Comparison of LRFD and Allowable Stress Design Methods for Steel Structures

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ABSTRACT

Steel structures have historically been designed by allowable stress design methods. Recently, the American Institute of Steel Construction has developed an alternate specification based on the Load and Resistance Factor Design Method. This method is a statistically based method which utilizes concepts of limit state design. It uses factored loads and employs resistance factors to account for uncertainty in the loads and computed strength of the structural components. The rationale for these load and resistance factors will be summarized. The general concepts of the design method will be discussed and compared to other design concepts such as allowable stress design and Ultimate Strength Design. Specific comparisons of design provisions will be made for bending members. The comparison will show that there are some specific economic advantages for structures designed by this method, and it will illustrate some advantages with respect to rationality of the design.

INTRODUCTION

Steel structures have historically been designed by allowable stress design (ASD) methods. The American Institute of Steel Construction (AISC) has been the agency, which has been responsible for this design method in the United States. The AISC Specification is intended for use in buildings, but it is often used as a basis for separate specifications developed for structures such as bridges or industrial facilities. The eighth edition of the specification[1] was published in 1980 and is the most commonly used ASD method in the US today. Recently, AISC has developed an alternate specification based on the Load and Resistance Factor (LRFD) Design[2] method. The LRFD Specification used a different format, and it used very different design concepts than those used for ASD. More recently a new Ninth Edition ASD design specification[3] has been developed. There are now two independent design methods for steel buildings in the US. The new ASD Specification uses a similar format as the LRFD Specification, because many engineers believe the new format is easier to use and interpret. The new ASD Specification also incorporates changes which were required to maintain rational design for steel structures. Many of these changes came to light as a result of the overall review required during the development of the LRFD Specification. However, the new ASD Specification continues to use the historic rational in the ASD Specification, and no effort was made to extend the advantages of the LRFD method to the new ASD provisions It is generally thought that the ASD method will disappear in the coming years through attrition as more engineers become familiar with the LRFD method, and so future changes and improvements to the specification will occur through the LRFD provisions This attrition is not totally obvious in US practice at this time. Some structura engineers use the LRFD method and are enthusiastic about its advantages. However, the great majority of steel structures in the US are still designed by an ASD method The Eighth Edition of ASD is still used for most of these designs Nevertheless, a increasing number of political agencies and model specifications are recognizing a adopting LRFD for building design, and some independent agencies such as the Americ

Association of State Highway and Transportation Officials (AASHTO) are developing new LRFD Specifications[4] for the design of bridges and other structural systems. The net effect of these developments is to provide wider familiarity, exposure and acceptance of the LRFD method.

The LRFD design method recognizes that there is variation in the loads applied to a member and in the behavior of the member subject to a given load. The LRFD method attempts to rationally deal with these variations in load and behavior. It utilizes concepts of limit state design, which have been employed in some specifications in Europe and other parts of the world. Limit state design makes a marked distinction between the strength and serviceability of the structure, and it requires that the designer separately consider potential problems and modes of failure of the member and the structure. The LRFD specification employs these limit state concepts but it also adds statistical considerations. The objective of structural design is to produce a serviceable structure which is strong enough to support the required loads. This normally requires that the strength or resistance of the structure, R, always exceed the loads, Q, on the structure for all conditions.

Many structural engineers believe that they are satisfying the required inequality between load and resistance with the ASD method. They also usually believe that the probability of structural failure is zero when these ASD provisions are satisfied. However, regular reading of any major newspaper will show that structures do occasionally collapse. It is also clear that more structures may have serviceability problems during the life of the structure. The LRFD method recognizes this small but finite probability of failure, and it attempts to deal with it in a rational manner. It attempts to design structures which have consistent reliability without severely complicating the design process. The method recognizes the true behavior of structures rather than an idealized mathematical model. The LRFD method provides a rational method where the reliability of the structure can be increased or decreased (and the probability of failure decreased or increased) for structures of greater or lesser importance. The discussion in this paper will focus on the AISC LRFD Specification[2] as used for building design, but many of the observations are also applicable to the wider range of LRFD provisions. The paper will present a general overview of the rational behind the LRFD method. The method uses resistance factors (ø factors)and load factors (γ), and the selection of the factors will be discussed in some detail. It is not practical to provide detailed discussion of all aspects of steel design in this paper and so the more detailed discussion of LRFD provisions as they relate to the design of flexural members will be provided. LRFD provision also results in significant changes in provisions for connection design, and a brief discussion of these changes will also be provided.

RATIONALE OF DESIGN

The design process consists of providing structures with a strength or resistance which is greater than the applied loads. In ASD, the designer attempts to compute the stress, f_a, in a member or connection and keeps this stress below the yield stress, F_y, or some critical buckling stress, F_{cr}, with a given factor of safety, FS. That is

$$f_a < (F_y / FS) \quad \text{or} \quad f_a < (F_{cr} / FS) \qquad \text{(Eq.- 1)}$$

