. Various computational and statistical manipulation programs are available to evaluate the test data.

(2) The plastic moment M_p is suitable as the nondimensionalized factor for rolled and welded beams and the yield moment M_y in compression flange for plate girders.

(3) Welded beams have a lower ultimate strength in mean values and a larger variation than rolled beams.

(4) There is a possibility that the similar type of strength function may be used for columns and beams.

(5) The beam strength curve of the ECCS can not indicate properly the strength of the test data even selecting a suitable system factor n.

(6) Test beams are divided into three groups of rolled beams, welded beams and plate girders and the mean strength curve and the lower bound curve are proposed for each group.

(7) Further investigation may be necessary for the interaction between the lateral-torsional buckling and the local buckling of flange and web, especially for plate girders.

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NOTATION

The following symbols are used in this chapter:

A_c = area of compressive flange;
A_w = area of web;
b = flange width;
E = modulus of elasticity;
h = beam height;
h_w = web height between flanges;
L = span length of member;
M = mean value;
M_E = elastic critical moment;
M_p = plastic moment;
M_u = ultimate bending moment;
M_y = yield bending moment;
S = standard deviation;
t_f = flange thickness;
t_w = web thickness;
$\bar{\lambda}$ = slenderness function;
σ_y = yield stress;
$(\sigma_y)_a$ = actual yield stress;
$(\sigma_y)_n$ = nominal yield stress;
ω = coefficient of variation.

Table 7-1 Number of Beam Data in NDSS

	Rolled Beams	Welded Beams	Plate Girders	Total
Japan	240	152	28	420
U.K.	35	0	14	49
U.S.A.	31	2	0	33
West Germany	27	0	0	27
Australia	15	0	0	15
Total	348	154	42	544

Table 7-2(a) Summary of Rolled Beam Data in NDSS

Boundary and Loading Conditions	Ref. Nos.	Shape (mm)	L (cm)	Number of Beams
(1) laterally supported point	21	200x100x5.5x8	70 - 150	18
	22	200x100x5.5x8	100,150,200,300	4
	2	203x133x5.5x7.8	186 - 603	9
		222x209x13x21	215,377	4
		309x102x6x9	105,377	10
		158x153x6.6x9.3	216 - 603	6
		254x102x5.8x6.8	360	1
(2)	16	200x100x5.5x8	90 - 130	4
		194x150x6x8	140	1
	26	256x146x6.4x10.9	250,300,350	3
(3)	21	200x100x5.5x8	80	6
(4)	16	200x100x5.5x8	90,130	3
(5)	21	200x100x5.5x8	70,80,90	13
(6)	16	200x100x5.5x8	70,90,140	3
(7)	18	200x100x5.5x8	150 - 800	21(7)**
(8) loading beam	3	203x 58x3.4x5.8	608,914	5
	7	200x100x5.5x8	350,600	2
(9)	11	261x151x7.7x12.3	244 - 610	6(2)**
	17	200x100x5.5x8	220 - 550	12
	23	200x100x5.5x8	150,200,260	75
	25	200x100x5.5x8	150,175,225,350	34
		150x75x5x7	175	10
	28	152x120x6x9(160x9)*	120,130,150,190	8
		152x160x6x9(120x9)*	130,190	6
(10)	9	254x118x7.9x12.7	152 - 915	14
		305x165x6.4x9.5	305 - 915	7
		457x191x9.5x14.3	305 - 1524	10
	25	200x100x5.5x8	225,300	26
		150x75x5x7	225	8
(11)	26	256x146x6.4x10.9	400,450,550	3
(12)	26	256x146x6.4x10.9	350,400,450	3
(13)	29	152x120x6x9(160x9)*	300,320,350	4
		200x100x5.6x8.5	350	1
(14)	29	152x120x6x9(160x9)*	130,150,190	6
		200x100x5.6x8.5	130	2
Total				348

Table 7-2(b) Summary of Welded Beam Data in NDSS

Boundary and Loading Conditions	Ref. Nos.	Shape (mm)	L (cm)	Number of Beams
(2)	4	454x220x6x12	200	4
		454x190x6x12	200	2
	20	250x125x12x12	110 - 160	12
	10	324x110x6x12	200	5
	13	237x108x6.5x16	60	1
		239x151x6.8x15.7	90	1
(6)	19	500x125x9x12	60 - 100	8
		375x125x9x12	90	2
		250x125x9x12	100	2
		175x125x9x12	120	1
(7)	18	250x125x9x12	150 - 450	9
		200x100x6x9	200 - 400	3
(8)	5	250x100x6x8	300,400,450	12(6)**
		300x100x6x8	300,400,450	6(3)**
		250x100x7x10	250,300,350	6(3)**
		250x120x7x10	250,300,350	3
		300x100x7x10	250,300,350	3
	6	300x100x6x8(150x8)*	300,400	2
		300x150x6x8(100x8)*	300,400	2
		489x120x10x19(150x19)*	400	1
		489x150x10x19(120x19)*	400	1
(9)	24	250x100x6x8	180,260	68
Total				154

Table 7-2(c) Summary of Plate Girder Data in NDSS

Boundary and Loading Conditions	Ref. Nos.	Shape (mm)	L (cm)	Number of Beams
(2)	14	816x84x3.2x8 818x100x3.2x9	168 168	1 1
	1	980x200x3.2x10 820x160x3.2x10	192 160	1 1
	15	818x195x6x14 830x180x6x25 1150x182x6x26	150 150 150	5 4 2
(8)	5	1000x130x6x10 1000x120x6x8 800x110x6x10 800x130x6x10 1000x120x3.2x10(160x10)*	410 410 280,330 280,330 280,330	1 1 3 2 6
(9)	12	432x89x4.8x25.4	437	2
(15)	29	515x120x6x10(180x12.5)*	162.5,310.4,449.5	12(6)**
Total				42

* : for tension flange
** : numerals in ( ) for annealed beams contained

Table 7-3 Sectional Properties of Test Beams

	Rolled Beams			Welded Beams			Plate Girders		
	Min	Mean	Max	Min	Mean	Max	Min	Mean	Max
$(h/b)\sqrt{\sigma_y/235}$	1.02	2.20	4.49	1.49	3.08	5.60	4.64	7.33	13.3
$(L/b)\sqrt{\sigma_y/235}$	7.40	27.5	198.4	0.51	26.6	62.8	8.80	26.8	54.7
$(h_w/t_w)\sqrt{\sigma_y/235}$	19.3	37.7	71.9	17.9	46.3	119.5	77.1	192.5	370.5
$(b/t_f)\sqrt{\sigma_y/235}$	9.36	14.6	25.9	6.57	14.5	34.3	3.40	14.4	22.8

h = beam height, b = compressive flange width
L = laterally unsupported length
t_f = compressive flange thickness
h_w = web height, t_w = web thickness
σ_y = yield stress (N/mm^2)

Table 7-4 Ratios of Test Results to Eq. (7-1) or (7-2)

System Factor n		Rolled Beams N=348 $(Exp)_a/Eq.(7-1)$ or $(7-2)$		Welded Beams N=154 $(Exp)_a/Eq.(7-1)$ or $(7-2)$		Plate Girders N= 42 $(Exp)_a/Eq.(7-1)$ or $(7-2)$	
		M	S	M	S	M	S
M_u/M_p Eq. (1)	n=2.0	1.071	0.114	1.050	0.134	0.891	0.124
	n=2.5	1.022	0.103	0.998	0.122	0.859	0.123
M_u/M_y Eq. (2)	n=2.0	1.175	0.113	1.161	0.137	0.981	0.124
	n=2.5	1.123	0.106	1.104	0.123	0.948	0.127

