

Concrete Design: U.S. and European Practices

The following four tables should be placed at the end of "General Approach to Safety, Serviceability, and Limit State Philosophy--European Concrete Committee," by R. W. Rowe (pp. 13-22).

Table 1 - Values of γ_m

Material	Limit state	
	Ultimate	Serviceability
Concrete	1.5	1.0
Reinforcing or pre-stressing Steel	1.15	1.0

Table 2 - Values of ψ_o

Action	ψ_o
1. Associated with	
Domestic buildings)	
Office ")	0.7
Retail Premises)	
Parking Garages	0.9
2. Associated with	
Wind	0.8
Snow	0.8
Wind and snow	0.8 & 0.6

Table 3 - Values of ψ_1 and ψ_2

Action	ψ_1	ψ_2
1. Associated with		
Domestic Buildings	0.7	0.4
Office "	0.8	0.4
Retail Premises	0.9	0.4
Parking Garages	0.7	0.6
2. Associated with		
Wind	0.2	0
Snow	0.3	0

Table 4 - Limit state of cracking - criteria

Environment	Load Combination	Steel					
		Sensitive			Not Sensitive		
		Limit State	w_k (mm)	c_{min} (mm)	Limit State	w_k (mm)	c_{min} (mm)
Very Aggressive	(1)	D	-	45	W	$\leq w_1$	35
	(2)	D	-	45	-		
Moderately Aggressive	(1)	W	$\leq w_1$	35	W	$\leq w_2$	25
	(2)	D	-	35	-		
Not Aggressive	(1)	W	$\leq w_2$	25	W	$\leq w_3$	15
	(2)	DorW	$\leq w_1$	25	-		

where $w_1 = 0.05$ mm

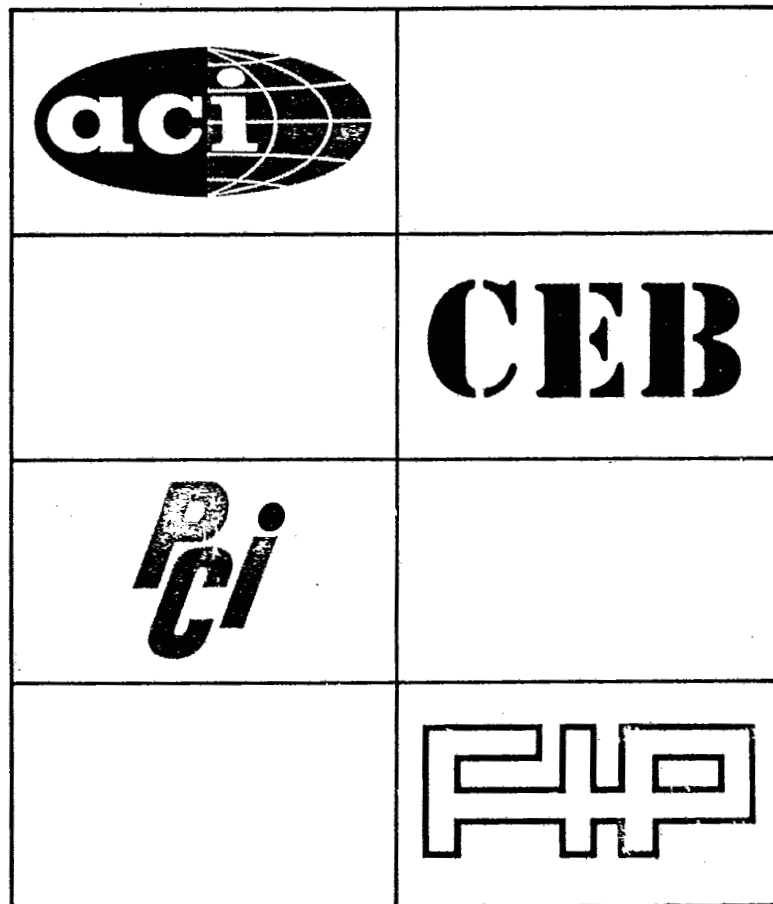
$w_2 = 0.2$ mm

$w_3 = 0.4$ mm

and c_m is the minimum cover to the steel.

CONCRETE DESIGN: U.S. and European Practices

ACI-CEB-PCI-FIP Symposium

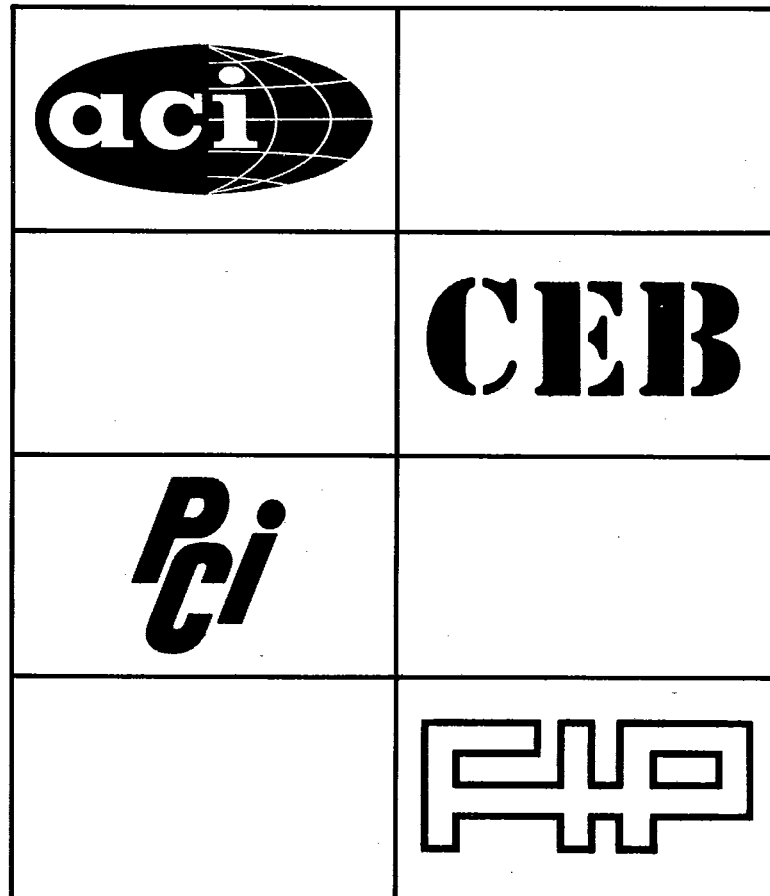


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CONCRETE DESIGN: U.S. and European Practices

Proceedings of the symposium cosponsored by the American Concrete Institute, the Comité Euro-International du Béton, the Prestressed Concrete Institute, and the Fédération Internationale de la Précontrainte. The symposium was held during the 1976 ACI Annual Convention, Philadelphia, Pa.



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Documents



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PREFACE

A special international symposium was held at the American Concrete Institute's Annual Convention in Philadelphia, Pa., in 1976. The symposium was cosponsored by ACI, Comité Européen du Béton (CEB) (now Comité Euro-International du Béton), Prestressed Concrete Institute (PCI), and Fédération Internationale de la Précontrainte (FIP). The purpose of the symposium was to achieve a cross-pollination of ideas between the United States and Europe, especially as it applied to the ACI Building Code. Papers concentrated on the innovations and changes which the organizations were introducing into their new codes. Speakers emphasized the philosophy underlying the code revisions and changes, and the effect these changes would have on design practice.

The symposium had been proposed by the ACI Board Committee on International Activities, of which Mete A. Sozen was chairman at the time. The symposium program was organized by a subcommittee comprised of James G. MacGregor, Chairman, Phil M. Ferguson, and Alan H. Mattock. James MacGregor also served as editor of this volume. Sixteen dignitaries representing CEB and FIP came from Europe to participate in the symposium, including Andrew Short, president of CEB, Yves Saillard, président délégué of CEB, and Arthur Hill, secretary-general of FIP. Ben C. Gerwick, Jr., president of FIP, George F. Leyh, executive director of ACI, and W. Burr Bennett, Jr., executive director of PCI, also participated.

The first session of the symposium was devoted to limit states design. The second session discussed shear and torsion. Papers in the third session dealt with special concepts and new applications for prestressed concrete.

This volume contains 20 papers developed from the symposium and follows the same format. In addition, addresses given by CEB President Short and FIP President Gerwick at the general session of the convention are included.

This symposium signified the continued active cooperation between ACI and other organizations on an international scale.

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COLLABORATION BETWEEN
COMITÉ EURO-INTERNATIONAL DU BÉTON AND AMERICAN CONCRETE INSTITUTE

Address given at the American Concrete Institute
1976 Annual Convention, Philadelphia, Pa., Mar. 31, 1976

By Andrew Short
President, CEB

I am grateful to the American Concrete Institute and its Board of Directors for the invitation extended to me to address this important Bicentennial Spring Convention held in Philadelphia, the "City of Brotherly Love," on behalf of the Comité Euro-International du Béton (CEB). It is a source of particular satisfaction to me that this splendid Philadelphia convention should include the Joint ACI-CEB-PCI-FIP Symposium on our technical work and attitudes to Code harmonisation. I have had occasion during the past year of visiting the United States as well as Mexico and Canada, and everywhere I received most generous and delightful welcome and friendship. I am most grateful for all the kindnesses extended to me, to my wife, and to my colleagues.

