
Report on a new approach for design loads for buildings

Foreword

This Published Document has been prepared under the direction of the Civil Engineering and Building Structures Standards Policy Committee by subcommittee CSB/54/3 of Technical Committee CSB/54. It advances work on the evaluation of partial safety factors for loads started by other research groups of which the most recent was the Code Servicing Panel of the Institution of Structural Engineers on the Rationalisation of γ -factors.

The use of partial safety factors for loads (γ_f) and the values assigned to them as part of the limit state format of design was first pioneered in the UK by the structural concrete code. Values of partial safety factors for load were chosen to suit the typical dead to imposed load ratios for concrete, and produced design outputs which generally matched permissible stress design outputs.

Limit state format was later also adopted by structural codes for other materials. However, the same factor values for concrete when applied to structures of lighter self-weights and dead loads generally produced poorly matched design outputs. The mismatch has either been reluctantly accepted or improved by unilaterally altering the load factors to suit the particular structural material. Technical Committee CSB/54 considers that as loading is generally independent of the structural material used, it should be covered only in loading codes and not specified or modified in individual structural material codes. It is hoped that this report will refocus the consideration of loading by engineers in this direction.

Current limit state structural codes specify single values of partial safety factors for dead and imposed loads for particular load combinations. This ignores the very obviously different variabilities for varying types of dead or imposed load, such as hydrostatic load contrasted with floor imposed load, which should be associated with appropriately different load factor values. More use can be made of consensus data based on the experience of engineers to identify the variability for at least the commonly used load types. This is more useful and logical than the present situation, and more practical than expecting to derive true characteristic load values from statistically based data, which for well known practical reasons continue to elude engineers. To this end, the report suggests that instead of

the system of partial load factors associated with characteristic loads, a prescription of up to four design values of each load type for direct use may be more appropriate.

It is hoped therefore that this report will be widely debated with a view to influencing the development of improved loading formats for structural design. Any changes which may find agreement will take some time to implement. Many of the missing details indicated in the report will need the consensus of practising engineers, whose comments Technical Committee CSB/54 would greatly welcome. It is anticipated that this report will be a useful UK contribution to the drafting of the Eurocode on Actions which is now in progress.

It is recognized that there may be research and development on the subject being undertaken elsewhere in the UK and overseas and consequently readers are invited to forward any comments or recommendations on any such data to the Secretary of BSI Technical Committee CSB/54, 2 Park Street, London, W1A 2BS.

The information and recommendations contained in the report are aimed at civil and structural engineers, architects, central government departments, local government authorities, professional institutions, research institutes, BSI code drafting committees and the construction industry.

The full list of organizations which have taken part in the work of the Technical Committee is given on the back cover. The Chairman of the Committee is Dr W W L Chan and the following people have made a particular contribution in the drafting of this Published Document.

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Section 1. General

0 Introduction

The Institution of Structural Engineers defines structural engineering as 'the science and art of designing and making, with economy and elegance, buildings, bridges, frameworks, and other similar structures so that they can safely resist the forces to which they may be subjected'. To carry out this task properly, a structural engineer requires information on the materials from which the structure will be manufactured, information on the loads which the structure will have to carry and information on how the structure will withstand the forces.

Even in today's advanced technological world the structural engineer cannot obtain exact information on any of these three topics. To complicate matters further every structure which is designed and erected is unique; even if only because no two sites are identical.

To overcome this lack of information an engineer introduces safety factors into his calculations. These safety factors alter the design calculation in one or more of three principal ways. They may effectively reduce the specified strength of the material, they may effectively increase the maximum loads which the structure is required to withstand or they may reduce the calculated carrying capacity of the elements in the structure.

Engineers long ago learnt that although individual structures were unique there were sufficient similarities, e.g. types of structural element construction material, for general rules to be produced for certain types of structure. These rules eventually led to the writing of codes of recommended practice which are the primary source of guidance for most structural engineers today. However, as knowledge increased and the codes of practice were refined it was realized that there were often different ways of obtaining similar results. In recent years there has been as much argument over these alternative ways as there has been over the technical information. Much of this argument has been over the manner in which the safety factors are introduced into the design equations and the form that the design equations should take.