This approach is depicted in the stress strain curve of Fig. 1. There are several problems with this method, however. First, the actual stress in the member or connection is nearly always computed by a simplified linear elastic analysis technique, and real structures are seldom linear elastic. In particular, steel structures often yield at relatively low load levels due to residual stresses or forces locked in during the construction process. The ultimate strength of steel structures often greatly exceeds the yield capacity predicted by linear elastic analysis, because of the great ductility of the material. Second, this approach uses a single factor of safety which must account for both variation in member behavior and uncertainty in the loading. Unfortunately, these variations may be very different for different conditions, and it is difficult to define a single factor of safety, FS, which covers the wide range of these variations. Engineers have recognized these problems, and the factors of safety used in ASD were typically selected with great care. The factors of safety often accounted for several problems or modes of failure acting together. As a result, this often lead to perceived irrationality in these safety factors. This also leads to extreme conservatism in some conditions, and fudge factors such as allowable stress increases were inserted in the design specifications to correct for these problems. One of the greatest problems with the the ASD method, however, was that it commonly mislead the engineer into believing the factor of safety, FS, used in the design equation was a true measure of the safety of the structure. Some structural engineers believed that the probability of failure of their structure was zero when the allowable stress equations were satisfied. This is clearly not the case, because the ASD method does not consider the variability that is possible in design.

Unfortunately, there is great variability in both the loads and resistance. This is illustrated in Fig. 2. The statistical variation in this figure is approximated as a normal distribution, but obviously other statistical distributions are possible. The statistical variations are caused by many factors. Variation in loads are associated with normal time dependent variations in loading, and potential changes in use of the structure over its useful life. Variations is resistance are caused by variations in yield strength and material properties, variations in geometry and size of members, inaccuracy of the mathematical models used to predict the strength of the member, and variability in the judgement and understanding of individual engineers. The variation and the resulting probability density functions shown in Fig. 2 are quite critical, since the overlap of the two density functions defines the probability of occurrence for that particular mode of failure. This probability of failure is small but finite. It is made increasingly small by separating the two functions by increasingly larger amounts. That is, the designer attempts to assure that the mean load, Q_m , is kept below the mean resistance, R_m,by a given margin, FS_m.

$$Q_m < FS_m R_m \qquad \text{(Eq -2).}$$

The margin, FS_m, must be selected to obtain the design reliability of the structure, and it must include both the variability in the load and the resistance. Since this variability can be very large, many different margins would be required to accommodate the wide range of combinations of load conditions and types of behavior noted in steel structures. The LRFD method simplifies this process and reduces the number separate variability considerations by separating the variability of the resistance from the variability in loading by introducing separate load factors, γ, and resistance factors, ø. The LRFD method also recognizes that normal design methods predict nominal loads, Q_n,and resistance, R_n, and these nominal values may be quite different from the mean values.

The resistance factor, ø, is dependent upon the variation of the member resistance and the accuracy of the mathematical model used to estimate the resistance. The resistance factor is less than or equal to 1.0. The resistance factor is smaller if there is great variation in the resistance of the member or if the nominal resistance tends to be larger than the mean resistance. The load factors, γ, depend upon the variation of the loads. The load factor is greater than or equal to 1.0, and it is larger for loads which have large variation or for loads which have a high probability of occurrence. The combined effects of these factors is that the nominal load and resistance are evaluated such that the combined effect of the two terms are to assure the safety and reliability of the structure through the equation

$$\gamma Q_n < \phi R_n. \quad \text{(Eq. 3)}$$

Allowable stress design methods use factors of safety which may be based on consideration of many factors and focus on a fictitious mode of failure rather than a true limit state. The load and resistance factors of LRFD are based on a statistical analysis of the available experimental results for each loading and mode of failure. The load and resistance factors are chosen to provide a consistent statistical reliability for all aspects of the design. That is, the LRFD method attempts to provide a comparable probability of failure (i.e. the overlap noted in Fig. 2) for all portions of the structure and all similar structures.

A simplified method of statistical analysis[5,6] is used to achieve this balance. This simplified method combines the two random variables (load and resistance) by subtracting the load from the resistance and the new statistical distribution for the difference between the two variables is as shown in Fig. 3. The probability of failure for a given structure or component due to a given limit state is the area under the probability density function with the new variable less than zero. A simple statistical method, the first order second moment method, is than used to estimate the probability of failure for each limit state. It should be noted that the probability density functions of Figs. 2 and 3 use a normal distribution, but true load and resistance are unlikely to have this idealized statistical distribution. Thus, a simple statistical method is needed to combine the two undefined density functions. The selected method, the first order second moment method, depends only upon the estimated mean values of the load and resistance and the variance, V, or standard deviation, σ, of the two variables. The simple analysis leads to a reliability index, β, for the component or structure where

$$\beta \cong \frac{\ln\left(\frac{R_m}{Q_m}\right)}{\sqrt{V_R^2 + {}_Q^2}} \quad \text{(Eq 4)}$$

A large value of the β results in a small probability of failure. It is desirable to have a large value of β and a small probability of failure for critical or important structures, but a smaller β and a higher probability of failure can be tolerated for less critical structures. A larger reliability index is achieved by moving the mean load and resistance farther apart. The required separation is dependent upon the variance of the two variables. A reliability index of approximately 3.5 was found to be desirable for the members of most building structures, since this implies a probability of failure for a given element of approximately 0.0001. The probability of failure for the total structure is likely to be much smaller (or the reliability larger), because a single local failure typically does not result in structural collapse. The LRFD Specification deals only with member or component reliability at this time, but there is interest in extending this to total structural reliability. This reliability index was found to be consistent with that historically achieved in building design, and it was believed to be a reliability which would be tolerable to the public and yet would not result in overly expensive structures.