The reasons for my presence before you today are not a secret. We have had some preliminary discussions and I now come to ask you to join forces with us, in CEB, on a world-wide scale for I think that this is a logical and indeed inevitable consequence and culmination of the activities which your organization and ours has been engaged in throughout the past two and a half decades. Moreover, in my opinion, this is a most favourable time for such a development for our technical differences are smaller now than I can remember then; if we fail to act now, there may not be another favourable opportunity for a long time.

Twenty-five years ago, soon after the War, some of the most experienced and knowledgeable Engineers in Europe were brought together to examine the possibility of designing concrete structures on the basis of a scientific recognition of agreed new knowledge, knowledge that was the fruit of research work and experience. They were soon joined by individual American colleagues; I should mention here in particular the names of a few of your honoured older statesmen like Ray Reese, Phil Ferguson and others. CEB was created

as the instrument of this endeavour, initiated by the French Confederations for Building and Civil Engineering with the support of the then newly created European Iron and Coal Community. Throughout its history The French Contractors' and professional engineering organisations continued to provide intellectual and financial strength for CEB. Thus CEB is the child of a marriage between industry and the professions; now, in adulthood it still looks to industry and the professions for inspiration and support while at the same time welcoming the understanding and additional support that is given it by Governments both at national and international levels.

It should be said firmly, however, that CEB is - and will in future continue to be - an independent force, independent of all the great component factors of the construction world as well as of Governments and their agencies. CEB is an unashamedly elitist organisation, seeking excellence in its work, for the benefit of mankind.

The first edition of the International Recommendations of CEB on Concrete Construction, published in 1963, was followed 7 years later by an improved and expanded 2nd edition created in very close co-operation with FIP. Both editions were found to have had an unprecedented influence on the thinking of Engineers everywhere.

By this time a small number of American Engineers participated enthusiastically in our work but, the idea of international co-operation on a broad scale has perhaps not yet touched the imagination of the great mass of American Engineers. We had correspondence and meetings with eminent members of the American profession. In particular we had most helpful consultations with the successive chairmen of your International Committee, with Dr. Eivind Hognestad and then with Professor Mete Sozen, planning future co-operation. We hammered out future action and we agreed on what should be done. The key to co-operation to-day, however is no longer merely the goodwill of your chief officers and elected officials which we already have. The key to that co-operation is your own willingness and persuasion.

But our organisations are not there merely to arrange conferences; CEB and ACI produce documents and these meetings were not without practical consequences. One of the most significant results was the adoption of common forms of notation which has now been accepted on CEB's suggestion, by the International Standards Organisation (ISO). The creation of a virtually common, internationally applicable system of notation for concrete construction and design was accomplished as a result of close collaboration between CEB and ACI.

There has been during the past few years, I feel, a perceptible change in attitude in influential and important sections of your community of Engineers in ACI however, towards the idea - to put it no more strongly than that - of a possibility of enhanced co-operation between our organisations and even of you joining CEB. Today CEB is a world wide organisation which extended its European image and is a truly international organisation of whose 35 members and associated countries more than half are outside Europe.

Meanwhile CEB came of age; a great effort was made to produce practical documents such as Design Manuals and Technological Manuals, to establish closer ties between scientific understanding and practical application. The application of the limit states theory and other modern methods of design was explored. CEB initiated the International Safety Commission and co-operates with all significant international technical organisations, largely through the International Liaison Committee for Civil Engineering.

CEB has also become a consulting organisation to the European Communities in Brussels without however losing its truly International Standards Organisations (ISO) of which the US is a member. In conjunction with the appropriate technical committees of ISO, CEB is now producing a system of international Standard Model Codes of Practice for structural engineering and for the safety of structures, and in this they are working in close co-operation with other international voluntary professional organisations such as FIP, CIB, CECM, IABSE, RILEM and so on. This fundamental detailed work, which will be set out by my colleagues in the course of the Symposium, will - I hope - form the foundation on which legislative action can be taken by the European Communities as well as by the Council for Mutual Economic Aid of Eastern European Countries and by ISO and so far as Building Regulations are concerned, by the UN European Economic Commission of which the United States again is a member. Within this System CEB is responsible, in co-operation with other international organisations for the creation of a volume on the Safety of Structures which will apply to all major methods of civil engineering construction, and of course, for the volume on concrete construction. Some of these volumes are already available in draft form, for example in our Bulletin 111 and are discussed throughout the world intensively. We have welcomed the ACI initiative for holding this Symposium today and tomorrow, because it will give us an opportunity to endeavour to harmonise our drafts and proposals with your documents. Indeed, this has been pursued throughout the last few years already, with a measure of success. Within this International System of Codes CECM are producing the Code for Structural Steelwork, CEB and CECM are jointly producing that for Composite Construction, CIB are responsible for the Codes on masonry and timber

construction. The CEB Model Codes represent in essence a broad frontal approach, a 'grand menu', accepting in many cases a number of alternative approaches of equal validity for different circumstances, albeit in Code form. The ACI Code, on the other hand is so shaped as to be capable, as it stands of direct and immediate application within a legal framework which is mandatory in character.

It should be remembered, however that subsequent CEB documents to be derived from the Model Codes for ISO or for the European Communities as a basis for EEC legislation will have to be similarly concise and directly applicable, as the ACI Code.

What is needed, at this stage, however, is an effective translation of our technical and scientific consensus as between ACI and CEB into a proper working machinery capable of day-to-day administrative, organisational action, work of practical management in which members of ACI traditionally excel. Indeed, I do not mean that we can here and now establish a precise framework for more intensive co-operation between our organisations. This is a matter for later and successive agreements which must be carefully worked out. One thing is certain: this co-operation must involve more than periodic informal discussions or participation in each others' working committees or indeed, participation in conferences like this one. It would be perhaps wrong for me to anticipate detailed discussions on these problems, but I can say this, that we in CEB have no brief which could involve rigid adherence to entrenched positions and attitudes either in the technical or in the administrative fields. On the contrary, we would greatly appreciate the privilege of receiving your ideas and would wish, wherever possible, to incorporate those in our documents; personally I see the International Model Codes as joint documents incorporating ACI views and ideas and published with a joint CEB-ACI 'imprimatur'. We should be particularly indebted to you if we could have the benefit of the great and successful experience of American Engineers and managers in the organisation of construction work in factories and on building sites for in that field your performance and teaching has been shown to have been of outstanding merit.

I shall now turn to the question of what benefits ACI would derive from co-operation with CEB on a more extensive scale. It would be tempting to repeat the current arguments, some more, some less persuasive, as to why international co-operation would be beneficial for American engineers and the US as well as for mankind in general. You will be familiar with most of these arguments, however. I would, nevertheless, mention the growth of building and civil engineering design and construction activities in Third World countries which now, owing to their ownership of the essential raw materials of which

industrialised nations are in need, are much richer than they ever thought themselves to be in the past. They look to you and they look to us to provide the basic engineering know-how as well as the necessary investments, at least initially, for the enormous programmes of construction with which they are faced. This could, of course, proceed according to the national standards, codes and building regulations existing in our various industrialised countries, leaving the host countries unable to compare realistically the economy and worthwhileness of tenders they receive since they are based on different rules and regulations. Moreover, technical legislation and documentation are just as much part of a nation's national culture as its language, art, or literature. In due course, nations of the Third World will insist on having their own. Which Recommendations will they adopt? Surely it would be helpful to everybody if they could direct their minds to an internationally accepted general set of codes and standards sufficiently flexible to be adoptable to individual, national or regional conditions and requirements and yet be based on the same scientific foundation.

In North America ACI has gradually perfected an extremely effective system of consultation between industry, the professions and Government at all levels, with the active encouragement and assistance of national agencies such as the National Bureau of Standards, of the Department of Commerce. In different ways and with somewhat varying success similar consultation procedures have been evolved in many of the member countries of CEB, but yours seem to be more sophisticated and complete. Their extensive application would be beneficial to the United States as well as everywhere else. They would ensure a desirable measure of continuity, but also a capacity to incorporate the developing results of scientific research and experience in standard and codes in an orderly and readily applicable manner throughout the world. In view of the increasing share of international consulting and contracting work undertaken by the US construction industry, which is the largest of your industries, this aspect should not be overlooked.

Is it not odd that these matters are organised in such a diffuse way? Technical rules vary from country to country, sometimes in a surprising and illogical way, almost irrespective of local conditions or workmanship requirements. Is it not time at least to unify our technical procedures, safety principles and design rules and thus provide a consistent machinery for orderly progress?

The existing differences in codes and regulations in different countries and indeed within the United States constitute a serious obstacle to the free interchange of goods and professional expertise between different countries.

