

Serviceability *of* Concrete

A Symposium Honoring Dr. Edward G. Nawy



American Concrete Institute®

Advancing **ACI** concrete knowledge

SP-225

Editor: Florian Barth

Co-Editors: Robert Frosch,
Hani Nassif, and Andrew Scanlon

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Serviceability of Concrete

A Symposium Honoring Dr. Edward G. Nawy

Preface

Edward G. Nawy Symposium

Friends, former students, and colleagues of ACI Honorary Member Dr. Edward G. Nawy collectively planned and organized a much overdue tribute held at the ACI Spring Convention in New York City on April 17 and 18, 2005. The Edward G. Nawy Symposium was sponsored by ACI Committees 224, Cracking; 421, Design of Reinforced Concrete Slabs; and 435, Deflection of Concrete Building Structures. The symposium and this special publication celebrate and honor the distinguished career of Dr. Nawy as an outstanding educator, committed researcher, influential author, and professional engineer.

The symposium consisted of three technical sessions with a total of 16 presentations. The symposium speakers and authors of articles in the volume consisted of friends, former students, and colleagues who traveled from around the world to join Dr. Nawy during this special tribute.

Dr. Nawy is a distinguished professor in the Department of Civil and Environmental Engineering at Rutgers University, The State University of New Jersey. His commitment to engineering excellence during his nearly one-half century of service to the public has earned him world-renowned recognition for his extensive work in the fields of reinforced and prestressed concrete, and particularly for addressing serviceability issues such as crack and deflection control. Not only has Dr. Nawy published over 175 technical papers during his career, he is also well known for his concrete design textbooks. The “Nawy Books” are the first choice for the concrete engineering community, and his detailed approach and timely editing in response to continuous code changes has resulted in his textbooks being the preferred choice of educators, as well.

This symposium and special publication also recognized the outstanding contribution Dr. Nawy has made to the American Concrete Institute. He joined ACI in 1959, and is the founding chair of ACI Committee 224, Cracking. Dr. Nawy has essentially attended every committee meeting of ACI Committee 224 since its formation in 1966, and is still an active contributor to the committee. He is an active member of ACI Committee 421, Design of Reinforced Concrete Slabs, and past chair and current member of committee 435, Deflection of Concrete Building Structures. He was the first chair of the ACI Chapter Activities Committee, where he helped implement key policies still used today. He is the recipient of numerous ACI and other academic awards and is an ACI Honorary Member.

This special publication offers technical papers on aspects of concrete serviceability, with emphasis on concrete cracking and deflection for prestressed and non-prestressed members.

Much of the effort of preparing this special publication was carried out by the technical session moderators, who deserve special thanks for their great support: Ted Krauthammer and Hani Nassif for Session No. 1; Debrethann Orsak and Andrew Scanlon for Session No. 2; and Robert Frosch and Harvey Haynes for Session No. 3. Of course, it goes without saying that this volume would not have been possible without the authors, the reviewers of the papers, and the continuous assistance of Todd R. Watson of ACI staff in guiding the editors through the process.

The editors of this Special Publication would like to express their gratitude to Dr. Nawy for his guidance, sound advice, and friendship over the years, and to all those who contributed in making the symposium and dinner a memorable event for Dr. Nawy.

Whether he is a mentor, teacher, colleague, or friend, we all have learned to depend on Dr. Nawy's achievements and on his dependable, hard-working character. Dr. Nawy has inspired us by displaying such exemplary conduct, outstanding dedication, and consummate technical knowledge throughout his many years of service to the engineering community and ACI.

Florian Barth
FBA Inc. Structural Engineers
Hayward, California
December 2004

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Theory of Elastic Analysis— Illusion and Superstition

by R. W. Furlong

Synopsis: Building Codes have specified for the purpose of design, that “*Theory of Elastic Frames*” be used for analyses of indeterminate structures. Sophisticated computer software has been developed based on the condition of elastic response to loading. Designers rely on computed results from such analyses as if those results were perfect and reliable evaluations of structural behavior. The influence of assumptions regarding accepted simplifications, member stiffnesses, load definitions and frame connectivity is addressed. Reliability from elastic analyses is revealed as an illusion, and less sophisticated alternate analytic requirements are suggested.

Keywords: analysis; frame analysis; structural design

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Introduction

*“All members of frames or continuous construction shall be designed for the maximum effects of factored loads as determined by the **theory of elastic analysis**, except as modified according to 8.4.”* ACI 318-02, “Building Code Requirements for Structural Concrete”¹, contains these words, and previous editions have contained the same mandate since Strength Design became part of the Code in 1963. Section 8.4 of ACI 318-02 permits negative moments determined from theory of elastic analysis to be modified by the designer within limits expressed in terms of anticipated flexural ductility. The Manual of Steel Construction – Load and Resistance Factor Design² – Section A5.1 permits the designer to use either elastic or plastic analysis to determine maximum effects of factored load. How did ACI318 reach its current position regarding analysis?

Basic civil engineering curricula in the United States before World War II offered analysis courses applied primarily to statically determinate systems. American college curricula for civil engineering began to include iterative moment distribution procedures^{3,4} only after World War II. Applications of the theory of elastic analysis to indeterminate systems have been familiar to most American structural engineers for a mere 60 years, perhaps two generations of designers. Sixty years ago, the analytic effort with slide rules remained rather tedious. In order to reduce required analytic effort, Section 702 of ACI 318-47⁵ and similar sections of subsequent issues permitted simplified substructures within monolithic concrete frames as acceptable for design.

The advent of digital computers after 1950’s expanded the degree of complexity that designers could consider, but designer efforts were hardly less tedious for the first generation of engineers trained for indeterminate analysis. Early computer use was quite expensive, often with the computational help of rented time on large computers. During the 1980’s, comprehensive digital computer programs began to appear on the desk of every designer, removing analytic tedium even for complex building structures. Today’s third generation of designers can perform from their desktop computers elastic analyses with impressive calculation precision and elegant graphical displays of results. After the somewhat tedious process of defining loads and structures for the computer, elastic analysis has become relatively painless, and designers can enjoy the cozy illusion that computed numbers are precise with analytic results that are completely acceptable in accordance with current design rules.

How reliable or “accurate” are results based on elastic analysis? Is there any other basis for determining required strength? How precise are the safety indices that have been derived from an elastic analysis? Should more than one analytic model be required for analyzing response to the variety of loading conditions specified for a complete design of specific buildings or bridges?

The Structural Model for Elastic Analysis

Elastic analysis requires that force and deformation be proportional and that the structural system be completely resilient. After loading is removed, deformations from the loading disappear. Material and geometric properties of structural members are unchanged as loads are applied or removed, and superposition of various load cases always produces a corresponding superposition of deformations.

Analysis requires a definition for each of 6 parameters for every segment of every member, material stiffness E , Poisson's ratio ν , the geometric properties of area A , flexural shape I , shear shape A_v , and torsion shape J . For purposes of this discussion let it be assumed that there are no loads that cause deformations involving significant twisting or unusually high shear. Without twisting, only 3 parameters, material stiffness E and geometric properties A and I need to be considered. Members themselves are generally represented as linear elements along which all force components act and all deformations occur. The linear elements are the geometric centroid axes of each line-element member.

Almost all members of building structures are perceived with a cross section that remains the same from one end to the other. Such members are classified as prismatic members, and the product EI and the product AE are constant from one end to the other. If the geometric properties A and I vary along the length of a member, the member is classified as non-prismatic. Analytic procedures are available to incorporate non-prismatic properties into an elastic analysis for a member. However, unless non-prismatic dimensions of members can be defined, reinforced concrete structures are considered to be composed of prismatic members.

What value should be used for E , for A and for I when reinforced concrete members are to be analyzed? Traditionally, any influence of steel reinforcement has been ignored during the frame analysis part of design, and values A and I have been based on the outline of the concrete cross section, commonly referred to as the "gross" section. Without any influence from steel reinforcement, a value for E based exclusively on concrete has been taken as the secant modulus $E_c = w_c^{1.5} 0.043 \sqrt{f'_c}$, or $4700 \sqrt{f'_c}$ (in MPa units for E and for f'_c). Thus, the products $A_g E_c$ and $I_g E_c$ have been used as stiffness values of prismatic members for frame analysis. A modest sophistication has been employed by some designers, who have used for beams a value I taken as only half that of the gross concrete outline. That sophistication was employed in recognition that beams were likely to be cracked by flexure under service load, whereas columns and walls were not likely to be cracked under service load conditions.

A further sophistication in ACI 318-47 permitted designers to use for design the forces at the faces of joints rather than the forces at the theoretical node at which line-element members actually intersect. Without additional advice regarding determination of "face-of-joint" forces, designers simply used the forces that would occur along prismatic members at the face of joint.

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Prismatic members with or without modified proportions of stiffness or without simplified substructure models continue to be acceptable for the elastic analyses that is required by current ACI 318-02 design rules.

Monolithic Reinforced Concrete Structures - Only a Concept

Each of the simplifying assumptions accepted by ACI 318-02 results in some variance between analytic results and results obtained from more complex procedures that have not been simplified. Variances from specific influences will be examined.

Actual reinforced concrete members are not prismatic. From joint node to joint node, cross sections along any member must vary as the effective cross section between the joint nodal point and face of joint will be much larger than the effective section between faces of joints. "Exact" elastic models should employ non-prismatic members with greater stiffness through joint regions at the ends of members. Also, and of more significance, concrete cracks must form in flexural tension before reinforcement helps significantly to resist applied moments. Such flexural cracking not only reduces effective flexural stiffness at cracked sections, but cracking alters the position of the elastic centroid of the cracked section, such that the neutral axis along a linear member cannot be a straight line from end to end after flexural cracking.

The influence of longitudinal reinforcement should not be ignored. The amount and the position of longitudinal reinforcement are not known for any structural component at the time designers begin the process of structural analysis. Longitudinal reinforcement is assigned on the basis of results obtained from the requisite elastic analyses, such that the amount and the location of reinforcement is not the same at every position along the length of each member. Steel is 7 to 9 times stiffer than concrete, but the amount of steel in any section is only 1 to 2 percent of the area of concrete. Nevertheless, in sections before concrete cracks, longitudinal reinforcement increases flexural stiffness as much as 10% to 15% above the value for concrete alone. After concrete cracks in tension, steel reinforcement influences flexural stiffness significantly.

