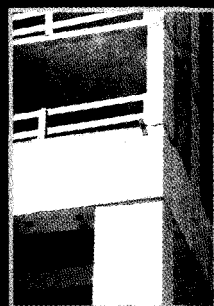


Structural Concrete:

(a) Very Good Control of Loads and Strength

Behavior to Implementation

(b) Poor Control of Loads and Strength



SP-198



Editors: Shafiqul Alam, Roger G. Green & S. Arif M. M. M. M.



A Symposium
Honoring
James G.
MacGregor



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Structural Concrete: Behavior to Implementation

A Symposium Honoring James G. MacGregor

Editors

Shuaib Ahmad

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PREFACE

Leadership, combined with innovation and integrity, are qualities inherent in James G. MacGregor's personal and professional activities. These attributes led a group known as the "Friends of Jim" to prepare and hold a national symposium of the American Concrete Institute in Seattle to honor Jim MacGregor.

The symposium offered papers on diagonal tension, reliability, columns, and engineering procedures and was well received. These papers have been published in the *ACI Structural Journal* or in this symposium volume. Papers offered for publication were reviewed according to ACI procedures.

The "Friends of Jim" began work on this volume and members of the ACI staff ably assisted them. Without this valuable assistance, considerable delay may have resulted.

Leadership is not a common quality in a modern society. Many aspire to leadership but few have the integrity, the compassion, the will to serve, or the people skills to be leaders. Such an individual is Jim MacGregor.

"The Friends of Jim"
2000

IN TRIBUTE TO JAMES GRIERSON MACGREGOR

A Leader in Structural Concrete Building Code Development and Education

On Wednesday, April 9 1997, colleagues, students, and friends of Dr. James G. MacGregor gathered for an all-day symposium at the American Concrete Institute Convention in Seattle, Washington. The James G. MacGregor Symposium was entitled Structural Concrete Behavior and Design.

The objective of the Symposium was to honor Jim MacGregor for his extensive contributions to research, to both professional and student education, to the prudent use of structural concrete, and to ACI. The Symposium also recognized through the various presentations, the calm and effective manner in which Jim MacGregor has made these contributions. The four technical mini-sessions were planned to relate to Dr. MacGregor's professional and research interests. These include Structural Reliability, Shear and Torsion in Structural Concrete, Stability of Columns and Frames, and Examples from Professional Practice. The papers presented in this Special Publication volume were an outgrowth from that Symposium.

While now in residence in British Columbia where he is very active in both ACI Building Code activities and in textbook writing, Jim MacGregor is more familiarly recognized as an Albertan. Born in Vegreville in the Canadian Province of Alberta in February 1934, Jim received his primary, secondary, and undergraduate education in Edmonton. His father was a highly respected author there. Upon completion of his B.Sc. in Civil Engineering with Distinction degree at the University of Alberta in May 1956, Jim traveled to Champaign-Urbana where he earned M.S. (1958) and Ph.D. (1960) degrees under the tutelage of former ACI President Chet Siess and numerous other Illinois faculty who staffed the Camelot of reinforced and prestressed concrete in the USA. Jim had the opportunity to work closely with Chet Siess on ACI 318 Building Code Committee proposals and was clearly infected by a lifelong desire to be of service to the concrete design and construction communities. His personal research contributions at Illinois were notable, in particular laying the foundation for design for shear in prestressed concrete.

Upon his return to the University of Alberta as a faculty member in 1960, he immersed himself in a broad range of activities that were to lead to international recognition as an expert in structural concrete behavior and in quantifying structural safety. He advanced rapidly through the professional ranks, in 1985 was named a University Professor and from 1987 to 1990 served as Chair of the

Department of Civil Engineering. Throughout his career he also carried out numerous consulting engineering assignments. He became a Professor Emeritus in 1994.

From the time of his return to Alberta, he was quickly recognized for his ability to synthesize – to find the similarities between an important engineering problem and the material he found in theses and dissertations, reports, published papers, committee documents, and textbooks. A 1968 ASCE State-of-the-Art Award was quickly followed by another 1974 ASCE State-of-the-Art Award, again for a committee report on shear. Throughout his career, Jim excelled in turning research results into specific ACI and Canadian National Building Code proposals. The Code formulation proposal reflecting the synthesized shear studies earned Jim the 1976 ASCE R. C. Reese Research Prize, and a paper on design of deep beams earned the ACI R. C. Reese Research Medal in 1987.

Throughout his career, Jim has been a highly effective Technical and Building Code Committee leader. Particularly in the area of shear and torsion in reinforced and prestressed concrete, he led the adoption of design models for one- and two-way shear, integrated torsion with shear and flexure, and developed strut-and-tie models for D-Discontinuity regions. While these developments encompassed his own tests, the latter were a small fraction of the overall base for the recommendations. ACI awarded him the Delmar L. Bloem Award for Distinguished Service in 1974, recognizing his technical committee leadership.

In parallel development, Jim continued his interest in slender columns that began while working as a research associate at Illinois. Working with the Joint ACI-ASCE Committee on Reinforced Concrete Columns, he was a principal author of the “Moment Magnifier Method” for slender column design, which in 1971 replaced the behaviorally incorrect “Load Reduction Factor” method that had historically been the accepted procedure in the ACI Building Code. His contributions were recognized by simultaneous award of the ACI Reese and Wason Medals in 1972 for the same paper on slenderness effects. He continues to provide leadership in improving the approximate column slenderness design procedures and in refining methods of calculation for the more “exact” second order analysis procedures. Another paper on frame stability received the ASCE Reese Award in 1979.

One of Jim’s most lasting contributions has been in the overall synthesis of a modern, reliability-based contribution to the philosophy of structural safety underpinning the North American concrete codes. During 1975-77, Jim undertook a traveling lectureship for the Canadian Society for Civil Engineering on the subject of “Safety and Limit States Design for Reinforced Concrete.” He again synthesized a great deal of previous studies and research in these fields, giving an insightful, practical, and inspiring framework for a modern structural safety approach for structural concrete. MacGregor’s contributions were not only in gaining general acceptance on the philosophical level, but very importantly were on the quantification level, particularly of the variability in member resistance. These landmark studies provided the basis for the LRFD code adopted by AISC, the basic load factors adopted by the ASCE 7 load standards, and the alternate load factors and resistance factors used in the 1995 ACI Building Code for design of composite steel-concrete structures. Recently ACI 318 voted to use these recommendations as the primary load and resistance factors for all structures in the ACI 318 2002 Code. The overall limit states-structural safety philosophy has been implemented in the Canadian codes. MacGregor

co-workers at the National Bureau of Standards program received one of ASCE's highest honors, the 1983 Norman Medal for this pioneering effort.

Jim Macgregor's greatest contributions are the long list of highly informative and useful papers that have graced the pages of major North American and international journals. Their quality has been vividly attested to by the recognition of his peers on prize committees in numerous societies. Certainly, in terms of a legacy, one must mention the capstone that Jim provided in two editions of his Prentice Hall text, *Reinforced Concrete: Mechanics and Design*. Its comprehensiveness combined with its originality established it as a major text in the field and an important part of the legacy of a giant among the leaders of structural concrete design practices. Truly an educator par excellence, Jim received the ACI Kelly Award in 1986, "for research and teaching of student and practitioner."

In addition to his technical leadership, Jim has been a major figure in the administrative leadership of ACI. A past member of the Technical Activities Committee, Jim served as a member of the ACI Board of Direction (1972-75 and 1990-95), held the office of Vice President (1990-92), and was President of the Institute, serving with distinction from 1992-93.

His pre-eminent stature was recognized by an honorary Doctor of Engineering awarded by Lakehead University in 1998 and an honorary Doctor of Science awarded by his beloved University of Alberta in 1999. He was elected a Fellow of the Royal Society of Canada in 1992, an Honorary Member of ACI in 2000, and has received many, many other accolades.

Jim MacGregor has been the greatest living synthesist in the reinforced concrete design and building regulation field. He has drawn on research in North America, Europe, and Japan to provide a basis for modern building code regulations to protect life and investment. He has an uncanny knack of being able to see proper relationships in very disparate studies. He has summarized and illustrated these in highly imaginative and highly creative design proposals, which form the basis for current international standards in load factors, resistance factors, structural safety, reinforced concrete shear and torsion, and reinforced concrete slender columns and stability. Building regulations throughout Canada, the United States, Mexico, South America, Australia and New Zealand and much of Southeast Asia directly use code language and procedures developed and pioneered by Jim MacGregor.

Jim MacGregor complimented his structural engineering interests with an even stronger interest and dedication to his family. His wife Barbara has shared Jim with ACI and other organizations, while remaining a strong partner in every sense of the term. She travels with Jim worldwide and helps keep him on an even keel. Their children, Bob, Joan, and Dave have given strong support and love to their parents. Tragedy struck their family when Bob was killed in a fire, taken at the height of his creativeness as a partner in a structural concrete bridge design firm. The family has grown even closer in the aftermath of tragedy. Jim and Barb enjoy hosting visits from their eight grandchildren at their lovely retirement home on the British Columbia Sunshine Coast.

Jim remains an active contributor to ACI committees and is completing the 3rd edition of *Reinforced Concrete: Mechanics and Design*. A long and productive career still has more chapters to unfold.

