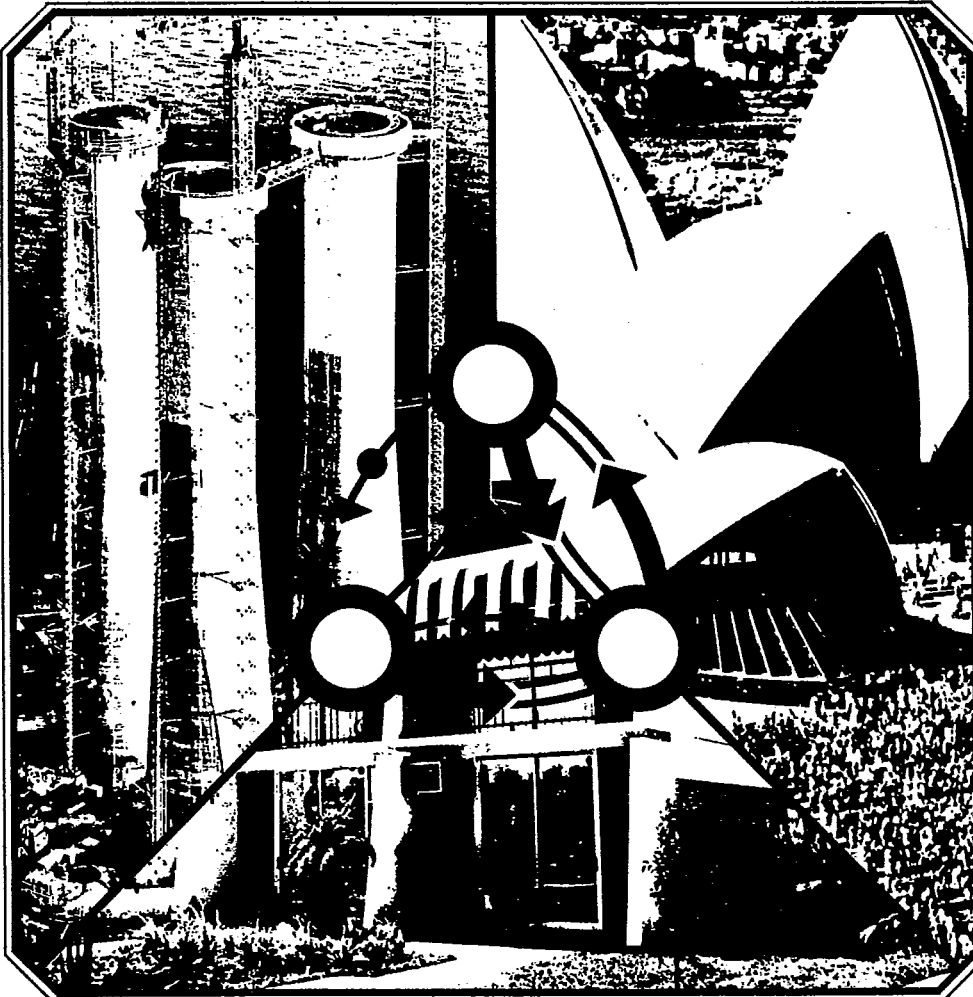


SIGNIFICANT DEVELOPMENTS IN ENGINEERING PRACTICE AND RESEARCH

A Tribute to Chester P. Siess



Publication SP-72

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Mete A. Sozen
Editor



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PREFACE

The "Three Sessions to Celebrate C. P. Siess' Contributions to Research, Education, and Practice," were organized at the request of Dr. Siess' students and friends by a committee on which J. H. Appleton, Eivind Hognestad, Armas Laupa, and M. A. Sozen served. The task of the committee was made much easier because of the enthusiastic and generous support of the American Concrete Institute staff, the Board, and the Technical Activities Committee as well as all those who participated in the event on 1 November 1979 in Washington, D.C.

Appreciation is due the session chairmen who were Dr. E. Zwoyer, Dr. Armas Laupa, Professor C. E. Kesler, Dr. E. O. Pfrang and Mr. J. R. Gaston. Because of the topical contents of their presentations, Dr. R. Woodring and Dr. R. D. de Cossio considered their material to be of temporal interest and elected not to have them included in the proceedings. Their contributions to the success of the session on education are gratefully acknowledged. Dr. Hognestad deserves praise for his penetrating as well as entertaining review of Dr. Siess' career. It is a pity the warmth, wit, and nuances of his delivery cannot be captured on paper. All participants are indebted to James Barry for the memorable party he organized for "friends and enemies of C. P. Siess" to top the day's activities.

Mete A. Sozen



Dedicated to
Chester P. Siess

CHESTER P. SIESS

Dr. Chester Paul Siess was born in Alexandria, Louisiana, to Leo Chester and Adele Liebreich Siess. He attended Rosenthal Grammar School and Bolton High School in Alexandria, where his father was a pharmacist. He continued his education at Louisiana State University in Baton Rouge and was a member of the ROTC. Huey Long was still an active patron of the LSU campus at that time. Siess had the highest scholastic record in the engineering school at the time of his graduation in 1936 with a bachelor of science degree in civil engineering. He was named the Outstanding Civil Engineering Graduate by the Louisiana Section of the American Society of Civil Engineers.

Joining the profession in a depression year, Dr. Siess's first job was that of Party Chief in the Rural Road Inventory program of the Louisiana Highway Commission. For almost half a year he documented the layout and condition of roads in several parishes. Organization of a soil testing laboratory within the Highway Commission brought him there as a Soils Engineer.

In September 1937 he started his graduate studies at the University of Illinois, Urbana, having been awarded a half-time appointment as a Special Research Graduate Assistant in the department of Theoretical and Applied Mechanics, working on a research project with Dr. N. M. Newmark, Dr. V. P. Jensen, and Prof. F. E. Richart. His M. S. thesis, which he completed in 1939, formed the nucleus of the University of Illinois Engineering Experiment Station Bulletin No. 336, "Moments in I-Beam Bridges," published with him and N. M. Newmark as the authors.

In June 1939 he joined Dr. R. B. Peck in the Chicago Subway Soils Laboratory. When the subway construction was completed in April 1941 he started working for the New York Central Railroad in Chicago where he was an engineer-draftsman in the Bridge Office. In Chicago he met Miss Helen Kranson from Marshall, Texas, who had gone there to work after a year's stint as a Freshman in Urbana.

In September 1941, Dr. Siess returned to Urbana as a Special Research Associate in Theoretical and Applied Mechanics. He was put in charge of experimental work, replacing Prof. Ralph Kluge who had taken a position at the University of Florida. He married Miss Kranson on 5 October 1941. Having completed his doctoral dissertation in 1948, in the course of which he developed a moment distribution procedure for two-way slabs, he joined the department of Civil Engineering in Urbana in 1949. By that time Judy, their daughter was born.

In the 1950's, the Structural Engineering Laboratory of the Department of Civil Engineering in Urbana was the scene of a number of new thrusts in experimental research on reinforced concrete. The fundamental research on prestressed concrete and reinforced concrete not only helped transform the basis of design in these materials but also helped train the professionals who would provide leadership for further developments and applications in the use of prestressed and reinforced concrete as a structural medium. The profession has recognized Dr. Siess's stewardship of this strong stream of research through a series of awards. His ASCE Ernest E. Howard Award (1968) citation reads, "Through his extensive research in reinforced concrete and prestressed concrete, the translation of his research results, his informative writings, his effective teaching and stimulation of students and coworkers, Chester P. Siess has made significant contributions to Structural Engineering, with worldwide influence."

In 1949 he received, jointly with N. M. Newmark, the ACI Wason Medal for their paper on Reinforced Concrete Slabs. For his contributions on the same subject ASCE awarded him the Research Prize (1956) and the R. C. Reese Award. His achievements on research related to understanding behavior toward design brought him honors from the Concrete Reinforcing Steel Institute (the CRSI Award 1956), the American Concrete Institute (the Turner Medal, 1964), the Reinforced Concrete Research Council (A. J. Boase Award, 1975), and the National Academy of Engineering (elected to the Academy in 1967). He is an honorary member of the American Concrete Institute (elected in 1969) and the American Society of Civil Engineers (elected in 1970).

He has served the American Concrete Institute in a series of technical-committee assignments and as its president (1974-75). He was the Chairman of the Reinforced Concrete Research Council of the American Society of Civil Engineers during the period 1968-1981.