Beams that support monolithic slabs have a T-shaped section for which the slab helps resist compression in positive moment regions between supports, but for which slabs can be cracked in flexural tension near supports. Dramatic changes that can occur in response to cracking are illustrated by an example given in Fig.1. The figure displays graphs of bending stiffness EI together with graphs of bending moment along a continuous 3-equal-span beam under constant uniform loading. The (black) graphs connecting diamond-shaped data points represent the prismatic beam before any cracking. The moment diagram for the prismatic beam customarily would be used for designing the beam. Maximum moments at joints would be -160 kN-m, and positive moments as high as 125 kN-m occur in each end span.

The (pink) graphs that connect square data points represent the same beam and loading after a flexural crack forms at interior Support 2, reducing EI to half its value before cracking. Analytically, that one crack reduces the negative moment at the 2nd

support from -160 kN-m before cracking to -30 kN-m after cracking, while the moment at the 3rd support increases from -130 kN-m before cracking to -340 kN-m after cracking. The positive moments in the left span increase to 185 kN-m. Before this scenario would occur, the large moment over support 3 would cause a crack at the 3rd support. The (blue) graphs connecting x's represents the same beam and load with cracks over both interior supports. That graph is similar to the graph for response of the prismatic beam, except that the maximum negative moments at interior supports are smaller (-130 kN-m) and positive moments are larger (+130 kN-m) than were the values before any cracks occur. Positive moment in the center span increased from 40 kN-m before cracking to 80 kN-m after cracking at the supports..

The (yellow) graphs connecting triangular data points represent the beam with a 50% EI reduction due to a positive moment crack only in the left span. That graph indicates a modest reduction from +130 kN-m to +110 kN-m in the left span, a modest increase in negative moment at Support 2 from -160 kN-m to 170 kN-m. However, at Support 3 there is a large increase in negative moment from -130 kN-m to -280 kN-m. The large increase in moment in a support implies that the positive moment crack in an end span would be accompanied by cracking also at support regions.

Finally, a (brown) graph connecting a lined x (*) represents the same beam and loading with positive moment cracks in both end spans and negative moment cracks over each interior support. That graph suggests that moments at supports increase from -160 kN-m before cracking to -205 kN-m after cracking while maximum positive moments in end spans decreased from 125 kN-m before cracking to 105 kN-m after cracking. There are no positive moments in the uncracked center span after cracking in end spans and at supports.

It is quite obvious that as reinforced concrete members become non-prismatic due to cracking, response to load differs significantly from response of the members before cracking. Flexural cracking reduces cross section effective flexural stiffness EI to values far less than the values for sections before cracking. An estimate of stiffness EI for sections after cracking can be derived for the section if specific reinforcement areas are known at the myriad of anticipated locations of flexural cracking. Even though elastic theory can be applied after a multitude of cracked sections might be defined, the process would be extremely more complex but no more certain of accuracy, than a study of prismatic members before cracking. Reinforced concrete flexural members are neither elastic nor prismatic under service loads, and their response diverges even further from that of prismatic models as loads approach strength limit values required for design. Is it logical to apply prismatic member elastic theory to unpredictably non-prismatic members in the design process? Is there an alternate to elastic analysis for determining reliable levels of strength in non prismatic reinforced concrete members?

How Precise Are Loads for Which Structures Are Designed?

Loads on buildings and other structures can be classified into three categories, 1) those which generate vertical forces due to gravitational action – gravity loads, 2) those

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which generate horizontal forces – lateral loads, and 3) those which create forces due to volume changes within the structural system – environmental loads.

Gravity loads include Dead Load, which is the self-weight of a structure plus the weight of permanent attachments to the structure. It is possible to estimate the magnitude and location of Dead Loads more accurately than the magnitude of any other loads. Even when magnitudes and locations of dead loads are known, estimates of their effect on indeterminate reinforced concrete structures are subject to the uncertainties described in the preceding section. All other gravity loads can be called Live Loads. The magnitudes and locations of Live Loads such as movable equipment and vehicular traffic can be estimated with precision. However, most buildings must be designed for estimated “occupancy” loads such as snow, rain, furniture, human assembly, floor cover and temporary storage. Since the magnitude and location of occupancy loads overall are impossible to define, estimates of such loading must be made. Building codes specify recommended magnitudes for many occupancy loads⁶. If the potential area to be loaded exceeds 20 or so sq meters, the intensity of live load is not likely to be maintained over larger areas. As loading area increases, occupancy Live Loads can be reduced from 100% to as little as 40% of the total possible magnitude if the area of occupancy loading is large enough. Forces from moving traffic and from vibrating equipment include some dynamic effects that may be estimated as a percentage of the static, non-vibratory weight. Totally apart from the vagaries of results from elastic analysis of indeterminate frames, no precise magnitude of any occupancy Live Load can be established.

Lateral loads include actions of soil, water and wind against structures as well as base displacements from earthquake motions. The magnitude and location of lateral loads are even less predictable than are those from gravity type loads. Lateral pressure from soil against walls is influenced by the nature of the soil, the stiffness of the wall, and by variables such as depth to water tables and superimposed overburden on backfill. Lateral pressures from wind, both positive and negative, are only as predictable as is a velocity and direction of wind, environmental turbulence, configuration and texture of a structure and the vertical elevation of exposed surfaces. Similarly, loading from earthquake motions is only as predictable as is the magnitude, direction, frequency and duration of ground motion. Since ground motion initiates movement of structures from inert positions to dynamically active conditions, the weight and location of inert masses together with the stiffness and strength of building components influence loading from seismic events.

Predicting the precise intensity of seismic, tornado and cyclone events is beyond current ability to predict the causative influences. Consequently, structural engineers must use “extreme case” models of such events, acknowledge that damage of building components will occur, and design structural elements with adequate toughness to sustain inelastic deformations without overall loss of capacity to support the structure.

Significant environmental loads on reinforced concrete structures can occur from shrinking or swelling of building components due to changes in temperature and humidity. Environmental loadings do not produce overall changes in the magnitude of

either gravity or lateral loading, but they may increase tension or compression forces to values higher than values due to gravity or lateral loading. The consequences of such environmental loading can influence apparent serviceability, but the overall strength of structures is not changed if the structure is ductile enough to sustain inelastic deformation without significant loss of strength. Thus, structural engineers safely can employ such devices as expansion joints and separation from expansive soil in order to maintain acceptable serviceability and appearance of ductile structural members without superimposing design forces from environmental loading.

Strength, Safety and the Analytic Method

Columns and walls must be designed such that supported beams and slabs will sag visibly to warn of distress from overload before columns or walls fail. Structural safety then becomes a function of flexural behavior. Shear failure can occur before significant inelastic flexural deformation of sections takes place. Therefore, both the analytic basis for predicting shear failure and the coefficient that reflects reliability must be set to values less than those applied for flexural failure. It is mandatory that flexurally ductile failure precede any other type of failure.

Safety requires that there must be strength adequate to sustain required forces at every component of a structure. Tools of structural analysis are applied in order to establish values for required forces, and safety is insured after principles of materials and mechanics are used to insure that designed strength values exceed values of required force. Whereas the theory of elastic analysis requires that all members connected at a joint rotate through the same angle when forces change, the theory of plastic frames does not require rotational continuity at joints. Instead, the theory of plastic analysis recognizes that prior to flexural failure, structural components can endure inelastic rotational deformation without significant loss of strength when forces change. Of course, both elastic analysis and plastic analysis as well as any intermediate method must insure equilibrium of required force prior to failure.

Flexural strength at any reinforced concrete section is reached at the onset of compression spalling of concrete. Sections in which tension bars must stretch 2 to 3 times the yield strain of reinforcement before bars rupture will not experience compression spalling of concrete before section failure. Flexural strength in any beam can occur only after the formation of an unstable mechanism consisting of virtual hinge regions in 3 positions, one at each end of a beam span and one between the ends of the span. After 3 hinges form, no increase in load is necessary before unlimited tension strains occur and compression spalling ensues. For the special case of a simply supported span, the ends are always hinged. The third hinge position will occur wherever capacity to resist positive moment is reached between the ends of the span. Similar, if not identical, levels of safety can be obtained either from results of plastic analysis or from results from elastic analysis or from any intermediate limit load analysis that recognizes inelastic flexural rotations without significant loss of strength.

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Structures Influenced by Lateral Displacement – Sway Frames

Structures for which lateral forces impose significant forces on columns must be considered as unbraced frames or frames that sway. Structures in which lateral bracing resists all lateral forces such that columns are not affected by lateral forces are considered to be braced or non-sway structures. Shear walls provide lateral bracing in non-sway reinforced concrete structures. Only gravity-type loads need to be considered for the design of beams and columns in non-sway structures. Under lateral loading, shear walls must be sufficiently stiff that lateral displacements between floors are so small that forces at beam and column joints are virtually unchanged from values under gravity loading alone. Inelastic displacements due to seismic loading of non-sway frames obviously compel beam and column members to help resist displacement, but the beam and column members can be designed on the premise that seismic-augmented loads act against a non-sway structure.

Structures without stiff lateral bracing rely on the flexural stiffness actions of beams and columns to resist lateral loads. When there is lateral displacement between two levels, structural mass on the displaced upper level increases overturning loading in proportion to the amount of lateral displacement. The increased overturning load in its turn again creates more overturning action, but in stable structures the process converges to a condition of static equilibrium. Iterative applications of elastic analysis can be employed to estimate the iterated magnitude of beam and column forces at joints. Analytic inconsistencies between actual values EI and values EI used for analysis as previously described for gravity loading apply also to behavior for lateral loading. The consequences of inaccuracy are similar to those for analysis of resistance to gravity type load; estimated forces at joints will differ from actual forces, but as inelastic deformation of ductile components occurs, failure cannot take place until the designed strength of components is reached.

Stiffness values used for analysis must be smaller than those actually anticipated prior to failure, in order that harmful effects from lateral displacements are not underestimated. After more than 5 decades of silence regarding acceptable stiffness assumptions for elastic analysis, ACI Committee 318 provided advice in ACI 318-95⁹ with the stipulation that results from elastic analysis for lateral load cases are acceptable if stiffness values EI are not greater than $0.35I_gE_c$ for beams and cracked walls, nor $0.70I_gE_c$ for columns and uncracked walls. These stiffness values, intended for use to produce estimates of joint forces from elastic analyses of reinforced concrete structures subject to lateral loads, may be used also for analysis of response to gravity type loads. Second order displacements need not be considered in analyses of response to gravity load. It has been demonstrated that the actual stiffness values in beams change significantly in response to loads such that almost any flexural stiffness values including those specified in ACI 318-95, Section 10.11.1 can be useful for analysis of gravity load forces. Analytic results will be useful, but hardly any more “accurate” for design than any sets of forces in equilibrium at the required load condition.