I do not believe that this is any longer a question of greater or lesser advantage to this or that professional interest or this or that country. Indeed the formation of common Codes of Practice and Standards, a common basis for technical regulation, has become national Government policy in all industrialised and most developing countries in the world. A deliberate policy declaration to that effect was signed last year by 35 heads of state, including the President of the United States which clearly states in the Final Act of the 1975 Helsinki Conference on Security and Cooperation in Europe under Act 3 'Provisions concerning trade and industrial co-operation', 'Harmonisation of Standards' that.

The participating States,

'Recognizing the development of international harmonisation of standards and technical regulations and of international co-operation in the field of certification as an important means of eliminating technical obstacles to international trade and industrial co-operation, thereby facilitating their development and increasing productivity,

reaffirm their interest to achieve the widest possible international harmonisation of standards and technical regulations:

express their readiness to promote international agreements and other appropriate arrangements on acceptance of certificates of conformity with standards and technical regulations;

consider it desirable to increase international co-operation on standardisation, in particular by supporting the activities of inter-governmental and other appropriate organisations in this field'.

Many people realise by now that perhaps the only part of the Helsinki declaration capable of effective realisation now or in the short term was precisely this brief technical part for certainly, harmony among engineers in the technical field seems much easier to achieve, with all its complications, than in the political or social fields between Nations!

So far as Governments and their official organisations are concerned, they must be clearly bound by international agreement to press for the harmonisation of technical rules in the construction field. The question for us now in the professions and industry is not whether this harmonisation should in fact take place at all but whether we should undertake to have a hand in shaping and furthering this activity.

If you decide: it shall be without us, senze noi, there is nothing further to be said. I feel, however, that such a negative would be a mistake. It would confer sole, unqualified and unprotected moral as well as legal authority on official bodies in the USA as well as in Europe at the national levels as much as at the international level, to legislate and determine alone basic requirements for safety and health; indeed not only those but equally questions of detailed design and construction rules, a function which was reserved by general agreement, until now for industry and the professions. This would have far reaching and, on balance, undesirable consequences.

You may decide, on the other hand, that you want to help in guiding these essential harmonising and codifying operations on international as well as on national levels because you have - as I believe that you have - an essential contribution to make. That contribution would then be made more effectively by making use of existing recognised international professional machinery, together with us in CEB, based on our long history of common interests and activities. Undoubtedly such a combined undertaking would also strengthen your position in the national context.

I am for offering this confident and respectful offer of friendship and joint action for common policies, activities and publications, perhaps in the context of the possible formation of a Joint National United States CEB Committee;

not merely because this would bring tangible material and political advantages to ACI as well as to CEB; to American Engineers as well as to Engineers throughout the World; to the people of the United States as well as to mankind as a whole;

not merely because our Governments in their wisdom have resolved to convert into public policy that which has always been a common, spontaneous endeavour over the years in CEB as it has been indeed in ACI;

I ask you to make this act of faith together with us in CEB simply because in all the circumstances and with all our frailties and doubts, this is the proper and sensible thing to do!

THE ROLE OF THE FÉDÉRATION INTERNATIONALE DE LA PRÉCONTRAÎNTE

Address given at the American Concrete Institute
1976 Annual Convention, Philadelphia, Pa., Mar. 31, 1976

By Ben C. Gerwick, Jr.
President, FIP

The International Federation of Prestressing (FIP) is proud to participate in this historic meeting, appropriately held in this historic city and time.

This joint symposium has as its objective the increased understanding of the practices and codes, including their underlying philosophies, which are employed for concrete construction in Europe and America. This, of course, has been a continuing process for many years, fostered by a series of purposeful individual exchanges. However, this meeting brings the work into sharp focus, and should result in a highly constructive exchange.

FIP is devoted to the development of recommended practices for design and construction of prestressed concrete and related aspects of concrete construction, and to the dissemination of information concerning effective and safe utilization. We consciously endeavor to respond in a timely manner to the challenges of societal needs, such as energy, housing and transport.

FIP's response takes two forms, technical and organizational. In the technical field, our Commissions produce reports and Guides to Good Practice. These are made available for use in connection with the production of the FIP/CEB Recommendations, whose third edition will be issued in 1977.

FIP sees as one of its primary functions the timely development of guides or recommendations in newer fields of utilization, such as concrete ocean structures; concrete pressure and storage vessels, ranging from nuclear containment to oil and LNG storage; and the exciting new field of concrete ships. We attempt to integrate advanced concrete technology with the requirements of the User Industry.

The Federation also recognizes its obligation to developing countries and seeks to produce guides for the efficient use of prestressed and precast concrete in such fields as mass housing, port structures, and utilities. Other Commissions and groups are engaged in preparing reports on Fire Resistance, Partial Prestressing, Prestressed Lightweight Concrete, Steels for Prestressing, Practical Construction, Construction in Seismic areas, and Special-Purpose Concretes.

Our work is disseminated through the media of Congresses, Symposia, seminars, and especially through our publications.

Turning attention to the organizational response, the founders of FIP wisely created a federal form of organization in which Member Groups from all nations of the world can participate and exchange views and information on developments. Such a federation can only be successful if we have full participation, both by member groups and individuals: thus we encourage this professional involvement, carrying out our work on a consensus basis. We have been fortunate to achieve a productive and cooperative relationship with our member groups from far parts of the world, spanning across political, ideological, and economic boundaries.

These exchanges occur in both directions and are synergistic in effect. Our reports on research and development go out from Europe and North America to the more distant countries, and to the lesser-developed nations. From them we receive back data on experience, questions concerning problems, and an impressive array of creative applications and development.

Our meetings seek to stress this world-wide involvement: this past December, we held our Executive Committee meeting in Bombay; we have forthcoming meetings of our Administrative Council in Greece and Mexico. We will hold a series of Symposia in Sydney, Australia this coming September and our VIII Quadrennial Congress in London in May 1978.

We are dedicated to the closest possible cooperation with all organizations interested in structures, and especially to CEB, ACI, and PCI, who have done so much to foster the development of practices and knowledge and codes concerning concrete.

We believe that FIP has the opportunity to serve as a link between these fine organizations and the numerous concrete and engineering organizations spread throughout the world.

We look forward to this present meeting and the renewed opportunity to work together towards our common goals, meeting the challenges and opportunities of the expanding role of concrete. Above all, we seek to conduct our work so as to stimulate individual creativity within the framework of cooperative endeavor in construction for the benefit of mankind.

SESSION I

LIMIT STATES DESIGN



SP 59-1

GENERAL APPROACH TO SAFETY, SERVICEABILITY, AND LIMIT STATE
PHILOSOPHY--EUROPEAN CONCRETE COMMITTEE

By R. W. Rowe

Synopsis: The developments, relating to structural safety, which have taken place in recent years are reviewed with particular reference to the activities of the Joint Committee on Structural Safety. The interpretation of these developments in code of practice form is discussed and suggested clauses presented.

Keywords: building codes; cracking (fracturing); limit state design; loads (forces); prestressed concrete; reinforced concrete; safety factor; serviceability; structural design.

R. E. Rowe is Director of Research and Development, Cement and Concrete Association, Wexham Springs, Slough, England, and also chairman of the Editorial Group of the Joint Committee on Structural Safety. He has contributed in many fields related to Research and Development on concrete and concrete structures and has been closely associated with the U K Code for Structural Concrete, both in buildings and bridges.

1. Introduction

When writing on this same subject in 1970¹, the introduction consisted of a catalogue of committees which was extensive; progress has clearly been made, at least in co-ordination, in that now there are two principal committees, the Joint Committee on Structural Safety (CEB-CECM-CIB-FIP-IABSE) and ACI Committee 348. Further impetus to cooperation between the various groups has been the desire to rationalise the treatment of structural safety and serviceability as applied to the different construction materials by the introduction, wherever possible, of agreed partial safety factors.

In Europe, the extension of the original economic community from six to nine, the interest of the United Nations Economic Committee for Europe with its involvement with the COMECON countries and the current activities of various ISO and CEN committees have also given an impetus to these activities. Thus it can be seen that there is very considerable international interest and activity in the whole field and perhaps the most noticeable recent change is the attempt being made to ensure the greatest possible participation of practicing designers in the various discussions.

The CEB/FIP Recommendations of 1970 naturally formed the basis for the revisions and, since these were the result of many years of work, it is hardly surprising that the latest draft proposals of CEB/FIP differ only in the sense of providing amplifications or adjustments; therefore I should apologise in advance for re-stating certain aspects which have already appeared in print.

Rowe, R. E., "Current European Views on Structural Safety"
Journal of the Structural Division A. S. C. E.
Vol 96, No ST3 Proc Paper 7132 March 1970 pp 461-467.

2. Design - Objectives and General Recommendations

The aim of design is clearly defined as being the achievement of acceptable probabilities that the structure being designed will not become unfit for its required use during some defined life. From this it follows that the structure, and its elements, should be designed to sustain, with an appropriate degree of safety (or, alternatively, structural reliability) all loads and deformations liable to occur during construction and in service, to perform adequately in normal circumstances and, equally important, to possess appropriate durability throughout the design life.

