Structural behaviour including hybid construction

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Structural Behaviour Including Hybrid Construction

Comportement sous charges en incluant les constructions hybrides

Tragverhalten, einschliesslich hybride Tragwerke

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1. INTRODUCTION

In tall buildings,short columns with stocky sections are generally used and sections of beams are also relatively stocky compared with those used in bridges. This selection were motivated from the structural consideration(strength and deformability requirements against large external forces),and from the functional requirements to keep the members as compact as possible. In such a situation, characteristic behaviors of structures using extra-high strength steels will mainly develop in their post yielding and post buckling region.

On the contrally,rather slender and thin-walled members are used in large span bridges. To use the higher strength steels advantageously in these structures, the effective slenderness of members and width-thickness ratio of plate elements must be kept as small as possible,as is obvious from the fact that all slender compression elements have the same Eulerian strength regardless of the yield strength of the material. The optimum combination of those two ratios in the design of a member is a matter of consideration. Residual stress and initial imperfection will affect the strength of those members with intermediate slenderness. The investigation of post buckling and thus ultimate strength state of plate and box girders including the consideration of fatigue effect is another important problem.

This report summarizes the status of knowledge on the topics above mentioned. The problem of crack formation which might occur during or after welding is also surveyed briefly though this topic would be dealt with more extensively in subtheme Vc,"Fabrication and election problems" in this session. Though the assigned theme is the structural behavior of extra-high strength steels,it is considered to be better that the behavior of extra-high strength steels should be discussed in the comparison with those of mild steels and high strength low-alloy steels which are currently in use all over the world.

This report will,of course,not be complete since it is impossible to know of,and evaluate,all the research which has been conducted everywhere. The writers would,therefore,welcome any corrections and supplements as well as contributions based on new developments.

2. MATERIALS

A large number of steels are currently used for structural applications. These steels may be grouped into three or four general classifications: carbon steels(A), high strength low-alloy steels(B), heat treated high strength carbon steels and heat-treated constructional alloy steels(C). Heat treated high-strength carbon steels,introduced in 1964,are carbon steels that have been heat-treated by quenching and tempering to obtain high strength and toughness and are currently produced as plates. A minimum yield stress 552 MN/m^2 in thicknesses of 19 mm and less,and 483 MN/m2 in thicknesses over 19 to 38 mm is available,and thus it will fill the need for a constructional steel intermediate between B class steels and C class steels above cited. Steels in this category are,however,not yet covered by specifications. All the steels discussed are weldable with no loss of strength, but the welding materials and procedures must be in accordance with approved methods. Typical stress-strain curves for those structural steels are shown in Fig.1.

A knowledge of the stress-strain relations take on during the elastic and plastic ranges of behavior is an essential requisite to structural analysis. In elastic range there are accepted average values of the modulus of elasticity E, while the characteristic values in plastic range are not so firmly recognized,though they are essential to calculation of inelastic strength and deformation of members and frames. Properties which characterize the plastic range are: σ_y =yield stress level,ε_{st}=strain at initial strain hardening,$E_{st}=(d\sigma/d\varepsilon)\varepsilon_{st}$=strain-hardening modulus,and $Y=\sigma_y/\sigma_u$=yield ratio of material (σ_u=tensile strength),which sometimes can be used as an alternative of $E_{st}$and ε_{st} for large strain range. Values of E_{st} and ε_{st} are plotted in Figs.2 and 3 respectively,which were collected from available reports. They are including SS41,G40.8,A36 for carbon steels(A),SM50,SM53, G40.11,Fe52,A441,A572 for high strength low-alloy steels(B) and SM58,HW80,A514, (CT100) for heat treated constructional alloy steels(C).

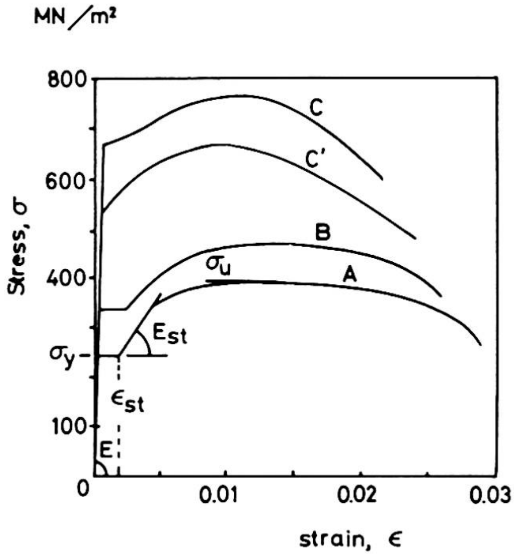


Fig.1 Typical Stress-Strain Curves for Structural Steels

Specified mechanical properties of various grades of high strength structural steel through various countries are tabulated in APPENDIX[1,2], which will offer a convenience to the reader who may be unfamiliar with a designation used in this report and wishes to identify it with grades with which he is familiar. Typical mild steels are included for reference and which are identified by # mark. More detailed information of those groups of steel is given in Refs.3 and 4.

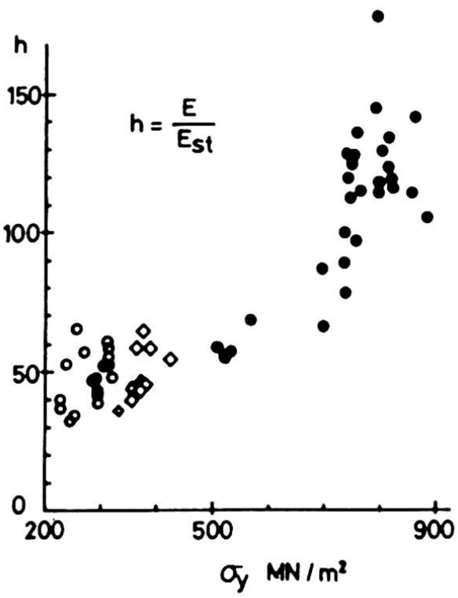


Fig.2 E_{st} vs. σ_y Relation

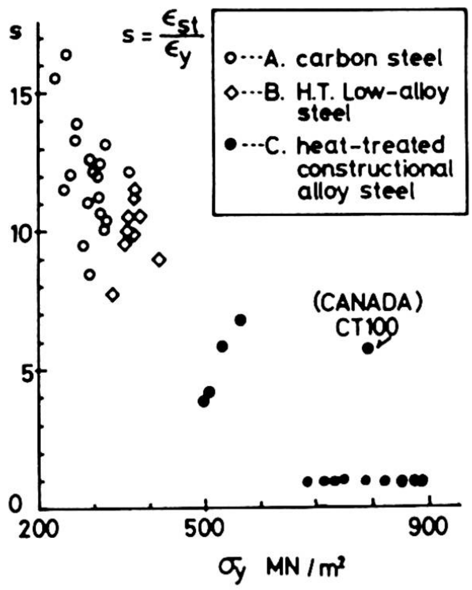
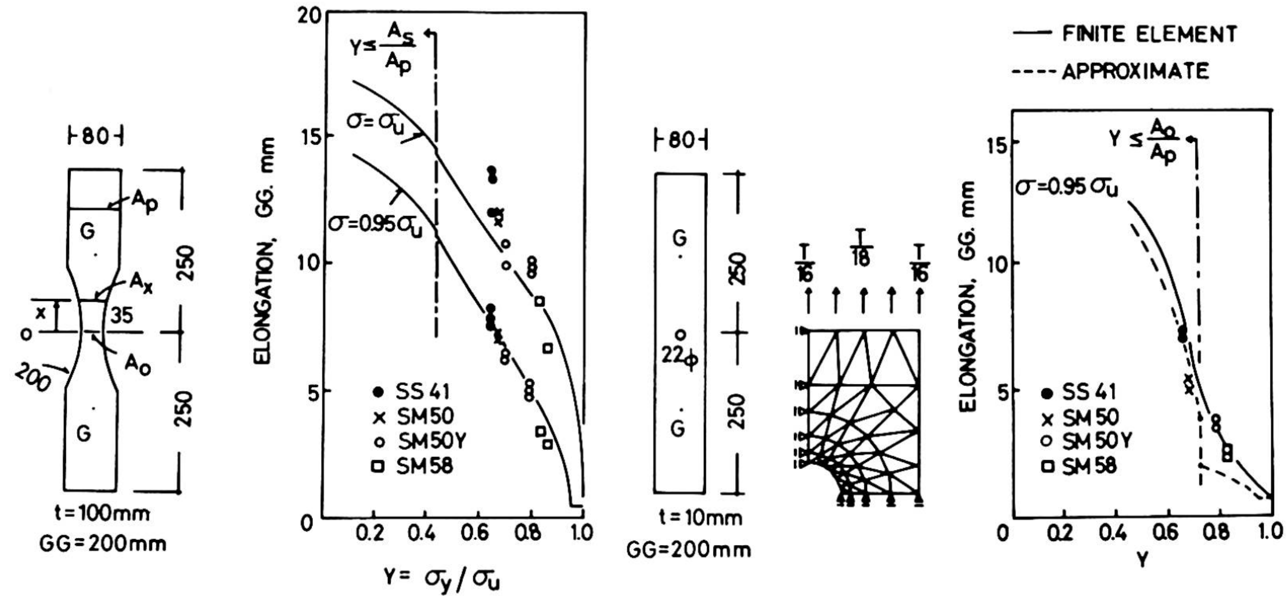


Fig.3 ε_{st} vs. σ_y Relation

3. TENSION MEMBERS

Although tension members are commonly designed on an elastic basis,it is nevertheless considered desirable that as this level is exceeded and the structure proceeds towards its ultimate load,capacity for distortion should still exist in the member. It is the recognized design philosophy that structures in strong seismic area should have enough plastic deformation capacity in horizontal direction so as to be able to absorb the input of earthquake energy. In Tokachioki Earthquake(1968,Japan),diagonal bracings of many steel structures had been broken off at their rivetted or bolted holes or at their threaded parts of end connections without substantial plastic elongation of the members as a whole.

