Limitations on design based on ultimate strength

To the Editor: The article, "Can Design Be Based on Ultimate Strength?" by Robert L. Ketter and Bruno Thürlimann in the January issue, was a concise and informative presentation of the fundamentals of design for ultimate capacity. The discussion of the design of redundant structures would, however, have been more complete if certain limitations of the technique had been noted.

Among these are, briefly:

1. The very simple analysis permitted by the introduction of the known flow moment at the points of maximum moment is not valid for moving loads. When loads may assume any position, a complete conventional analysis is required before the ultimate capacity can be determined.

2. In those cases where the capacity is obtainable directly from the known position of the yield hinges, the computation provides no information pertinent to the distribution of moments, shears, and deflections under working loads.

3. In steel, economy of design is largely limited to the case of simple rolled sections. Wherever it is intended to use cover plates or variable depth, the resistance of the member at the various critical sections may be adjusted, without significant increase in cost, to provide virtually full efficiency of material.

Walter E. O'Leary, J.M. ASCE
Hardesty & Hanover
New York, N.Y.

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Should design be based on ultimate strength?

To THE EDITOR: The article by Robert L. Ketter and Bruno Thürlimann, "Can Design Be Based on Ultimate Strength?" in the January issue, was an extremely interesting and well presented compilation of information on the subject of ultimate strength "analysis" and design. The writer believes, however, that a further examination of their examples will show that ordinary construction may profit little by the ultimate strength theory beyond the 20-percent stress increase for negative moments which is now present in the AISC code.

It is interesting to compare designs on the authors' two-span beam example with the concentrated loads placed in their much more usual positions at the center of each 20-ft span. Elastic analysis results in a negative moment of 188 ft-kips at the center support.

$$S = \frac{188 \times 12}{24} = 94. \text{ An 18WF55 is required [AISC 15(}a\text{) (3)]}$$

Using the authors' ultimate strength derivations for this case:

$$S = \frac{1.88 \times 50 \times 120 \times 120}{1.14 \times 33 \times 360} = 100.$$

An 18WF60 is required.

The required beam weights are now just the reverse of the authors' case—in the favor of *present* design methods. In further consideration of problems of this type it should be noted that *deflection* is often the controlling feature of continuous beam selection, as in highway bridges with continuous stringers and depth limited for clearance.

It is even more interesting to compare designs on the authors' rigid-frame example. The writer analyzed the given frame by customary elastic methods, and his design was based on a close observance of the present AISC specification. This resulted in a permissible design using a 12WF27 for both beam and columns—identical with the authors' proposed ultimate strength solution. The elastic approach requires investigation of the rigid-frame joint—especially if a uniform factor of safety is to be achieved. But it is to be noted that the ultimate strength approach *assumes* that the joint can remain relatively rigid under full plastic moment—a most difficult thing to achieve under usual framing and architectural clearance conditions.

The fabrication of connections which are capable of sustaining assumed moments is the little-mentioned weak link of ultimate strength analysis. There has been considerable research indicating that much care must be taken in evaluating moment vs rotation capacity of structural-steel rigid-frame joints. (See "Design and Research for Welded Structures," by LaMotte Grover, ASCE Proceedings. Separate No. 343, Nov. 1953, pp. 343–17, and the writer's discussion of same, Sept. 1954). However it should be mentioned that there is little if any available research on this subject with respect to reinforced-concrete rigid-frame joints constructed according to some of the ordinary reinforcement splicing practices.

NORMAN B. JONES, J.M. ASCE
Structural Engineer
Whittier, Calif.

Factor-of-safety concept needs clarification

To THE EDITOR: The article, "Can Design be Based on Ultimate Strength?" in the January 1955 issue, by Robert Ketter and Bruno Thürlimann is most welcome. It is a good thing to remind the practicing engineer that a simpler, safer, and more economical design method is available. How long will the inertia of the profession force us to waste both the client's money and our own time—and to create inferior structures to boot?

There is one consideration that could have been brought out more clearly in an article as basic as this. That is the concept of the factor of safety. Dead loads can be computed exactly; live loads have to be guessed. Dead loads will be as located by, and of the magnitude prescribed by our drawings—but almost anything can happen as far as live loads are concerned. Just think of a grand piano being dropped on a floor designed for 40 psf, or the common tendency of all crane operators to try to pick up everything and anything they can get a hold on.

Applying a factor of safety of 1.88 or 2.5, or anything any code prescribes, to dead load allows for our inability to compute more exactly. In all modesty, unless we displace the decimal point, we should be able to do better. But the same factor of safety applied against the grand piano or the crane operator might be catastrophic.

Applying a factor of safety for total load—and this happens when operating with "allowable stresses"—is quite meaningless. For example: two structures are designed for a so-called factor of safety of 1.88 for total load. The first carries a 75-percent live load and a 25-percent dead load. The real factor of safety for overloading by live load is (188 − 25)/75 = 2.17. The second structure carries a 25-percent live load and a 75-percent dead load. The real factor of safety for overloading by live load is (188 − 75)/25 = 4.52. That is, the second structure is more than twice as safe as the first.

Design based on ultimate strength will automatically correct this anomaly, because the load factor of safety will be different for dead load and live load.

FRED. P. WIESINGER, A.M. ASCE
Project Engr., Paul Rogers and Associates
Chicago, Ill.

