

NEW DEVELOPMENTS IN CONCRETE BRIDGE DESIGN - ULTIMATE STRENGTH DESIGN

by

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This paper comments on the development of ultimate strength design and relates it to current AASHO specifications. It discusses briefly the ultimate strength and serviceability provisions of the BPR Criteria.

Early proposals for the design and analysis of reinforced concrete were not confined to the elastic or straight-line theory. Ultimate strength theories date prior to 1900. However, the straight-line theory gained general acceptance and was utilized by the first Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, which was organized in 1904. The straight-line theory has become so firmly established that its shortcomings or limitations are frequently overlooked.

Column tests initiated by the American Concrete Institute in the 1930's indicated that stresses calculated by the straight-line theory would not check with measured values. The results of the tests led to the design formulas in the 1940 Joint Committee Code. These were modified ultimate strength designs for axial loads. For eccentric loads they combined modified ultimate strength design with straight-line theory. If such a procedure is used to plot an interaction diagram, which shows the relationship of load to moment, discontinuities will be exhibited. Such design formulas, however, were used in the present AASHO specifications.

At working loads and stresses the entire cross-section of a prestressed concrete member is effective. A flexural overload causes cracking and a marked change in effective cross-section. It is possible to design a prestressed member with satisfactory working stresses but with little capacity beyond cracking. To ensure adequate overload capacity, an ultimate strength method is essential. The Bureau of Public Roads formalized such a method in 1954 in "Criteria for Prestressed Concrete Bridges." Present AASHO specifications include such provisions for flexure of prestressed members.

In 1955, a Joint ASCE-ACI Committee published a "Report on Ultimate Strength Design" containing design recommendations. These were incorporated in the Appendix of the 1956 ACI Building Code Requirements. The 1963 ACI Building Code contained expanded provisions in the body of the Code. It is generally thought that the next ACI Building Code will place working stress design in the Appendix.

The concern of the profession with ultimate strength design has interested bridge engineers. California, Texas, New York and Washington have built experimental bridges designed by ultimate strength methods. Last year, the Bureau of Public Roads issued "Strength and Serviceability Criteria Reinforced Concrete Bridge Members - Ultimate Strength Design," which may be used for the design of bridges on Federal highway systems.

Forces in concrete structures designed by these criteria are to be determined by elastic analysis. Limit design, which utilizes plastic hinges, is not to be used. However, in recognition of the fact that moments can redistribute due to plasticity, negative moments at the supports may be adjusted by not more than 10 percent, provided that the adjusted moments are used to calculate other moments in the span under the same loading condition.

Recognized AASHO loads are used and all loads are multiplied by factors which vary with loads and load groups. Most flexural members in superstructures will have their design controlled by the expression on the top of Figure 1. The factor of safety is made up of the multipliers, called load factors on the left, plus ϕ , called a strength modification factor on the right. The load factors produce a greater factor of safety for live loads than for dead loads. Since any overload is more apt to be a live load than dead, this is one example of how ultimate strength design makes more realistic evaluations. The strength modification factor provides for material and dimensional variation and adjusts the factor of safety for different types of failure and members. For flexure, $\phi = 0.9$, but for shear it is 0.85, and for tied columns it is 0.7. For some calculations it is convenient to move ϕ to the left side of the equation, leaving the pure ultimate strength on the right and the most probable safety factors on the left.

Assuming that most bridge designs are presently intended for intermediate grade reinforcing steel, it is possible to derive a similar equation for working stress design. The difference between the internal moment arm for ultimate strength design and working stress design is only five or six percent and is neglected in the comparison presented on Figure 2. It should be obvious that use of the provisions of the criteria for short spans will not be advantageous for intermediate grade steel. One should not expect the criteria to produce improvement in economy in slabs, but they should result in economy for larger spans.

