It has been suggested that curves be drawn giving coefficients for varying column stiffness similar to those now in use for cases of equal spans. The writer has not thought the idea very practical at this time. The present system of coefficients does not distinguish between live and dead load, and does not use the span length between centers of intersection of the members. Such curves as proposed would, then, involve four variables—ratio of dead to live load, ratio of girder stiffness to column stiffness, ratio of clear span to span length between intersection of members, and the moment coefficient. The difficulties involved in presenting curves having four variables are very great.

There exists some difference of opinion among designers as to what combinations of loads should be considered in determining maxima, especially in regard to the weight to be given in cases of split loading. This question at least should be settled before any attempt is made toward further standardization.

4

THE RELATION OF ANALYSIS TO STRUCTURAL DESIGN

Synopsis

Confusion sometimes exists in structural design as to the use to be made of analyses. The designer soon realizes that precision is futile in some cases and important in others, and the experienced designer realizes fully that analysis of the conventional type is frequently a poor guide to proper proportions. That analysis shows a certain member to be overstressed commonly indicates that the member should be made larger; but the over-stress sometimes has little importance and may be disregarded. In some cases where the over-stress is serious, the best solution is not obvious; sometimes the structural layout should be changed entirely.

Critical study soon leads to recognition of important differences between load-carrying stresses and stresses which produce no appreciable resistance to the applied loads. The latter may be due either to external movement of abutments or to internal distortions, or they may be due to deformation induced in one part of a structure as a result of that in another part. The load-carrying stresses may also be diveded into two groups. The distinction in this case, however, is based upon response to changes in design; these sub-groups are not always clearly distinguishable, but they have characteristics so widely different in certain cases as to force their differentiation.

A classification is presented herein, with the idea of suggesting a convenient arrangement of certain familiar characteristics rather than with any wish to define the groups formally. The designations suggested, therefore, are for convenience of reference only. The non-load-carrying stresses will be distinguished as Deformation Stresses and Participation Stresses; and the load-carrying stresses as those normal in their characteristics and those which are hybrid.

Note.—Presented at the meeting of the Structural Division, New York, N. Y., on January 17, 1935. Discussion on this paper will be closed in January, 1936, *Proceedings*.

Deformation stresses are a consequence of strain and strain is a consequence of internal or external movements not due to stress in the structure, such as abutment movements, shrinkage, or the effects of temperature change.

Participation stresses are similar to deformation stresses, but they are due to a quite different cause. They include what are known as "secondary stresses" in bridge trusses and "participation stresses" in bracing systems as special cases. The designation is used herein for want of a better one, but the term is used in a wider sense than usual.

The primary action of most structures is such that the stress in any one part is independent, or nearly independent, of that of the other parts. This is termed normal, structural action. The group indicated includes all structures statically determined and, for good reasons, it includes also most of the forms of indeterminate structure that experience has shown to be useful.

There is a type of structure in which one member cannot be designed separately but must be designed with due consideration for its effect on other members. Such action is referred to as "hybrid," because it has some of the characteristics both of normal structural action and of participation action. The group of structures seems to be quite large and to have characteristics of great importance to designers.

Although the designations assigned herein may be new, the concepts involved are not new. What the writer wishes to do is to classify and arrange certain views of structural design which have an honored tradition in American practice. The paper is not intended to be quantitative, except in so far as quantitative statements may help in defining qualitative action. The classification proposed has some value in reconciling discordant views held by practical designers and those held by theoretical analysts, and seems, further, to have value in reconciling conflicting views held by theoretical students in the field. It is also of value in anticipating the characteristics of proposed structural types.

I.—Types of Structural Action

Deformation Stresses.—The outstanding characteristic of deformation stresses is that the strains in the structure are fixed and that the stresses are deduced from the strains. The stresses themselves are not fixed at all, but depend entirely upon the stress-strain relation of the material.

