What's wrong with load factor design?

A. N. BEAL

Mr Beal

'Limit state approach' has been said to mean many things; the principal innovations concern the treatment of a structure's safety factor. Two basic ways of applying it are identified:

- (a) the 'stress factor' method, represented by the equation $F \leq P/K$;
- (b) the 'load factor' method, represented by
 - (i) $KF \leq P$, or
 - (ii) $k_1F \leq P/k_2$.

There is no difference between the answers produced if both use the same safety factor and analysis.

60. 'Limit state' codes adopt the load factor approach (ii) and bring four innovations:

- (a) Design based on factored loads.
- (b) The safety factor is divided into a series of partial factors.
- (c) Loads and partial factors are intended to be derived from statistics and probability theory.
- (d) Maximum and minimum factors are applied locally to dead load for worst effect (CP110, BS 5628 and Merrison IDR only).

61. About half of the Paper is concerned with item (d). This is for two reasons: firstly, this is the only area where any analytic difference is proposed, and secondly, it is quite selective in the structures it affects—simply supported beams are unaffected, whereas for balanced or continuous structures it can be quite an onerous requirement. A continuous structure is required to carry several different patterns of dead load, rather than one; a balanced structure is made unbalanced. This requires a thorough justification, as continuity and balance have traditionally been regarded as fundamental principles for design which combines economy with safety and elegance.

Mr K. Sriskandan, Department of Transport

This Paper contains many statements and views which, at best, I can only describe as different.

63. The Author talks about working stresses giving way to what is known as limit state theory. A limit state is a performance criterion and relates to a limiting

condition beyond which a structure or element is assumed to become unfit for its purpose. The adoption of a limit state format is in no way dependent on whether a code has a probabilistic or deterministic safety basis. Limit state theory and the probabilistic safety methods have been developed in parallel and have been used together in recent codes of practice.

64. The probabilistic methods referred to in CEB bulletin 112⁶ range from the fully probabilistic methods (level III) to the much simpler partial safety factor format (level I). The partial safety factor format is defined as a design method in which appropriate levels of structural reliability are provided on a structural element or exceptionally on a structural basis by the specification of a number of partial safety factors related to some predefined characteristic values of the basic variable.

65. The partial safety factor format is the one which has been used in all the recent UK codes and incorporates partial factors on loads and on material strength. For some reason the Author has chosen to describe it as load factor approach, and in §9 he outlines what he calls differences in what was intended to be a unified approach.

66. The fundamental equation for unification of safety using the partial safety factor approach is the one described in ISO 2394⁷ and used in volume 1 of the CEB/FIP model code.⁸ This equation is

$$\text{function } [\gamma_{n1}, \gamma_{n2}, \gamma_{f1}, \gamma_{f2}, \gamma_{f3}, \text{ effects of } Q_k] \leq \text{function } [f_k/(\gamma_{m1}, \gamma_{m2})]$$

where Q_k and f_k represent the characteristic loads and strengths respectively. There are seven partial factors (γ) which can be grouped together if required and they allow for the following:

γ_{n1} | —mode and consequence of failure

γ_{n2} |

γ_{f1} | —unfavourable deviation of loads from the characteristic

γ_{f2} | —reduced probability of various loads acting together at their characteristic value

γ_{f3} | —adverse modifications of load effect due to incorrect design assumptions

γ_{m1} | —possible reduction in strength of materials in the structure as compared with the characteristic

γ_{m2} | —possible weaknesses in structure arising from other causes.

67. In all UK codes this fundamental approach has been used consistently. γ_{n1} and γ_{n2} have not been used explicitly because it was considered possible to subsume them in other factors. In the two building codes and the Merrison interim design and workmanship rules there are only two factors, γ_f on loads and γ_m on materials. All of these codes deal with one type of structure and therefore there was no need to have γ_{f2} separate. Additionally, as each code deals with only one material there was no need for γ_{f3} to be separate. Therefore γ_f covers γ_{f1} , γ_{f2} and γ_{f3} . Similarly γ_m covers γ_{m1} and γ_{m2} .

68. However, BS 5400⁹ deals with bridges of different materials. Thus γ_{f1} and γ_{f2} are combined to form γ_{fL} , but γ_{f3} is used separately for each material.

69. So I would say that they are all consistent to the extent that they are derived from the same basic equation and the only differences are the necessary ones. If there has not been any consistent approach on the building side it may be due to the

fact that code committees have not been properly co-ordinated, but there is not anything wrong with the limit state approach.

70. In (b) of §9 the Author says that in CP 110 only one factor is interposed, but paragraph 2.3.3.2 of CP 110 reads: 'When assessing the strength of a structure, or any part or section of a structure, the appropriate values of γ_m are 1.5 for concrete and 1.15 for steel, except ...'

71. There could be a criticism of CP 110 that in the later design rules there is no requirement to use γ_m explicitly—a value is already subsumed in the design tables. This could lead to some confusion in the designer's mind as to whether he should use γ_m or not in each case. However, I have not heard of any problems arising from this.

72. In §5 the Author states the opposition's reason for changing to the partial safety factor format; namely, to treat variability of loads, materials and structural behaviour, each in an independent manner so that improvements in knowledge of these can be incorporated into the codes to lead to more economical designs. However, it should also be noted that if suitable partial factors are adopted, then greater economy will result over the whole family of structures as soon as a change is made from the single global safety factor to the partial factor method. It has been demonstrated that the greater the number of suitable partial factors the nearer are all designs to the target reliability. With only one safety factor as in the stress factor approach, the designs will diverge widely from the target reliability—they will not have a consistent safety margin as the Author states in §4—and this will result in very uneconomical designs.

73. I am not as pessimistic as the Author that we will not learn enough about loads and materials and therefore gain benefit from this knowledge. Research is geared to acquiring this knowledge and the results are promising. There is no doubt in my mind that we must have a design format which will benefit from this improved knowledge. The resulting design process may be slightly longer, but the overall cost—which is what we should be interested in—can be reduced.

Mr C. J. Dewhurst, W. G. Curtin and Partners

As a practising engineer I am basically in agreement with the Author's theme. I am interested in the design of hybrid buildings, where, with the variation of materials, one can have considerable problems in the use of partial safety factors. A house can be constructed in four or more materials, using different partial safety factors—partial factors on the materials and partial factors on the various elements in the design. This can lead to quite a confusing state by the time the engineer has got down to the design of the foundations, when he must come back to a basic stress analysis. I feel, therefore, that the codes of practice should, in the future, be on a stress analysis basis, enabling the engineer to see, as he goes from one material to another, the stresses and loads he is dealing with.

Professor J. Heyman, University of Cambridge

Mr Beal's Paper gives a timely discussion of the assumptions and aims of structural design, and he draws attention to some anomalies inherent in the use of probability concepts. However, his Paper has its own inconsistencies, perhaps arising from a rather imprecise definition of some of the terms.

76. Underlying the discussion there are two fundamental confrontations, that between stress factor design and load factor design on the one hand, and between

Fig. 5. (a) Two-span beam under uniform load; (b) 'elastic' distribution of moments; (c) 'plastic' distribution

elastic design and plastic design on the other. There are cross-links between these two, but these are not always straightforward. For example, most people would think of plastic theory as a load factor design method, but Mr Beal states (correctly, I believe) that plastic theory as applied to steel portal frames is a stress factor approach (§52).

