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USING ACI 318 - THE EASY WAY

Over the last two years, numerous CRSI Chapters have had Mr. Fling make his presentation at local chapter breakfast/luncheon meetings. This article illustrates how a practicing engineer can successfully use the building code as it now stands without spending an excessive amount of time doing so. Mr. Fling discusses a number of techniques for designing safe, economical structures while maintaining a profit on minimal fees which he and his partners have developed for their own consulting firm.

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USING ACI 318 THE EASY WAY

by Russell S. Fling

Engineers who feel the ACI Building Code (318-77) is an exercise in puzzles, games, math, and logic can benefit from this article by utilizing ways of simplifying the Code. Computations of concrete shear, stress, column design, torsion, etc., can be solved with common sense. The author has developed a time and money saving approach to the use of the Code. Construction economies are also suggested.

Keywords: *building codes;* deflection; *reinforced concrete;* reinforcing steels; shear properties; shrinkage; stresses; *structural design;* torsion.

Most engineers like puzzles and games involving math and logic. Many of us were raised on a steady diet of such puzzles from childhood. We chose engineering as a life profession since it promised the opportunity to make a living while exercising our penchant for interesting games. Perhaps this explains why ACI Building Code (318-77) has become a compendium of lengthy equations and intricate problems of logic.

If you started reading this article hoping to find an elegant new mathematical twist to reduce your labor in using the ACI Building Code, read no further. I will make no such promises but rather point out an approach that permits you to devise your own shortcuts.

Likewise, we engineers are only human and like to complain about things that bother us. If you are reading this article hoping to find more fuel to feed the fires of complaint about code complication, read no further. This article has other purposes.

There is a distinction between how to simplify the Code itself and how to simplify the use of the Code as it stands. The former is a difficult task and cannot be accomplished by individual action. The latter is much easier. Everyone has the opportunity and prerogative to simplify the use of the Code to suit his individual preference and the situation at hand.

Being a naturally lazy person, I have developed a time-saving approach to the use of the Code with the inspiration and assistance of my colleagues and my employers during the past 30 years. In sharing these ideas with you, I hope they will inspire your thoughts along the same line.

WHY IS THE CODE SO COMPLICATED?

The classical first step in solving any problem is to define it. Let us examine some of the factors which make the present code seem so complicated. As a base period for comparison, I will use 1949 for no reason other than that it is the year I first started practicing structural engineering and is therefore a time period about which I can speak with some authority concerning the condition of the reinforced concrete design. Even then, engineers complained about the complexity of the reinforced concrete design. In the last 30 years reinforced concrete design has admittedly become more complicated, but so has the design of other structural materials, the U.S. Internal Revenue Service code, and life itself.

In 1949 we used an allowable steel stress of 20 ksi, even though at least three grades of steel were available, some of which are essentially the same steels available today. By contrast, today we are permitted by the Code to design using steels with a yield point of 40 ksi, 60 ksi, or with prestressing steels having an ultimate strength of 150 to 270 ksi. Today, selection of the proper steel to use as well as complying with the additional code provisions related to steel stress is time-consuming.

The office where I worked in 1949 used 3000 psi concrete for everything: footings, columns, beams, slabs, joists, etc. Other firms may have standardized on 2500 psi or another concrete strength, but rarely were strengths higher than 3000 psi used except by venturesome souls under the behest of clear economic advantage. Today, concrete strengths from 3000 to 9000 psi varying by 1000 psi increments or less are in common use, and most important, individual projects use two or more levels of concrete strength. Again, the selection of the correct strength to use as well as following the mathematical requirements of the Code is time-consuming.

We used no admixtures of any description in 1949: no air entraining, no set retarding, no water-reducing, no pozzolans or any other type of admixture other than an additional sack of cement. Today, admixtures are used on virtually every job.

In 1949, we simply estimated the actual loads and used them in the design. Loads were crude and standardized, and little thought was given to whether an 8 in. concrete block wall actually weighed 51 psf or not. Today, we agonize on the magnitude of the actual loads, then apply an esoteric factor depending on the nature of the load. It is not always clear what the factor should be or exactly how the loads and factors should be combined to arrive at a rational design.

