

Composite and Hybrid Systems

Editors: Riyad S. Aboutaha
Joseph M. Bracci

SP-196

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Composite and Hybrid Systems

Editors

Riyad S. Aboutaha

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Dedicated to a leader in Engineering Practice and Education - Walter P. Moore, Jr.

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Cover Photo: "Relocating beam plastic hinge region by use of headed bars."
Courtesy of Riyadh S. Aboutaha

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PREFACE

This special publication is in memory of the late professor Walter P. Moore, Jr., a pioneer structural engineer in composite systems, and a leading structural engineering educator. The first paper in this publication documents his professional career and highlights his contributions in the design and construction of composite systems.

In 1998, ACI Committee 335, Composite and Hybrid Systems, sponsored four technical sessions in Houston, Texas, and Los Angeles, California. The papers presented in these sessions covered a wide range of practical case studies and research projects on steel-concrete and polymer-concrete composite systems. These papers should be of interest to practicing engineers, educators, and researchers in that they demonstrate the effective use of different materials for the construction of composite structures.

Bahram Shahrooz was chairman of ACI Committee 335 when this venture was proposed, and at the time of the sessions. Riyad S. Aboutaha, Joseph M. Bracci, Gajanan M. Sabnis, and Sam X. Yao served as moderators for these sessions.

Acknowledgments

The editors would like to thank the authors and presenters for their contributions to the four technical sessions and this volume, the reviewers of the original manuscripts for their constructive comments and suggestions, and the ACI staff for their great help in organizing the sessions and preparing this special publication. Special thanks are also due to the members of ACI Committee 335 for their support of this project.

Riyad S. Aboutaha
Joseph M. Bracci
Editors

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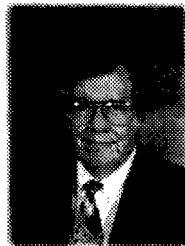
In Tribute to Walter P. Moore, Jr.

A Leader in Composite Building Design and Engineering Education

by Lee W. Slade, P.E.

Walter P. Moore, Jr., Ph.D., PE, NAE, was an internationally-respected structural engineer and educator who left a deep imprint on the way that major buildings are designed and on the way that students are educated to design them. He designed a broad spectrum of major buildings throughout the United States and was an industry leader in the incorporation of composite technologies into important structures. He advocated the combination of structural steel and reinforced concrete into composite elements that were superior in performance, economy, and constructability. Dr. Moore also taught a generation of designers how to collaborate and embrace new technologies and new materials.

For 40 years Walter P. Moore, Jr. was associated with Walter P. Moore and Associates, Inc., the Houston-based structural and civil engineering practice that his father established in 1931. As President and Chairman of the firm from 1975 until 1995, Dr. Moore expanded the horizons of the firm and helped it establish a reputation as a national leader in the design of high-rise buildings and sports facilities. Dr. Moore personally designed a number of award-winning buildings including the soaring steel cantilevered amphitheater at Miller Outdoor Theater in Houston's Hermann Park.



He led teams that delivered a succession of high-profile structures, including the crowned 60-story headquarters tower of NationsBank in uptown Charlotte, North Carolina, at completion the third tallest concrete frame building in the world.

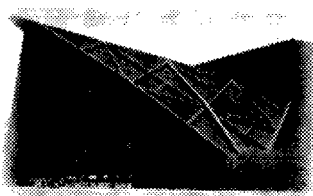
Dr. Moore's success was built on his core beliefs.

He believed in life-long learning. He felt that the most innovative and meaningful contribution came from those with a strong foundation of technical training, overlain with practical experience, input from industry, and continued curiosity. He earned his Ph.D. in structural engineering from the University of Illinois at Urbana after earning his Bachelors and Master of Science degrees at Rice University. His commitment to learning continued throughout his life as he served as an adjunct professor at Rice, teaching courses in both theoretical structural design and pragmatic practice issues. He served on engineering curriculum advisory boards at Rice as well as Texas A&M University, working to keep both schools vital and connected to the business of engineering. In January, 1994, he pursued his passion for teaching on a full time basis with his appointment as Professor in the College of Architecture and Department of Civil Engineering at Texas A&M University in College Station, Texas. His teaching responsibilities included coursework in both the architectural and civil engineering curricula. In addition, Dr. Moore served as Director of The Center for Building Design

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and Construction and The Center for construction Education. These appointments, which also began in January of 1994, were sponsored jointly by the College of Architecture and Department of Civil Engineering. In this dual role, Dr. Moore helped forge a stronger link between academia and practice by establishing a forum to examine issues central to the building design and construction industry.

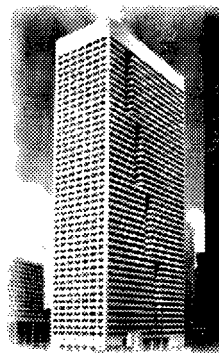
Walter P. Moore, Jr. simultaneously held the Thomas A. Bullock Endowed Chair in Leadership and Innovation at Texas A&M. This chair was established to further activities that promote leadership, management, and innovation in the design and construction industries.