Another important characteristic is that strength has nothing, or practically nothing, to do with the problem. The strains are fixed by the over-all dimensions of the structure and by the amount of deformation to be

accounted for. The designer finds it convenient to predetermine such strains, reduce them to equivalent stresses, and deduct these stresses from the working stresses available for load-carrying capacity.

Participation Stresses.—This term is used to include what are commonly known as secondary stresses in bridge trusses, participation stresses in bracing systems, participation stresses in cross-frames due to unequal deflection of trusses or girders, cross-flexure in the vertical members of trusses, secondary flexure in slender columns of bents, secondary flexure in slender spandrel columns of open spandrel arches, and secondary flexure in slender columns of buildings.

Participation stresses have many of the characteristics of deformation stresses: The strain is fixed; the stress is a consequence of the strain; and the strain, in general, can be affected in an important way only by changing over-all dimensions and not by changing the strength of the member. Thus, it is generally known that increasing the moment of inertia without changing the depth of truss members will affect the secondary stresses only as the primary stresses are affected.

Another important characteristic of these stresses is that they do not increase in proportion to the load up to rupture, but increase less rapidly after the yield point of the material has been passed. They are then clearly somewhat less dangerous than primary load-carrying stresses.

The interest of the designer in these secondary stresses is not to determine their value exactly but rather to be sure that this value is not too high. In order to find what value is too high, it is not sufficient to approach the problem from the analytical viewpoint. The values permissible in design vary with the material used, with the importance of the member involved, and with the type of failure that would result. At present, secondary stresses are accepted as a necessary evil. Try to keep them within a reasonable figure, and otherwise forget about them. Any effort to change this viewpoint represents a radical departure in thinking in the field of structural design. (In 1934, a valuable paper on the "Effect of Secondary Stresses Upon Ultimate Strength," was presented[3] by John I. Parcel, M. Am. Soc. C. E., and Eldred B. Murer, Jun. Am. Soc. C. E. Surely, the idea of discounting secondary stresses is not new in America; but accurate data as to the amount that may be discounted are much needed.)

An important difference between participation stress and deformation stress lies in their relation to the properties of the material. The two are affected in the same way by departures from Hooke's law for, in both cases, it is the strain that is fixed, the stress being a consequence of the strain. In so far as the ratio of stress to strain, however, changes with time (time flow of concrete) the effect is pronounced and direct in the case of deformation stresses, whereas time flow as distinguished from plastic flow has no effect at all in the case of participation stresses because, although the stress for a given participation strain is less as time goes on, the strain itself is in the same ratio greater because of flow under primary stress.

[3] *Proceedings*, Am. Soc. C. E., November, 1934, p. 1251.

If the participation strain is linear, as in the case of cross-bracing, the designer can control the stress only by changing the length of the member, which usually means that he cannot control it at all. If the strain is angular, the designer can control the maximum participation stress by changing the length of the member (which is usually impracticable), by changing the depth in the plane of flexure, and, to some extent, by changing the form or variation of section along the member.

Participation stresses are dangerous to the extent that they impair the primary load-carrying capacity of the member. Any generalization about them that does not consider the nature of failure from primary load is misleading.

Normal Structural Action.—The term is used herein to describe the action of those structures or structural parts in which it is possible to determine at the beginning the approximate magnitude of the forces in action, and in which the magnitude of these forces is affected comparatively little by the relative stress intensity in the parts of the structure.

The most obvious example is the ordinary statically determinate structure. In this type primary stress in one member is not affected at all by the stresses in any other member. All stresses are determined directly by statics, and the members are then proportioned for the forces which act upon them.

Most of the classical forms of indeterminate structures act normally, although their action is less definite than for structures statically determinate. A good example is the ordinary continuous truss, which, in practice, is often designed at once without any exact analysis being made after the design is complete. The experienced designer knows that for these structures such analyses will not indicate any important changes in his design.

Other examples are ribbed arches and spandrel-braced arches of steel, the so-called "rigid frame" bridges hinged at footings so far as the forces and moments at the knee are concerned. Most cases of continuous girders are included in this type.