Rational Estimates of Required Strength and Ductility

Every member of a structure must be strong enough to sustain the actions of loads that act on the structure. Columns and walls must be proportioned and reinforced such that thrust as well as flexural action and shear cannot cause visible cracks or rupture. Beams must be reinforced such that tension forces from shears, joint moments and span moments never cause visible cracks or rupture. Estimates of required strength can be obtained with the theory of elastic analysis, but such estimates also can be obtained with the theory of plastic analysis.⁷ The theory of elastic analysis requires that deformations be compatible at joints, but that stipulation is not required with plastic analysis. Reinforced concrete beams easily can be reinforced for plastic moment capacities that need not be the same at every point along a beam span. However, both positive and negative values of plastic moment are the same when plastic analysis is applied with structural steel rolled shape members.

The degree of flexural ductility necessary for hinge mechanisms to develop under gravity loading is greater when all plastic moments on a beam are the same, than the flexural ductility necessary if plastic moments can be proportioned with negative plastic moments larger than positive plastic moments. Concrete cross sections may have the same shape throughout continuous beams, but the amount of flexural tension reinforcement determines plastic moment capacity. Thus, it is customary and simple to design for different plastic moment capacities for positive and for negative moments of reinforced concrete beams. Under the action of gravity loading, the basic moment value for spans designed by plastic analysis is the magnitude of maximum moment in the span under the action of required transverse loading. That quantity will be designated as M_{su} , the maximum moment created by required loads on the span if it were simply supported.

It has been demonstrated that reinforced concrete beams can be designed for gravity loading with moment capacities simply assigned on the basis of desired limit strength (plastic moments).⁸ In order to insure adequate strength and ductility until plastic hinge mechanisms develop, some restrictions must be specified if limit design with plastic analysis is permitted for constant depth reinforced concrete beams that support only gravity loads:

a) Reinforcement shall be continuous across interior supports extending at least the tension bar development length beyond each face of the support.

b) Plastic moment values:

Negative moment capacity at ends of spans shall be no less than $0.6M_{su}$.

Positive moment capacity within spans shall be no less than $0.4M_{su}$.

c) Shear capacity values:

Interior spans shall be adequate to sustain required loading on spans with no rotational restraint at either end.

The exterior end of exterior spans shall be adequate to sustain required loading on exterior spans with no rotational restraint at either end.

The interior end of exterior spans shall be adequate to sustain 115% of required loading the spans with no rotational restraint at either end.

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Reinforced concrete columns can be designed for gravity loading with axial forces above each level based on contributing areas and with required end moments not less than $0.3M_{su}$ for either span supported by the column at the same level.

The plastic analysis procedure is not well-suited to applications with combined loading on multi-story frames that can sway. Not only must the most severe load combinations be considered, but the sequence of loading also must be considered for plastic analysis. Even with its known inaccuracies, an elastic analysis procedure may offer the least complicated analytic procedure for estimating forces required to design members in frames that sway.

The theory of elastic analysis can provide reliable estimates of strength required at all critical points of reinforced concrete structures. Numerous reasons have been given to explain why such estimates differ from values that would occur if required loading actually were applied to members of the structure. Discrepancies between estimated and “actual” design forces may exceed 25% of the estimated values, but flexurally ductile concrete members will deform inelastically to accommodate required loading without rupture. Discrepancies between estimated and “actual” service load forces, even with environmental loading, generally will not be large enough to reveal undesirable cracking. Also the theory of elastic analysis offers the least complex technique to be used for the various load combinations required for designing members that resist combined load cases on frames that sway.

Conclusion and Recommendation

The magnitude of internal forces necessary for designing reinforced concrete structures can be derived either from applications of theory of elastic frames or from theory of plastic analysis or from any limit strength condition between elastic response and fully plastic response. The accuracy, here defined as the ratio between estimated internal force and actual internal force, can vary from 2 to $\frac{1}{2}$ when the theory of elastic frames is employed for analysis. A similar accuracy can be obtained from analyses based on the assignment of limit forces or from a plastic analysis of response to required loads.

An elastic analysis for estimated forces will produce values that differ significantly from actual forces that can occur within flexurally cracked members. Although precision from such analysis is an illusion, the procedure identifies for all possible load conditions the limit strength values at all critical locations. If identified strength is provided at each critical section, and general detailing requirements insure ductility, failure cannot occur before any one of numerous possible load cases acts against a member.

The magnitude of internal forces necessary for designing reinforced concrete structures also can be derived from applications of plastic analysis. A plastic analysis for estimated forces necessary for designing laterally braced (non-sway) reinforced concrete structures may be less complex and easier to apply than elastic analysis. General detailing requirements insure ductility adequate for reinforced concrete members to

experience inelastic deformation until required plastic strength limits are reached. Certainly, plastic analysis is quite adequate for reviewing or designing any member of a non-sway frame.

Applications of plastic analysis to sway frames may become far more complex than applications of elastic analysis for sway frames. The complexities grow rapidly as load cases and load sequences must be considered in plastic analysis, whereas load sequences are not relevant in elastic analysis with its superposition of load case maxima. On the basis of analytic simplicity, elastic analysis is recommended for design of members in unbraced frames (sway frames). While recognizing that precision is illusory, in order to insure that second order effects of displacement are never underestimated, it is mandatory that overall stiffness models be more flexible than any actual behavior.

Elastic analysis should be permitted but not required for designing reinforced concrete structures. It is recommended also that plastic analysis procedures be permitted.

Precision from elastic analysis is an illusion. Elastic analysis becomes a superstition only when its use is required by design codes.

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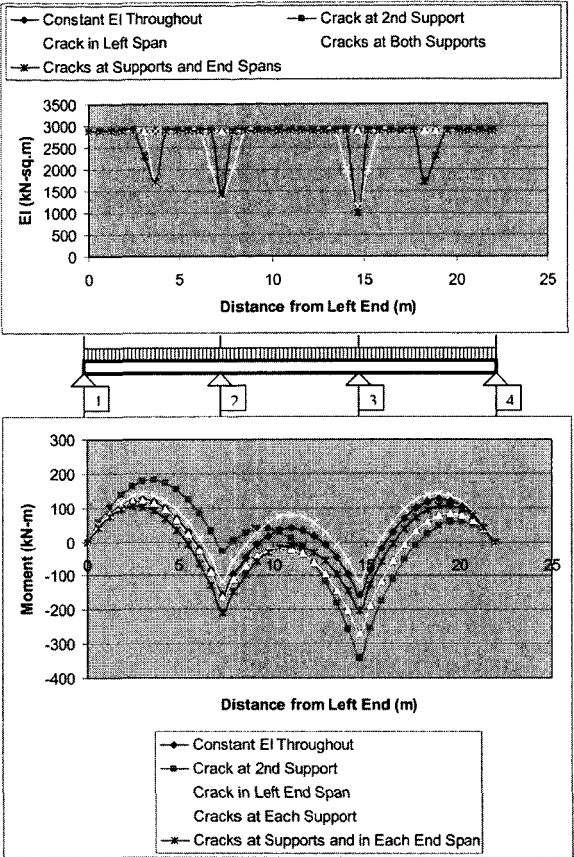


Fig. 1 – Effect of discrete cracks in 3-span continuous beam

Prescriptive or Performance Design for Fire?

by P. J. E. Sullivan

Synopsis: Three approaches to fire design are currently available:

- (1) The direct application of data from furnace tests on simply supported members heated at the ISO 834 Time/Temperature standard rate.
- (2) Conventional ultimate limit state design in bending for fire, taking into account idealised material losses in strength at elevated temperatures.
- (3) Complete analysis of structural members using iterative finite difference or finite element techniques to estimate temperature gradients and determining the resulting stress and strain resultants.

Prescriptive design falls into the first category and the thickness of the protective cover to the steel together with the minimum dimensions of the element is tabulated in building codes.

Performance design would be carried out by the other 2 methods, where the applied heat impinging the surface due to a fire is assessed, the temperature distribution evaluated and the deterioration determined. This method is more appropriate for important structures with indeterminate members.

Keywords: design; fire; performance; prescriptive

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Biographical sketch of the author

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Research significance

Prescriptive fire design is based on protective cover derived from simply supported members conditioned to a standard moisture content and loaded with design loads in a furnace heated under a standard rate. This method is only strictly applicable to common and garden buildings where the elements constituting the structural elements may be assumed to be simply supported. However where buildings have indeterminate members in the structure the prescriptive design would be uneconomical and in certain cases may be unsafe and performance design as described in the paper should be undertaken.

Introduction

The objectives of designing for fire resistance are to limit heat transmission through walls and partitions and maintain structural integrity and stability for a structure under fire. The period of exposure is decided from the nature of the contents and the purpose of the structure. The standard form of exposure, originally derived from the results of burn out tests on structures, provides the link between structural components and fire tests ⁽¹⁾. Building regulations accept tabulated values provided in Codes of Practice of protective cover to structural steel and concrete. These values are derived from furnace tests on structural members subjected to a standard exposure condition extending over some specified period. Prescriptive design involves the selection of protective cover and minimum sizes of members from such Codes of Practice. This method is rather restrictive and uneconomic and performance fire design gives more scope to the designer to use normal design methods to improve the fire endurance of structural members.

Design methods

Three distinct approaches to fire design are currently available ⁽²⁾:

- (a) The direct application of data from full size furnace tests using the standard ISO 834 Time/Temperature curve, the values of which are tabulated in Building Codes.
- (b) Conventional ultimate limit state design as applied to fire, where the temperature gradients for a given standard fire severity may be used. These temperatures are used to assess the deterioration of the material

and thus the change of the moment of resistance of the member. Idealised material losses in strength due to elevated temperatures and temperature distributions are provided in codes of practice. This can be considered as an extension of the procedures used in current design for normal temperatures.

(c) Complete analysis of structural members using iterative solutions of finite differences or finite elements to estimate temperature gradients and finite elements to determine the resulting stress and strain resultants.