Further, it is emphasised that safety, serviceability and durability are dependent not only on the design calculations, but are a function of the supervision on site, the quality control process associated with both steel and concrete, the magnitude and control of unavoidable imperfections (tolerances) and the quality of the site labour.

In defining unfitness for use, the concept of limit states is introduced; thus two broad categories of limit states, namely the ultimate and the serviceability limit states, are defined.

- (a) Ultimate limit states - these correspond to the maximum load carrying capacity and may be reached due to various causes (loss of equilibrium of part of or the whole structure, rupture of sections, transformation into a mechanism, buckling, fatigue, etc.)
- (b) Serviceability limit states - these correspond to the criteria governing normal use (deformation, local damage, such as cracking and spalling, vibration, etc.)

In addition, there are certain other considerations which, while not treated as specific limit states, nevertheless have to be covered in the design process; these are durability, fire resistance and progressive collapse. The first two aspects are normally covered by detailing procedures and specification of materials and workmanship; the last may be treated in a number of ways.

3. Limit state design

3.1 General The limit state design procedure recommended involves treating the variability of the various actions on a structure (loads and imposed deformations) by defining these in terms of

characteristic values which, ideally, are based upon statistical data, appropriately interpreted. Obviously in many cases, adequate data will not be available and then resort to experience is necessary to arrive at appropriate characteristic values. By experience is implied the consideration of past performance of structures designed in accordance with established procedures embodying loading requirements which have evolved rather than being derived.

Similarly, the variability of the strengths and other properties of the construction materials is treated statistically by defining characteristic values related to appropriate standard tests.

In the design process, two principal partial safety factors are introduced namely the partial safety factor for loads and load effects (γ_f or overload factor) and that for materials (γ_m or understrength factor). Thus, if the characteristic load is F_k , then the design load would be $\gamma_f F_k$; similarly if the characteristic strength of the material is f_k , the design strength would be f_k / γ_m . Both γ_f and γ_m are dependent on many parameters including, principally, the limit state being considered and the consequences, in all respects of attaining that limit state. It is possible to assign parts of the partial safety factors to different aspects of loading, structural response, implications of workmanship and so on. Thus:

- γ_{f1} takes account of unfavourable deviations of the loads from the characteristic values;
- γ_{f2} (now designated γ) takes account of reduced probability of combinations of actions all at their characteristic values. (It is thus not strictly a safety factor but merely a factor used to give an appropriate value to a load combinations);
- γ_{f3} takes account of inaccurate assessments of the load effects (accuracy, or otherwise, of calculations, significance of tolerances, implicit in workmanship, as they affect the response of the structure);
- γ_{m1} takes account of the possible differences between the strength of the material in the structure and that derived from control test specimens;
- γ_{m2} takes account of the possible local weaknesses in the material arising from the known and specified workmanship and the possible inaccurate assessment of the resistance of elements derived from the strength of the material (including any dimensional variations in so far as they effect the resistance).

Additionally, γ_m and γ_f may be modified by a further factor, γ_n , to take account specifically of the nature of the structure and its behaviour and the seriousness of attaining a given limit state. In turn, γ_n , may be considered to comprise:

γ_{n1} to cover the type of structural failure (ductile, brittle), and γ_{n2} to cover risk to life and economic consequences of attaining limit state.

These aspects may be relevant in grading the different types of structures. Thus, agricultural buildings, domestic buildings, industrial buildings, etc., may need to be treated differently by virtue of totally different risks to human life and economic consequences associated with the same type of ultimate limit state.

The process outlined above for treating safety has, within the Joint Committee on Structural Safety, been termed Level I Method; it is recognised within all the participating groups on the Joint Committee that this level is that most appropriate to the formulation of design codes. It is, perhaps, worth digressing here to indicate the other levels which are possible. Thus, Level II methods may be propounded which incorporate safety checks only at a selected point (or points) in the data space; this point is identified by a suitable algorithm with an idealisation of the failure boundary in that region. Reliability levels can only be defined by safety indices or equivalent 'operational' probabilities and not in a relative frequency sense. To ensure an invariant measure of reliability and avoid further first order approximations, the overall problem should be formulated in the input space rather than the output space (i.e. in the space of the functionally independent or "basic" variables). Level III methods are those where safety is checked by an "exact" probabilistic analysis for the whole structural system, using a full distributional approach, and with reliability levels based on an agreed failure probability interpreted in the sense of relative frequency. Consideration is being given to these levels as tools in improving and refining the treatment at Level I and as, possibly, design tools in their own right in certain well defined and specialised structures. I should emphasise that, so far, in all these treatments more attention has been focussed on safety i.e. the ultimate limit state, than on serviceability.

In concluding this section some comment on simplification is pertinent; obviously in presenting a design procedure to a designer simplicity is necessary to ensure understanding. Of equal importance, however, is the need to simplify, as much as possible, the overall design process bearing in mind the frequently small part the structure plays in it. Provided the concepts are clearly expressed

and appropriate consideration given in drafting the procedure to the relative importance of various limit states, it is possible to simplify the presentation of level I methods. It is in this sense that the CEB/FIP documents are being directed in their various draft forms.

3.2 Treatment of loads and materials

With respect to loads, the characteristic values should, ideally, be determined from the type of distribution, the mean values and the standard deviations from the mean, to given some acceptable probability of their not being exceeded during some reference period. When such data do not exist (and it must be admitted that this is so for probably the majority of loads if one merely considers the question of adequate data) recourse is had to experience and well tried national values. However, it should be mentioned that the characteristic load, defined as above, must be related to the limit state and the selected performance criterion. To avoid an unwieldy treatment it is now suggested that, from a characteristic load defined for the ultimate limit state, other "effective" characteristic loads can be derived by means of ψ factors.

Thus: ψ_0 relates to that part of a load which occurs in combination at the ultimate limit state;

ψ'_0 relates to that part of a load which might be associated with an accidental loading at the ultimate limit state;

ψ_1 relates to that part of a load which is of frequent occurrence at the serviceability limit state;

ψ_2 relates to that part of a load which is of long term occurrence at the serviceability limit state.

For materials, namely concrete and steel, the characteristic strength is taken as that value of strength below which only a small percentage of the population of all possible strength measurements is likely to fall; for concrete the 5% fractile is normally adopted. In design, it should be emphasised that a characteristic strength must be specified together with workmanship requirements so that appropriate compliance rules can be introduced in the quality control process.

3.3 Methods of analysis

The methods of analysis adopted should be appropriate to the limit state being considered and be based on as accurate a representation of the behaviour of the structure as is practicable. Thus for the ultimate limit state, elastic methods with or without redistribution, non-linear methods, or plastic methods may be relevant whereas for serviceability, generally, elastic methods would be appropriate. Model analysis and testing techniques and the experimental development of analytical procedures are recognised with appropriate qualifications concerning simulation techniques, numbers of test specimen and so on.

3.4 Verification procedures

These may be expressed in the general form, for the ultimate limit state:

$$\text{function } (\gamma_{f_1}, \gamma_{f_2} \text{ (or } \psi), \gamma_{f_3}, F_k) \leq \text{function } \frac{(f_k)}{(\gamma_m)}$$

or the assessed loading effects on the structure and the elements of the structure should be less or equal to the resistance of the structure or its elements.

For the serviceability limit state, dependent on the criteria governing it, there will be an appropriate checking basis involving a computed crack width, deflection, vibration, stress, strain and so on with some stated acceptable value. Alternatively, these may better be dealt with by some "deemed to satisfy" rules governing, for example, span/depth ratio or bar spacing and cover.

To ensure comparability in the design of certain types of structure in different constructional materials, it is useful to consider the ultimate limit state inequality in the form,

$$\gamma_{f_3} \text{ (effects of } \gamma_{f_1}, \gamma_{f_2} \text{ (or } \psi), F_k) \leq \text{function } \frac{(f_k)}{(\gamma_m)}$$

This form may also be relevant for certain non-linear problems.

4. Application of principles in Code

4.1 Materials

The values of γ_m adopted in general, when treating the individual sections within a structure, are given in Table 1. However, depending on the degree of quality control, γ_m for

concrete may range from 1.4 to say 1.7 for the ultimate limit state. Further, for many serviceability calculations the significance of the strength of concrete (and its associated properties) is small but for others where, for example, estimates are required of the effect of cracking or redistribution of stresses in statically indeterminate structures, it is prudent to adopt a value of γ_m of 1.2 - 1.3.

For all limit states the overall structural analysis should be carried out utilising the relevant properties associated with the specified characteristic strength. This is not correct in fact since the structural response will depend on the average properties throughout the structure and, in any event, these vary with time. However, at the design stage, this is the only assumption that is possible, it is consistent with past practice and ensures that a single analysis may be used for all limit states.

4.2 Loads and load effects

(a) Ultimate limit state

The actual magnitude of the various partial safety factors γ_{f1} , γ_{f3} and ψ is still under discussion; what follows represents the position in the latest draft (i.e. November 1975) which has been circulated for comment.