In these structures it is possible to follow the procedure recommended in the textbooks. The engineer is told to guess at the sections, analyze, revise, and re-analyze. A bad first guess does not make much difference; the series of designs converges rapidly. In these cases, however, the designer can do much better than a bad first guess. Preliminary studies of pressure lines and of the properties of influence lines will enable him to make a good first guess—so good that the revision is trivial.

Hybrid Structural Action.—The term is used herein to mean structural action in which two or more parts participate in carrying loads to such an extent that if the strength of one part is changed the forces acting on other parts are largely affected. Clearly, there is some interaction in all indeterminate structures; the difference here indicated between normal and hybrid action is one of degree, and the two classes merge into each other. Participation stresses also are directly affected by changes in the primary stresses, but the relation is not reciprocal.

Hybrid structural action may be divided into two classes from the viewpoint of the designer's knowledge: Those in which the nature of the structural action can be foreseen; and those in which it cannot be foreseen. This says nothing except that the designer either does or does not know what he is doing; but the distinction needs to be made.

This type of action may be further divided into two classes, depending on whether the structure can or cannot be designed efficiently as laid out. It is very important to know this at once, but the knowledge depends on the designer's understanding of the problem.

A very simple example of what herein is called hybrid structural action occurs where two parallel beams are connected so that their center deflections must be the same under a single load at the center. If they are of the same span, depth, and material, they share the load in proportion to their strengths, and load can be assigned to one or the other at will. Even if the spans are different, this is still true if the depth-length ratio is the same for the two beams.

In this case the designer can foresee the action. He then designs as he chooses and knows that the stresses will be as assumed without further analysis; the analysis precedes any designing. Moreover, the structure can be designed efficiently since the stresses in the two beams will be the same. If the depth-length ratio is different for the two beams, however, they cannot be designed efficiently. No matter what the designer does, the relative stresses will be proportional to this ratio; but he can still foresee this fact. This case is simple; frequently the action of the structure is not easily predetermined and, in many cases, the efficiency of the design must be considered for several conditions of loading.

Near the center of long trusses having two diagonals in each panel, the designer may predetermine the stress intensities in these diagonals pretty accurately. In any panel the stress intensities in the diagonals are in inverse ratio to the squares of the lengths of these diagonals if there is no stress in the posts and if the stress intensities in the chords are equal and opposite. For dead load, these conditions in chords and posts are nearly fulfilled; for isolated loads, they are only approximately correct.

Knowing this and considering for the present only dead load, the designer can assign areas to the diagonals at will. If the chords are parallel, the diagonals can be designed efficiently; if the chords are not parallel, one diagonal must be inefficient. Influence lines may be helpful, but they fail to furnish at once a very illuminating picture of the essential structural action. For single concentrated loads the effect of the stresses in the posts and of unequal stresses in the chords may be pronounced. If single moving concentrated loads dominate the design, exact proportioning seems hopeless.

In these cases—parallel beams and double diagonals—the textbook recommendation to guess at a section, analyze, and re-design, to convergence will not work very well. If the layout is efficient, analysis will show the first guess to be right—and will show any guess to be right—but if it is inefficient, repetition of analysis and design will eliminate the inefficient member—after many repetitions—and result in a simpler structure.

Similarly, the king post truss cannot be designed for given working stresses in beam and sag rods except for a small range of the ratio of beam depth to sag depth, which ratio can be predetermined. The chief interest in the king post truss is that, geometrically, it is so obviously the familiar problem of a truss subjected to secondary stresses. Looking at the truss from this viewpoint, the engineer would first design it; then he would reduce the depth of the beam until the secondary stress in it was within safe limits, or he could, by adjusting the sag rods, produce initial stresses to offset the secondaries. If, however, he attempts to utilize these secondaries, to put them to work in carrying the load, an entirely different structural problem is presented, and new methods of study are required.