Method (a) has been in existence for some time and although has a number of shortcomings is a simple and satisfactory tabular design for simply supported members and common and garden structures. However, one has to bear in mind that the tabulated values have been derived from tests carried out in a standard furnace. This does not represent a fire test since by maintaining the heating rate to the ISO 834 curve, the fuel required for materials with a high thermal mass and low diffusivity would be higher than that required for the material with the lower thermal mass and high diffusivity. This is not the case with an actual fire, which has a calorific value dependent on the contents of the room in question and not necessarily on the thermal properties of the supporting structure. Structural members are tested as simply supported in a standard furnace and concrete is conditioned to a certain moisture level. Members in construction are generally not simply supported and concrete may have different moisture content dependent on whether the member is internal or external exposed to wet conditions. Fire-testing full-scale structural steel members in a multi-storey building ⁽³⁾ clearly demonstrated how inappropriate the use of protective cover is, when derived from standard furnace tests.

The general process of performance design (Methods b and c) involves an estimation of the temperature gradients for a given fire intensity resulting in material deterioration and the redistribution of stresses and strains, thus allowing the determination of the structural behaviour of the member. Other than the conventional methods of fire design as in method b, there are a number of computer programs in existence, which have structural and thermal modules capable of assessing the stress resultants at regular intervals due to both applied and thermal loading. The computer programs are being refined and improved as further research data become available. A number of Research Institutions and Universities are involved in studies on all aspects of behaviour of structures under fire which will ultimately lead to a more realistic fire design procedure.

A recent publication ⁽⁴⁾ showing some of these procedures, advanced analytical methods and improved risk assessment techniques have been made available to the experienced engineer to support performance-based design for the fire load case.

Complete 2-D/3D analyses of multi-storey frames subjected to fire in a compartment were first performed at Imperial College in the early 70's ^(5, 6, 7). These were University developed programs and unlike modern commercial programs, were not well structured into discrete modules. Using iterative techniques, the analyses provided a history of the

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variation of bending moments, axial and shear forces, deflections, curvatures etc due to the action of a fire. The formation of plastic hinges around the framework was then determined. This research discovered unexpected failure behaviour and will be described in a later section.

Ultimate Limit Fire Design

This design method has been developed from work done at P.C.A. and other Research Institutions ^(8, 9, 10) and is a useful design tool, which result in an improvement in fire design. The process is familiar to designers of concrete members at normal temperatures.

The principle of this method of design ^(11, 12) can best be illustrated by examining the behaviour of a uniformly loaded simply supported under-reinforced concrete beam or slab heated from the underside. Since the positive steel governs the failure criterion, at normal temperatures one can write:

$$gWL/8 = A_s f_y l_a \quad (1a)$$

where the right hand side (RHS) is the moment of resistance of the steel, the uniform load is W and g is the load factor.

As the temperature of the steel increases during a fire, its yield stress reduces by a factor say Y and therefore

$$WL/8 < A_s Y f_y l_a \quad (1b)$$

and substituting (1a) into (1b) we have

$$gY > 1 \quad (1c)$$

This means that cover is not the only criterion for fire resistance. We can also vary the amount of positive steel reinforcing bars (Y) or the lever arm (depth) or alter the load factor. This gives the designer more latitude with his choice of materials to improve fire resistance. The approach is also more rational and its exponents ^(8,9,10) called it as such.

The trade off between various design factors to achieve a required fire resistance becomes even more important when we have a more practical structural member such as a continuous beam. A uniformly loaded continuous reinforced concrete beam subjected to a fire will now be investigated.

In this instance we have:

$$M_c + M_s = gWL/8 \quad (2)$$

(before heating)

Where

M_c is the moment of resistance at the centre
 M_s is the moment of resistance at the supports
 g is the load factor against collapse.

The effect of fire is to reduce the moment of resistance at the centre to M_{ct} and the resistance at the support may reduce to M_{st} .

For the member to survive the following condition has to apply:

$$M_{ct} + M_{st} > WL/8 \text{ or } (M_c + M_s)/g \quad (3)$$

At normal temperatures one can equate M_c to the moment of resistance provided by the positive steel reinforcement, since it is desirable that the steel rather than the concrete should govern failure, to ensure ductility at the ultimate limit state.

Thus:

$$M_c = A_s f_y l_a \quad (4)$$

As the temperature of the steel increases, the yield stress reduces to Y_f and hence:

$$M_{ct} = Y M_c \quad (5)$$

and substituting equations (5) in (3) and defining r as M_s/M_c which is a measure of the restraint or continuity at the support at normal temperature, we have,

$$g Y > 1 - r (g M_{st}/M_s - 1) \quad (6)$$

Note as the fire is assumed to be below the beam the top steel is not affected by the fire

$$M_{st} = M_s \text{ and } M_{st}/M_c = r \quad (7)$$

Substituting (7) into equation (6) gives

$$g Y > 1 - r (g - 1) \quad (8)$$

i.e. Failure cannot occur if left hand side LHS is greater than the RHS

(Note when the beam is simply supported $r = 0$ and hence:

$$g Y > 1 \quad (9)$$

as in equation 1c for the simply supported case.)

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Hence the fire resistance of a member is increased by increasing the value of the LHS or by decreasing the RHS of inequality (9). Therefore when the ends of the beams have a fixity equal to 1 and, if the load factor $g \geq 2$, the RHS becomes 0 or negative. The LHS is therefore always greater than the RHS and the fire resistance becomes theoretically infinite.

With this rational approach the designer has available to him a wider choice of action to achieve a given fire resistance and a greater flexibility in the options he has for improvement, economy and safety in design. He can improve the fire resistance of the member by:

- a) Increasing the area of reinforcement at the support (or value of r).
- b) 1. Giving a greater protective cover to the bottom steel and/or
2. Changing the type or area of reinforcement and/or
3. Increase the lever arm (depth)
4. Reduce the intensity of the fire if this warrants it.
- c) Increasing the load factor g

These results illustrate the link between applied loading, fire intensity and the necessity for good detailing in determining fire resistance. If tabulated values recommended in Building/Structural Standards are selected, then the designer has little option in exercising his ingenuity to improve the fire resistance of his structure.

Shortcomings

The above examples only deal with flexure but other members subjected to other stress regimes can also be considered in a similar fashion. Greater difficulty may be encountered, where the member is designed for failure of the concrete to occur in compression. However a simplified method of design for fire exposed concrete columns due to Hertz⁽¹³⁾ has been included in reference 3 and the Danish code of practice DS411. The author considers effects of fires on various faces of the column and deals with eccentrically loaded columns with cracked and un-cracked cross sections.

As in normal design it has been assumed in the continuous beam example that full moment redistribution can take place and although this frequently happens, errors in detailing can reduce this ductility. The method results in a particular application of the Upper Bound Theorem of Plasticity where an estimate of the collapse load is made on the basis of an assumed collapse mechanism (see figure 1). The theorem requires the material to exhibit perfect plasticity and it is this property, which allows the separation of kinematic and static variables and enables conditions at collapse to be considered without reference to the previous history of loading and deformation.

The conventional ultimate load method for beams is applicable only where the span to depth ratio does not make shear the limiting criterion for collapse, but fire design on the

basis of limiting shear or compression has not yet been attempted. Comparison of the prescriptive and this "analytical" method of fire design can be found in a 1978 publication of the Institution of Structural Engineers (London). An example of a slab supported on edge beams for a prescriptive 2-hour fire resistance can in reality have an additional 1/2 hour fire resistance, when the nominal steel appropriately anchored over the slab and beam is taken into account.

It is because of the reservations in the above paragraph, that a complete structural analysis has particular appeal for a highly indeterminate structure.

Analytical procedure

In attempts to model more closely the behaviour of reinforced concrete elements, a number of research workers have developed computer programs capable of detailed analysis using finite elements ^(14, 15, 16) and finite strip methods ⁽¹⁷⁾ to simulate deflections, cracking and ultimate failure of reinforced beams, columns and portal frames. Although, in all cases approximations have had to be made in dealing with the mechanical and thermal properties of concrete, close correlation has been achieved with the behaviour of reinforced concrete beams tests in a standard furnace ^(18, 19, 20, 21, 22).

A number of programs are available, some of which have been developed by software houses and others by research establishments and are capable of performing thermal and/or structural analysis. Department of the UK Environment Reports provide a list of some of these programs and a summary paper ⁽²²⁾ published which includes 14 such programs developed in the USA and Europe. One of the U.K. software houses which markets a well-structured program Lusas, originating from Imperial College, has had a module developed dedicated to fire design ^(19, 20). Another program Sosmef ⁽²¹⁾ developed at the City University is a structural code capable of analysing structural steel/reinforced concrete elements using a finite difference formulation for compatibility of curvature with increase of temperature and deterioration of the material. Temperature distribution is derived from a Swedish based program and is at the front end of the program. Both these programs are written in Fortran and Ansis and can be used on a P.C. Most of the above programs do not take moisture into account, but a number of other programs have included this effect ^(23, 24, 25, 26, 27).

A technique novel to Civil Engineering but commonly used in Physical Science, discretises the domain by random lattice modelling using Voronoi Tessellation ⁽²⁸⁾. This has the advantage that irregular shaped elements can follow more closely the shape of aggregates and/or gel particles. Stress is then determined together with the possible fracture associated with heat and moisture movement within the concrete. Cracking is modelled using energy balance relations within the framework of non-linear fracture mechanics. The modelling of heat flow, moisture flow, and fracture are objective with respect to random geometry and size of the lattice elements. These routines, and their coupling, have already been implemented and verified for the problem of shrinkage cracking in cement-based composites, where the aggregate and sand particles are

accurately modelled^(29, 30). This method shows promise at predicting violent brittle failure, where no program has yet been able to predict.

Analysis of frameworks

The first program, known to the author, which was dedicated to fire design of a complete skeletal structure rather than to structural members is one developed at Imperial College^(5, 6, 7). The program assesses the performance of structural frameworks under different fire and environmental loading. Simple 2- & 3- dimensional grid analysis are carried out iteratively to a framework carrying design loads with one compartment subjected to a fire.

In analysing the framework, the program is designed to take into account the following factors: the reduction in compressive strength of concrete and the yield strength of steel and hence the change in neutral axis depth with increasing temperature. The high restraining bending and axial forces developing from the steep temperature gradients reduce due to creep at each level of temperature and the stress and strain resultants are examined at each discrete interval. To take into account the reduction of the modulus of elasticity of the concrete, the load deformation characteristics of the affected member is assumed to be linear in the elastic range and perfectly plastic in locations where plastic hinges form at failure. The moment capacity of each hinge in the fire zone is revised, so that when the next iteration proceeds any excess plastic moment of the failed nodes can be redistributed to adjacent joints.

Approximations used in the structural analysis include idealisations of material properties and use of the standard fire time/temperature curve, although any type of curve such as a sinusoidal shape to simulate a model fire can be inserted. It is also assumed that the temperature of the surfaces of the beams and columns of the compartment containing the fire increase at the same rate and are subject to a uniaxial thermal flow within the thickness of the members, resulting in parabolic temperature gradients.