We used to compute the required area of reinforcing steel by simply dividing the actual bending moment at a section by the allowable steel stress and the internal lever arm *($A_s = M/f_s jd$)*.* Today, the area of reinforcing steel cannot be calculated directly except by use of a quadratic equation so frightening that I have never seen it in print.

In 1949, we used to check the capacity of concrete in bending by dividing the bending moment by the width of the compression face and the square of the effective depth *($K = M/bd^2$)*. The resulting number was compared to a limiting value which was 236† for 3 ksi concrete and 20 ksi steel. By contrast, in ACI 318-77 the procedure for checking the concrete flexural capacity is presented as a lengthy procedure, not amenable to reduction to a simple formula.

Shear capacity of the concrete was estimated by dividing the external transverse shear at the face of the support by the web width and internal lever arm *($v = V/bjd$)*. Today, we use the transverse shear at an artificial distance away from the support, use the square root of the concrete compressive strength, and modify the result by additional factors designed to increase safety and give credit to increased shear capacity in certain instances. In 1949 stirrups were sized and spaced by slide rule‡ in seconds. I know of no such brief procedure using a pocket calculator.

In 1949, we computed the bond stress by dividing the transverse shear by the perimeter of the bars and the internal lever arm. *($u = V/\Sigma_o jd$)*. It was then compared to an allowable bond stress. With a little practice, the experienced structural engineer compared the bar perimeter to the beam width by inspection without making any calculations at all on

TABLE 1 — DESIGN ROUTINES

1949	1978
$f_s = 20$	$f_y = 40, 60, 270$
$f_c' = 3$	$f_c' = 3, 4, 5, 6, \ldots$
—	Admix
ACTUAL — LOADS — FACTORED	
$A_s = M/f_s jd$	$A_s = \sqrt{\ }$
$M = Kbd^2$	V. C.*
$v_c = V/bjd$	$V_c = 2\sqrt{f_c'}\, b_w d$
$u = V/\Sigma ojd$	V. C.*
$P = f_s A_s + f_c A_g$	V. C.*
—	Torsion
—	Cracks
—	Deflection

* Very Complicated, similar to V. D.†
† Very Difficult

95 percent of the beams. Today, we have changed our approach to concepts of anchorage and development lengths with a considerable increase in design effort.

In the late 40's and early 50's steel was still in short supply. If a bar had any bumps on it at all, it was called a "deformed bar" and allowable bond stresses were twice as high as for a plain bar. Nevertheless, we had to make extensive use of "merchant bars" — smooth bars of unknown chemistry and strength. Structural engineers almost always called for 180 degree hooks on bars at the ends of beams. The contractors just as universally cut off these hooks because the bars would not fit within the forms otherwise. Not knowing this, we engineers spent a great deal of time and money specifying hooked bars.

The design of columns was a paragon of simplicity. We simply added the individual vertical load carrying capacities of concrete and longitudinal reinforcing steel to obtain the capacity of a column. Bending moment was normally ignored if it was due to frame action. We never worried about column slenderness or buckling. By comparison, the procedures for column design today are so complicated that hardly anyone attempts it by manual methods. Instead, engineers resort to published graphs and tables or computers.

In 1949, we rarely worried about torsion, as that was something that happened to drive shafts in automobiles and other pieces of equipment but not to concrete. In 1955 I became concerned about the effect of a slab cantilevered from the side of a beam and spent several days researching how to compute the torsion in the beam. The only literature I found gave a funny formula of obscure parentage. In spite of the indifference to torsion, we rarely had problems with it. Today we know fairly well how to design to resist torsion (but not how to compute its magnitude). Since we know now how to design to resist torsion, now we must.

*For those more studiously inclined, the ratio of an internal lever arm to effective depth, j, could be computed or selected from an appropriate table. It miraculously always turned out to be .875 ± 1 or 2 percent. Virtually all engineers of the day used 7/8 for the value of j.

†A number that is forever indelibly engraved in my mind and that of many other structural engineers, no doubt.

‡A plastic or aluminum (antique models: bamboo and ivory) tool, about 2" x ¼" in cross section by 12" long, inscribed with logarithmic scales, arranged for graphical solutions of mathematical problems in multiplication, division, trigonometric and/or exponential functions.