Since 1968 he has been a member of the Advisory Committee on Reactor Safeguards (U.S. Nuclear Regulatory Commission) which he served as Chairman in 1972.

He is currently Chairman of ACI Committee 318 which is charged with the task of preparing the next version of the Standard Building Code.

The University of Illinois has been blessed with a number of great teacher-researchers in structural engineering over the years. The shadows that men like Cross, Newmark, Richart, Wilson, Westergaard and Talbot cast are awesome. The most important recognition Chester Paul Siess has received is the firm belief of those students who have known him in the classroom and in endless discussions in his office (where if the proverbial pistol of his argument missed fire he beat you down with its butt end) that he belongs in that select class.

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OPENING REMARKS

By Joseph H. Appleton

A tribute to Chester Siess could take many forms--a recitation of his accomplishments, a discussion of his teaching and research methods, a look at his participation in technical committee work, or an examination of his interaction with his students and associates. The tribute that we are mounting today will be recognized as one only if the papers presented are judged to be up to Chester's standards. For the papers are principally concerned with work performed by the authors who were students or associates of his. Unless they digress from their topics, they will not touch on many of the more important things we experienced with Chester. Students quickly learned that they were a part of his family--being with Helen and Judy was particularly enjoyable. They found that ways were found for them to attend ACI meetings. How many of us have followed through on that point? Both associates and students found Chester to be open, inquisitive, and receptive. No development of his ever seemed new; because he had tried the ideas out on his students and associates long before they were published or formally presented. You couldn't steal one of his ideas, because he would give it to you if you would pursue it. I could go on and on with glimpses of my 32 years of knowing and associating with Chester. But most of what I would say would be the same that most of you have experienced another day, another time. Thank you for coming to share this day with Chester. He probably feels that it is not so much his day, but your day and my day.

SESSION 1 RESEARCH



SP 72-1

Load and Resistance Factor Design

By Ivan M. Viest

Synopsis: Load and Resistance Factor Design is a major advance toward rational design of steel framed buildings. It combines the limit states of strength and serviceability with a modern probability-based approach to structural reliability. After a historical introduction, the method is defined and a generalized LRFD format developed. The discussion centers on sources of variability in design, on limit states and on probabilistic concepts underlying the numerical values of load and resistance factors. The results of a design evaluation of LRFD are presented and steps are described that are being taken toward future adoption of the method into the AISC Specification.

Keywords: buildings; limit state design; load factors; loads (forces); probability theory; reliability; resistance factors; safety; serviceability; structural design; structural steels.

2 Viest

ACI Fellow Ivan M. Viest chaired the advisory committee on AISI research project at Washington University that developed the Load and Resistance Factor Design. He holds engineering degrees from Slovak Technical University, Georgia Institute of Technology and the University of Illinois. His experience has included engineering design, construction, teaching, consulting, extensive research studies and management. Currently, he is Assistant Manager of the Sales Engineering Division of Bethlehem Steel Corporation.

Two major structural developments of the last 30 years will have a profound influence on future design: (1) extensive experimental and analytical studies of structural members and connections and (2) probabilistic studies of the safety and reliability of civil engineering structures.

The first development has already resulted in a quantum improvement in the understanding of structural response to loading. Numerous design changes were adopted during the past 20 years. Structures designed today are lighter and more economical than the designs of - say - 1950 vintage.

The second development is less well-known. Probabilistic approaches to structural safety are relatively new. Over the centuries, structural safety was achieved empirically - through trial and error. Present factors of safety seem to have originated with the use of iron in construction, as described in the following quotation from Professor Pugsley's book, The Safety of Structures: (1)

"When one's personal safety is ... at stake in, say, a surgical operation, it is natural to ask that the operation shall be done by a man who has done it many times and with uniform success. And if the operation is an unusual one ..., then it is all the more desirable that the surgeon should be both of high repute in related work and known to be a fine practitioner ... The same sort of approach was from the earliest times applied to the safety of major structures. One sought out ... a designer of proven success ... a Stephenson or a Brunel was chosen and he could do no wrong. But as accidents nevertheless occurred, these leading engineers were forced to call in each other to investigate each other's mistakes, and so there grew up a system of accident inquiries or investigations ... progressively associated with some agent of Government, in the early days the Board of Trade.

"Thus it came about that with the general introduction of wrought iron in structures some officials of the Board of Trade became knowledgeable on matters affecting structural safety ... and began to recommend, and later insist upon,



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certain design features making for safety. In particular ... the Board, in about 1840, insisted upon a limiting working stress in wrought iron of 5 tons/in.², the forerunner of many such limitations to come ...

"The limit ... for wrought iron corresponded to a factor of safety of at least 4, and set a standard that was promptly applied to mild steel structures when they came into general use ...; hence the limit of 7-1/2 tons/in.² ... set by the first London County Council rules for the use of steel in building construction."

Structural engineers have been working with factors of safety ever since. They have adjusted their values from time to time on the basis of increased knowledge and both good and bad practical experience.

A radical change was brought about by the aircraft industry during World War II. The leaders in aircraft design realized that many design parameters - such as material properties and loads - are random. Starting from this premise, these leaders began collecting statistical data for use with mathematical theory of probability in a rational approach to the question of safety.

Two outstanding civil engineers, Professors Freundenthal in the United States and Pugsley in Great Britain, took part in this work. Shortly after the war they introduced these concepts to civil engineering by creating two committees - one in ASCE, one in ICE - to investigate ways of using probabilistic concepts in the design of civil engineering structures.

Both committees completed their work in the mid-50's. Freundenthal's paper, Safety and Probability of Structural Failure, (2) is considered by many the most basic and definitive paper on probabilistic design.

As is often the case, these early studies brought out many new concepts that departed from traditional norms. They also indicated that existing statistical evidence was too meager to permit early adoption of these methods by the practitioner. Another decade of intensive effort followed. Not until the late 1960's did these developments reach a point where one could see the implementation of these new concepts into practical design.

By that time the American Iron and Steel Institute completed the development of load factor design for steel bridges, and then embarked on similar work for steel buildings. A contract was awarded in 1969 to Professor T. V. Galambos of Washington University at St. Louis. In consultation with his advisory committee, Galambos decided to base the new method, the Load and Resistance Factor Design for steel buildings, on probabilistic concepts. The method was first published by AISI in January 1978 as Research Bulletin No. 27. (3) Shortly after, it was described in a general



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manner in three papers published in the First Quarter 1978 issue of the AISC Engineering Journal. (4-6) A more detailed treatment was presented in a series of eight papers in the September 1978 issue of the ASCE's Structural Division Journal. (7-14)

Independently of these studies, Committee A58 of the American National Standards Institute has been working to place design loads on a consistent statistical basis. These improved load standards will be included in the revision of A58.1-1972 (15) scheduled for 1980. Beyond 1980, a subcommittee established to develop load factors common for all materials is reviewing a proposal drafted by Drs. Cornell, Ellingwood, Galambos and MacGregor.

LRFD Defined

First, what is load and resistance factor design (LRFD)?

Load and resistance factor design is a method of proportioning structural members and connections so that the strength and serviceability limit states are greater than the corresponding factored load combinations.

This definition can be illustrated with the limit state of fracture at a net section of a tension member, under a combination of dead and live loads.

Strength T_u of such a tension member is equal to the area of net section, A_n , times the tensile strength of steel, F_u . The load combination involves axial tensile loads P_D caused by dead load and P_L caused by live load.

Using load factors proposed by Cornell, Ellingwood, Galambos and MacGregor and the resistance factor from Reference (3), the definition of LRFD may be written as

$$0.74 T_u \geq 1.2 P_D + 1.7 P_L$$

where .74 is the resistance factor, 1.2 is the dead load factor and 1.7 is the live load factor.

The expression can be made more general by using symbols for the numerical values of load and resistance factors:

$$\phi T_u \geq \gamma_D P_D + \gamma_L P_L$$

ϕ stands for resistance factor, γ_D and γ_L for dead and live load factors corresponding to the load effects P_D and P_L .

These factors account for various uncertainties in design. The resistance factor accounts for uncertainties in the resistance for a particular limit state. Its magnitude depends on variability associated with that particular limit state.

For example ϕ usually accounts for (1) variability in mate-

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rial properties such as tensile strength of steel; (2) variability in member geometry such as cross-sectional area, seldom exactly as given in a handbook because of practical tolerances; and (3) variability in the ratio of the strength used in calculations to the actual member strength represented by available test data.