He believed in collaboration, especially between engineers and architects. He enjoyed the creative tension that often exists between the creative and aesthetic focus of architects and the more technical, pragmatic focus of engineers. He often stated that engineers and architects are linked together, and that both are stronger when they work together. His design work with a wide variety of talented architects bore this out.

He believed in practical innovation. The structural engineer's role in building design, according to Dr. Moore, is to creatively find the most appropriate structural solution for the specific challenge, without a prejudice for a particular material or construction technique. He started every project with an open mind regarding the most appropriate structural system. As a result, he and his firm developed a broad range of expertise in designing buildings in structural steel, reinforced concrete, precast concrete, timber, and composite mixes of steel and concrete. In each case, his willingness to continue to learn, and be influenced by the needs of the construction industry led him to stretch the envelope of structural design innovation.

Never was this approach more intensely and effectively applied than during the oil-fueled construction frenzy of Houston from the mid 1970s through the early 1980s. During this period, conditions were ripe to encourage rapid development of composite blends of structural steel and reinforced concrete. Developers demanded that their buildings be distinctive in form and façade, and needed them to be completed within time frames that grew increasingly shorter as interest rates rose. Tenants grew to expect broad, column-free lease spaces, with sky lobbies and increasingly open window vistas. Architects responded with signature building footprints, soaring building lobbies, and articulated building facades. A premium was placed on speed of construction and structural economy to allow the available construction dollars to be spent on high quality finishes in memorable ground floor lobbies, expensive stone façade cladding, and luxurious elevator cabs - not on the hidden structural systems.



Though the plentiful materials and labor market in Houston had generally made reinforced concrete the preferred structural system for most buildings of size, Walter P. Moore, Jr. was

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among the first to explore alternative solutions as the buildings grew taller. Schedules left no time for the backshoring typically used in reinforced concrete towers. Further, the increasing height of the buildings demanded that they be lighter and lighter atop the soft alluvial soils of Houston. The blended systems pioneered by Walter P. Moore, Jr. and his firm responded to this set of design conditions. These buildings were real-life applications of his core beliefs.

One and Two Houston Center are 46- and 42-story high-rise office buildings, which anchor the Houston Center development in downtown Houston. For their design in the mid-1970s, Dr. Moore utilized composite columns to speed construction. **First City Tower** is a striking 49-story office building notable for three huge vertical notches in the exterior façade that precluded a conventional tube wind frame. Dr. Moore's firm instead utilized an innovative lateral system that featured composite stub girders linked with shear walls. **One Atlantic Center**, a 50-story tower in mid-town Atlanta, utilized composite perimeter columns around a slip-formed core wall which was the tallest in the world at the time of completion.



For **312 Walnut** in Cincinnati, Ohio, Dr. Moore's firm utilized composite shear walls to laterally brace the 35-story tower, the first major building to be designed using the new Load and Resistant Factor Design (LRFD) processes. Each of these pioneering applications of composite technology was recognized in the industry with one or more structural design awards.



Walter P. Moore passionately believed that engineers should be leaders, and contribute to the profession and society in ways beyond just good design. He personally was a contributor and leader of numerous professional organizations and technical societies. He chaired ASCE's Code Committees ASCE 7-93 and ASCE 7-95 and was an active member of ACI's Committee 318, (Standard Building Code), Committee 335 (Composite and Hybrid Structures), and the Technology Transfer Committee, among others. Dr. Moore also participated heavily in the steel industry organizations, including the National Specifications Committee for over a decade.



His career body of work was recognized in a number of important ways. He was elected to the National Academy of Engineering and named Fellow of the American Society of Civil Engineers as well as honorary member of the American Institute of Architects. In 1992 the American Concrete Institute recognized his contributions to the design of concrete buildings with their Alfred E. Lindau Award. Two years earlier he chaired the Fourth World Congress on Tall Buildings held by the Council on Tall Building and Urban Habitat.

Academia honored him as well, with both Rice University and the University of Illinois giving him their Distinguished Engineering Alumni Award. In 1995, Rice University

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recognized his contributions in fields outside of engineering with their Distinguished Alumni Award.

Walter P. Moore, Jr.'s career in consulting engineering and education was cut tragically short in 1998 when he was involved in an automobile accident. He sustained injuries to which he eventually succumbed on June 21, 1998. He left Mary Ann (Dillingham) Moore, his lifelong wife plus three grown children and four grandchildren. Thanks in large part to his foresight in putting in place an effective leadership and ownership transition, his firm, Walter P. Moore and Associates, remains vibrant and retains a position as a national leader in innovation and designers of composite structures.

Walter P. Moore, Jr. is greatly missed, though his work lives on in the accomplishments of the hundreds of engineers, architects, and educators whom he influenced during his career.

Walter P. Moore, Jr., (1937-1998), lived his life with passion, wit, style, integrity, and "ingenuity".

