REPORT OF PROJECT COMMITTEE ON THE USE OF HIGH ELASTIC LIMIT STEEL AS REINFORCEMENT FOR CONCRETE

By H J. Gilkey, *Chairman*, and G C Ernst

Iowa State College

SYNOPSIS

By means of the analysis of the problem presented in this preliminary report and the accompanying review of the existing literature pertaining to the subject, the Committee has been able to study the known facts concerning the use of high elastic limit steel as concrete reinforcement and to point out the factors upon which further research is needed

The lower ductility of high elastic limit steel bars does not constitute a valid objection to their use Bars rolled from steel railway rails exhibit satisfactory uniformity As longitudinal reinforcement in columns this grade of steel supplies additional strength Fatigue and impact loading and volumetric change in concrete do not present any special problem

Among the factors needing research are welding, bond strength, deflection of flexural members, effects of tensile cracks, identification of grades and severe impact

At the same working stresses high elastic limit steel can be safely and satisfactorily used interchangeably with softer material but if economy is to be achieved advantage must be taken of the higher possible working stresses for both steel and concrete Research is needed to clear up questions bearing upon this point The report analizes the possibilities of saving through the use of higher working stresses and alterations in design procedure and applies them to the redesign of a deck girder highway bridge A considerable saving in material is shown to be possible

FOREWORD

This investigation represents two phases of a general project which is designed to define the proper status of high elastic limit steel in the field of reinforced concrete construction In a sense, the first phase may be considered to be an extension or a generalization of the earlier investigation (Proc Highway Research Board, Vol 11, Part II, 1931) which was devoted exclusively to rail steel

The second phase is a reconnaissance into the economics of high elastic limit steel and constitutes one of the several small projects mentioned in Chapter I

The conclusions from each separate investigation need to be considered jointly with the evidence available on other aspects of the

general problem Chapter I supported by the references of Chapter II supplies the introduction to the problem in its entirety

The present project has been a cooperative undertaking between the Highway Research Board, R. W Crum, Director, and the Engineering Experiment Station of Iowa State College, Dean T. R. Agg, Director The detailed work has been performed by George C Ernst, Special Research Assistant, Engineering Experiment Station, under the immediate direction of H J Gilkey The results have been submitted to a committee appointed by the Executive Committee of the Highway Research Board The committee consists of:

H J Gilkey, Professor and Head of Theoretical and Applied Mechanics, Iowa State College Chairman.

A L Gemeny, Senior Highway Engineer, U. S. Bureau of Public Roads

F E Richart, Research Professor of Engineering Materials, University of Illinois

Searcy B Slack, Consulting Engineer, Decatur, Georgia

COMMITTEE FINDINGS

The committee is in substantial agreement with the findings as set forth in this report and feels that the following recommendations are justified

1. When handled, fabricated and placed with reasonable care, high elastic limit steel is entirely suitable for reinforcement for concrete within the range of current working stresses

2 Used with increased working stresses, high elastic limit steel offers considerable economy but in considering the material for such use account needs to be taken of several factors

(a) Provision must be made for the added anchorage requirement per unit cross-sectional area of bar.

(b) Design provision must be made for any additional hazard against diagonal tension..

(c) For severe exposure (as regards corrosion of the steel) caution needs to be exercised until more is known of the extent to which the widths of tensile cracks in the concrete may be widened with possible increased corrosive attack

3. Whether used with or without increased working stresses, the committee strongly recommends that every different grade of reinforcement be required to carry a distinctive label to be placed as one step in the manufacturing process For materials that require differences in the techniques of transporting, handling, fabricating and placement, it is important that there be no possibility of interchanges that cannot be recognized readily.

4. It is desirable that the separate small investigations outlined be carried out and that the results be published as they become available

INTRODUCTION

The elastic limit of steel may be raised by increasing its carbon content or by cold working The usual forms of high elastic steel are as follows

- Increased carbon content (hot rolled)
  - Billet steel (intermediate grade, high carbon).
  - Rail steel (heads, webs, flanges)
- Cold worked
  - Cold drawn wire
  - Cold twisted square bars
  - Cold rolled material
  - Isteg (two mild steel bars twisted together at pitch of 10 or 15 diameters)

Some of the questions bearing upon the use of high elastic limit steels in reinforced concrete apply equally to all of the kinds listed above, but each type also presents its own special problems

Billet steel and rail steel are used for all sizes of reinforcing bars The cold drawn wire is used mainly in welded mesh for slab reinforcement or as spirals for column reinforcement Cold twisted square bars were used widely a number of years ago but their use nearly ceased when it was demonstrated that twisting did not materially increase the bond resistance While it was known generally that the cold twisting of hot rolled steel of structural grade greatly raised its yield point, it is probable that attention was focused regretfully upon the accompanying decrease in ductility, rather than upon the increased yield point as a desirable by-product of the twisting Up to the present time, cold rolled material has not assumed importance as reinforcement for concrete

The "Isteg" bar is a strand of two mild steel bars twisted together with a pitch of 10 to 15 diameters of one bar The twisting raises the yield point from about 70 per cent to over 90 per cent of the ultimate strength of the steel This has not been used in the United States, having been developed in Germany since about 1929 In Prussia working stresses of 25,500 lb per sq in are permitted against 17,000 lb per sq in for the untwisted material Greater bond resistance is also claimed for the Isteg bars which, if true, is important especially when higher working stresses are used in the steel without increase in the strength of the concrete

For many of the uses of steel the high ductility of the softer grades is a desirable property and its presence contributes to the safety of the structure The material is given that toughness which enables it to absorb impact or shock loads without fracture It also permits a member to be forced into position, if necessary, or to undergo a re-distribution of stress which may relieve local over-strain without serious consequence

This added factor of safety is so real in many cases that it is not sur-

prising, perhaps, that engineers have been slow to recognize that ductility in concrete reinforcement is useful only in so far as it enables steel to be bent and handled without fracturing it The brittleness of the concrete itself is such that the yield point of the steel always marks the limit of resistance of the member. Since the yield point, or elastic limit, varies inversely with the ductility, it is evident that, viewed from this angle, the use of less ductile steels will, with no reduction in safety, produce stronger and more economical members Because of a growing realization of this, there is now a pronounced trend toward increased use of higher elastic limit steels Only recently intermediate grade steel has quite generally replaced the structural grade and the hard grades are coming more into favor than heretofore. Up to the present time, however, the increase in working stresses allowed by most specifications has been small and only a portion of the potential economy of high elastic limit steel has been realized In other words, whatever advantages there might be have gone largely to increased factor of safety rather than to decreased construction cost [1]

It should not be inferred from the foregoing comments that the use of stronger steels is a recent innovation Rail steel and other high elastic limit steels have had their advocates and have been on the market for many years There have been and still are, however, certain questions concerning their use which must be answered The recent trend toward a more discriminating appraisal of some of the factors involved, gives rise to a need for an immediate re-study of the evidence available and a search for such new data as may be required to clear up points which are still in doubt

This report is intended to be a step in that direction. Chapter I is a statement and a review of the leading questions which have been raised An effort is made to indicate the extent to which a particular question may already have been answered or may be apparent rather than real An effort is also made to indicate the nature of the researches which are needed

Chapter 2 summarizes some of the literature which has formed the general background for many of the statements made in Chapter I Without claiming for it the inclusiveness of a bibliography, it is intended to be, nevertheless, a chapter of summarized sources

Chapter 3 is a theoretical exploration of the possible economies that might be realized through the use of high elastic limit steel, taking into account the necessary changes in design procedure

Chapter 1 Questions and Their Status

Among the questions relating to the use of high elastic limit steel as concrete reinforcement are the following.

[1] A number of specifications have, however, allowed substantially increased working stresses in welded mesh made from cold drawn wire

1 Does lack of ductility constitute a valid objection to its use?

2 Is there a possibility that sound bars cast as reinforcement in a member might be ruptured under sudden or impact loading of the member?

3 Is there the danger that insipient undiscovered cracks may be developed in bending or in handling and that bars thus weakened might be cast in a member and subsequently fail?

4 Does the possible presence of transverse fissures in rails from which reinforcement is rolled constitute a hazard for this class of high elastic limit material?

5 Rails are of various (52)[2] origins and are collected from all parts of the country Are the diversities of composition and physical properties sufficient to produce an objectionable lack of uniformity in rail steel reinforcement?

6 It is possible that there may be increased use of pre-fabricated reinforcing units and that welding might be used in preparing such assemblies Do the different types of high elastic limit steels weld satisfactorily?

7 The drawing of wire produces a uniformity and smoothness of surface longitudinally that might be expected to decrease its bond resistance Might this have a significant bearing upon its usefulness in comparison with other reinforcing steels?

The foregoing questions apply to high elastic limit steels when used interchangeably with steels of softer grade and without an increase in working stresses If increased working stresses are contemplated, each of the foregoing questions becomes more important and in addition there are others to be considered

8 Most important of these is the question of adequate bond The possibility of insufficient anchorage for reinforcing bars probably constitutes the greatest present hazard in reinforced concrete design An increased working stress in the steel imposes a bigger bond stress to be developed by a bar of a given size Can the need for more effective anchorage be met sufficiently to safeguard the use of higher stresses in the steel?

9 Heat treatment will relieve the effects of over-strain in steel Is there a possibility that over-strained material such as cold drawn wire or cold twisted bars might be annealed gradually under service temperatures, thus bringing the yield point and elastic limit back to the value it possessed before the material was over-strained? The possible annealing of wire from the heat of welding is discussed under 6

10 With increased stresses in the steel there is no change in its modulus of elasticity, and the deformation of the bars would be increased in proportion to the increase in stress In flexural members this means

[2] Numbers in parentheses refer to list of references at end of Chapter 2

that deflections would be proportionately greater Might not the extra deflection be objectionable in the case of slender (long span) beams?

11 The cracks in the concrete on the tensile side of members would be proportionately wider because of the increased steel stresses Is there not a possibility that this increased width of tensile crack might permit corroding agencies to attack the steel?

12 Does sustained static loading present any problem that differs from that for the more ductile steels?

13 Does fatigue or repeated loading present a problem?

14 How important is the matter of possible confusion between similar appearing bars of different grades or kinds and how big a problem would it be to avoid such confusion?

15 What would be the effect of higher working stresses upon present design procedure and upon the mechanics of reinforced concrete, including economical stress combinations, and structural layouts?

16 How would the problems connected with this material as compressive reinforcement differ from those which it presents as tensile reinforcement?

17 It is evident that higher stresses in the steel should be accompanied by the use of stronger concretes As the cement content of concrete increases, the amount of volume change from chemical heat and moisture change also increase Is this a serious consideration in reinforced concrete construction?

DISCUSSION OF THE QUESTIONS

1 *Lack of ductility* (See also questions 2 and 3)

Roughly the ultimate compressive deformation of concrete of almost any quality is 0 002 while its ultimate tensile deformation is less than one-tenth as much [3] Under sustained loading in air it is possible for concrete to deform three or four times these amounts but even such deformations are much less than that taken by any usable grade of steel within its yield point range It is evident, therefore, that the beginning of the yield point range of stress (the elastic limit) gives the maximum usable deformation of a reinforced concrete member Beyond that strain the concrete cracks badly and the two materials cease to function as a unit The question of ductility reduces then to whether a high elastic limit steel is sufficiently ductile and tough to be handled, bent and placed without damaging it Apparently this was never an important problem with regard to the cold twisted material, nor has cold drawn wire of acceptable grade been subject to serious breakage under use For the steels in which the elastic limit was increased by raising the carbon content, more attention has been given to this point for the rail steels than for the billet steels of similar grade It seems proper to

[3] See Proc Am Soc C E , Jan 1935 p 133

assume that in this respect results from tests on bars rolled from rails are reasonably representative of high carbon steel in general

The most exhaustive as well as one of the most recent and authoritative investigations of rail steel is recorded in Part II Vol 11 Proceedings of the Highway Research Board, 1931 (45) This extensive investigation includes reviews of other important investigations of rail steel with a view to settling the two very important questions of uniformity, or dependability, and ductility These findings (45, pp 77, and 87) indicate definitely that breakage or even weakening in bending bars by approved current methods is negligible The very few bend failures recorded in investigations are failures to pass the standard tests as laid down by the A S T M rather than failures of bars being fabricated for service Of course, the bend test is properly much more severe as regards the sharpness of the bend permitted than is any defensible practice Studies of bent bars which were made in connection with the investigation alluded to above (45) resulted in the development by Professor C H Scholer (50, p 231) of an improved device for performing the bend test The indications are that the principles involved will soon be adopted as standard It is apparent that no sound bar of the grade of rail steel is likely to fail to pass any existing or probable specification for the bend test Moreover, the same principles can probably be incorporated in the machines commercially used for fabricating bars which would still further reduce the punishment which a bar undergoes in the bending process

The job mortality which has focused attention on ductility has existed mainly because of carelessness in handling and of wholly improper methods of bending steel of hard grade It must be recognized by anyone proposing to use any of the harder grades of steel that it is less ductile and that the technique of handling must be somewhat more refined than that which may be used for softer grades The limitations imposed are not difficult to meet and involve mainly a recognition that there is a difference For the most part the rail steel companies have themselves solved the bending problem by guaranteeing their product and by shop bending prior to delivery Proper handling after the steel reaches the job is essential, however, because of the time element and other inconveniences incidental to replacing broken bars

From the foregoing, it seems reasonable to conclude that the question of ductility has been answered fully by previous tests and that, with reasonable care in bending and handling, the lack of ductility constitutes no valid objection to the use of high elastic limit steel as concrete reinforcement

2 *The possibility of sound bars being ruptured within a member under impact loads on the member*

It requires little of thoughtful consideration to recognize that there can be no serious uncertainty on this point. Nevertheless, the question

has been raised repeatedly and will continue to be a bar to the use of the harder grades of steel as long as there are materials engineers who believe that this might occur. Among the more recent mentions of the point are those in the report of the Highway Research Board (45, pp 69 and 76) That there should apparently be no evidence on record to indicate the possibility of sound bars fracturing when embedded in concrete subjected to impact seems reasonable when one visualizes the condition which exists The steel is packed in a matrix of concrete through which all stress has to be applied. The concrete is relatively friable and is more brittle than the steel It seems inevitable that the concrete would have to be thoroughly shattered before steel of any grade could be damaged from impact applied through concrete Drop impact tests of reinforced concrete were made in connection with the shipping board researches (15, p 43) and it is understood that some specimens were reinforced with high carbon steel bars. Apparently all failures were of the concrete and neither the kind nor the amounts of reinforcement seemed to be a determining factor

It may be stated then that there appears to be little probability of sound bars being ruptured within a reinforced concrete member because of shock or impact and that there is no apparent need for tests as an added check on this point Some simple form of drop test could be arranged easily for demonstration purposes, however.

3 *Possibility of members which are weakened by undetected insipient cracks being cast in the concrete and subsequently failing*

Reported experience with the high elastic limit steels indicates that insipient failures are rare at best When a hard bar cracks the break may be expected to extend entirely across the section. The behavior is thus under the bend test, when failure does occur If it be again assumed that rail steel is representative of high elastic limit steels generally in this regard, it appears, from the studies of Pearce (45 pp 14, 68, 69, 70) that a failure of a steel bar within a member, from an insipient crack, is an unheard of phenomenon Even if such a failure did occur, however, it would probably for several reasons be localized in its effects·

(a) In any important reinforced member several bars are stressed jointly and if one of them were to fail the sound bars remaining could in general be expected to carry the load at an increased stress thus reducing the factor of safety but not always resulting in the failure of a member.

A coincidence as rare as that of insipient cracks occurring in several bars near the same cross-section can probably be disregarded Pertinent to this phase is a trend in some quarters toward the use of more and smaller bars because of their added effectiveness in bond Moreover, the smaller the bar the less is the structural disturbance caused in the

bar by bending it to a given radius To whatever extent this trend becomes the practice of the future, the possible hazard from a defect in a bar will be still further minimized

(b) A reinforcing bar is anchored to the surrounding concrete by bond and a fractured bar would affect the member for only a limited portion of its length It would not be analogous to the snapping of a cable or a tie.