Hybrid structural action also occurs in the queen post truss, in systems of intersecting beams, in Vierendeel girders, probably in most slabs (at least where variation of depth is involved), in some problems of continuous beams, and in many problems of continuous frames. It is discussed subsequently, in the case of bents, of rectangular wind frames, and of arches integral with their spandrel structures.

Study of a Two-Legged Bent Illustrating the Types of Structural Action.—The distinctions herein presented are well illustrated by studies of a two-legged bent carrying vertical loads. It is assumed that the members are rectangular and homogeneous. It is proposed to study the flexural stresses in the columns. The girder section is assumed to be the same throughout the discussion but first the depth, and then the width, of the column is varied.

If the column is very narrow the girder acts practically as a beam simply supported. The rotation of the top of the column must be the same as that of the end of the girder. The angular strain in the column, therefore, is fixed, and the flexural stress varies almost directly with the depth. If the column is extremely rigid it takes nearly the full fixed-end moment in the girder and the flexural stress varies nearly inversely as the square of the column depth. Between these conditions is one in which the flexural stress in the column is nearly independent of the depth; the column is picking up moment as fast as it can take it.

If the width of the column is increased, it will be found that in the first stage (column slender) the flexural stress in the column is scarcely affected at all; in the last stage (column stiff) this stress varies inversely as the width; and in the transition stage, the increase in width reduces the flexural stress somewhat but not at all in proportion to the increase in strength.

The first stage represents a structure essentially statically determined (post and lintel or column and beam), but with participation stresses in the columns. When the column becomes very stiff the structure—so far as the columns are concerned—is normal in its action, the stresses in the columns being determined for a fairly definite moment. In the transition stage the structural action is hybrid and does not respond readily to ordinary design procedure; increase in depth may either increase or decrease the flexural stress or leave it unchanged; and increase in width (which amounts to increase in moment of inertia) produces comparatively little effect.

Deformation stresses would be produced in this structure by change of temperature. As the depth of column is increased, the deformation stresses at the top of the column would vary at first almost directly as this depth, but later would be relieved by flexure of the girder.

Perhaps the most important fact revealed in this case is that if the flexural stresses indicated in the hybrid stage are dangerously high, there does not seem to be very much that can be done effectively, as long as the girder section is constant. This column may be thrown from the hybrid stage into the normal stage of action, however, by reducing the stiffness of the girder. In the rigid-frame bridge this is done by reducing the center depth of the girder.

II.—General Remarks on Hybrid Structural Action

It is difficult to identify hybrid action in an unfamiliar structural type, but after one has come to recognize the type he begins early to suspect its existence in certain cases. Probably the chief identifying characteristic of the type is that it responds sluggishly or erratically to traditional methods of structural design. Successive cycles of design and analysis may indicate a trend, but produce only slowly a definite and satisfactory conclusion. If there are discontinuities in this design procedure the traditional process may be quite misleading.

Traditional processes are not very helpful in this field, although they still have their place. In these cases there are usually many variables and the curves of variation present maxima and minima. It should not be necessary to point out to scientific men the extreme difficulty—the grave danger—of applying purely empirical methods to such problems. It is impossible, in such a case, to generalize or extrapolate beyond the range of data presented and it is almost impossible to classify the data for study no matter how numerous these data are, unless such arrangement is based on an adequate theory.

It makes a good deal of difference what the designer wants to do. Where the action is normal for given proportions, there is usually only one answer and that is easily approximated at once and easily determined accurately by cut and try. Where the action is hybrid there are many possible structures; the designer must make his choice. In a sense he tells the structure what he wants it to do, and the structure will try to do it. If, however, it is something that the structure cannot do at all, the designer has erred; if it is something that the structure cannot do efficiently, the design is penalized. To apply the more erudite terminology of mechanics to this conception, the fiber stresses desired may involve incompatible strains.

This type of structural action is often best approached from a direct study of the fiber stresses. H. V. Spurr, M. Am. Soc. C. E., has done this in his wind-frame studies,[8] and, without discussing herein whether his method of design is necessary, sufficient, or invariably satisfactory, the writer feels that his contribution to the direct method of attack is of great value.