Load factors account for dead and live load uncertainties. Examples are (1) variations of live load with time; (2) variations caused by idealization of loads (in building design, we usually assume uniform load) and (3) variations in the location of the loads on the structure.

Additionally, load factors account for uncertainties in structural analysis. Generally, we analyze structures as if they were two dimensional rather than three dimensional. We assume connections are either infinitely rigid or with no rigidity at all, whereas every connection has some rigidity, but never infinitely rigid. In analysis we also assume that a member has no depth, while in a structure all members must have some depth.

To make the LRFD equation completely general, general symbols can now be substituted for the limit state and force effects. R_n is used for nominal resistance, that is, resistance computed from the appropriate design formula. Q_i is used for load effects, where i represents various types of loads like dead load, live load, wind, snow, etc. This general format is

$$\phi R_n \geq \sum \gamma_i Q_i$$

Load effects Q_i may be axial forces as in the example; or moments, shears or torsions. They are computed from a structural analysis using code specified loads just as today.

Limit States

As to the resistance R_n , it is computed from formulas given in the specifications from various limit states. Limit states can be classified into two groups: strength limit states representing the capacity of a member or connection to survive extreme loads; and serviceability limit states that must be checked to avoid malfunctioning of the structure during routine use.

Some examples of strength limit states are fracture on the net section of a tension member, local buckling of a column with axial load, lateral-torsional buckling of a beam column and maximum plastic strength of shear connectors in a composite beam.

Examples of serviceability limit states are (1) permanent sag or drift due to yielding, (2) excessive elastic sag or drift, (3) major slip in high strength bolted joints and (4) unacceptable vibrations. Vibrating floors can interfere with personnel using a building or with the functioning of machinery. Excessive elastic drift during windstorms or earthquakes may result in cracking of partitions.



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Looking back at the general LRFD format, it is apparent that items for design consideration are not much different from those in strength design in concrete today. The difference lies in the manner of deriving specific numerical values of load and resistance factors. This step is, of course, accomplished by specification writers rather than by designers.

Today, structures are designed for a particular load by using particular single-valued expressions for strength. In reality, however, neither item can be determined with such singular certainty. Test results of 185 beams failing in elastic lateral-torsional buckling, shown in Figure 1, illustrate this point. Figure 1 is a histogram, a plot of the ratio of test to predicted values of strength against the number of tests or frequency of any of these ratios. For example, there were 14 tests in which the ratio of test to predicted value was 1.0. For most of the 185 tests the ratio was close to 1.0, while extreme values of 0.72 and 1.34 were obtained only once each.

Structural Reliability

For the purposes of analysis, the histogram of Figure 1 may be replaced by the smooth curve shown in the top half of Figure 2.

However, variation occurs not only in resistance but also in load effects, as illustrated in the lower portion of Figure 2.

Each of these distributions can be characterized by its mean value; R_m for mean resistance and Q_m for mean load effect, and with standard deviation σ_R or σ_Q to measure horizontal spread. Approximately 95% of all events fall between the vertical lines bounding the cross-hatched areas.

In Figure 3, resistance and load effect are plotted on the same graph. As long as load effect Q is represented by a point on the Q -curve to the left of point A, and resistance is represented by a point on the R -curve to the right of point A, the requirements of LRFD are satisfied and the structure is safe. On the other hand, when the load effect falls beyond point A and the resistance falls short of point A, failure occurs. The cross-hatched area under point A is related to the probability of failure.

It can further be observed in Figure 3 that probability of failure is inversely proportional and, therefore, safety of a structure is proportional to the difference between R_m and Q_m . In other words, the area under A is decreased and the safety of a structure is increased by spreading the Q and R curves further apart. One can also decrease the area under A to increase safety by decreasing variability, that is, by making one or both curves steeper. In other words, the less variability in Q and R , the smaller the area under point A and, therefore, the higher the safety.

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These two observations can be written in the form:

$$\beta = \frac{R_m - Q_m}{\sqrt{\sigma_R^2 + \sigma_Q^2}}$$

where σ_R represents the standard deviation of R and σ_Q represents the standard deviation of Q . This equation was derived from the theory of probability; it is known as the safety index.

Safety index β is related to the probability of failure, the cross-hatched area under point A. As an example, for normal distributions, $\beta = 5$ represents a failure probability of about 3 times 10^{-6} , or that failure would be expected to occur in three out of one million cases.

However, two rather important points must be emphasized. First, probability of failure depends very much on distribution curve shape. Sufficient data is not available to determine these shapes accurately; accordingly, the probability just cited may be substantially in error. Second, while our knowledge of the behavior of individual members and connections alone is quite accurate, our knowledge of the behavior of the same members and connections in a finished structure needs considerable additional studies. Generally speaking, the strength of individual members and connections furnishes minimum values such that current knowledge combined with LRFD concepts results in safe design. Considerable further research will be needed before one can design structures that are utilized to their capacity limits. Probabilities of failure, associated with various β values, must be judged in this context.

Load and resistance factors are not only functions of β , but of coefficients of variation in resistance V_R , engineering analysis V_E , and the load effects V_L . Additionally, they are functions of the ratio of mean to nominal load or resistance. Galambos expressed these functions in closed forms; for example, his equation for resistance factor is:

$$\phi = \frac{R_m}{R_n} e^{-0.558V_R}$$

On the other hand, the method proposed to ANSI A58 involves an optimization procedure that leads to invariant values of load and resistance factors.

One can now choose a β ; then compute the numerical values of factors ϕ and γ .

This has been done by Galambos, who evaluated various resistances as well as various coefficients of variation. He then set out to select a particular value of β for his design criteria.

One could select β on the basis of a chosen probability of

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failure. But in view of current uncertainties concerning relationships between β values and actual probability of failure, it seems preferable to follow the path elected by Galambos. He calibrated the new design against current designs. Or, expressed in another way, he selected a representative safety index value that characterizes the level of safety inherent in current design practice. On this basis, he decided to use $\beta = 3$ for members and $\beta = 4.5$ for connections.

The resulting resistance factors ϕ are shown in Table 1. They vary from .65 for buckling of long columns to .88 for yielding on the gross section of a tension member.

Galambos' resistance factors were rounded off to two decimal places in order to preserve apparent differences for specification writers. But for practical design, ϕ values are likely to be rounded off to one decimal or the nearest .05. It can be readily seen that such rounding will result in very few different resistance factors.

The load factors γ proposed to ANSI A58 are shown in Table 2 for seven combinations of dead, live, wind, snow and earthquake loads. It should be noted that the resistance factors in Table 1 are not strictly compatible with the load factors shown in Table 2, since the two sets of numbers were computed from different theories. However, preliminary studies have shown that resistance factors compatible to the load factors in Table 2 are essentially the same as those shown in Table 1.

Comparative Designs

Galambos selected β values by calibration of the new design against current designs. While this was done first on a theoretical basis, a practical check followed. This was performed by two well known consulting firms; Sverdrup & Parcel of St. Louis and LeMessurier Associates of Boston. They selected four buildings for redesign of certain members by LRFD.

One building was a large, basically one-story aircraft repair facility in Texas. The redesign included a typical roof purlin, two trusses on a 50-ft span, and one interior and one exterior column.

All steel was ASTM A36. Live loads were 20 psf for the roof, 50 psf for the mezzanine floor, and 70 mph wind. Crane and monorail loads were included. Both columns resisted some wind forces by strong axis bending.

Another building was a brewery with a heavily loaded brew-house floor framed with steel beams and plate girders. Floor dead load was 135 psf with a live load of 300 psf. Comparative designs were made for a typical simply supported filler beam and a plate girder.

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Still another building was a six-story passenger car parking garage. Floor live load was 50 psf. Beam, girder and column elements were redesigned.

The last structure was a 16-story office building. Floor framing in this study supported a 3-1/4 in. lightweight concrete slab over a 1-1/2 in. steel deck. Comparative designs were made for typical 32-ft filler beams and 40-ft girders for three different live loads: 50 psf LL with 20 psf partitions, 100 psf LL and 100 psf reduced LL.

Wind forces were resisted in the longitudinal direction by moment resisting framing while, in the short direction, simple connections were used and wind was resisted by bracing in the elevator walls shaft. Design live load was 50 psf, reduced, plus 20 psf partition allowance. Wind loading was 90 mph. ASTM A572 Grade 50 steel was chosen whenever practicable. Columns and girders of an interior and an exterior moment-framed wind bent, with wind bracing, were redesigned.