SP 196-1

Analytical Modeling of Through Beam Connection Detail

by A. Elremaily and A. Azizinamini

Synopsis: The “Through beam connection detail” has been identified as an ideal rigid connection for attaching steel beams to concrete filled tube (CFT) columns. A combination of analytical and experimental studies is being conducted to comprehend the behavior of this detail. The test specimen consisted of a CFT column and a steel beam passed through the column to represent an interior joint in a building. This paper presents a summary of the finite element analysis that was conducted to comprehend the force transfer mechanism and identify locations of potential stress concentration. The analytical results were verified by comparison with the experimental results. Both the experimental and analytical results showed the capability of the connection to develop the full plastic bending strength of the connected beam. The elements that contribute to the connection strength were identified as: the beam web, the steel tube, and the concrete core.

Keywords: composite structures; concrete filled tubes; connections; finite element

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Atorod Azizinamini is an Associate Professor of Civil Engineering and the director of the National Bridge Research Organization (NaBRO) at the University of Nebraska-Lincoln. He is also the chairman of the Steel Bridge committee of the American Society of Civil Engineers and is a member of several other committees related to bridge engineering.

INTRODUCTION

Composite construction consisting of concrete filled tube (CFT) columns with relatively thin-walled steel tubes, has been used in building construction in U.S. and far east. In general, in this type of construction, steel beams are framed to columns at each floor level. Detail and design criteria for connecting steel beams to CFT columns are almost non-existent. On the other hand the economy of this type of connection depends, to a large degree, on utilizing a suitable connection detail. The overall objectives of the investigations being conducted are to develop an economical connection detail for connecting steel beams to CFT columns and to provide accompanying design provisions.

A previous pilot study on a specific connection detail referred to as a through beam connection detail was conducted at University of Nebraska-Lincoln by Azizinamini and Parakash (2). Another study by Alostaz and Schneider (3) was recently completed at University of Illinois. This study examined the feasibility of using several connection details and concluded that the behavior of the through connection detail was the best.

A combination of analytical and experimental studies is being conducted to provide a thorough understanding of the behavior of the through beam connection detail. A three dimensional nonlinear finite element model was generated for the connection using ANSYS (1) finite element program. The experimental results were used to verify and fine tune the numerical model. The theoretical model was then used to identify the elements that control the joint behavior and evaluate their contributions to the connection strength.

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TEST SPECIMEN

A cruciform shaped specimen representing an interior connection in a plane frame was tested to understand the force transfer mechanism. Details of the test specimen are shown in Figure 1. The specimen consisted of a concrete filled tube column and a steel beam passed through the column. The upper and lower portions of this cruciform type test specimen represent the distance from the floor to mid-heights of the lower and upper stories. The length of the beam represents the distance to the inflection point on each side, which is assumed to be half the span. Loads were applied at the beam ends to simulate member shears at inflection points which occur under lateral loading of the frame. Axial load was applied on the column to represent the reaction of upper stories due to gravity loads. The magnitude of the axial load was taken as 20% of the squash load, P_o , given by the following equation:

$$P_o = A_s F_y + A_c f'_c$$

where

- A_s = cross sectional area of the steel tube
- A_c = cross sectional area of the concrete core
- f'_c = concrete compressive strength
- F_y = yield stress of the steel tube

The specimen is approximately 2/3-scale compared to the member sizes needed for the prototype building. The specimen had a 16"φx1/4" pipe and a W18x35 beam. The measured yield stresses of the tube and the beam are 64 ksi and 52 ksi, respectively. The measured concrete compressive strength is 5.0 ksi. Full penetration weld was used to attach the beam to the pipe. The total height of the column was 9 ft. Lateral reaction points on the column were located 6 in. away from the column ends resulting in a distance between the lateral reaction points of 8 ft. The total beam length between the loading points for all the specimens was 13 ft-6 in. The loading points were located 1 ft away from the beam end. The total length of the beam was 15 ft-6 in.

The specimen had a column-to-beam bending capacity ratio of 1.2, which satisfies the code requirement that the column should be at least 20% stronger than the beam. Failure of the specimen was due to plastic hinging in the beam outside the joint.

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GEOMETRICAL MODEL AND SOLUTION METHOD

Figure 2 shows the finite element mesh. The X-axis aligned with the beam longitudinal axis, the Z-axis aligned with the column longitudinal axis, and the Y-axis was perpendicular to both longitudinal axes of the column and girder. Due to symmetry in the in the X-Z plane, one half of the connection was modeled. Reaction points were located on the center lines perpendicular to the plane of loading at the top and the bottom of the column. Reaction points at the column top were restrained from translation in the X and Y directions, but the displacement along the Z-direction was not restrained to allow for the application of the column axial load. Reaction points at the bottom of the column were restrained from translation in the X, Y, and Z directions.

To ensure uniform distribution of axial loads at the column ends, the model included rigid steel cap plates. These steel cap plates were located at column ends using shell elements. The column axial load was applied in the form of uniform pressure on the rigid caps. The column axial load was applied first. Then, the girder was loaded monotonically in small increments. Full Newton-Raphson procedure with line search and automatic time stepping was used for the analysis. The frontal solver method was used to solve the simultaneous equations.

MATERIAL MODELS

Since concrete is a brittle material it tends to develop tensile fracture perpendicular to the direction of the largest tensile strain. Under uniaxial compression, tensile cracks develop on planes parallel to the direction of the compressive stresses. Under triaxial compressive stresses, the mode of failure is crushing of concrete.