The criteria provide that all flexural members are to be under-reinforced. In a load to failure, an under-reinforced beam would exhibit distress first by yielding of the tension steel. As the steel yielded, strains on the compressive face would increase until they reached a critical magnitude. This critical magnitude is conservatively assumed to be 0.003. A sufficiently large percentage of steel would cause this critical strain at the same time the steel yielded. This percentage of steel is called "balanced." The strain

ULTIMATE STRENGTH

$$\gamma [D + \beta(L + I)] = \phi [A_s f_y (d - \frac{a}{2})]$$

$$1.35 [D + 1.67(L + I)] = 0.9 [A_s f_y (d - \frac{a}{2})]$$

$$1.5D + 2.5(L + I) = A_s f_y (d - \frac{a}{2})$$

WORKING STRESS ASSUMING $f_y = 40,000$ psi

$$D + L + I = A_s j d \frac{f_y}{2}$$

OR $2(D + L + I) = A_s f_y j d$

Figure 1. Design Equations.

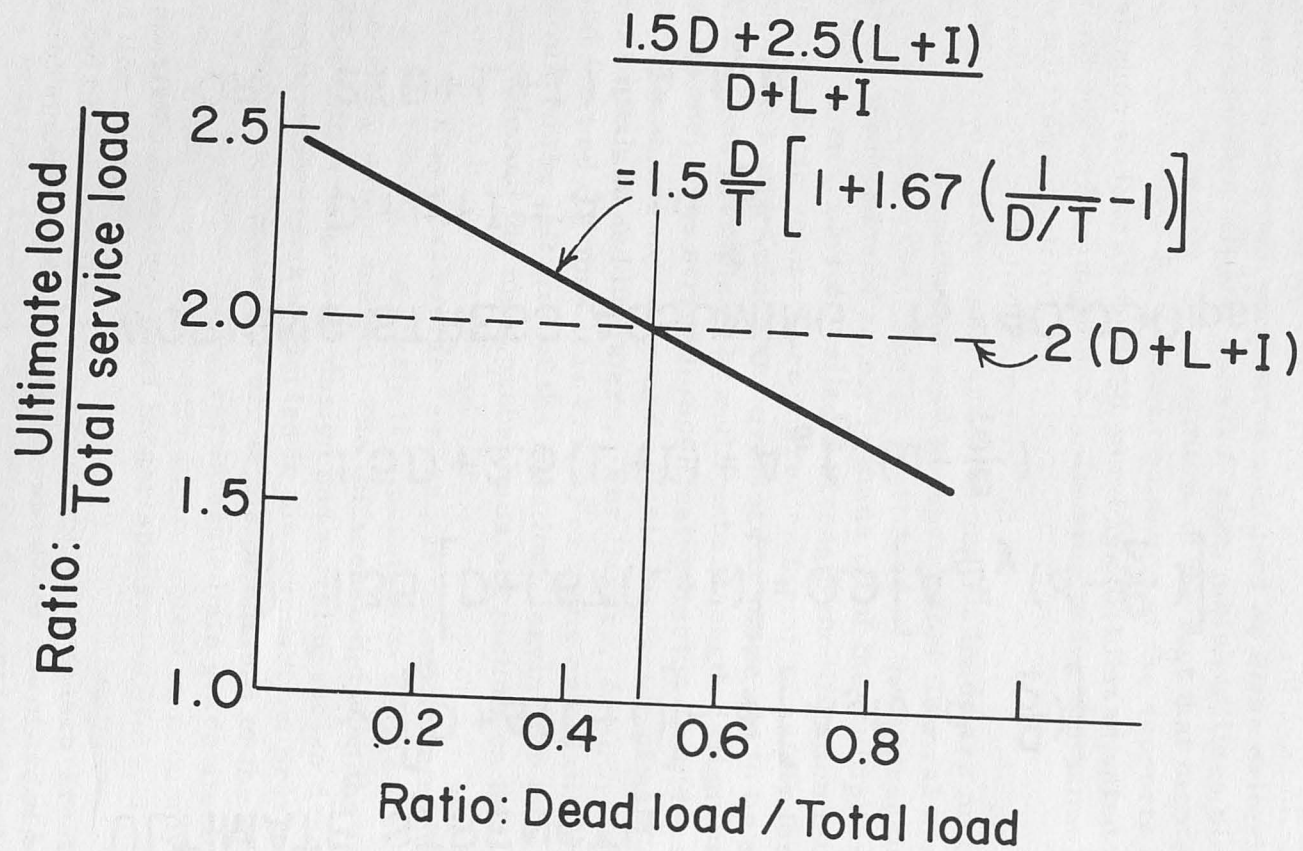


Figure 2. Comparison of Ultimate Strength and Working Stress Designs.