It appears, therefore, that the possibility of an insipient crack being present or of its creating a hazard, if present, is so remote that it too can be disregarded Nevertheless limited research on the subject should suffice to remove the element of conjecture and would therefore be desirable

4 *Transverse fissures and other major defects in rails from which high elastic limit bars are to be rolled*

Several mutually opposing factors bear upon the frequency of transverse fissures and other major defects present in rails removed from service. Fissure frequency seems to be definitely on the decrease, due probably to improved manufacturing and inspection methods in spite of the fact that the heavier rail sections and wheel loads in current use probably tend to increase them Track surveys can now be made by means of the Sperry Detector Car (45, p 10), but the frequency of defects in rails removed from service should obviously be greater than that for track in place Offsetting this is the culling over and cutting out of defective parts of rails prior to rolling as well as the detection and elimination of further defects which show up during slitting and rolling Under reasonable inspection and reputable handling only small defects could remain in the parts to be rolled and many of those not strictly transverse would be so distributed along the bar as to be made harmless from the standpoint of objectionable localization In other cases the concentration across the section of a small bar would automatically produce rupture and the weak spot would be self-eliminated

Normal wear and obsolescence account for the removal of many rails from service Of those removed because of breakage or a recognized defect the location of the weak spot is known and the defective portion may be culled out prior to re-rolling It seems probable, therefore, that an undetected defect could occur only once in many lengths of rail In turn, each flawed rail will form several lengths of bar, all but one of which will usually be sound. From these considerations and those discussed under 3, it appears that the possible presence of major defects in the rails offers little of hazard to the quality of rail steel reinforcement except under a laxity of inspection that would hardly be tolerated by a reputable manufacturing organization or by a responsible purchaser There are apparently no data that would seem to indi-

cate that transverse fissures or other major flaws in rails constitute a hazard for bars rolled from them

The results from the recent exhaustive study of rail steel reinforcement (45) appear to justify the conclusion that, under current methods of inspection, manufacture and marketing, possible defects in the rails present neither a question nor a hazard

5 *Possible non-uniformity of rail steel because of the wide range of origins of the rails*

This is a very important question but it seems to have been answered fully The previously mentioned committee of the Highway Research Board (45, p 9) set as one of its two major objectives the answering of the question, "Are rail steel bars sufficiently uniform in their characteristics so that compliance with a specification may be expected within a reasonable margin?" The report which summarizes all recent tests of importance shows a uniformity that would hardly be believable were it not for the varied and unimpeachable character of the tests cited Moreover, Bessemer rail as a source of rail steel bars seems destined to disappear altogether A negligible percentage of the rails produced since 1922 are of Bessemer origin Since Bessemer steel in this country is admittedly less uniform than is open hearth steel and since the procesess of manufacture and the specifications under which purchases are made are constantly becoming more unified, it seems to be inevitable that the trend will be definitely toward greater uniformity of material rather than less In the early days it is probable that there were instances in which materials other than rails were used, which might well account for some of the doubts which have been expressed regarding the uniformity as well as the suitability of rail steel The report mentioned (45) appears to settle the question of uniformity for all rail steel which has been rolled under reputable auspices There are now but eight steel plants in the U S equipped for rolling the rails in general use (52, p 234)

6 *Welding qualities of high elastic limit steel*

Cold drawn wire is habitually used in welded mesh and there appears to be no reasonable question regarding the ability to produce satisfactory welds There is a question, however, regarding the extent to which the heat of welding may lower the elastic limit by relieving the overstress which raised it Such data as are available seem to indicate that tensile tests across welds do not show an abnormal amount of breakage at the weld This point is probably not serious, therefore, although it will stand further investigation In general, high carbon steels do not weld as satisfactorily as do the softer grades and further research on this question would seem to be desirable before the use of welded units of high elastic limit steels in forms other than mesh becomes gen-

eral The same also applies to welding as a means of splicing column bars (35) (41)

7 *Bond resistance of drawn wire as possibly influenced by the smoothing effect of drawing*

This is a question that does not appear to have been answered It is reasonable to expect a lowered bond resistance per unit of surface contact since the die tends to eliminate all roughness in the direction of drawing which is also the direction of bond stress When drawn wire is used as mesh, especially if welded, the joints supply a desirable mechanical anchorage tending to compensate for the smoothing effect to whatever extent it may be present Moreover, the fact that wire is always of small diameter gives it a high ratio of surface area to cross-sectional area which makes it relatively more effective per inch of length in developing bond resistance than are larger bars of similar surface texture From unpublished tests made by the Wire Reinforcement Institute (41) it appears, as would be expected, that the welded intersections of mesh reinforcement do add greatly to its resistance against being drawn through an aperature in a steel plate and it is reasonable to assume considerable added anchorage resistance in concrete It appears, however, that further bond tests are desirable, especially for the unwelded wire, and that such devices as artificial roughening by pickling, scoring or rusting, as well as special anchorage by looping, knotting, etc , might well be tried Whether it should be permissible to use the same bond stresses for unwelded mesh as for bar reinforcement appears to be a legitimate question which takes on added importance if increased tensile stresses are to be permitted For both woven and welded mesh further tests are needed to determine what special anchorage allowance should be accorded to intersections when they are embedded in concrete

8 *Bond for high elastic limit steels in general*

The importance of bond cannot be over estimated, especially if increased working stresses are contemplated for the steel without a corresponding increase in the strength of concrete to be used Bond resistance varies with the compressive strength of concrete and the permissible bond stress is normally specified in terms of compressive strength It is apparent then that if the strength of the concrete is stepped up in the same ratio as the stress in the steel no recognized hazard is introduced by an increase in steel stress For maximum economy a stronger concrete should be used with increased steel stresses, and it is apparent that bond constitutes no more of a problem for high elastic limit steel (at an increased working stress) than for any other grade of steel (at ordinary stress) providing appropriate strengths of concrete are used It cannot be denied, however, that one of the great-

est needs in the field of reinforced concrete generally (regardless of the grade of steel used) is a thorough-going search for improvement in the anchorage of bars While the results of such a research are not necessarily related to one class of steel to the exclusion of another, they would benefit concrete practice generally and might in addition allay fears or prejudices which may or may not be well founded For this reason such an investigation is recommended An adequate coverage of the field calls for a project of some magnitude and should include studies involving the size element, type of deforming lug, surface texture of bar, design of various forms of mechanical anchorage including bending, threading, and equivalent straight lengths of bars.

9 *Age-annealing of cold worked material*

Whether long exposure at ordinary temperatures might gradually relieve the overstrained condition which causes cold worked material to have a higher elastic limit is another problem which seems not to have been answered fully although the authors know of no evidence to indicate that a serious hazard exists. The answer to this problem should be known, however, before high working stresses are permitted in important members reinforced with overstrained material and which may be subjected to long-time loading. This question assumes added importance if temperatures may be somewhat above ordinary as in boiler rooms and some industrial structures

10 *Possible undesirable effects of increased deflections from increased tensile stresses*

The data available from which to predict deflections of reinforced concrete members are meagre. Besides the uncertainties of the immediate or elastic behavior are those added by the plastic flow in the concrete which, in the case of plain concrete members, may amount to several times the elastic deformation There is a definite need for experimentation with beams subjected to sustained flexural loading Such an investigation should include varied percentages of steel, and a rather wide range of stress in both steel and concrete The use of compressive reinforcement may be expected to decrease the long-time deflection by retarding the flow in the concrete under sustained stress This case should be included in the plan for an appropriate research project A yielding or creep in bond would be equal in importance to flow in the concrete

11 *Possibility of corrosion of the steel because of the wider tensile cracks in the concrete if higher steel stresses were used*

While various accelerated corrosion tests have been conducted (15, p. 25) there are apparently no conclusive data regarding what may constitute the maximum permissible width of crack for different types

of exposure of members For bridges under which steam locomotives pass and for members exposed to salt spray the problem might differ materially from that for interior structural members This constitutes a needed research

12 *Special problems characteristic of high elastic limit steel under sustained static loading*

For this type of loading there are probably no problems that differ essentially from those for softer reinforcement except that they are magnified proportionately as stresses are increased However, a slight overload with the milder steel may produce some set and result in permanent cracking of the concrete whereas the harder grades retain their elasticity through a greater range of stress and make the cracks more or less self closing when load is decreased As stated under question 10, not enough is known about the deflection of concrete reinforced with any steel and further experimentation is needed, not so much from the standpoint of probable safety as to enable the designer to forecast deflections more accurately than is now possible as well as to determine the value of the greater tendency of tensile cracks to be self closing upon the removal of load when high elastic limit steel is used

13 *Fatigue or repeated loading*

The endurance of steel is so far in excess of that of concrete in the number of permissible repetitions of stresses within the working range that repeated loading tests could be expected to net information of interest relative to the concrete only In no way does the question appear to be related especially to the use of high elastic limit steel and there appears to be no research that would produce results bearing primarily upon this as a problem of high elastic limit steel

14 *More than one grade of reinforcement which is not easily distinguishable from others*

This is an important question It seems reasonable that every bar of either high elastic limit billet steel or rail steel should be identified by a mark either rolled in or stamped If the same rolls were used interchangeably for different grades of material, perhaps all bars could make one pass through a small supplementary machine that would supply the identifying mark This is a manufacturing problem which demands solution Handling requirements differ for the different grades even though no differences in working stresses are contemplated

15 *Effect of higher working stresses upon present design procedure and economy*

Second in importance only to the question of safety is that of possible economy in the use of higher strength steels with increased working

stresses Undoubtedly the use of higher working stresses in the steel would call for higher strength concretes and for various modifications in details, layouts and design practice Studies are needed to determine what some of these trends might be

16 *High elastic limit steel as compressive reinforcement*

As compressive reinforcement high elastic limit steel appears to offer no special problems Shrinkage and plastic flow in the concrete produce compression in the steel in excess of the proportion it carried originally, which was presumably proportional to the ratio of the respective values of Young's moduli (not altered by the grade of the steel) The yield point of the harder steel is not reached as soon as for softer grades but when it is reached, (regardless of the grade) the steel deforms at approximately constant stress and the concrete must then reassume some of the compressive stress formerly unloaded on the steel when in its unyielding range of stress The yield point strength of the steel plus the ultimate strength of the concrete represents the approximate limit of resistance for a reinforced compressive member (49), (42) and it follows, therefore, that the use of high elastic limit steel as compressive reinforcement will produce a definite increase in the strength of the member High elastic limit steel has been used in column investigations sufficiently to demonstrate that the behavior is essentially as outlined More investigational work is needed to establish its relative effectiveness as compressive reinforcement in beams, (34) although the nature of the action is reasonably clear

17 *How important might be the effects of the increased volumetric changes that would occur if richer concrete mixtures were used?*

There is a trend toward the use of stronger concrete mixtures in building construction With increased stresses in the steel, the trend would doubtless be still more pronounced The volumetric changes (shrinkage from drying out, swelling from saturation and also from the chemical heat of hydration and subsequent shrinkage as heat is lost) become more pronounced as the cement content is increased This is, however, a problem which is applicable mainly to large structures such as dams and which happen, for the most part, to be entirely without the field of reinforced concrete Ordinary reinforced structures are sufficiently small to dissipate their heat of hydration readily and no appreciable temperature rise occurs Many of them are never alternately wet and dry Moreover, the structures are usually small enough or are divided into units which are small enough to permit ready adjustment to change of volume from any cause It may be stated, then, that the question of increased volumetric changes because of the use of stronger (richer) concrete mixtures can be classed as not related in any essential respect to the use of high elastic limit steel as reinforcement An ex-

ception is the use made of either bars or mesh in slabs for controlling shrinkage by concentrating it near the margins of members or at predetermined expansion joints Only low percentages of steel are normally used for this purpose and the amount has, up to the present time, been more or less arbitrarily assigned With increased richness of mixtures corresponding increases are needed in the amount of steel used or else in the permissible working stresses The increased elastic strength of high elastic limit steel could be counted upon to offset more or less automatically any increased tendency to develop shrinkage cracks in such members from ordinary increases in richness of mixture used for them

SUMMARY

The conclusions reached may be summarized as follows·

Under reasonable inspection and suitable handling, the lower ductility of high elastic limit steels constitutes no valid objection to their use as reinforcement for concrete The concrete is more friable or brittle than is steel of any grade

When culled and rolled under responsible manufacturing conditions, transverse fissures and other defects stand little chance of influencing adversely the quality of bars rolled from rails

As a material, the rail steel of this country is now quite uniform in composition and physical properties Bars rolled from rails appear to exhibit no objectionable lack of uniformity

For the most part, high elastic limit steels have been used interchangeably with softer grades and at the same working stresses For such uses they are safe and satisfactory if reasonable care is used in handling

Considerable economy could be achieved by using increased working stresses for high elastic steels and efforts should be made to clear up such questions as have not yet been fully answered Increased working stresses have been accorded to cold drawn welded wire mesh by some building ordinances and no adverse experiences have been recorded although additional information is needed on some questions which bear upon such use

As longitudinal column reinforcement high elastic limit steel supplies added strength With suitable arrangement for load transfer near the ends of the bars and at junctions with beams and girders, bond is not a vital consideration

Fatigue and impact loading of reinforced structural units and volumetric change in the concrete present no special problems in connection with the use of high elastic limit steel reinforcement

Questions which appear to need further experimental study are

Can welding be applied safely and economically to

(a) High carbon steel? (b) Cold worked steel, without annealing it?

Is it proper to allocate equal bond resistances, per unit of contact area to cold drawn wire and to hot rolled bars?

What added effectiveness in bond due to mechanical anchorage can be expected for wire mesh:

(a) When welded? (b) When woven?

To what extent, if any, might cold worked material undergo age-annealing under different possible environments?

If higher working stresses were adopted for high elastic limit steel, is there any possibility that objectionably large deflections might be attained by flexural members?

To what extent could the deflections of flexural members under sustained static loading be decreased by using compressive reinforcement to decrease or offset the plastic flow in the concrete?

Would the width of tensile cracks in the concrete under increased stresses in the steel permit corrosive damage to the reinforcement under any of the conditions of probable use?

How would the use of increased working stresses affect the relative proportions of members and the details of design in order to secure a maximum of economy and service?

High elastic limit steel requires reasonable care in handling Whether increased working stresses are authorized or not, each different grade of steel not identified fully by its appearance should be clearly identified by a stamped or rolled-in impression Can practicable methods be devised for accomplishing this as part of the manufacturing process?

While increased working stresses for high elastic limit steel of any kind should be accompanied by a corresponding increase in the strength of the concrete employed, which would supply automatically, therefore, the added bond resistance needed, nevertheless a thorough-going research on bond would be a valuable contribution to the general field of reinforced concrete Such a project could be justified as a high elastic limit steel investigation by the fact that it should supply answers to questions which, while no more applicable to one grade of steel than to another, under the condition stated, have nevertheless been linked to the use of this material in the minds of many potential users of high elastic limit steel Bond is directly pertinent when increase of steel stress is contemplated without change in quality of concrete

Moreover, for the sake of those who continue to be fearful that high elastic limit steel bars might be ruptured in such service as that to which a railway or a highway bridge is subjected, it is probably desirable that some form of severe impact demonstration test be conducted. Perhaps the dropping of a beam or of weights on a beam would be suitable

Experiment is also desirable to clear up the question of whether a high elastic bar is capable of harboring an insipient failure i e of cracking only partly across its section.

CHAPTER 2. REFERENCES

It is the intent to include sufficient comment with each of the following references to indicate the general nature of the investigation. The arrangement is approximately chronological and the numbers are for reference purposes

In drawing conclusions from any tests in the field of concrete, it is important that cognizance be taken of the auspices under which the tests were conducted and interpreted and also upon the state of the art at the time the work was done. This applies especially to reports of stresses in steel and concrete and to deflections of beams Accurate measures of elastic behavior were not current prior to the development of the strain gage about 1910 Nor were accurate diagnoses of the primary causes of failure always made, especially in connection with early tests Nevertheless, study of the more carefully conducted and reported of the early tests will often supply much valuable information additional to that which was recognized and understood when the tests were reported

1 Howe, Malverd A. "Cross Bending Tests on Beams of Reinforced Concrete" *Journal*, Western Society of Engineers Vol. 9, p. 239. 1904.