The concrete material model in ANSYS accounts for both cracking and crushing. The cracking is modeled through an adjustment of the material properties which effectively treats the cracking as a smeared band of cracks rather than discrete cracks. The presence of a crack at an integration point is represented through modification of the stress-strain relations by introducing a plane of weakness in a direction normal to the crack face. Also, a shear transfer coefficient is introduced which represents a shear strength reduction factor for those subsequent loads which induce sliding (shear) across the crack face. If the crack closes, then all compressive stresses normal to the crack plane are transmitted across the crack. Since cracked concrete cannot transfer tensile stresses, tensile strength should drop suddenly after cracking. To avoid numerical problems, tensile strength right after cracking was assumed to drop

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to 99% of its original value. Tensile strength was then gradually decreased with crack widening until it reached a value of zero.

If the material at an integration point fails in uniaxial, biaxial, or triaxial compression, the material is assumed to crush at that point. Crushing is defined as the complete deterioration of the structural integrity of the material (e.g. material spalling). Under conditions where crushing has occurred, material strength is assumed to have degraded to an extent such that the contribution to the stiffness of an element at the integration point in question is ignored.

The tensile strength was assumed to be $7.5\sqrt{f'_c}$ psi. The ratio of the ultimate biaxial compressive stress to the ultimate uniaxial compressive stress was specified as 1.2.

A rate-independent plasticity model was used to simulate the inelastic behavior of the steel components. Von Mises' yield criteria was used to define the material yield surface, and an associated flow rule was used to determine the plastic deformation. The associated flow rule with the von Mises yield criterion is known as Prandtl-Reuss flow equation.

A bilinear kinematic hardening model was used to simulate the elastic and inelastic behavior of all steel components. Kinematic hardening assumes that the yield surface remains constant in size and the surface translates in stress space with progressive yielding. Material properties measured from testing coupons obtained from the test specimen were used in the finite element model.

ELEMENTS

Eight-node brick elements (SOLID65 of ANSYS) with three translational degrees of freedom at each node were used to model the concrete core. This element allows cracking in three orthogonal directions at each integration point. Four-node shell elements (SHELL43 of ANSYS) with six degrees of freedom at each node were used to model the steel tube and the girder. For both elements extra displacement shapes were excluded to help accelerate convergence.

The interface between the steel and concrete was modeled by using contact elements (CONTAC49 in ANSYS). The contact element can transfer compressive and frictional forces only. The coefficient of friction between steel and concrete was assumed to be 0.3. The weld was idealized by providing common nodes for the girder and the steel tube at the weld locations.

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FINITE ELEMENT RESULTS

In order to verify the analytical model, the results from the finite element (FE) analysis are compared to the experimental data. Figure 3 shows the experimental and analytical load-deflection curves at the beam end. A good agreement between the experimental and analytical results is observed. This indicates the capability of the FE model to predict the behavior of the joint in both the linear and the nonlinear stages. Also shown in Figure 3 is the load at the cantilever tip (P_p) that is required to develop the plastic moment capacity of the beam. Both the analytical and experimental loads exceeded P_p , which indicates the capability of the connection to develop the plastic flexural capacity of the beam.

Figure 4 shows the Von Mises stresses in the steel tube and the steel beam. Stress concentrations are found at the intersection of the beam flange and the tube. The distribution of the hoop stress in the steel tube at the tension flange level is shown in Figure 5. This figure shows that high hoop stresses are transferred to the tube at the tip of the tension flange.

The variation of the axial stress in the flange along the beam longitudinal axis is shown in Figure 6. It is noticed that there is a significant stress drop in the beam flange once it enters the tube. Due to the curved shape of the tube the stress drop at the flange edges is higher than at the flange centerline. The axial stress in the tension flange just inside the tube is approximately 70% of the stress value just outside the tube. This indicates that a portion of axial tensile force in the flange is transferred to the tube as shear force through the weld. This behavior agrees with the experimental results which indicated an axial strain drop in the flange, once the beam enters the tube. At the compression side, the stress drops almost to zero once it enters the tube. This indicates that axial compression force in the flange is transferred to the concrete core by bearing of the tube wall against the concrete. The same behavior was also observed experimentally. The gradual reduction in the axial tensile stresses in the flange, inside the tube, is due to transfer of the flange force to the concrete core by friction.

The minimum principal stresses in the concrete core are shown in Figure 7. High vertical and horizontal bearing stresses are generated in the concrete panel near the compression flange. The resultant of these forces causes the formation of the diagonal compression strut which contributes to the joint shear capacity. The formation of the compression strut is confirmed by the distribution of the minimum principal stress in the concrete panel as shown in Figure 7.