diagram with balanced p is shown to the left in Figure 3. Knowing the concrete strain and the steel strain at yield, the depth to the neutral axis can be calculated. The criteria limit the percentage of steel to $0.5 p_b$ and this strain diagram is shown at the right. Using half of the depth to the neutral axis which p_b required and the known concrete strain, the steel strain may be calculated. Note the large plastic strain which will occur before the concrete fails. The ACI Code limits the percentage of steel to only $0.75 p_b$, but the more restrictive limitation of the criteria will probably not be a handicap, because it will seldom be economical to use more than $0.5 p_b$.

In the case of flanged members, the steel is considered to be divided into a portion which balances the flange area and the remainder which works with the web. The ACI Code limits the portion which works with the web to $0.75 p_b$, but the criteria limit the portion working with the web plus half of that working with the flange to $0.5 p_b$. This is a severe limitation and will frequently control a design.

Figure 4 shows a typical ultimate strength interaction diagram, the relationship of axial load to moment, for a column. Notice that there are no discontinuities. Such curves can be plotted rapidly with few points. These were plotted with only four. The first point was for axial load alone. The next point was for balanced load. This is the load at which critical strain is reached in the concrete at the same time the steel on the tensile face yields. The other two points are arbitrary locations of the neutral axis on either side of the balanced locations. Such calculations involve only basic principles. If it is preferred, design aids such as ACI SP-7, which present similar curves in non-dimensional form, are widely available. The column equations presented in the Appendix to the Criteria are not recommended for the design of columns. They are not particularly easy to use and the interaction diagrams are more accurate.

The preceding applies to resistance against collapse, which is generally considered to have the dominant role in ultimate strength design. However, in the BPR criteria, working load serviceability often dominates the design. This includes fatigue considerations, crack widths, and live load deflections.

The fatigue considerations limit the live load stress range to $0.5 f'_c$ in the concrete and 20,000 psi in the steel. The latter will be a controlling factor in the design of bridge decks. As stated previously, the criteria should not be expected to reduce material requirements for bridge decks.

The crack width limitations appear arbitrary. The author knows of no definitive research on the effect of crack width on corrosion of reinforcing steel. The Corps of Engineers initiated a research program in 1950. Their specimens are exposed to the tides on the coast of Maine and contain steel permanently stressed to as much as 50,000 psi. At the time