Tests at Rose Polytechnic on 18 concrete beams reinforced with steel of 60,000 lb per sq in elastic limit The sizes used were from 5 in to 21 in deep and all were 12 in wide The overall lengths were from 12 to 19½ ft

2 Talbot, A N. "Tests of Reinforced Concrete Beams" Univ. of Illinois Engr. Exp Sta *Bulletin No 1.* Sept. 1, 1904. Also A S T M *Proceedings* Vol 4, p. 476 1904

The tests were of beams with both high and low elastic limit steel Eleven beams 13½ in deep, 12 in wide, and 15 ft 4 in long, with steel of 58,000 lb per sq in elastic limit or greater were tested Stresses due to maximum load were from 53,000 to 65,000 lb per sq in for steel No figures were given for concrete stress Percentages of steel advocated were ½ to 1½ for steels of 33,000 lb per sq in elastic limit and ½ to 1 for steels of 55,000 lb per sq in elastic limit The statement was also made in the summary that "so far as strength of beams is concerned, steel having a high elastic limit is advantageous, it being assumed that there is sufficient provision against the slipping of rods and shearing failures"

3 Turneaure, F E. "Tests of Reinforced Concrete Beams" A S T M *Proceedings* Vol. 4, p 498. 1904.

Beams 6 in x 6 in x 60 in were tested at the University of Wisconsin during 1902–3 with bars having elastic limits from 45,000 to 75,000 lb per sq in Reinforcement, varied from 0 89 to 1 07 per cent Calculated stresses in the concrete were 2,100 to 3,000 and for steel were 46,000 to 78,000 lb per sq in Ultimate deflections were less than 0 3 in

4 Marburg, Edgar "Tests of Reinforced Concrete Beams" A S. T M *Proceedings* Vol. 4, p 508 1904.

The beams tested at the University of Pennsylvania in 1904 were 8 in x 8 in with 8 ft and 5 ft spans with steel percentages of 0 84–1 19 for steels with 58,000 lb per sq in elastic limit or greater Stresses given for loads producing first visible crack were 45,000 lb per sq in for steel with 1,700 lb per sq in for concrete (0 84 per cent steel), and 30,000 lb per sq in for steel with 1,500 for concrete (1 19 per cent steel) The average maximum deflection was given as 0 57 in

5 Harding, J J "Further Tests of Reinforced Concrete Beams Conducted by the C M & St P Ry" *Journal*, Western Society of Engineers, Vol 10, p 705 December, 1905 Also *Engineering News* Vol 55, p 168 Feb 15, 1906

Thirty beams (sets of 3 alike) tested in the fall of 1904 were 12 in wide with depths from 18¾ in to 23¼ in and percentages of steel from 0 68 to 0 75 Deflections were not reported

6 Luten, Daniel B "Hard and Soft Steel in Concrete" *Engineering News*, Vol 56, p 62 1906

A general discussion of advantages and disadvantages of both types of reinforcement

7 Talbot, A N "Tests of Reinforced Concrete T-Beams" Univ. of Illinois Engr Exp Sta *Bulletin No 12* Feb 1907

In the investigation reported there were four tee beams tested to failure (series of 1906) These beams were 10 ft span, flange thickness of 3¼ in , depth to steel of 10 in , and 8 in breadth of stem, and flange widths of 16, 24, and 2 at 32 inches with percentage of steel in the stem equal to 1 05, 0 93, 1 05 and 1 05, respectively The average elastic limit of the steel was 53,800 lb per sq in Stresses developed in the steel were 64,200–57,500–55,700 and 57,400 lb per sq in , respectively The concrete stresses were not given but the elongation at yielding of the steel (all beams broke by reaching of the yield point of the reinforcement) was about 0 0005 per unit of length Deflections for a stress of 35,000 lb per sq in in the steel were 0 24 in , 0 25 in , 0 31 , and 0 31 in , respectively

8 Withey, M O "Tests on Plain and Reinforced Concrete" Series of 1906–1907 Univ of Wisc *Bulletins 175 and 197* Engr Series Vol 4, No 2, pp 1–136 1906–1907

In the summer of 1906 reinforced concrete beams were tested having steel percentages of 0 6, 1 67, 1, 2 5 and 2 9 with various grades, sizes and types of reinforcement The maximum stresses for these percentages were respectively 62,000 –53,000–50,000-27,000 and 46,000 lb per sq in for the steel and 1500–1600–2000 –1800–2900 lb per sq in for the concrete

9 Talbot, A N "A Test of Three Large Reinforced Concrete Beams" Univ of Illinois Engr Exp Sta *Bulletin No 28.* Oct 1908

The results are reported on tests (April 1908) of three large slab beams reinforced longitudinally with bars in which the elastic limit varied from 51,260 lb per sq in to 52,770 lb per sq in The elastic limit of the web reinforcement

varied between 45,800 lb per sq in and 51,100 lb per sq in The beams were 25 ft long, 6 ft 3 in wide and 34 in deep and weighed about 35 tons each They were taken from 119 made for use in the bridge floors over subways for the Illinois Central Railroad The beams tested satisfactorily in every respect and indicated that this type of steel might be used for beams of exceptional size such as is common for bridge girders The steel stresses developed were from 52,000–54,000 lb per sq in , and the concrete stresses approximately 3000 lb per sq in

10 Talbot A N and Slater, W A "The Turner-Carter Building Tests" Univ of Illinois Engr Exp Sta *Bulletin No 64*, Jan 1913

Reinforcement in beams and girders of this building (tested in Sept 1911) had an elastic limit of about 50,000 lb per sq in The age of the floor at test was about 50 days Stresses calculated with a moment coefficient of 1/12 were 31,000 lb per sq in for the steel and 1,300 lb per sq in in concrete without sign of distress The observed stresses in this case were 11,000 and 1,100, respectively Deflections were rarely over 0 10 in

11 Hatt, W K "Report on Investigation of Reinforcing Bars Rolled from Steel Rails" A S T M *Proceedings* Vol 13, p 96 1913

Extensive tests were made on rail steel bars from Sept 1912 to Feb 1913 The majority of these bars were rolled from Bessemer rail Exceptionally favorable results were obtained from the cold bend tests which were made with a pin diameter as specified in the proposed Standard Specifications for Rail Steel Concrete Reinforcement Bars presented for adoption at the annual meeting of the A S T M on June 24, 28, 1913 The twisting head of a Riehle torsion machine was used in bending This method of bending was criticized in the discussion of the report as not being representative of the methods generally employed It was replied that the method used gave uniform procedure and control over time of bending Some of the results are summarized as follows

Plain and deformed under $\frac{3}{4}$ in	97 per cent bent to 180 degrees
Plain and deformed $\frac{3}{4}$ in or over	97 6 per cent bent to 90 degrees
Twisted bars under $\frac{3}{4}$ in	96 5 per cent bent to 180 degrees
Twisted bars $\frac{3}{4}$ in or over	93 2 per cent bent to 90 degrees

The results from the yield point and ultimate tensile strength tests were satisfactory

12 McMillan, F R "Time and Shrinkage Affect Stresses and Deflections of Reinforced Concrete Beams" *Engineering Record*, Vol 72, p 251 August 28, 1915

In summary of tests in 1913 of beams under time loading it is pointed out that excessive shortening due to shrinkage may be 5 to 15 times that expected from ordinary calculations

13 Smith E B "The Flow of Concrete Under Sustained Load" *Proceedings* Am Conc Inst , Vol 13, p 99 1917

In 3-in diameter cylinders 24 in long, tested in 1914, the flow was shown to be 100 per cent in 4 or 5 days and another 400 per cent in about 20 days In two beams 5 in x 8 in x 10 ft with 0 75 per cent steel the deflection was doubled in 48 days and the elastic recovery was shown to be about 25 per cent in 2 days

14 Talbot, A. N and Gonnerman, Harrison F "Test of Flat Slab Floor of Western Newspaper Union Building" Univ of Illinois, Engr Exp Sta, *Bulletin No 106* 1918

This building, tested in 1917, had reinforcement with an elastic limit of 63,000 lb per sq in and an ultimate strength of 101,300 lb per sq in These tests showed satisfactory interaction of high elastic limit steel with concrete and in particular it should be pointed out that the portion of the Western Newspaper Union Building tested had been in use for nine years, during seven of which the floor had been occupied by printing presses The maximum observed stresses in the floor were as follows

Bars
- f_c for upper side of slab
  - Diagonal—57,300 lb per sq in
  - Rectangular—42,000 lb per sq in
- f_c for lower side of slab
  - Diagonal—24,000 lb per sq in
  - Rectangular—30,000 lb per sq in

Concrete E_c—4,000,000
- f_c for upper side of slab—3,880 lb per sq in
- f_c for lower side of slab—4,400 lb per sq in

Strain gages were used and the stresses observed were only those due to the test loads applied

15 Slater, W A "Structural Laboratory Investigations in Reinforced Concrete Made by the Concrete Ship Section, Emergency Fleet Corporation" *Proceedings* Am Conc Inst, Vol 15, p 24, 1919

Accelerated corrosion tests of reinforcing bars not imbedded in concrete as well as the effect of width of cracks on the leakage of water under pressure Drop impact tests on slabs were also reported on p 43 with photographs of the slabs after test Destruction of the concrete without breakage of the reinforcing was apparent It is understood that high carbon bars were used in some specimens

16 Slater, W A, Hagener, Arthur, and Anthes, C F "Tests of a Hollow Tile and Concrete Floor Slab Reinforced in Two Directions" *Technologic papers 220*, 1921–1922, the Bureau of Standards, Vol 16, p 727

These tests (Oct 1919) upon a tile and concrete floor slab reinforced in both directions with a steel having a yield point of 54,000 lb per sq in and made with 2000 lb per sq in concrete, showed good interaction between the steel (stresses to yield point values occasionally) and concrete

17 Larson, Louis J and Petrenko, Serge N "Loading Tests of Hollow Tile and Reinforced Concrete Floor of the Arlington Building, Washington, D. C." *Technologic Paper 236*, Bureau of Standards, Vol 17, p. 405 1922–1924

Tensile tests on the reinforcement of this building showed an average yield point of 53,500 lb per sq in Concrete tests showed an average ultimate strength of 3,350 lb per sq in and an average modulus of elasticity of 4,100,000 Average measured stress in the steel in gage lines crossed by cracks was about 19,500 lb per sq in Maximum observed unit deformation in concrete was 0 0005 Time loading for 92 hours increased the deformation about 25 per cent in both the concrete and the steel

18 Interstate Commerce Commission Report "Formation of Transverse Fissures in Steel Rails and Their Prevalence on Certain Railroads" Government Printing Office 1923

In a report by the Interstate Commission in 1933 on the "Formation of Transverse Fissures in Steel Rails" it was claimed that such fissures could be produced experimentally at will and located any where according to the manner of loading the rail This report appears to be in conflict with later and more extensive investigations made by the Bureau of Standards (in which it was stated that fissures were believed to be present in new rails as a result of the manufacturing process)

19 Kommers, Jesse B "Comparative Tests of New Billet Steel and Rerolled Steel Reinforcing Bars" Wis Engr Exp Sta, Engineering Series 1252, Vol IX, No 3, 1924

Altogether, 160 bars were tested in 1921 and 1922 The rerolled steel was marketed as conforming to rail steel specifications Each kind of steel was tested against A S T M specifications, A15-14 for the new billet, and A16-14 for the rerolled The effect of pre-bending upon strength constituted another aspect of the investigation Deformed bars of rerolled steel all showed satisfactory strength, but 42 5 per cent failed to meet the ductility requirements Deformed bars of intermediate grade new billet steel all showed satisfactory ductility but 50 per cent failed to meet the ultimate strength requirement Deformed bars of hard grade new billet steel showed 5 per cent which failed to meet ductility requirements, and 65 per cent which failed to meet the ultimate strength requirements It is apparent that neither the re-rolled nor the billet steel would have passed acceptance tests under specifications A16-14 and A15-14, and that these tests do not supply a basis for conclusions regarding the relative uniformity and dependability of the two kinds of steel as then marketed

20 Freeman J R Jr, and Derry, A F "Effect of Hot-rolling Conditions on the Physical Properties of Carbon Steel," *Technologic Paper 267*, Bureau of Standards, Vol 18, p 547, 1924–1925 (See reference No 44)

21 Slater, W. A, Lord, A R and Zipprodt, R R "Proof and Tests of Web Reinforcement Formulas," *Technologic Paper 314*, Bureau of Standards, Vol 20, p 387 1925–1926.

Among later tests of some importance were the investigations made during 1918 and 1919 by the U S Bureau of Standards with the Emergency Fleet Corporation on concrete ships The reinforcing bars were made from steel rejected from shrapnel manufacture and had an elastic limit varying from 55,000 lb per sq in to 70,000 lb per sq in depending primarily upon the bar size, the larger bars having the lower values This type of steel was used for web reinforcement as well as longitudinal steel and with concretes of from 2000 lb per sq in to 5500 lb per sq in ultimate strengths Results showed satisfactory interaction between the steel and concrete

22 Mylrea, T. D "Bond and Anchorage in Reinforced Concrete." *Journal*, Western Society of Engineers, Vol 31, p 11, Jan 1926

Discussion of items pertinent to bond and anchorage in reinforced concrete beams, such as relation of bend to crack formation, effects of end slips, beams with bent up bars, different methods of anchorage, stresses at bends of hooked bars, and reinforcement against splitting

23 U S Dept of Commerce Recommended Building Code Requirements for Working Stresses in Building Materials 1926

The Code presumes that in the case of overload of a reinforced concrete structure, high elastic limit steels would not produce a permanent crack where mild steel would Statement was also made that "no question of public safety is involved" in the use of rail steel reinforcement

24. Richart, F E "An Investigation of Web Stresses in Reinforced Concrete Beams," *Bulletin No 166*, Univ of Illinois Engr Exp Sta June 1927

Tests (from 1910 to 1922) on beams with different types of web reinforcement showed that the full value of high elastic limit steel could be developed with adequate web reinforcement Deflections not reported

25 Freeman, J R Jr, Dowell, R L, Berry, W J "Endurance and Other Properties of Rail Steel," *Technologic Paper 363*, Bureau of Standards, Vol 22, p 269, 1927–1928 (See reference No 44)

26 Maney, G A "The Sustained Loading of Concrete Beams Reinforced with Compressive Steel," *Engineering and Contracting* Vol 68, p 329 Aug 1929

These tests of sustained loading of beams carrying compressive steel, compared to those without, indicate that the compressive steel reduces the deflection of the beam as much as one-third in some cases Plastic flow and shrinkage doubled the deflection in two months

27 Freeman, J R Jr, and Solakian, Haig N "Effect of Service on the Endurance Properties of Steel Rails" Bureau of Standards, *Journal of Research* Vol 3, p 205 August 1929 (See reference No 44)

28 Saliger, R "Versuch mit Stahlbewehrten Betonbalken," *Report*, Second International Congress for Bridge and Structural Engineering, p 156, Wien 24–28, IX, 1928, Wien, Verlag Von Julius Springer, 1929

Tests in 1928 compared beams containing steel averaging about 67,200 lb per sq in elastic limit with those containing steels of 47,800 and 40,000 lb per sq in elastic limit Total lengths were 8 85 ft with clear spans of 7 85 ft, flange width of 15 in, stem width of 6 3 in flange thickness of 3 94 in, a total depth of 12 6 in, concrete covering over steel of 0 63 in, and 0 475 in between layers of steel Percentages of reinforcement were 0 5, 1 1, and 1 7 Vertical stirrups and bent up bars were spaced the same for all beams Loads were applied at the third points In all, there were 8 beams with steel of 40,000 lb per sq in elastic limit, 18 beams with steel of 47,800 lb per sq in elastic limit, and 6 beams with steel of 67,200 lb per sq in elastic limit of which two contained a large number of small bars instead of the usual practice of a small number of large bars Concrete strengths of 3,350 and 4,050 lb per sq in were used in beams with 0 5 per cent reinforcement and 3,440, 4,050 and 4,530 lb per sq in in beams with 1 1 per cent reinforcement, the higher strength concretes being used with the higher strength steels The beams containing 0 5 per cent reinforcement showed an increase in load carrying capacity of 56 per cent when large bars of 67,200 lb per sq in elastic limit steel were used in place of the 40,000 lb per sq in elastic

limit bars When small bars were used, this increase was 64 per cent One beam using the large bars of high elastic limit steel broke due to failure in bond The beams containing 1 1 per cent reinforcement showed an increase in load carrying capacity of 55 per cent with large bars of 67,800 lb per sq in elastic limit steel in place of the 40,000 lb per sq in bars Both beams with 1 1 per cent of high elastic limit steel in the form of large bars broke due to failure in bond All the remaining beams failed from reaching the yield point of the steel None of the beams containing 1 7 per cent of steel of 67,200 lb per sq in elastic limit were reported It was significant that all failures of bond occurred in beams reinforced with large bars of high elastic limit steel The use of smaller bars apparently improved the bond resistance

29 Slater, W A, and Smith, G A "Tension, Bend and Impact Tests on Reinforcement Bars," A S T M *Proceedings*, Vol 29, Part 2, p. 183, 1929

Tension, bend and impact tests (made during 1921–1922) on reinforcing bars were reported in 1929 by Slater and Smith with a view to improving bend test methods The $\frac{7}{8}$-in square bars (not now standard for all portions of U S) tested had a yield point of 66,000 lb per sq in and an ultimate tensile strength of 123,300 lb per sq in on the average The physical properties of this high-carbon steel showed no doubtful qualities excepting in the bend tests The diameter of the mandrel was three times the thickness of the bars The results showed 3 passing 165–180 degrees bend and 12 passing the 90 degree bend out of a total of 22 bars

30 Saliger, R "Tests on Columns with High Tensile Steel Reinforcement" (Versuch und Sanlen mit hochwertiger Stahlbewehrung) *Beton und Eisen*, Vol 29, No 1, p 7012, Jan 5, 1930

Tests on columns reinforced by the Bauer system of spiral reinforcement made in units Ten columns were tested with vertical reinforcement of about 9 per cent of core area, and spiral of 2 per cent Up to stresses of about 1200 lb per sq in the concrete and steel act together but for higher concrete stresses (as at 2100 lb per sq in) the outer shell of concrete separates from the core long before failure of column, and it is stated that design should be on core area rather than on gross area

31. Freeman, J R Jr, and France, R D "Endurance Properties of Some Special Rail Steels," Bureau of Standards, *Journal of Research*, Vol 4, p 851. June, 1930 (See reference No 44)

32 Freel, W I "Observations of an Exposed Reinforced Concrete Beam," A C I *Proceedings* Vol 26, p 278, 1930

Discusses the effect of cracks on corrosion of steel in a beam tested to failure and thereafter exposed to the weather without carrying load Minute cracks did not affect steel while large cracks at points of failure did permit corrosion, as would be expected

33 Mylrea, T D "Studies of Shear in Reinforced Concrete Beams," *Transactions* A S C E, Vol 94, p 734 1930

Outlines methods of taking care of shear in reinforced beams in order to make the concrete and steel act as nearly as possible like one homogeneous material

Suggestions offered are, crossing bars to opposite side of beam, bending bars with extremely large radii, bending bars into flange, providing anchorage capable of developing ultimate strength of the steel without appreciable slip

34 G A Maney "Shrinkage of Concrete as Factor in Compressive Steel Stress," *Engineering and Contracting*, Vol 69, p 234, June, 1930

Further tests on sustained loading of reinforced concrete beams carrying compressive steel (see No 26) Conclusions given are (a) plastic flow is negligible in increasing compressive steel stress, shrinkage being the controlling factor, (b) the amount and effect of shrinkage are predictable

35 Saliger, R "Tests of Columns Reinforced with Welded High-Grade Steel (Versuche und Sanlen mit Geschweisster Hochwertiger Stahlbewehrung) *Beton und Eisen*, Vol 29, No 17, pp 310–315, Sept 5, 1930 Continuation of tests reported on Bauer system (see No 30)

Principal findings show that presence of welds in high elastic limit reinforcement reduced the effective compressive stress (of the longitudinal reinforcement) by about 10 per cent

36 Glanville, W. H "Studies in Reinforced Concrete, II Shrinkage Stresses," Dept of Sci and Indus Research, *Building Research Technical Paper No 11*, London, 1930

An investigation of stresses in a reinforcing bar due to shrinkage of the concrete Description of the shrinkage stress distribution along a bar is given An approximate method is suggested for allowing for the creep in the concrete when calculating shrinkage stresses A formula is given for the steel strain due to shrinkage of concrete in a symmetrically reinforced member It is stated that, with high percentages of steel or with beams reinforced at the bottom only, some release of stress must result from cracking of the concrete in tension Shrinkage stresses vary but little with normal variations in mixtures, including variations in water content The reason given is that the different moduli of elasticity, shrinkages and creeps generally balance one another at least to a great extent Normal portland, rapid-hardening portland, and aluminous cements were investigated

37. Glanville, W H Dept of Sci. and Indus Research, *Building Research Technical Paper No 12*, London, 1930

It is stated that the effect of creep in plain concrete may be allowed for by a reduction in the modulus of elasticity (that is, if 'e' is the elastic strain under unit stress and 'c' is the creep under unit stress in the time under consideration, then the effective modulus of elasticity would equal $1/(e + c)$ This is not safe for reinforced concrete except as a rough guide Formulae for columns are developed in the paper and checked by experiments on small reinforced concrete columns One series of tests show a measured steel stress of 6,600 lb per sq in on loading and 20,000 lb per sq in after 8 months under load It is stated that for small percentages of steel the yield point may be reached but for quantities as great as 5 per cent a stress of 30,000 lb per sq in will not be exceeded even when high strength concretes are used The creep of steel is suggested as a possible factor in column design where the stresses in the steel are high The relation between steel percentage and initial concrete stress for a single ultimate steel stress is given as being practically linear Normal portland, rapid-hardening portland, and aluminous cements were investigated.