The load combinations are given by:

$$(a) \quad 1.2 G_{\text{mean}} + (1.4) (F_{1k} + \sum_{i=2}^n \psi_{oi} F_{ik})$$

$$(b) \quad 0.9 G_{\text{mean}} + (1.4) (F_{1k} + \sum_{i=2}^n \psi_{oi} F_{ik})$$

G_{mean} is the mean self weight of the structure
 where F_{1k} is considered as the Basic action in the combination
 F_{ik} is considered as the Accompanying action in the combination;
 in general only two accompanying actions need to be considered.

Values have been postulated for ψ_o and, clearly, these will need to be refined; however the suggested values to date are given in Table 2.

When treating an accidental, but defined loading e.g. earthquakes, the design load is given by

$$(c) 1.2 G_{\text{mean}} + 1.0 F_{1k} + \sum_{i=2}^n \psi_{oi}^1 F_{ik}$$

where $\psi_{oi}^1 F_{ik}$ represents the likely value of any other variable action which could act at the same time as the accidental action F_{1k} .

So far, no mention has been made of γ_{f3} ; it is suggested that $\gamma_{f3} = 1.07 - 1.15$ yielding in most cases design loads or, more accurately, design load effects derived from the design loads, of

$$(a) 1.3 G_{\text{mean}} + (1.5) (F_{1k} + \sum_{i=2}^n \psi_{oi} F_{ik})$$

$$(b) 1.0 G_{\text{mean}} + (1.5 - 1.6) (F_{1k} + \sum_{i=2}^n \psi_{oi} F_{ik})$$

$$(c) 1.3 G_{\text{mean}} + 1.1 F_{1k} + 1.1 \sum_{i=2}^n \psi_{oi} F_{ik}$$

The above equations can be enlarged to include prestressing forces and the effects of superimposed permanent loading but these would merely complicate the presentation.

4.2 Loads and load effects

(b) Serviceability limit states

The combinations of loads for this limit state are given as:

(1) Frequent combinations

$$1.0 G_{\text{mean}} + \sum_{i=1}^n \psi_{1i} F_{ik}$$

where $\psi_{1i} F_{ik}$ represents that value of a load deemed to be of frequent occurrence for the assessment of the defined criterion governing the limit state.

(2) Quasi-permanent (long term) combinations

$$1.0 G_{\text{mean}} + \sum_{i=1}^n \psi_{2i} F_{ik}$$

where $\psi_{2i} F_{ik}$ represents that value of a load deemed to be relevant in assessing the long term performance of the material or structure.

(3) Infrequent (or Total) combinations

$$1.0 G_{\text{mean}} + 1.0 F_{1k} + \sum_{i=2}^n \psi_{1i} F_{ik}$$

where a single load is taken at its characteristic value while all others are at their frequent value.

For these limit states γ_{f3} is taken as 1.0 and suggested values of γ_1 and γ_2 are given in Table 3.

(c) Criteria for Serviceability limit state

Dealing specifically with the cracking criteria, these have been categorised as:

- D - decompression limit state i.e. No tension
- F - crack formation limit state i.e. Tensile strength limit
- W - crack opening limit state i.e. specified crack width.

and in treating the steel it may be categorised as sensitive to corrosion when it is of small diameter ($<6\text{mm}$) and not sensitive when it is of large diameter ($>6\text{mm}$); equally one can distinguish prestressing and reinforcing steel.

Table 4 shows the suggested treatment for different environments and different combinations.

5. Effects of CEB/FIP in practical design

At first sight, limit state design appears to be totally different from and, perhaps, alien to the established design processes with its "new" terminology and such emotive phrases as "accepted risk of failure". However, with a little study it soon appears as merely a somewhat more formal presentation of what is done by most designers, although implicitly and intuitively, now. As a basis for design it does offer the means of improving both safety and economy; the former, by ensuring more appropriate treatment of the inherent probability aspects in safety with attention directed to the most sensitive aspects; the latter, by a more rational use of the properties of materials in the assessment of the strength of sections and, more importantly, by ensuring that the criteria governing performance are looked at more rigorously than in the past.

Much depends on the way in which any code, based upon limit state concepts, is put together. In this respect, experience in a number of countries suggests that, after an initial antagonistic reaction (because it is not the same as the old code and must be studied), it then becomes accepted and assimilated probably more rapidly than has been the case in the past.

SP 59-2

LOADINGS--CEB APPROACH

By Henri Mathieu

Synopsis: This paper is a summary of an important chapter of the CEB Safety Manual, published in 1975. It consists in a review of the general problems met about loads in the management of structural safety. It deals with the different qualitative classifications of actions, their idealizations and representative values. Other problems (e.g., combinations of actions specific problems during erection) are dealt with in other chapters of the manual.

Keywords: building codes; buildings; loads (forces); reinforced concrete; safety factor; structures.

Henri Mathieu was a cadet in the Ecole Polytechnique of Paris, and was promoted in 1948 Ingénieur des Ponts et Chaussées (member of the French Corps of Civil Engineers). He spent 13 years as a resident engineer in Normandy, where he built many roads and bridges, and an important part of the U.S. Air Base of Evreux. For 13 years he has been the head of the Bridge Division in the French Ministry of Equipment, and held responsible for the standard bridges and the control of design. Since 1972 he has also been technical co-director of CEB.

Although the subject "loadings" is only a small part of the general problem of safety of structures, its scope is very broad; different actions are being studied by different specialists. Thus, this paper can only outline the investigations carried out by CEB in this field.

1 - In Europe, although loadings to be introduced into the calculations are usually unified at a national level, they vary from one country to another. A study by Dr. M. Stiller⁽¹⁾ shows that depending on the country:

- in the best case (dwelling rooms), such loads range from 150 to 200 kg/m²;

- in the worst case (hospital passages), they range from 150 to 500 kg/m².

These differences are obviously not consistent with a unification of safety factors, nor, in general terms, with any optimization of safety levels. As no international agreement can be contemplated without a search for a socio-economical optimum, the 1970 CEB International Recommendations for the Design and Construction of Concrete Structures,⁽²⁾ dealing only with design regulations, could lead to a harmonization of European design regulations, but not to a unification.

2 - Four years ago, five international technical associations established in Europe the Joint Committee for Structural Safety under the chairmanship of Mr. J. Ferry-Borges. American and Canadian specialists have participated in almost all its meetings. Basically, this Committee is responsible for the study and development of the general concepts of safety. However, it was very soon realized that

the various concepts, either semi-probabilistic or probabilistic, could not be applied in practice if the necessary data regarding the distributions of actions were not available.

Thus an important part of the activity of the Joint Committee, under the personal leadership of Mr. Ferry-Borges, was dedicated to a study of the general features of some ten particularly common actions. These studies were published as "Basic Notes on Loads"⁽³⁾ including, for each action studied, an idealization of the distribution as a sufficiently complete probabilistic model, even involving the dynamic aspect of certain actions. Those models may be used:

- either directly, by probabilistic methods,
- or to deduce representative values, to be used by semi-probabilistic methods.

3 - Finally, CEB itself, in association with the CECM issued, from 1973 to 1975, a "Manuel de sécurité des structures" (Principles of structural safety),⁽⁴⁾ intended to constitute one of the chief bases of the 1977 proposed CEB International Recommendations. This manual is a general study of semi-probabilistic methods, aimed at establishing practical rules. Each problem is analyzed first from a theoretical viewpoint and is then studied numerically using available physical data. As regards the actions, the essential bases are the models recommended by the Joint Committee.

The chief parts of the Manual, as regards loadings, are:

- Chapter 6, with 55 pages of analysis of the general properties and representation of actions;
- Chapter 7 (combinations of actions), Chapter 9 (acting load effects), and Paragraphs 12.4 to 12.6 (safety during construction).

4 - This paper will not deal with the latter chapters, which involve applications and some particular cases. The reader is referred to the appropriate CEB bulletins.

4.1 - The first problem dealt with in Chapter 6 is that of the definition of individual actions. What is an action? This is not a matter of philosophy, but a fundamental question for the study of action combinations. The answer can only be practical and approximate.

An action is a group of forces applied to a structure, which can be statistically studied with sufficient approximation as a whole,

independent of other groups. Such a possibility usually results from the existence of a common cause for these forces, but this cause is only the result of an often subjective and rough interpretation. In fact, we are often led to group several elementary actions into a composite one; for instance, the loads induced on floors by furniture and people, and even the loadings exerted on several stories.

A support reaction must be considered, according to the design problem, either as a part of the applied actions, or as an action-effect. Such distinctions are necessary for the determination of the mode of application of safety factors in the case of an element of a structure, or when a structure supports another structure (for instance, the abutment of a bridge).

4.2 - Among the various possible classifications of actions, one appears fundamental for setting safety rules : the classification into permanent (G), variable (Q), and accidental (A) actions.

Actions to be considered as permanent are those which, in service, present one of the following characteristics:

- either their mean values do not vary significantly with time;
- or their magnitudes change only rarely in the course of time and never for short periods;
- or, finally, they vary continuously, always in the same sense, up to a finite limit.