Table 7-5 Strength Variations on Different Boundary and Loading Conditions

Boundary and Loading Cond.	N	M	S
(1)	52	1.035	0.057
(2)	8	0.976	0.041
(3)	6	0.971	0.050
(4)	3	0.961	0.008
(5)	13	1.022	0.057
(6)	3	0.970	0.016
(7)	21	1.079	0.108
(8)	7	0.870	0.106
(9)	151	1.033	0.105
(10)	65	0.999	0.127
(11)	3	1.153	0.035
(12)	3	1.022	0.008
(13)	5	1.036	0.034
(14)	8	0.971	0.028

Table 7-6 Ratios of the Test Results to the Proposed Mean Curves

Type of Process			Symbol	N	M	S
Rolled Beams M_u/M_p	mild steel	as-delivered	○	310	0.999	0.091
		annealed	⊖	9	1.075	0.036
	high strength steel	as-delivered	●	29	1.024	0.058
		annealed		0	—	—
Total				348	1.003	0.089
Welded Beams M_u/M_p	mild steel	as-delivered	△	116	1.041	0.114
		annealed	⊿	9	1.011	0.117
	high strength steel	as-delivered	▲	26	1.040	0.088
		annealed	⊿	3	1.185	0.084
Total				154	1.042	0.111
Plate Girders M_u/M_y	mild steel	as-delivered	□	26	1.007	0.113
		annealed	⊟	6	1.012	0.137
	high strength steel	as-delivered	■	10	1.036	0.101
		annealed		0	—	—
Total				42	1.015	0.112

high strength steel : $\sigma_y \geq$ 430 N/mm^2

(a) $(b/t_f)\sqrt{\sigma_y/235}$ of Rolled Beams

(b) $(h_w/t_w)\sqrt{\sigma_y/235}$ of Rolled Beams

(c) $(b/t_f)\sqrt{\sigma_y/235}$ of Welded Beams

(d) $(h_w/t_w)\sqrt{\sigma_y/235}$ of Welded Beams

(e) $(b/t_f)\sqrt{\sigma_y/235}$ of Plate Girders

(f) $(h_w/t_w)\sqrt{\sigma_y/235}$ of Plate Girders

Fig. 7-1 Histograms of Sectional Properties

(a) Measured Yield Stresses

(b) Measured Yield Stresses nondimensionalized by Nominal Values

Fig. 7-2 Histograms of Yield stresses

Fig. 7-3(a) Histograms of Initial Out-of-Crookedness

Fig. 7-3(b) Residual Stress Distributions

(a) non-dimensionalized using M_p

(b) non-dimensionalized using M_y

Fig. 7-4 Test Results of Rolled Beams and the ECCS Beam Curves

(a) non-dimensionalized using M_p

(b) non-dimensionalized using M_y

Fig. 7-5 Test Results of Welded Beams and the ECCS Beam Curves

(a) non-dimensionalized using M_p

(b) non-dimensionalized using M_y

Fig. 7-6 Test Results of Plate Girders and the ECCS Beam Curves

Fig. 7-7(a) ECCS Beam Functions

Fig. 7-7(b) Results of Regression Analysis of Rolled Beams

Fig. 7-7(c) Results of Regression Analysis of Welded Beams

Fig. 7-7(d) Results of Regression Analysis of Plate Girders

Fig. 7-8 Strength Variation and JRA Design Formula

Fig. 7-9(a) ECCS Unified Column Functions

Fig. 7-9(b) Results of Regression Analysis of Rolled Beams

Fig. 7-9(c) Results of Regression Analysis of Welded Beams

Fig. 7-9(d) Results of Regression Analysis of Plate Girders

(a) Rolled Beams

(b) Welded Beams

Fig. 7-10 Beam Data and Proposed Curves

(c) Plate Girders

Fig. 7-10 Beam Data and Proposed Curves

Chapter 8. EVALUATION OF COMPRESSIVE STRENGTH OF STEEL PLATES

8-1 INTRODUCTION

Buckling and post-buckling behavior of rectangular plates loaded in compression on two opposite edges have been investigated extensively (30,31) since Bryan obtained the first buckling solution in 1891.

In 1932, von Kármán derived an approximate formula for the effective width of simply supported edges. In 1945, Winter (32,34) applied the effective width concept to light-gage steel design. Since then various formulas have been suggested for the ultimate strength and design of the cold-formed sheet and welded plate using the effective width concept.

Experimental investigations have been made of the local buckling of steel plate elements to obtain the strength reduction due to initial imperfections in medium thickness plate, and the strength increases in thin plates.

Extensive tests have been conducted in United Kingdom. They include model welded box columns of the Forth Bridge Towers (7), and the studies of Dwight · Harrison (1) on welded box columns, and of Dwight · Ractliffe (6,11), Moxham (14,15), Little (16) on single plates and box columns. In Japan, tests have been made by Fujita (2), Yoshiki (5), Yokoo (8), Uchiyama (9), Tanaka (10), Fukumoto (12), Nishino (4,13), Hasegawa (17). Little (39), Massonnet (53) and Usami · Fukumoto (23,25) have investigated the local and overall interaction behavior of box columns.

Theoretical ultimate strength approaches to the elasto-plastic behavior of compressed plates with initial imperfections have been made in Japan by Okamura · Yoshida (48), Watanabe (49), Komatsu · Kitada · Miyazaki

(50), Ueda (55) and Fujita (54). Komatsu and Kitada (51,52) made ultimate strength analyses with initial imperfections of $\sigma_{rc} = 0.4\ \sigma_y$ (σ_{rc} = compressive residual stress, σ_y = yield stress) and $w_o = b/150$ (w_o = initial out-of-flatness, b = plate width), which are typical of the worst initial imperfections. They proposed a lower bound design curve and compared it with available test data and existing design formulas in specifications.

Faulkner (46) made an extensive review of effective plating and a list of the important effective width formulas, and Hasegawa and Usami (56) introduced more recent effective width formulas.

This chapter makes a survey of the ultimate compressive strength of plates which appeared in available test reports. Comparisons are made statistically between the reviewed test data and the existing theoretical and empirical formulas for the ultimate strength of plates.

8-2 PLATE TEST DATA AND DATA-BASE

(1) Number of Test Data

The purpose of this section is to review and store the data of uniformly compressed plates with clearly defined support conditions. The test data may be categorized by the cross-sectional shapes and the support conditions as (a) single plates, (b) square boxes and (c) cruciform shapes. Test data for rectangular boxes are also stored for comparison with the square box test results. Tubular box sections are also surveyed and included in the data-base. The data for the compression flange of H and box beams are not included because of the small number of test data and unidentified support conditions.

Table 8-1 shows a total of 793 individual test results which is

categorized by cross-sectional shapes, fabrication process, method of testing and country of origin. In Europe, 476 tests are further classfied into 323 of U.K., 148 of West Germany and 5 of Sweden. The data stored in the data-base are from Refs. (1)-(29). Test data in Ref. (3) have not been included in the analysis hereafter because of the unidentified edge condition.

(2) Data Items

The data-base of plates consists of a) σ_{cr} = maximum average compressive stress, b) σ_y = yield stress, c) width - thickness ratio with the following additional items:

1) Cross-sectional shape --- single plate, square box column, rectangular box section and cruciform section.
2) Dimensions of plate --- thickness, width, length, aspect ratio and cross-sectional area.
3) Fabrication method --- welded built up, cold-formed, mechanical cutting, gas cutting; After treatment --- as-delivered, annealed and gas heated.
4) Yield stress --- nominal yield stress, measured yield stress, method of test (compression or tension).
5) Support condition --- unloaded and loaded edges (simple support, clamped, free and others).
6) Initial imperfections --- measured residual stresses and measured initial out-of-flatnesses.
7) Other measured items --- load-average strain curve and average strain at the maximum load.
8) References --- reference number, reference and the country where the

test was conducted.