Cracks were assumed to be a part of concrete that you accepted along with its better qualities. No one worried about performing structural design to minimize their occurrence.

The common wisdom of 25 or 30 years ago was that concrete simply does not deflect, or at least not enough to be concerned about. If one wanted to satisfy one's academic curiosity, the PCA had a little bulletin that suggested ways of computing deflection, but rarely did anyone do so. Today, deflection is a major concern because higher strength concrete and steels have permitted shallow members that do exhibit substantial deflection and frequently cause problems.

These design routines are summarized in Table 1.

Added complications of the ACI Building Code have not been without their benefits. As a result of refinements in the last 30 years, structures designed under ACI 318-77 use perhaps two-thirds of the concrete and reinforcing steel that structures designed under ACI 318-47 would have used. Fig. 1 illustrates the designer's dilemma. As the design complexity of the ACI 318 Building Code has increased, it has enabled designers to reduce the construction costs by a substantial amount. This comes about both from the use of higher strength concrete and reinforcing steel and also by a reduced factor of safety. The "price" for reducing the factor of safety is taking greater care in design to investigate more variables, define the limit of applicability, and take advantage of the latest in research results.

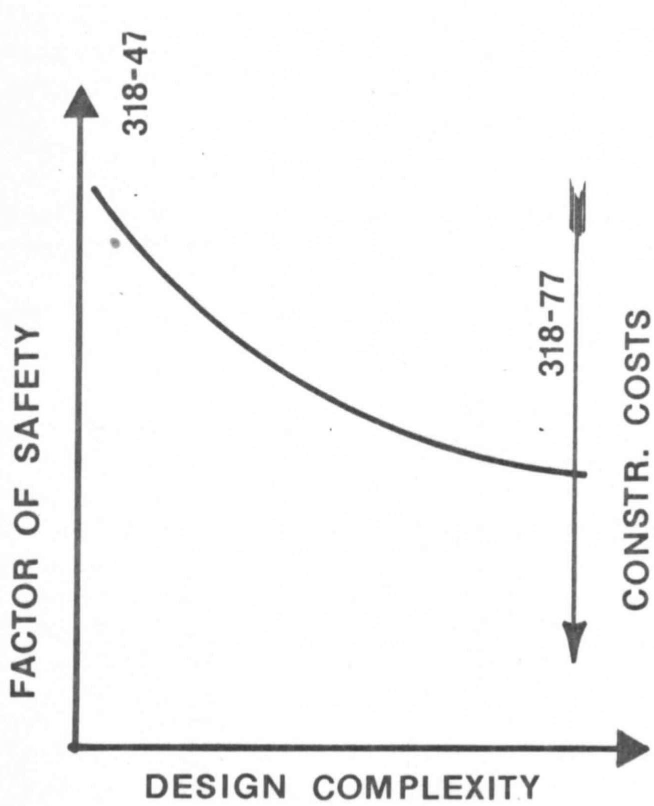

Fig. 1 — Designer's dilemma

Fig. 1 suggests an obvious way to simplify use of the Building Code. One need only return back up the curve and gain the simplicity of the 1947 Code at the expense of construction costs. While this is not a viable solution for a large and important project, it is certainly worth considering on small, incidental projects, especially those where design costs could exceed the potential construction cost saving. There will be more about this later. First let us discuss other ideas for saving design time without sacrificing the benefits of construction savings or assurance of safety.

SOLUTIONS

One of the easiest simplifications to implement is the determination of reinforcing steel area required by using the formula $A_s = M/30d$. M is the bending moment due to actual unfactored loads, assuming grade 60 rebars and 3 ksi concrete. This equation always produces safe designs when the live load is equal to or less than the dead load and usually produces safe designs for all other situations also. If 33 is used in the denominator instead of 30, the equation is usually still safe. In people-occupied buildings, live load rarely exceeds the dead load. The reader should derive this equation for himself before using it, to determine for himself the limits of its application, consistency of units, and to be able to justify his use if questioned by building code officials. From then on, the engineer will save a lot of time using this approach with no sacrifice in safety and little sacrifice in economy.