38 Slater, W A and Lyse, Inge First Progress Report on Column Tests at Lehigh University, A C I *Journal, Proceedings*, Vol 27, p 677. Feb 1931

In the report of series 2, a rail steel having an average yield point stress of 65,320 lb per sq in was used for longitudinal reinforcement in comparison with steels having yield point stresses of 40,000 to 57,000 lb per sq in Results from the tests are discussed on p 720 under article 21 concerning the "Relation Between Yield Point Strength of Longitudinal Reinforcement and Strength of Column" Figure 31 (p 717) shows the effect upon strength of column of increasing the yield point stress in the longitudinal reinforcement, the percentages being the same for all grades The columns with rail steel reinforcement show a considerable increase in load carrying capacity Results are also tabulated on p 727 showing the effect of grade of longitudinal reinforcement on column strength

39 Richart, F E and Staehle, G C Progress Report on Column Tests at the University of Illinois, A C I *Journal, Proceedings*, Vol 27, p 731 Feb 1931

In the report of series 2 a rail steel having an average yield point stress of 68,300 lb per sq in was used as longitudinal reinforcement in comparison with steels having yield point stresses ranging from 39,000 to 53,400 lb per sq in Results are shown in Fig 9, p 753, Fig 10, p 755 and in Table 10, p 758, and discussed on pp 753–757 The results of these tests confirm those reported from Lehigh University (A C I Journal Proc , Vol 27, p 677)

40 Davis, R E and Davis, H E "Flow of Concrete under Action of Sustained Loads" A C. I *Journal, Proceedings*, Vol. 27, p 837. March 1931.

The results of this investigation show that creep under sustained compressive load continues for a long time Specimens under load in dry air continued to deform after 3 years under load Creep is greater for lean mixtures The amount of creep often exceeds the initial deformation (total deformation more than doubled) Reinforcement reduces both shrinkage and creep After a period of sustained loading the steel stress is increased greatly The modulus of elasticity of concrete is raised by sustained loading but the compressive strength does not appear to be altered

41 Bradbury, R D "Report on Welded Wire Fabric as Reinforcing Material," Wire Reinforcement Institute, Washington, D C 1931

Quotes test results and discusses many of the items especially pertinent to this material A working stress of 25,700 lb per sq in was advocated for welded wire fabric made from cold drawn wire

42 Gilkey, H J and Raeder, Warren "New Formula for Concrete Columns Needed," *Civil Engineering*, Vol 1, No. 10, p 924. July 1931

This article gives a short review of investigations by Otto Graf, University of Illinois, W K Hatt, F R McMillan and M B Lagaard, R E Davis, and H. E Davis, and discusses data from a progress report of the A C I Column Research The effect of shrinkage and flow in the concrete is shown to be a major factor in producing stress in the steel Research conducted by the writers on vertical steel left protruding to assure yield point stresses showed that a tied

column will split at the yield point of the vertical steel, but that in a properly spiraled column a yield point stress is not a strength hazard and also that the value of "n" has little meaning in a spirally reinforced column

43 Wyss, Aug Th "Cracking of Reinforcing Bars at Cold Bends," Translation by Beyer, Albin H, *Civil Engineering*, Vol 1, No 14, p 1266 Nov 1931

An investigation of the cracking of reinforcing bars at cold bends showed the necessity for a revision in the practice of bending bars cold The research, conducted on mild steel bars, showed quite plainly the detrimental effect of sharp bends in any reinforcing steel

44 Quick, G Willard "Tensile Properties of Rail Steels at Elevated Temperatures," Bureau of Standards *Journal of Research*, Vol 8, Paper No 408 Feb 1932

Several investigations on rail steels and rails were conducted by the U S Bureau of Standards These investigations are more or less related to one another and are reported at various times from 1924 to 1932 The researches covered the effect of hot rolling conditions, effect of service, tensile and impact properties at elevated temperatures, and endurance properties of rail steel The principal results of importance regarding rail steel were that (a) finishing temperature is an important item of the rolling process, 700 degrees Centigrade seeming to give more satisfactory results than 1000 degrees Centigrade, (b) service does not affect the endurance or other physical properties of the steel in the rail, (c) transverse fissures seemed to develop from a defect produced by service, (d) internal failures or fissures may be due to thermal stresses developed in the steel while cooling through the "secondary brittle range," the slow cooling of rail steel, therefore, being essential, and (e) steel from fissured rails shows the same endurance and other physical properties as steel from unfissured rails

45 Morrison, R L and Crum, Roy W Report of Investigation of Use of Rail Steel Reinforcement Bars in Highway Construction, *Proceedings* Highway Research Board, Vol 11, Part II 1932

Report of Committee (R L Morrison, Chairman, R W Crum, Editor The report covers in considerable detail questions concerning (a) physical properties, (b) uniformity of material, and (c) method of manufacture The results from tests of the physical properties of the bars were exceptionally good, over 99 per cent of the bars passing A S T M Serial Designation A 16–14 The percentage includes rejections from manufacturing imperfections as well as from standard physical tests The investigation of the uniformity of a heat showed no abnormal deviations and indicated that the A S T M requirements were adequate to insure a consistently uniform product Report also includes summaries of other tests

46 Slater, W A and Lyse, Inge Third Progress Report on Column Tests at Lehigh University A C I *Journal*, Vol 28, p 159 1932

The purpose of the tests reported was to study the effect of high yield point wire spiral on column strength Results are shown in Fig 4, p 165, from which it may be concluded that the high yield point of the spiral increased the strength of the column

47 Richart, F. E and Staehle, G C Third Progress Report on Column Tests Made at the University of Illinois A C I *Journal, Proceedings*, Vol 28, p. 167 1932

The purpose of the tests reported was to study the effect of drawn wire spiral reinforcement Results are tabulated on p 172 from which it was concluded (p 175) that the "margin of strength attributed to the spirals was closely proportional to the yield point or useful limit of the two grades of spiral steel (hot rolled and cold drawn)," thus substantiating the tests at Lehigh University However, the breaking of the columns with cold drawn wire was described as being sudden and violent due to breakage of the spiral, but those with the hot rolled spirals passed the ultimate load slowly and without breaking of spiral

48 Posey, Chesley J "New Type of Reinforcing Bar Develops High Bond Stress," *Engineering News-Record*, Vol 110, No 15, p 461 April 13, 1933

Illustration and discussions of a new type of bar in which closely spaced notches (crenulations) are used to develop a higher bond stress than the usual deformed bar Results of tests show bar to be from 30–70 per cent more effective than the ordinary deformed bar

49 Reinforced Concrete Column Investigation Tentative Final Report, A C I *Journal, Proceedings*, Vol 29, p 275 1933

The principal results to be noted were that time yield and shrinkage had the effect of lessening the stress in the concrete and increasing the stress in the steel, the effect of shrinkage being much less than that of time yield under load Quoting from the Final Tentative Report, "The redistribution of stress due to time yield and the resulting development of high steel stresses approaching the yield point has no effect upon the ultimate strength of a column when tested to failure " The amount of increase in steel stress varied inversely with the percentage of vertical steel

49(a) Lyse, Inge "Tests of Reinforced Concrete Columns " Civil Engineering, Vol 3, No 7 (July 1933), p 366

Reports on tests supplementing those of "Reinforced Concrete Column Investigation" (Reference No 49) for longitudinal steel up to 17½% Includes tests with high elastic limit steel and also with welded connections Conclusions No 3 and 4 are as follows "3 The longitudinal reinforcement, up to 17½ per cent, added its full yield-point strength to the column This was the largest amount of such reinforcement used in the investigation " "4 High yield-point steel added about 20 per cent more than its yield-point strength to the strength of the column "

50 Scholer, C H "Bend Testing of Steel " *Proceedings* Highway Research Board, Vol 12, Part I, p 231 1933

The bending machine used was designed to conform to the tentative revision of the A S T M Standard Specifications A16-14 and A16-30, that the bend shall be made without restraint of the bar or any part of the machine Test data given in the report showed that it was difficult to get steel to fail in the bend test even when bent around its own diameter The machine appeared to be quite satisfactory for bending of high elastic limit steel bars as well as simple in its operation

51 Richart, F E, Brown, R L and Taylor, T G "The Effect of Plastic Flow in Rigid Frames of Reinforced Concrete" A C I *Journal, Proceedings*, Vol 30, No 3, p 181. January–February 1934

The tests were started in December, 1931, and the frames were subjected to sustained loading and observation for two years In the summary, it was stated that deformations due to time yield could be considerably reduced by the use of compression reinforcement Final displacements were from 3 to 6 times the initial and the shrinkage was only one-third to one-fifth as important as plastic flow.

52 Gennet, C W Jr "Defects in Railroad Rails" *Civil Engineering*, Vol. 4, No. 5, p. 233, May 1934

A resume of the status of the rail defect problem, covering such items as rail manufacture, rail fissures and failures, marking of rails, character of transverse fissures, detection of fissures, explanation of fissures, study of manufacture and composition, and the difficulties encountered in the solution of the problem Of particular interest is the summary of results of a Sperry Fissure Detector car operated on 30,000 miles of track

53 Emperger, Dr. Fritz von. "The Application of High-Grade Steel in Reinforced Concrete *The Structural Engineer*, Vol XII, No 3, p 160 March, 1934. (*The Journal of the Institution of Structural Engineers*)

Advocates use of high elastic limit steels with increased stresses and discusses especially the recently developed Isteg reinforcement

54 Vogt, Fredrik, and Gilkey, H J Arch Dam Investigation Vol II (1934) published by "The Engineering Foundation," 29 West 39th St, New York, under authorship of J L Savage, I E Houk, H J Gilkey and Fredrik Vogt, Performed 1928–29

Reports on sustained loading tests of plain concrete in compression and in flexure Results summarized on pages 518 and 519 Specimens loaded to 25 and 50 per cent respectively of ultimate strengths at time of loading Loads were applied at ages 3 days, 7 days, 28 days, and 3 months Specimens were kept under load in either air or water for from 5 months to a year Both compressive and flexural specimens loaded in dry air for a period of about 8 months took deformations of from 4 to 7 times the deformation that occurred immediately upon the application of load For the beams in air, the final total deflection equalled about $\frac{1}{700}$ of the span under a load which was half the ultimate After one month under load the deformations were from 3 to 5 times the initial For specimens which were kept submerged during the loaded period the deformations ceased to increase after the first month, having reached a total of only 1½ times the initial deformation Such data give some indication of limits to which it might be possible for concrete to unload upon reinforcement if present

55 Mylrea, T D "Tests of Reinforced Concrete T-Beams" A C. I. *Journal, Proceedings* Vol. 30, p 448. 1934.

Two beams reinforced with cold drawn nickel steel bars of 70,000 lb per sq in elastic limit and 3 beams with steel which had an elastic limit of 96,000 lb per sq in Concludes that with suitable anchorage of steel the full elastic strength of such steels can be developed with certainty.

56 Richart, F. E and Brown, R. L. "An Investigation of Concrete Columns" *Bulletin 267*, Univ. of Illinois Engr. Exp Sta. June 1934. See references 39, 47 and 49.

In addition to the material previously reported, Bulletin 267 includes as its series 31, page 78, heretofore unreported tests made in 1931 of high elastic limit steel as vertical reinforcement Quoting (p 81) "It may be concluded that the high-strength vertical reinforcement (with yield points of 71,700 and 96,400 lb per sq in) used in these tests was fully effective in producing column strength, etc" Some data were also obtained on dowel splices

57 Emperger, Dr Fritz von "Die Rizfrage bei hohen Stahlspannungen und die zulassige Blozlegung des Stahles" *Mitteilungen uber Versuche* Heft 16 Wein 1935 Similar to reference No 53 but in German

58 Gilkey, H J. *Proceedings* Am. Soc. C. E., Vol. 61, No 1 (Jan. 1935), p 133.

Lists unit deformations in compression, flexure, tension and torsion for two concrete mixtures at the ultimate load and also at 50% of the ultimate load for a variety of ages and curings for both the dry and saturated conditions at test

Chapter III Design Procedure and Possible Economies

In Chapter I attention was directed to a number of questions on which further research or study is desirable One of the most important of these was that of possible economy through the use of high elastic limit steel and the extent to which established design procedure might need alteration to best adapt it to such use

Without increase in permissible stresses the design problems for high elastic limit steels are identical with those for steels of any other grade and the only question of economy involved is that of relative cost of purchase and handling high elastic limit steel in comparison with other steels This investigation is prefaced, therefore, upon a presumption of increased working stresses in the steel.

A study of the savings that would be possible by the use of increased stresses supplies a basis for judging the extent to which special measures might be justified in modifying the details of design to permit their use.

A brief study will be made of the mechanics involved after which relative economies will be considered.

THE MECHANICS OF HIGHER DESIGN STRESSES

Flexural Members

Given in Figure 2 (solid lines) a beam of a usual concrete mixture reinforced with mild steel Three changes are possible

1 A stronger concrete may be used with an increased working stress, but with no change in the stress used for the steel

2 The steel stress may be increased without alteration of mixture or stress in the concrete.

3 A stronger concrete may be used with increased stress in both concrete and steel

The mechanics of each of these alternatives will be discussed briefly

1 *Increased stress in the concrete, no change in the steel* (*Fig. 2a dotted*) This represents the only alternative available when the grade of steel is specified Steel stresses are fixed but there is often leeway for the designer to specify the strength of the concrete within limits. This case is pertinent to the present study only as an alternative method of obtaining strength and it may be surveyed as an introduction to the other two alternatives each of which is germane While f_s itself is not altered, $\frac{f_s}{n}$ is increased because E_c increases with the strength of mixture

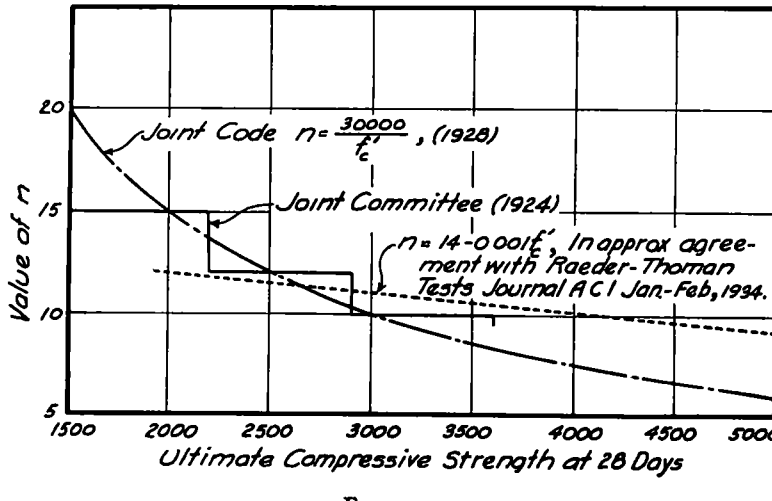

Figure 1. The Relation of $n(= \frac{E_s}{E_c})$ to f'_c (The Ultimate Compressive Strength of the Concrete.)

used and n is decreased accordingly Fig 1 shows what the relationship of E'_c to f'_c is assumed to be in the 1924 Joint Committee Report and in the 1928 Joint Building Code of the American Concrete Institute and the Concrete Reinforcing Steel Institute. There is shown also a third relationship based upon recent tests by Raeder and Thoman (Journal A C I, Jan -Feb 1934, p 236) which is probably a better approximation than either of the others Since the Joint Building Code formula is but a smoothed-out version of the Joint Committee specification and represents current design practice, it has been selected for these studies and n is assumed to vary with the ultimate compressive strength as shown on Figure 2, namely $n = 30{,}000/f'_c$ [4] For this expression, the