9) Identification --- name of specimens and sequential number.

Measured values of Young's modulus E and Poisson's ratio ν are not usually given in references, and so the values adopted for these material properties are E = 206 kN/mm^2 ($2.1 \times 10^6 kgf/cm^2$) and ν = 0.3, respectively.

(3) Material Properties of Test Specimens and Initial Imperfections

Material Properties --- Figure 8-1(a) shows a histogram of yield stresses of test specimens. Of the N = 789 test specimens, 13 % are of high tensile steels (σ_y > 430 N/mm^2) and the rest are of ordinary steels. Mean value of the ratio of the measured to nominal yield stress, $(\sigma_y)_a/(\sigma_y)_n$, is 1.13. The coupon test data include values for cold-formed square tubes which have high yield stress due to cold-forming. When these values (N = 71) are excluded in the data, the yield stress ratio becomes 1.09. The thicknesses of tensile coupons are from 2.1 mm to 22.3 mm, with a mean value of 7.1 mm. The tests have been made therefore for comparatively thin plates.

Initial Out-of-Flatness --- Figure 8-1(b) shows a histogram of (w_o/b) values for the N = 220 cases where measurements are available, in which w_o is the maximum out-of-flatness or the out-of-flatness of the plate center and b is the width of the plate.

The shaded areas are for the initial out-of-flatness of plates bent by cold pressing. A fitted Weibull distribution is shown by a solid curve. For 91 out of the 220 cases, the w_o value is larger than b/250, the most common value prescribed by codes as a fabrication tolerance.

Residual Stress --- Figure 8-1(c) shows a histogram of the ratio of the measured residual compressive stress σ_{rc} (which is uniformly

distributed over much of the plate) to the measured yield stress σ_y for the N = 248 cases where measurements are available. The mean value and standard deviation of σ_{rc}/σ_y are M = 0.23 and S = 0.145, respectively. The shaded areas are for N = 32 high strength steels (σ_y > 430 N/mm^2). The magnitude of residual compressive stresses may not be influenced by the yield stress of base metal and thus the σ_{rc}/σ_y values for high strength steels become small as shown in Fig. 8-1(c). The mean and standard deviation of the σ_{rc}/σ_y values of the histogram excluding the shaded areas are M = 0.25 and S = 0.15, respectively.

(4) Average Compressive Strain at the Maximum Load

There are significant differences in the load-shortening curves for different width-thickness ratios of plates, steel grades and initial imperfections.

The average compressive stress σ - strain ε relationships of compressed plates can be classified schematically into the following three patterns: a) for high b/t ratios, a linear elastic σ - ε relationship may be held to the peak load and then followed by abrupt descending curve, b) for low b/t ratios, a linear σ - ε relationship is followed by gradual change of slope to a yield plateau and then strength can be increased in the strain hardening range, and c) for medium b/t ratios, the σ-ε relationship is in between a) and b), i.e., the strength gradually decrease beyond the peak plateau.

The width-thickness ratio b/t is expressed as a modified plate slenderness

$$\bar{\lambda}=\sqrt{\sigma_y/\sigma_e}=b/t\sqrt{(\sigma_y/E)\cdot 12(1-\nu^2)/\pi^2 k} \qquad \text{----(8-1)}$$

in which σ_e is the critical stress given by the linear buckling theory and k is a buckling coefficient which varies with the aspect ratio of the plate (a/b) and the support conditions along the edges.

The commonly specified values of the width-thickness limits which will ensure adequate plastic hinge rotations in mild steel box sections (b/t = 30) and the flanges of H-sections (b/t = 10) correspond to $\bar{\lambda}$ = 0.578 (44) (see Fig. 8-2).

There are 355 cases of load-shortening curves available in the surveyed references. Figure 8-2 shows the nondimensional edge strain $\varepsilon_u/\varepsilon_y$ at the ultimate plate strength plotted against the plate slenderness $\bar{\lambda}$. The symbols used for the test points, which are summarized in Tables 8-2, 8-3, 8-4 and 8-5, allow the different groups to be identified.

The effective-width concept, which was proposed originally by von Kármán, predicts failure when the plate edge stress reaches the yield stress when no residual stress is present (i.e. $\varepsilon_u/\varepsilon_y$ = 1). However, if the tensile residual stress in welded built-up sections is equal to the yield stress, then the nondimensional edge strain $\varepsilon_u/\varepsilon_y$ at failure should be equal to 2.

From Fig. 8-2, the lower bound of the $\varepsilon_u/\varepsilon_y$ values in the range of 0.8 - 2.0 for $\bar{\lambda} > 0.7$. For $\bar{\lambda} < 0.7$, the values of σ_u/σ_y increase with a large scatter, and the edge strain may reach the initial strain hardening strain at $\varepsilon_{st}/\varepsilon_y$ = 9 - 17.

8-3 STRENGTH CHARACTERISTICS AND TESTING PARAMETERS

In order to compare the plate strengths of different cross-sectional shapes, with different fabrication processes and supported conditions, the ratios of the test points to those of a basic plate strength curve are

compared (41,42). The measured yield stress and cross-sectional dimensions are used to nondimensionalize plate strengths and plate slenderness $\bar{\lambda}$, and the basic strength curve is obtained from

$$\sigma_{cr}/\sigma_y = 0.984/\bar{\lambda} - 0.292/\bar{\lambda}^2 + 0.0334/\bar{\lambda}^3 \qquad \text{----(8-2)}$$

This equation represents the mean strength curve obtained through a regression analysis of the test points (N = 146) for the welded square box sections alone.

Fig. 8-3 demonstrates a scatter of the test points from Eq. (8-2), especially in the range of $\sigma_{cr}/\sigma_y > 1$ ($\bar{\lambda} \leq \bar{\lambda}_o = 0.579$) where the average compressive stress σ_{cr} exceeds the yield stress.

(1) **Single Plates**

A test set-up to satisfy simply supported conditions along the unloaded edges is quite troublesome and various efforts have been made (15).

Figure 8-4 shows test results of 413 of single plate tests plotted in the σ_{cr}/σ_y (maximum average stress/ actual yield stress) and $\bar{\lambda}$ coordinates. Symbols of the test points are summarized in Table 8-2 showing different fabrication conditions, yield stresses and aspect ratios α. In Fig. 8-4, Eq. (8-2), von Kármán's formula and Euler's $1/\bar{\lambda}^2$ are given for reference. Among the 413 test data, 313 were simply supported along the unloaded edges and 101 were clamped. 72 points were for $\alpha = a/b < 1$. No test data were available for the clamped edges with $\alpha < 1$. The test values of α ranged from 0.167 to 6.0.

Coefficients k in Eq. (8-1) for plates subjected to pure compression are

k = 4.0 for unloaded edges simply supported and $\alpha \geqq 1$

$= \alpha^2 + 1/\alpha^2 + 2$ for $\alpha < 1$

k = 6.97 for unloaded edges clamped and $\alpha \geqq 1$

The crosses + and arrows ↓ indicate the mean M and mean minus two standard deviations (M-2S) of all the single test points which fall into each 0.1 interval of $\bar{\lambda}$, where M = mean value and S = standard deviation. The lower part of the figure indicates the coefficient of variation ω of the test points over each $\bar{\lambda}$ = 0.1 interval.

Table 8-2 summarizes statistical results, that is, M, S and ω of the ratio of the test results to Eq. (8-2) for different groups of unloaded edge conditions, welding and annealing conditions, aspect ratios $\alpha \gtreqless 1$ and steel grades $\sigma_y \gtreqless 430$ N/mm^2. W represents the presence of the residual stress caused by weld beads along the unloading edges. In Table 8-2, the upper line indicates results obtained only for the test data which fall into the range of $\bar{\lambda} \geqq \bar{\lambda}_o = 0.579$ for which Eq. (8-2) yields $\sigma_{cr}/\sigma_y \leqq 1$, and the lower line indicates the ratio for all the test data including those in the strain hardening range.