Although the compressive strength in flexural members is far less of a concern today than it was 30 years ago, it can be checked by the same equation as used then, that is, $K = M/bd^2$. Due to ultimate strength concepts, K is now approximately 600 instead of 236 for the same set of material strengths. The engineer should derive this equation also for the same reasons mentioned above. The reduction in design time will be spectacular.

In maintaining the integrity of a beam the value of shear reinforcing is much higher in proportion to its weight and cost than is longitudinal steel. Much design time can be saved by ignoring the complicated refinements present in ACI 318-77 that allow an engineer to justify the use of fewer stirrups. A few extra stirrups will be required from time to time, but the cost will be much less than the cost of design time expended in trying to save them. If you are not already familiar with the concepts of truss analogy or stress volume in sizing and spacing stirrups, I recommend them. They are still valuable, time-saving concepts that do not conflict with ACI 318-77 requirements. Furthermore, sizing and spacing stirrups by slide rule is far simpler than any procedure using a pocket calculator. Description of these concepts is beyond the scope of this article but is available in the literature elsewhere.

Anchorage and development length provisions in the current Code are much more time-consuming than the former bond provisions. One way to avoid this design effort is simply to make bars long enough so that anchorage and development length need not be checked. The extra length in many cases is only inches and rarely more than a foot or two. Ironworkers rarely, if ever, take time to measure the location of rebars accurately, especially the end distance. Straight bars cut off in the middle of the span to fit the moment curve can be downright dangerous if not located accurately. Contractor and inspector efforts to insure such accuracy cost far more than the savings in steel. Top bars over the support should extend an equal distance on each side, as this is surely the way the ironworker will place them. A few more inches of extension will save construction effort, insure safety against unexpected moment patterns and placement errors, plus save the designer's time also.

Deflection computations can be eliminated by simply making slabs and beams about twice as deep as required by Table 9.5(a) of ACI 318-77. This will put the span depth ratios in the same range that we used 25 or 30 years ago. Another approach is to limit K to about 236 as we did then. With these procedures there were few deflection problems in 1949, and doubtless there would be few today.

There are only two reasons for trying to limit crack formation and width: appearance and durability. Look at the floor and the ceiling of the room you are in now, or in your office tomorrow morning. Are you concerned about unsightly cracks in the concrete? Chances are you are not, because you can't see the concrete. It is covered with some finishing material. Most people-occupied buildings in the United States today have conditioned air resulting in a humidity lower than the Sahara Desert. We know structures there can last thousands of years, so why are we concerned about the durability of concrete under such conditions here? Bridges, parking garages, and other structures subject to severe weathering exposure are another matter. In such cases, the most effective procedure is to proportion the structure so that tensile stresses do not exceed the cracking strength of plain concrete. Then, be sure the concrete is not cracked by overloading at an early age before the concrete has reached its mature tensile strength. Of course, one must use reinforcing steel in the normal fashion despite these precautions. This procedure does not result in structures as heavy as might seem at first as many, maybe most, structures meet the criteria already.

Column design offers vast opportunities for simplification by observing that the Code does not require you to avoid giving columns a strength exceeding the load by an arbitrary amount.* Fig. 2 shows a typical column interaction curve† for the 1977 Code and one for the 1947 Code. Column designs are not required to fall within a narrow band, say 10 percent, of the interaction curve. Rather, every design within the curve will meet the Code requirements. If column charts or tables are not available or do not meet your fancy, you can still design columns using the 1947 Code interaction diagram but with 1977 Code intercept points (Line A in Fig. 2). Better still, for more economical columns and only a little extra effort, use the minimum eccentricity intercept (Line B in Fig. 2). This procedure will always be safe, frequently be as economical as a more extensive design procedure, and simplifies design dramatically.