[4] According to the Raeder-Thoman tests, n changes at only about half the rate indicated by this expression The figures could easily be altered to make them

neutral axis does not shift but as the quality of the concrete is varied, f_c changes, and n also changes at a rate that leaves the position of the neutral axis unchanged as shown dotted in Figure 2a The effects are as follows

(a) Total compressive stress is increased (compressive area unchanged, average value of f_c increased)

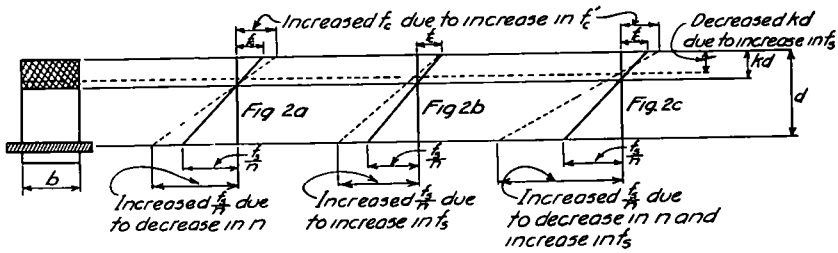

Figure 2 The Mechanics of Alternative Stress Combinations in a Reinforced Concrete Flexural Member

Tabulation to Show Relative Effects for Stress Increases of 50 Percent (The increases are intentionally extreme. The effect of more conservative increases can be estimated)

Items		Original Values (Condition for Solid Line)	f'_c Increased (Fig 2a, dotted line)	f_s Increased (Fig 2b, dotted line)	f'_c and f_s Increased (Fig 2c, dotted line)
f'_c		2000	3000	2000	3000
f_c		800	1200	800	1200
f_s		20,000	20,000	30,000	30,000
n		15	10	15	10
k		0 375	0 375	0 286	0 286
$\frac{M}{bd^2}$		131	197	103	155
p		0 0075	0 01125	0 0038	0 0057
Relative effectiveness per pound of	Steel	1 0	1.0	1 54	1 55
	Concrete	1 0	1 5	0 79	1.18

Straight line relationship assumed Values computed with the following formulas (usual notation),

$$1.\ \frac{M}{bd^2}=\frac{f_c}{2}\left(\frac{3k-k^2}{3}\right),\ 2\ \frac{f_s}{f_c}=\frac{n}{\left(\frac{k}{1-k}\right)},\ 3\ \frac{2f_s}{f_c}=\frac{k}{p},\ 4\ n=\frac{30000}{f'_c}\left(\text{See Joint Standard Bldg Code 1928}\right)$$

(b) More tensile steel must be added to maintain balance

(c) These two changes increase both the coefficient of resistance and the steel ratio required

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applicable to the Raeder-Thoman relationship Roughly, it could be expressed as $n = 14 - 0\ 001\ f'_c$ up to about 5000 lb per sq in for which n would equal 9 instead of 6 as it does according to the Joint Code expression

The structural elements might be adjusted to the changed condition in any of the following ways:

(a) Smaller beams could be used without change in center to center spacing or in span

(b) Increased spacing could be used for beams without alteration of their dimensions

(c) The same size beams could be used on longer spans without change in spacing

(d) The depth of a slab could be decreased or its span could be increased.

The first alternative would probably be the one selected in many cases since the other two each involve changes in layout It will be noted that this case affords a primary saving in concrete which, in amount, is directly proportional to the increase in working stress There are secondary savings of both steel and concrete because of the reduction in weight of member required to develop a given resisting moment Both the primary and secondary savings would be decreased by whatever amounts depths or widths had to be increased above the minimum required in order to accommodate the steel Steel is more expensive than concrete and the possibilities of savings from increased strength of concrete alone are not as great as those from either of the two alternatives that follow. This is especially true for T beams which usually have excess compressive area available without recourse to higher concrete strengths

2. *Increased Stress in Steel Without Change of Quality or Stress for the Concrete.* (*Fig. 2b dotted*) Were higher working stresses for steel to be authorized, this alternative would doubtless be used frequently, especially by routine designers of ordinary structures The effects are as follows.

(a) The neutral axis is raised.

(b) The coefficient of resistance is decreased (the same average compressive stress acts on a reduced area of concrete and the slight increase in moment arm (jd) is insufficient to overcome the reduction in total compressive force) [5]

(c) Steel percentages will be reduced in direct proportion to the increase in steel stress There will be secondary reduction corresponding to the increase in the effectiveness of the steel by virtue of the slightly increased jd arm and a slight secondary increase due to added weight of concrete required to develop the same resisting moment

[5] This decrease in coefficient of resistance may, however, be avoided by raising the concrete stress along with the steel stress by an amount sufficient to retain the same coefficient of resistance The percentage increase in the concrete stress for this condition is approximately one-half of the percentage increase in the steel stress (See Fig 4) This would violate the conditions laid down and would be a special case of alternative No 3

(d) The bond requirement is increased by the increased stress in the steel and may have to be met by special anchorage of one form or another or by the use of smaller bars since the grade of concrete has not been altered and bond stresses cannot be increased

Structural elements could be adjusted to the change in one of the following ways:

(a) The size of a rectangular beam or the depth of a slab could be increased [6] In general, no change in dimensions of a T-beam would be necessary since there is usually an excess of compressive area available except near the supports of continuous T-beams which may be safeguarded by the use of compressive steel, haunching, etc Moreover, it might sometimes be possible to decrease the stem size because of the decreased area of steel to be accommodated

(b) Rectangular beams could be spaced somewhat closer together

(c) The span of either rectangular beams or slabs could be decreased slightly.

The first alternative is the one which would require no change in layout The tabulation on Figure 2 shows for the illustrative case a reduction of 21 4 percent in the resisting moment for a 50 percent reduction in steel required which indicates a true saving in cost

3 *Increased Stresses in Both Steel and Concrete* (*Fig 2c dotted.*) From the standpoint of the mechanics involved this is the logical manner in which high elastic limit steel should be used

(a) The neutral axis is raised as for the previous case. (Fig. 2b.)

(b) The coefficient of resistance is increased when f_c is increased an amount equal to or exceeding approximately half the percentage of the steel stress increase (Fig. 4) in spite of the decrease in compressive area for rectangular beams and slabs

(c) For a percentage increase in f'_c at least equal to the increase in steel stress there is no added bond requirement The stronger the concrete the higher is the permissible bond stress.

Structural elements could be adjusted to the change in one of the following ways:

(a) By making beams smaller or decreasing the depth of a slab

(b) By spacing either rectangular or T-beams farther apart.

(c) By increasing the span of either a slab or a beam.

The first alternative involves no change in layout

The tabulation on Fig 2 shows, for the illustrative case, a primary reduction of 24 percent in steel and an increase of 18 4 percent in coefficient of resistance for a 50 percent increase in both steel and concrete stress In other words, a beam only 84 percent as wide and containing

[6] This increase in rectangular sections could be avoided as suggested in footnote 1 by an increase in concrete stress sufficient to bring the coefficient of resistance to its former value but which would violate the conditions set for alternative No 2

about 64 percent as much steel of the high elastic limit grade will develop the same resisting moment as the beam of Fig 2a The reduction in relative weight introduces an additional or secondary saving, by reducing the load to be carried

The foregoing may be summarized briefly as follows

1 Increasing the strength of the concrete without altering the stress used for steel reduces the weight of flexural member required to develop a given coefficient of resistance There is a saving in the amount of concrete but the only saving in steel required is that due to decreased weight of member (reduction in dead load)

2 Increasing the steel stress without alteration in quality of concrete will slightly increase the weight of rectangular member required for a given resisting moment but will save steel in practically the ratio of stress increase For this case bond requires special consideration since the strength of the mixture has not been altered and the bond stresses cannot be increased

3. Increasing both the steel stress and strength of concrete offers a maximum saving for both materials There is an added secondary saving from decreased weight of member If the relative increase in strength of concrete is at least equal to that in steel stress, no added hazard for bond is involved under current design practice which permits bond stresses to increase in direct proportion to the strength of the concrete.

Compressive Members

The mechanics of concrete compressive members is more direct, in some respects, than that of the flexural members and it is obvious that material can be saved by using higher strength concrete, higher elastic limit steel or both and that the greatest saving will be attained when higher stresses are used for both steel and concrete, as is the case for flexural members The trend of practice appears to be in this direction, for both types of members, but during periods of transition, it is to be expected that the advantages of higher strength concretes will be utilized frequently without change in the kind of steel, also, higher steel stresses may sometimes be employed without stepping up the quality of the concrete The effects of a 50 percent increase in the concrete stress, or in the steel stress, or in both, on the carrying power of both tied and spiralled columns are shown in columns c, d, and o of Tables I and II Columns f, g, and h indicate the percentages of reduction in area which are possible if the load be unchanged These tables are computed from the formulas recommended by Committee 105 of the American Concrete Institute in its Tentative Final Report of the Reinforced Concrete Column Investigation (Proc A C I , 1933, p 281 and 443) and include the maximum and minimum percentages of reinforcement and one intermediate percentage in each case

It is obvious from the formulas that if equal percentages of increase are used in the stresses allotted to the steel and concrete, the strength will be increased in the same percentage This is shown on the last line of each table, (Col c, d, e,) in which the supporting power is increased 50 percent It is likewise apparent that for no change in load the area of the column could be decreased one third (bottom lines of Col f. g, h)

For axial loading, as in a column, the added effectiveness per pound of steel or concrete is, of course, in the same ratio as the increase in the design stress allotted to it

TABLE I

TIED COLUMNS

$$P = A_g\ (0\ 2\ f'_c + 0\ 36\ f_y p_g)$$

f'_c	f_y	Percent Increase in P (A_g Constant)			Percent Decrease in A_g (P Constant)		
		Reinforcement in Percent			Reinforcement in Percent		
		0 5	2 25	4	0 5	2 25	4
a	b	c	d	e	f	g	h
2000	40000	0	0	0	0	0	0
2000	60000	7 6	22 4	29 5	7 1	18 4	22 8
3000	40000	42 4	27 7	20 5	29 8	21 6	17 0
3000	60000	50 0	50 0	50 0	33 3	33 3	33 3

TABLE II

SPIRAL COLUMNS

$$P = A_g\ (0\ 25\ f'_c + 0\ 45\ f_y p_g)$$

f_c	f_y	Percent Increase in P (A_g Constant)			Percent Decrease in A_g (P Constant)		
		Reinforcement in Percent			Reinforcement in Percent		
		1	4 5	8	1	4 5	8
a	b	c	d	e	f	g	h
2000	40000	0	0	0	0	0	0
2000	60000	13 2	30 9	37 1	11 6	23 6	27 1
3000	40000	36 8	19 1	12 9	26 9	16 0	11 5
3000	60000	50 0	50 0	50 0	33 3	33 3	33 3

Bond is important at lap splices and near the ends of bars which do not butt against one another or against plates but elsewhere it is less important in columns than in beams Again the use of stronger concrete automatically improves the bond resistance

Of the three alternatives, as applied to columns, it is evident that each will produce either a stronger column of a given size or a smaller column of a given strength If the size of the column were unchanged, the spacing could be increased If it is desired to avoid a change in layout, the strength will be held constant and smaller columns will be used

RELATIVE SAVINGS POSSIBLE FROM THE USE OF HIGHER DESIGN STRESSES

From the preceding glance at the mechanics of reinforced concrete flexural and compressive members, it is apparent that there will be some saving if higher steel stresses are used even though the concrete strength not be increased It is quite clear, however, that desirable use calls for increases in each design stress, and that, from the standpoint of bond and diagonal tension resistance, the increase in the strength of concrete used should be at least as great (in percentage) as the increased stress allotted to the steel It now becomes desirable to determine if possible just what stress combinations are best from the standpoint of economy and design practice

For T-beams which are part of a floor system, the usual case, the thickness of the T is the depth of the slab, the cost of the beam being then the cost of the stem only. This decreases with any increase in the working stress for steel The investigation then may concern itself primarily with rectangular beams and slabs although some portions are applicable also to T-beams

Returning then to rectangular beams and slabs it has been pointed out by Turneaure and Maurer in "Principles of Reinforced Concrete Construction," second edition, 1912, page 228, that the cost of a rectangular beam varies inversely as the depth, directly as the square root of the breadth, and directly as the cube root of the ratio of breadth to depth Curves are shown for relative unit costs of rectangular beams for constant depth, breadth, and b/d ratio respectively. These were based on the assumption that the unit cost of higher strength concrete was equal to that of low strength concrete The curves covered stresses of 400 lb per sq in to 700 lb per sq in for concrete and 10,000 lb per sq in to 20,000 lb per sq in for steel.

In Figure 3 this treatment is extended, for the case of b/d constant, to include higher stresses for the steel and has been modified to agree with the Joint Building Code practice regarding the assumed values of $n \left(n = \frac{30,000}{f_c^1}\right)$ The resulting curves can be considered representative of average conditions, since the current cost ratio is about 50

From Figure 3 it is apparent that maximum economy is attained for steel stresses above 23,000 lb per sq in only if the concrete stress is increased above the conventional 800 to 1000 lb per sq in and it is now desirable to determine more specifically what combinations are most suitable for representative or limiting conditions Two conditions have been selected for further study, viz

1 The maintenance of a *constant coefficient of resistance* as the stresses are increased For this condition the same cross-sectional dimensions may be retained for a given bending moment, if desired, the principal advantage being a saving in steel (as steel area is reduced) with no required change in layout or in the proportions of rectangular flexural members

2. The maintenance of a *constant steel percentage* as stresses are increased In this case similar proportions may be maintained throughout but a proportionately smaller beam can be used for a given resisting moment without change in layout Fewer beams of the same size could be employed of course, with corresponding savings in formwork in addition to the savings in steel and concrete

These alternatives probably represent the limits to which design practice could be expected to go

The method used was to work out general equations and plot curves from them giving the change in the properties of rectangular beams resulting from changes in the stress combination such that in the first

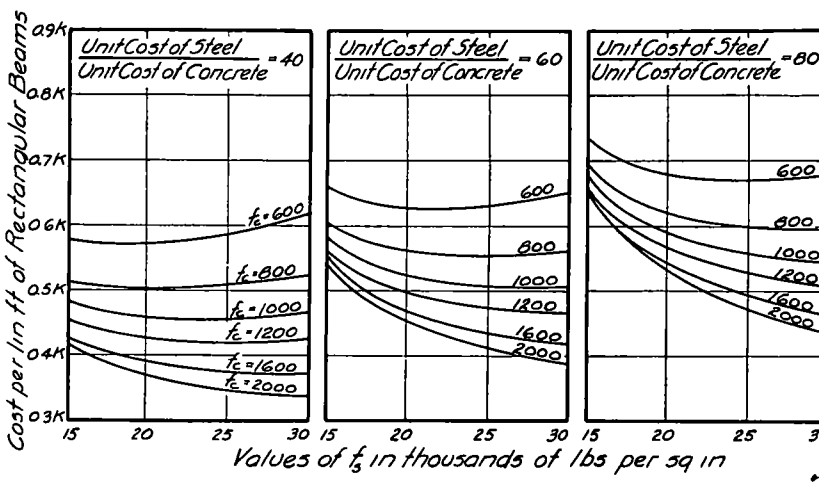

Figure 3 The Cost of Rectangular Beams with a Constant Ratio of Breadth to Depth Computed from the equation given by Turneaure and Maurer in "Principles of Reinforced Concrete Construction" 2nd edition, 1912, page 228

$$\text{The factor "K"} = \left[\frac{\text{Concrete Cost per cu ft.}}{144}\right] \sqrt[3]{\left(\frac{\text{Breadth}}{\text{Depth}}\right)\left(\text{Bending Mom. in inch lbs.}\right)^2}$$

case the coefficient of resistance remained constant, and in the second case the percentage of steel remained constant The general equations for the two cases are given below

u = ratio of new steel stress to original steel stress

z = ratio of new concrete stress to original concrete stress

v = ratio of new steel percentage to original steel percentage

ı = ratio of new coefficient of resistance to original coefficient of resistance

Case I Constant coefficient of resistance

$$z_1 = \frac{(8000 + f_s)\,(12000 + uf_s)^2}{(8000 + uf_s)\,(12000 + f_s)^2}$$

$$v = \frac{(8000 + f_s)\,(12000 + uf_s)}{u(8000 + uf_s)\,(12000 + f_s)}$$

Case II. Constant percentage of steel

$$z_2 = \frac{u(12000 + uf_s)}{(12000 + f_s)}$$

$$1 = \frac{u(8000 + uf_s)\ (12000 + f_s)}{(8000 + f_s)\ (12000 + uf_s)}$$

Figures 4, 5, 6, and 7 are plots of the above equations The effect of different values of the original working steel stress is so slight in the equations for v and i that it has been neglected These diagrams furnish a ready means of determining the amount of increase to be made in the working concrete stress for each case as well as the resulting effect upon steel percentage or coefficient of resistance.