Single plate strengths may be summarized as follows from Fig. 8-4 and Table 8-2:

(a) There is a significant loss in plate strength due to the presence of residual stresses (W). Plates with residual stresses (W) possess strengths between 5 % and 15 % less than those without residual stresses (C : as-cut and A : annealed). The test points for the plates are always below von Kármán's Eq. (8-10) for $\bar{\lambda} \geqq 1.0$.

(b) Plates with clamped unloaded edges possess strengths between 5 % and 15 % higher than those with simply supported edges as shown in Refs. (6), (15) and (22). However, the elastic critical stress σ_e according

to the linear buckling theory is 75 % higher for clamped edges than for simply supported edges. Therefore, the clamped plate test data plot a little lower than the simply supported plate data when k = 6.97 is used for clamped plates.

(c) Test data for $\alpha < 1$ are scattered in the vicinity of Eq.(8-2) curve for $\bar{\lambda}$ = 0.5 to 1.3. For $\bar{\lambda} > 1.3$, however, the test data become close to the Euler curve, showing column-like behavior, and the plate strengths in the range are quite different from those of plates with $\bar{\lambda} \geqq 1$.

(d) There is little difference in strength due to steel grades in the σ_{cr}/σ_y - $\bar{\lambda}$ coordinates. This may be partly due to the limited number of test data for high strength steels in the present survey.

(2) Square Box Columns

Figure 8-5 shows 224 test results for square boxes with uniform thickness. The value of k in Eq. (8-1) is 4.0 and b is the flange width between centers of webs for both welded and tubular box sections. The aspect ratios of test plates range from α = 3 to 8.7.

Table 8-3 summarizes statistical results as in Table 8-2 for welded and tubular square box sections. The plates are categorized into sub-groups with respect to fabrication conditions.

Plate strengths may be summarized as follows from Fig. 8-5 and Table 8-3.

(a) There is a significant difference in plate strength due to the presence of residual stresses (D). Plates with residual stresses (D) possess strengths between 5 % and 15 % less than those of annealed plates (A), which is similar to the conclusion reached for single plates.

(b) Tubular sections may possess relatively high strength compared with welded sections.

(c) There is little difference in strength due to steel grades in the σ_{cr}/σ_y - $\bar{\lambda}$ coordinates, which is similar to the conclusion reached for single plates.

(d) There is little difference in strength between single plate and square box tests for the same fabrication conditions.

(3) Cruciform Columns

Cruciform columns are generally used to investigate the compressive strength of the outstanding plate element of flanges in I shape beams and girders. A plate which composes a cruciform column has simply-free support condition on the nonloaded edges. The plate coefficient, k , is changed by its aspect ratio, α, and the support condition on the loaded edges.

The buckling load N_x of a plate with the uniform compression is generally expresses by a energy method (31,32)

$$N_x = D \frac{\int_0^b \int_0^a \{ (\frac{\partial^2 w}{\partial x^2} + \frac{\partial^2 w}{\partial y^2})^2 - 2(1-\nu)[\frac{\partial^2 w}{\partial x^2}\frac{\partial^2 w}{\partial y^2} - (\frac{\partial^2 w}{\partial x \partial y})^2] \} dxdy}{\int_0^b \int_0^a (\frac{\partial w}{\partial x})^2 dxdy}$$

---(8-3)

where, $D = Et^3/12(1-\nu^2)$, a = length of nonloaded edge, b = length of loaded edge, E = modulus of elasticity, t = plate thickness, ν = Poisson's ratio and w = deflection of plate. When loaded edges are simply supported, Eq. (8-4) can be assumed as an expression for the buckled surface.

$$w = A\sin\frac{\pi}{a}x(y+By^n) \qquad \text{---(8-4)}$$

From Eqs. (8-3) and (8-4) the plate coefficient can be obtained

$$k = (b/a)^2 + 0.608(1-\nu) \qquad \text{---(8-5)}$$

Substituting Poisson's ratio ν=0.3

$$k = (b/a)^2 + 0.426 \qquad \text{---(8-6)}$$

When loaded edges are clapmed, Eq. (8-7) can be assumed

$$w = A(1-\cos\frac{2\pi}{a}x)(y+By^n) \qquad \text{---(8-7)}$$

From Eqs. (8-3) and (8-7) the plate coefficient can be obtained

$$k = 4(b/a)^2 + 0.426 \qquad \text{---(8-8)}$$

The minimum plate coefficient of k=0.426, which is obtained by assuming the infinite aspect ratio, may be used as a conservative basis for design. However, it overestimates for evaluating test results. In this section, the plate coefficients of Eqs. (8-6) and (8-8) are used to evaluate test results.

Figures 8-6 shows 104 test results for cruciform columns. Table 8-4 summarizes statistical results for cruciform columns.

Plate strengths may be summarized as follows from Fig. 8-6 and Table 8-4.

(a) Annealed plates tend to have higher strength in the range of $\bar{\lambda} > \bar{\lambda}_o$ = 0.579 than as-welded plates. On the contrary, they have lower strength in the range of $\bar{\lambda} < \bar{\lambda}_o$.

(b) The strength characteristics of cruciform plates on $\sigma_{cr}/\sigma_y \sim \bar{\lambda}$

coordinates may be little different from single plates. This may be partly due to the limited number of test points in the range of $\bar{\lambda}$ = 0.5 to 0.8. Further investigation is needed for this range.

(4) Rectangular Box Columns

Four institutions had performed tests of rectangular box columns considering the plate buckling. Figure 8-7(a) shows 48 test results for rectangular boxes when their plate coefficients are used k=4.0. Usami and Fukumoto (23) proposed the equivalence plate coefficient k_{eq} of rectangular plates (Eq. (8-9)).

$$k_{eq} = \begin{cases} \left(\dfrac{4}{1+\alpha}\right)^2 & (b_e<b,\ d_e<d) \\ \left\{\dfrac{2}{1+\alpha}\left(1+\dfrac{\alpha}{0.75/R_f}\right)\right\}^2 & (b_e<b,\ d_e=d) \end{cases} \qquad \text{---(8-9)}$$

where,

b =flange width, d =web width, $\alpha = d/b$,

b_e = effective width of flange, d_e = effective width of web,

$$R_f = \sqrt{\frac{\sigma_u}{E}\frac{12(1-\nu^2)}{4\pi^2}}$$

Figure 8-7(b) shows test results, when plate coefficients are used $k=k_{eq}$.

Table 8-5 summarizes statistical results for rectangular box columns. The test results used k=4.0 lie higher strength due to the restraint of the adjacent panels. The equivalence plate coefficient is effective to evaluate the plate strength of rectangular box columns.

8-4 COMPARISON WITH ULTIMATE PLATE STRENGTH FORMULAS

Various effective width formulas for plates have been proposed by many researchers which are based on test results and post-buckling strength analyses. Faulkner (46) made an extensive review of effective plating in the plate analysis of stiffened plates. Important effective width formulas were listed and some reviews were made of the statistical characteristics of the test data.

The effective width formulas shown here are for comparison with the test data. The formulas are expressed using the slenderness parameter $\bar{\lambda}$.