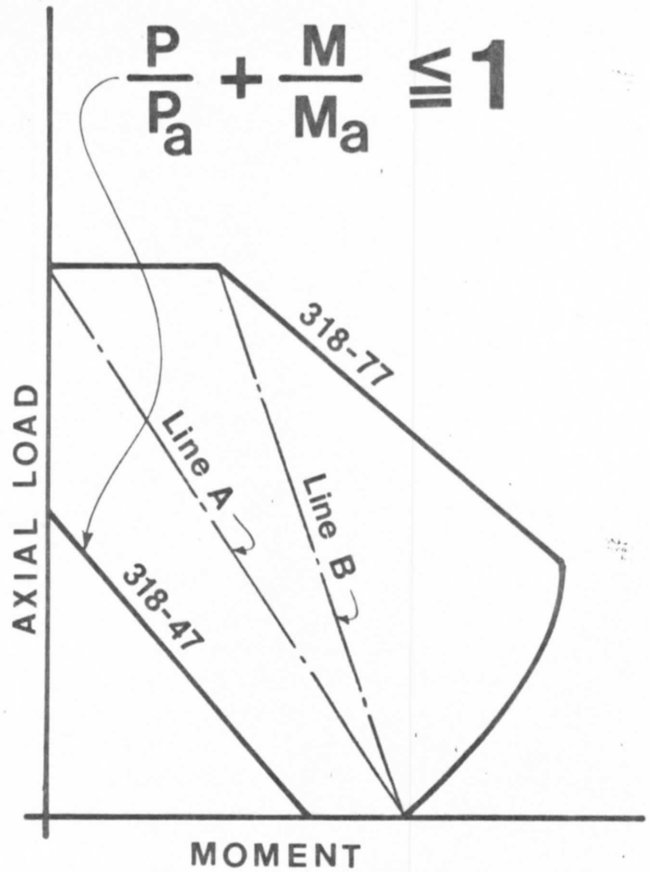

Fig. 2 — Typical column interaction

ACCURACY

By now you may be thinking that I advocate sloppy, careless or loose computational procedures. Not so. I believe it makes sense to examine the real accuracy of structural computations and be guided accordingly. Let us briefly look at some aspects of accuracy.

After examining a few hundred concrete test cylinder reports, it is obvious to anyone that even concrete from a single supplier will vary by 3 or 5 percent in unit weight. We add 1/2 to 10 percent reinforcing steel to the concrete for an additional 1 to 20 percent variation in weight. Considering the tolerance in concrete dimensions results in more

*The first words in the Code are: "This code provides *minimum* requirements —"
†Perhaps the sexiest curve in all engineering literature — something that only a few degenerate souls like me have noticed.

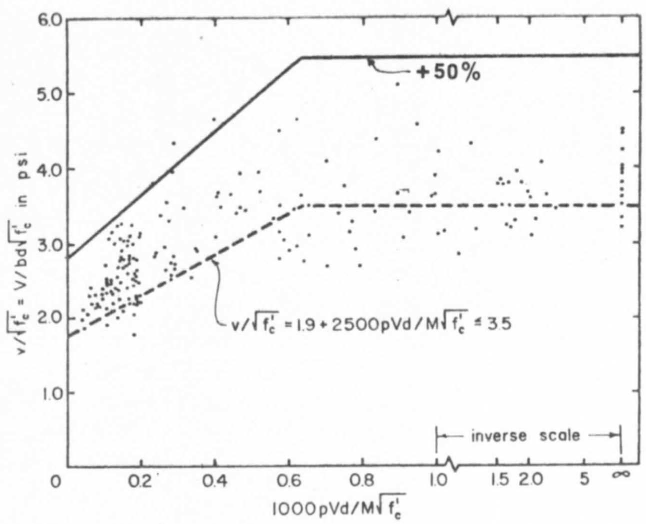

Fig. 3 — Shear and diagonal tension

variation in weight. Yet engineers universally assume the dead load of concrete can be determined by multiplying the theoretical volume by an assumed unit weight (150 pcf for normal weight concrete.) Rarely can the loading on a structure due to dead weight of concrete be determined within an accuracy of 10 percent by normal methods of calculation.

Live loads specified by building codes are usually higher than expected in actual service, but not always. Consider the school classroom when everyone rushes to the window to see the passing parade. Actual loading could be easily double the specified live load. On the other hand, I have yet to see a place of assembly loaded to the 100 psf normally specified. It is usually in the range of 3 psf to 8 psf, but occasionally may reach as high as 20 to 25 psf. Wind loads are even more variable, as some engineers have found to their sorrow.