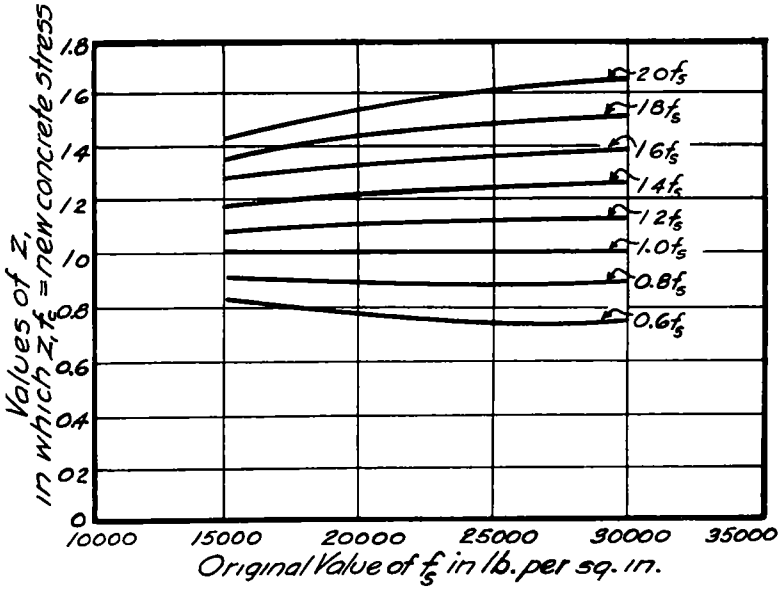

Figure 4 Curves Giving Change in F_c Sufficient to Keep $\frac{M}{bd^2}$ a Constant for a Given Change in f_s

Two commonly used stress combinations at the present time for reinforced concrete are 650 lb per sq in for concrete with 16,000 lb per sq in for steel and 800 lb per sq in for concrete with 20,000 lb per sq in for steel. Each of these combinations provides, under balanced design, a definite coefficient of resistance and percentage of steel

If it were desired to raise the steel stress to 25,000 lb per sq in or 30,000 lb per sq in and retain the same coefficient of resistance, the resulting changes referred to the 650–16,000 and the 800–20,000 combinations as bases are,

f_c	f_s	change in 'p'
650	16,000	0
820	25,000	38% decrease
920	30,000	49% decrease

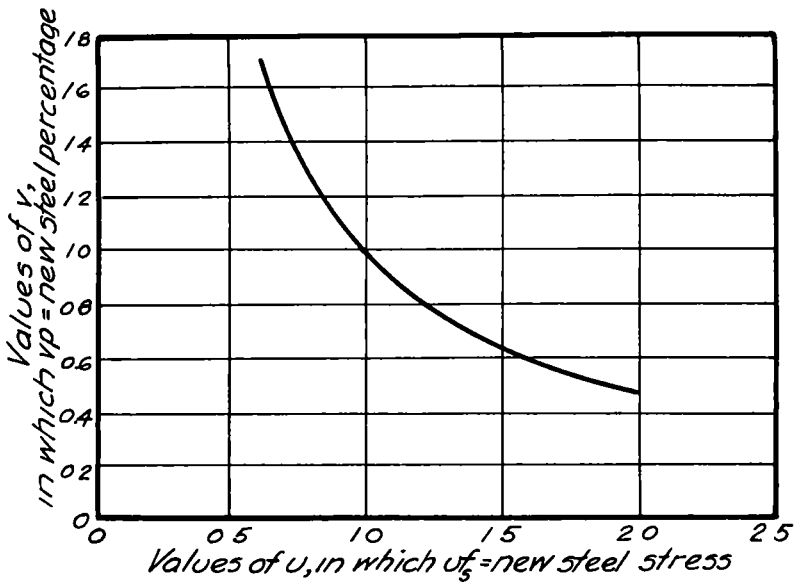

Figure 5 Curve Giving Change in p Due to a Change in f_c and f_s Such That $\frac{M}{bd^2}$ Is Constant (Effect of different values of original f_s assumed negligible)

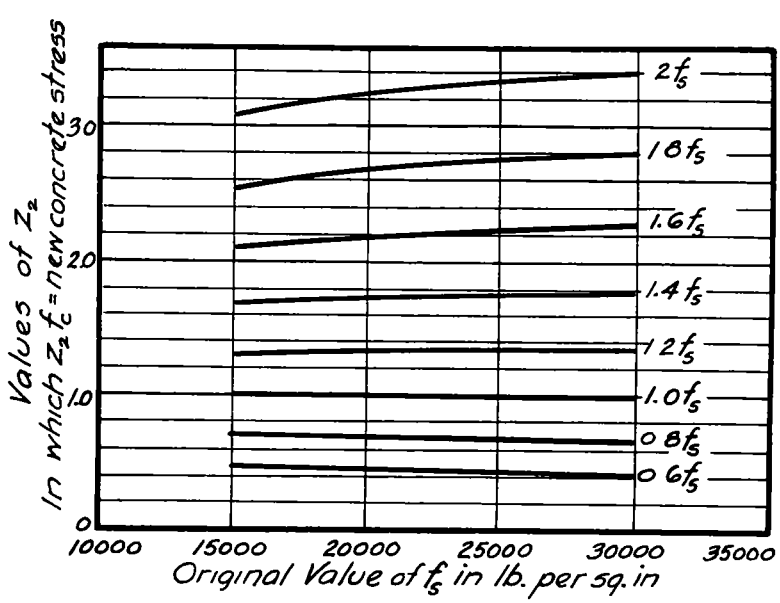

Figure 6 Curves Giving Change in f_c Sufficient to Keep p a Constant for a Given Change in f_s

f_c	f_s	change in 'p'
800	20,000	0
910	25,000	21% decrease
1020	30,000	35% decrease

On the other hand, if the same value of 'p' were desired, the resulting changes would be,

f_c	f_s	change in M/bd^2
650	16,000	0
1340	25,000	60% increase
1830	30,000	98% increase
800	20,000	0
1150	25,000	28% increase
1580	30,000	54% increase

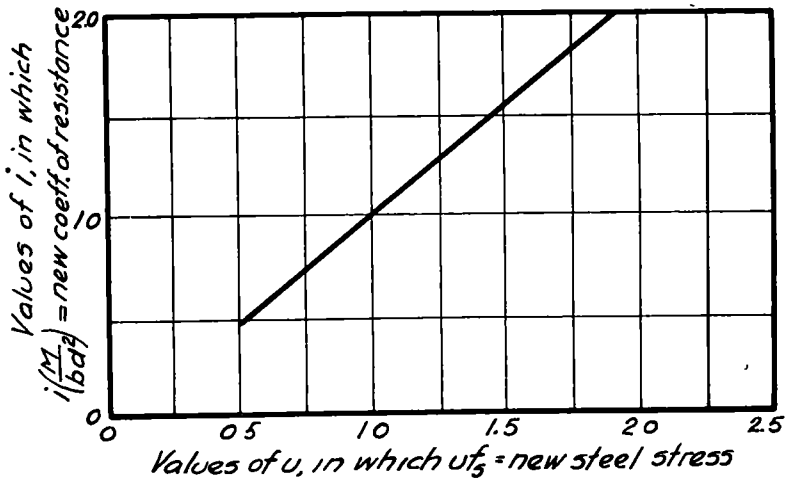

Figure 7. Curve Giving Change in $\frac{M}{bd^2}$ Due to a Change in f_c and f_s Such That p Is Constant. (Effect of different values of original f_s assumed negligible)

The stress combination of 800 lb per sq in for concrete and 20,000 lb per sq in for steel has been in general use for some time and it would seem rational to base further investigations upon stress combinations resulting from it as a base These, selected from the above in whole numbers would be,

For the same coefficient of resistance,

f_c = 900 lb per sq in with f_s = 25,000 lb per sq in
f_c = 1000 lb. per sq in with f_s = 30,000 lb per sq in

For the same percentage of steel,

f_c = 1200 lb per sq in. with f_s = 25,000 lb per sq in
f_c = 1600 lb. per sq in with f_s = 30,000 lb per sq in

Both sets of these are consistent with the values given in Figure 3 showing comparative costs

Possible Effect on Deflection Although a theoretical consideration of the deflection of concrete beams cannot be regarded as entirely satisfactory, it would no doubt indicate the general trend when higher working stresses are used

The deflection formula given by G A Maney in the A. S T M Proceedings, Vol XIV, 1914, pt II is,

$$D = K\frac{l^2}{d}(e_c + e_s)$$

in which K depends upon the condition of loading and method of support, e_c and e_s are the deformations in the concrete and steel respectively, l is the span, and d the depth of the beam If this be stated in terms of working stresses it takes the form,

$$d = K\frac{l^2}{d}\left(\frac{f_c}{E_c} + \frac{f_s}{E_s}\right)$$

The deflections, computed from the above equation, of the slab and girders of a highway bridge are given in Table III for the stress combinations listed above These deflections show no serious increase for depths equal to or greater than that of the 800–20,000 stress combination

PRACTICAL ASPECTS OF HIGHER DESIGN STRESSES

When changes are contemplated in codes covering design practice, it is always difficult to foresee all of the implications involved and to anticipate difficulties related to them For that reason it has seemed desirable to redesign an actual structure on the basis of the alterations which are under consideration For flexural members a highway bridge of 40-foot span has been selected and for compression members some typical column sections are compared

Choice of Specifications In the redesign of the structures a code on concrete construction and a set of specifications for highway bridge loadings had to be chosen to make the work comparable throughout It was decided to use the A C I joint code of 1928 for concrete construction and the current A R E A highway bridge specifications for live loading (Bulletin, A R E A, Vol 30, p 980) in so far as they both applied

Redesign of a Deck Girder Highway Bridge The designer of reinforced concrete structures is interested primarily in the final proportions and and total quantities resulting from the use of definite stress combinations Proportions of girders have been controlled, to a great extent, by such rules as making the depth of a beam between two and three times the breadth or making the depth in inches equal to the span in feet Criti-

TABLE III

PROPORTIONS, REINFORCEMENT, AND PROBABLE DEFLECTIONS OF SLAB AND GIRDERS OF A DECK GIRDER HIGHWAY BRIDGE

Class A Bridge, 40′ clear span, 24′ roadway, no sidewalks, top 1 inch of slab considered as wearing surface, L L of 2-15-ton trucks

Type of construction	Working stress		Slab		Girder				Probable Deflections in Inches $D = k \frac{l^2}{d} (e_c + e_s)$ Full working stresses assumed to be realized		
									Slab		Girder
	f_c	f_s	Total thickness (inches)	Steel Reinforcement (Top and Bottom the same)	Stem Width (inches)	d (inches)	Total Depth (inches)	Steel Reinforcement (tensile only)	Fixed Ends, Center Load l = Clear Span	Free Ends Center Load l = Clear Span	Free Ends lds @ ¼ Points l = Clear Span + l Support
6-Girders	650	16,000	8	¾″ rd @ 8″ c to c	17	35½	41	4–1¼″ round 4–1⅜″ "	0 0171	0 0342	0 595
	800	20,000	7½	⅝″ rd @ 7½″ c to c	16	31½	37	4–1″ square 4–1⅛″ "	0 0164	0 0328	0 825
	900	25,000	7½	⅝″ rd @ 10″ c to c	15	29½	35	8–1″ square	0 0198	0 0396	1 065
	1000	30,000	7½	½″ rd @ 7½″ c to c	13	29	34½	8–1″ round	0 0230	0 0460	1 265
5-Girders	800	20,000	8½	½″ sq @ 6″ c to c	16	39½	45	8–1⅛″ square	0 0272	0 0544	0 655
	900	25,000	8½	½″ rd @ 6″ c to c	16	36½	42	4–1″ square 4–1⅛″ "	0 0328	0 0656	0 855
	1000	30,000	8½	½″ rd @ 7″ c to c	15	35	40½	8–1″ square	0 0395	0 0790	1 040
4-Girders	900	25,000	9	⅝″ rd @ 9″ c to c	16	43	48½	4–1″ square 4–1⅛″ "	0 0447	0 0894	0 725
	1000	30,000	9	½″ rd @ 7″ c to c	16	38	43½	4–1″ round 4–1⅛″ square	0 0521	0 1042	0 955
	1200	25,000	8½	⅝″ rd @ 8″ c to c	16	33	38½	4–1″ square 4–1⅛″ "	0 0542	0 1084	1 000
	1600	30,000	8	½″ sq @ 7″ c to c	17	21	26½	8–1¼″ square	0 0700	0 1400	1 915
3-Girders	1200	25,000	9	⅝″ rd @ 7″ c to c	17	35	40½	8–1¼″ square	0 0950	0 1900	0 940
	1600	30,000	8½	½″ sq @ 6″ c to c	17	27	32½	8–1¼″ square	0 1255	0 2510	1 475

cism might be justified if higher working stresses were to be used in such a manner that they would produce proportions in serious conflict with present practice Let it be assumed, therefore, that there is to be a balance between the increased stresses and the loading such that the proportions of the final section will conform in a general way to present practice This balance may be obtained by varying the number and spacing of the girders The possibilities available in rearranging the members of a 40-foot deck girder highway bridge and the resultant effect upon the proportions of the members have been tabulated in Table III

The loading upon the girders was increased by decreasing the number of girders It should be noted that with the proper choice of the number of principal members a design may be had comparable either to the working stresses of 650 and 16,000 lb per sq in or 800 and 20,000 lb per sq in.

The probable deflections of the slabs and girders when the materials were assumed to be stressed to their full working values, have been included in the table and, as stated previously, indicate no serious condition among those in which the depth has been made equal to or greater than that of the 800 and 20,000 lb per sq in combination

Design of Columns In the design of typical column sections, the formulas recommended in the Final Tentative Report of the Concrete Column Investigation (Proc A C I, Vol 29, p 275 and p 443, 1933) were used. These formulas expressed the stress combinations in terms of the 28-day compressive strength of 6 by 12 inch cylinders of concrete and the yield point stress of the steel The stresses used for the column sections were,

f'_2	f_y
2000	40,000
3000	50,000
4000	60,000
5000	70,000

In the design of spiral column sections three total loads were chosen as representative of the range for ordinary spiral column construction The sections resulting from these loadings and the above stress combinations are listed in Table IV The percentages of vertical steel were kept as near to 1, 4 5 and 8 as the nearest column diameter in even inches would permit The table shows the effect of a change in stress combination, total load, and percent of vertical steel Considerable reduction in column diameter may be obtained by going to the higher stresses and percentages of steel as was shown in Tables I and II

The results from the design of the tied column sections are listed in Table V The same procedure was used with the exceptions of the lower loadings and the percentages of steel were kept as close to 0 5, 2 25 and 4 as the nearest column diameter in even inches would permit The table indicates approximately the same condition for tied columns as for spiral columns

TABLE IV

PROPERTIES AND QUANTITIES FOR SPIRAL COLUMNS

$$P = A_g\,(0.25\,f'_c + 0.45\,f_y p_g)$$

Load P Lb	Stresses Lb per sq in		Outer Diam In	Percentage of Vertical Steel p_g	Quantities (Spiral Steel Included)	
	f'_c	f_y			Concrete Cu Ft per Lin Ft	Steel Lb per Lin Ft
1,500,000	2000	40000	53	0.0100	15.3	150.4
	3000	50000	44	0.0102	10.6	99.5
	4000	60000	39	0.0096	8.3	77.5
	5000	70000	35	0.0100	6.7	61.6
	2000	40000	38	0.0453	7.9	209.3
	3000	50000	33	0.0439	5.9	155.5
	4000	60000	29	0.0467	4.6	127.0
	5000	70000	27	0.0435	4.0	102.5
	2000	40000	32	0.0763	5.2	237.6
	3000	50000	28	0.0755	4.0	178.5
	4000	60000	25	0.0750	3.2	142.3
	5000	70000	23	0.0755	2.7	118.8
1,000,000	2000	40000	43	0.0105	10.1	97.3
	3000	50000	36	0.0100	7.1	66.4
	4000	60000	31	0.0120	5.3	54.8
	5000	70000	28	0.0122	4.3	45.7
	2000	40000	31	0.0453	5.3	141.2
	3000	50000	27	0.0438	4.0	102.5
	4000	60000	24	0.0443	3.1	82.1
	5000	70000	22	0.0436	2.6	67.2
	2000	40000	26	0.0772	3.4	154.2
	3000	50000	23	0.0742	2.7	118.8
	4000	60000	20	0.0810	2.0	93.9
	5000	70000	19	0.0735	1.8	78.0
500,000	2000	40000	30	0.0117	4.9	51.3
	3000	50000	25	0.0118	3.4	35.2
	4000	60000	22	0.0118	2.6	26.2
	5000	70000	20	0.0110	2.2	21.4
	2000	40000	22	0.0450	2.6	71.5
	3000	50000	19	0.0440	2.0	52.2
	4000	60000	17	0.0448	1.6	42.4
	5000	70000	15	0.0495	1.7	37.5
	2000	40000	18	0.0810	1.6	76.6
	3000	50000	16	0.0775	1.3	58.2
	4000	60000	14	0.0820	1.0	49.0
	5000	70000	13	0.0791	0.9	43.1

TABLE V

PROPERTIES AND QUANTITIES FOR TIED COLUMNS

$$P = A_g\ (0\ 2\ f'_c + 0\ 36\ f_y p_g)$$

Load P Lb	Stresses Lb per sq in		Outer Diam In	Percentage of Vertical Steel p_g	Quantities	
	f'_c	f_y			Concrete Cu Ft per Lin Ft	Steel Lb per Lin Ft
750,000	2000	40000	45	0 0050	11 05	26 7
	3000	50000	37	0 0054	7 47	20 4
	4000	60000	32	0 0061	5 68	16 1
	5000	70000	29	0 0050	4 59	10 5
	2000	40000	36	0 0229	7 06	77 4
	3000	50000	31	0 0218	5 25	55 9
	4000	60000	27	0 0231	3 97	45 4
	5000	70000	25	0 0208	3 41	34 7
	2000	40000	31	0 0410	5 25	98 9
	3000	50000	27	0 0390	3 98	77 4
	4000	60000	24	0 0397	3 14	61 4
	5000	70000	22	0 0382	2 64	48 1
500,000	2000	40000	37	0 0050	7 47	19 5
	3000	50000	30	0 0058	4 91	13 5
	4000	60000	26	0 0063	3 69	12 0
	5000	70000	24	0 0050	3 14	8 4
	2000	40000	30	0 0213	4 92	51 0
	3000	50000	25	0 0227	3 41	38 1
	4000	60000	22	0 0232	3 64	30 1
	5000	70000	20	0 0231	2 18	24 0
	2000	40000	26	0 0372	3 69	68 0
	3000	50000	22	0 0395	2 64	50 7
	4000	60000	20	0 0362	2 18	38 2
	5000	70000	18	0 0381	1 76	32 1
250,000	2000	40000	26	0 0050	3 68	9 4
	3000	50000	22	0 0050	2 64	6 7
	4000	60000	19	0 0050	1 94	5 1
	5000	70000	17	0 0050	1 57	4 3
	2000	40000	21	0 0221	2 40	26 1
	3000	50000	18	0 0217	1 77	18 0
	4000	60000	16	0 0206	1 40	13 5
	5000	70000	14	0 0245	1 07	12 0
	2000	40000	18	0 0402	1 76	34 2
	3000	50000	16	0 0356	1 40	24 0
	4000	60000	14	0 0386	1 07	19 5
	5000	70000	13	0 0354	0 92	16 5

POSSIBLE ECONOMIES OF HIGH ELASTIC LIMIT REINFORCEMENT

In the above discussion of high elastic limit reinforcement no comments were made regarding the possible economy resulting from its use It was thought best to treat this item separately under three distinct subdivisions, (1) quantity of material, (2) formwork cost, and (3) increased cost of higher strength materials, including cost of added care in handling high elastic limit steel

Quantity of Material That considerable saving in quantity of material may be attained through the use of high elastic limit reinforcement in combination with high-strength concrete is shown in Tables I to VIII, inclusive.