Despite the crude assumptions, the analyses of a number of differently loaded frameworks together with their results have produced some interesting revelations. The order of formation of plastic hinges depends on the reinforcement details. In situations where reinforcing bars prevent the formation of hinges at the centre of the members, redistribution of excess moments may cause damage remote from the location of the fire. The deformation characteristics can, in certain circumstances, accentuate those imposed by loading whereas the combined stresses may be reduced. In other circumstances the resulting thermal stresses are additive whereas the deformations partly compensate each other.

Deformation patterns of various members subject to fires are shown in figure 2. It can be seen that the direction of deformation depends on the fixity and the relative stiffness of adjacent members. If the stiffness of the columns and beams are reversed, the deformations can be reversed for the same fire situation.

Remarks

Reinforced concrete has inherently a high fire resistance, but this is dependent on suitable design and careful detailing the reinforcement for a structure under fire, as detailing acceptable to normal temperature conditions may not be appropriate under fire.

Well-designed building construction with good detailing has a reserve of strength beyond that normally assumed. This applies to construction made with any material and the fire resistance of such buildings is normally higher than those assessed by standard building regulation rules. This was demonstrated by the result of a fire during construction of a completed structural steel framework, when most of the protection had not yet been installed⁽³¹⁾. The severe localised fire only affected the area in contact with the fire, but the remainder of the structure remained intact.

Testing on protected columns and joints but unprotected steel beams designed compositely with concrete floors in a full scale 5 x 4 bay x 8 storey structure performed extremely well in model fires⁽³²⁾. The model fire load was 40 kg/m³, which is 10% above that encountered in most office buildings. Although the beams had large deflections (557 mm) during the fire, the deflection recovered to 481 mm when the steel cooled. Localised buckling occurred but there were no signs of collapse despite the temperature of the fire reaching 763°C. In the prescriptive method a temperature of 550 would have predicted collapse.

Structural steelwork can be also designed using a fire engineering approach to justify economies. Composite circular steel columns in-filled with high strength concrete for a 30 storey building constructed in Hong Kong was found to require no external fire protection for both serviceability and ultimate states by assessing the column capacity derived from calculated column temperature profiles in fire⁽³³⁾.

Lifeboats with covered hulls, all made of plastic, are used as survival crafts for personnel in oil tankers and offshore oil drilling platforms in case of a fuel fire. The temperature of the hull is dependent on a water curtain to keep the accommodation bearable as the personnel manoeuvre the boat away from the blaze. The survival craft has to pass a fuel fire test in a boomed off area offshore over a 10-minute period. The author has tested some that have failed, because of a design fault and many others that have successfully passed the test, despite the fact that the temperature of the fire in contact with the boat reaches 1000°C within 10 seconds, and plastic melts at a much lower temperature.

With this in mind, a good designer can use his ingenuity to improve the performance of a structure, regardless of the material selected, not only in its service life but also under accidental loading. This would make the designer master of the material rather than be subservient to it.

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Conclusions

The tabular method of design is simple and effective in cases where the construction is essentially designed on a simply supported basis. However in most construction a certain amount of fixity exists in the structural members. The ultimate limit method of fire design, which has been codified in a number of countries for normal temperatures, offers the prospect of greater realism, flexibility and economy in design, but requires a clear appreciation of the approximations adopted and a complete understanding of the reinforcing details. The analysis of complete structures requires the use of a large capacity fast computer and can be expensive, but the result give a better understanding of the overall behaviour of structures subjected to accidental loading.

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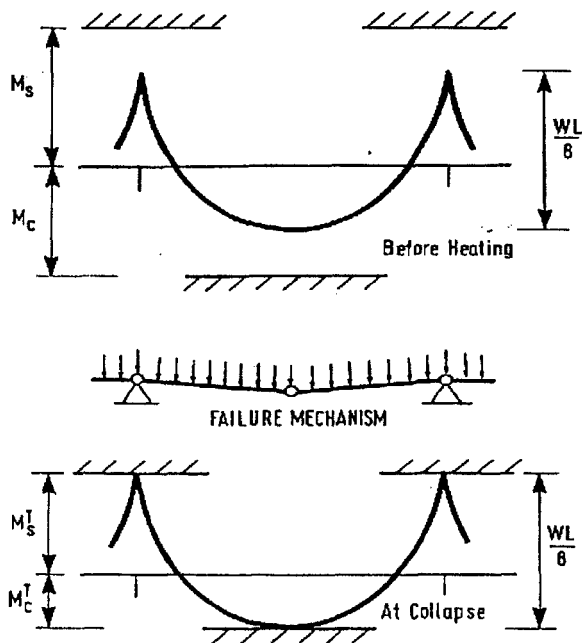


Figure 1: Redistribution of Bending Moments at Supports of a Continuous Beam

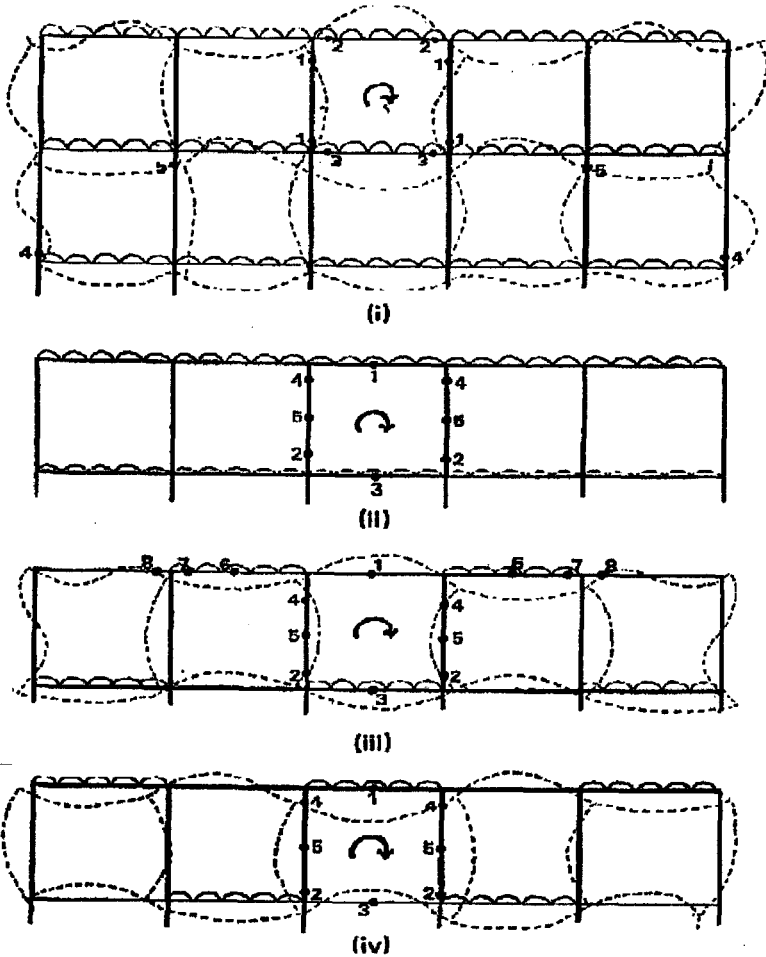


Figure 2: Hinge Formation and Deformation Due to UDL and Compartment Fire:

- (i) members falling near supports
- (ii) members falling at center
- (iii) alternate bays loaded including fire compartment
- (iv) alternate bays loaded over fire compartment

Performance of Geopolymer Concrete Under Sulfate Exposure

by S. E. Wallah, D. Hardjito, D. M. J. Sumajouw, and B. V. Rangan

Synopsis: As a relatively ‘new’ material, geopolymer concrete offers the benefits as a construction material for sustainable development. It utilises waste materials such as fly ash and has a very low rate of green house gas emission. This paper presents the study of the performance of fly ash based geopolymer concrete to sulfate attack. Test specimens were soaked in sodium sulfate solution and sulfuric acid solution for various periods of time, and the performance of geopolymer concrete is studied by evaluating the effect on the compressive strength, change in length and change in mass. Test results show that exposure to sodium sulfate has no significant effect on geopolymer concrete, whereas exposure to sulfuric acid affects the compressive strength.

Keywords: fly ash; geopolymer concrete; sodium sulfate; sulfuric acid

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INTRODUCTION

The development of environmentally friendly concrete for sustainable development is crucial for continued use of this material. In this respect, geopolymer concrete is emerging as an alternative to Ordinary Portland Cement (OPC) concrete as a construction material for sustainable development. Geopolymer concrete that utilises waste material containing silica (*Si*) and alumina (*Al*) such as fly ash is a good alternative because it has a very low rate of greenhouse gas emissions compared to OPC concrete. The term 'geopolymer' was first introduced by Davidovits (1987) to describe a family of mineral binders based on silico-aluminates. This inorganic polymer has a wide range of applications in civil engineering [1, 2]. In geopolymer concrete, the silica and the alumina present in the source materials are first induced by alkaline activators to form a gel. This geopolymer gel binds the loose aggregates and other unreacted materials in the mixture to form the geopolymer concrete. The chemical process involved in the formation of geopolymer binders is very different to that of OPC concrete.

Several laboratory studies have been reported to identify the parameters that influence the properties of geopolymers. Davidovits [1, 2] found that after a 4-hour curing period at 20°C, geopolymer mortar attains a compressive strength of 20 MPa and the final 28-day compressive strength is in the range of 70 to 100 MPa. Curing period and type of activators were found to be significant factors affecting the mechanical strength of fly ash based geopolymers [3, 4, 5]. Longer curing time and higher curing temperatures usually resulted in higher compressive strength.

In the previous studies, the authors [6] reported the results of the research into engineering properties of geopolymer concrete. The concrete mixtures were made using low-calcium class F fly ash, sodium hydroxide (NaOH) and sodium silicate (Na₂SiO₃) solutions as alkaline activators, and locally available aggregates. It was found that for curing temperatures up to 60°C, there was a significant increase in compressive strength, while the increase in strength was not significant when the samples were cured above 60°C. Also, curing for 24 hours increased the compressive strength substantially compared to a 4-hour curing. In addition, it was also shown that as the Na₂SiO₃-to-NaOH ratio increased the compressive strength increased significantly. Test results obtained

from specimens cured at 60°C for 24 hours revealed that the compressive strength of geopolymer concrete did not vary with the age of concrete because of the fast polymerisation process of the geopolymer gel [7]. The recent tests by the authors also identified the significant effect of water content in the mixture, expressed as water to geopolymer solids ratio, on the compressive strength of geopolymer concrete [8]. The fresh concrete could be handled for about 120 minutes without any sign of setting and without any effect on the compressive strength [9]. The test results also show that geopolymer concrete undergoes low creep [10]. This paper presents the study of the sulfate resistance of geopolymer concrete.