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Strut and Tie Models

by D. M. Rogowsky

Synopsis The evolution of strut and tie models is reviewed and their characteristics are discussed. From their evolution it is seen that more design effort should be expended on developing a reasonable truss than on refined calculations of nodal stresses and permissible concrete stresses. Strut and tie models are design tools. They allow engineers to put an appropriate amount of material in an appropriate place, and expeditiously demonstrate that there is at least one way for the structure to safely carry the design load. They are not very useful for checking designs. When one is planning the structural analysis, one needs to know if the design will be done on the basis of strut and tie models. Different models may be required for different load cases. Rather than modeling the whole structure, models may be applied to specific regions of a structure. Designs based on strut and tie models may require slightly more reinforcement than designs based on other methods since, in general, the contribution of concrete tensile strength is ignored. Finely tuned empirical equations will use less reinforcement but are restricted by the limits of the test data. Strut and tie models are general rational design tools that do not have such limitations.

Keywords: plasticity; reinforced concrete; strut; tie

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D M Rogowsky has been involved in the design and construction of concrete structures since 1973. As a practicing structural engineer, he used strut and tie models in the design of numerous structures over a period of some 15 years. He is currently a Professor at the University of Alberta.

INTRODUCTION

Strut and tie models are a useful but sometime misunderstood tool for the design and detailing of reinforced concrete structures. While rudimentary strut and tie models, also known as truss models, or as the truss analogy, have been used by engineers for almost 100 years [1,2], it is only recently that these models have been explicitly included in design codes. Figures 1 and 2 illustrate historical and current strut and tie models. Canadian [3,4] and European [5] codes provide rules for the use of strut-and-tie models. The formulation of appropriate rules for incorporation into American codes is a topic of current interest.

PRACTICAL SIGNIFICANCE

This paper provides a critical review of strut and tie models. The primary purpose of this paper is to provide readers with sufficient knowledge and insight to encourage the appropriate use of strut and tie models, and avoid the misuse of strut and tie models.

EVOLUTION OF STRUT AND TIE MODELS

A study of the evolution of strut and tie models provides useful clues regarding the important aspects of the models. As in most fields of study that evolve, the most important items are the first to be recognized and incorporated into practice. The most recent items, while being topics of current discussion in the literature, are probably the least significant.

Since the initial inception of reinforced concrete design at the end of the last century, engineers have idealized reinforced concrete members as trusses. Figure 1 illustrates an early design with strut and tie models. These pin-connected trusses, consist of steel tension members and concrete compression members. The essential feature of this approach is that one finds a system of internal forces that is in equilibrium with the externally applied loads, and support conditions. A further essential feature is that the designer ensures that the steel and concrete areas provided are adequate to resist the forces obtained in the truss analysis.

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The first step in the design is the selection of a truss configuration that is reasonable for the loading and support conditions. In particular, the tension ties should be located and oriented in the same manner as the desired reinforcement layout. For example, in Fig. 1 the designer wanted to use trussed bars for the shear reinforcement. This led to a strut and tie model with diagonal tension members, and vertical compression members in the web. Statically determinate truss models were used to facilitate analysis. A graphical static analysis is shown in the upper portion of Fig. 1(a). Historically, the analysis was based on service loads, and the steel areas were determined on the basis of allowable working stresses. The allowable steel stresses were on the order of $0.4f_y$, and were sufficiently low that both safety and serviceability, in terms of crack control, were satisfied. On occasion, the designer might choose to let the concrete carry tension, such as in the diagonals in the middle half of the beam in Fig. 1. The designer of the beam in question judged the concrete tension stresses to be sufficiently low that it was reasonable to not provide diagonal tension reinforcement in the middle of the span. The concrete compression members were checked to ensure that they fit within the member without overstress. Allowable stresses were on the order of $0.3f'_c$. These simple conservative concrete stress limits ensured that the steel yielded before the concrete crushed.

Joints of the truss were detailed to ensure that bars were properly anchored without creating overly congested joints. Explicit checks on the concrete stresses in nodal zones were seldom if ever done.

In the early decades of reinforced concrete, many designs were validated by proof loading the structure after it was completed. The beam in Fig. 1 was reported to have a deflection of approximately 3 mm under full load. After five years of service, there were no cracks discernible to the unaided eye. This type of first hand experience gave the designers of the day confidence in their ability to use strut and tie models. Over the last century thousands, if not millions, of reinforced concrete structures have elements that have been designed using strut and tie models.

During the first half of this century, while there was little development of strut and tie models, there were substantive improvements in other methods a reinforced concrete design. Specifically, there was a move away from strut and tie models toward a sectional design approach. For example, for flexure and axial load, we use a strain compatibility analysis at a "section". While not totally abandoned by the profession, the development of strut and tie models was "stunted".

In the 1960's strut and tie models experienced a resurgence as a result of research on the theory of plasticity as applied to reinforced concrete. Independent

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developments in Denmark and Switzerland arrived at similar results. Strut and tie models were shown to be a special case of the lower bound plasticity theorem. Nelson [6] summarizes much of the work done in Denmark. In North America, practice generally tends to follow the Swiss and German work.

Marti [7,8] provided a rigorous theoretical basis for strut and tie models and several detailed design examples. These papers provide the basic tools that one needs for the use of strut and tie models. He showed that strut and tie models are "statically equivalent" to stress fields. While strut and tie models neglect the concrete outside of the struts, stress fields account for all of the concrete carrying some stress. Strut and tie models are thus merely a type of short hand notation for the more general stress fields. Stress fields are illustrated in Fig. 3(c). Region AQP is a fan stress field with diagonal compression stresses passing through A. This stress field is represented by the leftmost diagonal strut in Fig. 3(b). Region ABOP is a diagonal compression stress field with compression stresses parallel to AP. This stress field is represented by the second diagonal strut from the left in Fig. 3(b). Thus, the diagonal struts in Fig. 3(b) correspond in location and magnitude to the stress resultants in the stress fields in Fig. 3(c). Free body diagrams for these fields are given in Figs. 3(d) and 3(e). The vertical forces along AP represent the forces in the stirrups. The horizontal forces along the top and bottom edges represent the flexural tension and compression forces. One should not take the struts too literally. The permissible stress in the struts is therefore not explicitly significant. Marti suggests a permissible stress of $0.6f_c$, but indicates that other values may be used if one considers the strains associated with the assumed stress fields. Reference 7 gives the complete design of this member on the basis of stress fields.

Schlaich et. al. [9] illustrate a large number of applications of strut and tie models. The strength of this reference is in the methods used to develop reasonable strut and tie models for a wide variety of situations. Strut and tie models are developed from elastic stress trajectories, and the "load path" method. Models which tend to minimise the strain energy in the tension ties are preferred. It is recognized that struts are generally not of constant width and usually become narrower as they approach the nodes. These bottle shaped compression struts are discussed, along with the possibility of strut splitting. Concrete tension ties are used for special cases such as the bottle neck region of a bottle shaped strut.

The use of strut and tie models has been experimentally validated by a number of researchers. References 10 through 15 are a representative sampling of the research. Figure 4 illustrates the testing of a dapped ended beam. MacGregor [16] discusses several different strut and tie models for a dapped ended beam.

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Even though the model shown in Fig. 4 was deemed to be the least reasonable, its behavior was satisfactory. The tests illustrate that there is considerable latitude in the selection of a strut and tie model.

CURRENT FORMULATION

MacGregor [16] provides an up to date synthesis of strut and tie models in a form that is compatible with ACI 318 load and resistance factors. A member resistance factor $\phi = 0.85$ is suggested for solutions based on strut and tie models. Steel stresses are permitted to reach yield. The permissible concrete stress in struts and nodes under factored loads is established as a function of concrete strength, the orientation of cracks with respect to the compression struts, and severity of cracks (i.e. magnitude of transverse strains). Different permissible stresses are given for ten different design situations. For most situations, it is conservative to use $0.5f'_c$. Concrete tends to be more brittle as the compression strength increases hence, some reduction in the ratio of permissible stress to compressive strength is warranted for higher strength concrete. This is a relatively minor adjustment for concrete strengths less than 6000 psi. Cracks which are at an angle to the compression strut can cause a sliding failure in the strut along a crack surface hence reducing the strength of the strut. Large transverse strains, as evidenced by severe cracks, can reduce the compression strength of a compression strut even if the cracks are parallel to the strut [17]. Design examples are solved in a step by step fashion.

While it requires some effort to work through the examples in references 16, 7, 8, and 9, this is an excellent method of learning how to use strut and tie models.

STRENGTHS AND WEAKNESSES

Strut and tie models have a number of characteristics that designers need to be aware of. These characteristics can be an advantage or a disadvantage depending upon what the designer is attempting to do.

Multiple Actions and Load Cases

With strut and tie models, shear, torsion, axial load and moment are handled simultaneously. Thus one cannot go from design envelopes for shear, torsion, axial load, and moment to designs based on strut and tie models. In general, one should check the entire structure for the load cases that give maximum shear, maximum torsion, maximum axial force, and maximum moment. When there is a possibility for reversal of forces in the struts and ties, due to moving loads or wind

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acting in the reverse direction for example, several different strut and tie models may be required for each load case. *Developing strut and tie models for a large structure with complicated geometry and dozens if not hundreds of load cases can be a daunting task.* The author has found it to be expedient to develop two or three strut and tie models to deal with the cases that are dominated by vertical and lateral load respectively. To deal with a large number of load cases, it is convenient to do the truss analysis with an ordinary elastic frame analysis program. To deal with shear or torsion reversals which change the sign of the force in web members, all panels are provided with two crossed diagonal web members. If the analysis program does not facilitate "compression only" members, the analysis is run twice. The first analysis is used to identify "compression struts" that carry tension. The axial stiffness of these members is made negligible by reducing the cross-sectional area or the modulus of elasticity. In the second run, even though the "compression struts" are still present, they carry negligible tension. Occasionally, the second analysis indicates additional concrete struts with significant tension. If this occurs, the process of softening these members is repeated and a third analysis is performed.