Results of these studies, based on load factors proposed originally by T. V. Galambos rather than on the load factors proposed to ANSI, are summarized in Figure 4. For each redesigned element, section weight obtained by AISC working stress design (1978 Specification) (16) are plotted against weights according to LRFD. (3) All were adjusted for design efficiency, i.e., the weights were multiplied by the ratio of applied force effect to the resistance of the cross-section. The 45° line represents cases where LRFD and AISC designs give the same steel section. For points falling below the line, LRFD is more economical. For points above the line, LRFD requires larger sections.

On the basis of Figure 4, it is reasonable to conclude that Galambos succeeded in making load and resistance factor design essentially equivalent to AISC design.

Concluding Remarks

The project at Washington University was completed in 1976. The proposed method was transmitted in 1977 to the American Institute of Steel Construction for consideration in future revisions of its Specification. AISC's Specification Committee divided the document among a number of task forces who are now reviewing it. The questions of accuracy and simplicity of use, as well as the twin questions of safety and economy, are important in this review. Assuming that the task forces will come up with favorable recommendations, it will then be up to the whole Committee to decide whether to include LRFD in future editions of the Specification.

It has been often said that one truly understands a technical relationship only when it can be reduced to a mathematical expression. This condition has existed in structural mechanics for over a century. But a rational approach to safety was not available;

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safety considerations have remained strictly empirical.

A rational consideration of safety is now a distinct possibility. Its adoption, in combination with design based on limit states, will result in two accomplishments: first, it will open the way to significant improvements of both safety and economy; and second, it will give the designer a far better understanding of the meaning of the design process.

And finally, with LRFD, one can see for the first time a clear path toward fully rational structural design.

Acknowledgments

This presentation is based on the references listed below and draws heavily on material in two papers presented at the 1977 AISC National Engineering Conference in Washington, D.C. (References 4 and 6). Figure 1 was taken from Reference 8 and Table 1 from Reference 5.

The phenomenological explanation of the meaning of the safety index β is an outgrowth of a discussion with Professor S. J. Fennes of Carnegie-Mellon University.

Finally, the author wishes to acknowledge the critical review of the manuscript by Messrs. D. H. Hall and R. H. Wildt.

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Member	Limit State	ϕ
Tension Member	Yield in Net Section	0.88
Tension Member	Fracture in Net Section	0.74
Column	Buckling	0.86-0.65
Beam, Plate Girder	Maximum Moment Capacity	0.86
Plate Girder	Maximum Shear Capacity	0.86
Composite Beam	Maximum Moment Capacity	0.84
Beam-Column	Instability	0.86
Fillet Welds	Shear	0.80
A325 H.S. Bolts	Shear	0.86
A490 H.S. Bolts	Shear	0.82

Table 2**Load Factors**

Load Combination	Y				
	D	L	W	S	E
D	1.4				
D + L	1.2	1.7			
D + S	1.2			1.7	
D + L + W	1.2	0.4	1.3		
D + L + E	1.2	0.4			1.5
W - D	0.9		1.3		
E - D	0.9				1.5

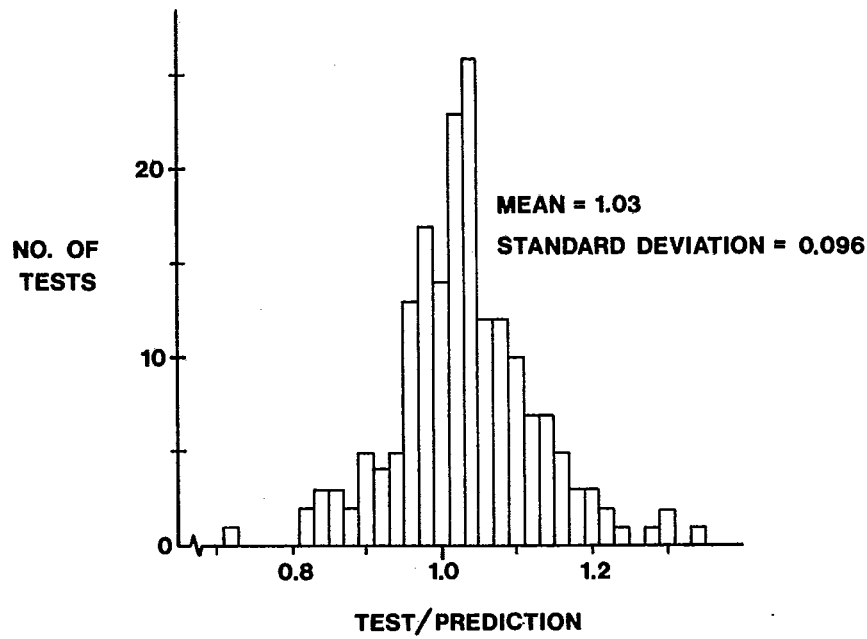
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Fig. 1--Histogram of beam tests

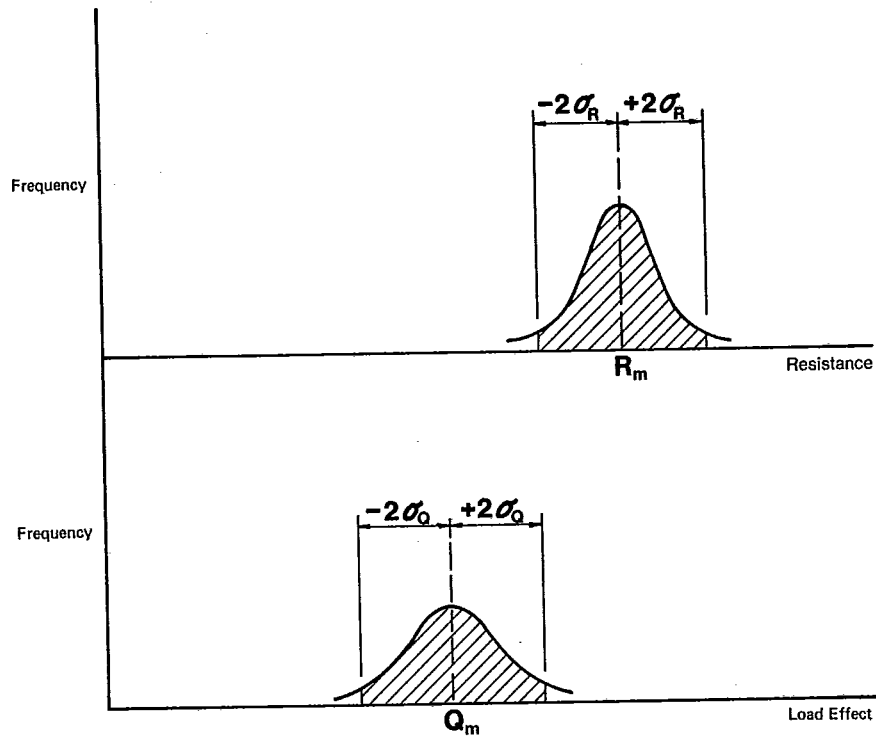
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Fig. 2--Probability distributions for R and Q

