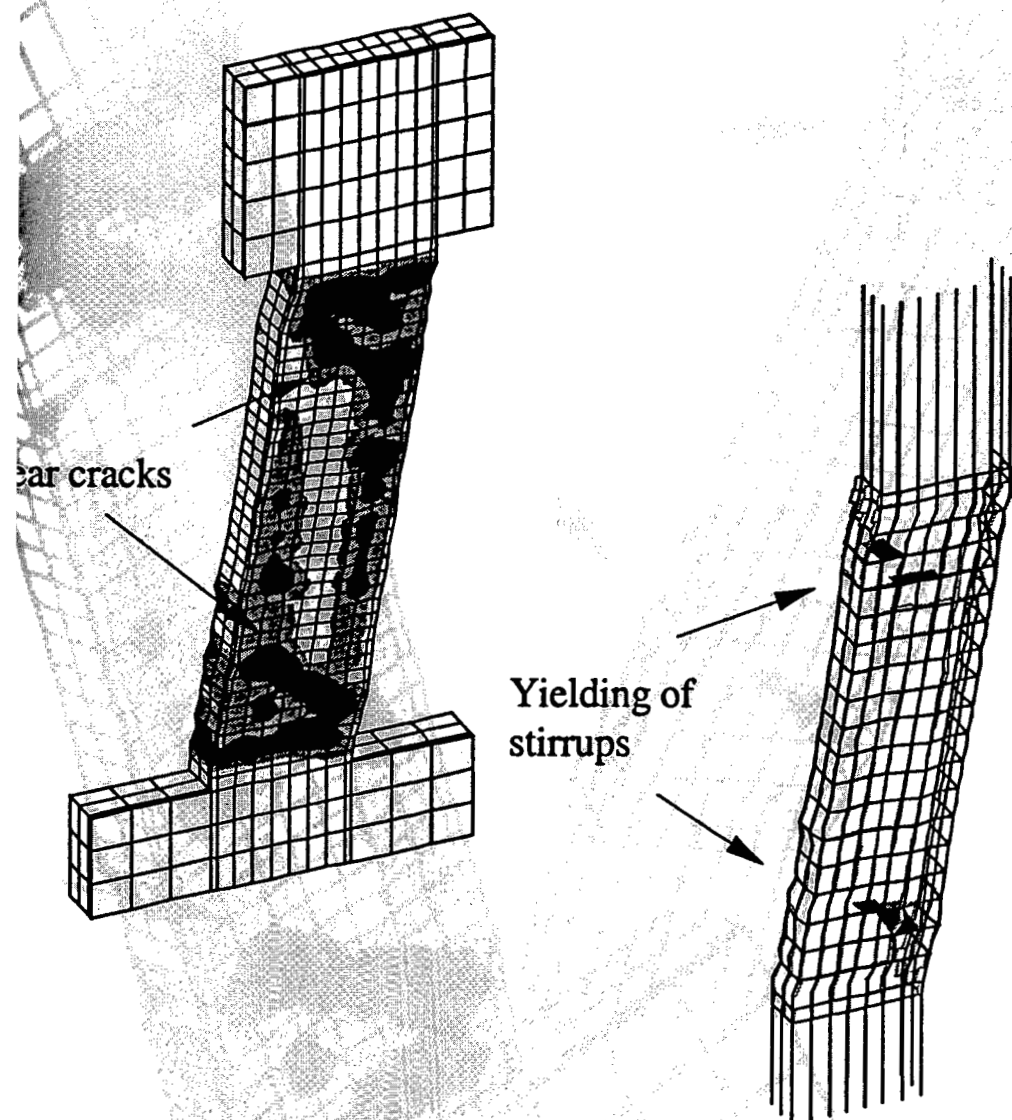


Finite Element Analysis of Reinforced Concrete Structures



Finite Element Analysis of Reinforced Concrete Structures

Editors

Kaspar Willam

Tada-aki Tanabe



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Printed in the United States of America

Editorial production: Bonnie L. Gold

Library of Congress catalog card number: 2001098847

ISBN: 0-87031-064-X

PREFACE AND DEDICATION

This Special Volume of the American Concrete Institute is dedicated to Alexander C. Scordelis, Professor Emeritus of the University of California at Berkeley, prominent structural engineer and educator. The 18 papers are written to honor Alexander Scordelis for initiating, developing, and leading the field of finite element analysis of reinforced concrete structures, a research field that has attracted eminent research scientists and engineers from all over the world.

It all started in 1967 with the publication of a paper in the *Journal of the American Concrete Institute* entitled "Finite Element Analysis of Reinforced Concrete Beams." At that time, the finite element method had reached a certain degree of maturity at the University of California at Berkeley. Given Scordelis's interest in the analysis and design of concrete structures, it was only natural that he would apply this new analytical tool to model the nonlinear crack and bond behavior of a reinforced concrete beam. Little did he know that this paper would become a classic and establish a new field of scientific inquiry.

In May 1977, Scordelis took the initiative to form the joint ACI-ASCE Task Committee on Nonlinear Finite Element Analysis of Reinforced Concrete Structures, FEARCS, in the form of a letter addressed to persons known to be active in this field. The formation of this committee was approved in August 1977, with Arthur Nilson as its first chairman. In 1982, this task committee published its first state-of-the-art report as a special publication of ASCE, which became by far the most frequently quoted reference on the subject. In 1985, the standing ACI-ASCE Committee 447 joined the JCI Research Committee on Finite Element Analysis of Reinforced Concrete Structures of the Japan Concrete Institute in Tokyo to organize a joint seminar, under chairmen Christian Meyer and Hajime Okamura. The proceedings contain 40 papers, which were published in 1986 in the form of a special publication by ASCE. The same committees held a second joint seminar on the campus of Columbia University in New York City in 1991. The proceedings of this meeting were arranged in nine topical sections that update the original state-of-the-art report of 1982. They were edited and published in 1993 by chairman Dr. Jeremy Isenberg under the heading "Finite Element Analysis of Reinforced Concrete Structures II."

The present volume of ACI features 18 papers that were presented during the ACI Fall 2000 Convention in Toronto. The three-session mini-symposium was sponsored jointly by the ACI-ASCE Committee 447, chaired by Kaspar Willam, and the JCI Committee on Post-Peak Behavior of Reinforced Concrete Structures Subjected to Seismic Loads, under chair Tada-aki Tanabe. It is the post-peak behavior of concrete and bond that has challenged the community of finite element analysts for a long time. Thereby, the prediction of ductility demands in earthquake engineering constitutes a major obstacle within the research field, which Professor Scordelis was so instrumental in creating 23 years earlier.

By dedicating this volume to Alexander Scordelis, the authors, former students, colleagues, and friends wish to applaud a long and productive career and wish him many more happy years.

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ALEXANDER C. SCORDELIS

BIOGRAPHICAL SUMMARY



Alexander C. Scordelis is the Bryon L. and Elvira E. Nishkian Professor Emeritus of Structural Engineering at the University of California at Berkeley, where he has been a member of the faculty since 1949. He has taught a large number of undergraduate and graduate courses in analysis and design of structures.

He has been actively engaged in research in a variety of fields throughout his career, including analytical and experimental investigations of reinforced and prestressed concrete beams, slabs, folded plates, and thin shell and bridge structures. The major

research contributions of Professor Scordelis have been on three principal topics: concrete thin shells, box girder bridges, and the nonlinear finite element analysis of reinforced and prestressed systems.

For concrete thin shells during the 1950s and the 1960s, he developed linear methods of analysis that could be utilized effectively with digital computers to analyze shell structures of all shapes and types. In recent years he has extended this work, using nonlinear finite element analysis to simulate the structural response of arbitrary reinforced concrete shell structures through their elastic, cracking, inelastic, and ultimate loads ranges, taking into account nonlinear materials, geometry, and the time-dependent effects of creep and shrinkage.

For many years he has conducted a continuing research program on concrete box girder bridges to study successively straight, simple, and continuous bridges, curved bridges, and skew bridges. Results of this research have been used widely in the United States and abroad. This work was then, and is now, being extended to prestressed concrete segmental box girder bridges and cable-stayed bridges. In these studies, the effects of prestressing, sequence of construction, and time-dependent effects are included in the analytical procedure.

He also has conducted a continuing research program on the finite element analysis of reinforced and prestressed concrete structures. His original papers on this subject in 1967 were the first to open up this new area of research, which is now being studied by researchers throughout the world. Results of research in

this field of study have been used in the analysis and design of bridges, buildings, nuclear containment structures, and offshore platforms.

Professor Scordelis has always had interest in applying the results of his research to practical engineering problems. During the past 40 years, he has been a consultant to various structural engineering firms and governmental agencies on more than forty major projects involving shell structures, bridge structures, reinforced and prestressed concrete structures, and computer solutions for complex structural systems.

Professor Scordelis is an Honorary Member of the American Society of Civil Engineers and a Fellow of the American Concrete Institute. He is an Honorary Member of the Association for Shell and Spatial Structures. He is also a member of the Structural Engineers Association of Northern California and the American Segmental Bridge Institute. He is a registered Civil Engineer in the State of California.

He has received many awards, including the ASCE Moissieff Award three times, in 1976, 1981, and 1992; the Western Electric Award for Excellence in Engineering Teaching in 1978; the Best Paper Award from the Canadian Society of Civil Engineers in 1982; the K. B. Woods Award of the Transportation Research Board, National Academy of Sciences in 1982; the ASCE Howard Award in 1989; and the University of California "Berkeley Citation" in 1990. In 1987 he was awarded the Bryon L. and Elvira E. Nishkian Chair as Professor of Structural Engineering, and in 1989 he was appointed by Governor Deukmejian to the Governor's Board of Inquiry on the 1989 Loma Prieta Earthquake. He is a Member of the Caltrans Seismic Advisory Board and the Chairman of the Golden Gate Bridge Seismic Instrumentation Advisory Panel. In 1994 he was appointed to the Blue Ribbon Panel for the review of the structural evaluation of the Kingdome in Seattle.

He was elected into the National Academy of Engineering in 1978 with a citation for "pioneering the development and application of advanced structural analysis to the design of record breaking and unique structural systems." In 1993 he received the highest honor of the ASBI, the Leadership Award, for "outstanding contributions in research and computer program development for analysis of segmental concrete bridges." In 1993 he received the Berkeley Engineering Alumni Society Distinguished Engineering Alumnus Award, which stated: "...who by his contributions to elegant structures and devotion to educating generations of Berkeley engineers has brought distinction to the College of Engineering and its alumni." In 1994 he received the highest honor, awarded only every three or four years, by two separate international organizations: first from IASS, the Torroja Medal, "for his contributions to the analysis and design of thin shell concrete structures"; and second from FIP, the Freyssinet Medal, "for his contribution in prestressed concrete structures."

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NLFEARC: Look Both Ways Before Crossing

by F. J. Vecchio and D. Palermo

Synopsis: A critical look is taken at the state-of-the-art in nonlinear finite element analysis of reinforced concrete structures. In examining the results of recent prediction competitions, the accuracy of such analysis procedures is gauged. Reasons for caution when applying nonlinear analysis methods are then identified and discussed. Finally, the results of a test program involving shear critical beams are presented in support of the contention that the behaviour of reinforced concrete is still not well understood. The tests represent a good challenge for validating current procedures.

Keywords: analysis; concrete structures; finite element; nonlinear; panel test; shear; shear beams

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Daniel Palermo, P. Eng, is a doctoral candidate in the Department of Civil Engineering, University of Toronto. He received his M.A.Sc. and B.A.Sc. from the University of Toronto. His research interests include constitutive modelling of concrete subjected to cyclic loading, and experimental testing of shear walls.