Building code provisions are based on thousands of laboratory tests. In formulating their recommendations, researchers and engineers, of necessity, must be conservative. Fig. 3 is taken from a typical research paper in which a proposed equation is compared to test results. I have added an additional line which is 50 percent above the proposed Code equation. Note that much of the test data is higher yet. Fig. 3 is the work of highly competent and widely respected researchers. Clearly, the accuracy of Code equations based on actual test results is no better than 20 percent and could range as high as 100 percent or more.

Codes are based on other assumptions, such as the homogeneity of concrete. Almost universally we engineers analyze a structure for distribution of moments by assuming monolithic, uncracked, plain concrete. This is a convenient assumption for maintaining the financial solvency of design firms but hardly in tune with reality. A few years ago I tested this assumption by analyzing a theoretical structure for varying conditions of moment of inertia caused by cracking and placement of reinforcing steel as well as for loading conditions during construction when only a portion of the structure is completed. Preliminary results indicated that moments in beams and columns in interior spans were typically 5 to 10 percent different from those computed by conventional analysis. In end spans, moments were typically in error by about 50 percent and frequently much higher. ACI 318-77 takes great pains to spell out the limits of redistribution of moments. The structure, being ignorant of how it is supposed to act, has gone about redistributing much larger moments with none of us being the worse for it.

After making design calculations, experienced engineers will select member sizes with an accuracy of about 10 percent. Steps of 20 or 30 percent are common. For example, the variation in area of single reinforcing bars ranges from 23 to 90 percent, although combinations can be selected with less variation. Only a rank amateur would vary concrete outlines by 5 percent or less (e.g., 12 inches, 12½ inches, 13 inches, etc.) due to changes in stress. Experienced engineers will vary the sizes by 4 inches or more.

Table 2 summarizes the above factors affecting accuracy.

In the face of all this, how can we be so sanctimonious about a calculation accuracy greater than 5 or 10 percent? (I am not talking about a general undersizing of the structure by this amount, only the accuracy of the calculations.) In days of old, we used to refer to slide rule accuracy of 3 decimal places. For engineers with weak eyes like mine, let's say the accuracy is 0.3 percent. With this accuracy, if 20 consecutive operations of multiplication or division are performed on a slide rule, the error will still be about 6 percent even if the maximum error occurs on each operation and in the same direction. Six percent is less than the error due to any one of the above factors and hardly cause for alarm. Is it any wonder the old timers thought slide rule accuracy was good enough? Today we are enamored with pocket calculators and high-speed electronic computers. Calculations are registered with 8 significant figures, or more. Are structural calculations made today on electronic gadgetry any more accurate than those of 30 years ago? I don't think so.

TABLE 2 — ACCURACY

ACCURACY OF:	
Loading:	10%→200%
Test results:	20%→100%
Assumptions:	5%→ 50%
Member select:	10%→ 30%
Slide rule:	±0.3%
$(1.003)^{20} = 1.06$	
Calculation series:	
$C^n = A$	
$(1.01)^9 \leqslant 1.10$	

I am not suggesting sloppy calculations merely because our assumptions are so imprecise. Rather, we should realistically appraise the real accuracy that is attainable. Most structural calculations involve 9 steps or less. If each step is performed with an accuracy of 1 percent, the end result will be accurate within 10 percent under all conditions and usually far more accurate than that. Considering the conditions of the real world, this should be sufficient accuracy for even the most meticulous person. Structural calculations would be a fruitful area of study for some group interested in tolerances and accuracy.

DESIGN COSTS VERSUS CONSTRUCTION COSTS (Your pocket versus someone else's)

Consider the case of a concrete column with four #9 vertical bars required by a shortcut, abbreviated method. The engineer suspects that he might be able to justify four #8 bars if he checked the design by more refined methods. The difference in weight is about 44 pounds for a column of average height. If steel costs $0.25 per pound, the difference in cost is $11.00. If the engineer's billing rate is $25.00 per hour, the break even point is $11.00/2 X $25.00 = 0.22 hours = 13 minutes. At this point, *society* has broken even. Eleven dollars has been transferred from one pocket and put into another pocket, but there has been no net savings in national resources. Note the factor of 2 in the denominator. One must assume that half the time a calculation of this sort is undertaken, it will be unsuccessful. To assume otherwise would render the calculation unnecessary. Simply use four #8 bars if you are sure it will work. At 13 minutes, society may have broken even, but the engineer is poorer by $11.00 and someone else is richer. This doesn't make much sense to most engineers if they are on a fixed fee, so I suggest the engineer not even start on such a calculation unless he expects to be able to complete it within five minutes or less. This allows him to discharge his professional obligation to his client without overpenalizing himself in the process. Furthermore, the average person greatly underestimates the time it will take to do something, so my advice is don't even start on this calculation unless you are sure it can be completed within one or two minutes including the time it takes to make the decision to proceed or not. Engineers should get in the habit of making an analysis of this sort very frequently if they wish to remain solvent.