STRAIN DIAGRAMMS FOR $f_y = 60,000$ psi

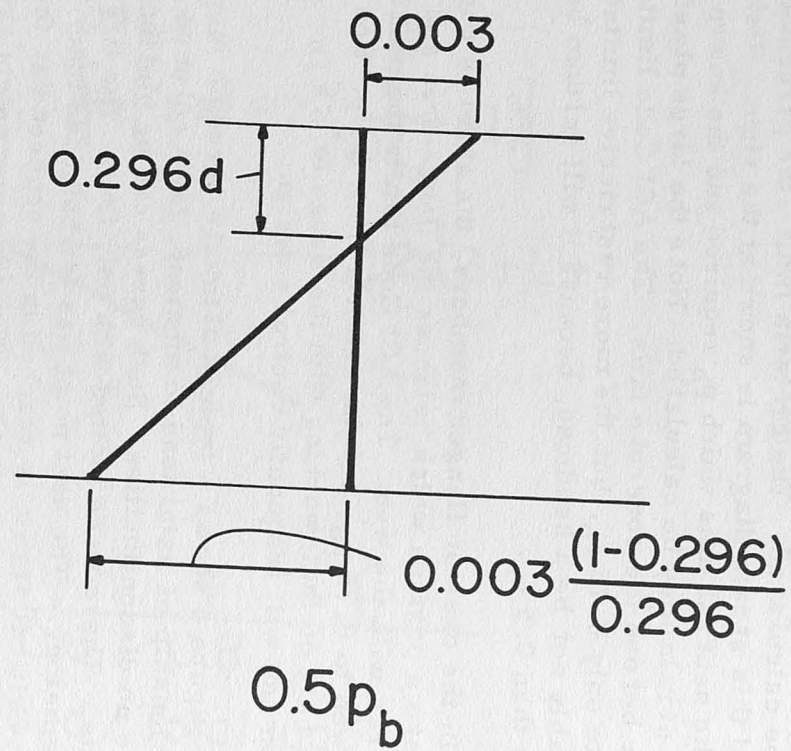
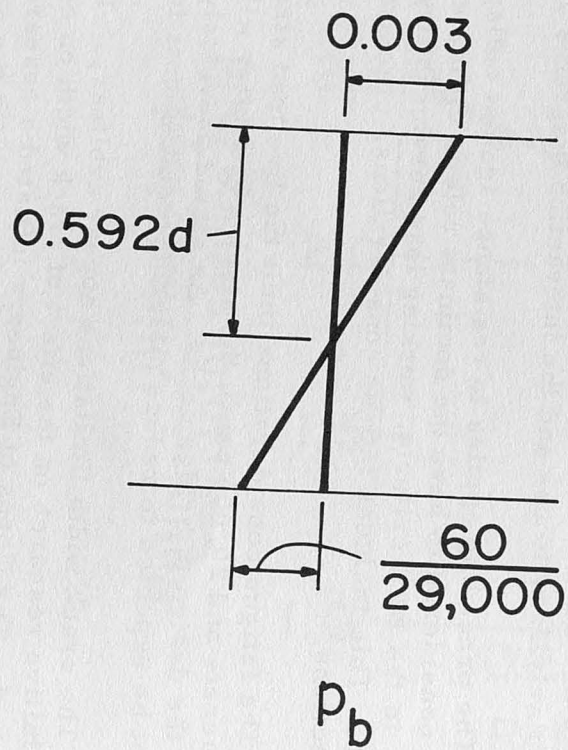


Figure 3. Strain Diagrams.