While strut and tie models can be used for "whole member" design, they are better suited to the design of details such as joints between members, corbels and dapped ends. The development of strut and tie models for these details is straight forward. The overall structure is analyzed in the normal manner, using beam elements, to obtain the forces going into and out of the detail region. Usually a single strut and tie model for the detail region will suffice, and only the load cases that maximize each of the forces acting on the detail need to be considered explicitly.

Non-unique Solutions

There are theoretically an infinite number of strut and tie models that can provide acceptable results. This allows the designer some latitude in dictating how the structure should carry the load. For example, one may wish to have the shear reinforcement inclined as in Fig. 1, or vertical as it is usually done in current practice. The debate on which is the "best" model for a given situation can be endless. Since every situation is different, the selection of the model should be left to the designer. Standardization, or codification of specific models is not necessary. It is also undesirable in that, due to our litigious environment, designers will not have the freedom to select more appropriate models for their application if they are not "in the code".

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Checking of a design done on the basis of a strut and tie model can be problematic. If the checker uses a model that is significantly different than the designer's model, the checker may find that there is not enough reinforcement for his model to work. This however does not prove that the design is inadequate. It simply indicates that the checker has not found a safe load path. To check designs based on strut and tie models, the checker should review the design model to see if it is a reasonable model. The computation can then be checked. If the checker considers the model unreasonable, use of kinematic methods of analysis in accordance with the upper bound plasticity theorem can be used to prove that the design is inadequate [18].

V_c, the Concrete Contribution to Shear Capacity

Strut and tie models provide a rational method for dealing with the enhanced shear capacity near concentrated loads and supports due to diagonal compression in the concrete. This is particularly significant in deep beams and corbels. Because the tensile strength of the concrete is ignored, the diagonal tension capacity of the concrete is lost when one uses strut and tie models. While members such as one-way slabs may be adequate without shear reinforcement when designed by ordinary theory, these members will require shear reinforcement when designed by strut and tie models.

Steel Detailing

Strut and tie models excel at identifying where reinforcement is required, and where it should be anchored for the ultimate strength of the member. Serviceability, in terms of crack control can be readily checked on the basis of the steel stress under service load. Compatibility induced cracking, and cracking due to imposed deformations is more difficult to calculate with strut and tie models, but it generally need not be calculated at all. One practical requirement for the use of strut and tie models is the provision of well detailed minimum reinforcement throughout the member [7]. The amount of minimum reinforcement provided should be sufficient to take over the tensile force from the concrete, when it cracks, without yielding. In ordinary buildings, this corresponds to a steel ratio of at least 0.002, but may be as high as 0.006 when the concrete strengths are high and the member can be subjected to direct tension over the entire cross-section.

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Generality vs Specificity

Finely tuned empirical equations for specific problems will always give more precise answers than general procedures such as strut and tie models. For example Jirsa et. al. [13] found that while dapped end test specimens designed by strut and tie models behaved satisfactorily, specimens designed on the basis of empirical PCI methods had comparable, or better behaviour and used a little less reinforcement. Empirical equations are only useful for problems that fall within the data base of experimental results. Extrapolating beyond the limits of the test data is dangerous. Strut and tie models are general rational tools that do not have such limitations.

CONCLUSIONS

Strut and tie models are a useful tool, but not the only tool in a designer's tool kit. Strut and tie models have evolved to the point that rules for their use are being formulated in design codes. The author would like to make a plea for simple uncomplicated, even if somewhat conservative, rules. The rules are not a replacement for knowledge on the part of the engineer.

The priorities when using strut and tie models are

- a model that follows a simple natural load path;
- a model that is in equilibrium with the external loads and support reactions;
- use of minimum reinforcement throughout the member;
- detailing of the reinforcement (location and anchorage at nodes) that is consistent with the model;
- use of reasonable concrete stresses (0.5f_c for ordinary applications);

Secondary considerations for using strut and tie models include;

- when planning the structural analysis for a structure, one needs to know if the design will be performed on the basis of strut and tie models or sectional design procedures since this can influence the structural analysis model;
- several different strut and tie models may be required for a structure that has multiple load cases or reversal of stresses;
- while strut and tie models are good tools for design, they are not necessarily good tools for checking designs;
- strut and tie models may require more reinforcement than specialised empirical equations for special cases;
- sophisticated strut and tie models and refined procedures for determining permissible concrete stresses can be used;

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Pursuit of the "best strut and tie model" is like pursuit of the "best car" – an enjoyable but generally non-profitable exercise. Most models, like most cars are adequate for the intended purpose.

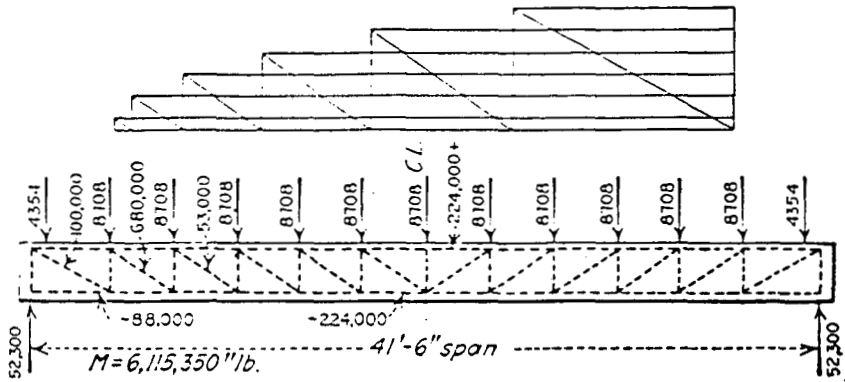
REFERENCES

1. Ritter, W., "Die Bauweise Hennebique" [The Hennebique System] Schweizerische Bauzeitung, Bd. XXXIII, No. 7, January 1899.
2. Morsch, E., "Der Eisenbetonbau, seine Theorie und Anwendung" (Reinforced Concrete, Theory and Application), Verlag Konrad Wittwer, Stuttgart, 1912.
3. Canadian Standards Association, "Design of Concrete Structures for Buildings (CAN3-A23.3-M84)", Rexdale, Ontario, 1984, 281 pp.
4. Canadian Standards Association, "Design of Concrete Structures (CSA Standard A23.3-94)", Rexdale, Ontario, 1994, 220 pp.
5. Comité Euro-International Du Béton, "CEB-FIP Model Code 1990", Lausanne, Switzerland, 1991, 437 pp.
6. Nielsen, M.P., "Limit Analysis and Concrete Plasticity", Prentice Hall, 1984, 420 pp.
7. Marti, P, "Basic Tools of Reinforced Concrete Beam Design", ACI Journal, Vol. 82, No. 1, Jan.-Feb. 1985, pp. 46-56. Also, Discussion, Vol. 82, No. 6, Nov.-Dec. 1985, pp. 933-935.
8. Marti, P, "Truss Models in Detailing", Concrete International, Vol. 7, No. 12, Nov.-Dec. 1985, pp. 66-73. Also, Discussion, Vol. 8, No. 10, Oct. 1986, pp. 66-68.
9. Schlaich, J, Schafer, K, and Jennewein, M, "Toward a Consistent Design of Structural Concrete", PCI Journal, Vol. 32, No. 3, May-June, 1987, pp. 74-150.
10. Rogowsky, D.M., MacGregor, J.G., and Ong, S.Y., "Test of Reinforced Concrete Deep Beams", ACI Journal, Vol. 83, No. 4, July-Aug. 1986, pp. 614-623.
11. Rogowsky, D.M., and MacGregor, J.G., "Design of Reinforced Concrete Deep Beams", Concrete International, Vol. 8, No. 8, Aug. 1986, pp. 49-58.