The load and resistance factors are then chosen to achieve the desired reliability index, and they are based on the separate statistical behavior (Fig. 2) of the load and resistance. Separate resistance factors must be obtained for each potential problem or mode of failure and so separate statistical analyses must be performed for each design limit state. Separate load factors must be obtained for each load case or load combination, and so separate statistical analyses must be performed for each load condition.

GENERAL DESIGN IMPLICATIONS

LRFD offers many advantages. It provides a consistent reliability and safety for all structures and components. It deals directly with individual limit states rather than combining them into a mythical factor of safety. It deals directly with the true behavior of structures rather than a highly idealized linear elastic model. It provides a rational mechanism for changing the reliability required for individual structures and for incorporating new research information into the design process. On the surface it requires a great deal of statistical analysis, however this statistical analysis is not required by the structural engineer unless the engineer wants to increase or decrease the reliability from that envisioned in the basic specification. The AISC LRFD Specification uses the target reliability to assign load factors for all load combinations, and to assign resistance factors to all limit states. Therefore, the structural engineer can directly use these design factors, and does not need to perform any statistical analysis. The statistical considerations are completely transparent to the engineer under these conditions. The actual design is performed in classical design methods where the strength and load are compared. However, the design is still different from basic ASD methods in that the comparison is made with factored loads, and the comparison is made with a comparison such as indicated in Eq. 3 rather than a comparison as indicated in Eq. 1.

The load and resistance factors were selected based on an extensive research program[6-11] into the failure modes and behavior of steel structures and structural components. In addition, the factors were calibrated to be consistent with past design practise. However, there are significant changes which can be noted when comparing structures which were developed with past ASD methods with those that can be achieved with the LRFD Specification. The ASD method sometimes used design criteria or factors of safety which were partially intended to assure serviceability of the structure along with the safety of the structure. LRFD recognizes that safety and serviceability are different issues and separates them completely. Serviceability is concerned with the ability of the structure to economically satisfy its desired function. Therefore, the serviceability requirements may vary dramatically with different types of buildings. Deflection limits are one very common serviceability limit for buildings, and deflections limits are typically very variable. Hospitals or other critical structures or structures with some stiff, brittle architectural elements often require very tight deflection limits, while some other buildings can tolerate relatively large deflections. Serviceability deflections are usually performed at service load conditions (unfactored loads or load factors of 1.0), or with smaller load factors and a smaller reliability index than are used for strength considerations. The AISC LRFD Specification largely sidesteps the issue of assigning serviceability limits. This is now viewed as a responsibility of the structural engineer since the engineer is most familiar with the design requirements of the total structure. As a result, it is now sometimes possible to design lighter structural components with the LRFD Specification than with the ASD Specification, if the ASD provisions had serviceability considerations built into them. Composite beams

(Chapter I of LRFD) are one type of component which may sometimes illustrate this effect. The LRFD Specification is not suggesting that serviceability limits are not important. It is suggesting that the limits are to variable to be included in a general specification. It is important that the engineer recognize the additional responsibility for establishing serviceability limits.

Deflections are usually a serviceability consideration, but deflections may also be important to the safety and stability of the structure. The stability and safety of the structure under combined loads (Chapter H of LRFD) are one example of this effect. The LRFD Specification now requires the designer to directly calculate these deflections and to use these deflections in the LRFD Specification. The ASD Specification often covered over these deflection considerations with additional conservatism or fudge factors.

The ASD and LRFD methods are likely to result in very similar designs with intermediate sized steel buildings, but individual aspects of the design may change somewhat. However, very tall buildings may require slightly less steel with LRFD design than with historic ASD design. This occurs because of the more rational treatment of load factors for dead and live load. Tall buildings are often dominated by dead load rather than live load. Live load is much more variable than dead load, and so a larger load factor is typically required for live load than for dead load. As a result, taller buildings may require somewhat less steel. Shorter buildings or light metal frames such as prefabricated metal buildings are dominated by live load and so LRFD will require bigger members for some of these structures. While these changes may seem arbitrary to some engineers, they are important because they assure that the probability of failure or reliability of the structure is similar for both class of structures.

The LRFD Specification recognizes that steel structures are not linear under many practical conditions. It includes provisions which are leading to increased use of nonlinear or approximate nonlinear analysis techniques in the United States. This is particularly true with combined load and stability considerations, and it is increasingly true with connection design. LRFD attempts to deal with structures as they actually behave rather than through some idealized (and often incorrect) model.

Finally, it should be noted that the AISC LRFD Specification was the first steel specification to be completely rewritten in many years. This operation allowed the writers to reorganize the material in a more logical manner, and to eliminate the overlap and scattered presentation included in the 1980 ASD Specification. This also simplified the application of research results to the specification. The LRFD Specification is divided into 13 chapters, and each chapter provides a relatively complete coverage of one aspect of the design. Detailed or highly specialized information is included in an Appendix associated with that chapter. The general design provisions are covered in Chapter A. Chapter B provides general design requirements such as definition of effective section and slenderness design limits. Tension members, compression members, and beams are covered in Chapters D, E, and F, respectively. Combined loads are covered in Chapter H, and connections are covered in Chapter J. Chapter K covers specific strength limitations often associated with connection design or other design details. New users require time to become familiar with this new format, but after they gain this familiarity, the new format is much easier to use with less shuffling through the manual. The latest edition[3] of the ASD Specification has also adopted this new format.