Rigid-Frame Bent Subject to Vertical Loads.—Consider a bent of the so-called rigid-frame type sometimes used for building frames consisting of a roof girder, curved or polygonal, carried by two columns. The pressure

[8] "Wind Bracing", by H. V. Spurr, McGraw-Hill Book Co., 1930.

line for dead load in this structure may lie anywhere between that for a simple curved beam supported on columns and that for a three-hinged arch (hinges at the center of the girder and at the bases of the columns).

The action of the structure may be studied by either of two methods. Choose some pressure line which is expected to give a good distribution of material, determine the moments for this pressure line, and then vary the moment of inertia along the section so as to satisfy the conditions of continuity. The depths of the sections may then be varied by choice. By this method the designer tells the moments where he wants them and then decides whether he likes the result.

As an alternative procedure which has some advantage, he chooses the desired pressure line, selects working stresses along this line, and then varies the depths to provide the requisite conditions of continuity. This requires judgment, but may be done quite well by eye. The moments of inertia may then be determined for these moments, stresses, and depths. The important fact is that the design may be predetermined—and predetermined over a wide range.

If at three points of the structure the moments of inertia are intentionally reduced compared with the sections elsewhere, the structure now becomes a normal structure (a three-hinged arch) with participation stresses at the weakened sections, which now act as hinges.

Rectangular Wind Frames.—The problem of analyzing rectangular frames for horizontal forces due to wind or to earthquake accelerations continues to occupy an important place in structural literature. It seems particularly illuminating to discuss the problem from the viewpoint proposed herein. It is assumed that the columns have been designed for vertical loads and that their sections will not be changed; discussion, then, is directed entirely to the design of the girders. The effect of offsets is not considered.

It is not intended to discuss the participation stresses induced in the girders of upper floors by departures from planarity at floor levels. The writer's studies indicate that the problem is neither so important nor so difficult as some have indicated.

Assume that the girders have been designed by some of the conventional methods. An analysis is now made and it is found that some girders are overstressed and some under-stressed. Tradition indicates re-design by increasing the size of the overstressed girder and decreasing that of the girder under-stressed. However, analysis will show in many cases practically the same stresses as before; in other cases, it may show a slight improvement in the stress distribution which will be further improved by repeated re-design to a certain point; but numerous cycles of re-design may be necessary to produce much improvement.

The girders of a symmetrical rectangular frame, carrying horizontal loads, may be designed on the following assumptions: (*a*) Points of inflection are at the mid-points of all members; (*b*) column shears are proportional to moments of inertia of columns; and (*c*) design stresses in girders are proportional to depth-length ratios of girders. If so designed, it will act as designed except near the fixed base. The analysis precedes and dictates the design; after design, no analysis is needed.

If points of inflection in the columns and distributions of column shears are assumed throughout the building, the designer can, by working from irrotational footings, determine relative stiffness, or K-values for the girders all the way up the building. Negative values for K would indicate impossible assumptions and positive values of K might be unsatisfactory because, for a given working stress, too deep a girder would be indicated or, for a given depth, too great a fiber stress would result.

The object is not to recommend this method of design, although it has value, but rather to indicate that as an alternative to the procedure, now popular, of assuming a structure and, by analysis, determining the stresses in it, presumably with a view to re-design, the designer may predetermine the function that the bracing is to perform, design directly, and then see whether the design is satisfactory as regards girder depths or working stresses. Of the two procedures, the second is often the more satisfactory. Each method is useful for certain purposes.

Note that the sensitiveness of the procedure depends upon the relative stiffness of columns and girders. If the columns are very stiff relative to the girders the moments can be applied almost anywhere without disturbing the desirable condition of inflection points very near the mid-points of the girders; the moments in the girders will be proportional to their K-values.