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The tensile and compressive forces in the beam flanges are transferred as horizontal shear forces through the connection zone. The connection zone is defined as the portion of the column bounded by the beam top and bottom flange levels. The concrete in the joint zone is divided into inner and outer concrete panels. The inner concrete panel consists of the concrete between the beam flanges while the outer concrete panel is the concrete outside the flanges. The joint shear is distributed between the beam web, the tube, the inner concrete panel and the outer concrete panel. Figure 8 shows the amount of shear resisted by each of the four elements. Figure 8 shows that all the elements are activated from the beginning of the loading. The web, however, seems to pick up more forces than other elements until it reaches its capacity. Figure 8 shows that the web yielded at a load of 47 kips. After web yielding, the increase in the shear force is distributed between the tube and the concrete. Figure 8 also indicates that the shear capacity of the tube and the concrete panel was not fully mobilized since the specimen failed by beam yielding outside the joint.

CONCLUSIONS

The through beam connection detail provides a good detail that is capable of developing the plastic bending capacity of the connected beam when the strong column-weak beam philosophy is followed. The tensile and compressive forces in the beam flanges are transferred as horizontal shear forces through the joint zone. The joint shear is distributed between the beam web, the tube, the inner concrete panel and the outer concrete panel. In order to design the connection, the shear strength of each element needs to be established. Experimental studies are being conducted on test specimens that were designed to fail in the joint zone in order to establish the ultimate strength of the connection.

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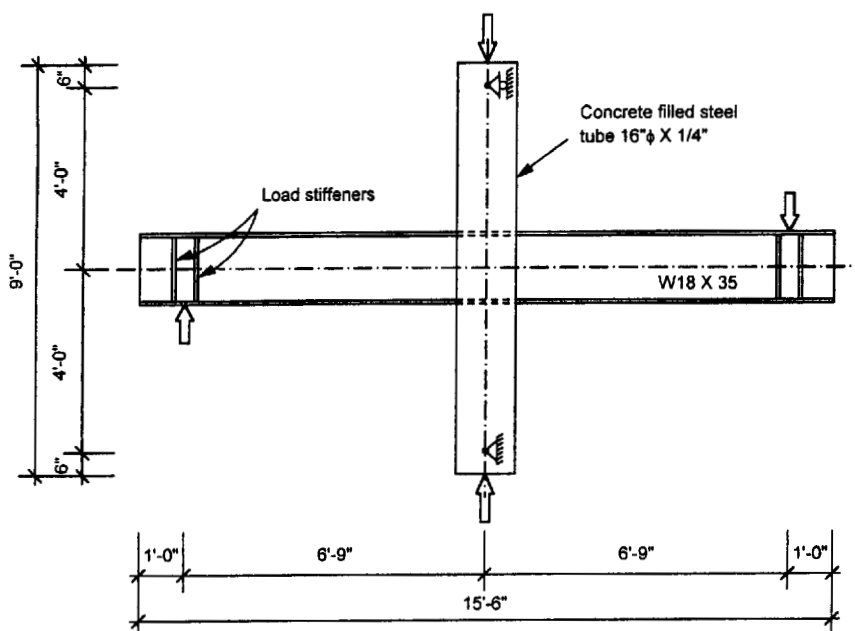