LOAD FACTORS

The load factors in the AISC LRFD Specification are based on an extensive independent research program[11,12] on the loadings in buildings. The AISC LRFD load factors were originally developed for the American National Standards Institute (ANSI) Standards[13]. The resulting load factors and critical load combinations (Chapter A of LRFD) are -

$$
\begin{gathered}
1.4\ D \\
1.2\ D + 1.6\ L + 0.5\ (L_r \text{ or } S \text{ or } R) \\
1.2\ D + 1.6\ (L_r \text{ or } S \text{ or } R) + (0.5\ L \text{ or } 0.8\ W) \qquad \text{(Eqs. 5)} \\
1.2\ D + 1.3\ W + 0.5\ L + 0.5\ (L_r \text{ or } S \text{ or } R) \\
1.2\ D + 1.5\ E + (0.5\ L \text{ or } 0.2\ S) \\
0.9\ D - (1.3\ W \text{ or } 1.5\ E)
\end{gathered}
$$

where D is dead load, L is live load, W is wind load, S is snow load, R is roof load, and E is earthquake load.

The AISC LRFD load factors and load combinations have a great deal of similarity with load factors used in American Concrete Institute (ACI) Ultimate Strength Design[14] , but they are clearly also different. In particular, the load factors assigned to dead load are somewhat larger in the ACI than in the AISC. This suggests that heavy structures can be designed with somewhat lighter members with LRFD than with the ACI Specification. This difference has caused some concern among engineers in the US. The AISC LRFD factors are believed to be a rational measure of the statistical variability of the loads, and as a result are believed to a rational use of limit state design concepts. The ACI load factors were designed some years earlier. They were intended to be rational. However, they were not based on a detailed statistical evaluation of loads and loading patterns, and the ACI factors also included other secondary considerations. In particular, concrete is a more variable material than steel, and the resistance factors tend to be lower than steel. This is particularly true with shear and tension design considerations. The concrete industry made a deliberate decision to use slightly larger load factors to avoid excessively small resistance factors when the Ultimate Design Specification was developed because of the negative reactions engineers have to material variability. This decision maintained the safety of concrete structures and it avoided the necessity of educating the engineering profession of the full variability which can be expected in design. Engineers in the US sometime question the rationality of the LRFD load factors in view of this difference. It is clear that there is a rational explanation for the difference, but it remains somewhat unsettling. It causes design problems, since many structures require design of both steel and concrete components, and the difference in load factors requires that they maintain two sets of factored loads to complete the design. Many engineers look on the LRFD design as a rational step forward, since AISC LRFD and ACI Ultimate Strength Design both use load and resistance factors, but the load factors remain an issue of some controversy in the US.

DESIGN OF FLEXURAL MEMBERS

It is not possible to cover the full range of design applications in this paper, and so a more detailed comparison between the ASD and LRFD provisions is made only for beams or flexural members Beams may exhibit a wide range of behavior. Figure 4 illustrates typical bending moment - deflection behavior for beams under a given loading. Multiple curves are shown in this figure to represent the wide range of behavior that can be achieved. Curve 1 represents a beam which can which can develop the full plastic bending moment, M_p, and can maintain this moment for large plastic rotation. A beam of this type is suitable for plastic design, because plastic design requires the ability of an element to develop a plastic collapse mechanism without a reduction in moment capacity. It requires adequate lateral bracing to control lateral torsional buckling, and the flange and web must have adequate thickness for avoid premature local web or flange buckling. Plastic design has long been recognized as an appropriate limit state for steel design, and it has historically been covered in Part 2 of the ASD Specification. Plastic design is covered in F1.1 of the LRFD specification. Table 1 provides a comparison of these basic provisions in the Eight Edition ASD[1] and the LRFD[2] Specifications, and it can be seen that they are not dramatically different. The LRFD Specification recognizes that the flexural behavior of steel members is more predictable and less variable than some other types of behavior. A ϕ factor of 0.9 is assigned because of this observation[7].

Curve 2 of Fig. 4 illustrates a beam which can reliably develop the plastic moment capacity but cannot necessarily maintain it for large plastic rotations. This has historically been the lower limit for compact section design. It has also been a acceptable limit for the last plastic hinge of a collapse mechanism of plastic design. Compact section design has historically appeared in para. 1.5.1.4.1 of the Eighth Edition ASD Specification and it is covered in F1.3 of the LRFD Specification. Table 2 compares these basic provisions. They look very different, but there is considerable similarity. The ASD provisions use an allowable stress of .66 F_y. This nominally appears to have a factor of safety of 1.5 when used in Eq. 1. However, when it is recognized that sections designed to these criteria can develop M_p, and M_p is 12% to 20% larger than the nominal yield moment (M_y = S Fy), the normal factor or safety against M_p of 1.7 is easily achieved. This represents one of the irrational elements of the ASD Specification since the true design considerations are disguised to the engineer. The provisions for compact section design are also different in that the ASD Specification is based on service loads rather than factored loads, but this difference is more a question of style than substance as will be illustrated in Fig. 5.