INTRODUCTION

Nonlinear finite element analysis of reinforced concrete (NLFEARC) has taken tremendous strides forward since initial applications about 40 years ago. Much research activity has occurred in the realm of constitutive modelling of reinforced concrete behaviour and in the development of sophisticated analysis algorithms. Occurring at the same time, and no less significant, were quantum leaps in computing technology, greatly expanding the size of analyses that can be considered and greatly reducing the time required for solution.

The state-of-the-art has progressed to the point where NLFEA is close to being a practical, every-day tool in the arsenal of design office engineers (1). No longer the purview of the researcher, it is finding use in various applications; many relating to our aging infrastructure. NLFEA procedures can be used to provide reliable assessments of the strength and integrity of damaged or deteriorated structures, or structures built to previous codes, standards or practices deemed to be deficient today. They can be valuable tools in assessing the expected behaviour of retrofitted structures, or in investigating and rationally selecting amongst various repair alternatives. And, in situations that haven't turned out well, NLFEA procedures are finding applications to forensic analyses and litigations that follow. Not far down the road, they will likely form the main engine in computer-based automated design software, in a form likely invisible to the user.

THE QUESTION OF ACCURACY

Despite the increasing sophistication of NLFEA tools, users must be ever mindful of the question of how accurately and reliably do they represent the

behaviour of reinforced concrete structures. In this regard, it is useful to examine the results of three recent 'prediction competitions'.

At the 1981 IABSE Symposium in Delft, a 'blind' competition was organized involving four panels tested in a comprehensive research program then underway at the University of Toronto. The panels were orthogonally reinforced, and subjected to uniform stress conditions: seemingly a very simple problem to model and analyse. The results of these panel tests were not disclosed prior to analysts submitting their *predictions* of strength and load-deformation response. Approximately thirty entries were received, many from the leading researchers in the field at the time. Shown in Figure 1 is the range of responses for Panel C, one of the better predicted of the four panels. The analysis results submitted showed a wide variation in predictions of the panel's shear strength, and an even wider divergence in computed load-deformation responses (2). Clearly, the collective ability to model nonlinear behaviour of reinforced concrete, particularly in shear-critical conditions, was not well advanced. This is not to suggest that the experimental results are represented by an absolute curve. Scatter also exists in the experimental data, and it would be more realistic to represent the data by a range of results. The analytical scatter could then be viewed with a clearer perspective.

More recently, in 1995, the Nuclear Power Engineering Corporation of Japan (NUPEC) staged a prediction competition involving a large-scale 3-D shear wall subjected to dynamic cyclic loading (3). The flanged shear wall exhibited highly nonlinear behaviour before sustaining a sliding shear failure along the base of the web. One facet of the competition required estimates of the ultimate strength, and corresponding displacement, of the wall as determined from static pushover analyses. Again, over thirty sets of predictions were received; the results are summarized in Figure 2. The predictions of strength, as a group, showed better correlation than were seen with the Toronto panels; however, the deformation estimates still showed large scatter. Nevertheless, it could be concluded that the ability of NLFEA to accurately capture the behaviour of reinforced concrete had measurably advanced. It should be noted, however, that this was not a completely 'blind' competition since some of the test results had been disclosed prior to the competition.

Finally, we have a series of large-scale columns tested at the University of California at San Diego. These columns were the subject of an informal competition organized recently by ASCE-ACI Committee 447. Many of the analyses undertaken pursuantly are documented in papers contained within this set of proceedings. As one reads through the papers, it can be noted that: i) a number of quite different analysis approaches were taken; ii) predictions of strength and pre-peak response generally correlate well with the experimental results; and iii) predictions of post-peak response are generally not as accurate and still require further attention. [Bear in mind, once again, that this was not a

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blind competition. Analysts had the opportunity to *calibrate* parameters, *optimize* material models, and *refine* analyses.]

Nevertheless, it is an inescapable conclusion that our ability to accurately model the behaviour of reinforced concrete structures has seen tremendous improvement over the past 20 years. It is approaching a stage of development where we may be inclined to proceed with a certain degree of confidence.

REASONS FOR CAUTION

Despite seemingly significant advancements in our ability to accurately model the response of reinforced concrete, the users of NLFEA procedures need to be mindful of several issues and caveats:

Diversity of Theoretical Approaches

A number of rather diverse approaches exist for NLFEA modelling of reinforced concrete structures. Among those available are models built on nonlinear elasticity, plasticity, fracture mechanics, damage continuum mechanics, endochronic theory or other hybrid formulations. Cracking can be modelled discretely, or using smeared crack approaches; the latter can range from fully rotating crack models, to fixed crack models, to multi-orthogonal crack models, to hybrid crack models. Some approaches place heavy emphasis on classical mechanics formulations; others draw more heavily on empirical data and phenomenological models. It can generally be said of any approach that it will be more suited to certain structure/loading situations and less so to others. No one approach performs well over the entire range of structural details and loading conditions encountered in practice.

Diversity of Behaviour Models

Reinforced concrete structures, particularly in their cracked states, are dominated in their behaviour by a number of second-order mechanisms and influencing factors. Depending on the particular details and conditions prevailing, a structure's strength, deflection, ductility and failure mode may be significantly affected by mechanisms such as: compression softening due to transverse cracking, tension stiffening, tension softening, aggregate interlock and crack shear slip, rebar bond slip, rebar dowel action, scale effects, and creep and shrinkage, to name a few. For each of these, a number of diverse formulations can exist. In the case of tension stiffening, for example, the

stiffening effect can be ascribed to a post-cracking average tensile stress in the concrete or, quite differently, to the constitutive response of the rebar steel. The user of NLFEA software must be aware of what mechanisms are likely to be significant in the problem at hand, be certain that it is included in the analysis model, and have some confidence that the model being used is reasonably accurate.

Incompatibility of Models and Approaches

The formulation and calibration of a concrete behaviour model, as it is being developed, is often dependent on the particular analysis methodology being used. As a consequence, some models cannot be randomly transplanted from one analysis approach to another, or haphazardly combined with other models. Often, they are developed in tandem with other complementary material models, or analysis approaches, and should not be separated. As a case in point, consider the observed and predicted behaviour of Panel PV19 (4); this was, in fact, Panel C from the 1981 Delft Competition, represented in Figure 1. An important mechanism influencing the shear strength and deformation response of this element was the softening of the concrete in compression due to transverse cracking, with the panel eventually sustaining a concrete shear failure. Shown in Figure 3 are the predicted responses obtained using the compression softening models of Vecchio and Collins implemented in a rotating crack formulation, and of Maekawa implemented in a fixed crack formulation (i.e., each correctly matched with crack model for which it was first developed). Both provide equally good simulations of response. The Vecchio-Collins formulation slightly over-estimates strength and slightly under-estimates ductility. Conversely, the Maekawa formulation slightly under-estimates strength and slightly over-estimates ductility. Either one, however, is certainly well within the margins of accuracy we can hope to achieve with NLFEA. But consider what happens if one implements the Vecchio-Collins model into a fixed crack formulation, or if one uses the Maekawa model in a rotating crack formulation. In both cases, the results are much less satisfactory; strength, ductility and failure mode are subject to significant miscalculation. As it turns out, a hybrid formulation between fixed and rotating crack models (5) provides the most accurate simulations in this case.

Experience Required

Use of NLFEA for modelling and analysis of reinforced concrete structures requires a certain amount of experience and expertise. Unlike, say, the use of plane section analysis techniques to calculate the flexural strength of a beam cross section, the application is rarely straightforward. Decisions made

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with respect to modelling of the structure and selection of material behaviour models will have significant impact on the results obtained. Again, unlike sectional analysis techniques, two analysts may very well get widely diverging results when modelling the same structure using the same analytical model and the same software. This is well known to instructors of graduate courses on NLFEARC, even when considering as simple a case as a simply supported beam with a mid-span load. Decisions made regarding mesh layout, type of element used, representation of reinforcement details, support conditions, method of loading, convergence criteria, and selection of material behaviour models, will result in significant scatter in the results. Add to this the increased likelihood of errors in input due to the relative complexity of NLFEA. [The scatter of results is only marginally reduced when the group of analysts is reduced to those actually working in the field of nonlinear analysis and modelling of RC structures!] Hence, there is ample explanation for the significant difference in accuracy of results obtained in 'blind' competitions as opposed to those obtained when the desired results are known in advance.

Reams of Results

NLFEA invariably produces huge quantities of data, typically spawning output files of several megabytes or larger in size for each load stage. Information may be provided on: stresses and strains at each integration point of each element, both with respect to local and principal axes, both for the element and for the concrete component; nodal displacements; sectional forces per unit width at each integration point; reactions; reinforcement stresses and strains; stiffness matrix coefficients, etc. Considering that typical problems can involve tens of thousands of degrees of freedom, the total amount of data quickly escalates to the point where the use of post-processors is virtually essential. Even then, the analyst must have an awareness of what to look for and how to interpret it. Still, even with the use of sophisticated post-processors and graphics capabilities, there is the possibility for misinterpretation. After all, it was the incorrect processing and interpretation of FEA data that was directly responsible for the collapse and loss of the \$500M Sleipner A offshore platform (6).

Incomplete Knowledge

To paraphrase Bob Dylan from one of his songs, 'we should know our song well before we sing it'. So, too, we should accept that we still do not understand well, let alone have accurate models for, many aspects of reinforced concrete behaviour. [See 'Challenging Set' below.] Applications of NLFEA should be done with a healthy dose of caution and scepticism. Wherever possible, analysis software and models should be validated or calibrated against

benchmark tests involving specimens of similar construction and loading details, dependent on mechanisms anticipated to be significant in the analysis problem at hand (as far as this can be done). And wherever possible, results should be supported by analyses based on different models or approaches.

Research Philosophy

Lastly, it must be said that the research community, and associated technical committees, have failed the profession in some respects. Many working in the area have directed their efforts to developing sophisticated models and methods of analysis, in many cases basing their work on esoteric models or rigorous application of classical mechanics approaches not directly suited to reinforced concrete. Advancements have been made in developing the NLFEA concepts and methodologies, but often with reinforced concrete being merely the *application*. Unfortunately, reinforced concrete is a complex and stubborn material that sometimes refuses to act according to accepted rules of mechanics or even common logic. Instead, efforts should be refocused toward better understanding and modelling reinforced concrete behaviour, with finite element analyses being merely the *tool*. Certainly, however, there is room and need to advance on both fronts.

A CHALLENGING SET OF TEST RESULTS

To reinforce the notion that we still do not know enough about the behaviour of reinforced concrete, and to provide a challenge to those who might think otherwise, consider two series of beams tested by Angelakos et.al. (7).

The first series involved five beams each of which was 6000 mm in length, 1000 mm in depth, 300 mm wide, reinforced with approximately 1.0 percent longitudinal reinforcement, containing no transverse reinforcement, and subjected to a monotonically increasing load applied at the midspan. [Beams DB120, DB130, DB140, DB165 and DB180; see Figure 4 for beam details.] The only variable was the compressive strength of the concrete, ranging from 20 MPa to 80 MPa. These beams failed in a shear critical manner, upon the formation of the first web shear crack. Current design code formulations would say that the shear strength of these beams is directly proportional to a 'concrete contribution' related to the tensile strength of the concrete, which in turn is normally related back to the compressive strength (typically, the tensile strength is taken as proportional to the square root of the compressive strength). Hence, the 80 MPa beam could be expected to have a shear strength of close to double that of the 20 MPa beam. Finite element analyses could also be expected to produce similar trends in predicted strength, since concrete tensile strength is an

over-riding parameter in most analyses in such cases. Shown in Figure 5 are the load-deformation responses measured experimentally. Note that there is little difference in the strengths and pre-peak deflection responses obtained; certainly nothing approaching the doubling of strength anticipated. In fact, the 80 MPa beam exhibits a shear strength lower than the 20 MPa beam. At work are mechanisms relating to smoothness of the fracture plane, aggregate interlock mechanisms, and crack slip mechanisms.