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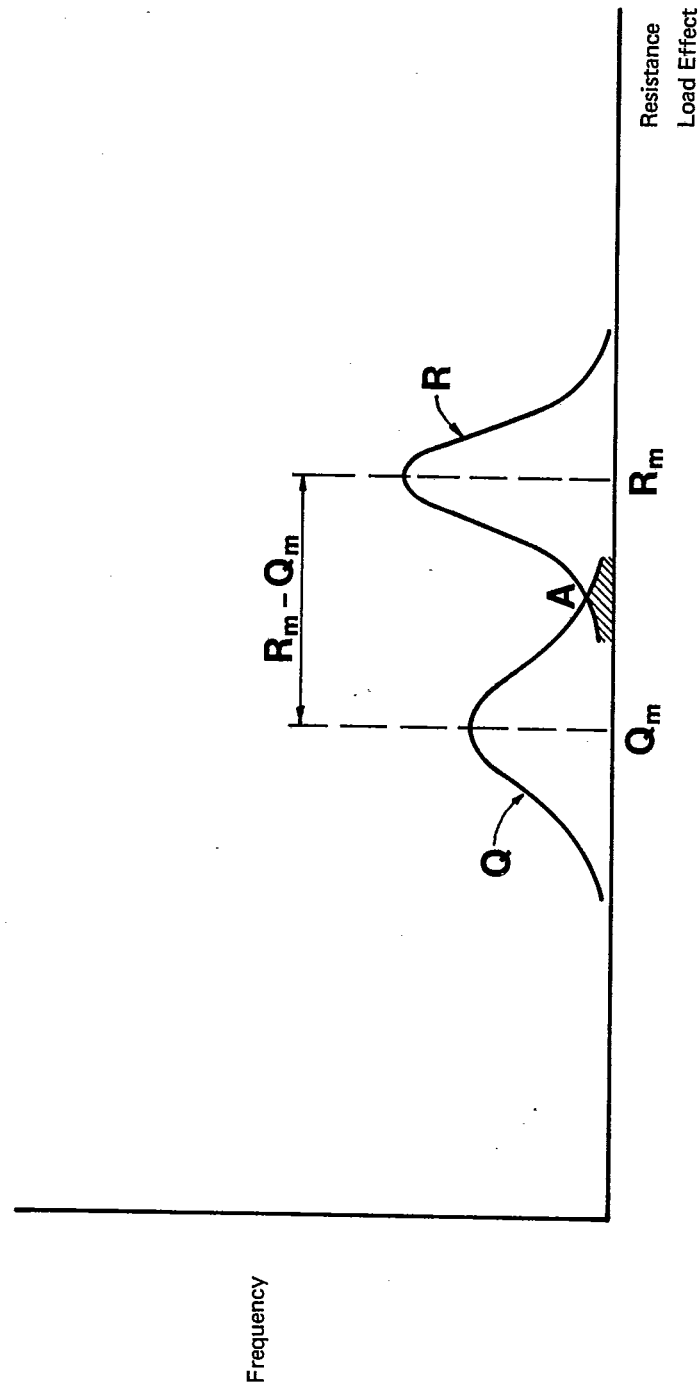


Fig. 3--Load effect versus resistance

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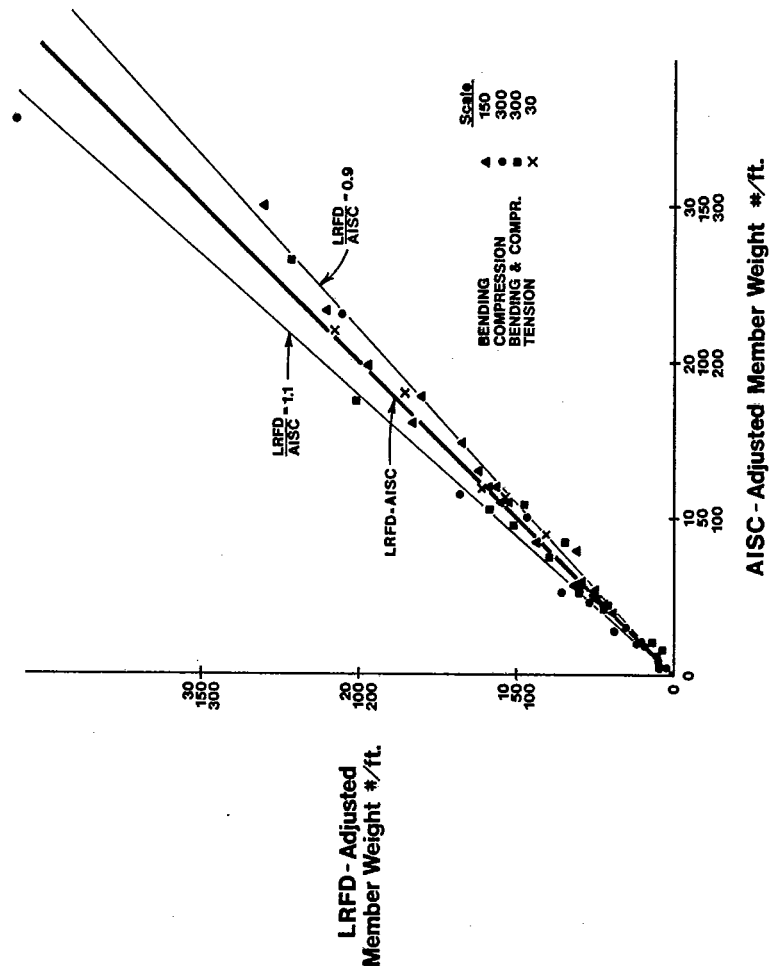


Fig. 4--LRFD versus AISC design

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Applications of Theory to Practice in Scandinavia

By R. Lenschow, S. O. Olesen,
and S. Sahlin

Synopsis: The use of concrete structures in off shore installations offers a challenge to structural engineers. Due to lack of direct experience from comparable, earlier concrete structures, it has often been necessary to rely more heavily on knowledge of the basic properties of concrete under extreme conditions. The paper describes a few examples.

Inclined slipforming of large concrete tower structures requires creep and shrinkage deformations within the first few days to be accurately estimated taking into account the effect of heat generation and age on the strength and stiffness properties of the concrete.

Off shore structures are generally exposed to fatigue loading from waves. Wave load histograms from the North Sea are presented and the problems in estimating the fatigue damage to the concrete is described.

The effect of concentrated impact loads on slabs and cylindrical shells is treated. FEM analysis as well as laboratory tests have been performed.

Finally, the strength of concrete under high water pressure is discussed.

Keywords: concrete construction; concrete slabs; cylindrical shells; fatigue (materials); impact strength; loads (forces); marine atmospheres; offshore structures; shells (structural forms); slipform construction; strength; structural design; structural engineering.

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INTRODUCTION

In the early 1970's due to the intensive search for oil and gas in the North Sea, the development of marine technology accelerated, the results being, from a structural point of view, huge concrete offshore structures. The procedure by which they appeared to be emerging out of the water was fascinating and amazing to professionals and laymen alike.

The concrete structures suddenly entered a field which had been dominated by steel structures and a few years later, several concrete platforms were under construction simultaneously in three or four countries. The height of such a structure may - complete with all installation - come close to 200 m and cost well over 2 billion US \$. Examples are shown in figure 1 and 2.

Since prospects are, however, that in future, we shall have to operate in greater water depths than today - which are maximum 160 m - and since the present concrete structures are not suitable for very much greater depths, considerable research and development work is being carried on in different parts of the world. Part of it is basic research, part of it concentrates on new concepts in design.

The present paper does not intend to give an overall picture of these activities but will deal with a few subjects of specific interest to those engineers with a general interest in concrete

Practice in Scandinavia 19**INCLINED SLIPFORMING**

For oil production at water depths of about 300 m different concepts are conceived. One is a floating anchored structure and another one is a three-leg structure as shown in figure 3. The latter will be discussed here. The legs are about 150 m long, and above the legs the design concept is similar to what is being used today. The legs are hollow reinforced concrete cylinders and they are inclined to give the structure sufficient stability under wave and wind actions.

Slipforming in a non-vertical position has not previously been used but since this procedure is necessary for the economical construction of this type of structure equipment for the slipforming in an inclined position has had to be developed. Figure 4 shows such equipment allowing continuous adjustment of slipforming direction.

In order to obtain a straight leg with a given inclination, the slipform should be constructed in such a manner that adjustments can be made at any time. The reference is a laserbeam fixed in the middle of the base of the leg. As shown in figure 5 an inclined tower or leg was made on land as proof that the new equipment functioned.

Another problem is to decide, how the center line of the slipform should be adjusted as it moves from the bottom to the top. Because of the inclination, the concrete section will be exposed to moment and shear force at a time when the material is very young and has low strength, low modulus of elasticity, and high creep. The creep is high also because the temperature increase under hydration is considerable in this thick walled structure (about 1 meter).

The properties of the slipformed concrete change continuously during the construction period as well as afterwards. Thus, the stiffness of a certain section increases with time. On the other hand, as the slipform moves upward, the eccentric loading on the section becomes greater so that cracking in the section may cause a significant loss of stiffness. The changing stiffness of all sections below the slipform determines the curvature and the deformation of the structure. In order to form a straight leg with acceptable accuracy, adjustments must be made which continuously sum up or integrate the situation in every element allowing for the changes in E-modulus, creep, shrinkage, cracking, and other enelastic behaviour of

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the reinforced concrete. Such an analytical model has been established, (1), and results appear in figure 7-9.

In this hypothetical case, the structure is 70 m tall ($\frac{1}{2}$ scale) and has been slipformed over 28 days. The heavy line indicated the form of the structure on the 28th day provided no adjustment has been made. The lower part of the tower has the greatest curvature, because of higher moment and because of cracked concrete. The cracked portion of the cross section decreases upward, and the upper 60% of the total height is uncracked.