In Ref.5,deformation capacity of tension plates with variable cross section was studied theoretically and experimentally. Consider a model shown in Fig.4(a), cross section of which varies continuously along the length and has the minimum cross sectional area Ao at O. The yield will occur at O when the tensile force equals $Ao\sigma_y$,but no plastic deformation can be observed at this state as yield line at O can not spread.



(a) Tapered plate (b) Prediction & Test results (c) Perforated plate (d) Finite-element subdivision (e) Prediction & Test results

Fig.4 Deformability of Tension Plates of Variable Section

The spread of plastic region along the length is only possible when strain-hardening takes place at O corresponding to the increase of tension. The maximum spread of the plastic region x is determined by the condition that $Ao\ \sigma_u = Ax\ \sigma_y$ or $Ao = Y\ Ax$, where σ_u is the tensile strength of the material,$Y=\sigma_y/\sigma_u$=yield ratio of material and Ax is the sectional area at x. As an extreme case,if the material be elastic-perfectly plastic,namely Y=1,no spread of plastic region could be expected and thus the bar would break off in a brittle manner as soon as the stress at minimum cross section reaches yield point. In Fig.4(b),test results are compared with the prediction obtained from the above simple analysis. When the change of sectional area along the length is steep as in cases of bolt or rivet hole,the effect of stress concentration can not be ignored. This case was analysed by means of finite element technique allowing for the elastic-plastic-strain hardening relationship of the material for the model (c) in Fig.4. Theoretical prediction and test results are compared in Fig.4(e). For both cases,correlation between test results and theoretical prediction seems to be satisfactory. From this study,it can be said that if a tension member system should have enough deformability,the following condition should be satisfied at its connections,

$$An \geq Y\,Ag \tag{1}$$

where, An=net cross sectional area at bolted,riveted or threaded part
Ag=gross cross sectional area of a tension member

Similar tests were carried out using A514 steel plates and arrived at the same conclusion[6]. Tension tests of large,bolted butt splices of A514 steel fastened by A490 bolts had also confirmed above conclusion[7],and pointed out that "A514 steel joints using A490 bolts do not produce yielding on the gross section if the elements of the joint are designed according to current AISC specification (1967)".

4.BEAMS AND BEAM-COLUMNS, -IN PLANE BEHAVIOR-

In-plane behaviors of beams and beam-columns will be discussed,i.e. it is assumed that local buckling and lateral-torsional buckling are precluded in the present discussion. Those problems will be surveyed later herein. As far as elastic behavior is concerned,essentially there would be no difference among different grades of steels since they have common modulus of elasticity. On the contrary, plastic behavior of those members will depend largely on the grade of steel.

Moment-axial thrust-curvature diagram of a wide flange section made of carbon steel(A) and of heat-treated constructional alloy steel(C) are shown in Fig.5 [8,9]. Assumed stress-strain relationship of these two steels are depicted in the figure.Though a typical wide-flange shape was chosen here,it is known that the moment-curvature diagrams are almost identical for all practical wide-flange shapes. Remarkable difference can be seen between these two diagrams and this difference will reflect on the load-deformation curves of members since the latter is obtained by integration of the former over the length.

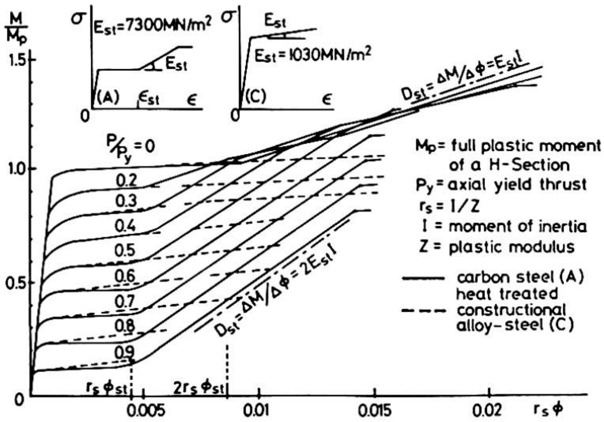


Fig.5 Moment-Thrust-Curvature Diagram

4.1 Beams

It is widely recognized that the load-deflexion curve of a beam subjected to uniform moment shows the similar pattern to that of stress-strain diagram of the material used since yielding is spreaded over the whole uniformly bent segment. Thus beams of A and B class steels show the flat plateaus after yielding which correspond to the yielding flow portions of stress-strain curves of their steels. Increase of the moment due to strain-hardening will be observed only after the large deformation had taken place which is immaterial from the structural view point. In beams of C class steels,the increase of moment due to strain-hardening will occur immediately after attaining full plastic moment because these steels have almost no yield plateau(see Fig.5),but as the strain-hardening modulus of these steels are very small,this upgrade slope is negligible,and thus the plastic behavior of beams under uniform moment is almost the same for all grades of steels and enough ductility can be expected.

In contrast,yielding in the beam under moment gradient can not spread unless the moment is increased,and so strain-hardening sets in as soon as the full plastic moment is reached,resulting in the upward swing of the curve. This behavior is illustrated by a schematic picture in Fig.6. The maximum load is approximately defined by M_p/Y,and hence the maximum increment of the moment of C class steels,Y of which is very high,is very small. Behavior of A class steels is shown in solid line and that of C class steels is shown in dashed line in the figure.

Although this difference will influence the sequence of hinge formation, rigidity and capacity of energy absorption of a structure in the plastic region when these beams are framed in, the more important is the problem of ductility. In Ref.10,it was reported that tension flanges of two A514 steel beams built up by welding,had ruptured at the vicinity of the concentrated load. These beams were simply supported and had a concentrated vertical load at mid-span as shown in Fig.7. Both beams developed full plastic moment in the vicinity of the concentrated load,and in the course of sustaining continuously increasing load abrupt unloading occured due to a rupture of the tension flange. In one beam,the rupture proceeded straight across the flange and vertically through the web and the weld of the stiffener. In another beam,the rupture extended diagonally across the flange and through the base metal of the web. Both ruptures stoped before reaching the compression flange. Yield stress and tensile strength of tension flanges of both beams were 766 and 849 MN/m^2 respectively and thus the yield ratio was 0.902.

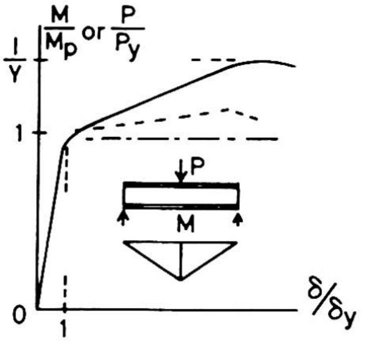


Fig.6 Lord-Deflexion Curve

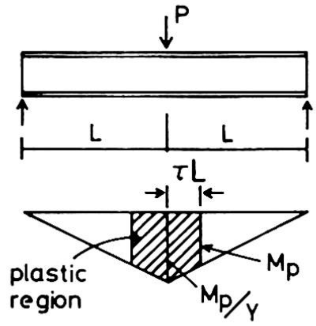


Fig.7 Spread of Plastic Region

This unusual behavior could be explained as follows: In Fig.7,when the maximum moment M_p/Y is attained at the concentrated load,the spread of the yielding of tension flange along the length of the beam τL is determined from the bending moment diagram as $\tau=1-Y$. Note that τ is only the function of the yield ratio of the material and independent of the span of the beam and the moment gradient. In the present case,Y=0.902 and τ is calculated to be 0.098 which means that only one-tenth of the span can be yielded even at the ultimate strength state,and then tension flange will break off without developing enough rotation of the beam.

In general,rotation capacity of a beam had been governed by the local buckling of the compression flange and/or lateral-torsional buckling. But for beams of extra-high strength steel with high yield ratio,deformation capacity of tension flange might become another criterion of the rotation capacity as seen above.

4.2. Beam-Columns

When a beam-column is subjected to end moments under constant axial force, the moment will reach a peak and thereafter unloading will take place,and frequently it will not be possible to achieve full plastic moment M_{pc}. This reduction in both bending and deformation capacity differenciates the performance of a beam-column from that of a beam. The reduction in moment capacity is due to the combined effect of the "secondary" moment introduced by the axial force times the deflexion and the reduction of stiffness due to yielding.

Moment-end rotation(M-θ) or moment-deflexion(M-δ) relationships can be obtained by integrating the moment-thrust-curvature relationship as was given in Fig.5. Though the determination of M-θ or M-δ curve is performed by numerical integration because of the complicated nature of cross sections,the numerical integration procedure is very simple in concept. The important point is the use of realistic stress-strain relationship of the material and thus the use of realistic moment-curvature relationship of the member section to assess the behavior of beam-columns made of different grades of steel.