Von Kármán originally suggested that Eq. (8-10) gives the maximum average compressive stress σ_{cr}.

$$\sigma_{cr}/\sigma_y = b_e/b = \sqrt{\sigma_e/\sigma_y} = 1/\bar{\lambda} \qquad \text{----(8-10)}$$

Winter (33,34) suggested that Eq. (8-10) should be modified for light-gage steel design to

$$\sigma_{cr}/\sigma_y = b_e/b = 1/\bar{\lambda} \cdot (1-0.25/\bar{\lambda}) \qquad \text{----(8-11)}$$

Lind (35) proposed Eq. (8-12) as a simple compromised based on experimental data for light gage steel plates (N = 336).

$$\sigma_{cr}/\sigma_y = b_e/b = 0.86/\bar{\lambda} \qquad \text{---(8-12)}$$

Möller (45) and Faulkner (46) proposed Eqs. (8-13) and (8-14), respectively

$$\sigma_{cr}/\sigma_y = 1/\bar{\lambda} \cdot (1 - 0.148/\bar{\lambda}^2) \qquad \text{----(8-13)}$$

$$\sigma_{cr}/\sigma_y = 1/\bar{\lambda} \cdot (1.05 - 0.277/\bar{\lambda}) \qquad \text{----(8-14)}$$

Dwight (56) proposed Eq. (8-15) in which the average compressive residual stress σ_{rc} is accounted for

$$\sigma_{cr}/\sigma_y = 1/\bar{\lambda} \cdot (0.85 - \sigma_{rc}/\sigma_y) \qquad \text{----(8-15)}$$

Usami and Fukumoto (23) proposed that the ultimate plate strength of high strength steel based on the box column test results could be obtained from

$$\sigma_{cr}/\sigma_y = 0.75/\bar{\lambda} \qquad \text{---(8-16)}$$

Figure 8-8 shows the formulas of Eqs. (8-10), (8-11), (8-12), (8-13) and (8-16). Table 8-6 summarizes statistical results for M and S of ratios of the test results for single plates and square boxes to Eqs. (8-10)~ (8-13) and (8-16). In Table 8-6, the upper line indicates the results obtained when only the test data (N is given in the table) which fall into the range of $\bar{\lambda} \geq \bar{\lambda}_o$ are used where $\bar{\lambda}_o$ is the limiting slenderness parameter for which $\sigma_{cr} = \sigma_y$ according to each formula. The lower line indicates the results obtained using all the test data (N = 340 for plates and 224 for box sections).

The mean ratios compared with the von Kármán formula are low showing that this equation overestimates the test data. The Usami and Fukumoto formula is clearly very close to the mean value of the test points for $\bar{\lambda} > \bar{\lambda}_o$.

Faulkner approximated the coefficient of variation ω for his reviewed test data on simply supported plates from his mean curve (Eq. (8-14)) as

$$\omega = 0.05 \qquad for\ 0.26 \leq \bar{\lambda} \leq 1.32$$

$$\omega = 0.038\ \bar{\lambda} \qquad for\ \bar{\lambda} > 1.32$$

The lower part in Fig. 8-5 indicates ω-values in the range of 0.049 to 0.101 for $\bar{\lambda}$ from 0.4 to 1.5, so that the Faulkner's lowest ω-value is close to the lower bound.

8-5 ULTIMATE PLATE STRENGTH FROM TEST DATA

Since there were significant losses and variations in strength due to the presence of residual stresses, plate strengths are categorized into two groups depending on the presence of residual stresses.

Ultimate plate strengths for $\alpha < 1$ are not analyzed hereafter because of insufficient test data.

(1) Plates with Residual Stress

Figure 8-9(a) shows results of 383 plate tests with residual welding stresses. Symbols of the test points are summarized in Table 8-7. Test data in this category are for single plates with weld beads along the unloaded edges, and for as-welded box columns.

In Fig. 8-9(a), the nearly equal length of the 2S arrows in the range of $\bar{\lambda} > 0.5$ may assume uniform variance of the test data through this range. A nonlinear regression analysis with an assumed uniform variance was therefore performed to obtained a mean curve of the test points. This mean curve is

$$\sigma_{cr}/\sigma_y = 0.968/\bar{\lambda} - 0.286/\bar{\lambda}^2 + 0.0338/\bar{\lambda}^3 \qquad for\ \bar{\lambda} \geq 0.571$$

$$\sigma_{cr}/\sigma_y = 1.0 \qquad for\ \bar{\lambda}<0.571 \qquad \text{---(8-17)}$$

and the standard deviation which is a root of conditional variance is S = 0.0871.

The mean minus two standard deviation curve is expressed as,

$$\sigma_{cr}/\sigma_y = -0.174+0.968/\bar{\lambda}-0.286/\bar{\lambda}^2+0.0338/\bar{\lambda}^3 \qquad for\ \bar{\lambda}\geq 0.389$$

$$\sigma_{cr}/\sigma_y = 1.0 \qquad for\ \bar{\lambda}<0.389 \qquad \text{---(8-18)}$$

The dotted curves in Fig. 8-9(a) represent the mean curve (Eq.(8-17)) and the mean minus two standard deviations curve, (Eq.(8-18)). Figure 8-9(b) shows the mean curve and the curve corresponding to the mean minus two standard deviations compared with histograms of test data and fitted normal distribution curves of 0.1 intervals of $\bar{\lambda}$. Cumulative probability of $p=10^{-8}$ to 10^{-2} is scaled on normal distribution curves. Table 8-7 summarizes the mean function (Eq.(8-17)) for differently categorized sub-groups. The results in Table 8-2 are for N = 289, the number of the N = 382 test points which fall in the range $\bar{\lambda}\geq\bar{\lambda}_o$ = 0.571. For $\bar{\lambda}<\bar{\lambda}_o$, σ_{cr}/σ_y = 1 is assumed which ignores the strain hardening effect.

From Figs. 8-9 and Table 8-7, the proposed formula (Eq.(8-17)) agrees quite well with the mean value of the test points for $\bar{\lambda}>\bar{\lambda}_o$ for all the sub-groups except for the single plates with the unloaded clamped edges. The M-2S Function is close to the lower bound of the test data and even close to the Euler curve $\sigma_{cr}/\sigma_y = 1/\bar{\lambda}^2$ when $\bar{\lambda}$ becomes large. The limiting slenderness parameter $\bar{\lambda}_o$ = 0.389 for the M-2S Function correspond closely to the lower bound of the scattered test points in the vicinity of σ_{cr}/σ_y = 1. The plate strength curve proposed by Komatsu and Kitada (51) falls in

between M and M-2S curves.

(2) **Plates without Residual Welding Stress**

Figure 8-10(a) shows results of 172 plate tests without residual stresses. As-cutt and annealed single plates and annealed square box sections are included in this category.

In the same way as for the plates with residual stresses, a nonlinear regression analysis with a uniform variance was used to develop the following mean function:

$$\sigma_{cr}/\sigma_y = 1.133/\bar{\lambda} - 0.384/\bar{\lambda}^2 + 0.0468/\bar{\lambda}^3 \qquad for\ \bar{\lambda} \geq 0.658$$

$$\sigma_{cr}/\sigma_y = 1.0 \qquad for\ \bar{\lambda} < 0.658 \qquad \text{---(8-19)}$$

with the standard deviation S = 0.104. The M-2S curve is

$$\sigma_{cr}/\sigma_y = -0.208 + 1.133/\bar{\lambda} - 0.384/\bar{\lambda}^2 + 0.0468/\bar{\lambda}^3 \qquad for\ \bar{\lambda} \geq 0.337$$

$$\sigma_{cr}/\sigma_y = 1.0 \qquad for\ \bar{\lambda} < 0.337 \qquad \text{---(8-20)}$$

Figure 8-10(b) shows a summary of the test results and the mean curve Eq. (8-19) and the mean minus two standard deviations Eq. (8-20). Table 8-8 summarizes statistical results for the ratio of the test points to the mean curve (Eq. (8-19)) for differently categorized sub-groups. As shown in Fig. 8-10, the M Function is close to the von Kármán curve, when $\bar{\lambda}$ becomes large. The proposed formula (Eq.(8-19)) agrees quite well with the mean value of the test points for $\bar{\lambda} > \bar{\lambda}_o$ = 0.658. The M-2S Function underestimates the test points for large $\bar{\lambda}$, since the uniform variance was assumed for much of the range.