EXPERIMENTAL PROGRAM

Materials and Mixture Proportions

Geopolymer concrete in this study utilised the low calcium (class F) fly ash from Collie Power Station, Western Australia as the source material. Table 1 shows the chemical composition of the fly ash as determined by X-Ray Fluorescence (XRF) analysis. It can be seen from Table 1, silicon and aluminium oxides constitute about 80% of the fly ash and the Si-to-Al ratio is about 2. From particle size analysis, as presented in Fig. 1, it was found that the specific surface area of the fly ash was 1.29 m²/cc. Graph A shows the percentage of the volume passing and Graph B shows the percentage volume for certain sizes. Local aggregates, comprising 20 mm, 14 mm and 7 mm coarse aggregates and fine sand, in saturated surface dry conditions, were used in the experimental work. The fineness modulus of combined aggregates was 5.0. The alkaline activators were analytical grade sodium hydroxide (NaOH) in flake form with 98% purity and sodium silicate (with SiO₂/Na₂O ratio of 2.00 and SiO₂=29.4% by mass). In order to improve the workability, high range water reducer superplasticiser of 1.5% by mass of the fly ash was added to the mixture.

The mixture proportion of geopolymer concrete selected for this study was one of the mixtures used in authors' previous research [6, 7, 9]. The details are given in Table 2.

Specimen Preparation

The sodium hydroxide flakes were dissolved in distilled water to make a solution with a concentration of 8 M at least one day prior to use. The fly ash and the aggregates were first mixed together in a pan mixer for about 3 minutes. The sodium hydroxide and the sodium silicate solutions were mixed together with superplasticiser and then added to the dry materials and mixed for about 4 minutes. Immediately after mixing, the fresh concrete was cast into 100 x 200 mm cylindrical molds in three layers and 75x75x285 mm prismatic molds in two layers. Each layer was given 60 manual strokes using a rodding bar and vibrated on a vibrating table for 10 seconds. About 30 minutes after casting, the specimens were placed in the oven for curing at 60°C for chosen periods. After curing, the specimens were left to air-dry in the laboratory.

Test Variables and Test Procedures

The specimens were soaked in sodium sulfate solution and sulfuric acid solution for intended periods of time and the changes in compressive strength, length and mass were measured.

The concentration of the sodium sulfate (Na_2SO_4) in the solution was 5 percent by mass and that of the sulfuric acid (H_2SO_4) was 2 percent by mass [11, 12, 13]. The pH value of the sodium sulfate solution was between 6 and 8 and that of sulfuric acid solution was about 2. In order to maintain these concentrations the solutions were replaced every month. The test specimens were immersed in the solutions one week after casting, and the solutions were made one day before the test specimens were soaked.

Test variables selected for this study are grouped into two series. In the first series, the changes in compressive strength and length of the specimens soaked in sodium sulfate solution were observed. Specimens were cured in the oven at 60°C for the periods of 24 hours and 48 hours. For the length change tests, three specimens were made for each variable and the changes in length were measured using a horizontal length comparator one hour after removing the specimens from the immersion tank. Four specimens were made for each variable for compressive strength tests. The tests were performed one week after removing the test specimens from sodium sulfate solution and tested in compression in accordance with the relevant Australian Standard for testing concrete.

In the second series, the changes in compressive strength and mass of the specimens soaked in sodium sulfate solution and sulfuric acid solution were observed. After each exposure period, the specimens were tested immediately after removing them from the solution. For change of mass tests, four specimens were made for each test variable and the mass was measured using a laboratory scale. Four specimens were made for each variable for compressive strength tests and the tests were performed in accordance with the relevant Australian Standard for testing concrete. For comparison, an additional set of specimens were soaked in tap water.

The test variables are summarized in Table 3. The results plotted in various Figures are the average of the values of the specimens in each category.

TEST RESULTS**Visual Appearance**

There was no significant change in the external appearance of the surface of specimens soaked in sodium sulfate up to 12 weeks. The same was true for the specimens soaked in tap water. However, the surfaces of specimens soaked in sulfuric acid solution started to erode even after one week of exposure.

Compressive Strength

Figures 2 and 3 show the variation in the compressive strength of specimens tested one week after removing the specimens from sodium sulfate solution after various weeks of exposure. For both curing periods (24 hours and 48 hours), the variation in compressive strength is not significant compared to the result obtained for the companion specimens left in the laboratory ambient conditions and tested one week after casting. Figures 4, 5 and 6 show the compressive strengths of specimens soaked in sodium sulfate solution, sulfuric acid solution and tap water for exposure periods of 4, 8 and 12 weeks respectively as compared to the compressive strengths of companion specimens left in the laboratory ambient conditions and tested one week after casting the specimens. It can be seen from these data that exposure to sodium sulfate solution or tap water up to 12 weeks has very little effect on the compressive strength. However, a significant change in compressive strength is observed in the case of specimens exposed to sulfuric acid solution. The compressive strength of these specimens decreased substantially as the period of soaking increased. For the 12 weeks soaking period, the reduction of compressive strength was about 30 %. It appears that the penetration of sulfuric acid may have affected the microstructure and decreased the bond between geopolymer paste and the aggregates, thus resulting in a decrease in compressive strength.

Change in Length

Figure 7 shows that the change in length of specimens soaked in sodium sulfate solution for various periods of exposure is very small indeed and less than 0.01%.

Change in Mass

The average unit weight of concrete in ambient conditions was 2356 kg/m^3 . This value did not change for specimens soaked in sodium sulfate solution. In the case of specimens soaked in sulfuric acid, the mass decreased less than one percent after 12 weeks.

CONCLUDING REMARKS

The performance of geopolymer concrete under sulfate exposure has been studied by soaking the specimens in sodium sulfate solution and sulfuric acid solution. After 12 weeks of exposure, by observing the change in compressive strength, change in mass and change in length of the specimens, the results showed that in form of sodium sulfate, sulfate attack did not have significant effect on geopolymer concrete. On the other hand, the sulfate attack in the form of sulfuric acid damaged the surface of the specimens and reduced the compressive strength of geopolymer concrete. Tests are continuing for at least one year in order to substantiate the trends observed so far.

ACKNOWLEDGEMENTS

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TABLE 1 Chemical composition of fly ash (% by mass)

SiO ₂	53.36
Al ₂ O ₃	26.49
Fe ₂ O ₃	10.86
CaO	1.34
Na ₂ O	0.37
K ₂ O	0.8
TiO ₂	1.47
MgO	0.77
P ₂ O ₅	1.43
SO ₃	1.70
LOI*	1.39

* loss on ignition

TABLE 2 Mixture proportions for geopolymers concrete

Materials	Coarse aggregates	Fine aggregates	Fly ash	Sodium hydroxide solution (8M)	Sodium silicate solution	Super plasticizer
Mass kg/m ³	1294	554	408	41	103	6

TABLE 3 Test variables and other details

Parameters	Specimens	Curing period	Exposure conditions
Series-1			
Compressive strength	Cylinders 100/200 mm	24 and 48 hours	Sodium sulfate solution
Change in length	Prisms 75/75/285 mm	24 and 48 hours	
Series-2			
Compressive strength, and change in mass	Cylinders 100/200 mm	24 hours	Sodium sulfate solution, sulfuric acid solution, and tap water

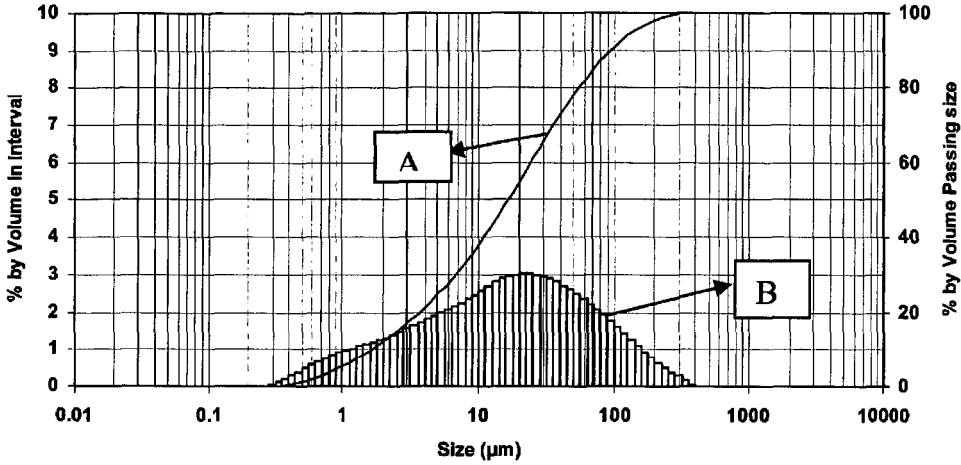


Fig. 1 Particle size distribution of fly ash

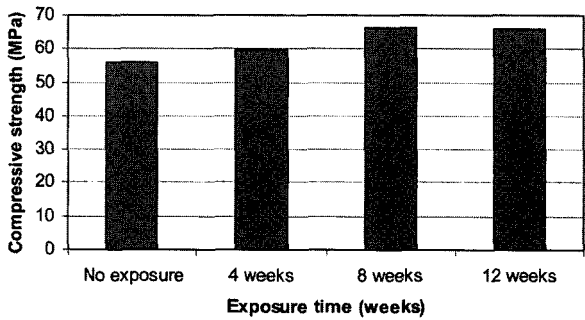


Fig. 2 Compressive strength after sodium sulfate exposure for specimens cured for 24 hours

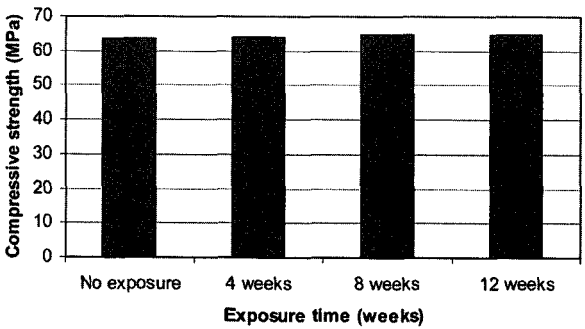


Fig. 3 Compressive strength after sodium sulfate exposure for specimens cured for 48 hours