Curve 3 of Fig. 4 illustrates a beam which can develop the nominal yield capacity, M_y, of the beam, but cannot develop the plastic moment capacity, M_p, of the beam. In most practical cases of bending of hot rolled wide flange shapes, this occurs because the unsupported length is inadequate, since most wide flanges have relatively thick webs and flanges. This design category is covered by Eq. F1-3 in F1.3 of LRFD, and it is scattered within para. 1.5.1.4.5, 1.5.1.4.6a, and 1.5.1.4.6b of the Eighth Edition ASD Specification. They again look very different, but there is considerable similarity The ASD provisions use an allowable stress of .6 F_y, and this results in a factory of safety of approximately 1.7 against the nominal yield. The LRFD Specification uses a smooth transition of the moment capacity in from that of curve 2 to curve 3, while the ASD uses an abrupt transition as illustrated in Fig. 5. This again represents an irrational element of the ASD provisions. The ASD Specification are again based on service loads rather than factored loads, but this difference is more a question of style than substance as will be illustrated in Fig 5.

Bending members begin to yield well before the nominal yield moment is achieved. This occurs because of the residual stresses which are present in hot rolled sections. Curve 4 of Fig. 4 shows a beam which is dominated by this behavior. It develops the moment capacity associated with local yielding due to residual stress but does not develop the nominal yield moment. Beams in this category usually exhibit a form of inelastic lateral torsional buckling The moment capacity of these beams has historically been defined by the greater value of Eq. 1.5-6a or 1.5-7 in para.

1.5.1.4.6a of the Eighth Edition ASD Specification and it is covered by Eq. F1-6 of F1.3 of the LRFD Specification. The LRFD provisions are more compact since they require a single equation rather than a pair of equations. This occurs because the ASD design equations are simplified, conservative approximations to the equation which models lateral torsional buckling. The simplification was needed many years ago because design was performed with slide rules. Most engineers in the US now use electronic calculators or personal computers for design calculations, and so more complicated equations can be handled with ease. Figure 5 will show that LRFD design will sometimes result in increased load capacity for beams in this zone, because the full equation is used directly in the specification rather than a simplified approximation.

Curve 5 of Fig. 4 illustrates a beam which fails by pure elastic lateral torsional buckling. The moment capacity of these beams has historically been defined by the greater value of Eq. 1.5-6a or 1.5-7 in para. 1.5.1.4.6b of the Eighth Edition ASD Specification and it is covered by Eq. F1-13 of F1.4 of the LRFD Specification. The LRFD provisions are more compact since they require a single equation rather than a pair of equations. The ASD equations are again simplified and suffer from the same problems noted for the inelastic lateral torsional buckling zone.

This discussion clearly illustrates that steel beam design may be controlled by a number of different limit states. Both the Eighth Edition ASD and the LRFD provisions are based on these same limit states, but LRFD directly deals with the limit states and uses fewer approximations. ASD employs a rather scattered reasoning. It has some abrupt, irrational transitions, and it employs some conservative approximation. The Ninth Edition ASD maintains the conservative approximations and the abrupt transitions, but it utilizes a more logical limit state organization. Hot rolled wide flange shapes usually satisfy compactness criteria for flanges and webs, and their bending capacity is dominated by their unsupported length. Figure 5 illustrates the effect of the unsupported length on the moment capacity for a W16x50 beam of A36 steel. The service bending moment is taken as being constant over the unsupported length, since this is usually conservative for most other moment diagrams. The service load bending moment is assumed to be equally distributed between live and dead load. This permits direct comparison between LRFD and ASD. The LRFD provisions result in a slightly larger moment capacity over most of the range of the unsupported length. This is partially due to the smaller load factor for dead load, but some of the difference can be attributed to the more rational treatment of lateral torsional buckling. The effect of the various zones of behavior or limit states can be seen in this figure. The abrupt transitions and conservative approximations in ASD can also be noted.

CONNECTIONS

The LRFD provisions are also producing significant changes in the connection design provisions. Many structural problems are associated with connection details, and failure of an individual connection often has more serious consequences than yield of an individual member. As a result, connections are designed to have a greater reliability index, β, than member design. A target index of 4.5 is used for strength considerations in connection design. Many aspects of connection design are serviceability considerations. In particular, high strength bolted connections have historically been design as bearing connections or friction connections. Bearing connections transfer load between attached elements by direct bearing of the steel on the bolt and shearing action of the bolt across the transfer interface. Thus, bearing bolts have always been checked for bearing stress and shear stress. Edge distance requirements are closely related to the bearing stress requirements. These provisions are strength provisions LRFD and ASD have similar limit state requirements, although they are often expressed differently, and the LRFD provisions are based on a reliability index of approximately 4.5.

Friction bolt connections require that the bolt be tightened to a tensile force which is 70% of the ultimate tensile force in the bolt. The tensile force in the bolt induces a clamping force across the joint, and so the force in the connection is transferred between the components through the friction force developed by the clamping action. Many engineers prefer this type of connection, because they believe that it results in better service behavior in the structure. ASD has historically required that friction bolts be checked for shear, but the shear check is a fictitious check. The ASD shear stress check for friction bolts is actually based on the friction. The ASD provisions require that friction bolts also be checked for bearing, since it is recognized that the apparent factor of safety against slip is smaller than the nominal factors of safety which are applied to strength. The LRFD provisions refer to friction type connections as slip critical connections. The friction capacity is still checked through a fictitious shear check, but the check is made for service loads rather than factored loads. The reliability index for the friction check is much smaller (in the order of 1.5), and it is clearly recognized that slip may occur during the life of the connection. Thus, the LRFD provisions retain the requirements that slip critical connections be checked for bearing with factored loads.