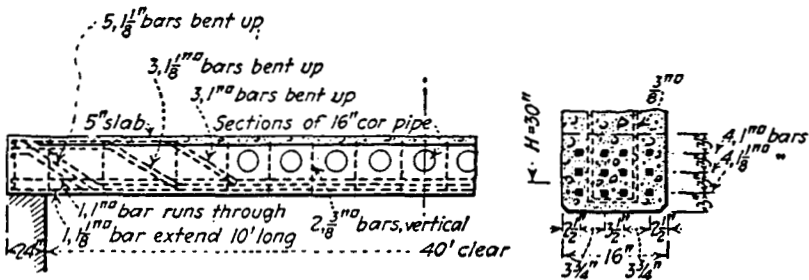
10 Rogowsky

12. Cook, W.D., and Mitchell, D., "Studies of Disturbed Regions near Discontinuities in Reinforced Concrete Members", ACI Structural Journal, Vol. 85, No. 2, Mar.-Apr. 1988, pp. 206-215. Also, Discussion, Vol. 86, No. 1, Jan.-Feb. 1989, pp. 108-111.
13. Jirsa, J.O., Bergmeister, K., Anderson, R., Breen, J.E., Barton, D., and Bouadi, H., "Experimental Studies of Nodes in Strut-and-Tie Models", IABSE Colloquium on Structural Concrete, Stuttgart, 10-12 April, 1991, IABSE Report, Vol. 62, Zurich, 1991, pp. 525-532.
14. Bergmeister, K., Breen, J.E., and Jirsa, J.O., "Dimensioning of the Nodes and Development of Reinforcement", IABSE Colloquium on Structural Concrete, Stuttgart, 10-12 April, 1991, IABSE Report, Vol. 62, Zurich, 1991, pp. 551-556.
15. Adebar, P., and Zhou, Z., "Bearing Strength of Compressive Struts Confined by Plain Concrete", ACI Structural Journal, Vol. 90, No. 5, Sept.-Oct. 1993, pp. 534-542.
16. MacGregor, J.G., "Reinforced Concrete Mechanics and Design", 3rd Ed., Prentice Hall, 1997, 939 pp.
17. Collins, M.P., and Mitchell, D., "Design Proposals for Shear and Torsion", PCI Journal Vol. 25, No. 5, Sept.-Oct. 1980, 70 pp.
18. Marti, P., "Dimensioning and Detailing", IABSE Colloquium on Structural Concrete, Stuttgart, 10-12 April, 1991, IABSE Report, Vol. 62, Zurich, 1991, pp. 411-443.
19. Hutchings, J.B., "Concrete T-Beams Designed as Truss", Engineering News-Record, Vol. 86, No. 5, February 3, 1921, pp. 233.

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(a)



(b)

Figure 1. Traditional Design Example [19]

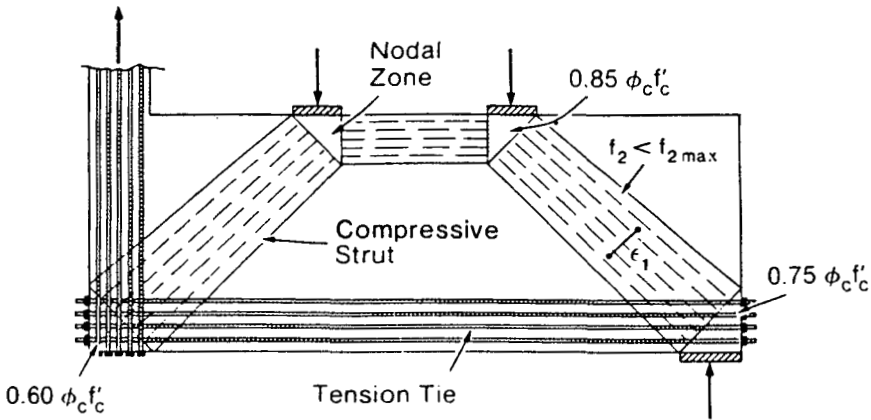


Figure 2. Components of a Modern Truss Model [3]

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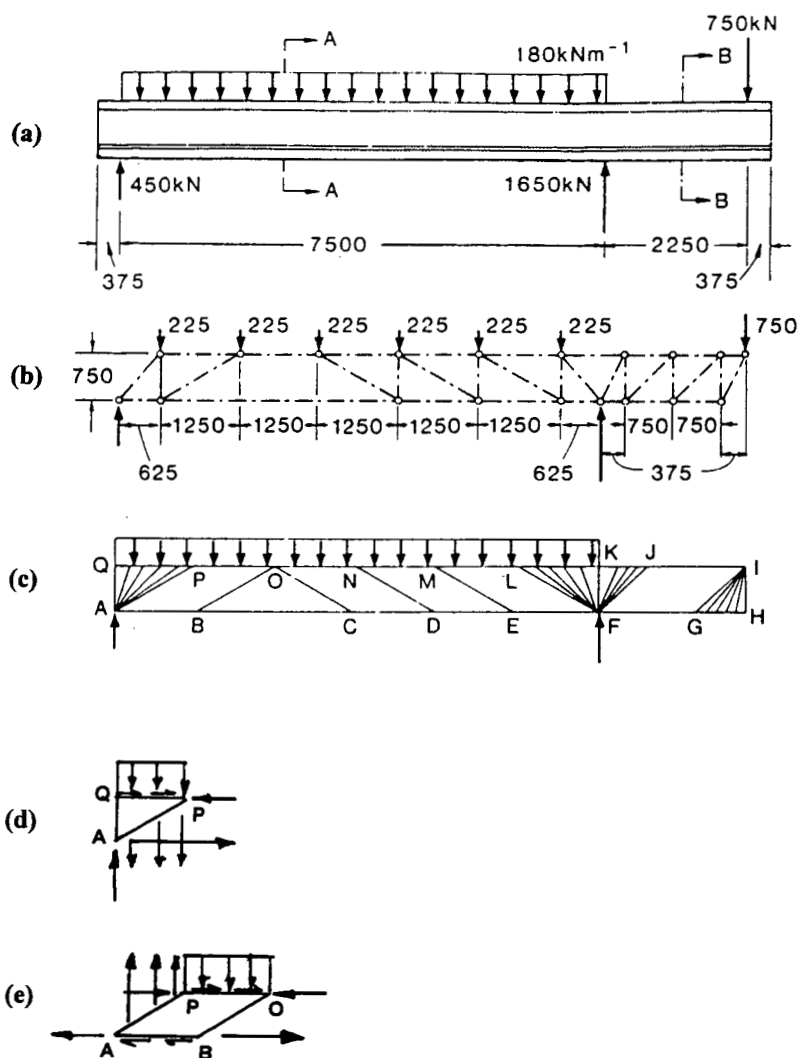
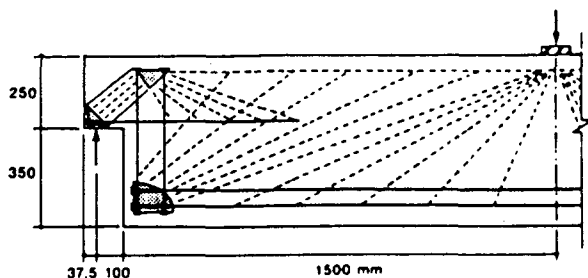
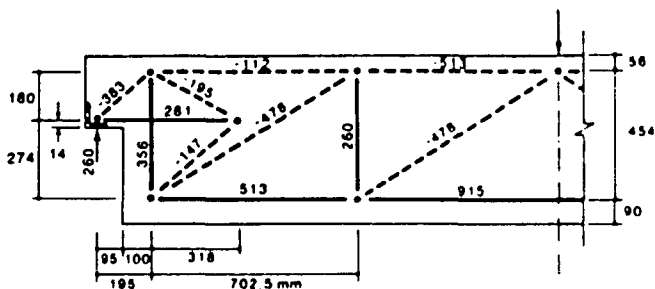


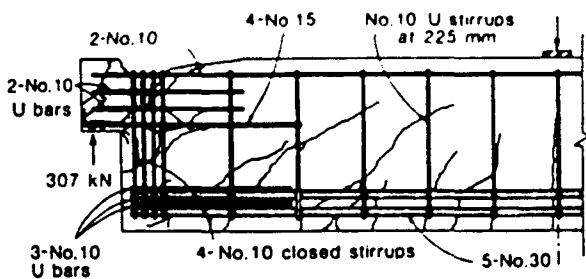
Figure 3. Example of Stress Fields [7]



(a) Strut and tie model



(b) Truss idealization and member forces in kN



(c) Reinforcement details and failure conditions

Figure 4. Experimental Verification of Strut and Tie Models [12]

Reliability Concepts and Realistic Design Standards

by R. G. Sexsmith

Synopsis: Professor Jim MacGregor began his doctoral studies at the University of Illinois just as his research advisor had become concerned with the safety provisions of the ACI building code. As MacGregor began his teaching and research career at the University of Alberta, developments in the reliability area became suitable for structural applications, especially in providing a basis for load and resistance factors. MacGregor was able to combine his exceptional talents in reinforced concrete behavior and design with the reliability-based code developments. This combination of expertise in reinforced concrete with an interest in safety requirements of codes has resulted in significant advances in the safe and economical design of concrete structures.

Keywords: reinforced concrete; safety index; structural safety

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INTRODUCTION

Professor Jim MacGregor has made many contributions to the understanding of behavior and design of concrete structures. Because of his keen interest in the appropriate implementation of design principles in the codes and standards, he also made significant contributions to the development of reliability concepts applied to reinforced concrete design. The development of the key concepts of structural reliability took place over the past thirty years, a time that has coincided with a large part of Jim MacGregor's career.

This discussion outlines the development of structural reliability concepts that have now become part of the basis for most design codes, and the role played by Jim MacGregor in applications to concrete design. His record of achievement in the integration of reliability concepts to concrete design practice provides inspiration to the profession, and is a fine example for both researchers and practitioners.

Jim MacGregor began graduate studies at the University of Illinois in the late 1950's, with Professor Chester Siess as his advisor – about the best start anyone could have had in the field of reinforced concrete. The 1956 ACI code (1) had included an abstract of the report of the ACI-ASCE Joint Committee on Ultimate Strength Design (2), the first proposal for load factors and limit state principles in North America. At that time George Winter of Cornell University was interested in a rational basis for safety in the code, and he and others had developed a design philosophy in which strength would have a small likelihood (about 10^{-3}) of being under the factored strength, and load effect would have a small likelihood (about 10^{-2}) of exceeding the factored load effect. The result was an approximate estimate of probability of failure of 10^{-5} over the structure's lifetime. Winter and Siess were both on the ACI 318 committee and had a great deal of influence on the code. It was during this time that Jim completed his Ph.D., and joined the faculty at the University of Alberta.