TABLE VI

QUANTITIES OF CONCRETE AND STEEL IN A DECK GIRDER HIGHWAY BRIDGE
Complete Data Given in Table I

Type of Construction	Working Stresses		Slab	Girder		Quantities in Slab and Girders—Web Temp and Shrinkage and Negative Reinforcement Included	
	f_c	f_s	Total Thickness In	Stem Width In	Total Depth In	Concrete Cu Ft	Steel Lb
6-Girder	650	16,000	8	17	41	1840	18800
	800	20,000	7½	16	37	1620	16000
	900	25,000	7½	15	35	1510	13600
	1000	30,000	7½	13	34½	1400	11200
5-Girder	800	20,000	8½	16	45	1760	15400
	900	25,000	8½	16	42	1690	13400
	1000	30,000	8½	15	40½	1600	12100
4-Girder	900	25,000	9	16	48½	1640	10700
	1000	30,000	9	16	43½	1550	9500
	1200	25,000	8½	16	38½	1380	11200
	1600	30,000	8	17	26½	1170	12800
3-Girder	1200	25,000	9	17	40½	1370	11700
	1600	30,000	8½	17	32½	1220	11400

Although, in the case of the highway bridge, some designs using shallow girders show a greater reduction in material than those with the deep girders (Table VI). it must be understood that there is some loss of stiffness in using shallow sections

The column quantities show consistent reduction with increased stresses regardless of load or percentage of steel

With respect to the highway bridge, the maximum reduction in concrete quantity was obtained through the use of the 1600–30,000 combination in which the same value of 'p' was retained as for the 800–20,000 combination The maximum reduction in steel quantity was obtained by the use of the 1000–30,000 combination in which the same value of $\frac{M}{bd^2}$ was retained as for the 800–20,000 combination

As may be expected, for the column sections, the 5000–70,000 combination with a high percentage of steel gave the maximum reduction in concrete quantity, while a low percentage of steel gave the maximum reduction in steel quantities

Summarized percentages of quantities for stress combinations giving the maximum reduction in any one material are listed in Tables VII and VIII It may be seen that by the use of the 1600–30,000 combination a saving of 28 per cent in concrete and 20 per cent in steel was made over that of the 800–20,000 combination for slab and girder construction For columns, savings between 50 and 60 per cent in both concrete and

TABLE VII

PERCENTAGES OF QUANTITIES FOR STRESS COMBINATIONS GIVING THE MAXIMUM REDUCTION IN ANY ONE MATERIAL FOR SLAB AND GIRDER CONSTRUCTION

Working Stresses		Concrete Percent	Steel Percent
f_c	f_s		
650	16000	114	117
800	20000	100	100
1000	30000	96	59
1600	30000	72	80

TABLE VIII

PERCENTAGES OF QUANTITIES FOR STRESS COMBINATIONS GIVING THE MAXIMUM REDUCTION IN ANY ONE MATERIAL FOR SPIRAL AND TIED COLUMNS

Column Type	Stresses		Concrete		Steel	
	f_c	f_y	Min Steel Percent	Max Steel Percent	Min Steel Percent	Max Steel Percent
Spiral	2000	40000	100	100	100	100
	5000	70000	43	52	40	47
Tied	2000	40000	100	100	100	100
	5000	70000	42	48	40	47

steel were made by the use of a 5000–70,000 combination in place of a 2000–40,000 combination

Formwork Cost An item of importance to be considered in concrete construction is the cost of the formwork With the fewer girders used for the higher stresses there is simplification of formwork as well as a reduction in its amount. The three girder construction for the 1600–30,000 combination had approximately 41 sq ft of form surface per lin ft of slab and girder, the 1200–25,000 combination had about 45 sq ft , while 6-girder construction for the 650–16,000 combination had about 61 sq ft , and the 800–20,000 combination had 58 sq ft

In building construction the total number of columns required would, in general, be unchanged and little simplification would be possible

unless the sizes of the panels were increased Some saving in time of placing of steel should be made with the smaller amount used

Cost of Higher Strength Materials The difference in cost between steels of structural, intermediate, and hard grade is at present slight The lesser amount of steel used would much more than offset any slight increase in cost of high elastic limit steel, including cost of added care required in fabrication and handling

In opposition to the increased cost for cement per cubic yard of stronger concrete are such items as

1 Decrease in volume of concrete required
2 Saving in form work
3 Increased workability of the richer mixtures
4 Some saving in aggregate for which cement is substituted

It may be stated that the economics will usually favor the use of the stronger mixtures, although the relative importance of the individual items listed will vary and some of them will prove difficult to evaluate

SUMMARY

From a study, both of the mechanics and the practical design aspects of reinforced concrete in flexural and compressive members, there seems to be no question about the savings in cost that could be realized through the use of high elastic limit steel with increased working stresses

Some savings are present when the stronger steel is simply used to replace a larger amount of softer grade, without altering the quality of the concrete, but the following points should be kept in mind when this is done:

1 The maximum saving not realized
2. Added precautions are required to insure adequate bond and diagonal tension resistance

The use of high elastic limit steel with increased stresses and a concrete of appropriate grade offers considerable economy with no increased hazard in bond or diagonal tension This practice may be recommended as safe and desirable

ACKNOWLEDGMENTS

The authors wish to acknowledge helpful suggestions and criticisms from members of the project committee as well as from Mr A R Lord, Consulting Engineer, Chicago, Ill. Mr R D Bradbury, Engineer-Director of the Wire Reinforcement Institute, Washington, D C furnished a recent report upon the use of welded wire fabric manufactured from cold drawn wire

To Professor Louis DeVries, Head of the Department of Modern Language at Iowa State College, is due the greatest of appreciation for the translation of several German reports of researches upon the use of high elastic limit reinforcement

DISCUSSION—USE OF HIGH ELASTIC LIMIT STEEL AS CONCRETE REINFORCEMENT

MR R D BRADBURY, *Wire Reinforcement Institute* The report of the Project Committee on Use of High Elastic Limit Steel as Concrete Reinforcement presents a very comprehensive preliminary outline of this subject and discusses in a most thorough manner many of the important considerations involved The report very properly points out the fact that so-called high elastic limit reinforcement may be classified according to the method by which the elastic limit is raised, namely, by the use of a high carbon content, or by the cold working of a low carbon steel This classification thus divides high elastic limit reinforcing steel into two general types, hot-rolled bars, and cold-drawn wire, each type possessing certain distinctively different stress-strain properties, particularly as regards the interrelationship of elastic limit, yield point, and ultimate strength

Yield Point While the general class of material under discussion is designated as "High Elastic Limit Steel," it is observed that throughout the report "yield point" is apparently considered as the practical limit of reinforcing utility without making any distinction as to its relationship to "elastic limit " Such a distinction is, of course, unnecessary in the case of hot-rolled bars, since they exhibit the property of a definite and distinct yield point at practically the same stress which marks the straight-line limit of stress-strain proportionality In hot-rolled bars, this point occurs at a stress corresponding to approximately 63% of the ultimate strength Cold-drawn wire on the other hand, although exhibiting a limit of straight-line proportionality at practically the same percentage of its tensile range as in the case of a hot-rolled bar, does not, however, reveal any stress-strain property similar to yield point action until a stress of from 90 to 95% of the ultimate strength is reached

The question thus arises as to whether a cold-drawn wire really possesses any definite yield point at all, since the only phenomenon resembling yield point action, namely elongation without increase in stress, appears so high in the tensile range as to be construed more properly as a necessary accompaniment of plasticity immediately preceding rupture However, if a yield point does exist in cold-drawn wire, one thing is certain, it consistently occurs at a very much higher percentage of ultimate strength than does the yield point of a hot-rolled bar It is therefore apparent that, in order to provide comparable yield point values in the two types of steel, the ultimate tensile strength of a hot-rolled bar must necessarily be materially more than that of a cold-drawn wire

Typical stress-strain properties of welded wire fabric are indicated by the following test results These tests were made on stock commer-

cial fabric in the testing laboratory of the Massachusetts Institute of Technology Specimens consisted of strands of No 3 gauge wire with No 8 gauge cross-wires welded at 12-in intervals Results are shown in Table I for two specimens tested across welds and two tested between welds Stress-strain diagrams are shown in Figure 1 for each specimen, also tabulated results indicating the relation between ultimate strength, proportional limit, Johnson's Elastic Limit, and "probable" yield point Typical behavior of cold-drawn wire is clearly indicated by the gradual stress-strain deviation well beyond the proportional limit and also the fact that no indication of drop-of-the-beam action appears until reaching a stress of approximately 95% of the ultimate strength.

TABLE I

TYPICAL STRESS-STRAIN PROPERTIES OF PLAIN AND WELDED WIRE

Specimen	Proportional Limit	Johnson s Elastic Limit	Recorded Yield Point	Ultimate Strength
A-1	48,500	52,200	74,200	76,800
A-2	49,800	56,000	75,000	78,000
Average for specimens across weld	49,200	54,100	74,600	77,400
B-1	50,500	56,400	73,900	78,000
B-2	47,800	56,000	75,100	78,800
Average for specimens between welds	49,200	56,200	74,500	78,400
Total Average	49,200	55,200	74,600	77,900

Wire tested was No 3 gauge (2437" diameter) with No 8 gauge (1620" diameter) cross-wires welded at 12-inch intervals "A" indicates specimen tested across a weld, "B" indicates specimen tested between welds

Reduction of area for all four specimens varied less than 1 per cent, showing an average of 56 7 per cent

Effect of Welding The report suggests, as a possible subject for further investigation, the effect of electric welding on tensile properties when cold-drawn wire is fabricated into a welded mesh It would seem that a satisfactory answer to this question is furnished by the typical test data cited above, which indicate quite conclusively that a strand taken from welded wire mesh and tested in tension across a weld develops practically the same values for all of the important properties, such as ultimate strength, elastic behavior, and ductility, as are revealed by testing the same strand of wire before welding

Although ample test data are available to support the general conclusion that welding does not materially affect the tensile properties of the wire, still the question may appropriately be asked as to what

practical difference does it make whether the wire is weakened or not By merely imposing the requirement that the prescribed tensile properties shall be met by tension tests made across a weld, properties of the wire after welding are directly revealed by the acceptance test itself This is very common practice and automatically eliminates any uncertainty that may exist concerning the effect of welding

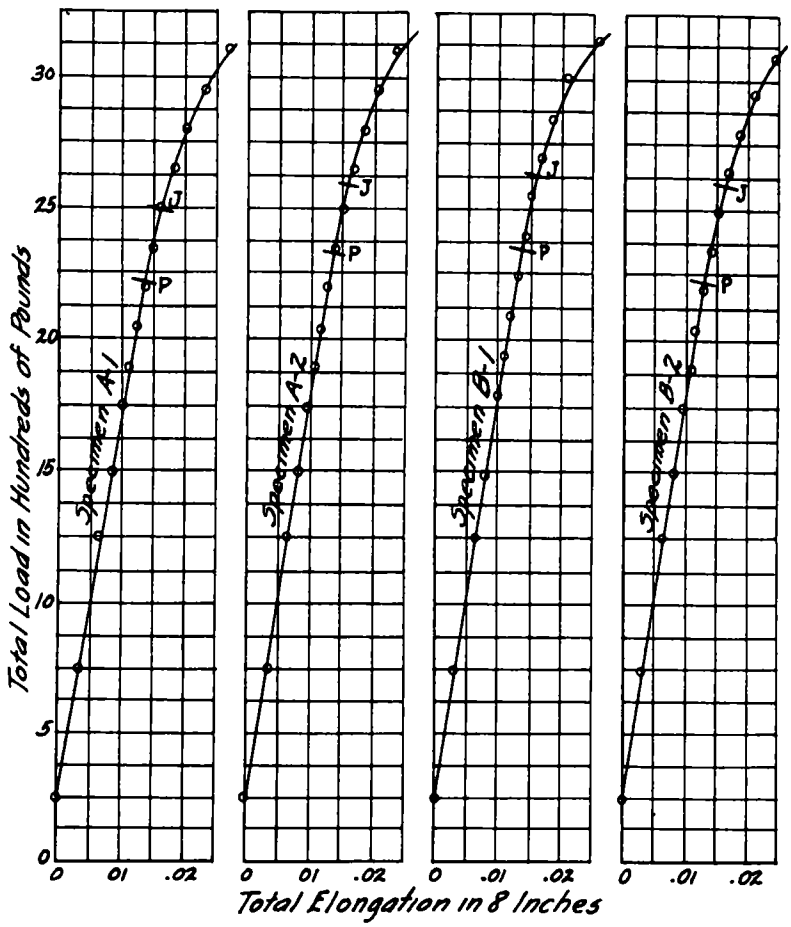

Figure 1 Typical Stress-Strain Curves for Plain and Welded Wire Each specimen, No. 3 gauge wire (.2437″ diameter) Specimens "A" tested across a welded joint Specimens "B" tested between welds. "P" indicates Proportional Limit "J" indicates Johnson's Elastic Limit.

Bond Resistance It is quite definitely known that the unit bond resistance of a clean bright strand of plain wire, as measured by initial slip, is somewhat less than that of a smooth hot-rolled bar But this also is a feature of questionable practical significance, since plain strands of cold-drawn wire are seldom if ever used as concrete reinforcement

except for spirals in concrete columns And a column spiral serves merely as a shell-like container to prevent lateral expansion of the concrete core under axial load—a structural action which does not impose any bond requirement in the sense of ability to transfer increment of tensile stress

In the case of flexural members such as concrete slabs of various kinds, bond is, of course, an all-important feature, but cold-drawn wire, when utilized as reinforcement for such members, is invariably used, not as strands of plain wire, but in the form of a fabricated mesh. When fabricated by electrically cross-welding longitudinal and transverse members, long experience and specific investigations have proved that the rigid anchoring effect of the welded cross wires provides adequate safeguard against slippage regardless of the surface bond of the wires themselves

That welded wire mesh as used for concrete reinforcement possesses adequate bond resistance to develop any allowable tensile stress that may properly be assigned to the material is supported by the results of a series of tests recently conducted by Prof E Warren Raeder at the University of Colorado Two conclusions reached by this investigation, in particular reference to the anchorage value of the cross wires, were as follows.