A second series of beams from the same test program involved four beams similar in dimensions and loading to the first [Beams DB120M, DB140M, DB164M and DB 180M; see Figure 4]. The principal difference was that these beams contained the near-minimum amount of shear reinforcement, at an amount of 0.22%. Again, the compressive strength of the concrete ranged from 20 MPa to 80 MPa. Shown in Figure 6 are the load-deformation responses recorded. Note that here, a small amount of shear reinforcement had a substantial influence on the strength and failure mechanisms observed. Although the higher strength concrete beams did exhibit a higher shear strength, there was still a good deal of perplexing behaviour observed. See Angelakos et al (2001) for a more complete description of the test program, and a more thorough discussion of results and significance.

These two series of test results will provide a stringent test of any NLFEA model. Analysts are encouraged to formulate all models, and select all design models and parameters in advance of any 'preliminary' analysis, and to conduct the analyses in a group and only once! *Know your song well before you sing it!*

CONCLUSIONS

The state-of-the-art in the nonlinear finite element analysis of reinforced concrete structures has advanced significantly in recent years. The speed and accuracy of analyses have progressed to the point where NLFEA software are close to being an indispensable design office tool, useful in many types of practical applications. In coming years, they may well form the heart of advanced software for automated design, albeit in a form transparent to the user.

However, NLFEA models and procedures for reinforced concrete remain complex, fragmented, and fraught with dangers. The user of these procedures must be experienced, cautious and somewhat sceptical.

Future work must be directed at developing improved material behaviour models that more accurately represent the behaviour of reinforced concrete under the diverse conditions encountered in practice. As well, efforts are needed

in reconciling the incompatibilities of models and approaches, and in reducing the potential for errors in modelling, analysis and interpretation of results.

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NOTATION

f'_c = compressive strength of concrete cylinder at 28 days
 f_y = yield strength of reinforcement
 ϵ_o = concrete strain at peak compressive stress
 γ = shear strain
 ρ = reinforcement ratio
 τ = shear stress

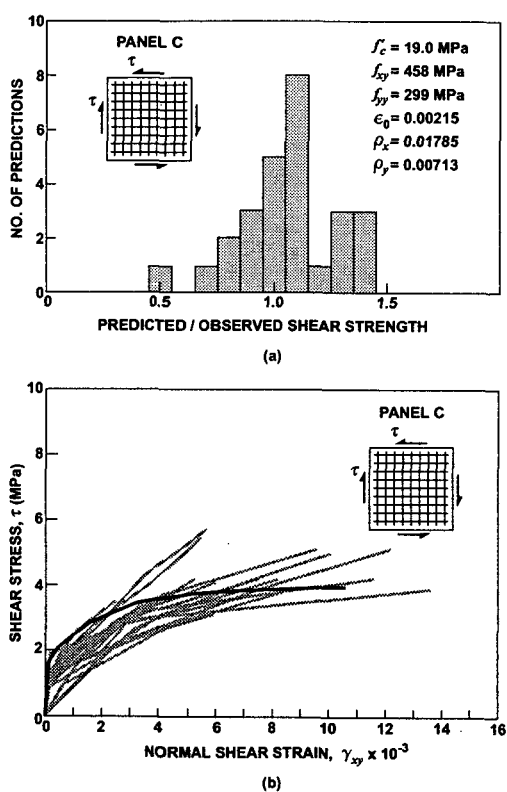


Figure 1: Delft Competition Results

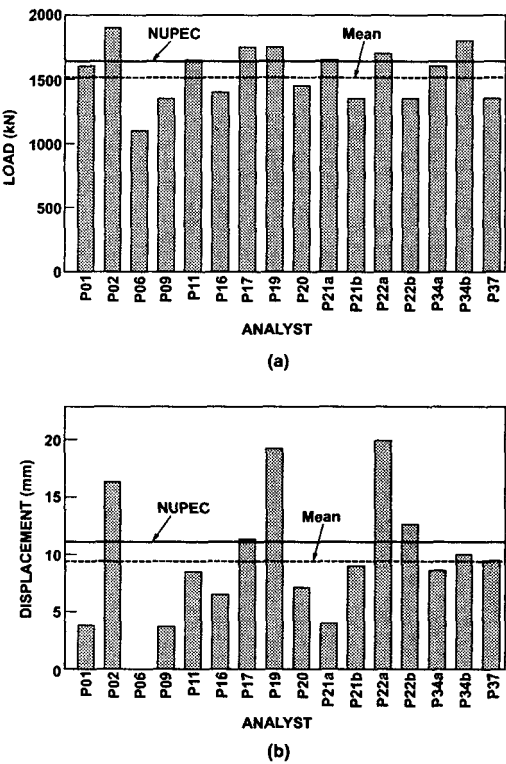


Figure 2: NUPEC Wall Prediction Results

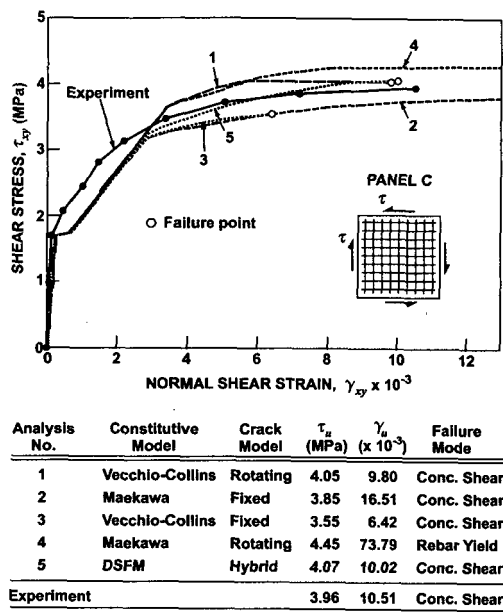


Figure 3: Predictions of Panel PV19

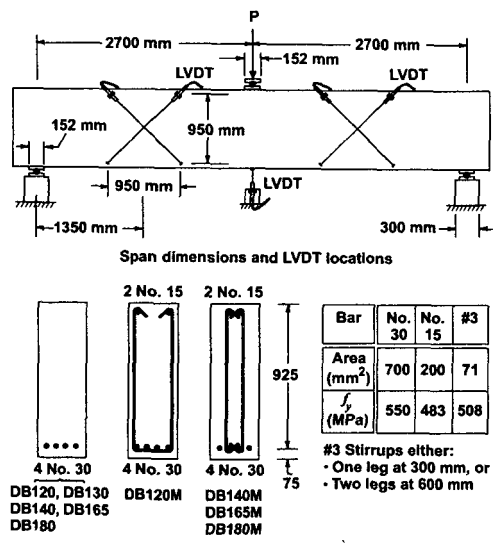


Figure 4: Details of Angelakos Beams

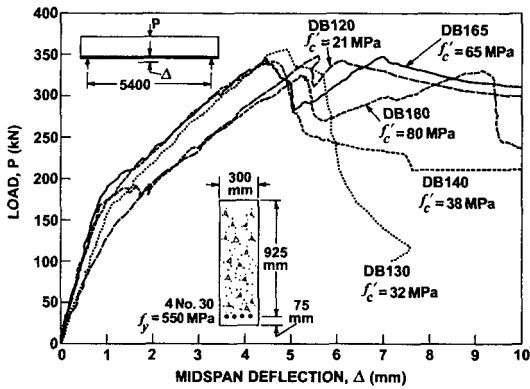


Figure 5: Results of Series I Beams

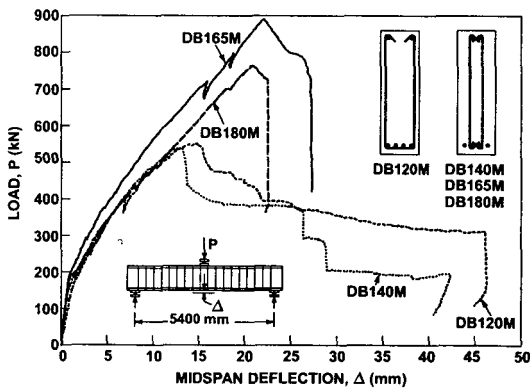


Figure 6: Results of Series II Beams

UCSD Shear Column Benchmark Tests

by R. K. Dowell and F. Seible

Synopsis: The paper presents the UCSD reinforced concrete shear columns, and measured test results, which participants have been invited to model as part of a benchmark analysis. Both “as-built” and seismically retrofitted column results are presented. The columns were 1/3-scale, rectangular bridge columns that were loaded in double bending which produces twice the shear force demand of an equivalent cantilever column. While the shear force capacity of the “as-built” columns was essentially the same, the shear force demand was different for each of the columns due to the use of Grade 40 and Grade 60 vertical steel and due to the different column heights. This resulted in one column failing in a brittle shear manner prior to reaching nominal moment and ductility 1, another failing at displacement ductility 1.5 and the third failing at displacement ductility 3. The challenges to the participants of the benchmark analysis are to capture the peak shear force at failure and post-peak force-deformation behavior for the three “as-built” columns. In the paper a discussion is presented regarding difficulties with modeling and testing reinforced concrete structures.

Keywords: ductility; reinforced concrete; seismic loading; seismic retrofit; shear capacity; shear columns; structural tests

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OVERVIEW

"As-Built" Columns

The benchmark column tests were conducted at the Charles Lee Powell Structural Research Laboratories at the University of California, San Diego and are discussed in detail in [1]. All 6 test specimens were 1/3-scale, rectangular reinforced concrete (RC) bridge columns loaded in double bending (Figure 1) under simulated seismic conditions. Due to the large shear force input of the columns, owing to their short length and double bending loading condition, they are considered "shear columns" with failure of the "as-built" columns from shear or a combination of shear and flexure. The three "as-built" columns had a 24" x 16" rectangular section with 22 #6 longitudinal bars (Figure 2b) spaced evenly around the section (longitudinal steel ratio of 2.5 %) and an axial load of 114 kips (nominal axial load ratio of 6 %, with axial load ratio defined as the axial load divided by the nominal axial capacity of the column). Columns R-1 and R-3 had a height of 96 in.(aspect ratio or shear span of 2) while Column R-5 had a reduced height of 72 in.(aspect ratio of 1.5). Column R-1 had Grade 40 longitudinal steel while Columns R-3 and R-5 had Grade 60 longitudinal steel (see Test Matrix in Figure 3).

Based on capacity design principles, the maximum shear force demand, V , on a bridge column is a function of the moment capacity, M , of the end sections (where plastic hinges are designed and detailed to occur) and the column height, H ($V=2M/H$). Thus Column R-1 had the least shear force input of the "as-built"

columns because of the reduced column end moment capacity (Grade 40 steel) and the increased height (aspect ratio of 2). Column R-5 had the greatest shear force input of the three “as-built” columns due to the Grade 60 longitudinal steel and reduced column height (aspect ratio of 1.5). Column R-3 had a shear force input somewhere between that of Columns R-1 and R-5 because of its Grade 60 longitudinal steel (same as Column R-5) and the larger aspect ratio of 2 (same as Column R-1). All three “as-built” columns had #2 rectangular hoops @ 5 in. spacing which approximates #4 stirrups at 12 in. spacing for the full-scale prototype column of 1960s era bridge design. Thus the shear force capacity of the three “as-built” columns is nominally the same but the shear force demand increases from Column R-1 to R-3 and from Column R-3 to R-5. The result is that (1) Column R-5 suffered a brittle shear failure prior to ductility 1, (2) Column R-3 experienced a brittle shear failure at displacement ductility 1.5 and (3) Column R-1 had a flexural shear failure at displacement ductility 3.