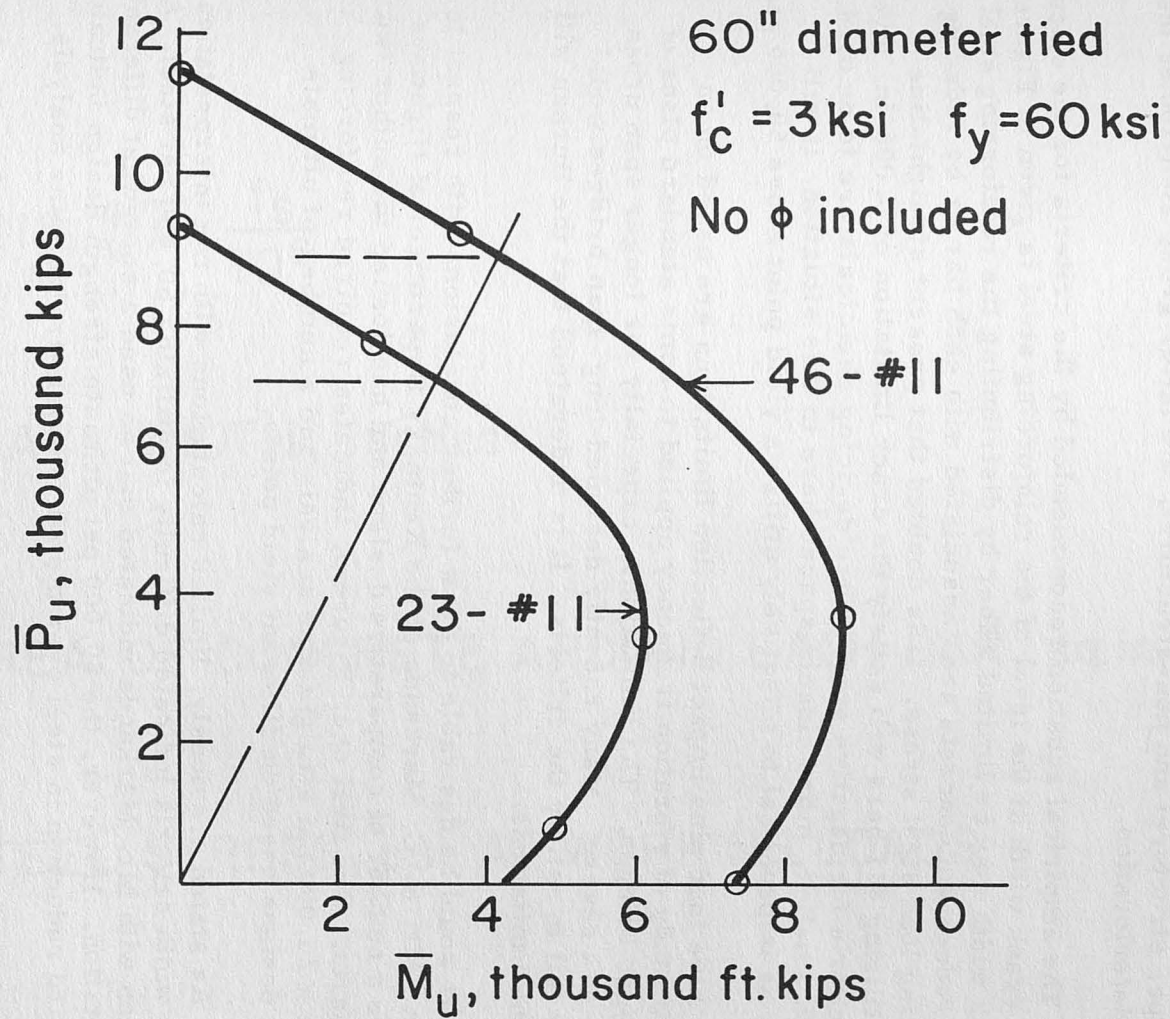


Figure 4. Typical Ultimate Strength Interaction Diagram.

of their last report in 1964, sufficient information had not developed to draw conclusions. Twelve State highway departments, the Bureau of Public Roads and the Portland Cement Association are cooperating in a comprehensive bridge deck study. Indications are that if cracks are divided into two groups, structural and non-structural, the latter group contributes the most to deterioration.

The empirical equation recommended by the criteria for the calculation of crack width at the level of the reinforcing steel is given in Figure 5. Crack width may be limited either by distributing the reinforcing steel so as to reduce the concrete area associated with each bar or by reducing the working load steel stress. It is doubted that present straight-line design utilizing #11 bars will satisfy the crack limitation of 0.006-in. given by the criteria for negative steel where de-icing chemicals are to be used. Even with smaller, more closely spaced bars in this situation, it will probably be impractical to effectively utilize a yield point above 50,000 psi.

Live load plus impact deflection limitations are based on an analytical study of vibration frequency applied to some standard plans of simple-span bridges. The limitations, especially for longer span urban bridges, are severe. Many recently designed long-span bridges would probably fail to satisfy the criteria. It is understood that the Bureau will modify the limitations.

It would be desirable to draw further conclusions with respect to economy. Mr. S. C. Markanda of the Kentucky Department of Highways has made a number of comparative designs and has loaned the author two of his charts. The first one, Figure 6, indicates required reinforcing steel for a 20,000 psi straight-line analysis and a number of ultimate strength designs based on different yield points.

As stated previously, limited calculations with my interpretation of crack width criteria indicated difficulty in utilizing 60,000 psi steel. A discussion with Mr. Markanda indicated that he used a somewhat different interpretation. However, the 50,000 psi ultimate strength design indicates a one-third reduction in steel over the 20,000 psi straight-line analysis.

The next chart, Figure 7, shows costs calculated with Kentucky units and indicates approximately a saving of 50 ¢ per sq. ft. of the 50,000 psi ultimate strength design over the 20,000 psi straight-line analysis. These are superstructure costs. Columns in substructures also should produce substantial savings, unless they are already at a minimum size for esthetic reasons.