77. Part of the difficulty actually lies in the changing use of words in the last 30 years. What used to be called simple plastic design gradually became known as limit design (particularly in the USA), and was then somewhat naturally assimilated into the philosophy of limit state design. It is this philosophy which, as Mr Beal's Paper shows, can now be seen only rather partially and dimly, as if in a fog.

78. Some light may be thrown if the aims of plastic design methods are once more examined rather closely. Such an examination was made in detail over ten years ago,¹⁰ and the following remarks are based on that analysis. Part (a) of Fig. 5 shows a two-span beam under uniform load, and part (b) gives the 'elastic' distribution of moments. The elastic designer will base the design of the beam on the value of moment ($Wl/8$) at the central prop.

79. However, this value is extraordinarily sensitive to the assumptions made in the analysis. If, for example, there is any differential settlement of the supports to the beam, then the central bending moment ($Wl/8$) may be either reduced or increased.

80. In fact, to ask for a description of the elastic state of a redundant structure is meaningless. If settlements, imperfect fabrication, erection stresses and so on could all be taken into account, then perhaps an 'actual' bending moment distribution

could be calculated. But this calculation would remain valid for only a short time. A shift of load might cause slip in a connection or settlement of a footing; the structure would continue to look much the same, but its elastic behaviour would have altered markedly.

81. By contrast, these practical imperfections have no effect on the single quantity calculated by simple plastic theory, namely the collapse load; the imperfections are 'wiped out' as the material is supposed to pass from the elastic to the plastic range. As plastic theory was developed, it was natural to think in terms of the collapse load of the structure, related to the working load by a load factor, and the theorems are often established in terms of load factors.

82. In fact, the 'plastic' bending moment diagram (part (c) of Fig. 5) is not the collapse distribution, but is supposed to be due to the *working* loads. Both the distributions in the figure are possible equilibrium states for the beam, and either may be used (by the safe theorem of plasticity) as a basis for design. In either case the designer will arrange for the largest bending moment to lead to a stress which does not exceed some permitted value, say 165 N/mm^2 for steel with a yield stress of 250 N/mm^2 .

83. The 'plastic' distribution is of course more economical in design than the 'elastic' distribution, but both are stress factor approaches as defined by Mr Beal in §52. The difference is that the 'plastic' approach gives a consistent safety margin, while the 'elastic' approach does not (and the first sentence of Mr Beal's §4 is incorrect).

84. The safety margin is the *hypothetical* increase that must take place in the loads in order that hypothetical collapse should occur, and the increase is related, in simple plastic theory, to the load factor. Its value was set empirically (at say 1.75 for a conventional framed structure) to give the same design for a simply supported beam whether the approach is elastic or plastic.

85. But the load factor *is* hypothetical, and its use does not imply that the designer envisages his structure as actually in some sense in danger of collapse. This point is well made by Mr Beal (§16); and further, his criticism of the use of probability theory is just. This matter too was discussed about ten years ago in relation to the significance of shakedown calculations.¹¹ The fact is that for some types of loading no real statistical analysis is possible.¹²

86. Mr Beal's difficulties with the multi-span beam (§§39 et seq.) really arise, however, from the use of elastic concepts as much as from the use of probability theory. Patterns of loading do not in general have much, or indeed any, influence on the plastic design of a continuous beam; each span can be designed pretty well in isolation, and the statistical arguments do not apply.

87. The difficulty with the cable-stayed bridge lies in the marriage of stiff and flexible structural elements; the whole structure is very sensitive to small deviations. This is not only a problem of theoretical analysis; it is a problem also of practical construction.

88. Thus Mr Beal's examples do not all substantiate his thesis, and there are pockets of fog that he fails to disperse. But it is clear that deeper thought should be given to the role of probability theory in structural design.

Mr D. D. Croft, Ove Arup and Partners

I am surprised that Mr Beal does not mention what I always thought was the main reason for changing from the permissible stress method to the load factor method,

and that was to cover the problem of non-proportional loading in which the effects of one load oppose those of another. The case most often cited is that of the cooling tower where the tension stresses in the windward face are opposed by the compression stresses due to self-weight. If the wind stresses are nearly balanced by the self-weight stresses then it is clear that any small increase in wind load above the design value due to unforeseen factors without any corresponding increase in vertical load will cause a disproportionate increase in the net tension stress. In order to achieve a safe design, factors must be applied to the individual loads as with the load factor method and not to the net resultant stresses as implied by the permissible stress approach.

Mr M. J. Baker, Imperial College of Science and Technology

I find the Paper of interest, largely because it expresses some different points of view from those of codes of practice committees of the last few years. There is a need for considerably more discussion of the underlying principles in structural codes and this Paper provides one such opportunity.

91. In my opinion the misuse of the term 'limit state design' has caused much confusion. Limit state design is simply a procedure in which there is explicit recognition of a number of limiting design requirements and where action is taken to try to prevent these limiting conditions from occurring. The concept is a powerful one in that it should encourage engineers and their clients to select the relevant performance requirements for each structure and then to design the structure accordingly. Admittedly, some of this freedom of action is restricted by the mandatory status of most codes, but this is not too important provided that design does not become simply an exercise in satisfying the code requirements.

92. In practice, however, limit state design is often associated with a number of other concepts or procedures—for example, the use of partial safety factors as opposed to single safety or load factors, the use of 'characteristic' values, design for an acceptably small probability of failure, structural reliability analysis, etc. This is unfortunate because limit state design does not imply or necessarily require any of these concepts—only suitable calculation models for each limit state (e.g., deflexion, cracking, overturning, plastic collapse). However, many of these concepts are useful and are features of the hierarchy of safety checking methods mentioned by Mr Sriskandan.

93. I would like to suggest that perhaps a better title for the Paper might have been 'What's wrong with partial factor design and in particular with the use of partial factors on loads?'. Although the Author describes what he means by 'load factor design' this is not, in my opinion, commonly accepted usage and the criticism in the Paper is directed mainly at the use of the partial factor system of safety checking (level I).⁸ I do not agree, however, with many of the conclusions reached by the Author. For clarity, Table 1 summarizes the way in which safety factors are introduced into some existing structural codes.

94. In §5, the Author states that the advantage claimed for 'the load factor method' is that it allows the full application of probability theory to structural safety. In fact, this is not a claim that would be made by anyone working in the field of structural safety or reliability analysis. The partial factors that are used in deterministic level I codes may be derived by a process of probabilistic calibration,¹³ but this does not impose any constraint on the number of partial factors in the final code. In general the aim is to have as few partial factors as

Table 1. Use of safety factors in some current structural codes (excluding treatment of load combinations)

Code/standard	Structural analysis	Section design	Safety factors against collapse
BS 449	Elastic or empirical	Elastic	Single factor on stress
	Plastic collapse	Plastic	Single factor on total load
CP 114	Elastic or empirical	Ultimate state stress block	Factors on yield stress and compressive strength
CP 110 BS 5400 Part 4	Elastic (with limited redistribution of moments), empirical or yield-line etc.	Ultimate state stress block	Partial factors on yield stress, compressive strength and on each type of load

possible, and here I support the Author's view that undue code complexity is likely to lead to design error.