This report looks at one particular aspect of lack of information in the design process, that concerning the loads which the structure has to be designed to withstand, and proposes a new treatment aimed at clarifying this aspect for structural engineers. In doing this it has also been necessary to consider the context in which this new treatment should be used.

The principal recommendation of the report is that the loading code should specify design values directly. Generally there is no unique relationship between loads required for ultimate (safety) limit state calculations and serviceability limit state calculations and it is suggested that separate values should be specified for each. The report also gives a recommended treatment for load combinations.

The report is structured into four sections. Section 1 gives a general introduction, provides a brief historical review of structural safety and considers the type of loading information required for structural design. Section 2 defines the limit states and the overall treatment of uncertainty with which the later proposals conform; it also briefly considers probabilities of failure. Section 3 defines the different

design values required for use in structural design, indicates how they might be derived for the different types of load and develops the load combination rules. Section 4 considers the manner in which the design values could be given in the loading code and compares the proposed method with existing practice.

1 Scope

This Published Document is a report on how partial safety factors for loads could be treated in BS 6399 to give a common basis for design, irrespective of the construction material used.

Previously, for limit state design, the design value has been defined as the representative (or characteristic) value multiplied by the appropriate partial safety factor. However, in many cases use of a single characteristic value to determine several values for use in different load combinations is unrealistic.

This report by the subcommittee recommends that instead of including partial safety factors for loads in BS 6399 design values should be given for direct use. These design values are assumed to include all material independent uncertainty associated with the loads. Different design values will be required for different limit states and for different load cases within certain limit states. This implies that BS 6399 should specify several design values for each load: the report shows that a maximum of four are required. One of the design values may be appropriate for use in permissible stress design, where this is still used.

It is recognized that there may be occasions when designers will need to know what allowance for uncertainty has been included in the design values. It is therefore proposed that the background to the derivation of the design values should be given in a separate document.

There are eight principal recommendations.

- (a) For most loads, design values should be given in BS 6399 for direct use in structural design (see 5.2).
- (b) The uncertainties reflected in the design values should be only those associated with the loads (see 5.2).
- (c) For dead loads it may be more practical to retain partial safety factors for use with representative values of material densities (see 8.1.1).
- (d) The partial safety factors for dead loads should be adjusted to provide adequate reliability in combinations involving opposing loads and cases where dead load predominates (see 9.1.3).
- (e) There should be two types of design load, extreme and severe, for the two types of limit state: extreme for ultimate and severe for serviceability (see 7.1).
- (f) For all loads, other than dead loads, two values, principal and accompanying, should be given for each type of design load for use in load combinations (see 7.4.1).
- (g) A load combination should comprise the dead loads, the principal value of one other load and the accompanying values of all the remaining loads (see 7.4.2).

(h) There should be a special rule for combinations involving accidental loads (see 7.4.3).

The report includes some preliminary comparisons with current practice which confirm the feasibility of the basic proposals. For example, for a column with dead, imposed and wind loads:

Combination equations at the ultimate limit state using BS 8110 method

$$(1) \quad 1.4 G_k + 1.6 Q_k \\ = 1.4 \times 400 + 1.6 \times 200 = 880 \text{ kN}$$

$$(2) \quad 1.4 G_k + 1.4 W_k \\ = 1.4 \times 400 + 1.4 \times 300 = 980 \text{ kN}$$

$$(3) \quad 1.2 G_k + 1.2 Q_k + 1.2 W_k \\ = 1.2 \times 400 + 1.2 \times 200 + 1.2 \times 300 = 1080 \text{ kN}$$

Combination equations at the ultimate limit state using proposed method

$$(i) \quad G_{\text{max, extreme}} + Q_{\text{princ, extreme}} + W_{\text{accomp, extreme}} \\ = 1.2 \times 400 + 300^* + 0^* = 780 \text{ kN}$$

$$(ii) \quad G_{\text{max, extreme}} + Q_{\text{accomp, extreme}} + W_{\text{princ, extreme}} \\ = 1.2 \times 400 + 120^* + 400^* = 1000 \text{ kN}$$

Values marked * would be obtained directly from the proposed new loading code.