Figure 1 Test Specimen

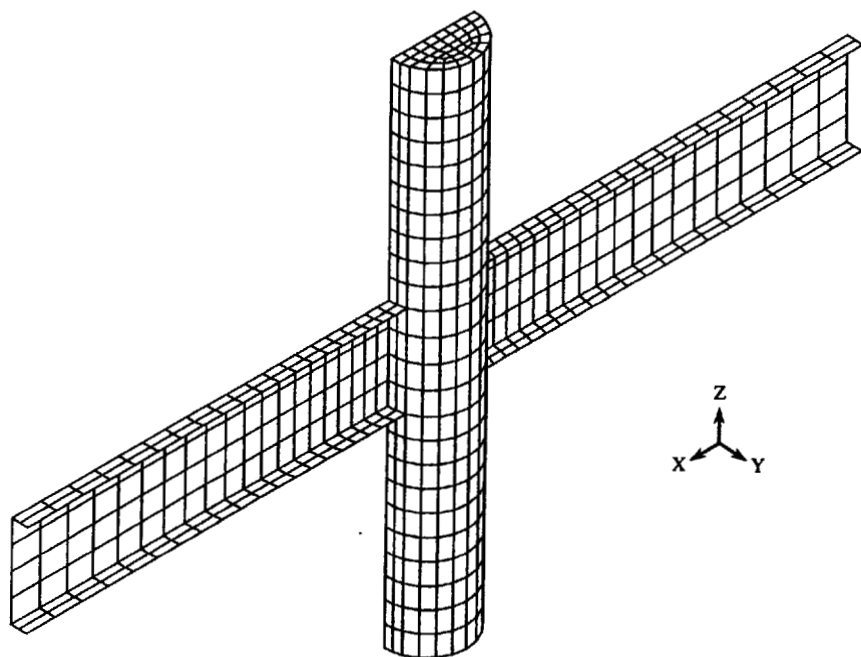


Figure 2 Finite Element Mesh

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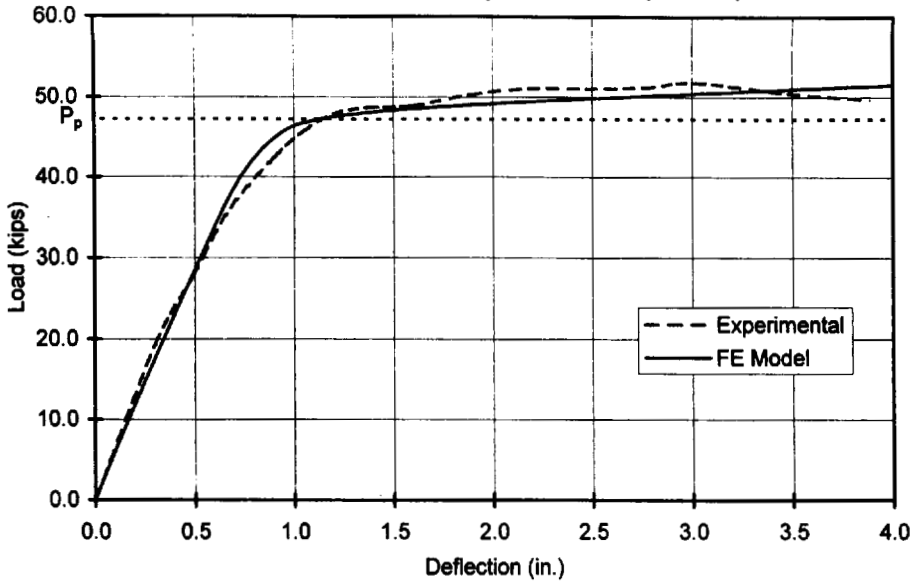


Figure 3 Load-Deflection Relationship at Beam End

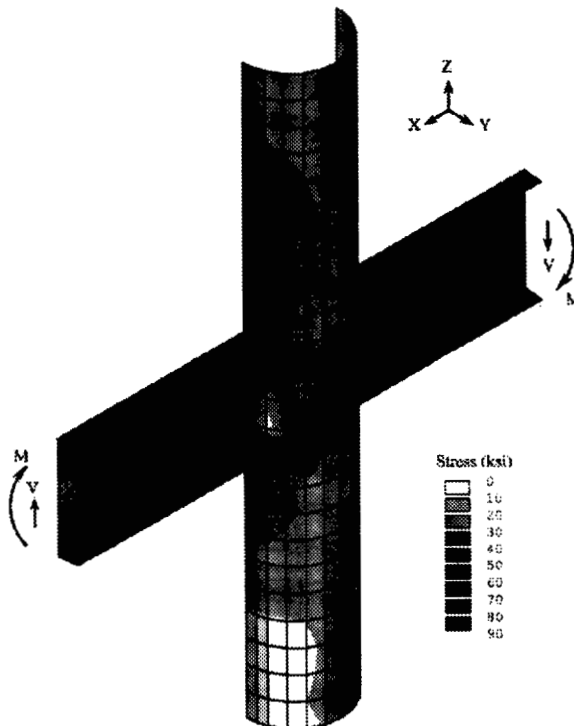


Figure 4 Von Mises Stress in the Steel Tube and Beam

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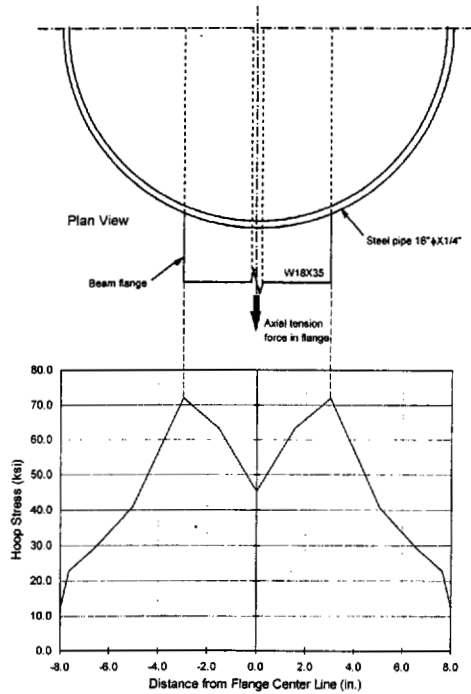