95. The Author also criticizes the way in which partial factors for some new codes such as CP 110 have been chosen so that the 'overall factors of safety' remain about the same. I am also critical of such procedures. However, better ways of determining partial factors do exist. At Imperial College we have recently completed the probabilistic calibration of the draft BS 5400 Part 3 for the design of steel bridges. This work, undertaken in conjunction with the Flint and Neill Partnership, has shown¹⁴ that components of bridges designed to the existing standard BS 153 vary by as much as ten orders of magnitude in their notional failure probability, and that by introducing a partial factor format the following can be achieved simultaneously:

- (a) the average safety levels implied by design to BS 153 can be maintained;
- (b) the safety of the least reliable components as designed to BS 153 can be considerably improved;
- (c) average savings of about 6% in the total amount of steel used can be achieved.

This might seem remarkable but is fairly easily demonstrated.

96. The Author argues (§8) that the aim of consistent structural member reliability is perhaps not appropriate for members carrying a high proportion of dead load because such members often have the highest failure consequences. This is true but simple analysis shows that such members are so many orders of magnitude more reliable that they are just uneconomic and are grossly overdesigned by codes such as CP 114, BS 449 and even CP 110. Some change is therefore desirable.

97. Discussion of the merits of partial factor design versus procedures using factored stresses would not be complete without mentioning the problems that arise when some loads act in a resisting or stabilizing capacity. In Fig. 6, the reinforced

Fig. 6. Reinforced concrete column

concrete column ABCD is loaded by an axial force P and a moment M which are assumed to vary independently (e.g., live load and wind load). The column will fail either by developing high tensile stresses at A or by compression at B. For compression at B, the effects of P and M are additive and adequate safety could be provided by a permissible stress type of approach. For tensile failure at A, however, P acts as a resisting variable and M is the only load. In this case, a permissible stress approach is a very poor basis for design, particularly if the actual values of P and M are subject to considerable uncertainty and at the same time their nominal effects are large and almost equal. In this case the only sensible design approach is to have a load reduction factor on P and a load enhancement factor on M . Neglect of this principle was the main cause of the collapse of the Ferrybridge cooling towers in 1966.

98. The Author suggests (§19) that safety factors should be increased to account for 'engineer error' or errors in design and construction. Evidence suggests that this is not a good way to overcome this problem. These errors, which are often the real causes of structural failure, are more effectively counteracted by improved design checking procedures and site inspection. In fact, a reduction in safety factors would result in some savings in material costs which could be more effectively used in checking that what should be there is there. By way of example, Fig. 7 shows

Fig. 7. Slab thickness measurements from a single structure

some of the results of a recent survey¹⁵ involving 272 thickness measurements of nominal 150 mm thick reinforced concrete slabs in a single structure. With a range from 111 mm to 198 mm there is considerable room for improvement.

Dr W. B. Cranston, *Cement and Concrete Association*

Mr Beal makes the point that state codes normally require analysis for only one limit state. He is strictly correct but this statement could be misinterpreted. A good designer will know, out of the various performance criteria that his structure has to obey, which is the most critical. He will design for that critical limit state and then do some simple checks for the others. For some structures, particularly where he is not experienced, he may find that the wrong limit state has been chosen and the design will have to be started again. In my view this is the major advantage of limit state design, because it makes the designer think about the performance criteria relevant to his structure. The permissible stress approach is too often treated just like a recipe—used to cover all the ills of the structure, but without any clear knowledge of how or why.

100. Mr Beal is quite right when he points out that ‘The combination of probability functions is a tricky process’. It is my personal view that for quite a long time to come, certainly another decade, the decisions on partial safety factors for structural codes are going to have to be made by committees on the basis of what they feel is right from experience. Probability theory may well be of assistance, but judgement must be the dominant factor.

101. It follows that one should not see limit state design as necessarily requiring the use of sophisticated probability theory. Many limit states can be dealt with, as stated above, by very simple checks which do not even involve calculations, far less probability calculations.

Mr A. T. H. Swinney, *William Haley and Partners*

Mr Beal has produced a very interesting and thought-provoking Paper which almost has me convinced of his arguments. It is necessary in considering the merits of his case to define our role as engineers and identify our aims. In its simplest form our objective is to produce the most economical structure which reflects the realities of its purpose as defined by the structural configuration, imposed loads and construction materials.

103. An engineer whose work involves the use of several different building materials has indicated his preference for a single all-embracing safety factor. Certainly this can be provided, but at best it will be a rough approximation; at one extreme the resulting design will be ultra-conservative and expensive, while at the other, less reliable materials and uncertain loading conditions may result in a potentially dangerous design.

104. Mr Beal shows that with two load factors for dead loads (viz. 1.4DL and 1.0DL) a continuous beam is designed against a probability of $1/k^2$ or even $1/k^3$, while a simply supported beam is designed against a probability of only $1/k$, and questions the need for the most onerous conditions being imposed on multi-span beams. His conclusion that this does not make engineering sense is correct but this is no argument for abandoning the partial safety factor approach.

105. The answer is not that we require fewer partial safety factors to fulfil our role, but that we need more: factors to cover variations in material properties;

factors to cover variations in load distribution; factors to cover different structural arrangements etc.

106. One of the main arguments put forward in defence of the 'stress factor' approach is simplicity of use and elimination of potential sources of error brought about by the more complex 'load factor' approach. While long numerical operations are certainly a major source of error the advent of the micro-computer will very soon eliminate any need for such operations to be undertaken by hand. Furthermore, such machines will eventually be part of every engineer's equipment (much as modern electronic calculators have all but eliminated the slide-rule) and he will have available to him not only efficient 'number-crunchers', but storage capacity which will provide the appropriate partial safety factors for any given structural problem once it has been defined by the engineer. As an indication of the trend in this direction, I have programmed an HP67 pocket calculator to undertake the design of concrete sections in accordance with clause 3.3.5.1. of CP 110, based on the stress-strain curves of concrete and reinforcing steel, on only two magnetic strips (i.e., in about 500 steps), taking into account all variables. The capacity of a micro-computer is considerably greater than this and the use of multiple partial-safety factors becomes an attractive proposition providing a more rational basis for design.

Mr L. Crystal, Seltrust Engineering Ltd

I think this Paper needed to be written, if only to start a debate as to the value of load factor design. I view this from the point of an older man trained in the methods of Hardy Cross analysis without the benefit of computers, and having to cope with later codes from CP 114 to CP 110, seeing the volume of paperwork increasing and having younger engineers much more competent in that field of analysis who are producing designs that I have to check and approve and then be responsible for.

108. I think that what is wrong with load factor design is not the ideas behind it but the inability of some designers to appreciate what really is the meaning of serviceability limits.

109. Let us take as an example a reinforced concrete slab which the designer assumes is a nominal 150 mm thick and which he probably accepts as being maybe $\pm 3-4$ mm. But in his analysis of the serviceability limit, in which he might even go as far as calculating the width of cracks, he loses sight completely of what can happen on a site. He loses all sensibility as to whether a crack of width $\frac{1}{4}$ mm will really propagate up through the slab, for say 10 mm, and thereby reduce the effective cover, maybe in a corrosive atmosphere. When tackled on this, too often the people concerned produce computations, mini-computers and the rest of it to show that 'there are the calculations; my calculations are right, and therefore the structure is safe.'