NOTE 1. This comparison is only to show the simplicity of the proposed method: there is no significance in the difference between the resulting maximum values derived for the two methods.

NOTE 2. The titles of the publications referred to in this document are listed on page 28.

2 Definitions and symbols

2.1 Definitions

For the purpose of this Published Document the following definitions apply.

2.1.1 accidental load. A load which, in any given structure, is not likely to occur with a significant value during the period of time under consideration. It usually arises from events such as external impacts, explosions and fires.

2.1.2 accompanying value. A reduced value which represents a longer term value (compared with the principal value) for that parameter.

2.1.3 design value. The value of a parameter used directly in the design equation (it includes an allowance for uncertainties associated with that parameter).

2.1.4 environmental load. Load generated by climatic factors.

2.1.5 extreme design value. A design value for use in ultimate (safety) limit state calculations.

2.1.6 limit state*. Limit beyond which a structure or part of a structure is unfit for use, being unable to fulfil its functions or satisfy its design requirements.

2.1.7 load combination. A combination of load design values acting simultaneously on a structure or structural element.

2.1.8 permanent load. Load which is likely to act throughout a given period and for which variation in magnitude with times is negligible in relation to the mean value; or for which the variation is in one sense and a limiting value is attained.

2.1.9 principal value. The peak value for a parameter.

2.1.10 ultimate (safety) limit state*. Limit state corresponding to the maximum load carrying capacity and related to safety or major economic loss. Included are collapse or excessive settlement, failure of a member or plastic yielding leading to a mechanism, instability (buckling) and overturning. Large movements of part of a structure may lead to an ultimate (safety) limit state being exceeded in another part of the structure.

2.1.11 serviceability limit state*. Limit state related to the normal use of the structure. Included are durability, vibration, excessive distortion and cracking or other local damage.

2.1.12 severe design value. A design value for use in serviceability limit state calculations.

2.1.13 variable load. Load for which variations in magnitude with time are not negligible in relation to the mean value.

2.1.14 weight density. The weight per unit volume.

2.2 Symbols

E earth pressure
 G dead load
 Q imposed load
 S snow load
 W wind load

subscripts:

accomp accompanying
 extreme extreme
 k characteristic
 max maximum
 min minimum
 princ principal
 severe severe

3 Historical review

3.1 General

This section will briefly consider the different design philosophies used up to date and the ways in which they introduced safety factors. It will also consider and summarize the conclusions of previous UK committees who reported on structural safety.

3.2 Safety factors

Safety factors were originally used in structural design to ensure that structures were designed so as not to operate close to their limits. Later it was realized that these factors were guarding against ignorance and variability. With the passage of time the ignorance, e.g. the ignorance of wind

* Denotes definition different from that given in BS 6100.

loads which resulted in the collapse of the Tay Bridge, has been partly reduced by research so that the principal criterion for using safety factors today is to guard against variability, e.g. the uncertainty in the capacity of a particular steel beam due to the variability in dimensions and yield strengths for nominally identical ones. Here there has been some advance and some of the variability can now be quantified in statistical terms; this has led to the introduction of partial safety factors.

It has been suggested that safety factors cover, to a certain extent, change or use. They are not intended to do this, and can only do so if it can be shown that uncertainties allowed for in design have been reduced. Often an appraisal of an existing structure can show it to be capable of carrying a load higher than that for which it was designed because, for example, the actual strength of the material used is found to be higher than assumed, or alternative load paths can be identified. However, as more sophisticated structural analysis techniques become available and a better understanding of material strengths is developed it is less likely that future structures will be found to be capable of carrying higher loads on appraisal. This is not something that the safety factor should allow for, rather it should be considered explicitly by the engineer during the design.