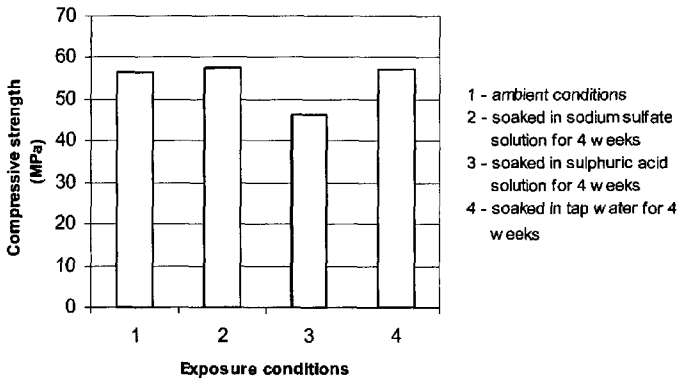


Fig. 4 Compressive strength after 4-weeks of exposure

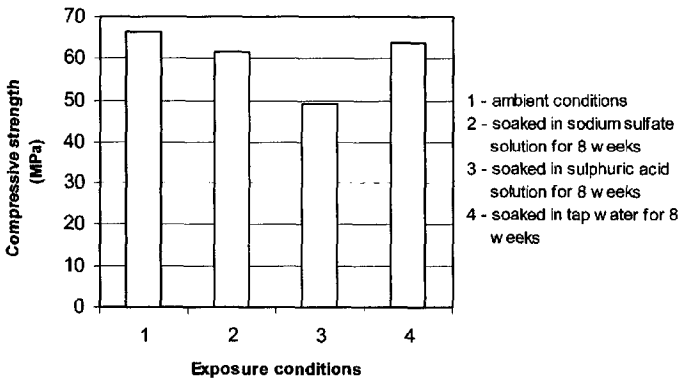


Fig. 5 Compressive strength after 8-weeks of exposure

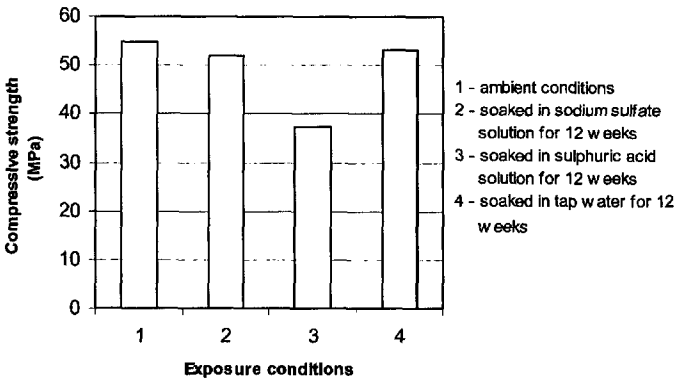


Fig. 6 Compressive strength after 12-weeks of exposure

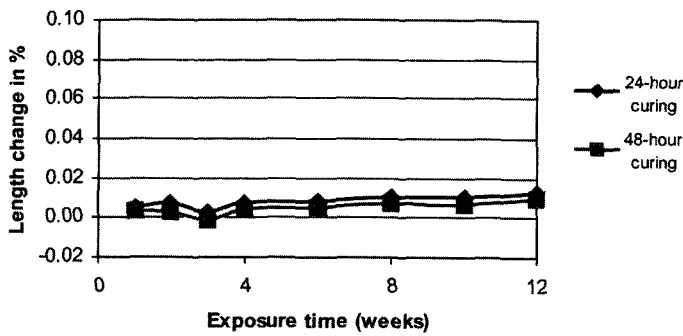


Fig. 7 Length change after sodium sulfate exposure

Bacterial Concrete— A Concrete for the Future

by V. Ramakrishnan, R. K. Panchalan, and S. S. Bang

Synopsis: This paper presents the results of a novel technique in remediating cracks and fissures in concrete by utilizing microbiologically induced calcite (CaCO_3). *Bacillus Pasteruii*, a common soil bacterium was used to induce calcite precipitation. This technique is highly desirable because the mineral precipitation induced as a result of microbial activities, is pollution free and natural. The effectiveness of this technique was evaluated by comparing the compressive strength and stiffness of cracked specimens remediated with bacteria and those of the control specimens (without bacteria). Experimental investigation was also conducted to determine the strength regaining capacity (modulus of rupture) of cracked beams remediated with different concentrations of bacteria. This paper also presents the results of a durability study on cement mortar beams treated with bacteria, exposed to alkaline, sulfate and freeze-thaw environments. Different concentrations of bacteria were used for the investigation. It was found that the use of bacteria improved the stiffness, compressive strength, modulus of rupture and durability of concrete. Scanning electron microscope (SEM) was used to document the role of microbiologically induced mineral precipitation in improving the strength and durability aspects of concrete. Energy Dispersive X-ray (EDX) spectrometer analysis of the precipitated crystals indicated abundance of calcium and the precipitation was confirmed to be calcite.

Keywords: *Bacillus Pasteruii*; bacterial concrete; calcite crystals; crack remediation; durability; EDX spectrometer analysis; SEM; strength; strength regaining capacity

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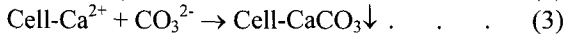
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INTRODUCTION

Natural processes, such as weathering, faults, land subsidence, earthquakes, and human activities create fractures and fissures in concrete structures and historical stone monuments. These fractures and fissures are detrimental since they can reduce the service life of the structure. In the case of monuments and buildings of historic importance, these cracks tend to disfigure and destroy the structure. Use of synthetic agents such as epoxies for remediating these structures will reduce the aesthetic appearance of the structure. Therefore a novel technique for remediating damaged structural formations has been developed by employing a selective microbial plugging process, in which microbial metabolic activities promote calcium carbonate (calcite) precipitation [1 to 4]. This technique is called microbiologically enhanced crack remediation (MECR). This technique comes under a broader category of science called biomineralization. It is a process by which living organisms form inorganic solids. Living organisms including bacteria have the ability to form bones and teeth, to natural pearls and shells. As a microbial sealant, CaCO_3 exhibited its positive potential to selectively consolidate simulated fractures and surface fissures in granites [2]. Microbial calcite precipitation was quantified by X-ray diffraction (XRD) analysis and visualized by SEM [5, 6].

In natural environments, chemical CaCO_3 precipitation ($\text{Ca}^{2+} + \text{CO}_3^{2-} \rightarrow \text{CaCO}_3 \downarrow$) is accompanied by biological processes, both of which often occur simultaneously or sequentially. An endospore forming, soil microorganism, *Bacillus pasteurii*, is known to have an ability to precipitate calcite in the environment. This microbiologically induced calcium carbonate precipitation (MICCP) comprises of a series of complex biochemical reactions. As part of metabolism, *B. pasteurii* produces urease, which catalyzes urea to produce CO_2 and ammonia, resulting in an increase of pH in the surroundings where ions Ca^{2+} and CO_3^{2-} precipitate as CaCO_3 . Possible biochemical reactions in medium to precipitate CaCO_3 at the cell surface that provides a nucleation site can be summarized as follows.



Earlier it was reported that sand consolidation by *B. pasteurii* reduced porosity by up to 50% and permeability by up to 90% in the areas where the cementation took place [1]. Microbial calcite plugging was selective and its efficiency was affected by the porosity of the medium, the number of cells present and the total volume of nutrient added. The sand column loaded with bacteria was so tightly plugged that the column was fractured with a mechanical knife for examining. In a study conducted by Zhong and Islam [4], an average crack width of 2.7 mm and a mixture of silica fume (10%) and sand (90%) showed the highest compressive strength in the microbial remediation of granite.

Concrete crack remediation by microorganisms was significantly different from that of granite remediation, mainly due to the fact that concrete maintained high levels of pH. An extreme alkaline environment of pH around 12 is the major hindering factor for growth of *B. pasteurii*, whose optimum pH for growth is around 9. However, *B. pasteurii* has the ability to produce endospores to endure an extreme environment.

EXPERIMENTAL INVESTIGATIONS

Strength Characteristics

The effectiveness of microbiologically induced calcite (CaCO_3) precipitation in remediating cracks in concrete was evaluated by comparing the compressive strength and stiffness of cracked specimens treated with bacteria with those of the control specimens (without bacteria). Experimental investigation was also conducted to determine the strength regaining capacity (modulus of rupture) of cracked beams remediated with different concentrations of bacteria. Most of the earlier research work by Ramakrishnan et al. [5 to 7] concentrated on studying the strength aspects. Brief summaries of these investigations are presented below.

The effect of microbial calcite precipitation to various depths of cracks on the stiffness of cement mortar beams.

Cement mortar beams of dimensions $152 \times 25.4 \times 25.4$ mm ($6 \times 1 \times 1$ in.) were prepared according to ASTM C305-94. The specimens were cured in water for 28 days and then they were kept exposed to air for another 3 months. Artificial cracks were cut on the completely cured cement mortar beams using a Hillquist saw. The width of the cut was maintained constant at 3.2 mm (0.125 in.), while the depths of cut were 3.2 mm (0.125 in.) and 9.5 mm (0.375 in.). For each crack depth, a total of 10 specimens were prepared. The first 5 specimens were used as control without any filling in the cracks and were left exposed to air. The cracks in the remaining 5 specimens were filled with a mixture of sand and *B. pasteurii*. Sand used for the filling passed through US standard sieve No. 16 and retained on US standard sieve No. 40. The sand mixed with bacteria suspension to a final concentration of 6.2×10^{10} cells per ml was forced into the crack

with a knife-edge. Then the beams with bacteria in their cracks were placed in a tray containing Urea- CaCl_2 medium and cured for 28 days. The medium was replaced after 14 days and extreme care was taken not to disturb the precipitation of the calcium carbonate on the specimens during the renewal of the medium.

The control beams and the beams with bacteria were tested for their stiffness after 28 days using one point loading with a span of 127 mm (5 in.). Load-deflection values were recorded using a Data Acquisition System. The load values were recorded to an accuracy of 0.45 kg. (1 lb.) and the deflection values were recorded to an accuracy of 0.025 mm (0.001 in.). The data was then analyzed and the load-deflection graphs were drawn. The slope of the load-deflection curve was calculated which was taken as the stiffness of the cement mortar beam. Figure 1 shows the stiffness values for both control and treated specimens. The stiffness values of beams whose cracks were filled with bacteria and sand were found to be higher than those of the control specimens. This was also true for beams with both crack depths. As expected, the beams with a deeper cut showed a lower stiffness value when compared to those with shallower cut. The results showed a significant increase in the stiffness of beams with 3.2 mm (0.125 in.) depth of cut. The microbial precipitation due to the bacteria present within the crack showed more effective remediation in shallower cracks than in deeper ones, increasing the stiffness of beams by 24% and 14% with 3.2 mm (0.125 in.) and 9.5 mm (0.375 in.) cuts respectively.