Retrofit Columns

In addition to the three “as-built” column experiments, three retrofit column tests were conducted to demonstrate that shear degradation and failure observed in the “as-built” column tests, from simulated seismic loading, can be mitigated by adding elliptical steel shells to the section up the member height (Figure 2a). The elliptical steel shell provides the necessary shear capacity and confinement to change the dominant shear behavior and failure mode to flexure, which allows the nominal flexural strength of the member to be obtained and large displacement ductility capacity to be reached. The void between the elliptical steel shell and the rectangular concrete section is filled with concrete so that hoop tension, developed in the shell, directly confines the section. As indicated in Figure 3, a 1-inch gap has been provided at the column ends between the steel shell and concrete infill and the connecting footing and load stub. This limits (1) undesirable flexural strength enhancements at the column ends, (2) shear demand on the column as well as (3) forces that are transmitted to the connecting members. Retrofit Columns R-2, R-4 and R-6 are companion tests to “as-built” Columns R-1, R-3 and R-5, respectively (see the Test Matrix in Figure 3). Note that the “retrofit” test specimens are new test specimens constructed as “as-built” specimens and retrofitted with the steel shell and concrete infill.

Loading Protocol

All of the “as-built” and retrofit columns were loaded in double bending by applying the horizontal actuator load at mid-height of the column (shown in Figure 1), producing twice the shear force demand than a similar cantilever

column. A vertical actuator was provided to ensure zero rotation of the load stub and to counter the weight of the large steel loading arm. The axial load of 114 kips was provided by high-strength rods attached to the top of the load stub and the underside of the high-strength floor. Slotted holes in the footing allowed the rods to move without bearing on the test specimen. A load cell at the top of the load stub ensured that the axial force in the rods remained constant while the column member elongated due to plastic hinging and diagonal shear cracking.

The loading sequence consisted of even push and pull cycles that were initially conducted in force control and then were switched to displacement control following theoretical first yield of the longitudinal reinforcement. First yield displacement was found by averaging measured push and pull displacements at theoretical first yield force, determined from moment-curvature analysis. The idealized yield displacement, which defines displacement ductility 1, was determined by multiplying the measured first yield displacement by the ratio of nominal to first yield moment. With displacement ductility 1 defined, the remaining loading takes place in displacement control with three complete push and pull cycles at each target displacement based on displacement ductilities 1, 2, 3, 4, 5, 6 and 8.

TEST RESULTS

“As-Built” Columns

Test results for “as-built” Columns R-1, R-3 and R-5 are given in Figures 4, 5 and 6, respectively. In each of these figures (a) is the overall force-deformation response, (b) the crack patterns at failure, (c) the force-flexural deformation response and (d) the force-shear deformation response. To better understand the behavior of these shear-dominated columns, the total column top displacement was separated into flexural and shear components. This was accomplished by separating measured displacements from the three panels of horizontal, vertical and diagonal displacement potentiometers up the column height (Figure 1) into average curvatures and shear deformations per panel. The average curvature of a panel is the difference in vertical displacements divided by the distance between potentiometers and divided by the panel height. Flexural displacement within a panel height is the slope at the bottom of the panel multiplied by the panel height plus the average curvature multiplied by the square of the panel height divided by two. The total top displacement is the summation of the three panel segment displacements.

Column R-1 reached displacement ductility 3 before suffering a flexural shear failure (Figure 4a). This is not considered a brittle shear failure as the column reached its nominal flexural strength and had some ductility capacity with little

loss of strength. Of particular interest in the separated flexural and shear deformation results is that with each cycle to displacement ductility 3, following shear failure, the flexural deformations decreased as the shear deformations increased (Figures 4c and 4d). Column R-1 had the least severe shear failure of the three “as-built” columns due to the Grade 40 longitudinal steel and shear span of 2.

Column R-3 suffered a brittle shear failure at displacement ductility 1.5, followed by a severely pinched response (Figure 5a). The Grade 60 longitudinal steel increased the shear demand over Column R-1, which reduced the displacement ductility capacity from 3 to 1.5, and changed the flexural shear failure of Column R-1 to a brittle shear failure (Figures 5a and 5b). It is clear from viewing Figures 5c and 5d that the failure can be attributed to shear, with a sudden increase in shear deformations while the stable flexural deformations remained small. Also, the photographs in Figure 5b demonstrate a massive shear failure near the top of the column. Note that for Column R-3 the flexural and shear deformation components are given only to displacement ductility 1.5 because some of the linear potentiometers reached their displacement limit following shear failure.

Column R-5 had the greatest shear force demand of the “as-built” columns due to the Grade 60 longitudinal reinforcement and the reduced column height of 72 in. (shear span of 1.5). As the shear force capacity is nominally the same for the three “as-built” columns, Column R-5 suffered the greatest shear damage with a brittle shear failure prior to ductility 1 (Figure 6a). Following shear failure the shear deformations increased to beyond the flexural deformations (Figures 6c and 6d). Note that for Column R-5 the flexural and shear deformation components are given only to ductility 1 because some of the linear potentiometers reached their displacement limit following shear failure.

The peak flexural and shear displacements at the 1st and 3rd cycles are plotted in Figure 7 against the total measured displacements at the column top for the three “as-built” columns. Also plotted is the summation of flexural and shear deformation components, which demonstrates relatively close agreement to the total applied displacement. This validates the method used to separate flexural and shear displacement components and validates the measured displacements from the linear potentiometers. What is clear from the three “as-built” column tests is that (1) initially the flexural deformations are far greater than the shear deformations, (2) following shear failure the shear deformations increase more rapidly than flexural deformations and (3) following shear failure the 3rd cycle shear deformations are significantly larger than the 1st cycle shear deformations, with corresponding decreases in flexural deformations.

Retrofit Columns

Overall force-deformation results are given in Figures 8a, 8b and 8c for retrofit Columns R-2, R-4 and R-6. Column R-2 is the companion column to “as-built” Column R-1 and, similarly, has the least shear force demand of the three retrofit columns due to the Grade 40 longitudinal steel and larger aspect ratio of 2. The 3/16-in. elliptical steel shell provided enough shear strength and confinement to allow stable, full hysteresis loops to displacement ductility 10 with no sign of shear degradation (Figure 8a). With Grade 60 longitudinal steel and aspect ratio of 2, Column R-4 had a larger shear force demand than Column R-2; this resulted in a more pinched hysteresis response as shown in Figure 8b. Even so, the behavior was flexure dominated with nice stable hysteresis loops to displacement ductility 8 compared to its companion “as-built” Column R-3 which suffered a brittle shear failure at displacement ductility 1.5. The force-deformation results from Column R-6 indicate increased pinching due to the large shear force demand (Grade 60 longitudinal steel and shear span of 1.5) on this column coupled with the reduced shear capacity due to the thinner 1/8-in. thick steel shell (Figure 8c). However, the results show stable hysteresis loops to displacement ductility 7 with little force degradation between cycles. This is a tremendous improvement over companion “as-built” Column R-5 that had a brittle shear failure prior to ductility 1. Note that for Column R-6 the force limit of the actuators was reached in the pull direction just beyond displacement ductility 3, resulting in the asymmetric cycles presented in Figure 8c.

The peak flexural and shear displacements at the 1st and 3rd cycles are plotted in Figure 9 against the total measured displacements at the column top for the three retrofit columns. Although the shear deformations are not insignificant they never dominate the behavior as in the “as-built” columns, with the ratio of shear to flexural displacements remaining approximately constant. Also different from the “as-built” columns is that no additional shear deformations developed between the 1st and 3rd loading cycles at all displacement levels, indicating that no cyclic degradation or shear failure had occurred. Of particular interest is that while the shear force remained approximately constant following ductility 1, the shear deformations continued to increase with increased member displacement, which is similar to flexural behavior.

DISCUSSION OF MODELING & TESTING RC MEMBERS

Modeling RC Members

Reinforced concrete structures are difficult to model within the context of finite element analysis. This difficulty arises from the fact that the material is not

homogeneous, but made-up of concrete and steel components which act together to form a very strong and relatively inexpensive building material. Concrete is good in compression and weak in tension while the steel reinforcement is good in tension, and thus admirably takes the place of the concrete where it has cracked, regardless if the cracks are from flexure or from shear. The structure will progressively soften, however, as a result of cracking. It should be noted that the steel reinforcement may be defined here as weak in compression due to its propensity to buckle unless fully restrained. Thus the relationship between the concrete and the steel is fully symbiotic with their weaknesses being supported by the strengths of the other. Typically, concrete is modeled with a smeared crack approach. This allows the use of a Mohr's circle strain analysis with the understanding that several cracks are included and average strains are used.

Flexural RC bridge columns have traditionally been modeled by simple moment-curvature analysis. In this approach the member displacement is found as the summation of plastic and elastic deformations. For members with a large shear force, the UCSD shear equation [2, 3] provides a realistic force level at shear failure but does not provide post-peak member response. A more detailed assessment of the force-deformation response of a RC member can be accomplished through 2-D or 3-D finite element modeling. In the most comprehensive approach of 3-D finite element modeling the concrete is modeled as solid brick elements and the reinforcement is modeled as sub-elements to the concrete, with the 1-D rebar stiffness added to the stiffness matrix of the 3-D concrete element [4].

While modeling RC structures with the embedded rebar method appears to be an efficient finite element approach, there are several problems which arise, most notably that the bars do not have the ability to buckle as they are modeled as 1-D elements. Bar buckling is significant because bridge columns with well-confined end regions typically fail in the laboratory at large ductility by buckling of the vertical reinforcement, between or over several hoops or spirals, followed by fracture of the vertical bars upon straightening in the next half loading cycle. This failure can be interpreted as a low-cycle fatigue fracture, with the large compressive strains on the inside of the buckled wave reversed in the next half cycle as the bar is straightened, resulting in fracture of the bar. Note that buckling of the vertical steel is often accompanied by fracture of the transverse hoops or spirals.

A second difficulty with the embedded rebar approach is that the bars are typically assumed to have perfect bond between the steel and the surrounding concrete (no relative slip). This is an important consideration when modeling the critical region between a column and connecting member, such as a footing or superstructure, where strain penetration is expected to be approximately 20 bar diameters [2]. Strain penetration is the phenomenon of tension rebar strains at the critical interface, with peak curvature, extending or penetrating into the

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connecting member over a given development length. Integration of these strains in the connection zone results in a definite displacement at the interface between the column and footing, for example, which is clearly observed in tests and contributes to the overall member displacement. In the finite element analysis of the connection zone, strain penetration occurs only by tensile cracking and deformation of the local concrete elements in the footing or superstructure, rather than relative slip between the rebar and concrete. This will often cause unrealistically large deformations of the solid brick elements, which can result in numerical difficulties and loss of convergence in the analysis.

The strain penetration and buckling problems may be resolved by replacing the embedded rebar elements in the critical region with individually modeled nonlinear beam elements, using the same properties as the steel reinforcement. In addition, two nonlinear spring elements are required at each node between the concrete and beam elements. One spring is calibrated to represent the relative slip of the rebar within the concrete and the other spring represents the transverse ties or hoops, which help restrain the vertical bars from buckling. This spring must be stiff in compression so that the bars do not buckle in toward the concrete. Also very important to buckling is that the shape of the steel material loading and unloading responses be represented with a realistic stress-strain hysteresis model that follows various Bauschinger curves that depend on the maximum plastic strain and strain history. This is significant because the tangent of the stress-strain curve represents the current modulus of elasticity, E , at each integration point and plays an important role in buckling. The nonlinear beam elements should include several integration points around the rebar section to capture sudden strain reversals associated with buckling. A low-cycle fatigue model can be included to obtain rebar fracture following buckling.