(3) **Effect of Initial Imperfections**

As shown in Figs. 8-1, N = 248 of measured residual stresses have been

reported in the surveyed references, and histograms showing variations of the measured data. The effect of initial imperfections upon the ultimate plate strength has been analyzed in different investigators using the measured magnitude of imperfections. Detailed statistical correlation of initial imperfections with ultimate plate strengths is not given in this dissertation because of the shortcomings of the enough data available. However, the ultimate strength data of plates which have large magnitude of initial imperfections are shown in following figures.

Initial Out-of-Flatness --- From Fig. 8-9(a), the test points which have the initial measured maximum out-of-flatness $w_o > 0.004b$ were chosen to investigate the reduction of plate strength due to the magnitude of the initial out-of-flatness. Figure 8-11 shows test results of 52 plates with $w_o > 0.004b$ and with residual stresses. The mean curve of Eq. (8-17) is also drawn for comparison. Values of M = 0.964 and S = 0.0634 were obtained by statistical manipulations of the ratio of the chosen test points to the mean curve (Eq. (8-17)). The test points are plotted considerably below the mean curve for $\bar{\lambda} < 1.3$.

Similar results can be seen for the case without residual stresses, as shown in Fig. 8-12. N = 39 with $w_o > 0.004b$ which were chosen from the N = 172 tests shown in Fig. 8-10(a), and they were compared with the mean curve of Eq. (8-19). Values of M = 0.959 and S = 0.0755 were obtained from the ratio of the chosen test data to the mean curve of Eq. (8-19).

Residual Stresses --- Plates with large magnitude of residual welding stresses ($\sigma_{rc} \geqq 0.4\ \sigma_y$) which were chosen from N = 383 test results. There were N = 27 of these, mainly scattered near the limiting value $\bar{\lambda}_o = 0.571$. The ratios of the chosen test points to the mean curve Eq. (8-17) are M = 0.994 and S = 0.0944, and no significant effect of the residual stress can

be detected from these values as shown in Fig. 8-13. There are no test data available for plates with large initial imperfections of $\sigma_{rc}/\sigma_y > 0.4$ and $w_o/b > 0.004$.

Figure 8-11 shows that the ultimate plate strength which is significantly influenced by the magnitude of the initial out-of-flatness in the medium range of $\bar{\lambda}$. Even when the N = 52 test data for $w_o > 0.004b$ are included, the scattered test points are closely bounded by the M-2S curve (Eq. (8-18)).

When the test data with large initial imperfections are excluded, a regression analysis using the N = 306 test data of Fig. 8-9(a) leads to the mean curve approximately by

$$\sigma_{cr}/\sigma_y = 1.023/\bar{\lambda} - 0.339/\bar{\lambda}^2 + 0.0458/\bar{\lambda}^3 \quad for \quad \bar{\lambda} \geqq 0.571 \qquad \text{---(8-21)}$$

and with a conditional standard deviation S = 0.0864.

Eq. (8-21) is only slightly higher than Eq. (8-17), and there is no significant difference between them.

8-6 CONCLUSIONS

The following main conclusions have been drawn from the results of the experimental data-base approach.

(1) A numerical data-base system has been developed for experimental data on steel plates. Details of cross-sectional shape, fabrication processes, material properties, initial imperfections, and the others have been stored in the data-base, from a total of 793 plate tests conducted in Europe, North America and Japan. Various computational and statistical manipulation programs are available to evaluate the

test data.

(2) No clear difference between plate strength by the single plate test method and the square box test method may be found from the test data.

(3) The plate strengths are categorized into two groups depending on the presence of the residual welding stresses. Two plate strength formulas which explain more accurately the test data are proposed from the statistical results based on the experimental data-base.

(4) Reduction of plate strength due to the magnitude of the initial out-of-flatness is predicted in the medium range of plate slenderness from the results of the experimental data-base approach.

(5) Further experimental investigations are needed for high strength steel plates with plate slenderness parameters near the limiting value $\overline{\lambda}_o$.

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NOTATION

The following symbols are used in this chapter:

a	= length of unloaded edge;
b	= length of loaded edge;
b_e	= effective width;
E	= modulus of elasticity;
$(Exp)_a$	= $\sigma_{cr}/(\sigma_y)_a$
k	= plate coefficinet;
M	= mean value;
N	= number of data;
S	= standard deviation;
t	= plate thickness;
w_o	= initial out-of-flatness;
α	= aspect ratio;
ε	= strain;
ε_u	= strain at the maximum stress;
ε_y	= yield strain;
$\bar{\lambda}$	= nondimensional slenderness;
$\bar{\lambda}_o$	= limiting nondimensional slenderness;
ν	= Poisson's ratio;
σ	= stress;
σ_{cr}	= maximum stress;
σ_e	= elastic critical stress;
σ_{rc}	= compressive residual stress;
σ_y	= yield stress;
$(\sigma_y)_a$	= actual yield stress;
$(\sigma_y)_n$	= nominal yield stress; and
ω	= coefficient of variation.

Table 8-1 Number of Plate Data

Type of Profile	Europe	North America	Japan	Total
Single Plate	362		55	417
Welded Square Box	74	8	93	175
Square Tube			49	49
Welded Rectangular Box	20		6	26
Rectangular Tube			22	22
Cruciform	20	12	72	104
Total	476	20	297	793

Table 8-2 Comparison of Test Results for Single Plates

Unloading Edges	Fabrication Conditions		Exp)$_a$/ Eq. (8-2) N	M	S	ω
Simply	W M ($\alpha \geq 1$)	○	106	0.997	0.097	0.097
			124	0.999	0.113	0.113
	W M ($\alpha < 1$)	⊘	4	0.904	0.080	0.088
			6	0.899	0.063	0.070
	W H	⊖	6	0.982	0.075	0.077
			6	0.982	0.075	0.077
	W* M	✦	8	1.102	0.093	0.084
			8	1.102	0.093	0.084
	C M ($\alpha \geq 1$)	⊘	52	1.128	0.090	0.080
			66	1.123	0.104	0.093
	C M ($\alpha < 1$)	⊕	57	0.946	0.172	0.182
			67	0.938	0.162	0.172
	A M	●	23	1.095	0.118	0.108
			29	1.041	0.152	0.146
	A H	✦	6	1.098	0.071	0.065
			6	1.098	0.071	0.065
Clamped	W M	ϕ	46	0.925	0.084	0.090
			69	0.935	0.073	0.079
	C M	✦	20	1.058	0.095	0.090
			32	1.021	0.094	0.092
Total			328	1.012	0.132	0.130
			413	1.004	0.133	0.132

W = weld with both side, W* = weld with center,
C = as-cutting, A = annealed,
M = mild steel ($\sigma_y < 430\ N/mm^2$),
H = high strength steel ($\sigma_y \geqq 430\ N/mm^2$)

Table 8-3 Comparison of Test Results for Square Box Columns

Fabrication Conditions			Exp)$_a$/ Eq. (8-2) N	M	S	ω
Welded Boxes	D M	□	79	0.990	0.095	0.096
			107	0.990	0.093	0.094
	D H	⊟	31	1.025	0.058	0.057
			39	1.005	0.068	0.068
	B M	⊞	1	0.996	--	--
			1	0.996	--	--
	A M	■	14	1.128	0.079	0.070
			26	1.057	0.103	0.097
	A H	⊞	1	1.225	--	--
			2	1.128	0.138	0.123
Tubes	D M	⊕	19	1.093	0.066	0.060
			38	1.042	0.083	0.080
	A M	⊕	4	1.247	0.078	0.063
			11	1.082	0.147	0.136
Total			149	1.032	0.102	0.099
			224	1.015	0.097	0.095