Figure 5 Hoop Stress Distribution at the Tension Flange Level

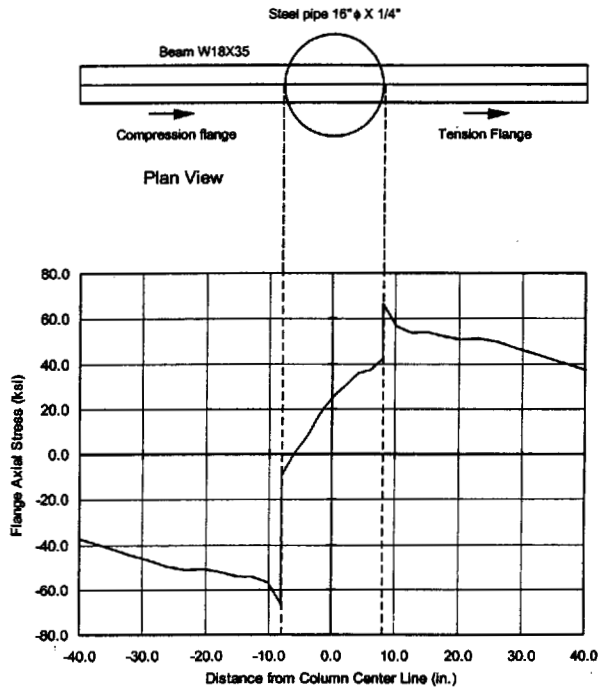


Figure 6 Variation of Axial Stress in Top Flange along the Beam Axis

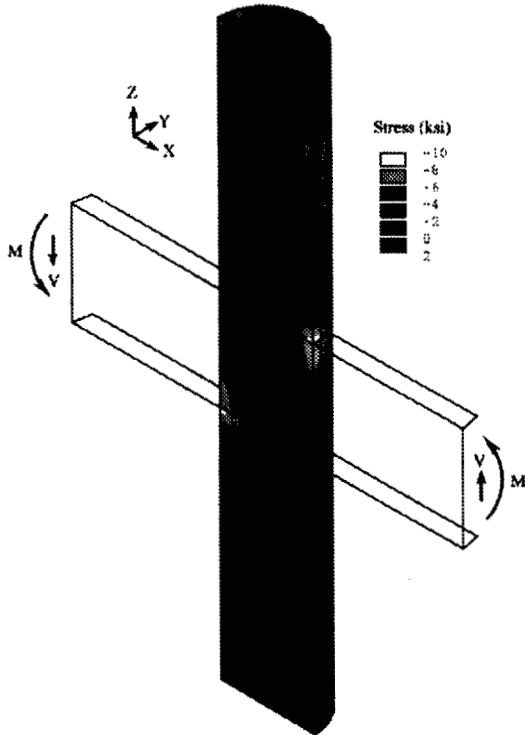


Figure 7 Minimum Principal Stress in Concrete Core

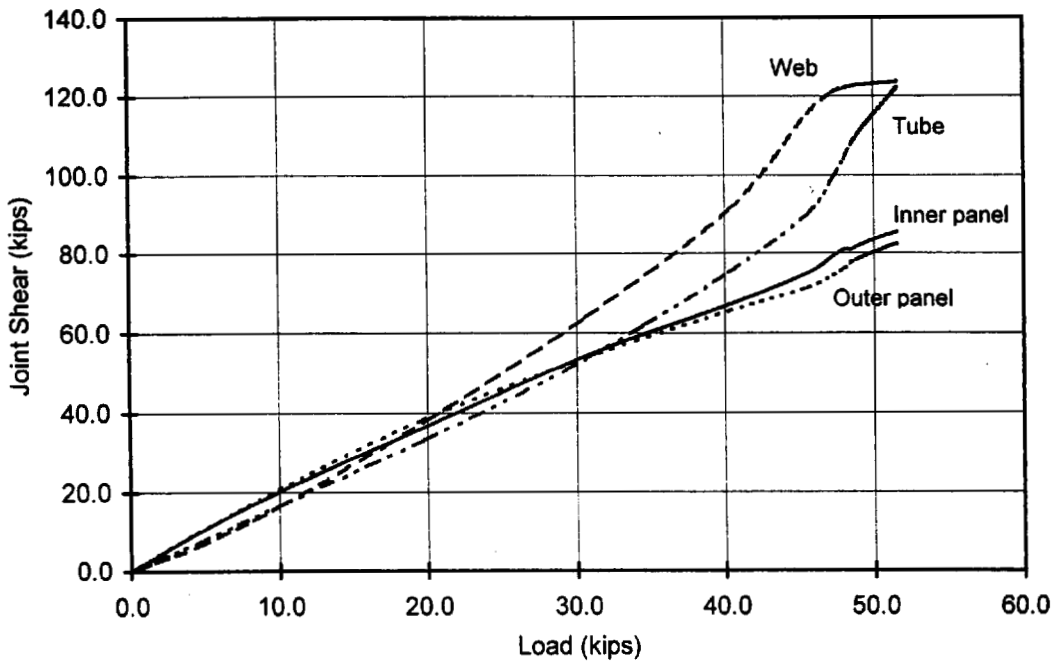


Figure 8 Joint Shear Force Distribution

SP 196-2**Advanced Composites for the Navy
Waterfront Infrastructure****by L. J. Malvar**

Synopsis: Since 1992, the U.S. Navy, Naval Facilities Engineering Service Center (NFESC), has been involved in the study and use of advanced fiber reinforced plastic (FRP) composites for its waterfront infrastructure applications. Carbon FRP (CFRP) and glass (GFRP) composites were assessed and demonstrated at NFESC and at various sites throughout the Navy. CFRP sheets were used to upgrade reinforced concrete beams, slabs and piles. GFRP shells were also used for pile upgrade. Some of these components were reinforced and prestressed with CFRP rods and tendons. Finally, other composites, such as epoxy coatings for rebars and engineered wood were addressed. Long term durability issues are still being investigated. This paper represents an overview of the assessment and use of FRP materials in Navy reinforced concrete waterfront structures.

Keywords: carbon fiber; composites; fiber-reinforced plastics, infrastructure; reinforced concrete; structural upgrade

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Dr. L. Javier Malvar is a Research Materials/Structural Engineer at the Naval Facilities Engineering Service Center. He has been involved with composite materials for infrastructure applications since 1992.