The general conceptual framework for connection design has also changed in the LRFD Specification. The ASD provisions have historically considered all connections as Type 1, Type 2 or Type 3 connections. Type 1 connections are rigid frame connections which experience no deformation within the connection. Type 2 connections are pin connections which rotate freely and experience no rotational resistance. Both of these connection types are easily visualized in design calculations but never achieved in practice. All connections develop some deformation under loading, and they all develop some rotational resistance. This is depicted in Fig. 6. Figure 6 shows typical moment-rotation behavior for steel connections. Figure 7 illustrates typical connection details for some of these connections. The rotations are connection rotations rather than member rotations. A Type 1 connection should lie on the vertical axis on this plot, and a Type 2 connection should lie on the horizontal axis. No practical connections fill either of these requirements. ASD Type 3 connections were semi-rigid connections which basically covers all of the connections illustrated in Fig. 6. Unfortunately, the ASD provisions provided complete guidance for designing Type 1 and Type 2 connections but no guidance for Type 3 connections.

The LRFD Specification has attempted to deal with this dilemma in a more rational manner. It recognizes that all connections have some flexibility and that this flexibility affects the design of both the connection and the structure. The primary connection types are Restrained, Partially Restrained, and Unrestrained connections. The specification recognizes that virtually all connections fall into the partially restrained category, and the specification requires that the effect of the connection flexibility be considered when evaluating the stiffness and stability of the structure. This has started a whole series of connection research studies in the US. The studies have attempted to assign strength and stiffness characteristics for a wide range of connections. They have examined the details of connection design, and have resulted in improved design procedures. Nonlinear analysis programs have been developed to incorporate this connection behavior into the structural analysis. The result of this work is that there are a number of recent advances in the engineers understanding of connection behavior and connection design.

SUMMARY AND CONCLUSIONS

This paper has presented a brief overview of LRFD design in the United States and a comparison of these design provisions with allowable stress design. There are similarities in these design methods and there are marked differences. The differences are concentrated in the rational and logic behind the design rather than design process itself. The paper has provided a detailed discussion of the design rationale with particular emphasis on flexural members.

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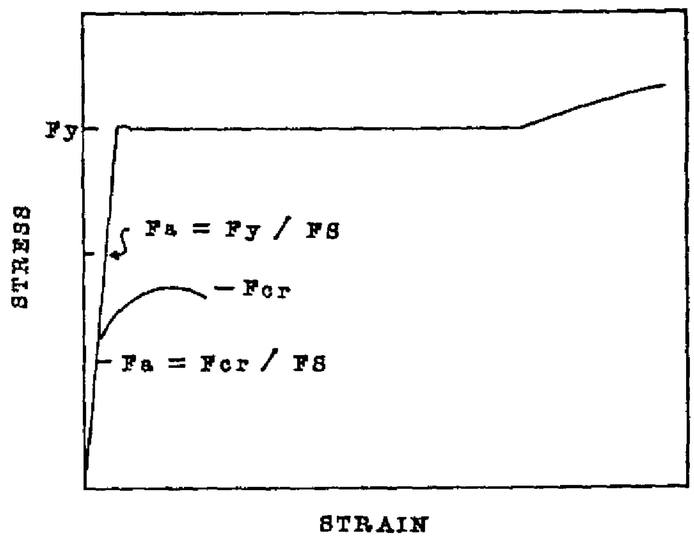