D = as-delivered, A = annealed,
B = gas-flame was run down the center-line of four sides,
M = mild steel (σ_y<430 N/mm^2),
H = high strength steel ($\sigma_y \geqq 430$ N/mm^2)

Table 8-4 Comparison of Test Results for Cruciform Columns

Fabrication Conditions		Exp)$_a$/ Eq. (8-2) N	M	S	ω
D M	△	17	1.125	0.067	0.060
		38	1.143	0.100	0.088
D H	⊿	17	1.189	0.053	0.044
		20	1.165	0.078	0.067
A M	▲	15	1.240	0.117	0.094
		24	1.163	0.145	0.125
A H	⊿	14	1.219	0.089	0.073
		22	1.098	0.187	0.170
Total		63	1.191	0.093	0.078
		104	1.142	0.131	0.115

D = as-delivered, A = annealed,
M = mild steel (σ_y<430 N/mm^2),
H = high strength steel ($\sigma_y \geqq 430$ N/mm^2)

Table 8-5 Comparison of Test Results for Rectangular Box Columns

Fabrication Conditions			Exp)$_a$/Eq. (8-2)(k=4.0) N	M	S	ω	Exp)$_a$/Eq. (8-2) (k=k_{eq}) N	M	S	ω
Welded Boxes	D M	□	8	1.073	0.741	0.069	8	0.904	0.028	0.031
			8	1.073	0.741	0.069	8	0.904	0.028	0.031
	D H	⊟	6	1.127	0.041	0.036	6	1.042	0.082	0.079
			6	1.127	0.041	0.036	6	1.042	0.082	0.079
	A M	■	9	1.261	0.049	0.039	9	1.074	0.066	0.062
			9	1.261	0.049	0.039	9	1.074	0.066	0.062
	B M	▬	3	1.138	0.095	0.084	3	0.951	0.054	0.057
			3	1.138	0.095	0.084	3	0.951	0.054	0.057
Tubes	D M	▮	17	1.235	0.119	0.096	17	1.189	0.125	0.105
			22	1.195	0.131	0.109	22	1.160	0.125	0.108
Total			43	1.188	0.113	0.095	43	1.075	0.141	0.131
			48	1.175	0.115	0.098	48	1.073	0.135	0.126

Table 8-6 Comparison between Test Results and Various Design Curves

	Single Plate (α≥1) N	M	S	Square Box N	M	S
Von Kármán	122	0.783	0.093	52	0.758	0.069
	340	0.882	0.175	224	0.950	0.189
Lind	166	0.895	0.114	83	0.882	0.084
	340	0.938	0.156	224	0.986	0.161
Winter	246	0.958	0.110	130	0.962	0.096
	340	0.982	0.147	224	1.023	0.147
Möller	243	0.888	0.105	125	0.896	0.096
	340	0.933	0.156	224	0.985	0.165
Usami	213	1.006	0.130	110	1.000	0.093
	340	1.010	0.154	224	1.042	0.138

Table 8-7 Comparison between Test Results and Proposed Curve "with residual stress" (W)

				Exp)$_a$/ Eq. (8-17) N	M	S	ω
Single Plates	Simply	W M	○	106	1.009	0.097	0.096
		W H	⊖	6	0.996	0.077	0.077
	Clamped	W M	ϕ	48	0.935	0.083	0.088
Welded Boxes		D M	□	79	1.002	0.096	0.095
		D H	⊟	31	1.038	0.059	0.056
Tubes		D M	ϕ	19	1.105	0.067	0.060
Total				289	1.004	0.097	0.096

Table 8-8 Comparison between Test Results and Proposed Curve "without residual stress" (W/O)

				Exp)$_a$/ Eq. (8-19) N	M	S	ω
Single Plates	Simply	A M	●	23	0.997	0.107	0.107
		A H	⊖	6	0.989	0.053	0.053
		C M	⊘	46	1.036	0.087	0.084
	Clamped	C M	ϕ	17	0.975	0.093	0.095
Welded Boxes		A M	■	13	1.036	0.075	0.073
		A H	⊟	1	1.123	--	--
Tubes		A M	ϕ	4	1.148	0.079	0.069
Total				110	1.021	0.094	0.093

Fig. 8-1(a) Histogram of Yield Stress

Fig. 8-1(b) Histogram of Initial Out-of-Flatness

Fig. 8-1(c) Histogram of Residual Stresses

Fig. 8-2 Edge Strains at the Ultimate Strength

Fig. 8-3 Results of Regression Analysis

Fig. 8-4 Test Results of Single Plates

Fig. 8-5 Test Results of Square Box Columns

Fig. 8-6 Test Results of Cruciform Columns

Fig. 8-7(a) Test Results of Rectangular Box Columns (k=4.0)

Fig. 8-7(b) Test Results of Rectangular Box Columns ($k=k_{eq}$)

Fig. 8-8 Various Strength Curves of Plates

Fig. 8-9(a) Test Results and Proposed Design Curves "with residual stress" (W)

Fig. 8-9(b) Summary of Test Results and Proposed Design Curves "with residual stress" (W)

Fig. 8-10(a) Test Results and Proposed Design Curves

"without residual stress" (W/O)

Fig. 8-10(b) Summary of Test Results and Proposed Design Curves

"without residual stress" (W/O)

Fig. 8-11 Effect of Initial Out-of-Flatness

"with residual stress" (W)

Fig. 8-12 Effect of Initial Out-of-Flatness

"without residual stress" (W/O)

Fig. 8-13 Effect of Residual Stresses

Chapter 9. CONCLUSIONS

9-1 SUMMARY

The work described in this dissertation consisted of two main phases; (1) extensive test programs for the statistical determination of the ultimate strength of laterally unsupported beams and (2) an establishment of a numerical data base for the ultimate strength of structural members and component plates.

(1) Test Program --- Three experimental programs on the behavior of steel beams were performed from the statistical viewpoint in this dissertation. Three groups, varing in length, of nominally identical 25 rolled beams were tested under a concentrated load applied vertically at the midspan of the compression flange. Two groups, varing in length, of nominally identical 34 welded beams were also tested under the same conditions as that of the rolled beams.

The variation of the geometrical and material imperfections such as initial out-of-straightnesses and residual stresses was measured and the influence of the variation of various parameters on the scattered test points of the lateral buckling was reviewed. The buckling behavior, buckling strength and strength variation of rolled and welded beams were compared.

Next, twenty-one two-span continuous welded beams were tested under a concentrated load. The test results of continuous beams were compared with those of the simple beams.

(2) Numerical Data Base --- A part of the work undertaken in this dissertation was an establishment of the numerical data base NDSS consisting of information regarding the ultimate strength of steel

structures and structural components. Experimental data of steel columns, steel beams and steel plates have been assembled and stored in the NDSS on a world-wide basis. Material properties for tensile tests in Japan were also stored in the NDSS.

In order to compare the test results with available formulas, statistical assessments were made, and the effects of initial imperfections such as residual stress and initial crookedness were discussed considering their variations. New design formulas for columns, beams and unstiffened plates were proposed to explain the test data from a statistical viewpoint more accurately.

9-2 CONCLUSIONS OF EACH CHAPTER

The main conclusions of each chapter have been drawn as follows.

Chapter 2

Seventy-five rolled beams of three different lengths were tested under a concentrated load applied vertically at the midspan to investigate the strength variation of laterally unsupported beams. The variation of the geometrical and material imperfections was also measured. The influence of the variation of various parameters on the scattered test points of the lateral-torsional buckling was studied.