Other challenges in 3-D finite element modeling of RC structures are to properly account for (1) shear strength and shear stiffness degradation of concrete at wide-open flexural or tensile cracks, (2) non-healing cracks at variable crack angles and (3) the rapid expansion or dilation of confined and unconfined concrete under large compressive strains. Rebar hoop strains and vertical bar buckling are directly related to each other and to the nonlinear expansion of the concrete.

Testing RC Members

Structural testing of RC components is, in many ways, similar to performing detailed nonlinear finite element analyses. Typically, only a portion of the global prototype structure is tested, requiring special attention to the boundary conditions and loading in the test setup. These loading and boundary conditions must be valid for dead load and any additional loads of interest, such as seismic

loading. As it is often not feasible to test at full-scale, scaling effects need to be considered in terms of the test setup and applied loads as well as in the interpretation of the results and application of these results to the full-scale prototype structure.

To minimize scale effects the largest possible scale is typically used within budgetary, time and physical space constraints in the laboratory as well as within actuator force and displacement capacities. Therefore, somewhat of a dilemma exists for large-scale structural testing in terms of repeatability of test results and reduction of errors, in that only one test is usually conducted. This is true for the three “as-built” and three retrofit column tests discussed in the paper, with each test representing a different column. Thus it is of paramount importance that prior to building and testing large-scale test units, the objectives of each test are clearly defined and it is determined how the data of interest will be measured. Typically, strains, deformations and forces are recorded. However, only a limited number of data collection channels are available for a given test and educated decisions must be made regarding the most significant data to measure.

After the test it is often realized that some of the measurements were not needed, while other measurements would have been useful. Strain gages typically fail by a strain of about 0.02, and thus their usefulness is limited to the elastic and small strain ranges, whereas rebar in plastic hinge regions often reach strains several times this value at large ductility. Deformation measurements are typically useful for the entire range of testing, and can yield important results such as the shear and flexural deformations presented in the paper, as well as curvature and displacement profiles up the column height. At large ductility, strains can often be approximately determined from the measured curvature data. It should be noted that sometimes displacement potentiometers stick and result in errors. Following the test, all of the data is reduced and graphed in various ways, which typically reveals any errors in strain gage and displacement potentiometer readings. It is good practice in structural testing to provide symmetrically placed gages and potentiometers for redundancy of data.

Test units are loaded in force or displacement control. For the shear columns discussed in the paper, lateral or seismic loading was initially applied in force control and was switched to displacement control following theoretical first yield of the vertical reinforcement. While the remainder of the lateral loading was performed in displacement control, the axial loads remained in force control. If possible, it is recommended that for component and sub-assembly testing the structure be made statically determinate following the test by measuring any necessary boundary forces. Being able to determine the applied forces of each member inspires confidence in the post-test analysis, especially if the observed behavior was unexpected. Often the intended and measured loadings are not the same due to friction and other causes. Without this knowledge it is often not clear if differences between post-test analysis and

measured test results are due to the analysis model or to the different loading conditions.

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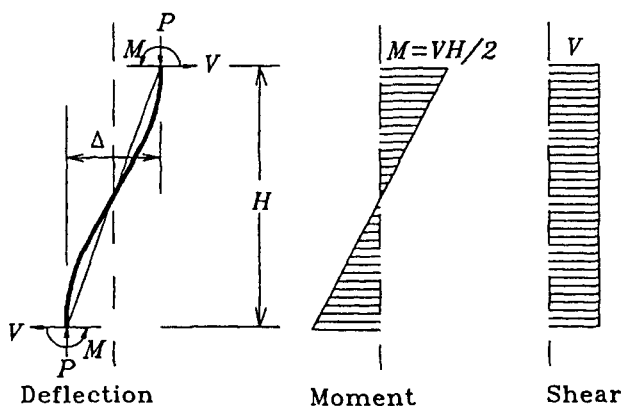
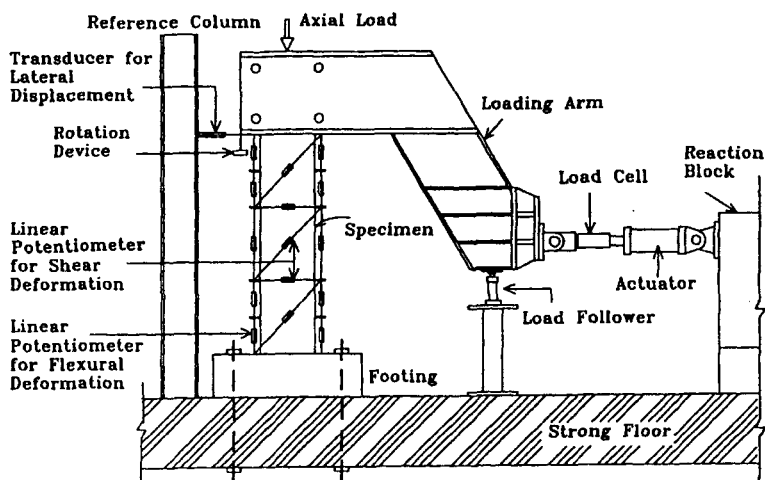
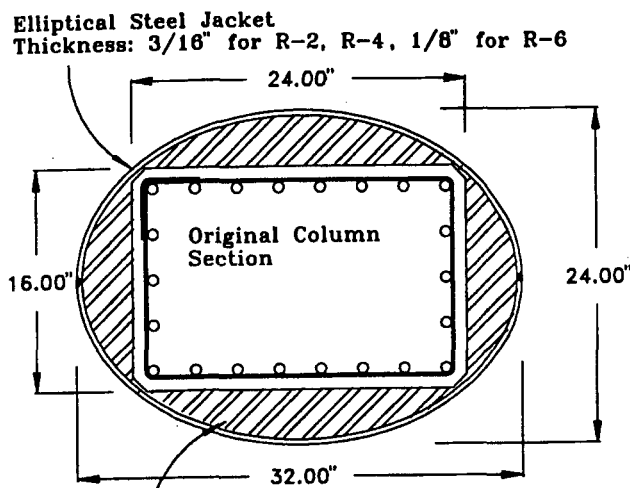
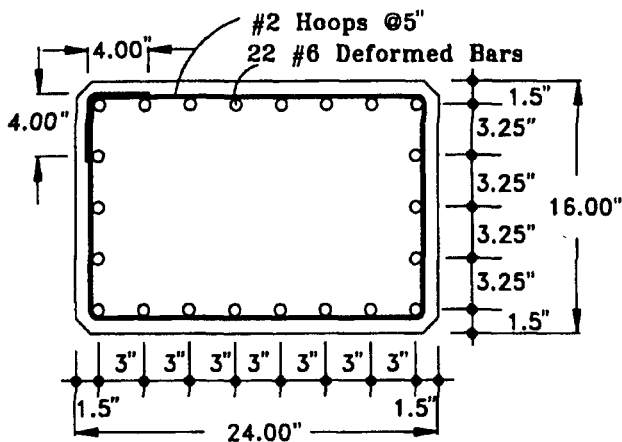


Figure 1. Test Setup and Loading Condition



(a) Retrofit Column



(b) "As-Built" Column

Figure 2. Section Details for "As-Built" and Retrofit Columns

Test Specimen	Longitudinal Steel	Axial Load (kips)	M/(VD)	Retrofit
R-1	Grade 40	114	2	"As Built"
R-2	Steel Area=2.5%			3/16in. Elliptical Jacket
R-3				"As Built"
R-4	Grade 60		1.5	3/16in. Elliptical Jacket
R-5	Steel Area=2.5%			"As Built"
R-6				1/8in. Elliptical Jacket

Note: 1kip=4.45kN; 1in.=25.4mm

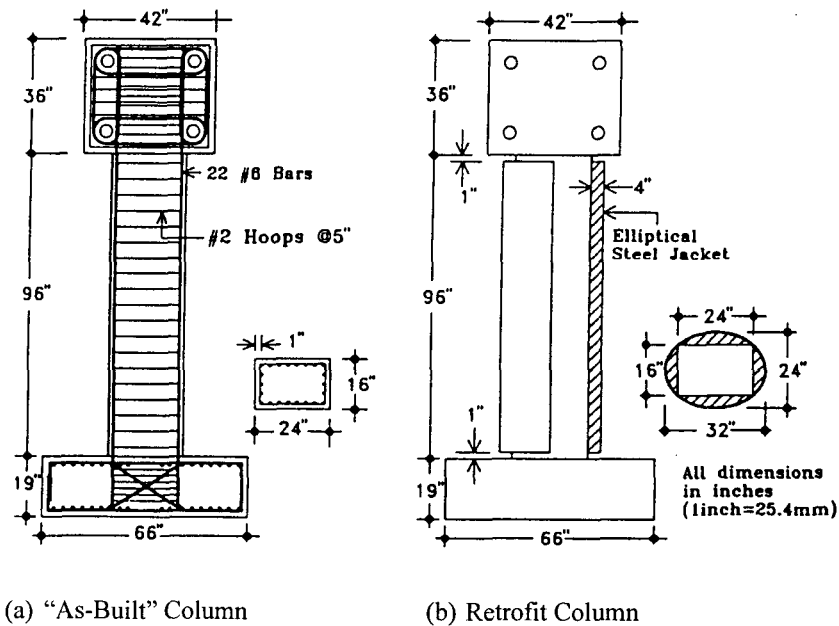
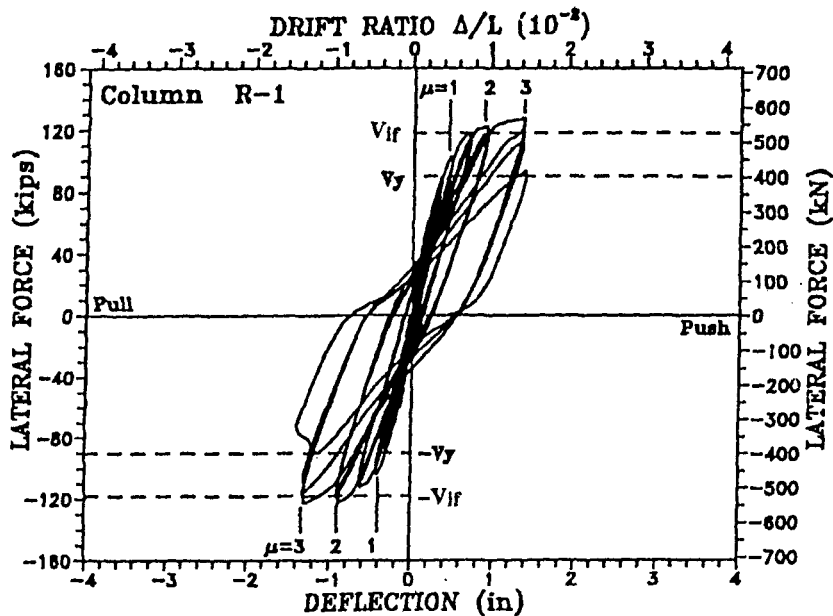
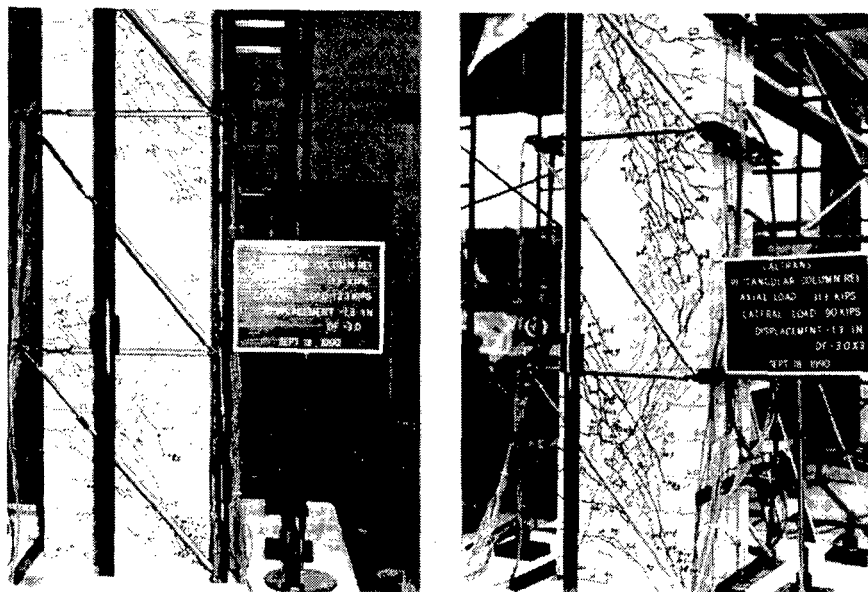