"Beams reinforced with a single longitudinal wire with three cross wires welded on it failed in some manner other than bond, and they developed approximately the same nominal bond stresses as did plain reinforcing bars with semi-circular hooks on the ends "

"If there are at least two cross wires on each side of the center of the beam, and the ends of the beams are properly built to prevent spalling off, the failure of the beam will probably occur in some manner other than bond "

Age-Annealing The committee appears to be somewhat apprehensive concerning the possibility of an annealing action taking place as cold-drawn wire ages under exposure to atmospheric temperatures, the report stating that this might be particularly important in locations such as boiler rooms and certain industrial structures where temperatures somewhat above the ordinary may occur It is unfortunate that the committee has chosen to designate this presumed phenomenon by compounding the two words "age" and "annealing," because the phenomenon which the committee apparently has in mind does not continue indefinitely with age, neither is it manifested as an annealing tendency

Significance of the term "aging" as applied to cold-worked steel may be illustrated by the statement that a piece of steel tested immediately after it has undergone substantial cold working will reveal a slightly different elastic behavior than will the same cold-worked steel if allowed to rest for a period of several weeks at room temperature before being tested. This difference in elastic behavior resulting from aging is

manifested principally by a stabilization or "setting" of the elastic limit, but relatively the position of the elastic limit may be either higher or lower than that of the unaged material, depending upon composition of the steel, and the amount of cold working it has undergone

The phenomenon of so-called aging is explained metallurgically by the fact that the atoms of the steel, which have been highly disorganized as a result of the deformations produced by the cold working, tend naturally to rearrange themselves in more normal positions, and, by virtue of their vibratory motion, are able to accomplish some slight rearrangement even at ordinary atmospheric temperatures However, at such temperatures, atomic energy is insufficient to materially overcome the effect of cold working and any change in atomic arrangement that occurs under such temperature conditions can progress only with diminishing intensity and only to a very limited degree of development

If one fails to consider the fact that annealing does not begin until temperatures are reached which are very much higher than even unusual atmospheric temperatures, it is very easy to reach deceptive conclusions as to possible effects of aging on cold-worked steel For instance, one may argue that, since atomic energy tends to relieve the strains produced by cold working, and since some relief is accomplished even under ordinary atmospheric temperatures, any condition tending to raise the surrounding temperature will increase atomic energy and, as a result thereof, will cause a more rapid manifestation of the aging phenomenon, and also a greater degree of strain relief The first conclusion is correct but the second conclusion may be either correct or erroneous depending upon the intensity of heat imparted to the steel

As an illustration, boiling freshly cold-worked steel in water for a few hours will accomplish the same amount of aging effect as is produced by weeks or months of aging at room temperature But, even if the heat treatment be extended to temperatures as high as 400 to 500°F, atomic rearrangement is merely accelerated without being carried beyond a certain minor degree of development In any event, the effect on tensile properties of aging under anything like normal conditions of practice is developed in a comparatively short period of time, and, at most, is quantitatively so small as to have no practical significance whatever from a reinforcing viewpoint

This whole phenomenon of aging of cold-worked steel might be described as essentially the attainment of a state of "atomic stabilization," it is not an annealing action in any sense of the term Annealing, or in other words a tendency to restore the original properties of the unworked material, requires much more than minor atomic rearrangement—it must be accompanied by an appreciable change in grain structure And change in grain structure does not even begin to appear until a temperature of about 570°F is attained When it is also realized that, in order to bring about a complete change in grain struc-

ture and relieve all strains, the steel must be heated to a temperature of some 1200°F or more, it becomes quite apparent that the highest temperatures that could conceivably occur in the normal use of a building or any other structure would necessarily be so low in comparison with required annealing temperatures as to preclude all possibilities of any long-time annealing effect of the reinforcing steel

MR C S POPE, *California Division of Highways* From my inspections of numerous concrete highways and concrete structures in which steel is used as reinforcement, I am more and more impressed by the fact that there are an enormous number of cases in which steel cannot be considered as acting as a permanent reinforcement I have seen innumerable cases in removing reinforced concrete highway slabs where steel was practically eaten away by rust at cracks to such an extent that it was almost valueless This is also an extremely unfortunate and almost universal condition with relation to concrete piles and other reinforced concrete structures submitted to the action of sea water It appears to me that one of the greatest needs for the proper permanency of reinforced construction is to secure a type of reinforcement which will not be acted upon by the elements either within or without the concrete, and I think this is a far more important matter for consideration than any other matter relating to reinforcement The best reinforcement under adverse conditions has been found practically valueless in many instances

MR B FRIBERG, *Laclede Steel Company* Some research has been done in the building construction field, establishing the considerable economy to be gained by the use of high working stresses in concrete Little attention has been given the question to what extent increased working stresses in the reinforcing steel can be economically and technically justified The submitted data, therefore, offer a considerable addition to the research in this field, and the economy that may be obtained through higher working stresses in the reinforced concrete material as established by the research would certainly seem to be a great inducement to further work, pointing toward a wider field for reinforced concrete In many instances, the use of higher working stresses in the concrete will be impaired without the possibility of using higher working stresses in the reinforcing steel It seems, therefore, correct to consider increased steel working stresses in connection with increased concrete working stresses, realizing at the same time that for one of the most common concrete design elements, the "T" beam, and for reinforced concrete columns in accordance with the A C I. Column Investigation, at least, higher steel working stresses are economically justified without consideration of simultaneously-used higher concrete working stresses.

The Committee Report presents a detailed study on the influence of varying concrete and reinforcing steel stresses on tensile reinforcement and column reinforcement The A C I Column Investigation in its final report indicates very directly the structural advantage of high elastic limit steel as reinforcement in compression This research has an important bearing upon highway structures as well, especially as regards compression reinforcement in double reinforced concrete girders and in concrete arches where savings comparable to those shown by the Committee for reinforced columns may ultimately be realized For fixed arches and other statically indeterminate structures in particular the higher working stresses, resulting in more slender sections, have an increased advantage from the standpoint of temperature stresses

The stress in the reinforcing steel is undoubtedly in present concrete design practice the factor determining the strength of a flexural member The factor of safety with relation to the steel working stresses varies from 1 8 to 2 5, depending upon the grade of steel used, whereas the factor of safety for the concrete in compression is in most instances 3 5 or higher when the strength of the concrete in bending is taken as the basis The use of high elastic limit steel for concrete reinforcement should accordingly even with present design practice receive serious consideration

When the possibility of providing higher tensile reinforcing steel stresses is considered, the appearance of the first visible crack would, according to past research, seem to place a rather distinct limit on the amount of increase which can practically be permitted In the concrete beam tests recorded, cracks seem to have appeared in each instance rather close to the permissible stress in the steel, the factor of safety against crack being on the average 1 2 In all these tests the bends made in the tension reinforcement seemed to be quite sharp, resulting in high, premature, crack inducing concrete stresses inside the bends To highway engineers it would also seem immediately apparent from the considerable experience with curing of concrete pavement how the appearance of the first visible crack might be delayed with improved curing practice for the parts of concrete structures under tension, which structural parts are apt to get the least curing

Another aspect of these characteristics of reinforced concrete is that, according to research of Professor Spangenberg, Munich, (Second International Congress, Bridge & Structural Engineers, Vienna, 1928, Reports page 170), the first crack in the tensile zone of concrete beams occurs at correspondingly higher steel stress when the design working stress in the reinforcing steel is increased, (i e the steel area decreased), his tests including steel working stresses from 14200 lb per sq in up to 28500 lb per sq in The same research does establish a very distinct change with changing concrete working stress, however, expressed in translation as follows

"Doubtless the increase in steel working stress has far less influence upon the crack occurrence than the increase in concrete working stress, so that in this respect the question of application of high grade steel in combination is not unfavorable "

This statement is in full accord with deflection findings of Messrs Gilkey and Ernst in regard to both instantaneous and time deflection of flexural members as influenced by concrete and steel working stresses [1]

The occurrence of the first crack is also delayed with decreasing size of the individual bars making up the reinforcing area

From Table III of the Committee Report showing the deflection of a 40 foot concrete girder bridge, it seems at first hand that the deflection when high reinforcing steel and concrete stresses are used would be exceptionally great. The deflection there shown seems to be, however, the combined dead load and live load deflection For a comparison with, for instance, a steel structure, the deflection under live load only is of more importance The dead load deflection, as well as the deflection due to plastic flow, can be provided for in the construction An approximate adjustment of the deflection figure in the worst case of the table (girder deflection 1 915 in) for this condition would give a live load deflection of about 55 in or $\frac{1}{870}$ of the span, a deflection figure within requirements for most cases The particularly small live load deflection of concrete bridge structures is in fact one of their structural advantages over the lighter steel structures, as is repeatedly pointed out by designers

The question of fatigue loadings on reinforced concrete structures received attention at the Second International Congress for Bridge and Structural Engineers in Vienna, 1928, Report pages 492–505, the research reported involving one million loadings and steel stress up to 27000 lb per sq in The most important result is quoted in translation· "The herewith described tests show primarily that in no case the repeated loadings changed the ultimate load" (Page 497) Tensile zone cracks observed with steel stress of 27000 lb per sq in were practically unchanged at 0 1 mm loaded and 0 04 mm unloaded after one million loadings

The future will probably see high strength alloys used for concrete reinforcement with strengths in excess of those now obtainable in commercial grades and with adequate fabrication possibilities Proposed Swiss stress regulations reported to the First International Congress for Concrete and Reinforced Concrete at Liege, 1930, Report I, page 214, would permit 23000 lb per sq in on high elastic limit steel with yield point of 50000 lb per sq in To a considerable extent such reinforcing materials are available in the market today, both in rolled carbon steels and in cold drawn steels It may be mentioned in this connection that

[1] Unpublished progress report presented at Meeting of Highway Research Board, December, 1934

beyond a slight decrease in strength of up to three per cent within the first short time after cold drawing there would not seem to be any "age annealing" of cold drawn wire used for concrete reinforcement under ordinary temperature Standard marking of high elastic limit steel reinforcing seems unnecessary at least until definite standard grades are established While marking of standard grades of reinforcing bars may be of help in quickly identifying them on the job, there should be no necessity whatever of providing rolled in or stamped identification on any one special grade of high elastic limit bars to identify them in the shop The importance of keeping different grades of material separate is so easily recognized by any one familiar with the numerous grades of steel made for different purposes in the average steel mill that further identification beyond that already used in the mill would seem an undue hardship Careful and responsible inspection furnish entirely adequate guarantee to the builder that the steel intended for use is obtained when the specifications are agreed upon

The question of proper fabrication of reinforcing bars so that sufficiently great bonding radii are obtained is one, however, the importance of which cannot be exaggerated The specifications for reinforced concrete now in common use do not give enough importance to the advantage of large bending diameters A small bending radius, besides often furthering the appearance of the first visible crack, starts the ultimate failure of a concrete structure at the inside of bends and hooks The few tests available on this feature indicate considerable improvement to be possible when we come to appreciate in practice the advantages of locating the tensile reinforcement along the stress trajectories of a beam

From the structural standpoint, we should rather be concerned with how large bar bends we can obtain and use in our reinforced concrete structures than how sharp bend can be made on a particular reinforcing bar In connection with that question, improvements upon the conventional anchorage hooks and splicing of reinforcing bars should also be sought

Mr H J Gilkey and Mr G C Ernst, *Author's Closure* The authors feel that the discussions offered are constructive and that they constitute a useful supplement to the matter presented in the paper.

Referring to Mr Bradbury's discussion, the authors agree that the term yield point is a misnomer if applied to cold drawn wire Certainly the practical limit of proportionality for such material is but slightly below its ultimate strength Thus the ultimate strength of drawn wire is virtually the equivalent of the yield point for hot rolled bars The fact that a hot rolled bar does not rupture at the tensile yield point is immaterial in a reinforced concrete member because the deformation taken is enough to wreck completely the concrete in the vicinity of the

bar The data on tests across welds are reassuring as regards that phase of the problem

In the almost total absence of published data on the various questions which relate to cold drawn wire, Mr Bradbury's discussion is especially welcome and reassuring Few persons are better qualified to discuss these points and the authors know of no evidence from either service use or tests that tends to discredit cold drawn wire as suitable reinforcement for concrete There remain, nevertheless, certain questions upon which experimental evidence should be procured and made a matter of published record

One point not mentioned by either Mr Bradbury or the authors in their discussion of bond is the fact that drawn wire is used largely as reinforcement for slabs in which shearing stresses are rarely important Since the bond stress is a function of the shear it follows that the bond requirement is much less for long span or slender beams (of which, the slab is an example) than for shorter or stockier beams Moreover, as was mentioned in the paper, the wire is always of small diameter and has a relatively high superficial area available for bond.

Mr Friberg makes an excellent point in the opening paragraph of his discussion in pointing out that the strength of the concrete need not be increased in order to attain distinct economy by using high elastic limit steel with higher design stresses in T beams and also in columns under the recently evolved A C I Code In other words, the benefits from the use of higher steel stresses are available for these members either with or without the additional benefits that accrue from the use of stronger concrete

Mr Friberg also mentions cracking For flexural members it must be recognized that the presence of cracks on the tensile side of a beam is one thing and their detection is quite another Experimenters have usually recorded the loads at which cracks first became visible There was, of course, always pronounced cracking at the yield point of the steel or earlier if anchorage was inadequate Such devices as white washing the beam and microscopic examinations refined the determination and strain gage readings on the concrete gave still earlier indications of the presence of cracks If beams are reinforced with large bars, cracks occur at greater intervals and will be correspondingly wider and more readily subject to detection From observations of ultimate deformations by one of the authors on tensile specimens and beams of plain concrete[2] it is evident from the limiting deformations of which concrete is capable that cracking must be present for stresses in the tensile steel which are well below 10,000 lb per sq in (sometimes as low as 3500 lb per sq in)

[2] Arch Dam Investigation, Vol II, published by Engineering Foundation, May, 1934, pp 502, 504, 550, 552 (54) See also Proc Am Soc C E , Jan 1935, p 133 (58)

That it is only as the steel stresses have exceeded the proportional limit and have approached the yield point, that cracks become wide enough to constitute a hazard either from the standpoint of lack of homogeneity of the beam or corrosion of the steel seems to have been amply demonstrated by the behavior of exposed reinforced concrete structural members in service for many years under a variety of conditions Whether the steel in structures which are subjected to the more severe exposures would continue to be amply safe-guarded under elastic stresses considerably above those which have been used heretofore con-

TABLE 9

PROBABLE DEFLECTIONS, DUE TO DEAD LOAD, LIVE LOAD AND TOTAL LOAD
Supplements Table III, Chap III

No Girders	Stresses			Slab Deflections (Fixed Ends, Center Load)			Girder Deflections Clear spans all 40′		
	f_c	f_s	Clear Span (Ft)	DL Defl	LL Defl	Total Defl	DL Defl	LL Defl	Total Defl
6	650	16000	4 6	0 00066	0 01644	0 0171	0 296	0 299	0 595
	800	20000	3 9	0 00062	0 01578	0 0164	0 370	0 455	0 825
	900	25000	3 9	0 00077	0 01903	0 0198	0 469	0 596	1 065
	1000	30000	3 9	0 00089	0 02211	0 0230	0 531	0 734	1 265
5	800	20000	5 2	0 00192	0 02528	0 0272	0 297	0 358	0 655
	900	25000	5 2	0 00232	0 03048	0 0328	0 384	0 471	0 855
	1000	30000	5 2	0 00280	0 03670	0 0395	0 462	0 578	1 040
4	900	25000	6 2	0 00426	0 04044	0 0447	0 322	0 403	0 725
	1000	30000	6 2	0 00498	0 04712	0 0521	0 416	0 539	0 955
	1200	25000	6 2	0 00517	0 04903	0 0542	0 429	0 571	1 000
	1600	30000	6 2	0 00759	0 06241	0 0700	0 714	1 201	1 915
3	1200	25000	8 6	0 01450	0 08050	0 0950	0 386	0 554	0 940
	1600	30000	8 6	0 01913	0 10637	0 1255	0 496	0 979	1 475

Deflections are computed by Maney's formula (See Chap III) assuming full working stresses to be realized For separating Live and Dead Load Deflections a ratio of Max Live or Dead Load Moment to Max Total Moment was used

stitutes one of the problems which needs further study, as was pointed out in Chapter I As regards the cracks, then, there seems to be little question about their presence on the tensile side of any economically designed flexural member but the question is rather one of how objectionable they might be for given conditions

The comments relative to the deflections listed in Table III are germane and Table IX is appended to cover the points raised The highest of the authors' computed live load deflections are still well above those which Mr Friberg obtained In the computation of the live load moment, the effective portion of the front and rear *axle* loads was deter-

mined by assuming the distribution to each girder to occur as though the slab were a simple beam Upon this basis the live load moments were approximately one-eighth greater than the dead load moments Impact was taken at $33\frac{1}{3}$ per cent of live load moment. In the table containing the deflection due to dead and live load separately, the deflection given for live load is actually that from live load plus impact This accounts for a portion but not for all of the difference noted by Mr Friberg The Maney formula gives high deflections in comparison with some others and the lack of agreement is an added indication of the need for more experimental evidence on the subject of deflections Some of these needed data are now being obtained as a current phase of the High Elastic Steel Investigation

From the data on Figure 9 the live load deflection in no case exceeds $\frac{1}{400}$ of the span length and is much less for most cases, especially for the slabs Such deflections cannot be objectionable from the standpoint of appearance In its relationship to the width of crack opening, however, the total deflection is the one to be considered and there is as yet no adequate basis for such a quantitative consideration

The data on fatigue loading and age-annealing of cold drawn materials at ordinary temperatures are timely

In the emphasis placed upon the need for identifying-marks on bars, the authors and the committee have in their minds a consciousness that bars do get misplaced between the mill and the forms Such scrambling is more likely to be found on small jobs than on large ones and might, in unusual cases, come from a willful substitution on the part of a jobber in an effort to cover up a shortage of the grade of steel ordered Marked bars should go far toward removing the existing consumer-prejudice against having more than one grade of reinforcing steel on the market The fact that bars do get misplaced is not necessarily a reflection upon steel mill practice although the suggested remedy seems to be a steel mill problem

Mrs Pope's comment applies to all grades of reinforcement and emphasizes the need for adequate protection of reinforcing metal, especially when the exposure is severe as for a pavement