(1) The main beam parameter that has the greatest influence upon the variation of the ultimate strength is the variable value of the actual full plastic moment that contains actual yield stress and cross-sectional dimensions. The degree of correspondence between the ultimate strength and the plastic moment becomes smaller with an increasing slenderness function.

(2) The coefficient of variation of the ultimate moment that is nondimensionalized by the actual plastic moment becomes smaller. These coefficients of variation are 24 %, 40 % and 45 % less than those of the maximum loads for the A, B and C series, respectively.

Chapter 3

Sixty-eight welded beams of two different lengths were tested under the same conditions of the rolled beams in Chapter 2. The buckling behavior of the welded and rolled beams were directly compared.

(1) The main parameter that has the greatest influence upon the variation of the ultimate strength of the welded beams as well as that of the rolled beams is also the actual plastic moment.

(2) The coefficient of variation of the ultimate moment that is nondimensionalized by the actual plastic moment becomes smaller. These coefficients of variation are 22.6 % and 1.8 % less than those of the maximum load for the D and E series, respectively.

(3) Under the same support and loading conditions for welded and rolled beams, welded beams have a lower ultimate strength than rolled beams in the mean values. The coefficients of variation of ultimate strengths of D-series and E-series beams are 0.079 and 0.110, respectively. These values are considerably large than values of rolled beams.

Chapter 4

CONCLUSIONS

Twenty-one two-span continuous welded beams were tested under a concentrated load to compare the lateral-torsional buckling behavior of simple and continuous beams.

(1) Ultimate strengths of continuous beams are higher than those of simple beams and are scattered between the elastic buckling strength curves of simple beams and of propped cantilever beams which are considered to have a nonloaded span with infinite rigidity.

(2) Test results of continuous beams are scattered on the lower part of results of simple beams in the plot of P_u/P_y versus $\lambda=\sqrt{P_y/P_E}$. The coefficients of variation were close to the values of the simple beams.

Chapter 5

A data base consisting of information regarding the ultimate strength of steel structures and structural components was established. Mechanical properties of structural steels were discussed from the statistical viewpoint.

(1) A numerical data base system was built for material properties of steels, steel columns, steel beams and unstiffened steel plates.

(2) Variations and fitted distribution functions of material properties are made clear from the data base approach. The mean value and the coefficient of variation of yield stresses nondimensionalized by nominal values are 1.168 and 0.111, respectively.

Chapter 6

CONCLUSIONS

The strength of steel column was evaluated by using the data base approach. After various computational and statistical manipulations, proposals for new multiple column curves were made.

(1) The column strength of various sections were compared. Welded H-section columns possess less strengths with increased scatter compared with rolled H-section columns.

(2) The ECCS and SSRC multiple column strength curves were compared with test data allocated to individual curves and it is made clear that multiple column curves should be used.

(3) New multiple column strength formulas which explain the test data more accurately were proposed using the statistical results which were based on the experimental data base.

Chapter 7

The beam strength on the lateral-torsional buckling was evaluated from the data base approach. Statistical results were compared with available beam strength formulas.

(1) The plastic moment M_p is available as the nondimensionalized factor for rolled and welded beams, and the yield moment M_y in compression flange is suitable for plate girders.

(2) Welded beams have a lower ultimate strength in the mean values and a larger variation than rolled beams.

(3) Test beams were divided into three groups consisting of rolled beams, welded beams and plate girders and the mean strength curve and the lower bound curve were proposed for each group.

Chapter 8

The ultimate strength of uniformly compressed plates was evaluated using the data base approach. Comparisons were made statistically between reviewed test data and existing theoretical and empirical formulas for the ultimate strength of plates.

(1) No clear difference between plate strengths by the single plate test method and by the square box test method can be found from the test data.

(2) Reduction of plate strength due to the magnitude of the initial out-of-straightness is predicted in the medium range of plate slenderness using results of the experimental data base approach.

(3) Plate strengths were categorized into two groups depending on the presence of residual stresses. Two plate strength formulas which explain the test data more accurately were proposed from the statistical results which were based on the experimental data base.

9-3 CONCLUSIONS

Some of the important conclusions have been drawn as follows.

(1) Statistical buckling tests were carried out for 75 rolled beams and 68 welded beams under a concentrated load applied vertically at the midspan in the inelastic range. The main imperfection parameter that has greatest influence upon the variation of the ultimate strength is the variable of the actual full plastic moment that contained

actual yield stress and cross-sectional dimensions. The degree of correlation between the ultimate strength and full plastic moment becomes smaller with an increasing slenderness function.

(2) Welded beams have lower ultimate strengths, $M_u/(M_p)_a$, than rolled beams in the mean value. Large variation of the ultimate strength in welded beams may be due to the large variation in initial lateral crookednesses and residual stresses.

(3) Twenty-one two-span continuous beams with nominally identical cross-sections were tested in seven groups, each group having a different span length under a concentrated load. The parameter (P_p/P_E) versus ($\lambda = \sqrt{P_p/P_E}$) can be effective for evaluating the ultimate strength of both simple and continuous beams.

(4) A numerical data base consisting of information regarding the mechanical properties of steels, steel columns, steel beams and unstiffened steel plates was established.

(5) Variations and fitted distribution functions of material properties are obtained from the test data of 2154. The mean value and the coefficient of variation of yield stresses nondimensionalized by specified minimum values are 1.168 and 0.111, respectively, and the yield stress frequency distribution has a positive skewness.

(6) A total of 1665 individual column test results which had been stored in the experimental data base was retrieved and a statistical

appraisal was carried out to evaluate the experimental column strength. Comparisons were made between surveyed test data and the ECCS and SSRC multiple column curves. New multiple column formulas which explain the test data from a statistical viewpoint more accurately were proposed.

(7) Statistical assessments were made for test data of 544 beams to evaluate the beam strength under various conditions and to be compared with available strength formulas. Beams were divided into three groups; rolled beams, welded beams and plate girders, and the mean strength curve and the lower bound curve were proposed for each group.

(8) Statistical investigations of unstiffened steel plates were performed to evaluate their strength characteristics under various conditions. The results of these investigations were compared with the available strength formulas. Plate strengths were categorized into two groups based on the presence or absence of residual welding stresses. Two plate strength formulas which explain the test data more accurately were proposed using the statistical results based on the experimental data base.

9-4 FUTURE WORK

The following subjects are required for future research and development.

Numerical Data Base

CONCLUSIONS

(1) Experimental data of structural items such as beam-columns, stiffened plates, plate girders, structural systems, etc. need to be stored in the data base.

(2) Data of initial imperfections such as residual stress and initial out-of-straightness need to be measured and collected for structural members in the practical use.

Beams and Columns

(3) Effective length concept of columns and beams need to be re-evaluated in the inelastic range.

Beams and Plates

(4) Computer simulations for beams and plates need to be performed by using the statistical parameters which are estimated from the data base approach.

(5) The interaction strength between the lateral-torsional buckling and the local buckling of flange and web, especially for plate girders, needs to be investigated experimentally and theoretically.

(6) An experimental investigation for high strength steel plates with plate slenderness parameters near the limiting value is needed to obtain for increased efficient use of high strength steels.

AUTHOR'S RESEARCH ACTIVITY

Publications

1. "Strength Variation of Laterally Unsupported Beams", Journal of the Structural Division, ASCE, Vol. 106, No. ST1, 1980, pp. 165-181 (co-authored with Y. Fukumoto and M. Kubo).

2. "Information Processing and Retrieval of Ultimate Strength of Steel Structural Members", 5th Symposium of Computer Utilization of JSCE, November 1980, pp. 98-101 (with Y. Fukumoto, in Japanese).

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