Figure 3. Test Matrix and Reinforcement Details for "As-Built" and Retrofit Columns

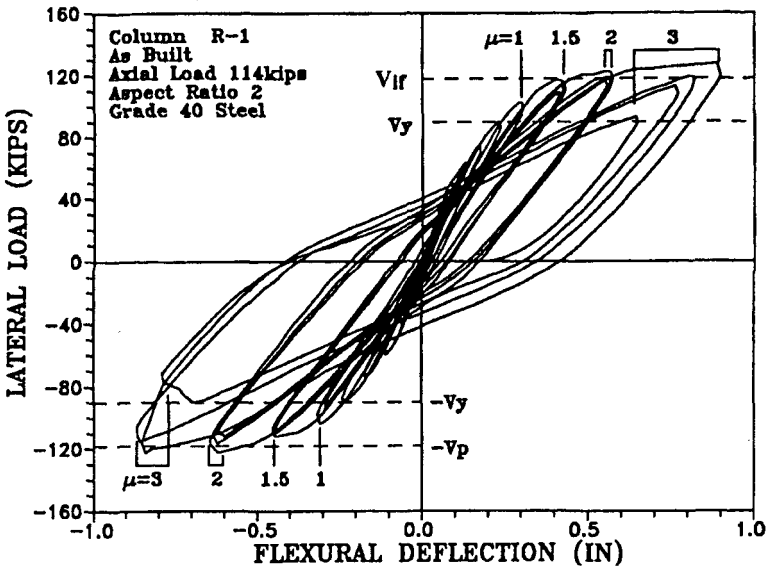


4 (a) Overall Force-Deformation Response (Column R-1)

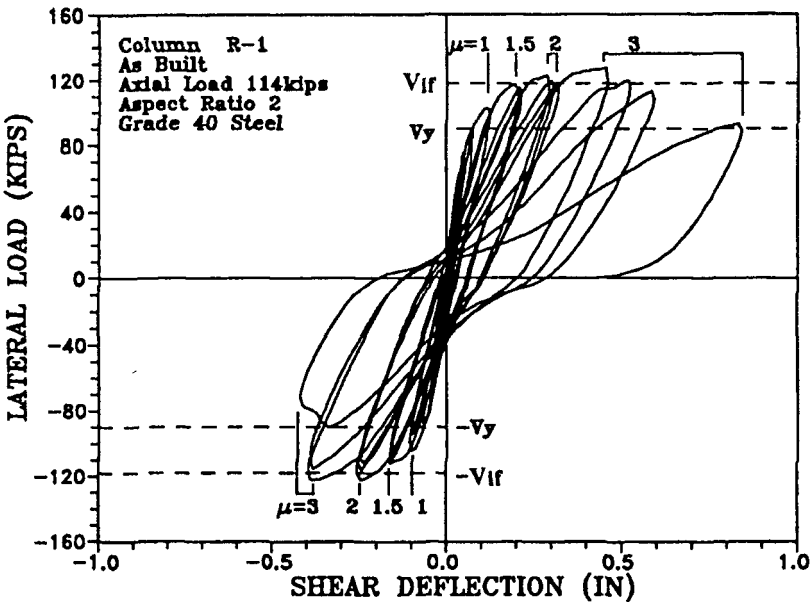


4 (b) Crack Patterns at Displacement Ductility 3 (Column R-1)

Figure 4 continued on the next page.

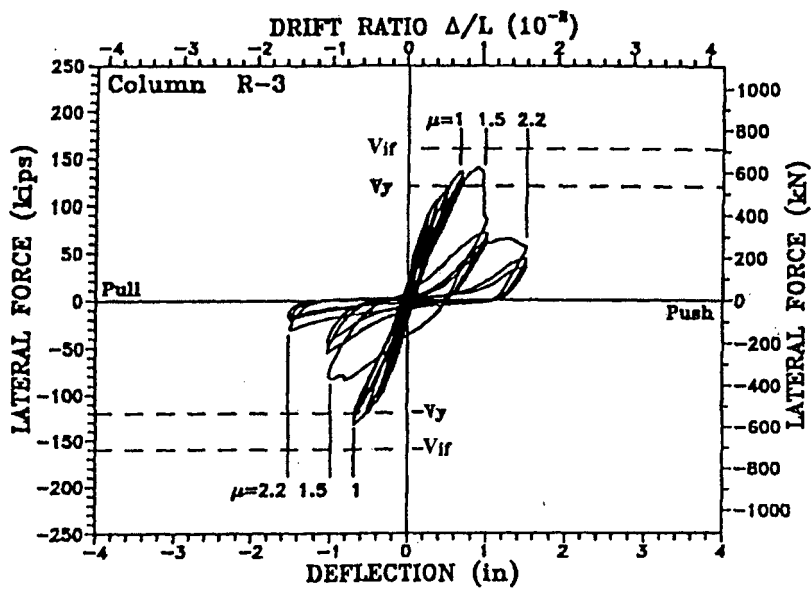


4 (c) Force-Flexural Deformation Response (Column R-1)

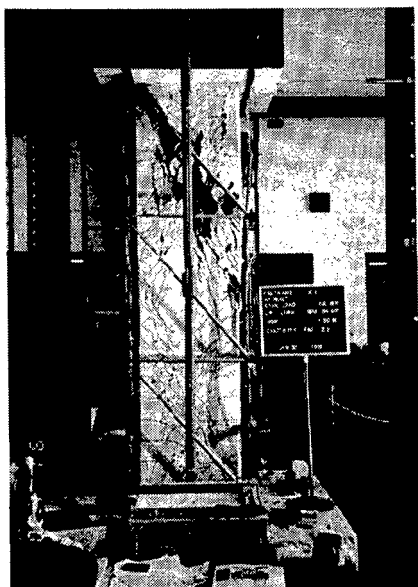


4 (d) Force-Shear Deformation Response (Column R-1)

Figure 4. Behavior of “As-Built” Column R-1

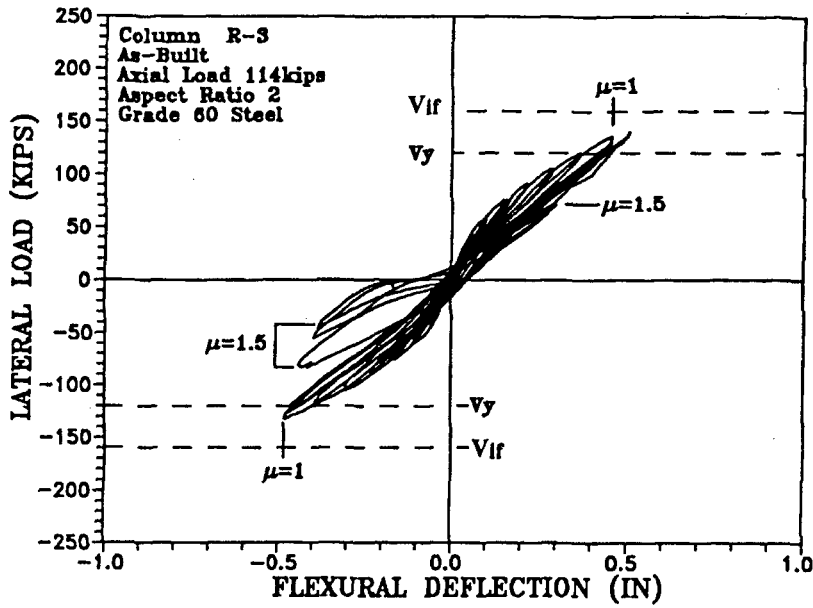


5 (a) Overall Force-Deformation Response (Column R-3)

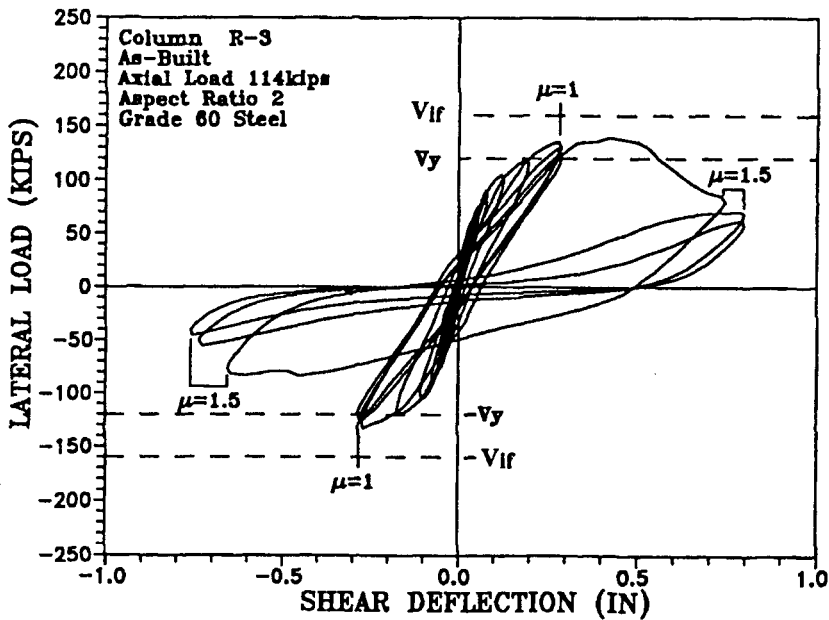


5 (b) Crack Patterns at Displacement Ductility 2.2 (Column R-3)

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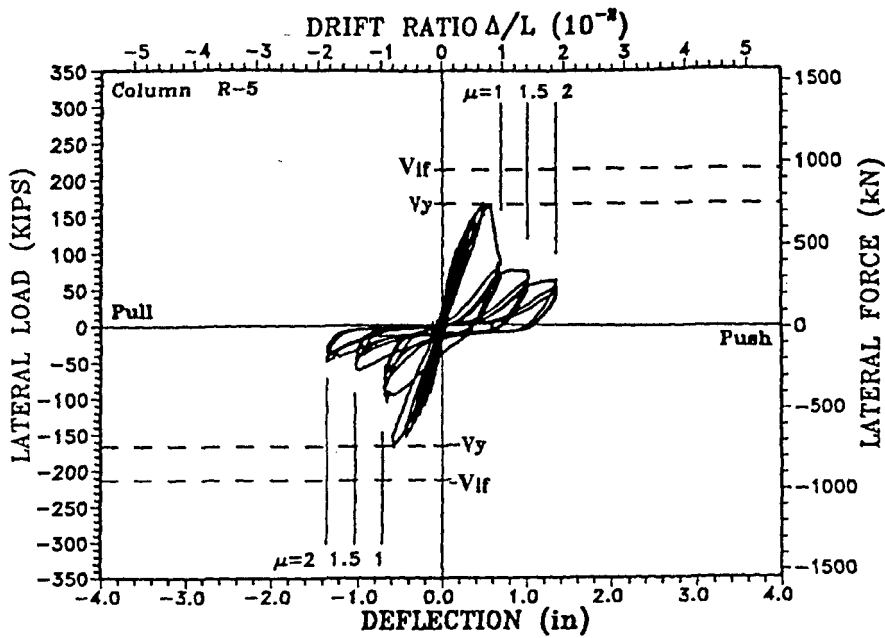