Actions which fulfill none of those requirements and display frequent and significant variations, thus permitting a statistical study, are the so-called variable actions.

The definitions of permanent and variable actions are different during construction.

This classification is only approximate; some intermediate cases are not dealt with, for instance the weight of mobile partition walls in buildings. But, whatever the classification made, many permanent actions exist, and the properties of some of them are very different from those of the dead weight of structures.

Finally, we consider as accidental those actions with a low probability of occurring with significant values for a given structure; they cannot therefore be statistically studied with accuracy. A typical example is the action induced by the impact of a vehicle on a structure.

Intermediate cases exist between variable and accidental actions also; for instance, an earthquake may be variable or accidental, according to the region. There exist intermediate cases even between permanent and accidental actions; for instance, some types of soil collapse.

This classification has many consequences which affect the following notions:

- 1) the definition of representative values
- 2) the combination of actions
- 3) the γ_F factors (load factors)

Differences result from the following general properties: the nature of the statistical variability is different for G and Q, notably as regards the "time" dimension; data are quite limited for A. Moreover, as a rule, the coefficient of variation is rather small for G, medium for Q, and very high for A; as a consequence, the costs of safety vary widely for the three classes of actions, and the optimum decision cannot be equal safety for all three.

However, there are exceptions to these properties, for instance:

- the thrust of soil is a permanent action, but its coefficient of variation is often rather high;
- the maximum water pressure in a tank is a variable action, but its coefficient of variation may be zero.

Such exceptions may justify some modifications of safety rules or of the numerical values of coefficients.

4.3 - Actions can be classified on the basis of many other criteria; the most important are the following:

- 1) Static or dynamic character of actions; certain dynamic actions must be treated by very different methods in which a structure's own vibration periods play a very important role.
- 2) Variable or accidental actions resulting from a stationary or nonstationary process; when a process is not stationary, the application of statistics must usually be completed by additional investigations; moreover, the safety level varies with time.

Of course conventional regulations are set up for static actions resulting from a stationary process.

3) Dependence or nondependence upon human activity; the properties of actions depending upon a large number of persons are similar to those of natural actions, though often multimodal. Extreme values of the distributions of actions depending upon a small number of persons result from human errors. For both types, taking the various parts of their distributions into account raises contractual and juridical problems. Finally, it may be necessary to define such actions through nominal values without any statistical basis.

4) Dependence or nondependence upon structural response; there are several types of actions dependent on the response of a structure, for instance, dynamic actions, imposed deformations, etc. For such actions, measurement of the forces exerted on a structure must be distinguished from the measurements of their causes.

5) Continuously or intermittently applied actions; when an action is applied intermittently, it may be necessary to introduce a parameter measuring the relative duration of the absence of the action.

6) Bounded or unbounded actions; when a variable action is bounded and its boundary is attainable, it may happen that the safety factor with respect to this boundary must be reduced by about 10%; this brings the safety factor back to values of the same order as those applied to permanent loads.

7) Degree of "freedom"; a so-called free action is an action exerted on two parts of a surface independently (for instance, on a floor). Live loads are usually more or less free; and, more generally, freedom is rarely total. Taking the freedom of loading into account is a very difficult, only partly studied problem. In fact, a free action is a composite action that cannot be measured by means of a single parameter. In practice, to deal with these actions, one introduces the supplementary notion of "load case" based essentially on the use of influence surfaces.

4.4 - This enumeration, although incomplete, shows that the study of actions involves difficulties which are too numerous and too varied to permit a single theory.

The studies of the Joint Committee are separate investigations of the most common actions. Each of these actions involves only a small number of particular details, thus permitting their study. Such actions can be represented by a single random parameter, or reduced to this case by means of more or less rough assumptions. On the

other hand complementary specific idealizations were defined for the dynamic aspects of wind and earthquake.

However, relatively often engineers will encounter actions presenting peculiarities which are not considered in the examples dealt with by the Joint Committee. For instance, when a dam retains a water volume of variable level, problems of nonlinearity appear: shear forces and bending moments in the dam are second-order and third-order polynomial functions of the level of water and the level of the dam section studied.

4.5 - In the simple or simplified case of an action which can be represented by a single random parameter, establishment of a representative model of the parameter leads to two types of questions: What are the parameter properties to be represented? How will they be represented?

The simplest model for representing a variable action gives only the distribution of the periodic maxima. Such models are insufficient for the study of action combinations, as the duration of each individual action application is a major element of such a study.

The simplest approach to completion of the model, is to ascribe a fixed value (elementary interval) to the duration of an individual application. This is the procedure adopted in the "Basic Notes on Loads" of the Joint Committee. However, more sophisticated models were worked out and used both in the "Manuel de sécurité" and "Basic notes."

4.6 - In development of semi-probabilistic methods, the models can be used to determine representative values of the actions. They could also be used in more probabilistic methods.

5 - The representative models of the actions defined by the Joint Committee have not yet been definitively published. However, they proved to be very useful for the "Manuel de sécurité"⁽⁴⁾ and are now being used in ISO work.

In January 1976, I prepared a preliminary draft of Appendix 2 of Volume I, CEB Bulletin 111,⁽⁵⁾ to present the essential data contained in the "Basic Notes," and link them to the provisions of CEB Bulletin III.⁽⁵⁾ The essential purpose of this appendix is to permit trial applications of the regulations in the Bulletin, and to decide on the simplifications to be made.

5.1 - These models can be divided into two types: those representing natural actions (chiefly S_n , W , E_q), and the exploitation load

models (dwellings, offices, stores, etc.). Besides the models and the representative values which can be deduced, Appendix 2 also contains general indications for each class of action, some particular rules for each class, and indications concerning other actions (temperature, water pressures, various types of live load, etc.).

5.2 - The principal difficulty met in studying natural actions is of a practical nature. It is that the relevant statistics can only be obtained after many years. Data gathered from different places add to knowledge, but they also raise new difficulties and uncertainties. In the best of cases, values with average return period greater than 100 to 200 years cannot be known with sufficient accuracy.

In the case of exploitation loads, difficulties result mainly from the following causes:

- statistics concern only the principal uses of buildings;
- loads are in most cases more or less free;
- they are often the sum of elementary actions (for instance, furniture and concentration of persons), the properties of which differ greatly.

As a consequence, in the range of extreme values, the distributions mainly result from changes in the use of buildings; they are multimodal, and do not correspond to statistical extrapolations. Moreover, it is difficult to apply the statistics for a given period and to use them to study the loading distribution on a surface of complex shape. Therefore the establishment and use of the "Basic Notes" requires certain hypotheses which can be used to generalize the available statistical information.

5.3 - Idealizations of the distributions of periodic maxima of the various actions were proposed by small groups of specialists, directed by Mr. Ferry-Borges in the context of the Joint Committee. In most cases, they are extreme value laws of type 1.

Subsequent discussions, notably by Mr. Delesques (CECM), led to the belief that such laws are valid up to return periods of 100 to 200 years, but that beyond this limit, their validity is completely unknown. As a consequence, the results of probabilistic methods are not only conventional, but completely hypothetical since they result from "curve tails" which, in fact, are completely unknown. We are thus led to pose the question: is it an improvement to base safety on pure assumptions?

In fact, the laws of extreme values of type I were adopted primarily because of their practical advantages: in particular, they permit the transfer of distributions from one time interval to another without difficulty.

Numerous complementary idealizations are proposed in the "Basic Notes." Some of these introduce a reduction of exploitation loads as a function of the loaded surface and number of stories, others introduce concentration factors, which permit replacement of actual loads by equivalent distributed loads.

Finally, other problems are more briefly studied, as for example:

- the spatial distribution of snow on a roof, of wind pressure on a surface with any shape, both by means of constant (deterministic) coefficients.

- The directional distribution of wind and earthquake; a relationship can be found between the average return periods of omnidirectional distributions and the distributions of pressure in a single direction; but if this correspondence is a simple relationship between these periods, its determination raises some problems, as the distribution is not uniform in all directions and the magnitude of wind action on an inclined surface is not a mere component, but a function of the angle of inclination, which results from the wind's aerodynamic properties. According to the most recent investigations, this ratio can be taken as 2.5 which leads to a characteristic value of the wind in one direction with a 50-year return period, instead of about 125 years for all directions.

6 - For the study of limit states, especially for concrete structures, and for the combination of actions, several representative values have been defined for each variable action. The corresponding terminology is not yet definitively established, and simplifications will perhaps be made.

6.1 - Presently, those values are as follows:

- characteristic values F_k , for the study of the ultimate limit states;

- serviceability values, F_{ser} , for study of the limit states of crack formation and deflection; When the coefficient of variation of the maximum in 50 years is small enough, they are equal to the characteristic values;

- frequent values $\psi_1 F_k$ for study of the limit states of crack width; usually, they are the values exceeded about 5% of the time;

- combination values $\psi_0 F_k$ and quasi-permanent values $\psi_2 F_k$ included in the formation of action combinations, which are sets of representative values for several actions. Usually, combination values have average return periods of about 30 years, and quasi-permanent values represent the average value with time.