It may be noted that adjustments securing that the slipform is at all times in the same position relative to the laserbeam are insufficient. Although the top of the leg would be in the correct position at the moment when the slipforming has been finished, the axis of the leg would be curved, and later on creep would furthermore bring also the top of the leg out of position. Adjustments must therefore allow for the deformations which will occur later on in the part of the leg already erected.

As mentioned above, hydration in the thick walled structure will cause a considerable temperature rise in the middle of the wall, which will have an effect on the rate at which strength and stiffness are developed. The temperature rise, however, is likely to be non-uniform over the thickness and therefore stresses will be introduced and cracking may result. It has been demonstrated, (2) that it is possible in a rather simple and rational manner to predict the time variation of the temperature profile under given circumstances. Thus it is also possible to choose rationally a production process which will keep temperature differences within acceptable limits and ensure that the form is not removed until the required maturity has been obtained.

IMPACT RESISTANCE OF CYLINDRICAL SHELLS

The design of off-shore structures must take into account that ship traffic represents a risk of collisions between the structure and a ship. Platforms are serviced to a large extent by smaller vessels, and large tankers operate within short distances. It is evident that any platform - steel as well as concrete - will suffer a complete collapse if a supertanker makes a full speed head on collision. The risk that such an event could happen is extremely small and it has so far been accepted. However, the less serious but much more likely situation of a supply vessel hitting the



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structure with low speed should not cause major damage to the structure.

A prediction of the effects of a collision between a ship and an offshore structure requires an analysis of the transfer of energy from the ship to the structure and of the dissipation of energy in the structure. During the impact, part of the kinetic energy is dissipated in plastic deformations of the ship, part results in local damage of the structure, part is transformed into potential energy in the structure and its foundation and finally some energy is dissipated in the water surrounding the structure. Among the many questions involved, special interest has been attached to the question whether a local failure in the legs of a platform would be a likely result of an impact situation. This problem has been studied in different manners.

In reference (3), an analytical approach is used to determine the stress, strain, and crack propagation in a flat reinforced concrete plate struck by a solid object, see figure 10. At the instant of impact, shock waves will start travelling in both bodies with the speed of sound referring to the respective materials. The wave front will be reflected at the opposite surface of the slab, see figure 11 and start travelling as a tension wave inwards. As time lapses, numerous reflections occur and after a while the track keeping of wavefronts becomes impracticable. The only possible means for a reasonably detailed analysis is a numerical process on a computer. Figure 12 shows an example of such results simulating tests shown in figure 10 and based on a time integration of the dynamic equations for the FEM-discretised bodies. The results showed sensitivity to changes in the assumed yield and failure criteria and in the yield functions. The lack of experimentally verified models for concrete response to multiaxial loadings at high strain rates thus becomes a limiting factor for the use of analytical tools.

The calculations cited did not include large deformations and large strains but the calculations show clearly the progression of compression waves resulting in some crushing, shear cracks, bending cracks, yielding of reinforcement, and different possible failure mechanisms. The time history was followed into the nonlinear regions of the materials to the point where apparently most of the resistance of the plate was supplied by the reinforcement on the impacted side and by a cone of compressed concrete outside the shear cracks. Furthermore radial and circular bending cracks was clearly developed as can be seen from the figure.

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Yielding of the reinforcement was also indicated before the impactor bounced away and the post impact vibrations went on and eventually decayed. The plate was only "nearly failing" since the pulse was too short to produce a "quasi static" loading with prolonged yielding as would be expected from a long not solid impactor. It is conceived that energy initially is lost in the formation of shear and bending cracks but that eventually the major part of the impact energy is dissipated in yield lines in the slab. Therefore a design method based on the yield line theory could in this case be expected to give reasonable results.

As the impact velocity is increased the vibration of the slab will be more and more dominated by the higher modes, which again means that a punching type failure may occur which invalidates a yield line analysis. Also an increased bending strength would produce a shear failure.

The actual impact velocities involved at sea are of the same order of magnitude as in the example and this similarity also applies to the vibrational frequencies of the slab and that part of an off shore structure which is close to the impact point.

Information of this type together with available test results, (4), with proper account for material nonlinearity and strain rate sensitivity of material, supported an early hypothesis that the collision problem for the actual offshore structures could be treated more or less as a static problem.

The analysis of cylindrical concrete structures subjected to static concentrated loads normal to the axis, however, also becomes difficult when the inelastic range is reached. This problem has been studied experimentally on cylindrical test specimens as shown in figure 13, (5). The effect on failure load and failure mode due to variations in amount of reinforcement, in longitudinal prestress, in loading area, and in the boundary conditions were studied.

At low levels of load, the behaviour of the specimen was well predicted by an elastic analysis. Extensive cracking gradually developed along generatrices on the inside of the cylinder under the loading point ("12 o'clock") and on the outside of the cylinder at "3 and 9 o'clock". Yielding in the steel was measured at several points and the load-deformation curve showed in figure 14 also suggests that a considerable amount of inelastic deformations takes place before failure eventually occurs as a punching shear failure.

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The failure load was found to depend on the boundary conditions. In some of the tests the cylinder as a whole was acting as a beam supported on two "gable walls", see figure 13. In other tests the cylinder was supported directly opposite the loading point resulting in a 30-40% decrease in load capacity. A variation in the longitudinal prestress in the cylinder wall however, did not seem to affect the failure load. These observations may indicate that the punching shear failure was triggered by bending moments in the ring direction exceeding the yield moment.

Although the punching shear failure classification may be incorrect, the failure load was predicted fairly well (within the range of parameters studied) by the same formula as is normally used to predict the punching failure load in reinforced concrete slabs.

The problem is certainly not fully examined through this investigation. For the practical purposes involved, however, it was important to know that a load capacity of this magnitude could be expected. The load capacity is higher than the load required to plastify the ship hulls assumed to be the impacting body, (6). It means again that the major part of the impact energy will be dissipated in the ship itself and thus the ability of the concrete structure to absorb energy becomes less important.

WAVE LOADS AND FATIGUE

It can be said with some certainty that wave loads will not cause fatigue failure of the present offshore structures within their lifetimes. However, along with an increased concrete quality and more sophisticated calculations, resulting in highly stressed structures, the fatigue life can be expected to become one of the critical factors when future offshore structures are designed, (7).

Admittedly, we know too little about the fatigue of reinforced concrete structures to predict their fatigue life, even within reasonably wide limits. We have some knowledge, maybe enough, about the fatigue life of reinforcing steel. We have recently acquired more information about plain concrete, and, moreover, investigations on reinforced concrete specimens are in progress. But there is yet a long way to go. Generally, fatigue life is described by means of the Wöhlers diagram, which shows how many loadings a specimen will take at a specific stress level before failure. There is a difference of opinion as to the extent to which the number of repeated loads affect the reduction of strength of a cross section. Figure 15

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shows three examples of the Wöhlers diagram as interpreted by three different institutions, according to their individual experience.

The curves in figure 15 assume that all repeated loadings are equal which is certainly not true for wave loads. For variable loads, Miner introduced the hypothesis that each time a load of a certain magnitude is repeated, the same amount of damage to the section will occur. Applying Miners rule, which is used all over the world, damages from loads of different values can be added until the damage is 100%, which means failure.

Wave load data are often presented in a histogram. Such histograms relating to the North Sea are shown in figure 17. They show that within a 25-year period one 30 m high wave and for instance about one million 8-10 m waves can be expected.

For a specific cross section the wave load histogram can be transformed into a number of loadings at each stress level. The damage according to Miners rule and Wöhlers diagram is found, and it is thus possible to determine the fatigue life. However, results corresponding to each of the three Wöhler diagrams are completely different. The difference in the relative damages due to various wave heights are shown in figure 18. According to the Norwegian Petrol Department regulations practically all fatigue damage is due to waves larger than 15 m, while Det norske Veritas is of the opposite opinion, i.e., that practically all fatigue damage is due to waves of height less than 15 m. This clearly illustrates that we know too little about the fatigue of concrete structures.

The fatigue research going on in the USA, Germany, France, Holland, Scandinavia and other places will probably change the picture. Figure 19 shows some typical results. It appears that a load does not cause the same damage regardless of the preceding load history, which is contrary to Miners rule. The damage is much greater early and late in the load history. With different load levels, the sequence of loads is therefore not immaterial.