CONSTRUCTION ECONOMY

By now, you may be wondering how anyone who approaches design in such a manner can remain in business in today's competitive climate. Owners and contractors are simply too sophisticated and too demanding to permit an engineer the luxury of design shortcuts at the expense of construction dollars. Let me assure you that it is not only possible to survive, but to prosper with this philosophy. A substantial proportion of the work of our design office has always been performed for contractors and fabricators who believe we know how to design economical structures. Following are some tips, culled from many sources, on how we do this.

SIMPLIFY CONSTRUCTION by selecting just one framing method for a given structure, orienting the framing members in just one direction, avoiding unnecessary special equipment, using methods familiar to the contractors in the area, and by anticipating the problems the contractor may encounter.

SIMPLIFY DRAWINGS by eliminating extraneous information and paying careful attention to good drafting techniques. Simple drawings are usually easier to prepare and that saves the engineer money. Contractors are more inclined to bid lower on drawings that are easy to understand. Field personnel will have less difficulty reading simple drawings, spend less time building the structure, and spend less of the contractor's money.

COOPERATE WITH THE CONTRACTOR if you expect to get a low bid on the next job. He is only human and has the same wants and desires that engineers have. (E.g., a home in the suburbs, vacations in Florida, college education for his children, etc.)

BE CONVENTIONAL if you want to save money. I did not tell the contractors for my first hyperbolic paraboloid shell what it was called, not even a hypar shell. Rather, I told them it was a flat slab, 3 inches thick with one corner lowered about 2 feet from the others. We had a prebid conference to demonstrate how plywood will warp to fit, how a few extra triangular pieces of plywood were necessary, and to reassure them on some other details. Only later did I tell them that they had just completed an immensely difficult, intricate piece of structural engineering.

SAVE FORMWORK by using concrete and reinforcing steel. In most structures the formwork costs