Figure 1. Idealized Representation of the ASD Method

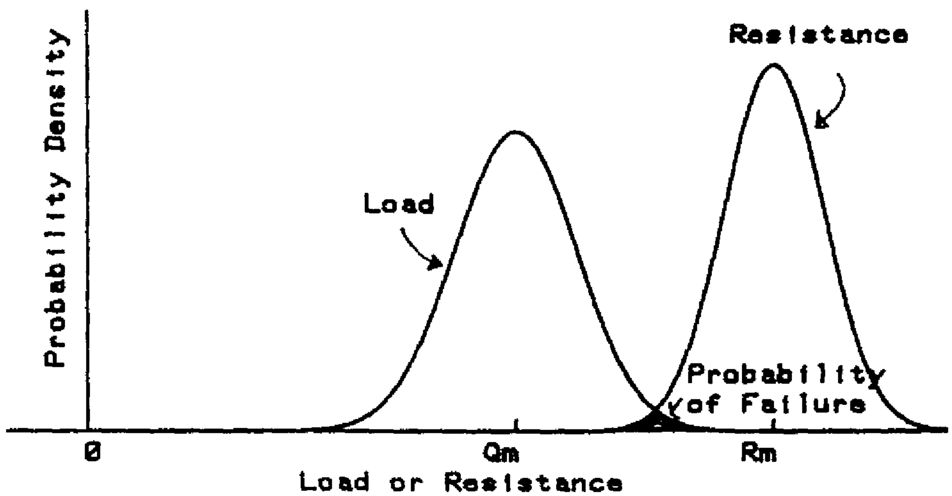

Figure 2. Statistical Variability in Load and Resistance

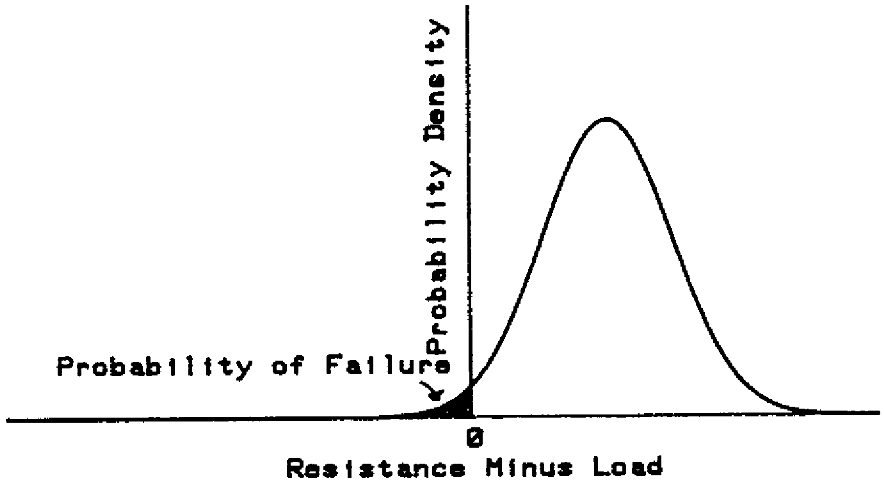

Figure 3. Statistical variability and Probability of Failure for a Component or Structure

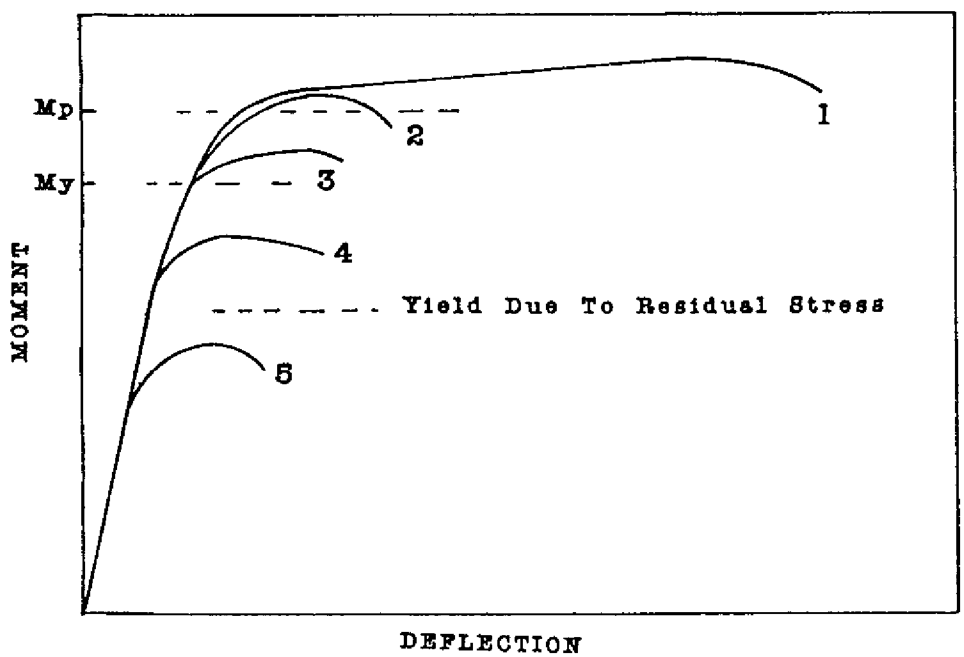

Figure 4. Typical Bending Moment - Deflection Curves for Steel Beams with a Given Loading