3.3 Design philosophies and safety factor formats

Before the nineteenth century 'structural engineering' was primarily concerned with the construction of bridges, churches and cathedrals. The construction material was normally masonry and stability of blocks piled on top of each other, differential settlement and overturning were the main problems encountered. The introduction of cast iron, initially to replace masonry arches in bridges, but then in girder form for mill buildings, led to concern about catastrophic failure under load¹. Initially a safety factor was defined as the ratio of the load required to break the girder to the maximum load it was expected to carry. By the time wrought iron and mild steel were brought into use, beam theory was beginning to be understood and the safety factor became linked with breaking and working stresses.

Engineers knew that for structural elements made from wrought iron and mild steel the margin between the elastic limit and the ultimate strength was large. They also knew that if the elastic limit was exceeded the structure would suffer permanent deformations. They therefore decided that a structure would be safe and perform acceptably if the elastic limit was not exceeded. However, recognizing that there were uncertainties in the carrying capacity of structural members, design was carried out using a working stress which was a proportion of the stress at the elastic limit. The uncertainties which they considered to be covered by the safety factor included²:

- (a) variations and imperfections in the material and the workmanship;
- (b) corrosion or other deterioration of the material;
- (c) secondary and other stresses not taken into account in the calculations;
- (d) the possibility of the imposed load being higher than specified;
- (e) the dynamic effect of moving loads;

(f) the effect of repetition of stress on the elastic strength;

(g) the unlikely but possible combination of several adverse events (although, for economy reasons, they also allowed an increase in stress of up to 25 % for the unlikely combination of maximum dead, imposed and wind induced stresses).

This concept of design using working stresses later became known as permissible stress design. It was based on the general assumptions that the structural material was homogeneous, obeyed Hooke's Law, was perfectly elastic, had the same modulus of elasticity in tension and compression, deflected such that plane cross-sections remained plane, and was isotropic³.

The introduction of reinforced concrete in the early 1900s caused problems because few of the basic assumptions for working stress design applied. However, for the sake of consistency (and simplicity) working stress design was initially adopted as the basic concept for design for this 'new material'. As the understanding of the behaviour of reinforced concrete grew, engineers realized that the non-linear behaviour of concrete could be dealt with better by considering its ultimate strength. This, however, required a knowledge of the loads likely to cause failure at the ultimate strength, consequently the loads used in the working stress design should have been increased by multiplying them by a load factor. Instead, when the load factor method of design was introduced into the design codes it was in terms of expressed working stresses adjusted by modifying the method accordingly⁴.

In hand with the development of the load factor method went the increasing application of statistics in predicting material strengths. Data on concrete strength were being gathered because the construction process required cubes to be tested to prove that the concrete met the strengths envisaged in design. Realization that neither working stress methods nor load factor methods were completely adequate for design in reinforced concrete led to the introduction of the concept of limit state design where a number of different limit states were considered independently of each other. These limit states were grouped according to whether they were likely to cause structural failure [the ultimate (safety) limit states] or likely to impair the use of the structure for its purpose [the serviceability limit states]. However, the introduction of this concept⁵ was confused because at the same time the treatment of safety using more than one safety factor, called partial factor format, was introduced. The partial factor format was based on the idea that a more uniform level of safety would be obtained for structural elements if the uncertainty (represented by the partial safety factors) was applied directly to the parameter to which it related. The partial factor format adopted was basically a two factor system with partial safety factors being applied to the material strength and to the loads.

The limit state concept and partial factor format were originally developed in Russia for steel structures and were first adopted in the UK for the code of practice for the design of concrete structures⁶ in 1972. In 1978 a limit state code for the design of masonry⁷ was introduced, followed by a limit state code for the design of steel bridges⁸ in 1982 (this code having been calibrated⁹ using procedures given in CIRIA Report 63¹⁰). When the same concept and partial factor format were adopted in the UK for the design of steel structures¹¹ in 1985 the partial safety factor for loads was considered to consist of two parts, one part covering uncertainty in the performance of the structure and the second part covering uncertainty associated with the loads, although for convenience the two parts were not separated.