5 (c) Force-Flexural Deformation Response (Column R-3)

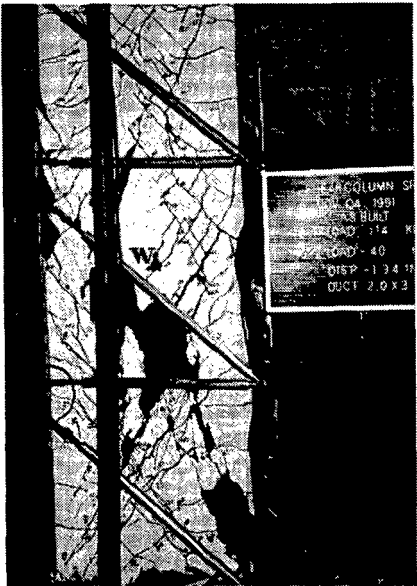


5 (d) Force-Shear Deformation Response (Column R-3)

Figure 5. Behavior of "As-Built" Column R-3

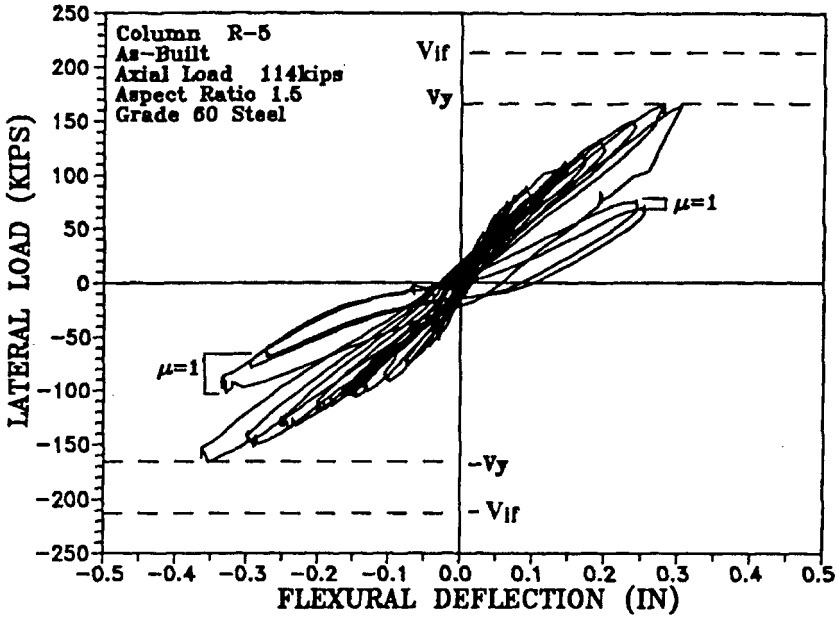


6 (a) Overall Force-Deformation Response (Column R-5)

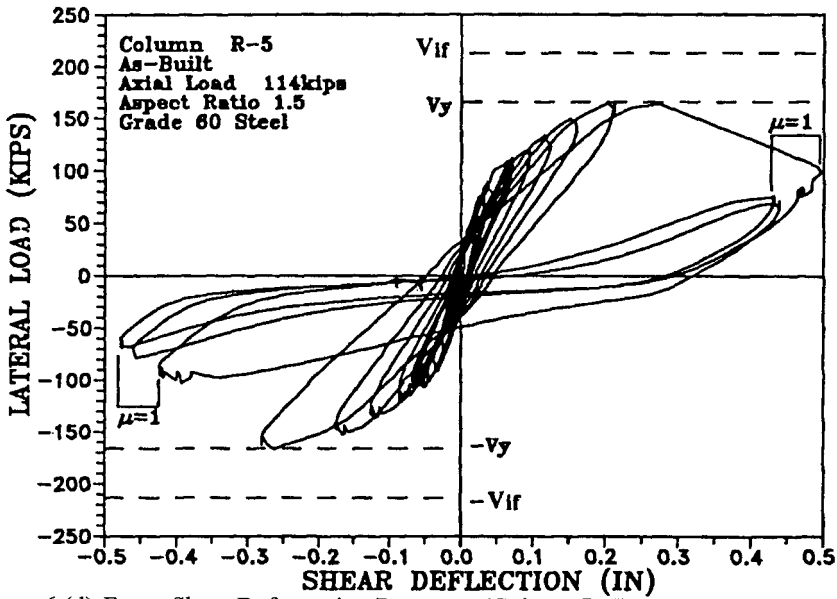


6 (b) Crack Patterns at Displacement Ductility 2 (Column R-5)

Figure 6 continued on the next page.

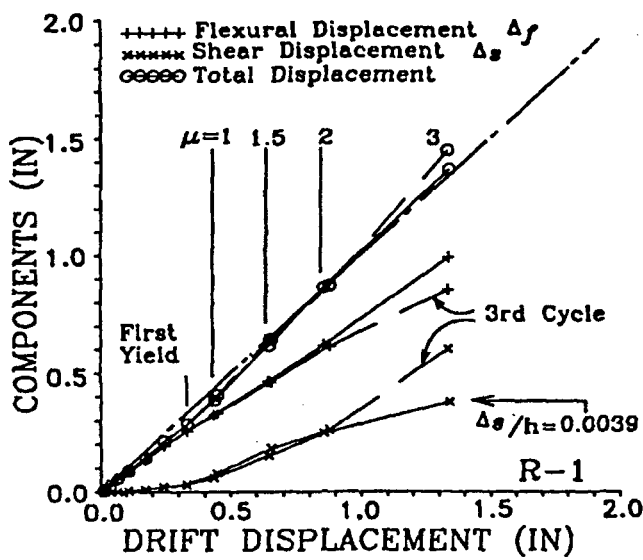


6 (c) Force-Flexural Deformation Response (Column R-5)

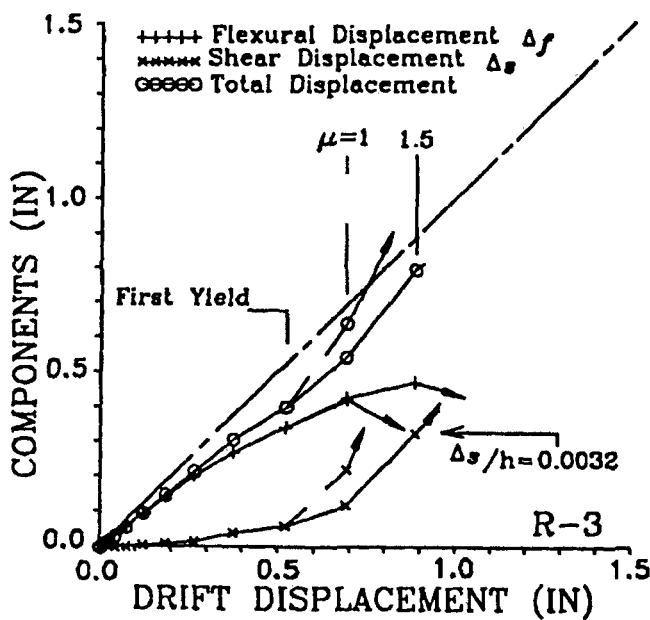


6 (d) Force-Shear Deformation Response (Column R-5)

Figure 6. Behavior of "As-Built" Column R-5

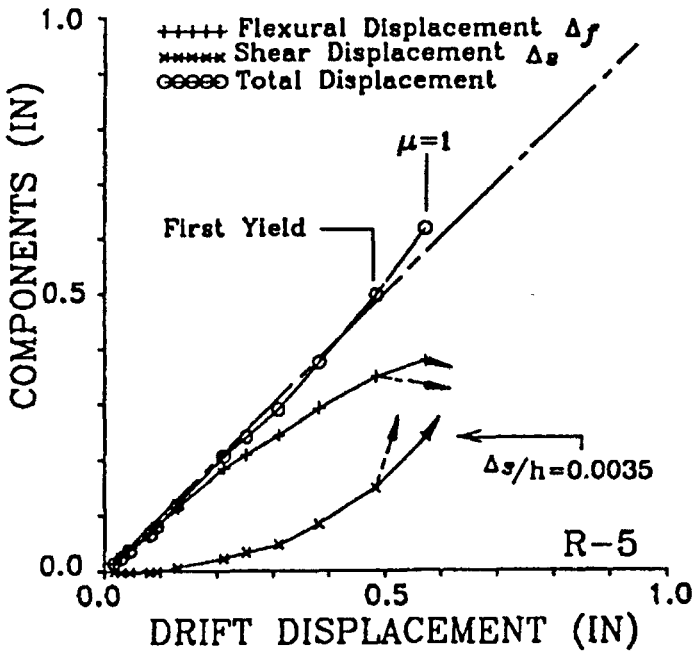


7 (a) Column R-1



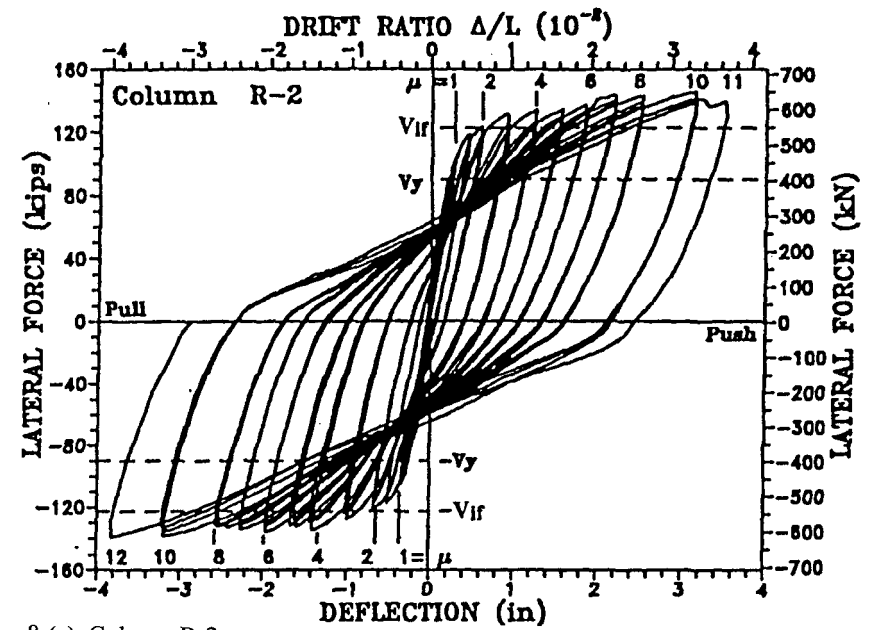
7 (b) Column R-3

Figure 7 continued on the next page.