What Jim developed under the tutelage of Chester Siess at Illinois, and brought to Alberta, to the ACI, and to the profession of engineering was a profound knowledge and understanding of the behavior of concrete, and a foresight

of what was needed to develop the subject further. Because he worked closely with Professor Siess on code-related matters, he also became interested in the safety factors and in the need for approaches that would help to make the choice of safety factors more rational. Thus his interests were focused on fundamentals of behavior and concerns of the code.

The 1963 ACI code (3) incorporated ultimate strength into the body of the code in parallel with working stress design. Approximate probabilities were estimated for exceedence of factored loads and for shortfalls of factored strength, maintaining an overall probability of failure of 10^{-5} . Reliability needs were recognized, but actual developments in structural reliability were taking place in the aircraft and space-industries, with little applicability to the realities of concrete.

RELIABILITY CONCEPTS

By the mid-1960's, the aircraft and spacecraft industries, and many other fields, had begun to embrace probability-based reliability methods, and researchers were taking an interest in applying these methods to the problems of structural safety. ACI started a Committee on Structural Safety (ACI 348) which was charged with development of safety concepts within ACI.

Probability-based reliability methods have had some special difficulties in application to structural engineering. Structural reliability is extremely high, thus failure rates are so small as to be unquantifiable. Data on material strength and extreme loads is correspondingly scarce. Structures are one-of-a-kind, hence frequency interpretations of probability may be meaningless. Finally, the concept of failure probability for a civil engineering structure has not been accepted by most designers and clients. These difficulties have been gradually overcome in recent years, but were quite pronounced in the mid-sixties.

In 1967 Allin Cornell presented, in a memorandum to the ASCE Task Committee on Structural Safety, a method of estimating safety on the basis of the expected values and coefficients of variation of load and strength. This was later published in the ACI Journal (4), generating much interest among those concerned with structural safety. Cornell's proposal, for calculation of a "safety index" or "reliability index" is illustrated in figure 1. The upper part of the figure contains a probability distribution of load effect L and of resistance R . When $R < L$, failure occurs. If a function $Z = R - L$ is defined, failure corresponds to $Z < 0$, and the probability of failure is the area under the density function of Z to the left of $Z = 0$. Cornell's proposed method was based on the function $Z = R - L$, and an alternative $Z = \ln R/L$, again where $Z < 0$ corresponds to failure. The lower part of the figure

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illustrates the distribution of Z and the corresponding probability of failure. From the figure, the mean of Z is

$$\bar{Z} = \beta \sigma_z \quad (1)$$

Where σ_z is the standard deviation of Z , and β , the reliability index, is a measure of the distance in units of standard deviation of the mean of Z from the failure event $Z=0$. The reliability index is then

$$\beta = \bar{Z} / \sigma_z \quad (2)$$

β is a measure of safety that avoids the idea of "probability of failure", and is based only on estimates of the means and standard deviations of the load and resistance variables, thus avoiding many of the difficulties related to scarce data. If it is assumed that R and L are normally distributed, then $Z = R - L$ is also normally distributed, and the probability of failure can be directly related to β using tables of the normal probability function. Similarly, where R/L can be assumed lognormally distributed, the corresponding distribution of $Z = \ln R/L$ is normal, and again the probability of failure is directly related to β .

Cornell's contribution generated much interest because it recognized major limitations to probability applications in civil engineering structures. Complete probability models, of interest in the aerospace industry, were impractical because a knowledge of probability distributions of the variables was simply not available, and could not be available, for many civil engineering structures. A method based on means and standard deviations, and avoiding the dreaded "probability of failure" was very attractive.

Equation (2) can be used to determine the safety index when the means and standard deviations (or variance) for the load and resistance are known or assumed. Conversely, if a target safety index is established, load and resistance factors can be chosen to achieve the target when the variance of the variables is known or assumed. Prior to the introduction of the safety index concept by Cornell, little effort had been made by structural researchers in the identification of the needed data on loads and resistances. Cornell's contribution increased the interest in gathering and classifying information on the mean values and variability of loads and resistances.

Since the original idea was introduced in 1967, the idea of the reliability index has evolved, with contributions by many. The original concept did not require assumptions of probability distributions of the load and resistance variables, but the reliability index could be directly related to probability statements if normal or lognormal variables were assumed. It was only a matter of time before the idea of transforming named probability density functions to normal density functions opened the method to applications where such other distributions were known. Another major area of development was the recognition that the two variables, load and resistance, were insufficient. Ditlevsen showed that it was not always possible to identify "load" and "resistance" uniquely. The need for extensions to many variables was soon met, and several authors, e.g. Rack-

witz and Fiessler (5), produced efficient computer codes to deal with a large number of reliability problems.

The definition of the failure surface (the function of variables expressing failure, e.g. $R-L = 0$) became a means of classification. A first order approximation of the failure expression is called the First Order Reliability Method (FORM). If a non-linear failure expression is expressed up to second order approximations, the method is called a Second Order Reliability Method (SORM). Currently there are applications with discontinuous failure expressions, many random variables, and many different probability models. In all cases the goal is to assess the reliability. In many cases the mathematical advances have gone far beyond the real level of practical information about the variables, their variability, and the structural behavior and load behavior being represented.

CONTRIBUTIONS TO STRUCTURAL SAFETY BY MACGREGOR

While reliability theory developed as outlined above, Jim had become an internationally known expert in the behavior and design of concrete structures, and had maintained his interest in the design codes. He realized that practical applications in structural reliability depended upon a solid information base, especially in the poorly understood area of quantifying variability. In the mid 1970's a series of projects aimed at quantifying uncertainty in concrete began, much of the work with Sher Ali Mirza. Several publications involved Monte Carlo simulation (6,7). Others were studies of variability (e.g. 8). There were also publications on codes in the area of limit states design (9). MacGregor's expertise in reinforced concrete had prepared him to begin the specific developments in reliability that depend on material knowledge as well as reliability concepts.

A major event in structural reliability development took place in the summer of 1979, when Bruce Ellingwood organized a workshop with the goal of developing probability-based load criteria for the American National Standard A58. The project was entitled "Development of a Probability-Based Load Criterion for American National Standard A58", and sponsored by the National Bureau of Standards. The participants were Ellingwood, Ted Galambos, Jim MacGregor, and Allin Cornell. MacGregor and Galambos represented concrete and steel, respectively, while Cornell and Ellingwood were the probability experts, and Ellingwood had assembled much of the available information on loads and other materials.

The four researchers showed the value of cooperation to advance a topic. Over a period of a few months, generally in residence at the NBS, a landmark publication was produced (10). It collected in one place the best available information on bias and variability in materials and loadings, and used first order reliability methods to assess reliability index for current design standards. Inconsistencies in safety were identified, and recommendations for load and resistance factors were provided that would lead to more consistent safety. Jim MacGregor was uniquely qualified to contribute to this important document. His intimate knowledge of

concrete behavior, understanding of the practical requirements of codes, and background in studies of variability of concrete, provided the right combination to develop and enhance the synergy with the other participants. The publication remains a basis for code developments in all the civil engineering materials.

During this time, Jim was active in code development in Canada. He chaired a task force on loads for the National Building Code of Canada and has served as chairman of the Canadian Concrete Code, in addition to many other similar activities.

MacGregor maintained extensive research in the reliability area since the NBS project, as exemplified by numerous publications in this area (e.g. 11). Some of his most recent projects involve variability in concrete and its modeling. He has, more than any others, combined reliability and concrete, to the benefit of all.

STRUCTURAL SAFETY TODAY AND IN THE FUTURE

We now take for granted that behind the code load and resistance factors is a probability basis that is founded on the goal of consistent safety. Due to the efforts of Jim and others (e.g. Galambos in the steel area), the codes have simple formats, similar to the format introduced in the 1963 ACI Building Code. In the concrete area, MacGregor promoted successfully the need for separate resistance factors on the steel and the concrete materials. This has been adopted in Canada but is still to come in the United States. It looks to the future, where the use of very high strength concrete requires a separation of the σ factors.

Outside of the codes, reliability applications are now routinely made on large projects to develop appropriate load and resistance factors. This was done for the Alex Fraser Bridge in British Columbia 15 years ago, for many offshore structures over the past decade or two, and for many special or large projects with unusual risks. The most notable recent example is the Confederation Bridge, where many studies culminated in establishment of load and resistance factors for all aspects of the design, specifically tailored to the unique circumstances of this unusual bridge. Jim MacGregor was one of the major participants in the development of load and resistance factors for the bridge design (12).

In the mathematical development, the improved computing power and better algorithms have expanded the use of the SORM methods to deal with situations that involve large computational effort to determine the failure surface, irregular and discontinuous failure surface, many variables, inverse reliability (e.g. load factors directly computed for specified reliability index), and alternative computational methods such as importance sampling. The mathematical development has again outstripped our ability to make engineering design decisions.

The future will continue to yield more and better information, now on whole structures rather than elements, and on overall response in the

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range. It will require many to follow in the footsteps of Jim MacGregor, to keep focused on the behavior and design requirements and not only on the mathematical developments.