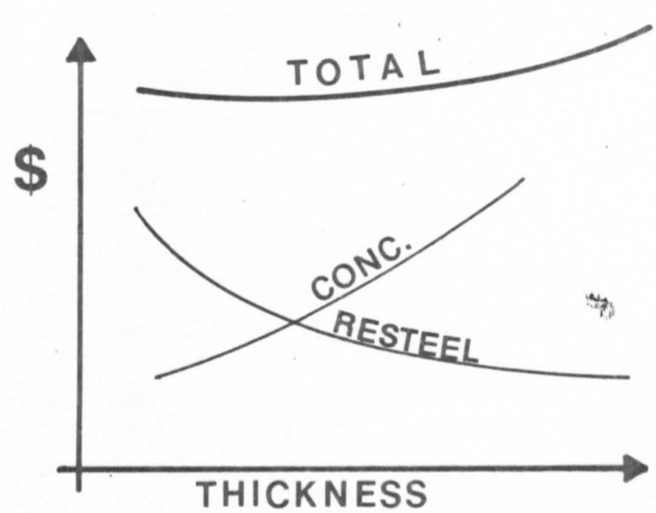

Fig. 4 — Solid slab costs

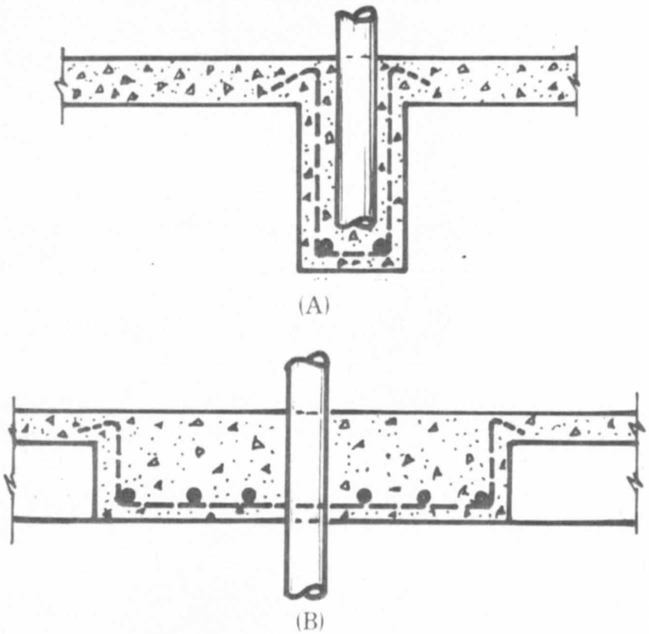

Fig. 5 — Concrete beam proportions

50 percent or more of the total cost of a concrete frame. For a given span and loading condition, the volume of concrete and reinforcing steel is about constant regardless of the framing method used. If money is to be saved, you must do it in the formwork. Therefore, great attention should be given to laying out a framing system that will minimize formwork labor and facilitate its reuse. In the process, one should consider simply making slabs or beams heavier in order to simplify or reuse the formwork. For example, in Fig. 4, in a solid slab, the cost of concrete goes up as the thickness increases, but the cost of reinforcing steel goes down, at least to a certain point. The total cost of concrete plus reinforcing steel remains relatively constant within a broad range. Minimum depth slabs are rarely the most economical, especially if formwork savings can be realized by increasing the dimensions.

Years ago, conventional wisdom assumed that the optimum proportions of a concrete beam were 2:1 (see Fig. 5A). Engineers insist on placing columns under the ends of beams to hold them up, so beams frequently coincide with column centerlines. Architects also like to place walls on column centerlines to avoid having the columns located in the middle of rooms where they interfere with foot traffic and have other undesirable effects. Toilet rooms back up to those partitions and the soil pipe is located on or near the column centerline, resulting in the condition illustrated in Fig. 5A. The engineer may design the beam in 15 minutes or one-half hour, but he will spend half a day trying to figure out how to get a 6 in. soil line through a 12 in. beam. A far better solution is to make the beam much wider than it is deep, as in Fig. 5B. Pipes, conduit and even duct work can pass through the beam vertically without distress. The ironworker can place the steel without removing all the skin from his knuckles. The mechanical engineer can run his duct work unobstructed throughout the structure. The cost of a wide flat beam may be a little higher in concrete and reinforcing steel than a deep narrow beam, but the structural engineer goes home at night with a few extra dollars in his pocket and without a case of migraine headache.

ONE COLUMN SIZE should be used wherever possible. Obviously this saves the cost of column formwork if the forms can be used repeatedly. Less obvious is that the beams or slabs around the column can also be reused without reworking. Column ties can be the same, saving fabrication and placing effort. Even more important, the cost of carrying load vertically on concrete is about one-third the cost of carrying it on reinforcing steel. It saves money to use large columns rather than reducing the size in upper stories. The one column size dictum applies horizontally throughout the structure as well as vertically from floor to floor.

UNIFORM COLUMN SPACING helps assure uniform loading and makes uniform size and reinforcing possible. Furthermore, the floor framing can be made uniform, thus facilitating repetition.

ONE BEAM SIZE should be used if at all possible even though spans and loading differ. A few dollars of extra concrete will be more than offset by the savings in time and labor and reduction in reinforcing steel.

ONE DECK DEPTH enables the contractor to use one length of shoring. Also, fewer problems will be experienced in coordinating concrete construction with mechanical trades, thus saving everyone time and money.

COST COMPARISONS on a typical bay should be made on as many different methods of framing as reasonably possible before one method is selected. Detailed analyses don't cost, they pay in the long run.

FINALLY, remember that simple construction means simple design and dollars in your pocket.

SI equivalents:
1 ksi = 70.31 kgf/cm²
145 psi = 1.0 MPa
1 sq ft = .0929 m²
1 pcf (lb/ft³) = 16.02 kg/m³
1 in. = 2.54 cm
1 lb = .4536 kg

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