110. I think that the serviceability limits, other than the ultimate, are more important. I think that in specifying various load factors and various material safety factors, the code misconstrues the case for the actual construction. There are tolerances which the contractor expects to have and to build in. Those tolerances, which are fairly liberal, cannot be assumed and cannot really be written into a code of practice. In my own company's sphere of work—we are dealing with heavy industrial plants—the earlier engineering designers would take a maximum permissible stress, for example, under BS 449 or BS 153, and then say: 'We will take 85% of these stresses.' Generally, the reason for that was that they did not know for

certain but could guess just what use the operators would make of the plant during its lifetime (which could be 30 years or more). I have known a case of an electric overhead travelling crane, although only designed to lift loads vertically, used to pull something horizontally by means of a block fixed to a stanchion, making the whole building sway.

111. Such situations are not taken care of by codes of practice. They are taken care of by engineers appreciating to what use their designs are put. I feel that the codes of practice describing load factor design, and introducing material factors and limit states, tend to cloud the issue for the young engineer who has not had the practice, has not been out, has not seen what is happening and cannot believe that somebody would literally cause 'wilful damage'. But wilful damage is not as rare as one might like to believe; it is done by operators who think that a building or structure is there for their purpose and it is safe for whatever use they may want to make of it. There must be many engineers who can recount stories of somebody going along with a welding torch and cutting a large hole in a steel girder web to pass a pipe through.

112. The serviceability limit states and their definition in terms of structure use ought to be more carefully looked at; the engineer, with some judgement and guidance from the codes, could then decide what are the loads and load combinations and how he treats them, to produce a safe and efficient structure, capable of a trouble-free, useful life.

Mr D. J. Adler, London Borough of Hammersmith and Fulham

I should like to thank Mr Beal for saying what many of us have thought for ten years, but have been too intimidated to utter. He mentions that structures do not understand safety factors. I shall go further and say that structures do not understand calculations. When we make a calculation for an element or for a whole structure, what we are doing is building on the experience of the past. We learn from our failures.

114. If a structure is standing today, tomorrow and next year, there is no guarantee that it will not collapse the year after. We can draw no conclusion about the method of design other than it *appears* to be satisfactory. But when there is a failure we obtain incontrovertible information. The Merrison rules followed from such a failure. When it happens we develop design methods that account for the failure and check that the structures that have not failed already can continue to stand. Such a method is really a sophisticated 'rule of thumb'. The mathematical analysis is a model-making process, the result of which has a similar relation to the real structure as an architectural model has to the real building.

115. Many engineers like myself spend their time designing everyday building structures. I have seen no reason to change the methods that have proved satisfactory in the past, and which are economical in design time—a factor that is of prime importance to my employers, the ratepayers. It may be that the methods of CP 110 are appropriate to more sophisticated structures of which I have no direct knowledge. My plea is that a code similar to CP 114 be retained for the everyday building structures.

Mr N. C. Train, W. G. Curtin and Partners

The basis of all structural design is the comparison of applied loads and strength of the structure. A factor of safety needs to be incorporated somewhere in this

comparison. Whereas with permissible stress design the comparison of load and structural strength leads to a safe structure, with load factor design such a comparison leads to failure since the safety factor has to be incorporated by the designer. This I believe is a dangerous design approach especially for non-structural specialists such as architects and surveyors who design their own lintels and beams. In order to help designers as a whole, I think it is a far better idea to have the factor of safety incorporated within a permissible stress and then to allow overstress as a positive input on the part of experienced designers.

117. An additional advantage in having a high strength reserve in members with high dead loads (§28) is in present-day building contracts where the superstructure drawings are still being finalized during the contract period. It is comforting to know that the substructures have some reserve of strength in case the superstructure loads start jumping down different loading paths.

118. With respect to Mr Beal's probability development (§42) I looked at what happens to the safety factor on a two-span beam taking a probability of occurrence as $1/k^2$ and using the method and values given by Bennett.¹⁶ For the two independent events coming together the equivalent factor of safety was 3.2. I think this demonstrates that a full probabilistic model for factors of safety fails with continuous structures.

119. My introduction to limit state design was a small project comprising three storeys of flats on ground beams with piles and retaining walls, which meant that I had the unfortunate condition of

- (a) safety factors for dead and live loads and materials from CP 110;
- (b) safety factors for the retaining wall overturning;
- (c) safety factors for the pile design.

Thus I had five separate safety factors for the preliminary design. If I had continued with a full design using BS 5628 for the masonry and also taking account of wind loads, I would have had additional safety factors for the wind loading and (from the masonry code) safety factors for site control and brick ties, which would then have added up to eight separate factors.

120. I have now completed a reinforced concrete frame office block, and in the interests of efficiency I have already started incorporating the factor of safety for materials in the tables in CP 110. So I now talk about a class 25 concrete as having a permissible stress at ultimate loads of 10 N/mm². I also talk in terms of high yield steel as having a permissible stress of 356 N/mm². All I now need to do is to take a firm stand and divide everything through by a factor of 1.5 and I will have a very good stress factor code for concrete.

Dr F. Walley, Consulting Engineer

Mr Beal has not mentioned CP 115. Has he had any difficulty with the limit states or partial factors in that?

Dr B. Simpson, Ove Arup and Partners

I would like to take up a point on the use of different materials together in the same structure or in a combined soil-structure system. Situations can occur, particularly in the ground, where the loads transmitted to one part of a structure depend critically on the strength or other properties (e.g., the density) of quite different materials elsewhere in the structure. For example, if one takes a frame building on

either a piled foundation or individual pad footings for the columns, built on ground suspected to be rather variable, then it is natural—and it is normally done as a matter of engineering judgement—to increase the strength of the pile caps or ground beams, as may be appropriate, so that load can be distributed from one point of support that is not working very well to others. This is not readily done, it seems to me, with a simple load factor or permissible stress approach. We have to take into account the possibility that one part of the structure is not working too well in order to design another part. The use of partial factors applied to each load and to the strengths of each point of support seems to give a rational method for doing this.

123. A similar problem arises in designing retaining walls to resist earth pressures. It is very difficult to choose a suitable safety factor (whether it is expressed as a load factor or a stress factor) for design of a reinforced concrete structure if it is considered in isolation, when the loads or the stresses in that reinforced concrete structure are a function of the soil that it is supporting. The use of partial factors for the soil strength or stiffness, as appropriate, in order to derive the loads on the structure, and comparison of these with the strength of the structure, factored to take account of its variability, seems to provide a rational framework for thought. I am aware that this method is not available by combining any two existing codes, but I think we are in a position to work towards this if we develop the limit state philosophy.

124. Although we cannot use probability theory directly, I think it is the case—and I think Mr Beal demonstrates this—that probability theory can be very useful in helping us to understand how to combine loads. Mr Beal shows that in the current CP 110 the method of using load factors for the design of a continuous beam leads to a nonsense, but he uses probability theory to prove that nonsense. I would suggest that although we may not be able to use probability theory in a numerical form, it can be helpful in showing us how to choose reasonable combinations of loads and factors of safety.

125. It is often stated that the permissible stress approaches tend to give simpler calculations which can be carried out in the design office more efficiently. I think that this may be true for simple structures, but certainly for more complex problems much more time is spent understanding the problem and developing a suitable design method than is required to do the arithmetic of the calculations. I would propose that the limit state approach gives a rational method of design and so is an enormous time-saver, even if it leads to slightly more calculation.