The effect of microbial calcite precipitation to various depths of cracks on the compressive strength of cement mortar cubes.

Cement mortar cubes of dimensions $50.8 \times 50.8 \times 50.8$ mm ($2 \times 2 \times 2$ in.) were prepared according to ASTM C305-94. The specimens were cured in water for 28 days and then they were kept exposed to air for another 3 months. Artificial cracks were cut on the completely cured cement mortar cubes. The width of the cut was maintained constant at 3.2 mm (0.125 in.), while the depth of cuts were 12.7 mm (0.5 in.), 19.05 mm (0.75 in.) and 25.4 mm (1 in.). For each crack depth, 10 specimens were prepared. The first five were used as control and the cracks in the remaining 5 were filled with bacteria and sand.

The specimens with bacteria in their cracks were cured in Urea- CaCl_2 medium, for 28 days. The medium was replaced after 14 days. The specimens were tested for their compressive strength using a Tinius Olsen compression-testing machine. Figure 2 shows the compressive strength comparison of control and bacterial specimens. The results showed an increase in the compressive strength of the specimens with bacteria and sand in their cracks, over the control specimens, with no filler material in their cracks. As expected, the compressive strengths of the cubes with deeper cuts were less than those of cubes with shallower cuts. The increase in the compressive strength of specimens with bacteria and sand in their cracks was evident for all the cubes with different depths of cracks. The increase in the compressive strength was most significant in the cubes with the deepest crack 25.4 mm (1 in.). The microbial remediation increased the compressive strength by approximately 80% when compared to that of control specimens.

The effect of different concentrations of bacterial cells used for crack remediation, on the compressive strength of cement mortar cubes.

Cement mortar cubes of dimensions $50.8 \times 50.8 \times 50.8$ mm ($2 \times 2 \times 2$ in.) were prepared according to ASTM C305-94. The specimens were cured in water for 28 days and then they were kept exposed to air for another 3 months. Artificial cracks were made on the completely cured cement mortar cubes using a Hillquist saw with a special arrangement to simulate a crack. The width and depth of cut were maintained constant at 3.2 mm (0.125 in.) and 25.4 mm (1 in.) respectively. All together 18 cement mortar cube specimens were made for this experiment. Microbial plugging of concrete cracks was examined with three different concentrations of *B. pasteurii* mixed with sand. Concentrations of cells were 4.3×10^8 , 8.6×10^8 , 4.3×10^9 cells per ml. The cubes were tested for their compressive strength after 28 days. Figure 3 shows the comparison of compressive strengths for different concentrations of bacteria. Specimens treated with bacteria showed higher compressive strengths than that of the control specimens. Among the tested microbial concentrations, a concentration of 8.6×10^8 cells per ml was found to induce the maximum compressive strength. This was taken as the optimum concentration. Specimens with higher concentration did not give higher compressive strength values probably because the greater population of bacteria did not have enough nutrients to multiply.

The effect of *B. pasteurii* with various concentrations, on the modulus of rupture of the cracked cement mortar beams.

In this study cement mortar beams of dimension $152.4 \times 25.4 \times 25.4$ mm ($6 \times 1 \times 1$ in.) were made. The specimens were cured in a moist cabinet for 14 days and then they were air cured for 7 days. Then cracks were cut in the specimen on the tension side. The depths of the cracks were 12.7 mm (0.5 in.). Care was taken while breaking the specimens so that the whole specimen does not break. The cracking of the specimens was done according to ASTM C 1399. The sample beam was turned on its side with respect to its position as molded and placed on top of the steel plate to be loaded with the specimen. The plate and the beam were placed on the support apparatus so that the steel plate was centered on the lower bearing blocks and the cement mortar beam was centered on the steel plate. Then the specimen and steel plate combination were loaded until the specimen cracked (only on tension side). The cracking load was recorded in the data acquisition system. The cracked beams after remediation with bacteria were tested to determine the regain in the modulus of rupture values.

A total of 15 cracked beams were made out of which 12 beams were treated with bacteria and the remaining 3 beams were not treated with bacteria (only Urea- CaCl_2 medium). Out of the 12 beams, 3 beams were treated with bacteria with a concentration of 10^7 cells/ml, 3 beams were treated with bacteria with a concentration of 10^8 cells/ml, 3 beams were treated with 10^9 cells/ml and 3 were treated with the optimum concentration of 8.6×10^8 cells/ml. Urea- CaCl_2 medium was added in each of the concentrations which served as the nutrient for the bacteria to grow in order to get the calcium carbonate precipitation. The beams were tested after 14 days of Urea- CaCl_2 curing according to ASTM C 293 (Standard test method for flexural strength of concrete using simple beam with center point loading). The beams were loaded centrally till it broke and the data obtained with the data acquisition system were analyzed to find the modulus of rupture of

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the specimens. The average values of modulus of rupture were calculated for each concentration and they were compared to those of the beams without pre-cracks (control). The results are shown in Figure 4.

The results are summarized as follows:

- i. Cement mortar beams (with cracks) treated with a bacterial concentration of 10^9 cells/ml regained their strength (modulus of rupture) by 58% when compared to that of beams without cracks. Whereas cement mortar beams with a bacterial concentration of 8.6×10^8 cells/ml regained their strength by 82%.
- ii. Cement mortar beams (with cracks) treated with a bacterial concentration of 10^8 cells/ml regained their strength (modulus of rupture) by 46% when compared to that of beams without cracks. Whereas cement mortar beams (with cracks) treated with a bacterial concentration of 10^7 cells/ml regained their strength (modulus of rupture) by 12% when compared to that of beams without cracks.
- iii. The strength regaining capacity of cracked cement mortar beams cured only in urea- CaCl_2 medium (without bacteria) when compared to that of beams without cracks was only 3%. This proves that bacteria play a predominant role in remediating cracked beams.

Durability Characteristics

Since earlier research work was mostly concentrated in studying the strength aspects, there was a need to determine the durability aspects of cement mortar with bacteria. Therefore recently investigations were carried out by Ramakrishnan et al [8, 9] to study the durability aspects of cement mortar beams made with different concentration of bacteria. The main objective was to determine whether the beams with bacteria performed better, when subjected to alkaline, sulfate and freeze-thaw attack.

The effect of different concentrations of bacteria (1×10^6 cells/ml, 1×10^7 cells/ml, 8.6×10^8 cells/ml and 1×10^9 cells/ml) on the alkali aggregate reactivity of cement mortar beams.

In this study cement mortar beams of dimension $285.75 \times 25.4 \times 25.4$ mm ($11.25 \times 1 \times 1$ in.) were made according to ASTM C305-94 procedure. A total of 15 beams were made. Three beams were made without bacteria (control) and three beams were made for each concentration of bacteria. The moulds were greased and the gauge studs were fixed such that 15.8 mm (0.625 in.) of the stud protruded into the beam. Care was taken while oiling the mould to make sure that the head of the stud was free from oil. These beams were placed in the moist cabinet for 24 ± 2 hrs. After demolding they were placed in Urea- CaCl_2 solution and cured for 7 days.

The test was done as per the requirements of ASTM C 1260 (Standard test method for potential alkali aggregate reactivity of aggregates - mortar bar method). The specimens were transferred into a plastic container containing tap water and were immersed completely. They were sealed and placed in an oven at $80 \pm 2.0^\circ\text{C}$ ($176 \pm 3.6^\circ\text{F}$) for 24 hrs. They were later removed one at a time and the zero reading was

taken. Then the specimens were placed in a container containing 1N NaOH and were placed in the oven. The process of drying the specimens and taking the reading was done in 15 ± 5 sec after removing the specimens from the container. After measurements were taken, the beams were left on a towel (for drying), until the length comparator readings were taken for all the beams. Readings were taken at every 3, 5, 7, 11 and 14 days. The comparison of the mean expansion readings for all the beams is shown in Figure 5. It was observed that the mean expansion of beams with bacteria was less than that of the beams without bacteria.

The results are summarized as follows:

- i. Beams with bacterial concentration of 1×10^6 cells/ml had 11% less mean expansion when compared to that of the control beams.
- ii. Beams with bacterial concentration of 1×10^7 cells/ml had 18% less mean expansion when compared to that of the control beams.
- iii. Beams with bacterial concentration of 8.6×10^8 cells/ml had 27% less mean expansion when compared to that of the control beams.
- iv. Beams with bacterial concentration of 1×10^9 cells/ml had 32% less mean expansion when compared to that of the control beams.
- v. The reduction in the mean expansion is due to the formation of calcite on the surface and interior of the specimen due to the metabolic activities of the bacteria.

The effect of different concentrations of bacteria (1×10^7 cells/ml, 8.6×10^8 cells/ml and 1×10^9 cells/ml) on the sulfate attack resistance of cement mortar beams.

In this study cement mortar beams of dimension $285.75 \times 25.4 \times 25.4$ mm ($11.25 \times 1 \times 1$ in.) were made according to ASTM C305-94 procedure. A total of 12 beams were made. Three beams were made without bacteria (control) and three beams were made for each concentration of bacteria. The specimens were cured in Urea - CaCl_2 medium for 7 days. The test was done as per ASTM 1012 (Standard test method for length change of hydraulic cement mortars exposed to a sulfate solution). Zero reading (initial reading) was taken before placing the specimens in 0.35 M sodium sulfate solution. Additional readings were taken using a length comparator at every 7, 14, 21, 28 and 56 days after placing them in sodium sulfate solution. The comparison of mean expansion readings for all the specimens is shown in Figure 6.

The results are summarized as follows:

- i. Beams with bacterial concentration of 1×10^6 cells/ml had 12% less mean expansion when compared to that of the control beams.
- ii. Beams with bacterial concentration of 8.6×10^8 cells/ml had 38% less mean expansion when compared to that of the control beams.
- iii. Beams with bacterial concentration of 1×10^9 cells/ml had 39% less mean expansion when compared to that of the control beams.

The reduction in mean expansion can be attributed to microbial metabolic activities, which in turn produced calcite precipitates in the paste matrix of the specimen making