As in developments in other fields, the goals in structural safety will move in the future. While the past goal has been consistent safety, the future should see safety variation consistent with the levels of risk and cost. It is time to put greater safety where there are greater consequences, greater safety where the cost of achieving it is relatively modest, reduced safety where the consequences are less severe or the cost of attainment is unusually high. This is optimization of expected cost, where costs are present expected values, and expectation is in the sense of probabilistic expectation. This does not depend as much on mathematical development as on understanding true behavior. Jim was great at using such understanding to achieve the present. Those who follow his example will develop the future.

ACKNOWLEDGMENT

To Jim, you have been an inspiration to many; you have made a difference to the world.

REFERENCES

1. ACI 318, "Building Code requirements for Reinforced Concrete, ACI 318-56", American Concrete Institute, 1956, Detroit.
2. ACI-ASCE Joint Committee, "Report of the Joint Committee on Ultimate Strength Design", Proceedings, ASCE, V. 81, No. 809, Oct. 1955. New York.
3. ACI 318, "Building Code requirements for Reinforced Concrete, ACI 318-63", American Concrete Institute, 1963, Detroit.
4. Cornell, C.A., "A Probability-based Structural Code", Journal of the ACI, Proc. V. 66, No. 12, Dec. 1969, American Concrete Institute, Detroit.
5. Rackwitz, R., and Fiessler, B., "An Algorithm for Calculating Structural Reliability under Combined Loading", Technische Universitat Munchen, Heft 17, 1977, Munich.
6. Grant, L.H., Mirza, S.A., and MacGregor, J.G., "Monte Carlo Study of Strength of Concrete Columns", Journal of the ACI, V. 75, No. 8, 1978, Detroit.
7. Ramsey, R-J., Mirza, S.A., and MacGregor, J.G., "Monte Carlo Study of Short-time Deflections of RC Beams", Journal of the ACI, V. 76, No. 8, 1979, Detroit.
8. Mirza, S.A., Hatzinkolas, M., and MacGregor, J.G., "Statistical Descriptions of the Strength of Concrete", Journal of the Structural Division of ASCE, V. 105, No.6, 1979, New York.
9. MacGregor, J.G., "Safety and Limit States Design for Reinforced Concrete", Canadian Journal of Civil Engineering, V.3, 484-513, 1976, National Research Council of Canada, Ottawa.
10. Ellingwood, B., Cralambos, T.V., MacGregor, J.G., and Cornell, C.A., "Development of a Probability Based Load Criterion for American National Documents

standard A58", NBS Special Publication 577, National Bureau of Standards, June 1980, Washington, D.C.

11. Mirza, S.A. and MacGregor, J.G., "Probabilistic Study of Strength of Concrete Members", Canadian Journal of Civil Engineering, V.9, No. 3, 1982, National Research Council of Canada, Ottawa.

12. MacGregor, J.G., Kennedy, D.J.L., Bartlett, F.M., Chernenko, D., Maes, M.A., and Dunaszegi, L., "Design Criteria and Load and Resistance Factors for the Confederation Bridge." Canadian Journal of Civil Engineering, V.24, No. 6, 1997, National research Council of Canada, Ottawa.

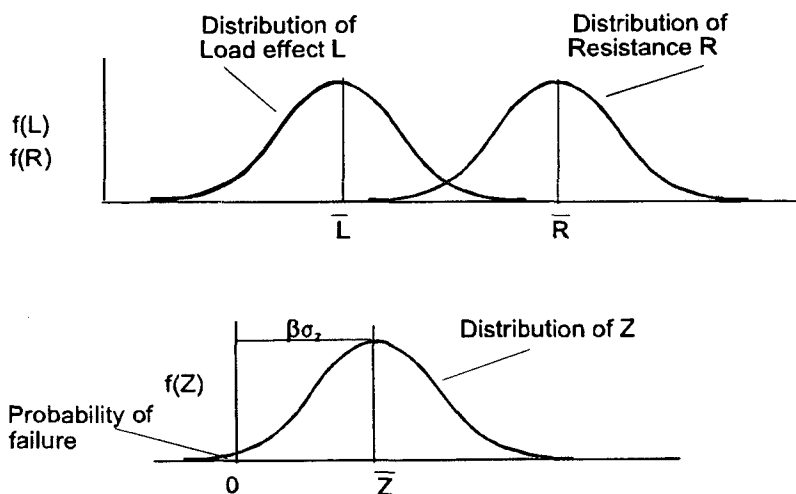


Fig. 1 Reliability index β

Structural Design for High-Strength Concrete— Important Code Aspects

by D. Mitchell

Synopsis: This paper describes some of the important aspects of code requirements for the design of structural members made with high-strength concrete. The provisions of the 1994 CSA Standard for the Design of Concrete Structures are highlighted. Comparisons are made between the provisions of the 1999 ACI Code and the 1994 CSA Standard. Some aspects of design from the 1990 CEB-FIP Model Code and the 1995 New Zealand Standard are also discussed.

Keywords: codes; design; flexure; high-strength concrete; shear

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INTRODUCTION

This paper summarizes the special design requirements for high-strength concrete in the 1994 Canadian Standards Association (CSA) Standard A23.3-94 "Design of Concrete Structures"(1). The differences between the provisions of the 1994 CSA Standard and those in the 1999 ACI Code (2) are discussed.

"LIMITS" ON SPECIFIED COMPRESSIVE STRENGTH OF CONCRETE

The provisions in the 1994 CSA Standard are applicable for designs having a specified concrete compressive strength not less than 20 MPa nor more than 80 MPa. The Standard permits designs with concrete compressive strengths greater than 80 MPa, provided that the structural properties and detailing requirements for these higher strength concretes are established for concretes similar to those to be used. An upper limit of 80 MPa was chosen for the compressive strength of the concrete because of the insufficient amount of test data on the response of structural elements constructed with very high-strength concrete. Furthermore, it is pointed out that high-strength concretes vary in their brittleness and need for confinement.

A cautionary note in the Standard indicates that designers planning to use high-strength concretes should determine whether such concretes are available in their region, and it also points out that it may be necessary to undergo prequalification of concrete suppliers and contractors.

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MODULUS OF ELASTICITY

The 1994 CSA Standard provides three methods for determining the modulus of elasticity, E_c , at the design stage, as follows:

(1) by testing cylinders of similar concrete.

(2) by determining the modulus of elasticity, E_c , for concrete with γ_c between 1500 and 2500 kg/m³ as

$$E_c = \left(3300\sqrt{f'_c} + 6900 \right) \left(\frac{\gamma_c}{2300} \right)^{1.5} \quad \text{MPa units} \quad (1)$$

where γ_c is the density of concrete in kg/m³.

This equation is based on the work of Carrasquillo et al. (3) and gives a more appropriate expression for the modulus of elasticity, E_c , for concrete compressive strengths greater than 40 MPa.

(3) by determining the modulus of elasticity, E_c , of normal density concrete with compressive strength between 20 and 40 MPa as

$$E_c = 4500\sqrt{f'_c} \quad \text{MPa units} \quad (2)$$

This simpler expression is suitable for use with lower strength concretes and is about 5% lower than that given in the 1999 ACI Code (2).

The CSA Standard also emphasizes that the modulus of elasticity of high-strength concrete is highly dependent on the properties of the aggregates used and hence can vary in different regions of the country (Baalbaki et al. (4)). If the modulus of elasticity is critical to the design, then a minimum value of E_c shall be specified and shown on the drawings.

SPECIAL CONFINEMENT REQUIREMENTS FOR HIGH-STRENGTH CONCRETE COLUMNS

Studies on high-strength concrete columns by Yong et al. (5), Cusson and Paultre (6), Ibrahim and MacGregor (7) and Polat (8) have indicated the need to provide a greater degree of confinement to improve the ductility of high-strength concrete columns. There is also a tendency for splitting cracks to form in high-strength concrete columns, resulting in premature spalling of the concrete cover and a reduction in the capacity as discussed by Collins et al. (9).

Reduced Tie Spacing Limits

The 1994 CSA Standard requirements for maximum tie spacings in columns made with normal-strength concrete are the same as those in the 1999 ACI Code. However, the tie spacing requirements for columns made with concrete having a specified compressive strength greater than 50 MPa are reduced by multiplying the tie spacing limits of the ACI Code by 0.75. Thus for higher strength concrete columns, the tie spacing shall not exceed the smallest of:

- (a) 12 times the diameter of the smallest longitudinal bars or the smallest bar in a bundle;
- (b) 36 tie diameters;
- (c) 0.75 times the least dimension of the compression member; and
- (d) 225 mm in compression members containing bundled bars.

The additional confinement for high-strength concrete columns is aimed at providing greater confinement and improved ductility.

Improved Anchorage Details for Ties

The 1994 CSA Standard also requires that ties in columns made with specified concrete compressive strengths greater than 50 MPa have standard tie hooks with a bend of at least 135°. The typical 90° bend anchorages are replaced with 135° bend anchorages for the ties in high-strength concrete columns in order that the ties remain effective even if some concrete cover spalling takes place.