Mr E. Dore, Cement and Concrete Association

I became converted to the limit state approach to design about 15 years ago with the publication of a paper by Rowe *et al.*¹⁷ I am still convinced that it is the right way to progress.

127. Mr Beal's Paper explains clearly that there is no real difference in result between formulae having a partial factor on the left hand side of the equation compared with formulae with partial factors on the right.

128. I was convinced that a limit state approach was necessary because we were sliding down a hill of theoretically reducing strength margins, without a rational basis for deciding the appropriate threshold beyond which the reduction in safety factors should stop. We have to turn to the use of probability theory to some extent. Probability theory has to be moderated by common-sense observations of how

structures become defective or fail. Mr Beal is right to complain that partial factors in CP 110 are not precise in their intentions, are not adequately subdivided and do not cover the abnormal situation. When the Institution of Structural Engineers wrote its report on the criteria for structural adequacy, it had some difficulty in defining appropriate standards for the accidental and unforeseeable category of loads. This underlines the fact that certain things cannot be dealt with probabilistically.

129. So I have read the Paper agreeing heartily with that half of it where the Author criticizes mere details of CP 110, such as load patterns, but not with the other half which advocates a return to permissible stress design.

130. On the question of difficulties and dangers of converting loads from an ultimate state design structure to a working state design foundation or the design of hybrid structures using a mixture of codes, I think this is merely a temporary problem. Surely when soils are tested they are taken to the limit of acceptable strain or failure and therefore foundation design could as easily be based on load factor methods compatible with the structure.

131. The Paper is a useful criticism of details of the code but does not present any reason for abandoning load factors or limit states.

Dr S. R. Cochrane, The Queen's University of Belfast

Mr Beal's Paper forces us to examine why widespread confusion should exist in the civil engineering profession over the requirements of the new probabilistic codes of practice and British Standards such as CP 110 and BS 5337. Mr Beal has usefully seized on the factoring up and down recommendations of CP 110 which must surely be a recipe for disaster sooner or later. Unfortunately he then generalizes much too freely regarding the probabilistic nature of material strengths, loads, load-span combinations and indeed crack control in water-retaining structures.

133. Rather than debate the validity of Mr Beal's statements one by one, it might be useful to discuss the different types of probability to which consideration is given in the most recent codes and British Standards, and examine their influence on the factoring process which Mr Beal rightly finds unsatisfactory.

134. The specification of a material's characteristic strength hinges, in CP 110, on the assumption that the material's quality is defined by an average or mean cube strength and the standard deviation of the cube strengths obtained, and that the cube strengths are normally distributed about the cube strength mean value. The choice of 95% or 99% certainty that a cube strength will not be below the chosen characteristic strength depends on the designer's requirements as to a chosen certainty of the sufficiency of his designs. Therein lies a philosophical problem for many engineers, in that they may well feel they are entitled to 100% certainty of sufficiency. Unfortunately the assumption of a normal or Gaussian distribution of cube strengths about the mean strength precludes that possibility. The assumption of a normal probability distribution implies that there is some possibility or probability of cubes being nearly infinitely strong or nearly infinitely weak. Practice and common sense would seem to refute these possibilities, given adequate supervision, and most engineers would prefer an absolute range of cube strengths to consider, such that a minimum cube strength could again be identified. This would appear to be entirely reasonable and there seems no valid reason for continuing with such a theoretical concept as the normal probability distribution when other much more suitable distributions could be used.

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Hong Kong Mass Transit Railway Modified Initial System: system planning and multi-contract procedures

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This Paper describes the detailed planning of the Modified Initial System of the Hong Kong Mass Transit Railway including development of the plan for its implementation and the selection of the form of contract. It also deals briefly with the cost-estimating and the verification of the estimates which led to the final decision to proceed with construction. The Paper sets the background for the three companion papers which describe the detailed structural design and construction of the railway.

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One does not have to agree with everything in this Paper to recognize its timeliness and value in calling for radical debate. Before the profession is irrevocably committed to what still seems to many a great and mystifying departure from tradition, a much wider scatter of informed opinion must be heard.

143. The Paper is concerned only with the load factor approach of new codes, and many will feel that, at some points, the Author overstates his case. But the debate should also concern itself with the limit state principle, which is really a distinct idea.

144. The problem with a serviceability limit state is that its definition is arbitrary, and not really different from the basis of traditional codes. The designer is asked to make what seems to him a large departure in thinking and format, for no result.

145. The problems with an ultimate limit state are that it can rarely be applied to a whole structure, which must in practice be considered member by member, and that in relation to an easy case such as a simply supported beam, where it would be valid, it is unnecessary after dealing with serviceability.

146. Undergirding everything in the new approach is the attempt to evaluate safety factors from first principles. It is no criticism of the amazingly impressive efforts that have been made, to say that the task is impossible. Apart from a few special types, structures are too diverse for too many reasons, and too much affected by too wide a variety of accidental influences, for statistical study to lead to a better result than experience. It will always be true in structural engineering that the only certainties are past successes and failures.

147. The structural industry can have only one good reason for changing its ways: economic pay-off. To impose new codes instead of revising old ones, without promise of profit, is to fiddle while Rome burns.

148. The purpose of these comments is not to pre-empt the conclusion. Perhaps there are good answers to all of them; but many people concerned with structural engineering still have not heard and really understood some of the answers, and need the opportunity to do so.

Mr J. E. Gray, Mara Institute of Technology, Malaysia

This Paper needs to be read in conjunction with various formal efforts to improve the load factor method, for example the report of the Code Servicing Panel.¹⁸ It is there emphasized that the factors which affect the final decision about the difference between allowable and ultimate load *or* stress can be subdivided into those relating to forces, to consequences of failure, and to materials, together with a consideration of the reliability of any equations used and the difference between the behaviour of assemblies of elements and separate elements.

150. Among the factors relating to forces is mentioned the 'reduced probability of the most severe effects of two or more transient loads occurring simultaneously'. According to Mr Beal this is not sufficiently taken into account in the existing code, and he demonstrates this lucidly. He is to be congratulated on helping to keep engineers thinking about the nature of factors of safety.

151. One result of all this thinking is that it is obvious that no structure can be summed up by one number (load or stress factor). To begin with there must always be consideration of the 'routine service' condition (often described by deflexion) and the ultimate or 'failure' condition. Sometimes one or other of these will

dominate to the extent that formal calculations are unnecessary to check the second.

152. The Servicing Panel distinguishes 'run of mill' or simple and repetitive structures where a number of the factors mentioned above could be 'lumped' together. In the extreme this could leave one number for working conditions and a smaller one for failure. It seems to me that whether this is called a stress factor or a load factor is a barren argument. Anyone who uses it should appreciate that it includes allowance for uncertainties in both load and stress, as well as some notion of what probability of failure can be accepted for the particular type of structure.

153. For more elaborate structures it becomes worthwhile to consider each component of the safety factor, but remembering, as Mr Beal points out, the well known statistical property that if such factors are multiplied, the probability of events they are designed to prevent reduces. Such consideration seems a reasonable effort compared with the other stages in design, and a proper development in engineering practice. It is not restricted to steel and concrete structures, but applies to all civil engineering work; for example, earth dams and hydraulic channels. The report of the ICE Working Party on Floods and Reservoir Safety¹⁹ indicates such thinking, as its Table 1 gives various proportions of 'probable maximum flood' to be taken according to the consequences of overtopping.