It has been shown that ultimate strength design is not entirely new and that present AASHTO specifications incorporate some ultimate strength provisions. Some aspects of the new BPR Criteria have been discussed. It is felt that the philosophical and practical advantages of ultimate strength design are such that it is the design method of the future.

$$w = 0.115 \sqrt[4]{A} \frac{f_s}{10^6}$$

w = CRACK WIDTH, inches

A = AREA OF CONCRETE AROUND EACH BAR, sq.in.

f_s = STEEL STRESS, psi AT WORKING LOADS

Figure 5. Empirical Crack Width Equation.

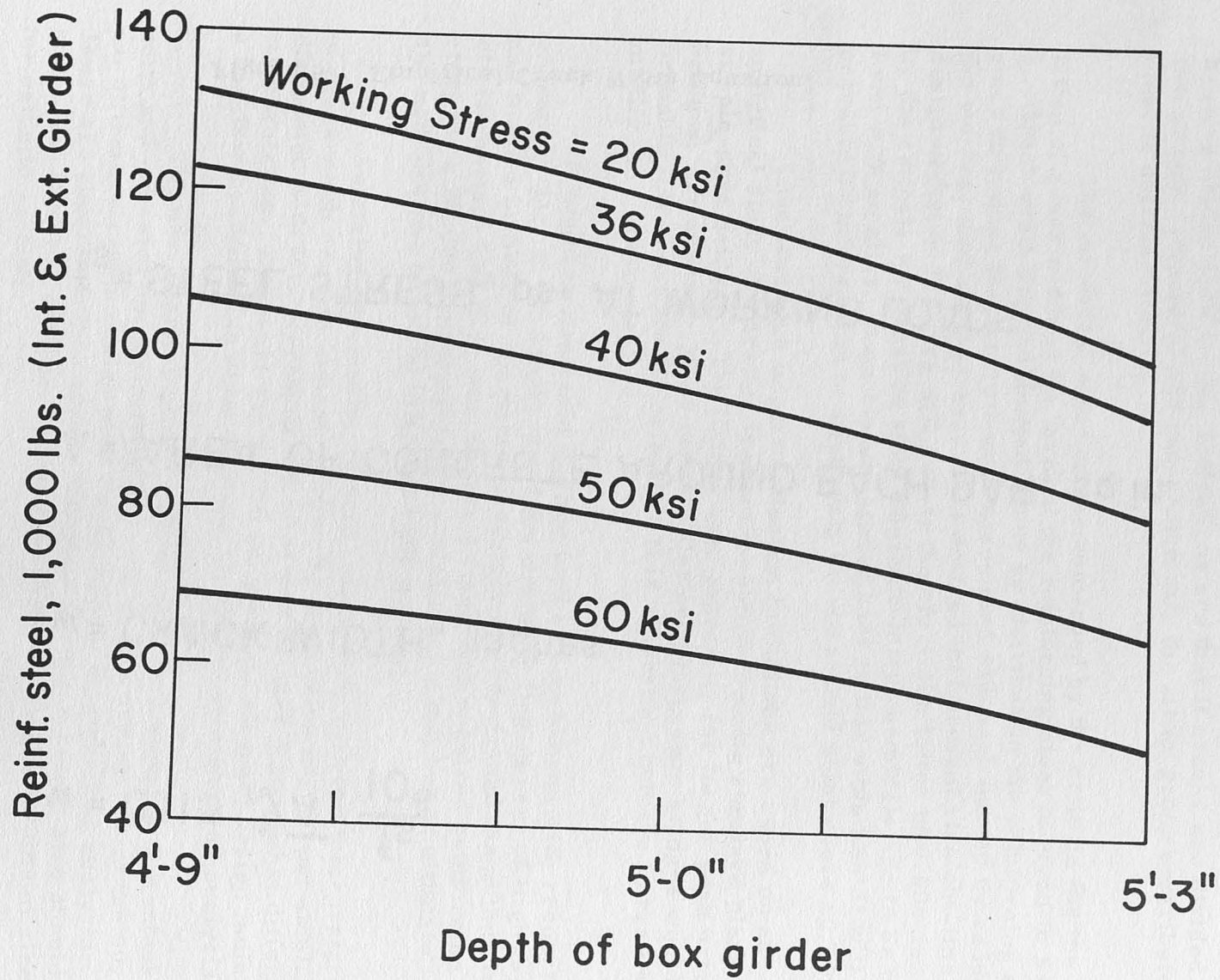


Figure 6. Required Reinforcing Steel.