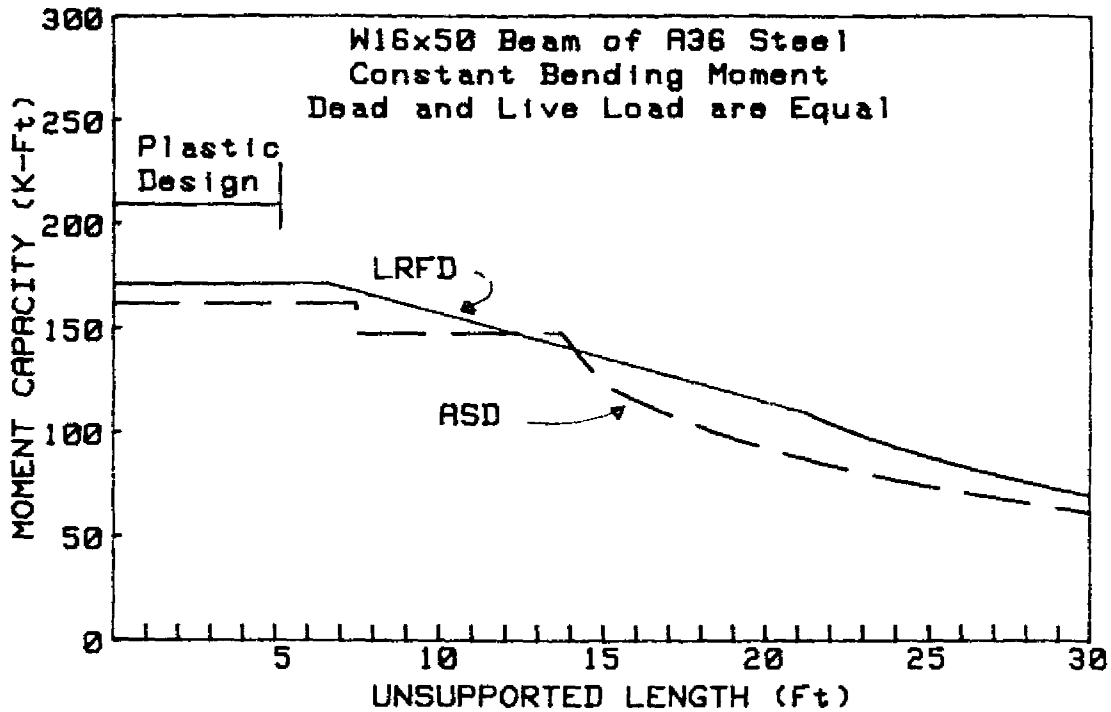

Figure 5. Comparison of the Load Capacity as a Function of Lateral Support for a Beam Designed by ASD and LRFD Methods

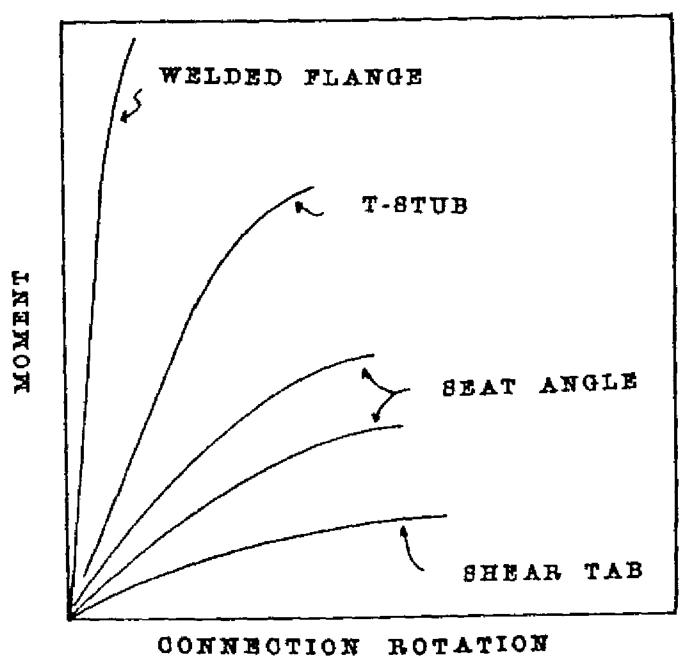

Figure 6. Typical Moment Rotation Behavior of Practical Steel Connections

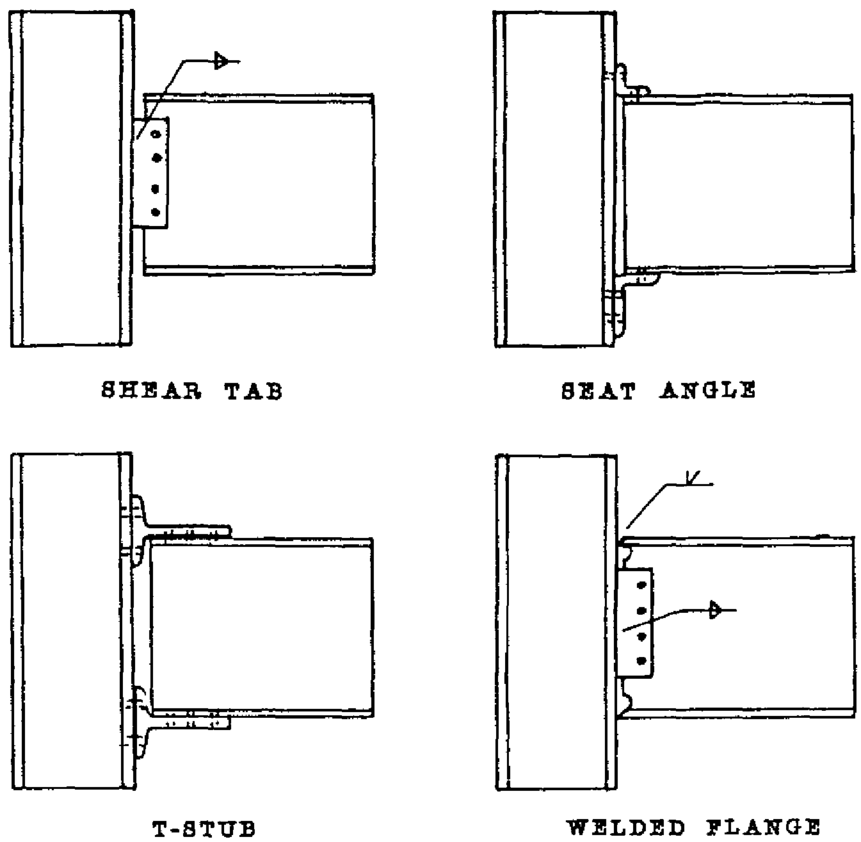

Figure 7. Typical Connection Details for Steel Construction