Can design be based on ultimate strength?

To the Editor: The discussions of our article, "Can Design Be Based on Ultimate Strength?" (January issue, p. 59), focus attention on several points that should be considered. In Mr. Jones' letter (March issue, p. 64) the question is raised as to the economic advantage of using plastic analysis over current AISC specification provisions (Section 15-3) which allow a 20 percent increase in allowable stress over an interior support. Mr. Wiesinger (March issue, p. 64) brings up a most interesting topic—the question of a prorated factor of safety based on degree of uncertainty for different types of loads. Mr. O'Leary (April issue, p. 63) lists several possible limitations on plastic analysis.

Commenting on Mr. Jones' letter, it should first be reemphasized that plastic analysis is a method of solution based on actual ultimate strength. For design by this method then, the given loads are stepped up by a given load factor of safety (margin of desired load capacity above the working load), and required member sizes

are determined which are capable of supporting these stepped-up loads at or below their ultimate strength. The actual selection of this load factor is arbitrary at the present time, not having been established by code. For comparative purposes it was decided that the criterion for its selection might properly be the ultimate load capacity of a simple beam divided by the working load according to the present AISC specification. As demonstrated in the original article, this value is equal to 1.88.

In the continuous beam example presented by the authors, maximum elastic moment occurred within the span. Comparative designs showed that the plastic analysis solution using a load factor of 1.88 resulted in a more economical choice of members than did the one based on the current AISC specification. However, in Mr. Jones' example, with the loads located so that the maximum elastic moment occurred over the interior support, for which the specification allows a 20 percent increase in allowable stress, the plastic analysis solution was slightly more conservative. The following conclusions can be drawn: (1) designs based on the present AISC specification have a variable factor of safety against ultimate carrying capacity, and (2) under certain circumstances these specifications allow the design of a continuous member having a slightly lower load factor of safety than for a simple beam (1.88).

So that there will be no misunderstanding, it should here be pointed out that, if a minimum load factor of safety against ultimate carrying capacity is specified (whether it be 1.88, as in the author's example; 1.84, as in the discusser's case; or any other prescribed value), plastic analysis *will always* result in the smallest required section modulus. Other solutions may give no larger members for certain special cases, but they will never result in smaller ones. Mr. Jones' more economical choice was possible only by a decrease in load factor of safety below that assumed in the original article.

Regarding Mr. Jones' statement that plastic analysis "*assumes* that the joint can remain relatively rigid under full plastic moment—a most difficult thing to achieve under usual framing and architectural clearance conditions," it should be noted that the various typical welded connections tested at Lehigh and reported in "Connections for Welded Continuous Portal Frames," by Topractsoglou, Beedle and Johnston (*The Welding Journal Research Supplement*, July and August 1951, and November 1952), do have adequate moment-rotation characteristics. If, however, the rigidity of any given connection is in doubt, a simple and relatively inexpensive solution would be to over-design it and thereby force the "plastic hinge" to develop in the beam or column outside the knee.

The question of safety factor raised by Mr. Wiesinger is of importance regardless of the method of analysis. We agree with the discusser's viewpoint that a more rational approach to this problem would be to take into account the uncertainty associated with the various loads, although a proper weighting of relative uncertainties with respect to this single aspect of design itself is not without uncertainty. Since, in plastic analysis, loads are "stepped-up" by a so-called load factor of safety, nothing prohibits the use of different load factors for different types of loads, using this method of analysis. This situation is not, however, unique to plastic analysis. It could also be taken into account in elastic design.

The discussion by Mr. O'Leary lists "certain limitations" of plastic analysis. The first of these concerns the problem of moving loads. Secondly, it was pointed out that solution based on plastic analysis gives no indication of shear and moment diagrams or deflections at working loads. The third listed limitation has to do with economy and members of variable depth.

With regard to the first of these, nothing prohibits the consideration of several different arrangements of loads to determine the most unfavorable position for design purposes. It should be kept in mind, however, that under certain circumstances, repeated combinations of loads less than those considered critical for static purposes can, when alternated to produce strain reversal, result in increased deformations at each cycle, and thus an eventual collapse of the structure. While recent tests have shown that this condition is not as critical as was anticipated, it is felt that further research work is needed before the methods of analysis discussed in the original paper are extended to the moving-load problem. This work is currently under way.

It is considered that Mr. O'Leary's second listed condition is not a limitation but rather an asset. If for some reason the shear or moment diagram were needed, a conventional elastic solution at working loads could be carried out, but this computation would not be needed in determining the member sizes throughout the structure. Deflections, however, may need to be considered depending on the type of structure. Regarding this problem, it has been observed that in most cases if a load factor of 1.88 is used in the plastic design of continuous structures, the resulting deflections at working loads will be less than for simple beams designed to carry the same loading on the same spans.

Regarding Mr. O'Leary's third point, even though variable-depth sections can be elastically designed so that they are "efficient" with regard to total weight of structure, the less highly fabricated prismatic shape may, in certain cases, have dollar-for-dollar more to offer. This is evident when considering the resistance to overload of each of the two systems.

Robert L. Ketter, J.M. ASCE
Research Instructor
Bruno Thürlimann
Research Assistant Professor
Fritz Eng. Laboratory, Lehigh Univ.
Bethlehem, Pa.