Improved Confinement for Fire Resistance

While there are a number of ongoing research projects aimed at developing a better understanding of the influence of high-strength concrete on the fire resistance of columns, a number of important observations have been made (Phan (10)). It has been found that high-strength concrete can experience "explosive spalling" when subjected to rapid heating. The rate of temperature increase, the type of aggregates, the details of reinforcement, the moisture content and the porosity of the concrete are all important factors which affect the spalling of the concrete. Due to the low permeability of high-strength concrete it tends to retain moisture and hence rapid heating leads to the development of

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large internal vapor pressures which results in explosive "spalling". Because the fire resistance is affected by loss of cover, it is believed that the 1994 CSA Standard with more closely spaced ties with 135° bend anchorages would improve the fire resistance of high-strength concrete columns. Research is currently underway to study the effects of these new details and the influence of steel fibers on the fire resistance of high-strength concrete columns.

DETERMINING FLEXURAL AND AXIAL LOAD RESISTANCES OF HIGH-STRENGTH CONCRETE ELEMENTS

In the determination of the factored resistance for flexure and axial loads the 1994 CSA Standard prescribes a plane-sections approach using one of the following two methods:

Integration of Realistic Stress-Strain Relationships

The position and magnitude of the resultant compression in the concrete may be found by first assuming a realistic stress-strain relationship between the compressive stress and concrete strain and then integrating these stresses. This relationship must account for the fact that as the concrete strength increases the compressive stress-strain curves exhibit greater initial stiffness, greater linearity and decreased ductility. Figure 1 shows plots of stress strain relationships which illustrate these features (11).

In using this approach it is necessary to account for differences between the in-place strength and the strength of standard cylinders. In this regard, the CSA Standard requires that the compressive stress-strain curve for the in-place concrete (both normal and high-strength) be based on stress-strain curves with a peak stress no greater than $0.9f'_c$.

Stress Block Factor Approach

The 1994 Standard provides an alternate approach using an "equivalent rectangular concrete stress distribution", together with a maximum strain at the extreme concrete compression fiber of 0.0035. A value of 0.0035 was chosen to better reflect the extreme concrete compressive fiber strain at flexural ultimate for a large range of concrete compressive

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strengths. The equivalent rectangular concrete stress distribution is defined by the following:

- (a) a concrete stress of $\alpha_1 \phi_c f'_c$ shall be assumed uniformly distributed over an equivalent compression zone bounded by edges of the cross section and a straight line located parallel to the neutral axis at a distance $a = \beta_1 c$ from the fiber of maximum compressive strain;
- (b) the distance c shall be measured in a direction perpendicular to that axis; and
- (c) the factors α_1 and β_1 shall be taken as:

$$\alpha_1 = 0.85 - 0.0015 f'_c \geq 0.67 \quad (3)$$

$$\beta_1 = 0.97 - 0.0025 f'_c \geq 0.67 \quad (4)$$

where f'_c is in MPa.

These stress block factors are more suitable for a wider range of concrete strengths than those in the previous 1984 CSA Standard (12). The stress block factors are intended to account for both the significant change in shape of the stress-strain curves as the concrete strength increases and the difference between the concrete cylinder strength and the in-situ strength of the column concrete.

Figure 2 illustrates the two analysis procedures for determining the factored flexural resistance of members. Figure 3 compares the factored moment resistances, M_r , using the new stress block factors of the 1994 Standard, with those determined using stress block factors of the 1984 CSA Standard, for a 400 mm by 600 mm deep beam singly reinforced with 3 No. 30 bars. The two approaches are very similar for the case of pure moment. This figure also illustrates that for this example, doubling the concrete compressive strength results in only about a 6% increase in pure moment capacity due to the fact that the capacity is controlled by yielding of the reinforcement.

Maximum Axial Load Resistance

The maximum factored axial load resistance P_{rmax} of compression members is determined from the following expressions:

(a) for spirally reinforced columns $P_{rmax} = 0.85 P_{ro} \quad (5)$

(b) for tied columns $P_{rmax} = 0.80 P_{ro} \quad (6)$

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where P_{ro} is the factored axial load resistance at zero eccentricity defined as:

$$P_{ro} = \alpha_1 \phi_c f'_c (A_g - A_{st} - A_t - A_p) + \phi_s f_y A_{st} + \phi_a F_y A_t - f_{pr} A_p \quad (7)$$

where

- ϕ_c = resistance factor for concrete, equal to 0.60
- A_g = gross area of section
- A_{st} = total area of longitudinal reinforcement
- A_t = area of structural steel shape
- A_p = area of prestressing tendons
- ϕ_s = resistance factor for reinforcing bars, equal to 0.85
- ϕ_a = resistance factor for structural steel, equal to 0.90
- f_{pr} = stress in prestressing tendons when concrete reaches limiting compressive strain
- f_y = specified yield strength of reinforcement
- F_y = specified yield strength of structural steel section

The value of P_{rmax} in the 1994 CSA Standard is a function of the stress block factor, α_1 , which varies with the concrete compressive strength. Figure 4 compares the P_{rmax} values calculated using the 1984 (similar to the 1999 ACI Code approach) and 1994 CSA Standards for a 450 mm by 450 mm column containing 8 No. 20 bars. For this column, containing 1.2% steel, the new provisions give about 4%, 9% and 13% lower capacities for concrete strengths of 30, 60 and 80 MPa, respectively.

Limit of c/d for Yielding of Tension Reinforcement

Due to the change in the maximum strain at the extreme concrete compression fiber from 0.003 to 0.0035 in the 1994 CSA Standard, the tension reinforcement in flexural members shall not be assumed to reach yield unless:

$$\frac{c}{d} \leq \frac{700}{700 + f_y} \quad (8)$$

where c is the depth of compression, d is the effective depth of the member and f_y is in MPa. For flexural members, without axial loads, the area of tension reinforcement is limited such that Eq.(8) is satisfied. In columns or walls, when c/d exceeds this limit, the stress in the tension reinforcement must be computed based on strain compatibility.

MINIMUM REINFORCEMENT FOR FLEXURAL MEMBERS

The purpose of providing a minimum amount of flexural reinforcement is to ensure a ductile flexural response. An insufficient amount of flexural reinforcement can result in the flexural capacity being lower than the cracking moment, resulting in a brittle response after cracking.

The 1994 CSA Standard gives three alternative approaches for satisfying the requirements for minimum reinforcement as given below:

(1) At every section of a flexural member where tensile reinforcement is required by analysis, minimum reinforcement shall be proportioned so that:

$$M_r \geq 1.2 M_{cr} \quad (9)$$

where the cracking moment, M_{cr} , is calculated using the modulus of rupture, f_r .

(2) In lieu of satisfying Eq. (9), minimum reinforcement may be determined from:

$$A_{smin} = \frac{0.2 \sqrt{f'_c}}{f_y} b_t h \quad \text{mm, MPa units} \quad (10)$$

where b_t is the width of the tension zone of the section considered. However, for slabs and footings only the minimum amounts for "temperature and shrinkage" need be provided.

(3) The requirements (1) and (2) above may be waived if the factored moment resistance, M_r , is at least one-third greater than the factored moment, M_f .

Older codes (12, 13) typically required a minimum reinforcement ratio that was a function of the yield strength of the reinforcement, but was not a function of the concrete strength. The 1994 CSA Standard and the 1999 ACI Code make the minimum amount of reinforcement a function of not only f_y , but also $\sqrt{f'_c}$ to account for the higher cracking moment as the specified concrete strength is increased. Figure 5 shows the variation of the reinforcement ratio, ρ_{min} , with increasing compressive strength. With the 1994 CSA provisions, a smaller amount of minimum reinforcement is required than previous code provisions for concrete strengths below about 40 MPa. Larger amounts of minimum reinforcement are required by the 1994 Standard for flexural members with concrete compressive strengths above 40 MPa.

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The purpose of minimum reinforcement is to avoid brittle failure of members once cracking has occurred and to help in providing control of cracks at service load levels. Figure 6 compares the minimum reinforcement ratio, ρ_{min} , required by the 1999 ACI Code and the 1990 CEB-FIP Model Code (14) for elements subjected to flexure as a function of the concrete compressive strength. The ACI requires larger values of ρ_{min} for larger members and significantly smaller amounts for slabs, because slabs are considered to be more ductile and have greater ability for redistribution than beams. On the other hand, CEB-FIP requires a larger ρ for thinner members than for thicker members, because thinner members are considered to have a higher cracking stress due to size effects. Although both approaches are based on sound principles the resulting provisions are quite different. It is important to note that the ACI and CSA specifications for the Grade 400 reinforcement have larger values of minimum elongation and larger ratios of ultimate stress to yield stress than that required in the CEB-FIP code. It is clear that more research is needed on determining rational amounts of minimum reinforcement for flexural elements.

TRANSMISSION OF COLUMN LOADS THROUGH FLOOR SLABS

When the specified compressive strength of concrete in a column is greater than that specified for a floor system, transmission of the column load through the floor system needs to be investigated. There are three alternate methods of dealing with this issue in the 1994 CSA Standard as described below:

(1) "Puddled" High-Strength Concrete in Slab - One method is to provide concrete of the strength specified for the column, f'_{cc} , "puddled" in the floor slab at and around the column locations. The top surface of the column concrete placed in the floor shall extend at least 500 mm into the floor from the face of the column to be considered effective (see Fig. 7(a)). This usually requires that the higher strength concrete in the floor in the region of the columns be placed before the lower strength slab concrete is placed to avoid accidental placement of the lower strength concrete in the column area. In addition, the lower strength concrete must be placed while the higher strength concrete is still plastic and must be vibrated to ensure integration of the two concretes.