The most recent research on this topic has been summarized in Refs.11 and 12. Among the many available solutions,that given in Ref.13 is refered here,because it represents so-called "exact" solution including the influence of residual stress on column strength,and is directly applicable to columns fabricated from as rolled wide-flange shapes which are subjected to bending about their strong axis.

In the development of the theory,it is assumed that, a)stress-strain relationship is elastic-perfectly plastic(no account for strain-hardening),b)the residual stress pattern used is typical for rolled columns of A36 steel and the maximum compressive residual stress is $0.3\sigma_y$ (there are some evidences to show that the magnitude of the residual stress is independent of the yield stress of the material which will be discussed later). Tests carried out on beam-columns of A class steels(A7,A36, A37,SS41)[14,15,16,17] and of B class steels(A441,SM50)[18,19,20] had confirmed this solution well. Based on that solution,tables for the ultimate strength of eccentrically loaded beam-columns have been furnished in Ref.21. Although the tables were furnished for steels with a yield stress of 33ksi.(228 MN/m^2),it is suggested the tables can be applied to steels of other yield points by substituting a modified slenderness L/r:

$$(L/r)_{mod.} = L/r\ \sqrt{\sigma_y/33}$$

In some design situations,particularly when resistance to earthquake motions and blast shocks is involved,it is necessary to count on the rotation capacity or deformability of beam-columns. In Ref.22, rotation capacity of beam-columns is surveyed. Available test results show that the rotation capacities are relatively small when compared to beams though they were for relatively long beam-columns under heavy axial load. Rotation capacities of the order of 4 to 13 were reported on tests of short beam-columns with one end moment[23]. Though above informations of rotation capacities are limited on beam-columns made of mild steels, AISI design manual on "Plastic Design of Braced Multistory Steel Frames" allow the higher strength steels to use the M-θ curves furnished for A36 steel by modifying the slenderness and rotation by the following equations:

$$(L/r)_{equiv.} = (L/r)_{actual} \cdot \sqrt{\sigma_y/36} \qquad (2)$$

$$\theta = \theta_{chart} \cdot \sqrt{\sigma_y/36} \qquad (3)$$

where σ_y is the yield stress of the particular steel expressed in ksi.[24].

The method of modifying the slenderness and the end rotation by the square root of the ratio of yield stresses as was proposed in eqs.2 and 3,and in Ref.21 seems to be reasonable as far as the elastic-perfectly plastic relationship of the material is assumed. When beam-columns become shorter,material properties in plastic region such as E_{st},ε_{st} and Y will play the more important role, while the effect of the secondary moment will become less significant,and thus the ignorance of the strain-hardening will cause the errors of conservative side. And if the M-θ charts are constructed on the basis of the realistic σ-ε relationship of a particular grade of steel,the adoption of the cited method of modification is only possible when the σ-ε relationship of the other grade of steel which is intended to apply is exactly similar to that used in making the charts.

Considering the substantial difference of σ-ε relationship which exists between mild steels and extra-high strength steels, the adoption of that modification method to short columns of extra-high strength steels would produce a poor prediction.

Cantilever beam-columns subject to horizontal shear force Q under constant axial force P at the top of them were analysed for two different grades of steels based on the realistic σ-ε relationships of respective steels,and the obtained interaction curves are shown in Fig.8[25]. Mechanical properties of the steels in plastic region are shown in the figure. Yield stress of SM58,though it belongs to C class steel,is not so high and E_{st} is not so low,but interaction curves differ remarkably from those of mild steel(SM41). Consider a curve for L/r=13.6 of SM41 in Fig.8(a). When the steel is replaced by SM58,the equivalent slenderness is calculated as $(L/r)_{equiv.}=13.6\ \sqrt{499/262}=18.8$ according to eq.2,and the curve for L/r=18.6 in the figure can be considered as the approximate equivalent curve to be applied. This curve shows the remarkable difference from the actual interaction curve for SM58 with L/r=13.6 as is shown in Fig.8(b).

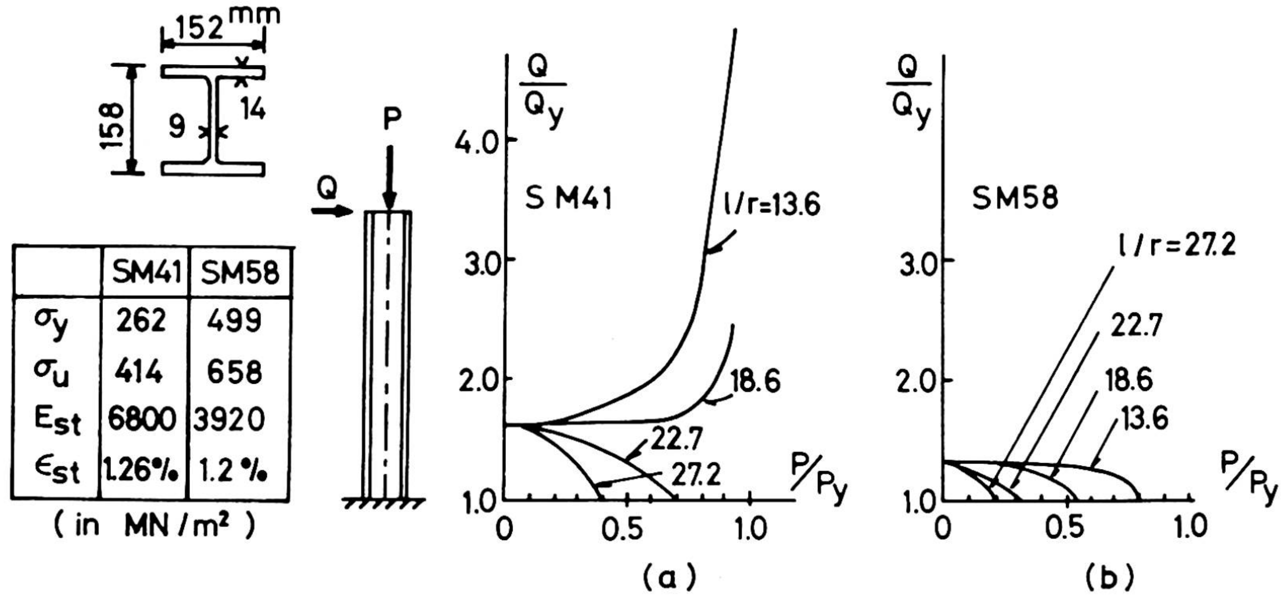


Fig.8 P-Q Interaction of Beam-Columns

5. BUCKLING

The previous discussion concerned the in-plane behavior of beams and beam-columns. Insofar as in-plane behavior is concerned,deformability of beams are unlimited unless flanges of them are broken off,and beam-columns,though they have peak moments due to the combined effect of the "secondary" moment and the yielding of material,often exhibit a rather slow falling-off in the descending part of their M-θ curves. This optimal behavior is curtailed by the occurence of lateral-torsional buckling and local buckling.

Flexural buckling is the only phenomenon when columns are compressed centrally which will scarcely occur in practice. Nevertheless,this concept forms a generally accepted basis for column strength and design.

Research on the buckling problems in the past decade was concentrated to those of members with intermediate range between the small and large slenderness wherein secondary factors,such as residual stress,initial crookedness and eccentricity have the greatest effect on the buckling strength of members. A comprehensive review of the research on these problems carried out by 1966 is given in Ref.11,and theoretical predictions,experimental results and design formulae reported by the end of 1969 are summarized in Ref.26. Main findings obtained from those researches which are pertinent to the high-strength steels are:

Columns

1) As for thermal residual stress introduced in rolled shapes,the magnitude of the compressive residual stress appears to be independent of yield stress level [19,27,28,29]. This gives a favourable situation to high-strength steel columns.
2) For welded built-up H-shapes(irrespective of grades of steel) the maximum tensile residual stress closely approaches the yield stress and the adverse effects of the concurrent compressive residual stresses are large[30].
3) The ultimate strength of A514 steel columns of circular cross section with initial curvature and with both concentric and anti-symmetric residual sresses,the former of which is introduced by cooling process and the latter is introduced by cold-straightening of columns,was investigated theoretically and experimentally [31,32,33,34]. This research verified the proposed numerical procedures to determine the ultimate strength of columns under these conditions.

Based on these researches,a systematic evaluation of the effect of residual stress and initial curvature on the column strength of wide-flange sections has been made[35]. Residual stress was held at a constant level of 69 MN/m^2. It has been shown that the maximum effect of either residual stress or initial crookedness,alone or in combination,always occurs at the slenderness which yields the Euler critical stress equals σ_y. The effect of residual stress is less pronounced for higher strength steels than it is for mild steel,thus as the yield stress increases,initial curvature and eccentricity take on increasing importance in relation to residual stress.

Beams and beam-columns

Recent investigations on lateral-torsional buckling of beams and girders have been reviewed in Ref.36. The strength of a laterally unsupported beam of relatively short length,like that of a corresponding column,will be determined by inelastic rather than by elastic behavior.

An approximate method of estimating the effect of plastic action on the buckling strength of beams and girders is to assume that the relationship between elastic and inelastic buckling strength is the same for beams as it is for columns. The inelastic buckling strength of beams can then be estimated from a column curve. This approach was evaluated by comparison with a theoretical solution[37],and also was confirmed by experiments[38].