154. Furthermore, the subdivision of a safety factor into partial factors, each designed to restrict probability of some event, has parallels in management games theory. Such logic would help to avoid the obvious error of assuming the same probability for all events to occur simultaneously as for one of them. In the older codes a similar object was achieved by allowing a lower (combined) factor of safety for combinations of live loads. In the extreme, the factor of safety for any conceivable coincidence of unfavourable events must exceed 1.0, and the question of multiplication of partial factors does not then arise; they are all 1.

155. No set of numbers by itself can sum up the safety of a structure. Humility and good engineering require defensive measures for when the 'unthinkable' happens: the fuse in an electrical circuit, the fuse plug in a reservoir.^{20,21} De Mello²¹ calls this the 'design choice of a universe', thus emphasizing one of the designer's responsibilities. In the case of building structures, such a choice would preclude card-house failures of Ronan Point type, for example. Disastrous failure would be localized and if possible slow, to allow evacuation or other defensive measure.

Mr G. F. Horner, Husband and Co., Basrah

I think we must be careful to separate the shortcomings which we believe are due to the 'load factor' approach from those associated either with the limit state philosophy, or even with the drafting of design codes in general. For example, the problem of age factor on timber stresses could be as easily covered by varying materials factors as by varying allowable stresses, and fatigue is perhaps better dealt with by the statistical approach which goes hand in hand with load factor than by other methods.

157. Presumably, the Author's objection is principally to the enforced use of a number of specified load factors, rather than to load factors *per se*. The idea of a factor of safety applied to a load as opposed to a stress is not uncommon, even when working to the old codes. Which of us has not added 10% or 20% or more to a loading when we were unsure about the exact value, or when failure of the part would have been particularly embarrassing? This is the basis of 'load factor'.

Indeed, we may still add a bit for safety, even now, if we are at all concerned about any of the factors listed in §23.

158. When discussing the causes of structural failure (16), the Author significantly ignores a fourth consideration in the choice of a load factor, namely 'the importance of the limit state being considered' (CP 110). The case of failure by 'statistical' variations of load is indeed rare, but this is surely due to the attention which has been given to design for structural strength in the past, and we have been right to do this. Failure of one part of a structure due to such 'statistical' variation would, I suggest, cause deeper concern to the owner and designer than failure due to exceptional circumstances. We may not even be aware of the cause of failure, and would certainly fear for the safety of similar parts elsewhere in the structure. We can hardly give this aspect too much emphasis.

159. Once the loads get outside the normal range, it is not clear that stress factor can cope any better than load factor.

160. While considering the event to be applied to a statistical analysis, variation in the sequence of construction should be added to the factors given by the Author. (It can be impractical to maintain support to all spans of a continuous beam, say, until the application of full dead and superimposed dead loads.) With all these variables, it is prudent to assume that the load per unit length or area may vary from point to point in such a way that the probability of it exceeding a given value is the same at any position, and has little or no correlation with the load at an adjacent position. The choice of the loaded length is then at the discretion of the designer (or rather, the code-writer).

161. The Author's discussion of dead load, then, requires closer examination. He rightly points out some woolly thinking on characteristic dead loads: the dead loads referred to in CP 3 are clearly average or expected values for the weights of materials, and not those with a 5% probability of being exceeded. To compensate, CP 110 for example applies a general factor of safety of 1.2, and the expected range of values of dead load is taken as $\pm 0.2 G_k$. This implies that the probability of the load at any point exceeding $1.4G_k$ is the same as the probability of its being less than $1.0G_k$. For any structure, the probability of $1.4G_k$ occurring everywhere is the same as the probability of any combination of loads of $1.4G_k$ and $1.0G_k$, be they distributed randomly, span by span, or according to the shape of the influence line.

162. If it is accepted that CP 110 and CP 114 give broadly similar results, we may say that designing to CP 114 corresponds to applying a γ_F of between 1.4 and 1.6 to the total applied loads, and we may thus compare the beam of Fig. 4 designed to the two codes in turn, for dead load only.

163. The loading given in Fig. 4 produces a sagging moment in the centre span of $0.08DL \times \text{span}$. The probability of this occurring is $1/k^5$. Under CP 114 the corresponding moment is $0.046DL \times \text{span} \times \gamma_F$; i.e., somewhere in the range $0.064\text{--}0.074 \times DL \times \text{span}$. The probability of this loading occurring is clearly $\leq 1/k^5$, and yet we have designed a weaker beam by 7–20%. In this case, and in general, CP 114 surpasses CP 110 in failing to design for 'the worst possible load with its own probability of occurrence'. Moreover the Author's suggested amendment on dead load stresses only makes matters worse for the stress-factor-designed beam.

164. If the above statistical argument is not wildly in error, it appears the Author has not been successful in criticizing load factor on its own ground; neither, more importantly, has he demonstrated any superiority of stress factor, apart from its acknowledged simplicity in use.

Mr S. Revess, Stephen Revess Associates

The Paper is very timely. The absurdity of selected worse possible loading on continuous beams is amply demonstrated by the Author's probability considerations of $1/k^2$ against $1/k$ for a simply supported case. I have used CP 110 to design a multi-storey car park and was dismayed by the lack of reality of loading pattern in relation to the parking layout.

166. The separation of live and dead loads in load factor methods does have a distinct advantage, however. It is very useful to investigate the stresses and corresponding deformation in various elements of the superstructure under dead load only with full allowance for time-dependent creep. This will give a realistic figure for cambers and soil settlement. Live loads in many classes of structure are of short duration and only a small fraction of the maximum allowed. In this way it is found that simply supported reinforced concrete roof beams need provision for significant deflection and that single-storey steel portal frames for warehousing on flexible raft slabs will not suffer from appreciable ground deformation even on poor subsoil. These are merely spin-off benefits but there are many other similar examples.

167. The essence of good practice is to give the designers a feeling for the reality of structural behaviour. Engineers must have a command of knowledge in this area, as well as being able to juggle with computations.

Mr A. P. Fawcett, Department of Architecture, University of Nottingham

There is a distinction of fundamental importance between the design process as a whole (with consideration of the several limit states) and the particular analytical methods employed for individual criteria of performance. Mr Beal appears to be confusing the latter with the former when he dismisses the new generation of codes on account of their 'load factor' basis. It is quite possible to justify the limit states of performance using 'stress factor' methods, but traditionally this has not been the case. On the other hand it is possible, and indeed common in present practice, for many of the performance criteria to be ignored in a supposedly limit state design procedure.

169. Mr Beal is mistaken in supposing that probability theory is the new '*enfant terrible*' of structural analysis. If he examines the basis of CP 112, for example,²² he will find that the variability of timber is accounted for in probabilistic terms. Timber, with its natural variability and its relatively low modulus of elasticity, is surely a prime candidate for limit state methods.

170. But Mr Beal's contribution is most valuable in highlighting the irrationality of many aspects of the new codes. One is hard put, for instance, to justify to innocent undergraduate minds the possibility of water achieving a density of 1600 kg/m^3 , as BS 5337 would have it. The inconsistent probabilities of failure demonstrated by Mr Beal, from consideration of dead load alone, necessitate a major rethinking of the partial safety factors. When it comes to serviceability—the source of most problems as the Author and others before him²³ have stated—the situation is yet worse. The CIRIA report on the rationalization of safety and serviceability factors,¹³ despite its title, did not examine the latter at all.