It seems futile to ask which of several conventional methods of analysis most nearly conforms to the results of so-called exact analysis; the answer depends on the proportions of the structure. Special attention is directed, therefore, to the dangers of induction from specific data whether obtained by computation, from models, or from tests in a laboratory, unless the range of the data covers all variables.

Arches Integral with Their Spandrel Structures.—An important example of hybrid structural action occurs in arches that are integral with their spandrel construction. It has long been recognized that there is interaction of the rib and the spandrels, but such interaction is commonly neglected in design.

In special cases applying to such structures, the engineer can see certain controlling relations. If the columns of an arch having open spandrels are quite flexible and very closely spaced, angular changes along the arch rib must be the same as those along the deck girder, since the vertical deflections of the two are every where the same. In this case, then, the entire structure could be designed at once as an arch made up of two members placed side by side as in a flitch beam. The relative flexural stresses in the rib and deck can be predicted and may be controlled by varying their relative depths.

Clearly, in this case, the designer may put all flexural resistance in the rib, or in the deck girder; or he may divide this flexural resistance between the two members at will; arches have been built of all three types—a stiff arch without stiffening girder; a flexible arch with stiffening girder; and a stiff arch reinforced with a stiffening girder. Of course, in any case, there must be some flexural resistance in the flexible member and some flexural resistance in the columns. These, however, will have the well-defined characteristics of participation stresses.

If the columns are not closely spaced the simplicity of the picture is marred both because the flitch-beam picture is less simple and also because the deck girder now has primary stresses as a continuous beam.

As the columns become quite stiff, however, the picture becomes very complicated. Such a structure is clearly hybrid in its action and the nature of the inter-relations is not at once apparent. Methods of studying such cases have been developed by Nathan M. Newmark, Jun. Am. Soc. C. E.[4]

It is important to question whether it is good practice to change from normal structural action with participation stresses to hybrid action with obvious reduction in the factor of safety of the rib, with little promise of economy, and with much complication and uncertainty in design. There is little, if any, evidence that the participation stresses in these structures are dangerous or even objectionable. The idea of putting secondary stresses to work is not usually very promising.

III.—General Remarks on Indeterminacy

Thirty-four years ago Mr. Frank H. Cilley presented a paper under the title "The Exact Design of Statically Indeterminate Frameworks. An Exposition of Its Possibility but Futility."[5] The main thesis was that "statical indetermination in a structure is always to be regarded as self-interference with efficiency." The paper followed a previous paper by the same author,[6] and revived a discussion of long standing as to the relative advantages of determinate and indeterminate systems. Two discussions of the paper were presented by distinguished American engineers and several by foreign engineers.

There is little question that Mr. Cilley's views represented those of many, and probably of a large majority, of the leading American structural engineers of his day. To-day, on the other hand, literature contains numerous articles extolling the virtues of indeterminacy and some writers even go so far as to attribute to indeterminate structures virtues which appear to be contradictory. They are referred to as "reservoirs of resilience," their rigidity is praised, as are their economy and their strength.

Since 1900, many indeterminate steel trusses have been built in America with claims, apparently well supported, of considerable economy. More important still, a new material—concrete—which is more conveniently made continuous, has come into general use.

The rather awkward methods of analysis current at the beginning of this century undoubtedly delayed the development of continuous structural types. At that time the analysis of some of the more complicated types now proposed

[4] Some phases of Mr. Newmark's extensive studies are reported in "Interaction Between Rib and Superstructure in Concrete Arch Bridges", by Nathan M. Newmark, Jun. Am. Soc. C. E. Thesis presented to the University of Illinois in 1934, in partial fulfillment of the requirement for the degree of the Doctor of Philosophy.

[5] *Transactions*, Am. Soc. C. E., Vol. XLIII (1900), p. 353.

[6] "Some Fundamental Propositions Relating to the Design of Frameworks", by Frank H. Cilley, *Technology Quarterly*, June, 1897.

was impracticable. The profession has made progress in this field. It may be well now to divert the attention of structural designers from the endless elaboration of analytical technique to the more important matter of interpretation of analyses.