A review of solutions of lateral-torsional buckling of beam-columns in inelastic range is given in Ref.39. Though few analytical solutions of this problem are available[40,41],the nonlinearlity of σ-ε relationship makes it extremely difficult to express explicitely the variation of the several quantities that must be evaluated. Because of this difficulty,several approximate types of solutions are generally adopted. Among those approximate solutions,the following modified interaction formula[42] is used in specifications most widely.

$$\frac{P}{P_u} + \frac{Mo}{Mu(1-P/P_e)} \leq 1 \qquad (4)$$

where,

P=applied axial load

P_u=axial load producing failure in the absence of bending moment(including the possibilities of buckling in the weak plane)

P_e=elastic critical load for buckling in the strong plane

M_o=maximum applied moment,not including the "secondary" moment.

M_u=bending moment producing failure in the absence of axial load(including the possibilities of lateral buckling)

Eq.4 was confirmed by an experiment[14].

More recent researches on buckling of members made of high-strength steels carried out after 1966,i.e. after the edition of Ref.11 are briefly reviewed below: Column tests of built up box-sections made of HW80 steel loaded centrally and eccentrically were carried out[43],and confirmed the theoretical estimations of Ref.35 cited previously. A series of research on British new high-strength steel Grade 55 were carried out in order to formulate design rules. This steel is denoted Pearlite-Reduced Structural Steel and minimum yield stress is 447 MN/m^2. 130 strut tests(concentric loading) were carried out as one project[44],and demonstrated that the test results closely related to the prediction of Perry-Robertson formula,and thus showed that the BS449 formula could be safely extrapolated to this steel. As another project,30 tests were carried out on beams of universal I-sections under a uniform bending,including the measurements of thermal residual stresses[29]. It was shown that the bending stresses for the design of beams in Grade 55 steel could be determined by the method given in BS153(1958) and BS449 (1959) with some amendments.

A simple method of design of laterally unsupported beams which covers all grades of steels was presented and this prediction was compared with test results given in Ref.29 above to show a good agreement[45].

Compression tests of square columns built-up by welding were carried out to determine the buckling strength of high-strength steel plate elements. Steels used were A514 in one test[46],and HW80 in another test[47]. The following conclusions were obtained in both tests:

1) Considerable post-buckling strength exists in a plate buckled in the elastic range,while a plate buckled in the elastic-plastic range has a relatively small reserve of post-buckling strength.

2) The effect of residual stresses on the buckling strength of a plate is less pronounced for these C class steels than it is for A class steels.

3) The plate elements(with intermediate width-thickness ratios) of these C class steels are stronger than those of A class steels when compared on a nondimensional basis.

The foregoing concerned the buckling of members of intermediate range. Ultimate strength and deformability of members in extreme ranges which subject to local and/or lateral buckling are another current topics: The research of the ultimate strength of thin-walled structures,in one extreme,is very important in the design of plate and box girders,and this topic,including that of hybrid girders, will be dealt with separately in another section later herein. The deformability and ultimate strength of members with very small slenderness or width-thickness ratio,in the other extreme,is a topic of increasing interest in relation to the development of the plastic design and of the earthquake resistant design wherein the assessment of the energy absorption capacity of members and frames is most important. Because the behavior is determined almost entirely by the plastic properties of the material in such a short and stocky members,characteristic behavior of extra-high strength steels will be paramount in this range,and this problem will be surveyed hereafter with a special emphasis layed on the deformability or ductility.

5.1 Local Buckling

In plastic analysis it is tacitly assumed that the moment capacity of the member will remain at the level of the plastic moment until enough hinges have developed to form a mechanism. It is,therefore,necessary that the moment capacity not be impaired by local or lateral-torsional buckling until the required rotation has been achieved. In earthquake resistant design,this requirement is more direct to secure the energy absorption capacity.

The first solution to the problem of local buckling in the strain-hardening range was given in Refs.48,49,the applicability of which was limited to A class steels(A36).

Two solutions are refered here which are capable to cover the higher strength steels:

1) In Ref.50,the limitations of width-thickness ratios of wide-flange sections are given as;

For flanges subject to uniform compression along the length,

$$\frac{b}{t} = \sqrt{\frac{G_{st}}{\sigma_y} + 0.381\left(\frac{E_{st}}{\sigma_y}\right)\left(\frac{t_w}{t}\right)^2\left(\frac{2bt}{dt_w}\right)^{1/2}} \qquad (5)$$

$$G_{st} = \frac{2G}{1 + \frac{E}{4E_{st}(1+\nu)}}$$

where, b=one-half of flange width

t=thickness of flange

t_w=thickness of web

d=depth of web

G=shear modulus

In the case of beams under moment gradient,

$$\frac{b}{t} = \frac{3.56}{\sqrt{\varepsilon y(3+ 1/Y)(1+ h/5.2)}} \tag{6}$$

where, $h=E/E_{st}$

The problem is treated as a classical buckling with bifurcation of the equilibrium position under assumptions that local buckling will occur when a)the average strains in the plate are at the strain-hardening strain ε_{st}, b)a long enough portion of the plate has yielded so that at least one-half(for uniform compression) or one full(for moment gradient) wave length of the buckle can develop,and the strain-hardening modulus in shear G_{st} is evaluated by assuming the discontinuous yield process.

2) In Ref.51,the deformation which a plate can develop without reducing its yield strength,expressed in terms of average compressive strain ε,is given as a function of width-thickness ratio and material property as;

For flanges subject to uniform compression along the length,

$$\frac{b}{t} = \left(\frac{2\Phi_1 + \Phi_2}{2\sqrt{2}} \right)\frac{1}{\sqrt{\varepsilon}} \tag{7}$$

where, $\Phi_1 = \dfrac{4(\alpha^2 -1)+(\beta_1-0.5)}{2\beta_1 +1}$, $\Phi_2 = \dfrac{\alpha^2 -1}{\alpha}$, $\beta_1=\sqrt{0.25 +4(\alpha^2-1)}$, $\alpha=1/Y$

For webs subject to uniform compression along the length(unloaded edges are clamped),

$$\frac{d}{t_w} = \sqrt{2}(\Phi_1 +0.5\Phi_3)\frac{1}{\sqrt{\varepsilon}} \tag{8}$$

where, $\Phi_3 = \dfrac{2(\alpha^2-1)-(\beta_2-1)}{\beta_2-1}$, $\beta_2 =\sqrt{1+4(\alpha^2-1)}$

Eqs.7 & 8 are upper bound solutions obtained by assuming a collapse mechanism and then applying the work theorem to it.

Available test results are plotted in Fig.9 for plates supported at one unloaded edge and free at the other. Those test results were reported in Refs.52,53 for C class steels,in Refs.51,54,55 for B class steels and in Refs.48,51,53,54,55 for A class steels respectively. These plotts are compared with the prediction of eq.7 in the figure. Considering the variety of the degree of restraint exsisting among those test specimens,eq.7 seems to explain the general tendency well,though it is too conservative for C class steels.

It should be noted that the test results on C class steels are showing lower $\varepsilon_{max}/\varepsilon_y$ values than those of A and B class even though compared on a nondimensional slenderness basis. This fact seems to show that the axial deformability of flanges is not only a function of the square root of the inverse of the yield stress, but also a function of the material properties in plastic range such as E_{st},ε_{st} and Y. This is the similar situation to that of in-plane behavior of beam-columns as was discussed in section 4.2 earlier.

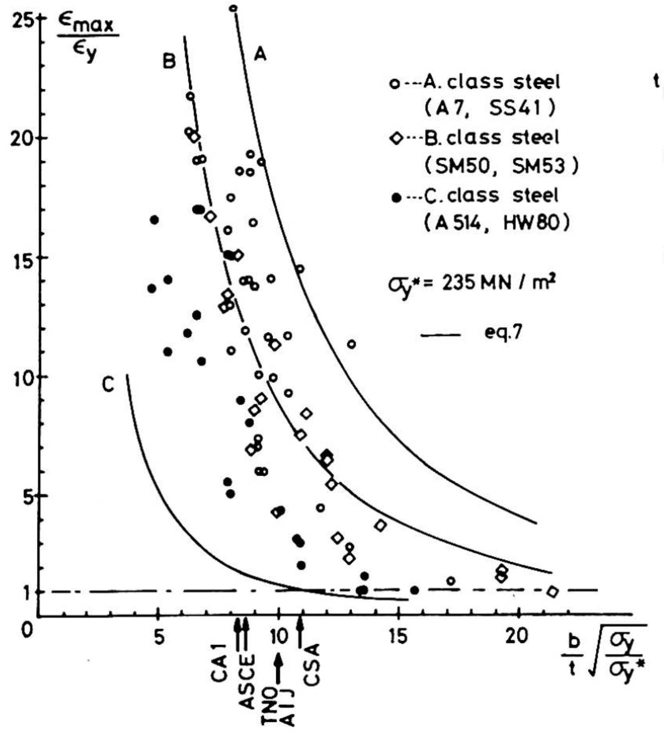


Fig.9 Deformability of Flanges

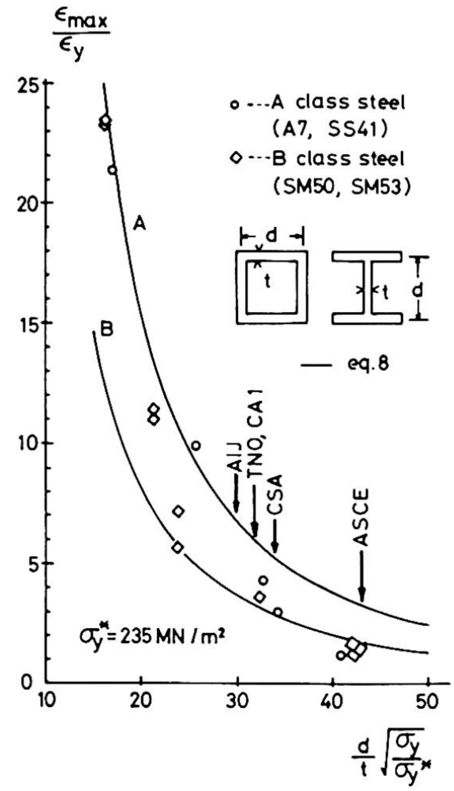


Fig.10 Deformability of Webs

Test results on plates supported on both unloaded edges are plotted in Fig.10. These were reported in Refs.51,55 for B class steels and in Refs.48,55 for A class steels.Prediction from eq.8 is also depicted in the figure. Though the number of test data is relatively small,both are in good correlation.