It is to be hoped that this interesting research will result in practical design rules before the fatigue becomes a critical factor in the future concrete offshore structures.



Practice in Scandinavia 25**STRENGTH OF CONCRETE UNDER HIGH WATER PRESSURE**

Design of concrete offshore structures is based on the ordinary strength of concrete. However, test results published in November 1976, (8) indicated a reduction in strength of up to 28% when concrete was subjected to high seawater pressure.

The effects of high pore pressure in the concrete have many interesting aspects and therefore a limited series of tests was carried out, (9). Hollow concrete cylinders were placed in a pressure chamber as indicated in figure 20. External and internal waterpressures were varied representing waterdepths from zero to 800 m. The variation of the saltwater pressure on the cylinder walls is shown in figure 21.

The concrete material is thus exposed to triaxial stresses, the magnitude of which will depend on the water penetration and the pore pressure. The triaxial stress situation was therefore well defined in the tests with waterpressure equal to zero and in the tests where the specimens were sealed. In those cases where water penetrates the concrete, the pore pressure is not exactly known.

As shown in figure 22, it was found that the triaxial stresses - as usual - lead to an increase in the load capacity. The increase, however, was smaller for the unsealed specimens where the pore pressure tends to reduce the effective stress between the material particles.

The figure also includes three results from specimens of a significantly lower strength. These specimens were exposed to water pressure corresponding to 800 m water depth, but they were tested in atmospheric pressure directly afterwards.

A simple explanation of these results would be that the internal pore pressure creates tensile stresses in the concrete which reduces the compression strength. The U.S. tests which were mentioned previously and which started the discussion about the reduction of strength under high water pressure, were carried out in the same manner, that is, the specimens were taken out of the high pressure and then tested. A reduction in compression strength was to be expected.

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CONCLUDING REMARKS

Concrete offshore platforms made a spectacular entrance in the market some years ago. Today the situation is rather complex. On one hand, only one single concrete platform is at present under construction while steel platforms are again dominating. On the other hand, it seems likely that the concrete platforms already in operation will continue to have low maintenance costs which in turn will probably make concrete platforms more competitive. It should be realized, however, that new concepts in design of steel as well as concrete platforms are needed in order to facilitate operations at water depths as large as 300 meter. The design of these new platforms requires the solution of a number of problems through coming research activities.

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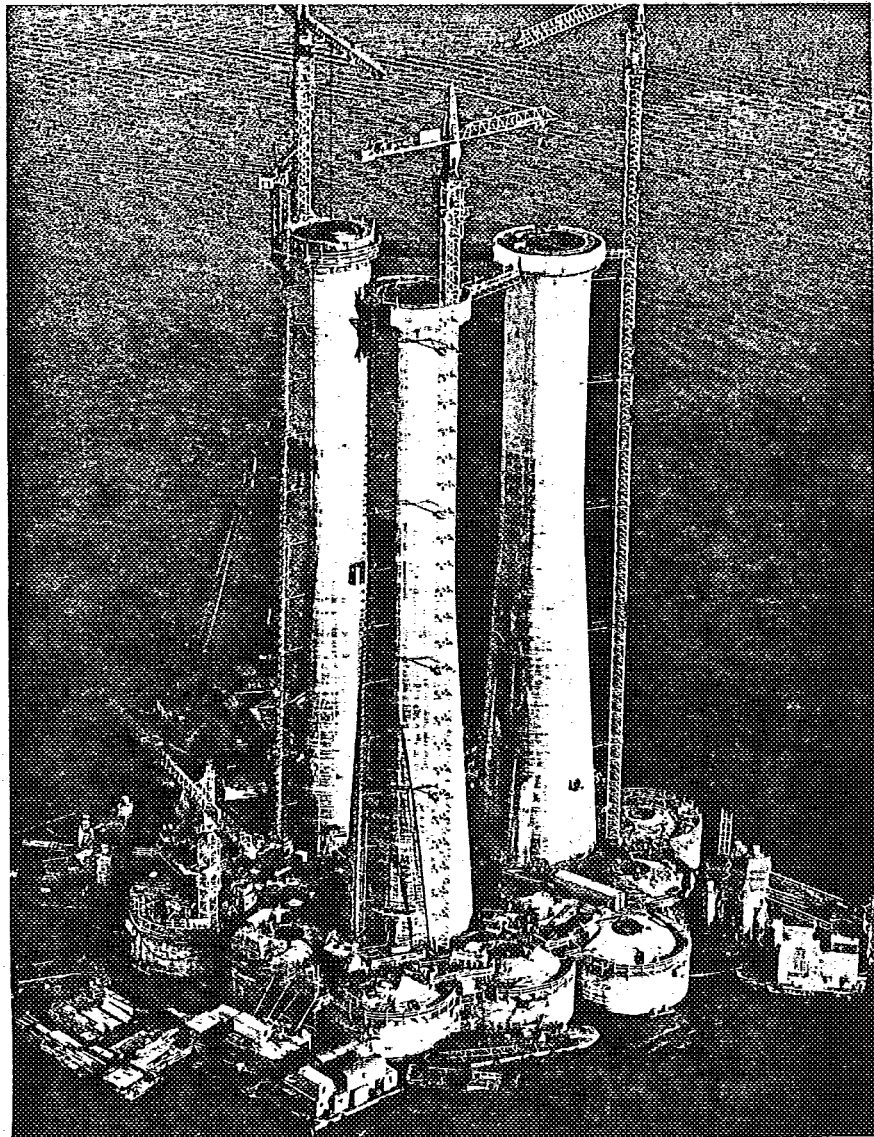


Fig. 1(a)--Concrete sea structure

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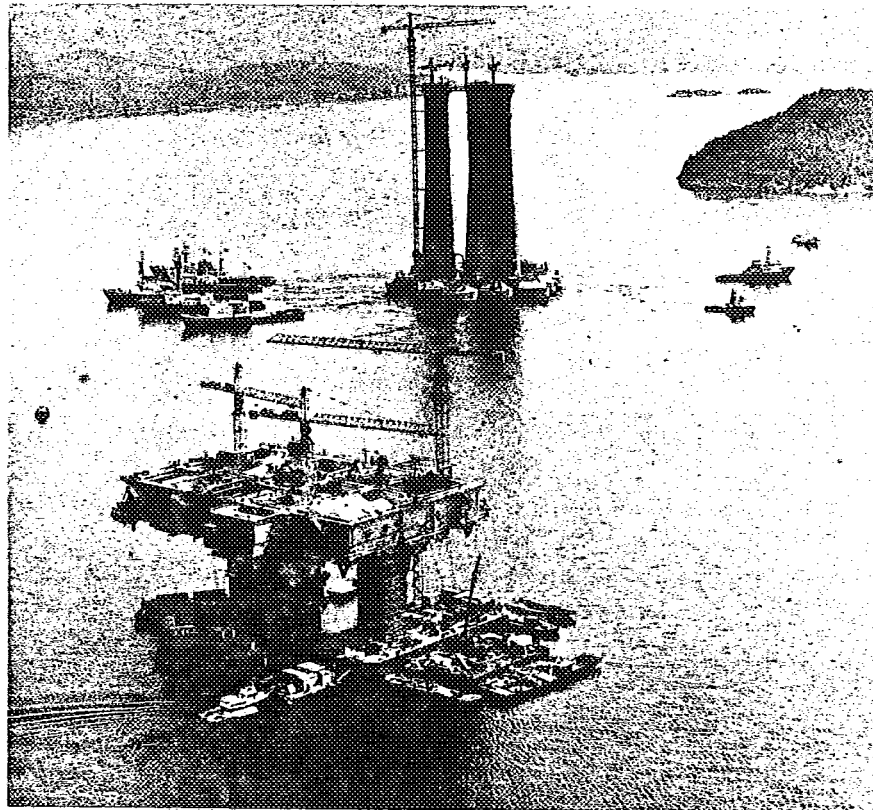