It appears that Mr. Cilley's paper was directed too much to a consideration of what is herein termed hybrid structural action. The degree of self-interference in normal structures is quite negligible. Each normal indeterminate structure usually has a determinate analogue in comparison with which it has certain virtues and which has certain virtues in comparison with it. No generalization is possible, but the indeterminate structure has won consideration and is often indicated.

Structures in which hybrid action predominates are also sometimes indicated. If their action can be clearly foreseen and if they are designed for one controlling load condition, they may be designed economically. Often they are indicated for reasons entirely apart from structural efficiency, as in the case of rectangular wind-bracing; but where their action cannot be clearly visualized, conventional procedures of analysis by computation or by model may furnish little help.

IV.—Concluding Remarks

The paper has indicated four types of structural action the characteristics of which make their separate discussion worth while. These are (*a*) the action that produces deformation stresses; (*b*) that which produces participation stresses; (*c*) normal structural action; and (*d*) hybrid structural action. Deformation stresses and participation stresses have many characteristics in common, but are essentially different in cause and sometimes in action. Hybrid structural action represents a transition stage between participation stress and normal structural action.

Deformation stresses cannot be avoided except by avoiding the deformation that produces them. They may be slightly modified so that a little more strain takes place at one point and a little less at another, but about the same total strain—angular or linear— will inevitably occur. Linear strain due to angular strain may be reduced by decreasing the depth.

The designer's interest in participation stresses is that they shall not be too high; he does not want their exact values. If they are too high, he changes either the over-all dimensions or the details of construction. What is "too high" will always remain a matter of judgment. In normal structural types only is the traditional procedure, of first computing the forces and then designing for them, applicable.

Hybrid structures may be designed in many ways. In order that analysis may guide to design, it should precede design so that the designer may see in what ways the structure can act. Then, in a quite literal sense, he tells it how to act and makes it act in that way. The difficulty is that he may blunder in trying to have it act in a way in which it cannot possibly act or that, of the many ways in which it can act, he chooses an inefficient one.

Structures characterized by hybrid action are difficult to design and are often inefficient in any case. Study of them is made difficult by the inadequacy of traditional methods; it is almost hopeless, and may be dangerous, to study them by emprical methods.

Generalizations as to relative advantages of determinate and indeterminate systems are difficult in any case. Some conflicting opinions in the literature may be reconciled by recognizing the distinction indicated.

5

LIMITATIONS AND APPLICATION OF STRUCTURAL ANALYSIS

I — AMID the many new developments in stress analysis, especially in the field of indeterminate structures, it is desirable to distinguish between analysis for research and analysis for design — Stress analysis for design should utilize familiar concepts, and the method should be flexible

Safe engineering means first of all correct calculation of stresses. This is true of a rigid frame or other indeterminate structure just as much as of a simple beam. Does it mean that the new systems of analysis brought out in recent years for the computation of stresses in intermediate structures are essential? Must the structural engineer use them? Do they assure safe building?

Professor Cross subjects these questions to searching scrutiny by dissecting the process of stress analysis into its elements. His conclusions are all in favor of simple, transparent methods as against complex and confusing procedure.

In the present first half of his study he proceeds on the view that a good method of analysis should use familiar concepts, should have a clear physical meaning at every stage and should be flexible. He holds that the quickest method is usually inferior to a slower but clearer one. He then tells what stress analysis really does and compares it with the process by which a picture puzzle is worked out. Incidentally he shows that the geometrical and the energy or least-work methods are identical.

On this foundation he will later show, in the second half of the article, that all analyses must be interpreted. The physical meaning of a stress and the extent to which chance factors may change the amount of the stress, have much to do with the safety of what the engineer builds. — EDITOR.

Evidence of increasing interest in the theory of structural analysis, if needed, is given by the number of articles appearing in this field. A review of the *Transactions* of the American Society of Civil Engineers shows that about five times as many such articles were published in the third decade of the century as in the first decade. Articles are appearing today which could not have found publication twenty years ago.