In both figures,the minimum width-thickness ratios specified by various specifications and reccomendations(AIJ[56],ASCE[12],CA1[57],CSA[58],TNO[59]) are identified,applicability of most of which are limited up to B class steels.

5.2 Lateral-Torsional Buckling

The solutions of lateral-torsional buckling in the vicinity of plastic hinges are available in Refs.60,61 for beams under uniform moment and in Refs.62,63, 64 for beams under moment gradient.

Beams under uniform moment[60]

The analysis is based on the experimentally observed behavior of a segment of a beam. The available test results indicate that the compression flange starts to deflect laterally as soon as M_p is reached. This lateral deflexion increases, while at the same time M_p is maintained and rotation in the plane of bending continues until local buckling occurs in the most compressed portion of the compression flange. Local buckling does not commence until the average strain is equal to ε_{st} at the center of the segment if b/t of the compression flange is equal to or less than the critical ratio given by eq.5. In the analysis it is assumed that the compression flange and one-half of the web act as a column under the yield axial load,and the reduced flange rigidity is evaluated on the basis of a discontinuous yield concept. Thus the critical unbraced length of a uniformely bent simply-supported beam is derived as,

$$\frac{KL}{r_y} = \frac{\pi}{\sqrt{\varepsilon_y}} \frac{1}{\sqrt{1+0.7R(\frac{h}{s-1})}} \tag{9}$$

where,

K=effective length factor,K=0.54 if the adjacent segments are elastic and K=0.8 if the adjacent segments are fully yielded

L=unbraced length of a beam

r_y=the least radius of gyration of the wide-flange shape

$s= \varepsilon_{st}/\varepsilon_y$

$R= \theta_u/\theta_p - 1$ =rotation capacity,in which θ_u is the rotation when the moment capacity reaches M_p on the unloading branch of an M-θ curve,and $\theta_p = M_pL/EI$ =the idealized rotation corresponding to elastic theory applied to the case where $M=M_p$(Fig.11).

Beams under moment gradient[62,63]

In Ref.63,a beam model which is partly elastic and partly strain-hardened is assumed. The differential equations of lateral-torsional buckling are solved,with the elastic moduli E and G in the elastic portion,and with E_{st} and G_{st} in the yielded portions. After some assumptions and approximations having been introduced,the final formula is derived as,

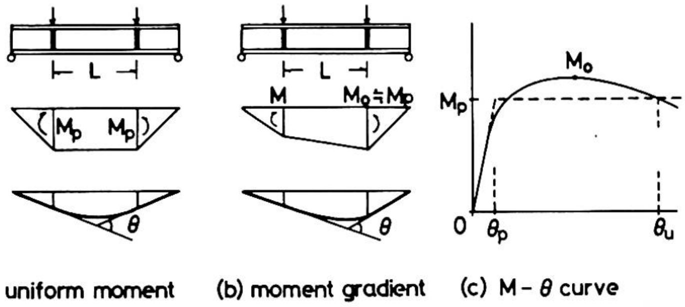


Fig.11 Moment-Rotation Relationship of Beams

$$\frac{L}{r_y} = (60 + 40 \frac{M}{M_p}) \sqrt{\frac{36}{\sigma_y}} \qquad \text{for } M/M_p > -0.625$$

$$\frac{L}{r_y} = 35 \sqrt{\frac{36}{\sigma_y}} \qquad \text{for } M/M_p < -0.625 \tag{10}$$

in which M and M_p are moments as defined in Fig.11(positive in the clockwise direction).

On the other hand,it was shown in Ref.62 that,for practical cases of inelastic beams under moment gradient,failure would be initiated by local buckling rather than by lateral buckling,and thus the lateral bracing spacing should be determined by considering the beam to be under uniform moment. Taking into account of this argument, the following empirical formula is suggested in Ref.12 as an alternative of eq.10,

$$\frac{L}{r_y} = \frac{1375}{\sigma_y} + 25 \qquad \text{for } -0.5 < \frac{M}{M_p} \leq 1.0 \tag{11}$$

In eqs.10 and 11, σ_y is expressed in ksi..

5.3 Rotation Capacity

In previous sections,local buckling and lateral-torsional buckling were treared as independent problem,namely provisions against local buckling and lateral-torsional buckling were made by limiting width-thickness ratio and unbraced length respectively based on an approximation given in terms of the strain-hardening strain ε_{st}. But actually those two phenomenon are intricately interconnected with each other. Hence,the deformability of beams and beam-columns can be assessed more directly by evaluating the rotation capacity R which had been defined earlier. Rotation capacity is also a good measure of earthquake resistance capacity of members and structures as R and dissipated energy are linearly related to each other.

Beams under uniform moment

The relationship in eq.9 connects the geometrical and material properties of a beam under uniform moment with the rotation capacity R. Many experimental investigations were carried out on beams under uniform moment with elastic and inelastic adjacent beams. Tests reported in Ref. 65 include A,B and C class steels,and beams tested in Ref.10 were C class steel,and those in Refs.66,67 were of B class steels and those in Refs.68,69, 70 were of A class steels. The comparison between those experimental rotation capacities and theoretical curves from eq.9 are shown in Fig.12. The theory explains the test results very well though it is rather conservative. But it does not give a useful information on beams with C class steel, because s= $\varepsilon_{st}/\varepsilon_y$ of C class steel is almost unity and then eq.9 always gives R=0 regardless of the value of λ.

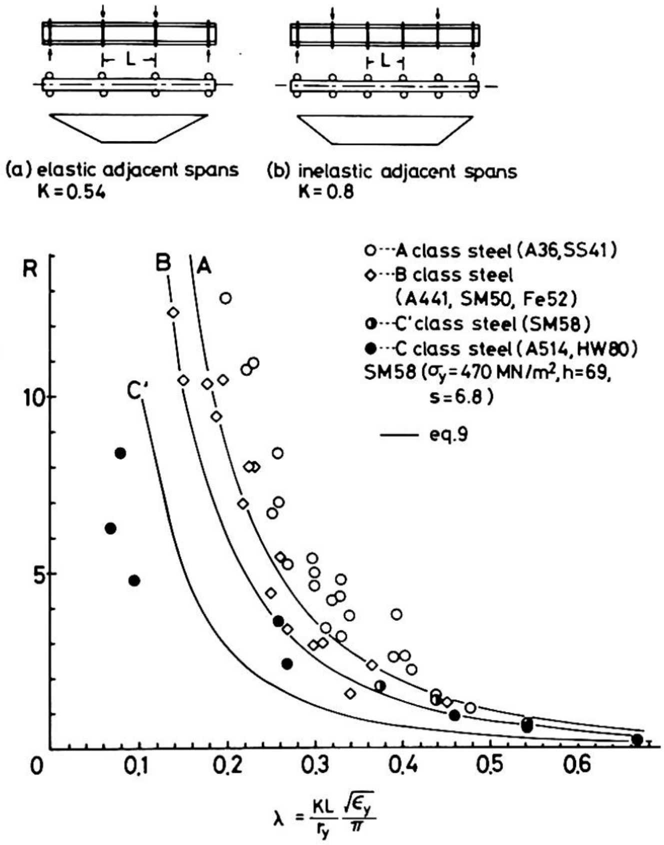


Fig.12 Rotation Capacity under Uniform Moment

Beams under moment gradient

There are some solutions of this problem[62,71],but they are not given in an analytical form and hence can not indicate the significance of the various parameters involved. A linear relation between R and a parameter $\Lambda^{-2}(t/b)(\sigma_{yo}/\sigma_y)$ was suggested in Ref.72. Available test results are plotted in Fig.13,in which a modified parameter $\Lambda^{-2}(t/b)(1+\rho)^2(\sigma_{yo}/\sigma_y)$ is taken in horizontal axis,where Λ=KL/r_y, $\rho=M/M_p$(moment ratio,positive in clockwise direction) and σ_{yo}=248 MN/m^2=reference yield stress. These tests were reported in Refs.73,74,75,76,77,for A class steels, in Ref.78 for A and B class steels and in Refs.67,79,80 for B class steels and in Ref.72 for A,B and C class steels. In the tests of Ref.78, (t/b) was variable being kept Λ as constant,while in Refs.72,77, Λ was variable being kept (t/b) as constant.