Moreover, after a recent study, we have been led to distinguish the long-term and short-term parts of these various representative values for concrete creep analysis.

The ψ factors are ratios of two representative values; they are introduced to facilitate presentation, but differ from one action to another, and cannot be generalized.

All representative values have different magnitudes during construction and in service. Finally, one can be forced to replace them by nominal values, as in the case of accidental actions, when statistical data are not available; for instance, in the case of live loads of an industrial building when the final utilization has not been determined.

6.2 - Choosing the characteristic values of variable actions was a difficult problem for CEB and other international associations. In the end, an essentially pragmatistical method was thought better than an a priori determined fractile. This proposition was submitted to the international enquiry of 18.9.74, and after improvement following the results of the enquiry, it now constitutes Appendix I of Volume I of Bulletin 111.⁽⁵⁾ I shall only give a brief outline.

Theoretically, characteristic values are deduced from the distribution of the maximum in 50 years, the so-called "reference period" which represents slightly less than the average lifetime of a structure. They are used to provide a homogeneous basis for the distribution functions of variable and permanent actions.

When the coefficient of variation of this distribution, according to the idealization adopted, is not notably greater than 0.2, an average return period between 125 and 200 years is chosen for the characteristic value, according to whether the idealization used seems to be more or less pessimistic with respect to reality.

When the coefficient of variation of this distribution is notably greater than 0.2, the characteristic value is chosen to have a higher average return period, such that the probability of reaching $1.4 F_k$

is thought to be negligible, the idealization adopted and the various data available concerning the possible extreme values being taken into account. The negligible value is estimated by judgement taking into account the economic consequences of the choice.

Those rules were chosen so as to consider values close to the practical limits of extrapolation of the statistics, and to be able to apply uniform values of the γ_F factors, whatever the coefficient of variation.

For wind, when the structure is mainly sensitive to wind in a single direction, the 50 year average return period is taken as the characteristic, all directions being combined.

7 - For permanent actions, theoretical calculations carried out to choose the safety factors indicate that the coefficient of variation is lower than 0.1, and that the characteristic values are 5% fractiles. It is normal that safety be slightly greater for permanent actions than for variable actions, as it is not expensive. It would be useless to determine other representative values.

In practice, as distributions are not well known, one chooses as representative values for most permanent actions, one minimum and one maximum value, corresponding to the maxima and minima which can reasonably be anticipated. In fact, they are nominal values, without statistical basis.

In the case of self weight, which is only one of the permanent actions to be considered, where the coefficient of variation is very small, it was agreed to use a unique representative value, which is the average value.

For accidental actions, whose dispersion is always very great, it is only possible to choose nominal values, replacing the extreme values $\gamma_{F1} F_k$; the other representative values are considered negligible.

8 - In this paper, I have only been able to present a very brief outline of studies which would require at least 200 pages to describe completely. Most of the studies have already been published in CEB Bulletins 106, 107, and 111. (4) (5)

These documents also contain all details concerning the background of action combinations and design action-effects.

In conclusion, I would like to emphasize the fact that all these studies have been carried out in accordance with the usual CEB

approach, that is, to stay as close as possible to physical reality, mindful of the fact that reality cannot be represented by one single mathematical formula.

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SP 59-3

SAFETY AND SERVICEABILITY PROVISIONS IN THE ACI BUILDING CODE

By George Winter

Synopsis: The development of the safety provisions of the ACI Building Code is traced through the four most recent editions, since 1956, with special emphasis on the present load and resistance factor (LRF) format and its relation to more rigorous probabilistic approaches to structural safety. Serviceability provisions regarding cracking and deflections at service loads and their background are also discussed.

Keywords: building codes; cracking (fracturing); deflection; load factors; reinforced concrete; safety factor; serviceability; structural design; ultimate strength method.

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Introduction

In the last three editions of the ACI Building Code, those of 1956, 1963, and 1971, the provisions for safety and for serviceability have undergone a radical evolution. During that same time, two other far-reaching changes have occurred, which have in turn affected the safety and serviceability provisions.

The first of these is the tendency toward the use of higher strength materials. Prior to 1956 the usual stipulated cylinder strength of concrete was 2,500 to 3,000 psi (17.2 to 20.7 MPa) with 3,750 psi (25.8 MPa) used in special cases. But, by 1971 the usual concrete strengths ranged from 3,000 to 4,000 psi (20.7 to 27.6 MPa) and concretes up to 6,000 psi (41.4 MPa) are no longer exceptional. Similarly, before 1956 the stipulated minimum yield strength of reinforcing bars was 33,000 or 40,000 psi (227.5 or 275.8 MPa). By 1971 the use of 60,000 psi (413.7 MPa) yield strength steel had become practically universal, with 75,000 psi (510.2 MPa) steel employed under appropriate conditions.

The second relevant change consists in the approach to calculating the strength, or allowable load, on reinforced concrete members. Prior to 1956 the carrying capacity of members was computed on a semi-elastic basis: that is, linear relations between stress and strain were assumed for both steel and concrete, with some exceptions, such as for axially loaded compression members. In contrast, by 1971 the dimensioning of members was based on the recognition of the inelastic behavior of both concrete and steel at loads approaching the ultimate strength of the member.

The development of the safety and serviceability provisions is intimately connected with these two basic changes.

Safety Provisions

In current design codes or specifications, safety provisions are formulated in one of the following three formats:

1. Allowable stress format. (AS-format)

In general terms, in this method some property of the cross-section, S , is multiplied by an allowable stress f_{all} . For a member to be safe, this quantity must be equal to or greater than the pertinent load effect or generalized force F (moment, shear axial load, etc.). Here, F is calculated for the stipulated design or service load L_{des} , i.e. $F(L_{des})$. Symbolically, then,

$$Sf_{all} \geq F(L_{des}) \quad (1)$$

2. Load factor format. (LF-format)

Here one calculates the expected (nominal) ultimate strength or resistance of the member, R , based on the best available experimental and analytical information. For a member to be safe, this ultimate strength must be equal or greater than the load effect F caused by the various types of design loads, each multiplied by an appropriate load factor γ . Symbolically, then,

$$R \geq F(\sum \gamma_n L_{des,n}) \quad (2)$$

Here the various load factors $\gamma_n > 1$ provide the required margin of safety. Their magnitudes can be adjusted to reflect the differing character and variability of the various types of loads (self-weight, occupancy loads, wind, etc.), which is not possible in the AS-format.

3. Load and resistance factor format. (LRF-format)

In this format, it is explicitly recognized that both the actual resistance of a member and the load effects acting on it are, in fact, variable quantities which differ in random ways from their nominal values R and $F(L_{des})$. In the LRF-format, the possibility that the actual resistance may be less than its nominal value, and the actual load effect larger than its design value, are recognized separately by understrength or resistance factors $\phi_m < 1$ and by load factors $\gamma_n > 1$. The desired safety margin is then provided by the condition

$$\phi_m R_n \geq F(\sum \gamma_n L_{des,n}) \quad (3)$$

Here R_n is the particular type of strength or resistance (bending, shear, compression, etc.) and ϕ_m the pertinent resistance factor. This format has the advantage that it lends itself well to the determination of the factors ϕ_m and γ_n by a combination of probabilistic safety theory and calibration against existing designs and structures. The approach known as limit state design, now in use or under development in many countries, is frequently expressed in the LRF-format.

Historically, the safety provisions of the ACI Code have developed as follows:

Prior to 1956: AS-format

1956 edition: AS-format or optionally LF-format

1963 edition: AS-format or alternatively LRF-format

1971 and 1977 editions: LRF-format or optionally AS-format

That is, in the 1971 edition, the basic design provisions are formulated in the LRF-format. Use of the old AS-format is still permitted, but is discouraged, particularly in connection with the new, higher strength materials. This is so because the nominal carrying capacity $S_{f_{all}}$ utilized in the AS-format and mostly calculated on a semi-elastic basis is increasingly inaccurate when applied to today's higher strength materials.

As indicated, the first radical departure from the traditional AS-format occurred in the 1956 edition of the Code. There, in a separate Appendix, equations were given for the first time for the nominal real strength or carrying capacity of members, R , as distinct from allowable capacities $S_{f_{all}}$. The load effects F were calculated for factored loads, namely either

$$F = F(1.2 \text{ DL} + 2.4 \text{ LL}) \quad (4)$$

$$F = F(\gamma_{DLL}(\text{DL} + \text{LL})) \quad (5)$$

Here DL is the dead load caused essentially by selfweight, and LL is the live load essentially caused by occupancy loads. The factor γ_{DLL} was to be taken as 2.0 for columns and 1.8 for beams. Additional provisions were made for wind loading in combination with dead and live loads.

This method, presented in an Appendix, did not gain wide acceptance and practical design continued mostly in the traditional

AS-format. However, this approach, which was known as "ultimate strength design" because of the use of real strength expressions R rather than allowable capacities $S_{f_{all}}$, represented the stepping stone to the introduction of a fully developed LRF-format of safety provisions in the 1963 Code edition.