While the limit state concept linked with the partial factor format was an acceptable alternative to permissible stress design for the design of the structure above ground, there were considerable doubts about its applicability for foundation design. The main criticisms were over the use of the partial safety factors because¹²:

- (a) the degree of certainty with which a given geotechnical parameter, such as undrained shear strength, can be assessed varies significantly from one project to another. Prescribed partial safety factors applied to mean or 'characteristic' values of the parameters are therefore not adequate for deriving design values intended to accommodate the variability;
- (b) it is not sensible to apply factors to water pressures and geological uncertainties, however these are often significant uncertainties and require treatment consistent with the other design variables.

To overcome these problems Simpson et al¹² proposed a treatment for geotechnical design which, while maintaining the limit state concept, adopted the use of directly assessed 'expected' and 'worst credible' values to be used (unfactored) in design.

While in the UK and Europe use of the limit state design concept with the partial factor format (whereby factors were applied to loads and material strengths) continued to develop, an alternative approach was being adopted in the USA. Known as the load and resistance factor design (LRFD) approach¹³ this was also based on the limit state concept and the partial factor format. However, it differed from the UK approach in that the partial safety factors were applied to the loads and to the resistance of the structural member, rather than to the loads and the material strengths.

To assist with the adoption of the LRFD approach, a report was issued in the USA in June 1980¹⁴ proposing load factors and load combination rules for use with all materials. The partial safety factors had been developed using probabilistic techniques. A target notional probability of safety (expressed as a reliability index) was developed from existing design practice for all materials. This target was then used to determine the optimum set of partial safety factors which most closely achieved it for all limit states in all materials.

A similar method for determining a uniform set of partial safety factors has since been used to develop proposals in South Africa¹⁵. In Australia probabilistic techniques have also been used¹⁶ to determine the capacity factors for their proposed new steel code¹⁷.

In Switzerland a somewhat different philosophy to that used in the limit state/partial factor format codes has been proposed. Known as the Hazard Scenario Concept¹⁸ the approach involves identifying all the hazards which may threaten the design, construction and use of a structure. A hazard scenario is then developed where one of the hazards is assumed to be the predominant hazard with all the other hazards acting simultaneously. The predominant hazard is assumed to act in its most adverse manner while the remaining hazards are assumed to be in some average state. There are similarities between this approach and that proposed by Simpson et al¹².

Another development in partial factor formats has been a proposal¹⁹ that the partial safety factors for loads and for material strength should be chosen so that they are solely dependent upon knowledge of the parameter to which they are applied. One other partial safety factor would be introduced which takes account of all the remaining uncertainty.

In Scandinavia the Nordic Committee on Building Regulations (NKB) Guidelines for loading and safety regulations for structural design²⁰ have recently been revised. These contain guidance on the values of partial safety factors for use in various circumstances. In the case of material properties the partial safety factors are related to the coefficient of variation of the quantity. In addition, a probabilistic procedure is given for calculating design values directly, which removes the need to consider partial safety factors altogether.

3.4 British committees on structural safety

3.4.1 ISE Special Committee on Structural Safety.

Appointed in December 1951 by the Institution of Structural Engineers (ISE) this committee's report was published²¹ in May 1955. Considering design by calculation, the report suggested that the ideal procedure should be regarded as comprising full calculations relating to the behaviour of the structure under both working load and collapse load conditions. It suggested that two different margins of safety should be considered, each expressed as a load ratio. These were:

- (a) *ultimate load factor*, the ratio of the ultimate load to the appropriate working load;
- (b) *limiting load factor*, the ratio of the limiting load to the appropriate working load.

The working load was defined as the greatest load that has a reasonable chance of occurring on the structure during its life, while used in the manner for which it was designed. The limiting load was defined as the load, distributed in the same way as the working load, at which some limiting condition, such as excessive deflection or cracking, first arises. The ultimate load was defined as the maximum load, distributed in the same way as the working load, that the structure can carry.

The report suggested that the ultimate load factor should be determined taking account of:

- (1) workmanship, having regard to inspection, maintenance and materials;
- (2) loading, having regard to control of use;
- (3) accuracy of analysis, having regard to type of structure;
- (4) danger to personnel;
- (5) economic considerations.