Test results are rather scattered. Roughly speaking,a bi-linear relationship can be observed between R and $\Lambda^{-2}(t/b)(1+\rho)^2(\sigma_{yo}/\sigma_y)$,and there seems to be a kind of yield point above which substantial increase of R could not be expected regardless of the value of $\Lambda^{-2}(t/b)(1+\rho)^2(\sigma_{yo}/\sigma_y)$. There are another group of test results which appears to form a similar bi-linear relationship. In these tests,the stiffness and strength of lateral bracings and their inevitable restraint against rotation about weak axis will influence the test results. The evaluation of effective length factor K is also a matter of consideration. These limited number of test results are concerned so far,rotation capacity is inversely proportional to the yield stress.

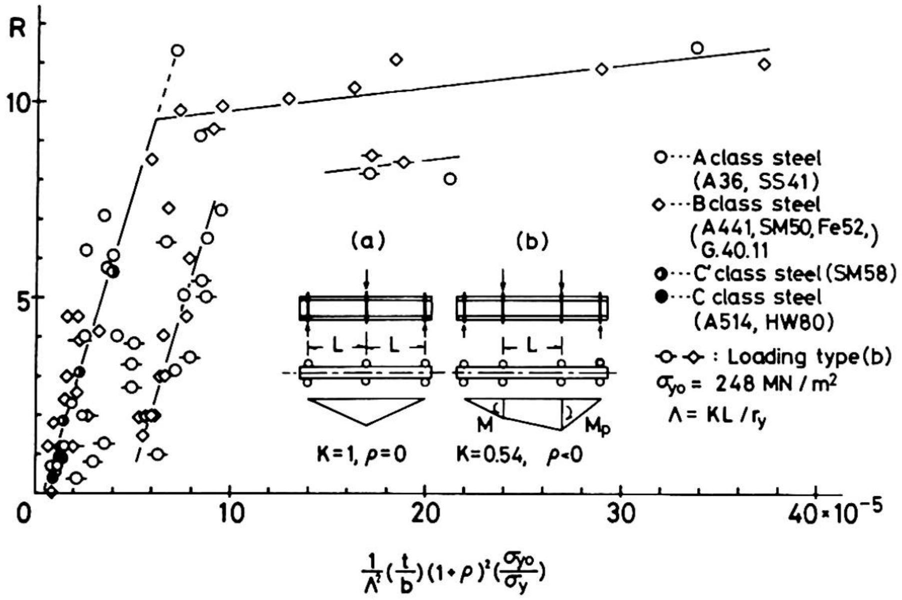


Fig.13 Rotation Capacity under Moment Gradient

Though above discussions were all on steel beams themselves,the restraining effect of floor slabs should be taken into account to understand the realistic behavior and to obtain the more economical design of tall buildings.

A number of experiments on the lateral-torsional buckling of unbraced wide-flange beam-columns were carried out[15,18,23,81,82,83]. But neither any analytical information nor systematic research are available yet on the rotation capacity of these inelastic beam-columns. In Ref.84,it has been recommended that for beam-columns in single curvature bending,the bracing rules for beams under uniform moment should be applied,and for the moment at one end only,the rules for beams under moment gradient apply if the axial load ratio P/P_y obey the relationship

$$\frac{P}{P_y} \leq \frac{1-L/r_x(1/\pi)\sqrt{\sigma_y/E}}{1+L/r_x(1/\pi)\sqrt{\sigma_y/E}} \qquad (12)$$

If P/P_y exceeds this limit,the bracing should be proportioned according to the rules for beams under uniform moment. In case of full double curvature the bracing spacing rules for beams under moment gradient apply.

6. FRAMES

The strong column-weak beam concept is popular in frame design,where the majority of the plastic hinges develop in the beams and the inelastic action in the columns is limited. The British approach to the plastic design of tall frames is based on the deliberate excludion of plastic hinges from the columns[85,86]. Most of earthquake resistant structures also have been designed according to this concept. This design method seems to has been adopted because a)the analysis becomes simple, b)the problem of lateral-torsional buckling of beam-columns has been studied inadequately as was reviewed in previous section. However,this concept should be carefully evaluated with the following facts in mind; 1)If the lateral-torsional buckling of beam-columns is studied adequately and the possibility of premature failure is eliminated,the plastic behavior of beam-columns may be superior than that of beams since the flexural rigidity of beam-columns in plastic range is approximately twice that of beams as is obvious from the M-P-ϕ relationship given in Fig.5. 2)Actual beams will be strengthened in such cases that, a) they are connected with floor slabs by shear connectors, and b)beams are jointed rigidly both in x and y directions to an internal column and the frame is subjected to horizontal force in oblique direction to the plane of bays(x and y),wherein both beams in x and y directions resist to the horizontal force synthetically. In such a situation,plastic hinges will apt to form in the column even if weak-beams were assumed in the initial design,and when columns were not designed adequately to prevent local and/or lateral-torsional buckling in plastic range, premature failure will occur in columns.

Present discussion,however,will be restricted to planer steel frames because the knowledge on the inelastic behavior of space frames and of composite beams is not enough to be reviewed generally.

Multistory frames can be categorized in two types: "braced frames" where the primary resistance to lateral loads,frame buckling and frame instability is provided by a vertical bracing system, and "unbraced frames" in which the bending resistance of the frame members themselves must account for the total frame strength and stiffness in resisting lateral loads and frame instability.

Plastic behavior of "braced frames" was investigated extensively at Lehigh University. Tests and analysis on subassemblages and full size frames were carried out for A class steel(A36) and B class steel(A441)[87,88,89,90,91]. Results of these researches were incorporated into AISC specification[92] and AISI design manual[24].

In Britain too, the research of the braced multistory frames was developed and design recommendations were published from Joint Committee of the Institute of Welding and the Institution of Structural Engineers[86,93],where the use of high-strength steel up to Grade 50 was permitted. One full-scale frame fabricated in Grade 43 steel[94],and another full-scale frame made of Grade 50 steel[95], both of which were designed in accordance with the recommendations of the reports, were carried out to establish the accuracy of the simplified design method proposed therein. Hybrid construction was adopted in the latter frame,namely Grade 50 steel was used in columns while Grade 43 steel was used in beams. Column-to-beam connections in this frame were of semi-rigid type,and the frame collapsed in beam mechanism type.

Research on unbraced frames is in progress. A large number of tests had been carried out on unbraced frames made of A class steels. Plastic design procedures of unbraced multistory frames are proposed[96,97,98]. A test on unbraced frame made of B class steel(A441) under combined gravity and lateral loads was reported in Ref.99. This frame is a hybrid frame,where A441 steel is used in columns and A36 steel is used in beam. The test showed the behavior of the frame could be predicted by methods conventionally used for mild steel frames.

When high-strength steels are used in unbraced tall frames subject to combined gravity and lateral loads,the following factors will influence their load-deformation relationship more strongly than they will do on mild steel frames: High strength steel columns will carry the relatively higher axial forces and these axial forces will a)introduce additional moments due to P-Δ effect, b)reduce the elastic stiffness of the beam-columns and c)introduce additional moments due to member shortening(bending deformation of frames). In plastic range, strain-hardening effects become remarkable. These effects are illustrated diagrammatically in Fig.14 taking an example of lateral load-deformation relationship of the lower story of a tall building. Q_y and Δ_y are lateral load and horizontal sway respectively,at which the first hinge develops in the frame. Of all,the effects of P-Δ moment and strain-hardening are dominating. Two pairs of model frames,one of which is

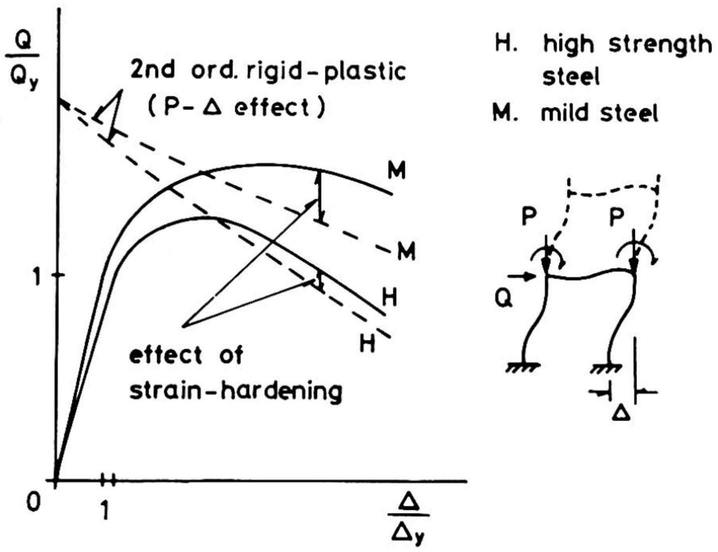
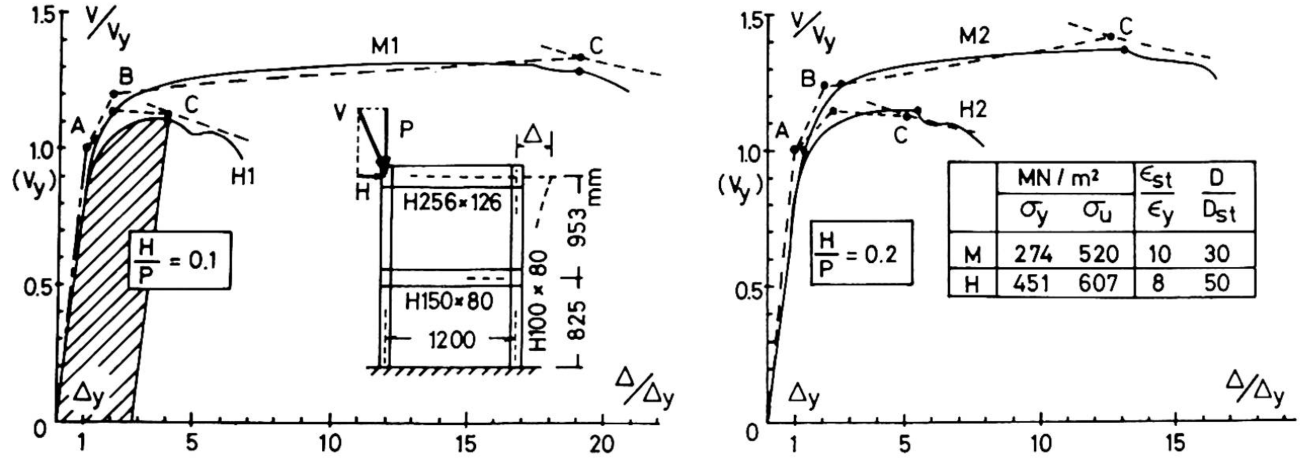