It was clear that the traditional design using the AS-format could not be suddenly discontinued by simple fiat. Therefore, the 1963 Code contained two complete alternative sets of provisions, one in the old AS-format and one in the new LRF-format, the use depending on the designer's preference.

At the time when that edition was in preparation, it was already realized that it is illusory to try designing for absolute safety. Considerable work was underway by various individuals and institutions toward the development of a theoretically consistent probabilistic approach to structural safety, but no useable method had yet been devised. Thus, the subcommittee in charge of formulating the safety provisions, of which the writer was chairman, had to develop its own methodology, utilizing whatever useful work by others was available.

It was decided to aim at a probability of failure of the order of $1/100,000$. It was realized that the probability of failure represented a combination of the possibility of strength deficiencies on the one hand, and excess loads or load effects on the other. The aim, then, was to keep the probability of strength deficiency to $1/1000$ and that for excess load effects to $1/100$. The product of these two probabilities would then represent the overall failure probability of the order of $1/100,000$. It is easy to aim at such probabilities, but it is next to impossible to achieve them without adequate statistical data on which to base numerical values of load and resistance factors.

Use was made of data available for at least one source of strength deficiency, namely frequency distributions of the actual strengths of the two materials, concrete and steel, as obtained in large-scale test series. Use was also made of such meager statistics on loads as were available. Combining these scanty data with engineering experience and intuition, and with comparisons with accepted design practice, the subcommittee drafted the following formulation: The factored resistance ϕR (see Eq. 3) was to be computed by means of accepted strength equations, but using materials' strength multiplied, for steel, by the resistance factor $\phi_s = 0.8$ and for concrete by $\phi_c = 0.67$. Thus, in Eq. 3.

$$R = R(0.67f'_c, 0.8f_y) \quad (6)$$

It was stated that only part of the resistance factors ϕ_c and ϕ_y accounted for actual possible strength deficiencies of the two materials. The rest was to provide for other possible sources of under-strength, such as dimensional deficiencies, effects of approximations and simplifications in structural analysis, sub-normal workmanship, etc. It is worth noting that the application of separate resistance factors ϕ to the specified strengths of concrete and steel is now employed in the codes of several European countries.

Regarding load effects F , the subcommittee recognized the greater certainty of dead loads as compared to live loads by corresponding load factors $\gamma_{DL} = 1.3$ and $\gamma_{LL} = 1.5$. This resulted in the load effects (cf. Eq. 3)

$$F = F(1.3DL + 1.5LL) \quad (7)$$

with similar expressions for combinations of wind with other loads. Thus, the basic safety requirement, Eq. 2, became

$$R(0.67f'_c, 0.8f_y) \geq F(1.3DL + 1.5LL) \quad (8)$$

(This brief presentation omits a number of secondary details).

When this draft was submitted for discussion to the Code Committee and the entire membership of A.C.I., it met with considerable criticism. The main objections raised were these: (a) the resistance factors applied to the strengths of concrete and steel could be misinterpreted by builders and others to imply that deficient materials were acceptable in construction provided their actual, tested strength exceeded $0.67f'_c$ or $0.8f_y$, respectively. (b) The significantly smaller value of ϕ_c for concrete than ϕ_s for steel may be exploited in the competition between steel and concrete construction as indicating that concrete is the less reliable material. (c) The effects of approximations and simplifications in structural analysis represented an uncertainty which affects the magnitude of the load effects F rather than of the member resistance R . Hence, they should be included in the load factors γ rather than the resistance factors ϕ . Considering the latter argument, it was decided to re-distribute the assumed probabilities so that a strength deficiency probability of 1/100 and an overload probability of 1/1000 was assumed, rather than vice versa.

On this basis the load factors were increased from 1.3 to 1.5 for dead load and from 1.5 to 1.8 for live load. In addition, individual resistance factors were applied not to the strengths of the two materials, but rather to the type of member or of internal action in the member. That is, different ϕ -values were developed

for flexure, shear, bond, anchorage, and for columns with ties and columns with spiral reinforcement. This way of using ϕ -factors has the advantage that types of member performance and of importance of members for the survival of a structure could be reflected in more detail in the ϕ -values.

Because the safety provisions of the 1971 Code are very similar to those of the 1963 edition, only the presently used values will be presented. There is one substantive difference between these two editions: in 1971 the load factors were lowered slightly, from 1.5 to 1.4 for γ_{DL} and from 1.8 to 1.7 for γ_{LL} . The Commentary states that this was done because of the more comprehensive provisions of the 1971 Code, improved concrete and steel quality control, and satisfactory experience gained with the LRF-format.

In brief, then, the safety provisions of the ACI Code utilize the LRF-criterion given by Eq. 3 with the following values:

Loading	Load factors γ		
	γ_{DL}	γ_{LL}	γ_W
dead plus live	1.4	1.7	
dead, live and wind, additive	1.04	1.275	1.275
dead and wind, counteracting	0.9		1.3

Type of action or member	Resistance factor ϕ
flexure	0.90
compression members with spirals	0.75
compression members with ties	0.70
shear and torsion	0.85
bearing on concrete	0.70
bending in unreinforced concrete	0.65

Except for minor adjustments, no significant changes of these factors are anticipated for the next edition of the Code.

It is evident from this history of these safety provisions, that the present numerical values of the factors ϕ and γ are based only

to a minor degree on objective statistical evidence. Such scarce data as existed at the time have been utilized, but the final values were largely determined by engineering judgement and calibration against established design practice and existing structures. In the foreseeable future, however, this is almost certain to change because, during the years when these provisions were in effect, great strides have been made in the development of rigorous, yet practically useable probabilistic approaches to structural safety.

It is fortunate that the method farthest developed in this country, the so-called first order, second moment theory, can be made to result in a criterion of the precise form of Eq. 3, i.e. the LRF-format. When statistical evidence is available on strengths, loads or other relevant quantities, this theory makes use of only two distribution characteristics: the mean value and the coefficient of variation. The safety criterion for the limit state k (e.g. flexure, shear, what have you) due to the load combination j can then be written as

$$\phi_k R_{nk} \geq \gamma_o \left(\sum_{i=1}^m c_i \gamma_i L_{ni} \right)_j \quad (9)$$

where R_{nk} is the nominal resistance for the state k , γ_o is an analysis factor of the order of 1.1, representing the effect of approximations and simplifications in the analysis, and c_i is a deterministic factor by means of which the factored nominal load $\gamma_i L_{ni}$ is transformed into the pertinent internal force (i.e. moment, shear force, etc.) According to the second moment theory, the resistance factor and the load factor are:

$$\phi_k = \frac{R_{mk}}{R_{nk}} e^{-\alpha \beta V_R} \quad (10)$$

$$\gamma_i = \frac{L_{mi}}{L_{ni}} e^{\alpha \beta V_L} \quad (11)$$

Here the resistances R and loads L when subscripted m are the mean values of the statistical distributions, and when subscripted n are the nominal design values (e.g. codified loads), α is a numerical quantity of no particular interest in this abbreviated presentation, and V_R and V_L are the coefficients of variation of the particular type of resistance or load, respectively. β is the safety index, which is of the order of 3 to 4 for a probability of failure of about 1/100,000. The same range for β is obtained by calibration against customary design practice.

Consequently, once one has settled on the desired safety index β , it is only necessary to obtain sufficient statistical information

on different types of strength (resistance) and of loads, so that the mean values and the coefficients of variation can be determined. Having these, the pertinent ϕ and γ can be calculated rather than being determined mostly by judgement. In regard to resistance, a considerable amount of such work has already been done. The Reinforced Concrete Research Council is in the process of instituting a major research project aimed at establishing resistance factors ϕ chiefly in the above context. Likewise, a Subcommittee on Load Factors of the Committee on Building Code Requirements for Minimum Design Loads in Buildings and Other Structures of the American National Standards Institute, ANSI A58.1, has recently been organized to develop, in the same context, load factors valid for all types of construction, not just for reinforced concrete.

As this much abbreviated and simplified presentation indicates, future safety provisions will be based to a much larger degree than now on objective evidence and on rational criteria, even though engineering experience and judgement will continue to play an important role in this area.

Serviceability Requirements

Once adequate safety against structural distress has been achieved in a design, it is still necessary to ensure that the performance of the structure will be satisfactory at service loads. In most cases this means that the magnitude of deflections and of tension cracking of concrete must be adequately controlled. Control of deflections, important in itself, also furnished some indirect control of undesirable vibrations.

Other things being equal, deflections as well as widths of flexural tension cracks at service loads are approximately proportional to the steel stress. In earlier Code editions through 1956 there was no need for serviceability requirements because fairly low steel stresses at working loads were obtained in low and moderate strength steels in conjunction with the AS-format. However, the general use of higher steel strengths, in conjunction with the LRF-format, resulted in sharply increased steel stresses at service loads so that control of member performance under working conditions became important. This is why the 1963 edition, for the first time, contains provisions for control of deflections. These have been retained and expanded in the 1971 edition together with new provisions for limiting the width of flexural tension cracks.