Fig. 1(b)--Concrete sea structure

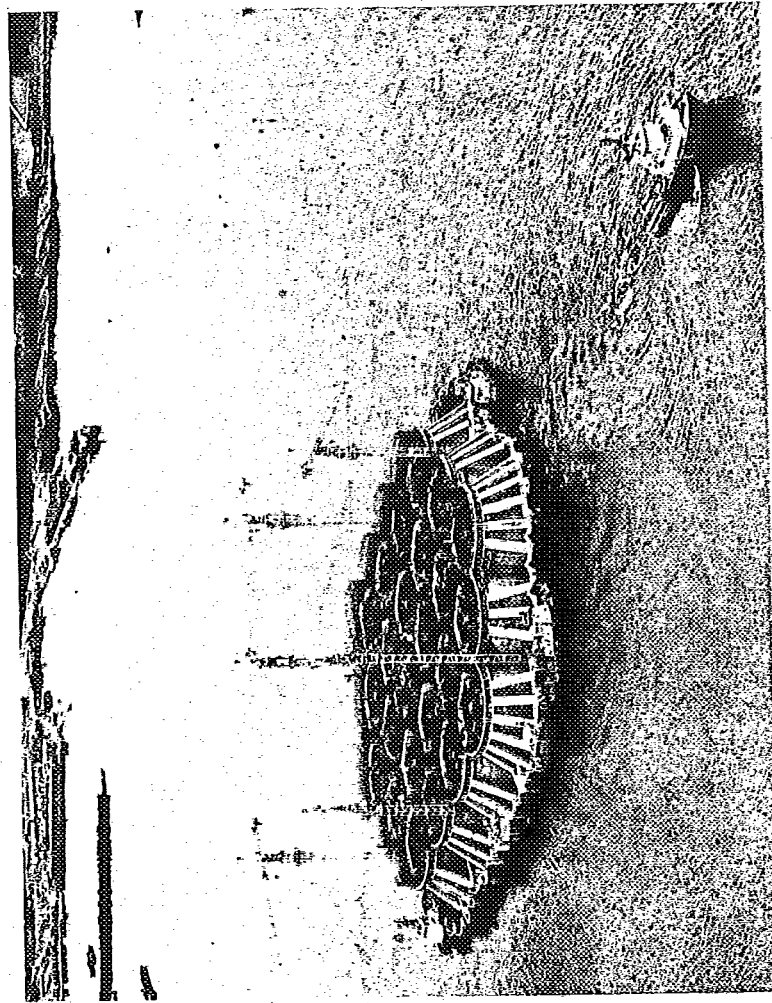


Fig. 2--Statfjord B platform under construction

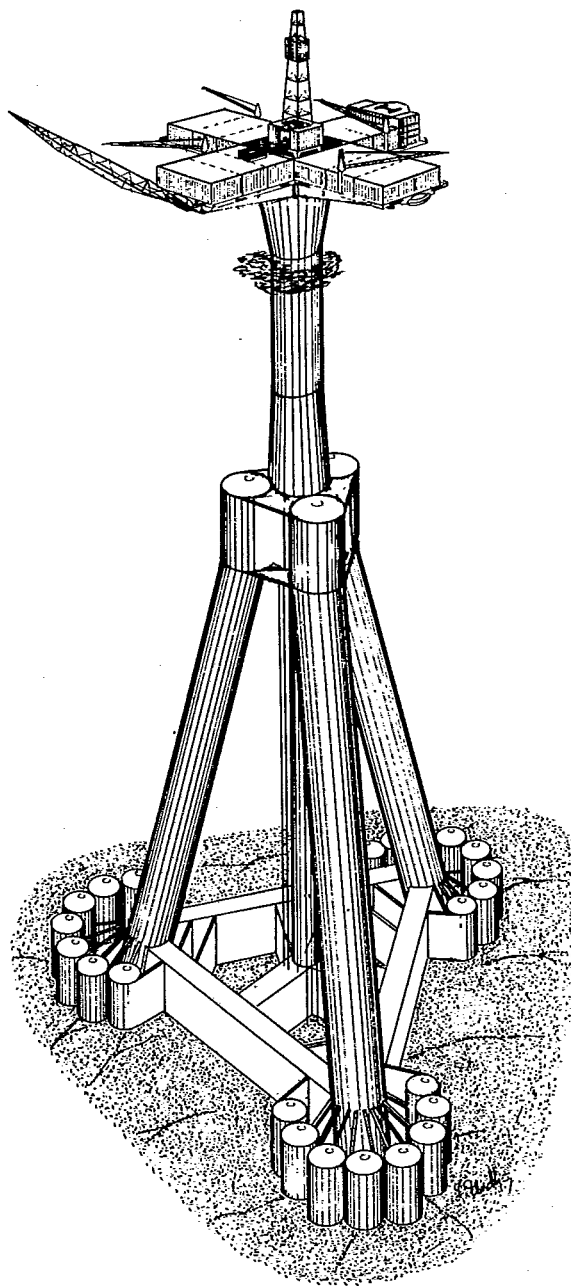
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Fig. 3--Tripod--A structure designed for oil production at 300 m depth

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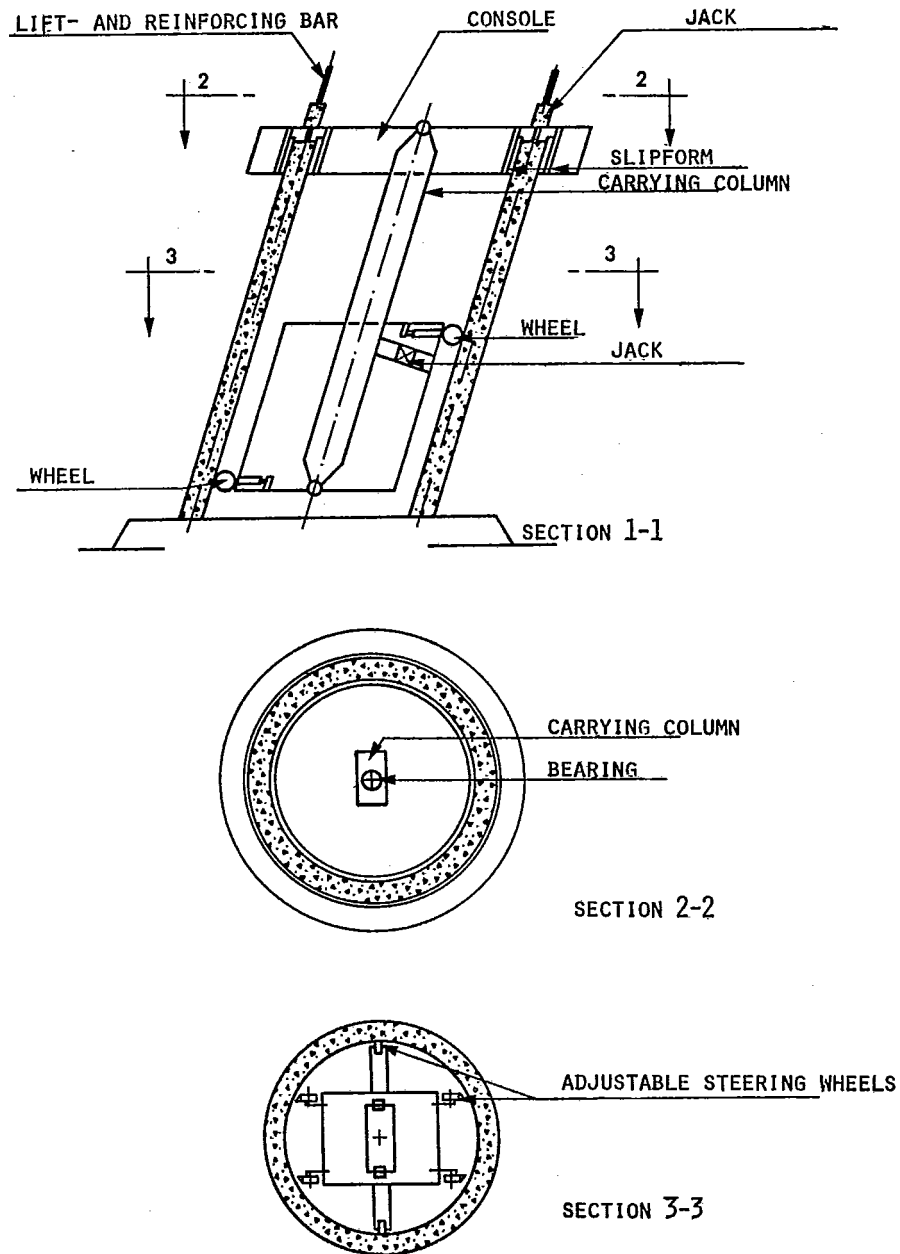


Fig. 4--Inclined slipforming device allowing continuous adjustment of slipforming direction