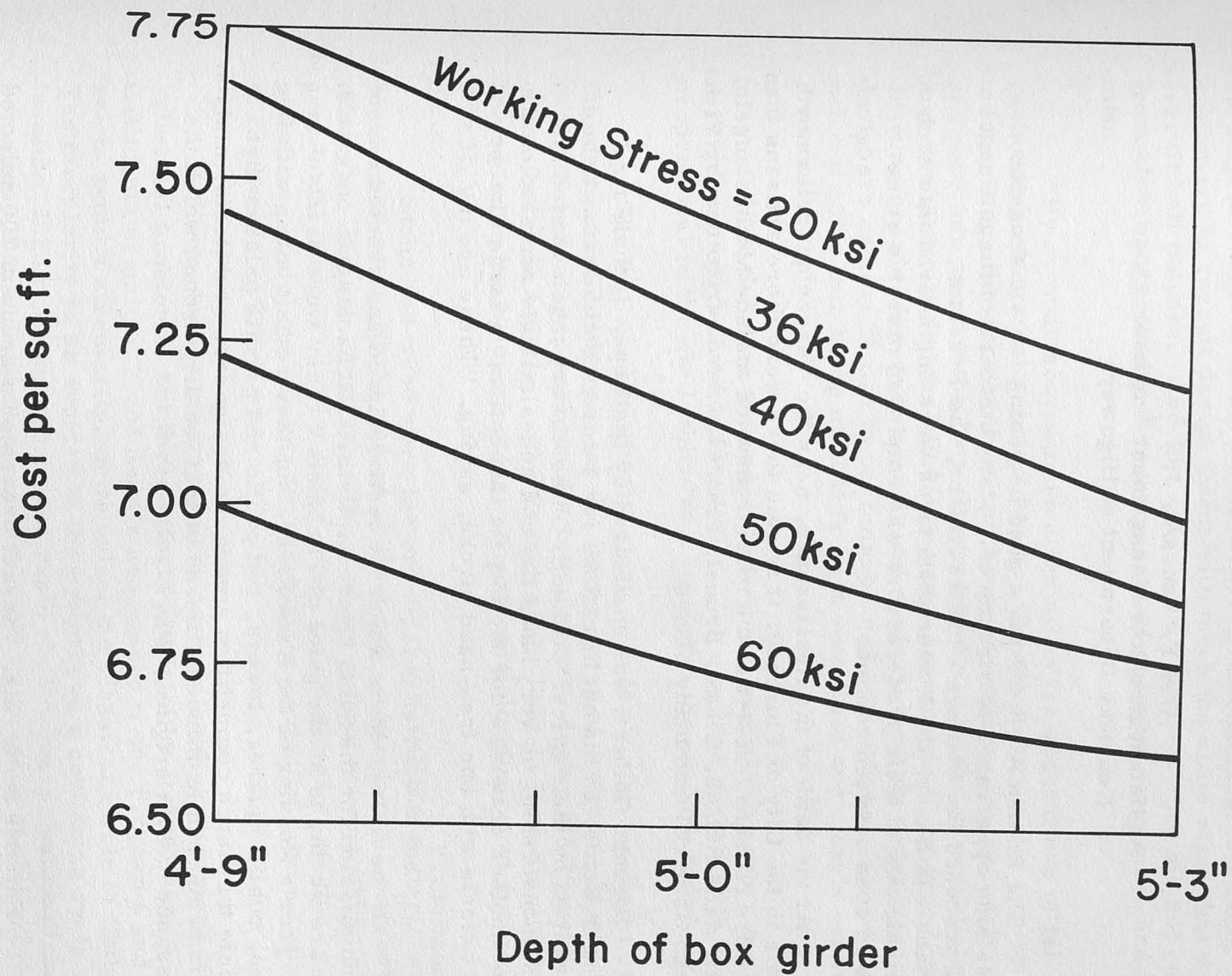


Figure 7. Comparative Costs.