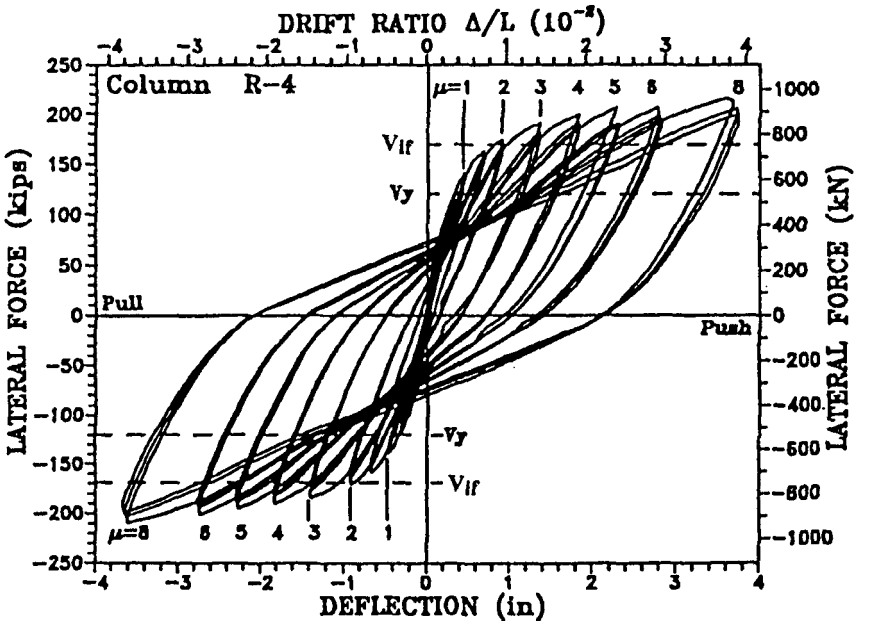


7 (c) Column R-5

Figure 7. Peak Flexural and Shear Displacements for “As-Built” Columns



8 (a) Column R-2



8 (b) Column R-4

Figure 8 continued on the next page.

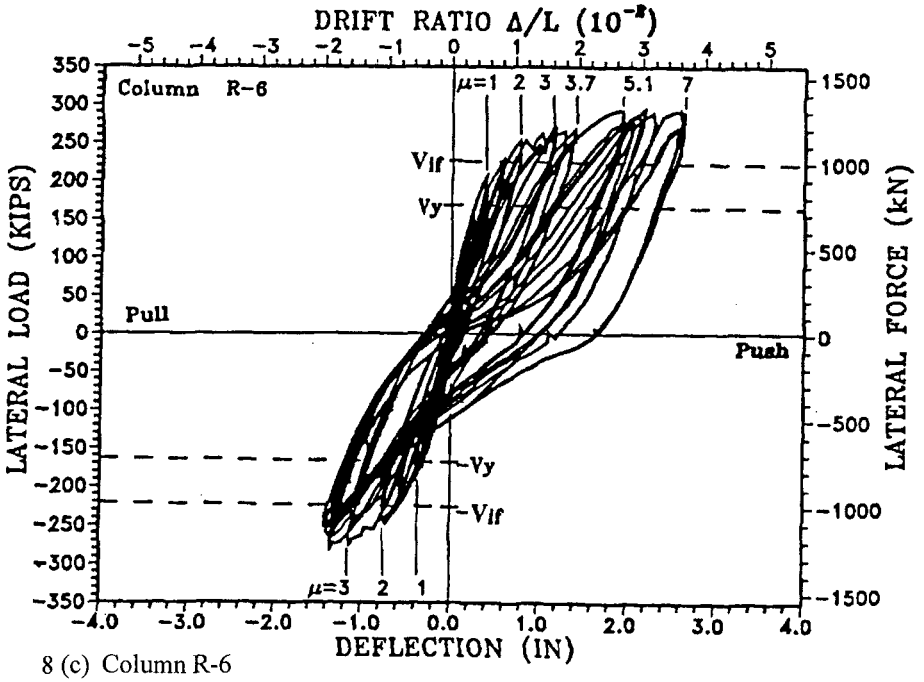
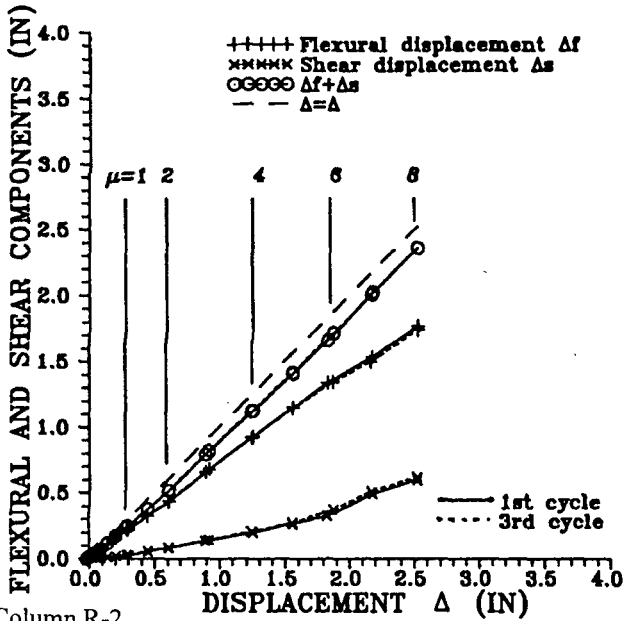
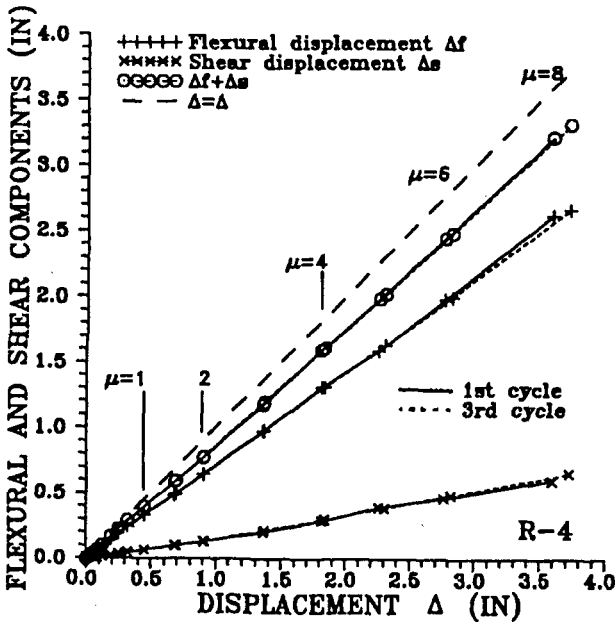


Figure 8. Overall Force-Deformation Results for Retrofit Columns R-2, R-4 and R-6

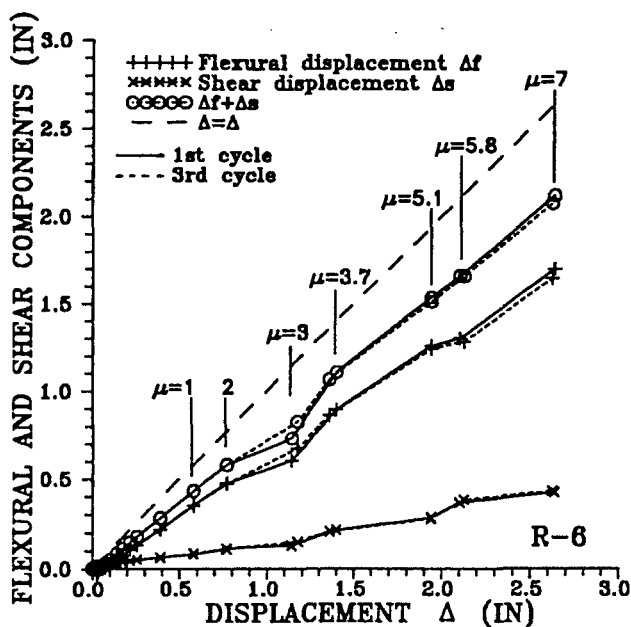


9 (a) Column R-2



9 (b) Column R-4

Figure 9 continued on the next page.



9 (c) Column R-6

Figure 9. Peak Flexural and Shear Deformations for Retrofit Columns R-2, R-4 and R-6

Seismic Behavior Predictions of Structures: A Local Nonlinear Mechanisms Approach

by F. Ragueneau and J. Mazars

Synopsis: This paper deals with the analysis of reinforced concrete structures subjected to seismic loading. The expressions of two constitutive relations based on damage mechanics are exposed. Physical behavior such as crack closure and frictional sliding are introduced at the local level and their influences towards structural computations (global damping) are exemplified by comparisons with experimental data.

Keywords: concrete structures; constitutive equations; seismic; simplified analysis

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INTRODUCTION

For civil engineering structures requiring large-scale computations, simplified analyses based on a multi-layered beam kinematics of a cross section are adopted. Such a pragmatic computational tool at the structural level allows (in terms of computational costs) for an enhancement of the material behavior descriptions at the local level. Physical mechanisms of non-linearities in both concrete and steel are described and introduced in the analysis through constitutive equations expressed within the framework of irreversible processes thermodynamics. Model parameters are identified for the representative volume element. This pertinent and physical description of the different sources of non-linearities greatly improves the structural analysis and above all, the modeling prediction ability for structures subjected to severe loadings such as seismic actions.

The failure mode of concrete specimen is related to the cracks nucleation and growth. Great attention must be paid to an accurate description of the effects of such discontinuity surfaces on the macroscopic response of the material. Recent shaking tables experimental programs on a concrete structure model pointed out the major influence of the crack-closure on the variation of the vertical load during an earthquake and of the frictional sliding of the crack surfaces towards structural damping. The objectives of this paper is to expose the main material features which have to be taken into account at the local level to deal with complex response of strongly loaded reinforced concrete structures. Based on a macroscopic description, damage mechanics allows to account for cracks, fundamentally discrete items, in a continuous framework, the finite element method. Different damage models accounting for unilateral effect and frictional sliding are described and computations show the major role played by these refined mechanisms on the global response. Such a local and physical approach towards local nonlinear mechanism modeling is exemplified through a structural case study: a building model subjected to seismic actions.

MATHEMATICAL TOOLS

Constitutive Relations

In this part, we briefly sum up two kinds of local material constitutive laws for concrete used in the following applications.

Scalar Damage Model with Unilateral Effect—Initially introduced by Kachanov (1) for creep failure problems, the damage mechanics formulation requires the addition of a new internal variable in order to represent the macroscopic loss of stiffness (2). This can be achieved in many ways. The classical one is to relate the damaged material's and the intact material's elastic properties. The most natural idea would be to think of the damage operator as an eighth-order tensor. But its identification is difficult. Defining a damage variable requires reducing the rank of damage operator while maintaining as well as can be the experimental properties of the material. In order to reach this objective, the effective stress concept is introduced. It induces a relation between the stress tensor and the effective stress tensor through a strain equivalence (the effective stress which, when applied to the undamaged material, produces the same strain). Therefore, in the basic form, a scalar variable can be used for the sake of simplicity. Numerous authors have proposed several expressions of different ranks for the damage variable. The fourth-order tensor was suggested by Chaboche. Second-order tensors are more frequently introduced (3). In this case, a problem exists in that the symmetry of the elasticity operator is no longer insured (4). Cordebois (5) postulated a state potential which depends only on the effective stress. Therefore, the symmetry of the elasticity operator is obtained through this energy equivalence.

Seismic loading, which includes cyclic aspects, produces microcracking in concrete. Some major phenomena have to be taken into account:

- decrease in material stiffness as the microcracks open,
- stiffness recovery as crack closure occurs,
- inelastic strains concomitant to damage.

To simulate this behavior we use a damage model (6), which incorporates two scalar damage variables, one for damage due to tension D_1 , the other for damage due to compression D_2 and which includes a recovery stiffness procedure and the description of isotropic inelastic strain. The Gibbs free energy is expressed as follows:

$$\chi = \frac{\langle \sigma \rangle_+ : \langle \sigma \rangle_+}{2E(1-D_1)} + \frac{\langle \sigma \rangle_- : \langle \sigma \rangle_-}{2E(1-D_2)} + \frac{\nu}{E} (\sigma : \sigma - Tr(\sigma^2)) + \frac{\beta_1 D_1}{E(1-D_1)} f(\sigma) + \frac{\beta_2 D_2}{E(1-D_2)} Tr(\sigma) \quad (1)$$

The total strain is: $\epsilon = \epsilon^e + \epsilon^{in}$