Fig.14 Load-Deflexion Curve of Frames

	M1	H1	M2	H2
V_y KN	1215	2332	823	1558
Δ_y mm	6	12	9	14.5



	MN/m² σ_y	MN/m² σ_u	ϵ_{st}/ϵ_y	D/D_{st}
M	274	520	10	30
H	451	607	8	50

Fig.15 Test Results of Frames of Different Grades of Steels

made of A class steel(SS41) and another is made of C' class steel(SM58), were tested to compare their deformability[100]. Frames were under combined vertical and horizontal loads,and were loaded up to the collapse in proportional loading condition. The ratio of horizontal loads to vertical loads was kept to 0.1 for one pair of frames(M1,H1) and to 0.2 for another pair of frames(M2,H2). Load-deformation curves of these frames are shown in Fig.15,where V_y and Δ_y are the resultant load and the corresponding horizontal deflexion respectively at which the first hinge develops in the frame. Frames made of A class steel are indicated by M and those made of C' class steel are indicated by H. Theoretical predictions are depicted by dashed lines. Difference of deformability (ductility) between frames of A and C' class steels is quite remarkable. Dissipated energies are calculated(with respect to H1 frame,for example,the energy was calculated from the shaded area,assuming that P≒V,and ignoring the work done by the vertical load), and they are 151 KJ for M1,82 KJ for H1 and 224 KJ for M2,208 KJ for H2. Again, mild steel frames are superior than high-strength steel frames from the view point of energy absorption capacity.

7. PLATE AND BOX GIRDERS

As the conclusion of the 8th Congress of IABSE,held at New York in September 1968,concerning the design of thin walled deep plate girders[101],it was pointed out that "the linear theory of plate stability is not an adequate basis for design of struts and girders consisting of thin-walled sections. Such design method must consider the initial geometrical imperfections of the plates as well as the residual stresses and the different yield stresses over the section and over the length of the structural member" and furthermore,the Association has recommended that the study on this theme should be continued. As a result of this recommendation a colloquium on "Design of Plate and Box Girders for Ultimate Strength" was held in London in 1971 under the sponsorship of IABSE[102],wherein main topics of a)ultimate strength of plate girders subjected to shear,-plate girders without intermediate stiffners, b)general analytical methods,-ultimate strength of plate girders subjected to bending and to shear and bending and c)hybrid girders,fatigue problems, effects of concentrated loads,box girders,special problems, were discussed thoroughly by some twenty specialists from all over the world.

The failures of four large steel box girder bridges occured between 1969 and 1971 emphasize the need for more research in this field. A striking test result on a box girder was reported in that Colloquium[103] that the mean collapse stress had been less than the critical stress given by the linear buckling theory for an ideally perfect flange. An international conference on "Steel Box Girder Bridges" organized by the Institution of Civil Engineers was held in London in 1973 where again the importance of the introduction of the ultimate strength concept to the design was emphasized[104].

It must be beyond the scope of this report to review and discuss on the structural concepts,the methods of analysis and the experimental observations presented in above colloquium and conference. But if one might be permitted to add a comment,most of the contributions presented seem to have been devoted to the construction of collapse models and the analyses of them,and the effects of plastic behaviors of comprising elements which show substantial difference among grades of steels were not counted on. These difference of material properties in plastic range were not taken into account even in the evaluation of test results,though a number of tests had been carried out on girders made of C class steels.

It might become possible to obtain the more optimum and the more reliable design,if the knowledge of plastic behavior of flanges and stiffeners made of various grades of steels,of which evaluations had been made in previous sections, were properly reflected on the research and analysis of those girders.

Based on the recent studies on hybrid beams(fabricated beams and girders which use a stronger steel in the flanges than in the web) the subcommittee of the Joint ASCE-AASHO Committee on Flexural Members had published a report in the form of design recommendations in 1968[105]. The subcommittee concluded that composite and noncomposite hybrid beams can be designed efficiently using an allowable stress based on the moment required to initiate flange yielding. This allowable flange stress is a function of the beam dimensions and the ratio of the yield strengths of the two steels,and is slightly lower than the allowable stress normally used for the flange steel. The bending stress in the web does not have to be checked when this reduced allowable flange stress is used. However,the shear stress in the web must be limited to the normal allowable stress for the web steel. The available fatigue data indicated that these hybrid beams can generally be designed for fatigue as if they were made entirely of the grade of steel used in the flanges. This Joint Committee report led to the adoption of design specifications for highway bridges and for buildings in USA: The American Association of State Highway Officials adopted provisions for both noncomposite and composite hybrid girders in 1969[106],and the American Institute of Steel Construction adopted provisions for noncomposite hybrid girders in 1969[92].

At present time,it is assumed that the hybrid beams will be designed on the basis of the allowable stress design,namely on the basis of the initial flange-yield moment. But it is desirable that both hybrid girders and homogeneous girders should be designed on the same concept based on the ultimate strength state. Ref.107 presented at the IABSE Colloquium seems to be on the verge of this approach.

Effect of web breathing on the fatigue strength is an important problem,especially for hybrid girders,which belongs to the problem of structural fatigue (or low cycle fatigue) rather than that of classical material fatigue, but the results of these tests should be carefully evaluated in consideration of the extent of the web breathing which likely to occur under the assumed service load level in association with the possible degree of imperfection of the actual girder.

8. FRACTURE AND FATIGUE

8.1. Welded Joint and Transitional Mode of Fracture

Over the period from 1937 to 1940,brittle fractures took place in succession and led to the failures of three steel Vierendeel type welded bridges in Belgium.

During the same period,brittle fracture was also observed in Rüdersdorf in Germany, an all welded steel plate girder made of St52. The fracture initiated from micro-cracks existed in heat affected zone due to welding.

As the results of an investigation to the cause of this fracture[108],it has been suggested that: 1)bead bend test proposed by Kommerelé simulates well the brittle fracture behavior of welded steel plates. 2)in order to prevent brittle fractures,adequate notch toughness is required as one of the material properties of steels. 3)notch toughness depends on temperature and 4)carbon equivalent,which plays a role for the initiation of cracks at heat affected zone,is an important factor together with such factors as structural restraints and pre-heating during welding. Since then continued efforts have been paid resulting in structural steels with greatly improved notch toughness. In Europe,however,there exists little intention of using extra-High strength steels for welded structures,which is due to the inherently more sensitive character of the steels to cracking at welds.

While in USA,quenched and tempered high strength steel with tensile strength exceeding 789 MN/m^2(C class steel) has been introduced around 1955 for structural applications in welded constructions. Coincident with the introduction of this A514 steel,similar heat treated steel with tensile strength exceeding 592 MN/m^2(SM58) has also been developed in Japan. The most important feature of this type of steels is an easy attainment of extremely high strength with few low alloy elements in spite of low carbon content(about 0.1% or more),and hence with low equivalent carbon content. This feature made it possible to perform crack free welding under moderate pre-heating condition. Moreover proper alloy elements and heat treatment contribute to maintain adequate notch toughness. The good weldability of the heat treated steels helped to increase their wider use in welded structures. It has to be noticed,however,that because of the decrease of notch toughness level by the formation of upper bainite arising from welding heat cycles at weld fusion lines,and such welding defects as cracks,undercuts,various angle changes of joints and residual stresses,fractures may take place under low stress level. In order to avoid these failures, US.Steel Corp., recommended to limit the amount of heat imput depending on the thickness of plates. Improvements have also continued with such intension as to make it possible to weld this type of steels with larger heat imput. Based on intensive research works[109],75mm thick 785 MN/m2 strength heat treated steel has been successfully used in the welded construction works of Osaka Port Bridge in Japan, which opened to traffic on July 1974.

Numerous surveys and investigations have been performed in relation to various accidents due to brittle fractures occured in such welded structures as ships and pressure vessels. One of the findings of these research works is that tensile tests on notched wideplate resemble the situation and satisfy the conditions at the locations of brittle fractures in practical structures[110]. Various reports were prepared on the initiation and propagation of brittle fractures[110,111]. Considering the stress intensity factors K_I computed based on the theory of elasticity at the tips of notches and the critical stress intensity factors K_{Ic} obtained experimentally from tensile tests on notched wideplate together with their temperature depending characters,the characteristics of brittle fractures are being explained in terms of equivalent notch effects which represent various defects and size effects [111,112,114]. Efforts[111] have been paid to evaluate the relationship among K_{Ic}, $_vE$(CVN) and yield stress σ_y. The efforts revealed that for steels with increasing yield stresses, a larger energy absorbing capacity $_vE$ is necessary to maintain the same K_{Ic} value. From the view point that the effects of plastification around the tips of notches have to be considered in addition to the characteristic factors based on the analysis of linear fracture mechanics, the concept of crack opening displacement(COD) is beihg proposed[111,113,114].