171. Variability of loads, materials and constructions, sometimes predictable, sometimes not, is a fact of life. To me it is a matter of extreme regret that the design philosophy which best frames the problem, namely the limit state approach, should

be getting confused with particular analytical procedures in documents which achieve statutory effect.

Mr Beal

Several of the contributors query the terminology and definitions used in the Paper. When it was being written, this was recognized as a problem—not only are various different terms in use, but they are given different meanings by different people. There are even different ideas as to what is meant by limit state approach. Rather than discuss these, the approach taken was to adopt simple terms which seemed appropriate to the points at issue and to define the sense in which they were being used. Thus ‘stress factor’ was taken to mean the application of the safety factor to permitted stresses, using unfactored loads, and ‘load factor’ was used for design with factored loads. The definitions of limit state approach given in the codes of practice are rather unspecific and so argument concentrated on the specific changes in design method introduced—the four points listed in §60.

173. **Mr Sriskandan** suggests that, contrary to §9, there is no inconsistency between the limit state codes, as they all derive their design formulae from the same basic equation. However, the design formulae of stress factor codes such as CP 114 and BS 449 can also be derived from this equation, by combining all the partial factors into γ_m , thus allowing the conclusion that *all* codes of practice share a common and consistent approach! In §9 it is pointed out that each of the codes requires a different form of calculation from the designer. In CP 110, the quoted permissible stresses already incorporate the materials factor γ_m and so the designer need not normally use it (unlike BS 5628). It should be remembered that the safety and economy of the structure depend only on the overall factor of safety which results from the calculations, regardless of the number of partial factors used in its derivation—they do not exist in the structure as built and are only an artifice for calculation.

174. The problem of hybrid structures raised by **Mr Dewhurst** is a serious one. Engineering common sense suggests that it is a bad idea to mix limit state and working stress methods in the same structure, yet almost all structures are hybrid to some extent. This means that it would be wrong to phase out CP 114 before limit state codes compatible with CP 110 are available and accepted for all other materials likely to be used in conjunction with concrete.

175. **Professor Heyman** raises the question of plastic design. The Paper’s conclusions apply for both elastic and plastic design—the designer can choose which of these is appropriate for the material or section being used. The patterns of loading investigated in §§39–46 do indeed cease to be relevant under plastic conditions; thus, if plastic design is accepted for concrete beams, the CP 110 dead load treatment must have a verdict of ‘irrelevant’ added to those of ‘illogical’ and ‘inconsistent’ reached in the Paper. Care must certainly be taken over the sensitivity of the cable-stayed bridge to small deviations; if the Merrison IDR dead load treatment is applied, the effect is very onerous and gives a very uneconomical design, ruling out the cable-stayed bridge in many cases. Yet experience suggests that the problem can be dealt with in practice—cable-stayed bridges designed to stress factor codes are well established as economical and safe in service. This confirms the Paper’s conclusion that the CP 110/IDR dead load approach is illogical and unnecessary and may be the reason for it being dropped from BS 5400. It is unfortunate that strengthening work required on some bridges to meet IDR requirements may only have been needed to sustain a loading which is not relevant.

176. **Mr Croft** suggests that the problem of overturning is the main reason for the change to the load factor method. If this is so, then a load factor code for steel buildings is unnecessary, as BS 449 already requires a stability check (clause 10b). It is surprising that CP 114 does not deal with the problem of stability as it is not a recent discovery—checking of overturning probably pre-dates St Venant and the Engineer's Theory of Bending—and it is clear that the omission should be rectified. The problem is rather different from the ordinary strength calculation, where loads are resisted by a material's tensile or compressive properties: in the overturning problem, at least part of the resistance is provided by the force of gravity. As the problem is different, so the relevant calculations and safety factors may also be different, and thus the fact that it must be separately dealt with in a stress factor code may turn out to be an advantage, focusing the designer's attention on a special problem and suggesting safety factors suitable for it rather than combining those specified with ordinary beam calculations in mind. The calculation is rather more awkward for a hybrid member such as a reinforced concrete column but it can still be done in quite a simple fashion. (If anyone is interested in revising CP 114, I would be pleased to suggest a suitable method.)

177. **Mr Baker's** statement that the probabilistic calibration of partial safety factors does not impose any constraint on their numbers seems to contradict Mr Sriskandan, who states that higher numbers of factors lead to more consistent reliability. His account of the probabilistic calibration of BS 5400 Part 3 is useful, although, as pointed out in the Paper, it is wrong to think of reliability calculations as the sole arbiter of safety factors—the complex of known and unknown effects covered by 'engineering judgement' must still be considered.

178. Detailed comment on BS 5400 Part 3 must await its publication, as must full examination of the claims made for its system of partial safety factors, but certain points of concern are clear in the current draft and in other (published) Parts of the code.

179. Firstly, BS 5400 Part 3 reverts to a 'two-factor' format, incorporating the 'analysis factor' of other Parts into the materials factor, thus compounding the differences in method criticized in the Paper (§9).

180. Secondly, comparison with other codes of practice is made difficult because, in BS 5400, some loads must be factored for checks under service conditions—class HA live load has a factor of 1.2 applied and a factor of 1.2 is also applied to superimposed dead load. In all other codes, nominal design loads are estimates of worst likely service loads and thus correct for checks on serviceability. As they are considered suitable for neither service nor ultimate conditions, the status of these nominal design loads in BS 5400 is not clear and begs the question of whether partial safety factor values are being adjusted to cover for changes or deficiencies in the specified loadings. If what is being proposed is an increase in HA loading of 20% (to cover for increased lorry weights?), then this would be much better dealt with in the loading specification; similarly, if the suggestion is that designers should add an allowance of 20% to superimposed dead load estimates to provide some margin against future alterations, this too would be more logically incorporated in the loading specification. If such a confusion does exist, then designers will be unable to deal logically with, for example, a form of live load not defined in the code, and the loading specification will be incompatible with any code other than BS 5400.

181. If these factors are removed to the loading specification, the factor for ultimate HA load will be reduced from 1.5 to 1.2 and that for superimposed dead

load from 1.75 to 1.4; the factor for steel dead load would remain at 1.05 and that for concrete dead load at 1.15. If these are then combined with the relevant material factor for a steel beam in bending (1.35), the theoretical range of safety factors is 1.42–1.89, with a likely practical range of 1.5–1.7. The comparable figures for concrete bridges, including the factor on reinforcement strength (1.15) and analysis factor (1.1–1.15) are a theoretical range of 1.45–1.85 and a likely practical range of 1.5–1.7.

182. If these inferences are correct, then I would strongly urge revision of the code—otherwise the confusion caused may well lead to the ‘fog’ referred to by Professor Heyman becoming completely impenetrable.

183. Thirdly: the above may be rather involved, but it must be resolved before Mr Baker’s claims of the virtues of the partial factor system of BS 5400 Part 3 can be examined, and comparison can be made with stress factor approaches. If the above inferences are correct and the loading specification is revised accordingly, then a comparison of BS 5400 Part 3 and BS 153 would show the BS 5400 overall factor of 1.5–1.7 (approx.) comparing with the BS 153 factor of 1.7. The variation of the safety factor in BS 5400 is mainly governed by the proportion of the element load which is dead load; a clause of the form suggested in the Paper (§29) would yield almost identical results in practice if inserted in BS 153. Such a clause might well be appropriate to steel design but it would probably not be appropriate for reinforced concrete because of possible serviceability problems (§28(c)).