(2) Accounting for Confinement Effects - Another approach is to take account of the confining effect of the floor slab surrounding the column (see Fig. 7(b)). The 1994 Standard prescribes an effective concrete compressive strength, f'_{ce} , which differentiates between the different

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amounts of confinement at interior, edge and corner columns as shown in Fig. 7(b).

(3) Use of Vertical Dowels - The third approach is to supplement the strength of the weaker concrete, "sandwiched" between the upper and lower column, by adding sufficient vertical dowels to enhance the capacity locally (see Fig. 7(c)), or by adding spirals or hoops in this critical region to increase the confinement and effective strength of the concrete in the slab region.

MINIMUM SHEAR REINFORCEMENT

The purpose of minimum shear reinforcement is to prevent brittle shear failures and to provide adequate control of shear cracks at service load levels (9). The 1984 CSA Standard (12), like the 1983 ACI Code (13), required a minimum area of shear reinforcement equal to $0.35b_ws/f_y$ (i.e., stirrups to carry 50 psi) which is independent of the concrete strength. As the concrete compressive and tensile strengths increase, the cracking shear also increases. This increase in cracking shear requires an increase in minimum shear reinforcement such that a brittle shear failure does not occur upon cracking. The 1994 CSA Standard (1) specifies the minimum amount of shear reinforcement as a function of not only f_y , but also $\sqrt{f'_c}$ to account for the higher cracking shear as the specified concrete strength is increased. Where shear reinforcement is required, the minimum area of shear reinforcement shall be such that:

$$A_v = 0.06 \sqrt{f'_c} \frac{b_w s}{f_y} \quad \text{mm, MPa units} \quad (11)$$

Figure 8 compares the 1994 CSA and the 1999 ACI required amounts of minimum shear reinforcement. The CSA requirements provide a more gradual increase in the required amount of minimum shear reinforcement as the concrete strength increases.

Tests carried out by Yoon et al. (15) on large beams with concrete strengths varying from 36 MPa to 87 MPa indicated that the amount of minimum shear reinforcement prescribed by the 1994 CSA Standard provides adequate control of diagonal cracks at service load levels and provides reasonable levels of ductility.

SIZE EFFECT IN SHEAR DESIGN

The influence of size on the shear stress at failure of members without transverse reinforcement is an important issue which is addressed in the 1994 CSA Standard. This effect is accounted for in the General Method for shear design, based on the modified compression field theory developed by Collins et al. (16). In this general approach, the ability of the concrete to transmit shear along the cracks is reduced as the size of the member is increased. The 1994 CSA Standard also provides a simplified approach to account for the size effect. In the simplified approach, the factored shear resistance of normal-density concrete members having an effective depth greater than 300 mm and with no transverse reinforcement, or less transverse reinforcement than that required by Eq.(11) is given by:

$$V_c = \left(\frac{260}{1000+d} \right) \phi_c \sqrt{f'_c} b_w d \leq 0.10 \phi_c \sqrt{f'_c} b_w d \quad (12)$$

where b_w is the web width in mm, d is the effective depth in mm, V_c is in N and f'_c is in MPa units.

Preliminary tests of large concrete beams without stirrups indicate that high-strength concrete members may be more sensitive to size effects than normal-strength concrete members (Collins et al (9)). Tests are continuing to investigate the influence of size, strength of concrete and the role of the amount of longitudinal reinforcement on the shear strength of members without transverse reinforcement.

GENERAL CRACK CONTROL REQUIREMENTS

An important issue is the amount of uniformly distributed reinforcement required in disturbed regions, such as deep beams, corbels, dapped ends, etc. The 1994 CSA Standard has provisions for the design of these important members using strut-and tie models, which were introduced in 1984 (12). Apart from providing tension ties required in these members the Standard also requires a minimum amount of uniformly distributed reinforcement in both directions in order to control cracking at service load levels. The Standard requires a reinforcement ratio for this uniformly distributed steel of 0.002 (i.e., 0.2%). For bridge members this crack control reinforcement is increased to 0.003 (1991 Ontario Highway Bridge Design Code (17) and the 1994 AASHTO LRFD Bridge Design Specifications (18)). The larger amount of crack control reinforcement

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for bridge members is due to the concern of corrosion protection. One question to be resolved is whether or not this minimum crack control reinforcement should be made a function of the concrete strength, following the same philosophy as used for the requirements for minimum flexural reinforcement and for minimum shear reinforcement. Preliminary research by MacLeod et al. (19)) involving the testing of full-scale bridge cap beams, indicates that:

- (a) a reinforcement ratio of 0.0018 for the uniformly distributed crack control reinforcement was sufficient to provide adequate crack control for normal-strength concrete (38 MPa) specimens, and
- (b) companion high-strength concrete (79 MPa) specimens required a reinforcement ratio of 0.003 to adequately control cracking at service load level.

DEVELOPMENT OF REINFORCEMENT

The 1994 CSA Standard has adopted the same approach as the 1999 ACI Code by limiting the maximum permissible value of $\sqrt{f'_c}$ to 8 MPa when calculating development lengths. More research is needed on this important topic since there is a tendency for more brittle bond failures in high-strength concrete members unless an adequate amount of transverse reinforcement is provided (20, 21).

BENEFITS FOR CONCRETE CAST IN CERTIFIED PRECAST PLANTS

The 1994 CSA Standard employs material resistance factors rather than capacity reduction factors when calculating the factored member resistance. For elements produced in certified manufacturing plants, the concrete material resistance factor, ϕ_c , may be taken as 0.65, rather than 0.60. This increased value of ϕ_c , reflects the better quality control achieved in certified precast plants.

CONCRETE COMPRESSIVE STRENGTH LIMITS FOR SEISMIC DESIGN

The "Special Provisions for Seismic Design" in the 1994 CSA Standard limits the specified compressive strength, f'_c , used in design to 55 MPa. These special provisions contain the seismic design and detailing requirements for ductile and nominally-ductile structural elements. This conservative approach was deemed necessary by the code committee

because of the brittle nature of high-strength concrete and because of the lack of tests of high-strength concrete elements subjected to reversed cyclic loading. The 1995 New Zealand Standard (22) limits the specified concrete compressive strength to 70 MPa for ductile elements and elements of limited ductility. The 1995 New Zealand Standard introduced new equations for determining the confinement required for ductile response of columns, based on studies by Watson et al. (23) and Watson and Park (24). The 1999 ACI Code (2) does not have an upper limit on the specified concrete strength for the design of ductile elements.

Considerable effort is being made to study the influence of high-strength concrete on the performance of ductile reinforced concrete elements. Research is continuing on the amount of confinement required and the influence of steel fibers on the response of columns, as well as the behavior of frame elements and walls. Preliminary results by Bristowe et al. (25) indicate that high-strength concrete coupling beams actually exhibit greater ductility than companion normal-strength concrete beams.

CONCLUSIONS

While there are many similarities between different codes for the design of high-strength concrete structural members, there remains significant differences in the approaches taken. It is clear that more research is required in many areas before rational approaches, based on sound principles, will be adopted. In particular, more experimental research on high-strength concrete elements is required in several areas including: columns; beams and slabs with minimum amounts of reinforcement; shear tests of large high-strength concrete members without transverse reinforcement; and elements subjected to reversed cyclic loading.

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REFERENCES

1. Canadian Standards Association, "CSA A23.3-94 Design of Concrete Structures", Rexdale, 1994, 199 p.
2. American Concrete Institute, Building Code Requirements for Structural Concrete (ACI 318-99) and Commentary, Detroit, MI, 1999, 369 p.
3. Carrasquillo, R.L., Nilson, A.H., and Slate, F.O., "Properties of High Strength Concrete Subject to Short-Term Loads", ACI Journal, V. 78, No. 3, May-June 1981, American Concrete Institute, Detroit, pp. 171-178.
4. Baalbaki, W., Benmokrane, B., Chaallal, O. and Aïtcin, P.-C., "Influence of Coarse Aggregate on Elastic Properties of High-Performance Concrete", ACI Materials Journal, Vol. 88, No. 5, Sept.-Oct. 1991, pp. 499-503.
5. Yong, Y.K., Nour, M.G., and Nawy, E.G., "Behavior of Laterally Confined High Strength Concrete under Axial Loads", Journal of Structural Engineering, Vol. 114, No. 2, Feb. 1988, ASCE, pp. 332-351.
6. Cusson, D. and Paultre, P., "High Strength Concrete Columns Confined by Rectangular Ties", Journal of Structural Engineering, Vol. 120, No. 3, Mar. 1994, ASCE, pp. 783-804.
7. Ibrahim, H.H.H. and MacGregor, J.G., "Flexural Behaviour of High Strength Concrete Columns", Structural Engineering Report No. 196, Dept. of Civil Engineering, University of Alberta, 197 p.
8. Polat, M.B., "Behaviour of Normal and High Strength Concrete under Axial Compression", Master's thesis, Department of Civil Engineering, University of Toronto, 1992, 175 p.
9. Collins, M.P., Mitchell, D., and MacGregor, J.G., "Structural Design Considerations for High-Strength Concrete", Concrete International, Vol. 15, No. 5, May 1993, American Concrete Institute, Detroit, pp. 27-34.
10. Phan, L.T., "Fire Performance of High-Strength Concrete: A Report of the State-of-the-Art", NISTR 5934, National Institute of Standards and Technology, Gaithersburg, Maryland.