8.2. High Cycle Fatigue

The fatigue strength of materials free of defects may proportionally increase with the increase of its static strength. On the other hand the influence of the defects present in practical structures which act to reduce fatigue strength is being accelerated with the increase of tensile strength. This is due to the fact that high cycle fatigue is governed by localized stress concentrations arising from structural shape and material defects. As a natural consequence,experimental results scatter largely from tests to tests. S-N curves do not necessarily represent fatigue strength of steels,but they are simply statistical representations of test results.

Due to the presence of numerous defects,the fatigue strength of welded joints with high strength steels is rarely improved compared with that with ordinary strength steels[114,115,116],nevertheless efforts are being continued to improve the fatigue strength.

In the field of fracture mechanics,attempts have been made succesfully[114, 116] to explain some of the characteristic behaviors of high cycle fatigue by making use of the concept of K_I and K_{Ic} similar to those familiar in the analysis of brittle fractures.

8.3. Low Cycle Fatigue

Low cycle fatigue is inherently different phenomenon compared with high cycle fatigue: the mechanism of low cycle fatigue failures is similar to that of static failures of structures,and hence low cycle fatigue strength depends largely on structural behaviors under static loading. One of the features of high strength steels under the circumstances for low cycle fatigue is that C class steels soften when subjected to repeated loading and plastic energy stored at the tip of a crack increases,whereas A and B class steels harden under repeated loading and the same energy decreases.

Country	Standards	Desig - nation	Min. Yield Stress MN/m^2	Tens. Strength MN/m^2	Class	Y	Remarks
INTERNATIONAL	ISO	#Fe 42	245	412 ∿ 490	A	0.594	
		Fe 52	343	490 ∿ 608	B	0.70	
AUSTRALIA	AS A 186 and A 187 (weathering)	#Grade 250	248	> 412	A	0.602	
		″ 350	344	> 481	B'	0.715	Columbium and/or Vanadium
		* ″ 400	412	> 515	B'	0.8	* up to 13mm
		** ″ 500	481	> 550	B'	0.872	** up to 9mm
BELGIUM	NBN 631	#Fe 42	250	420 ∿ 500	A	0.595	
		Fe 52	350	520 ∿ 620	B	0.672	
CANADA	CSA	#G 40.8	262	448 ∿ 586	A	0.585	
		G 40.11	345	483 ∿ 655	B	0.715	Weathering steel
		G 40.18	689	793 ∿ 931	C	0.87	
CZECHO-SLOVAKIA	CSN 73 - 1401	#10370	250	350	A	0.715	
		11523	360	520	B	0.692	
		11483	380	480	B	0.791	
EAST GERMANY	TGL, 12910	#St 38-s	230	380	A	0.605	
		11523	350	640	B	0.547	
		11483	370	620	B	0.596	
		St 52-3	350	520	B	0.673	
ENGLAND	BS 4360	#Grade 40	230	400 ∿ 480	A	0.575	
		″ 50	345	500 ∿ 620	B'	0.69	Columbium and/or Vanadium
		″ 55	430	550 ∿ 700	B'	0.782	
FRANCE	NFA 35·501	#Grade E24	235	363 ∿ 441	A	0.647	
		″ E30	294	461 ∿ 559	B	0.637	
		″ E36	353	510 ∿ 608	B	0.692	

(continued)

Country	Standards	Desig - nation	Min. Yield Stress MN/m^2	Tens. Strength MN/m^2	Class	Y	Remarks
FRANCE	NFA 35·501	Grade A50	294	490 ∿ 588	A	0.60	Impact tests Not Req'd. Limited Weldability
		〃 A60	333	588 ∿ 706	A	0.567	
		〃 A70	363	686 ∿ 833	A	0.530	
INDIA	IS 226, 961, 2062	#St42(S,W)	230	410 ∿ 530	A	0.56	
		St55 HTW	340	540 min	B	0.63	
		St 58 HT	350	570 min	B	0.613	
ITALY	UNI 5334.64	#Fe 42	230	420 ∿ 500	A	0.55	
		Fe50.1,50.2	290	500 ∿ 600	B	0.58	
JAPAN	JIS G - 3101 ∿ 3106	#SS41,SM41	235	403 ∿ 510	A	0.583	
		SM 50	315	490 ∿ 607	B	0.642	
		SM 50Y	353	490 ∿ 607	B'	0.72	Columbium and/or Vanadium
		SM 53	353	520 ∿ 642	B	0.678	
		SM 58	452	570 ∿ 718	C'	0.794	
	WES	HW 70	686	785 ∿ 932	C	0.874	
		HW 80	785	864 ∿ 1030	C	0.908	
		HW 90	883	951 ∿ 1130	C	0.93	
NETHERLANDS	EURO NORM 25 - 72	#Fe 360	225	360 ∿ 440	A	0.625	
		Fe 510	345	510 ∿ 610	B	0.677	
		Fe 590	325	590 ∿ 710	A	0.55	Limited Weldability
		Fe 690	355	690 ∿ 830	A	0.515	Limited Weldability
POLAND		#St 35	226	373 ∿ 464	A	0.606	
		18G 2A	342	490 ∿ 626	B	0.697	
RUSSIA	GOST 380 Gost 5058	#BG 3	240	440 ∿ 470	A	0.546	
		14Г2	330	470	B	0.702	
		15ГC	340	480	B	0.702	
		10Г2C	350	500		0.7	
		10XCHA	400	540		0.741	Columbium and/or Vanadium

(continued)

Country	Standards	Designation	Min. Yield Stress MN/m^2	Tens. Strength MN/m^2	Class	Y	Remarks
SWEDEN	SIS	#1412	260	430 ∿ 510	A	0.605	
		2172	300	490 ∿ 590	B	0.612	
		2132	350	510	B	0.686	
		2142	390	530	B	0.736	
SWITZERLAND	SIA 161 (1972)	#St 24/37	227	364 ∿ 443	A	0.623	
		St 36/52	345	512 ∿ 611	B	0.674	
U.S.A.	ASTM	#A 36	248	400 ∿ 552	A	0.62	
		A242, A441, AA441	317	462	B	0.686	A242: weathering steel
		A572 Gr 45	310	414	B'	0.748	Columnbium and/or Vanadium
		50	345	448	B'	0.77	
		55	379	483	B'	0.785	
		60	414	517	B'	0.8	
		A 588	345	483	B	0.713	Weathering Steel
		A 514	690	793 ∿ 930	C	0.87	
WEST GERMANY	DIN 17100	#St 37.283	240	370 ∿ 450	A	0.647	
		St 46.223	290	440 ∿ 540		0.658	
		St 52.3	360	520 ∿ 620	B	0.691	

1. Type: A = carbon steel, B = high strength Low - alloy steel, C = heat treated constructional alloy steel
2. Listed are weldable steels only
3. Mechanical properties of steels thickness of which are 16 ∿ 40mm are listed
4. $Y = \frac{\text{Yield stress}}{\text{Tensile strength}}$ = Yield ratio of material (maximum)
5. Typical mild steels are included for reference and which are identified by # mark.

REFERENCES

For brevity,the titles of a number of professional societies have been abbreviated as follows:

AIJ Architectural Institute of Japan
AISC American Institute of Steel Construction
AISI American Iron and Steel Institute
ASCE American Society of Civil Engineers
ASTM American Society for Testing and Materials
CRC Column Research Council,USA
IABSE International Association for Bridge and Structural Engineering
ICE The Institution of Civil Engineers,England
ISE The Institution of Structural Engineers,England
JSCE Japan Society of Civil Engineers
WRC Welding Research Council,USA

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SUMMARY

The state-of-the-art on the structural behaviour of members and structures made of high-strength steels is outlined. Characteristics of the structural behaviour of those made of high-strength steels are discussed in relation to those made of mild steel in order to provide the information by which a designer can decide whether a grade of high-strength steel should or should not be used in a particular part or in a particular structure in association with the imposed loading condition.

RESUME

L'état des connaissances actuelles est présenté sur le comportement structural des éléments et des constructions réalisés en acier à haute résistance. Les caractéristiques du comportement structural sont comparées pour des constructions en acier à haute résistance et en acier doux. Cette comparaison facilite le choix de l'ingénieur quant à l'emploi d'un acier à haute résistance ou d'un acier doux pour des éléments ou des ensembles de structure en fonction des conditions de charge imposées.

ZUSAMMENFASSUNG

Die Autoren berichten über die heutigen Kenntnisse über das Tragverhalten von Bauteilen und Bauwerken aus hochfestem Stahl und deren charakteristisches Tragverhalten bei Verwendung hochfester Stähle und von normalem Baustahl wird verglichen. Dadurch ergeben sich die Entwurfsgrundlagen, nach denen entschieden werden kann, ob in einem bestimmten Bauteil oder Bauwerk im Zusammenhang mit den vorgeschriebenen Belastungsannahmen ein spezieller hochfester Stahl angewendet werden soll.

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