It made no attempt to propose recommended numerical values for the load factors but gave several methods, based on the assessment of a number of particular design examples, which could be used to assess them.

The report recognized that some conditions, such as fatigue and vibration, instability due to aerodynamic causes and fire damage, have to receive special consideration.

3.4.2 CIRIA Study Committee on Structural Safety. This committee was formed in 1964 by the Construction Industry Research and Information Association (CIRIA) and produced two interim reports in 1965²² and 1968²³ and a final report in 1971²⁴. It restricted its considerations to structures under static or quasi-static loads and did not consider items such as fire or earthquakes. It considered that loads and material strengths could be represented by statistical distributions and identified the basic safety problem to be the chance of a weak structure meeting an extreme load, and hence the probability of failure resulting therefrom. It posed the question of what probability of failure is acceptable but recognized that the answer would need to take account of public reaction to structural accidents of different kinds.

The report considered separately:

- (a) serviceability of structures; and
- (b) behaviour when loaded into the non-linear range between yield and its collapse load.

For the latter it considered that a structure should not be excessively brittle so that it could collapse suddenly and without any warning.

The report then went on to identify ultimate (or collapse) limit states and serviceability limit states. It excluded blast loading, other explosive pressures, fire and impact from these limit states.

The studies of this committee were greatly influenced by work then going on within organizations such as the Comité Européen du Béton (CEB), the Conseil International du Bâtiment pour la Recherche, l' Etude et la Documentation (CIB) (in particular Thomas's paper²⁵), the Convention Européenne des Associations de la Construction Metallique (CECM), the International Organization for Standardization (ISO) and the American Society of Civil Engineers (ASCE). The report introduced the concept of characteristic values and used the same sub-division of partial safety factors subsequently included in ISO 2394²⁶. The second interim report²³ gave tentative values for the different partial safety factors.

3.4.3 ICE Committee on Safety in Civil Engineering.

Included for completeness, this committee was formed in 1967 by the Institution of Civil Engineers (ICE) to study failures or collapses of structures of temporary works and of plant. It thus looked at practical aspects of safety and its report²⁷ concentrated on:

- (a) safety of the structure during erection or demolition;
- (b) safety of construction plant; and
- (c) safety of personnel.

The report identified the principal types of failure and their causes and then gave information on prevention of failures and implications.

3.4.4 CIRIA study on rationalization of safety and serviceability factors in structural codes. This study resulted in a report¹⁰ intended as 'an interim stage in a continuing search for economy with reliability in the structural art'. In fact the report was one of the most comprehensive theoretical treatments of structural safety ever produced. It continued the work of the earlier British committees by introducing sophisticated analysis techniques for determining the partial safety factors for use in structural design calculations. It adopted the partial factor format proposed in ISO 2394²⁶ and CEB Bulletin 111²⁸ and recommended that the values of the partial safety factors should be determined by calibration with existing practice using an advanced first-order second-moment method to determine notional probabilities of failure. This approach was subsequently used in the calibration of the British steel bridge code, BS 5400 : Part 3^{8,9}.

3.4.5 ISE Code Servicing Panel on Rationalization of γ -factors. This committee was established in 1977 specifically to study the implications of the CIRIA study on rationalization of safety and serviceability factors. It concluded that the study's report was a research document which itself required extensive further work. The committee produced two reports^{29,30}. The first dealt primarily with the CIRIA report and the second gave recommendations for further action. The second report confirmed that calibration should be carried out against existing design practice as given in the existing design codes which were considered generally to give a reasonably acceptable level of reliability. It then gave a number of recommendations as guidelines to calibration exercises, which included a list of 12 different possible sources of uncertainty in design equations. Four of these sources of uncertainty were related to forces, two were related to consequences, four were related to materials and the remaining two related to the accuracy of the equation and the difference between the behaviour of assemblies of elements and that of the separate elements considered individually. It also gave a list of research which it considered to be necessary.

3.5 Structure/soil interface

Since the introduction of codes using partial safety factors there has been a problem at the structure/soil interface. This arises because the superstructure is designed using factored loads but the foundations are usually sized using unfactored or characteristic loads. The structural design of