184. Fourthly, Mr Baker notes the Paper’s argument in favour of maintaining high apparent safety factors on dead load. It is to be hoped that the other problems of defining safety factors referred to in §§23 and 26 have also been considered in the preparation of BS 5400.

185. Leaving aside the particular problems of BS 5400, Mr Baker mentions the problem of overturning, also raised by Mr Croft. It is hard to understand why a flaw in CP 114 which led to a serious collapse as long ago as 1966 has still not been corrected. With regard to ‘engineer error’, the problem is that, if it is not considered, the result may be a code of practice which can be shown to improve theoretical reliability greatly, yet which leads to a substantial increase in the failures which actually occur.

186. **Dr Cranston** values the recognition in limit state codes of a variety of different performance criteria which a structure must satisfy. Although the limit state codes formalize this somewhat, it has always been recognized and stress factor codes generally include provisions for the various problems of serviceability which may still arise in spite of adequate strength provision; the principal complaint about limit state codes is not this requirement, but the format of the basic strength calculation. If code committees ever do choose to define the safety factors of the limit state codes purely in terms of probability theory, they must face the fact that (if this Paper’s conclusions are correct) some of the answers they arrive at will be wrong.

187. **Mr Swinney** suggests that, for best results, there should be not fewer partial factors but more, with the microcomputer solving the complication of the arithmetic. The arguments about numbers of partial factors have already been considered but no mention was made in the Paper of computers. There are many different opinions on the value of computers (I use a slide rule) but in all cases it is true that the answers a computer gives are only as good as the information it is given—thus the Paper’s conclusions are intended to stand whether or not a computer is used. Unless the object of the exercise is to sell computers to engineers

who are unwilling to buy, codes must still remain as simple as possible and any increase in complexity must be fully justified in engineering terms.

188. **Mr Crystal** makes well the case for giving more attention to serviceability and his remarks about possible alterations or even 'wilful damage' should be heeded by us all.

189. **Mr Adler** makes a plea for a simple code for simpler structures which do not merit the elaboration of CP 110. The irony is that, as argued in the Paper, the CP 110 treatment of dead load produces highly illogical results when applied to complex structures. Thus if any concrete code should be restricted to simple structures, it is CP 110!

190. **Mr Train** makes clear the problems the proliferation of partial safety factors brings to practical design. I am sure his 'working stress' version of CP 110 will prove very popular.

191. **Dr Walley** points out that CP 115 (the code for prestressed concrete in buildings) has always required calculations for both factored and unfactored loads. Factored loads are required in CP 115 because, for pre-loaded elements (e.g., prestressed concrete), calculation of behaviour under working loads will not guarantee ultimate strength and vice versa—thus both must be checked. However, while factored load calculations are unavoidable for prestressed members, this has little bearing on the case for or against factored load design for steel, reinforced concrete or timber members.

192. It is difficult to see how the variation of foundation loads outlined by **Dr Simpson** could ever be fixed and defined in a code of practice—it is inevitably a matter of judgement. The maximum and minimum load factors given in present limit state codes have not been derived with this in mind and are not necessarily appropriate to it; indeed if the loads reaching the foundation already contain a variety of load factors, this will only make the problem more difficult to sort out.

193. **Dr Simpson** also points out that, although the Paper is critical of a strict reliance on probability theory, it uses probability theory to demonstrate the nonsense of the treatment of dead load in CP 110, BS 5628 and IDR. The object of that particular argument was to show that, irrespective of their individual merits, the two are incompatible—both cannot logically coexist in a code of practice. If we look at the dead load analysis from other points of view, it seems to have little to commend it: it brings complication, yet analyses a problem which seems unlikely to occur and which in any case ceases to exist under plastic conditions—all that is required is an adequate scope for redistribution within the structure. However, without it, it would be abundantly clear that CP 110 produces results little different from CP 114. As for probability theory, while it can certainly be useful in interpreting statistics on variations of loads, dimensions and strengths, it can only take its place alongside the other influences and effects which must be considered and it must always rank second to engineering judgement and careful thought.

194. **Mr Dore** claims that the problem of mixing approaches within a hybrid structure is a temporary one. Judging from our experience with CP 110, it is interesting to speculate how long it will take for engineers to agree on mutually consistent load factor design methods for all materials—concrete, steel, timber, masonry, soil and rock.

195. **Mr Horner** suggests that the 'load duration' factor on timber stresses could be dealt with by varying the materials factor. While it is clear that this would be the correct way for a 'limit state' timber code to proceed, the resulting code would be very complicated to use and it would bring extra confusion to the partial

factor format, with a 'load' effect appearing on the 'strength' side of the equation. The objection in the Paper is to the use of a factor on load to provide part or all of an element's reserve of strength; when the basic reserve of strength is provided for in the quoted permissible stresses, the designer is free to think clearly about likely maximum loads without confusion as to the relative status of maximum design loads (unfactored) and supposed ultimate loads (factored). Variations in the sequence of construction obviously complicate the dead load analysis, particularly as the designer may not know the sequence to be adopted at the time of designing the structure. It is not generally a serious problem in practice, because it does not affect statically determinate structures, and redundant structures normally have sufficient redistributive capacity to absorb the effects.

196. The CP 110 range of dead load factor from 1.0 to 1.4 has been interpreted in various ways; Mr Horner's interpretation offers the curious feature of a loading whose 'mean' value is 1.2 times as great as its 'characteristic' value. He points out that if the CP 114 loading is applied to the five-span beam in Fig. 4 the result is a probability of $1/k^5$ for a beam which is 7–20% weaker than the CP 110 section. However, rather than refuting the argument of the Paper, this actually confirms it: no matter how the span-by-span variation of dead load is applied to the beam, the result is a probability of $1/k^5$, although it has already been stated that the required level is $1/k$ (§39). The problem is inherent to the definition of dead load variation as being 'span-by-span' and this approach is thus incompatible with probabilistic design which aims at consistent results; as argued in the Paper (§§32–38), this is in any case not a logical definition of dead load variation. Returning to the beam designed to CP 114, there is no cause for worry: not only is the beam adequate for its own design loading (consistent dead load) but, with redistribution of moments, it would also sustain the loading shown in Fig. 4. It would thus be amply safe, while showing an economy of up to 20% compared with CP 110.

197. The wide variety of contributions and the level of interest shown is very encouraging and demonstrates the lively attention paid by engineers to the fundamentals of structural design. It seems to be accepted by most contributors that, in several respects, the limit state codes are unsatisfactory as they stand and also that statistical probability calculations must always be subordinated to engineering experience and judgement. The practical advantages of the stress factor approach have not been challenged and no insoluble theoretical weakness in it has been exposed. Clearly the lack of a requirement to check stability in CP 114 must be corrected as a matter of urgency, but this should pose no fundamental difficulty—BS 449 is satisfactory in this respect. Thus the conclusion reached in the Paper remains valid—the load factor method does not show the theoretical superiority it would require to offset its practical drawbacks and thus codes should revert to the traditional stress factor method.

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