INTRODUCTION

Since 1992, the U.S. Navy, Naval Facilities Engineering Service Center (NFESC), has been involved in the study and use of advanced fiber reinforced plastic (FRP) composites for its waterfront infrastructure applications. Advanced composites presented significant durability advantages for waterfront construction when compared to traditional materials, such as wood and steel. However, an understanding of these materials, and their short and long-term characteristics, was necessary to optimize their use in the areas where their properties made them best suited. The present paper shows a summary of development, test & evaluation, and application of these materials within the Navy infrastructure.

BACKGROUND

In fiscal year 1991 critical U.S. Navy waterfront deficiencies amounted to a total of \$1.3 billion, of which \$613 million were related to pier upgrades and repairs, and an estimated \$219 million were needed for new construction. About 75% of all Navy piers and wharves were over 40 years old and required increased repair and maintenance. Most of the deficiencies were due to corrosion, in particular to corrosion of steel reinforcement. To prevent this corrosion, the use of galvanized and epoxy-coated reinforcing bars (rebars) was investigated. A recent alternative was the use of corrosion-resistant fiber reinforced plastic (FRP) components including reinforcing bars, prestressing tendons, structural shapes, and unidirectional or woven fabrics. All of these were already commercially available but were used with restraint due to the limited knowledge of their long-term behavior and the lack of uniform codes or design guidelines. In the meantime research was advancing rapidly and several demonstration structures had been constructed in Europe. In Japan, a government-sponsored national research project on the use of new materials in construction was focusing on the use of continuous fiber products in concrete construction.

In 1992 the Naval Facilities Engineering Service Center (NFESC) started investigating the potential of these new materials for Navy waterfront infrastructure applications. FRP reinforcing bars for concrete reinforcement were first evaluated.

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In 1994 NFESC established the Advanced Waterfront Technology Test Site (AWTTS), to enable U.S. researchers to demonstrate their state-of-the-art composite applications on a waterfront reinforced concrete structure. This structure represents a typical Navy pier designed to withstand the highest existing Navy crane loads. It uses FRP materials as structural members or components. A parallel effort involved the assessment of structural upgrades using carbon FRP sheets.

In 1995 all components of the AWTTS were completed. Prestressing technology using graphite tendons was demonstrated in pile and deck construction. The feasibility of all-composite structural components was proven. Mechanical properties of FRP reinforcing bars had been evaluated. Structural upgrade using CFRP sheets was very successful and was scheduled for full-scale field applications. A first compendium was published in December 1995 covering research results through that date [1-12].

In 1996 half scale tests of composite upgrades were completed on the AWTTS reinforced concrete decks. Composite upgrades of operating piers at NAVSTA Norfolk, San Diego, and Pearl Harbor, were started. Additional work was completed in the areas of composite fender piles, composite hangers, and epoxy coatings for rebars. This work was included in the second compendium published in December 1996 [13-22].

The two first composite upgrades were completed in 1997 (NAVSTA Norfolk) and 1998 (NAVSTA San Diego). These efforts indicated that the mechanical characterization and use of these materials had matured. However, it became apparent that long-term characterization was still inadequate, and the 1998 technological development effort concentrated on durability issues. The latest compendium of publications gives an overview of NFESC's technology in advanced composites for infrastructure applications for calendar years 1997 and 1998 [23-30].

COMPONENT UPGRADE

Piles

In its seismic retrofit program, the California Department of Transportation (CALTRANS) has been a leader in assessing and allowing the use of FRP composite materials for column wrapping [31]. These wraps have been shown to significantly enhance the columns' ductility during seismic events. In the Navy, several of the same wraps were evaluated and used for upgrading piles, most notably at the pile to pile cap connection, where the pile would typically lose its concrete cover by crushing [24]. Among the types tested and used: a wet carbon FRP lay-up with underwater curing resin, which will retain the original appearance of the pile, and various prefabricated cylindrical glass shells which are placed around the pile, the gap between shell and pile being filled with concrete grout. Either system was shown to properly confine

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the pile concrete, allowing for increased ductility of the pile to pile cap connection [24] (Figure 1). Pile wraps have also shown potential application to the upgrade of structures to resist blast effects [32].

Beams

Some of the original work in this area was started in Switzerland [33]. NFESC carried out some testing to verify that both bending and shear enhancements were possible, and that the overall energy dissipation was also enhanced [3,8]. Satisfactory shear enhancement was obtained by wrapping CFRP sheets on 3 faces only (sides and bottom).

Slabs

Navy pier decks are characterized by reinforced concrete slabs over pile caps, with spans of about 20 ft (6.1 m). Although the FRP wraps were known to increase the bending capacity, the mode of failure for Navy slabs is punching shear under the high concentrated loads from mobile crane outrigger pads (up to 240 kips over 30 by 30 inch areas) (that is 1070 kN over 76 by 76 cm areas). Laboratory 1/5 and 1/2 scale tests showed that carbon FRP (CFRP) wraps were able to enhance the punching shear strength as well, by up to 50% [1,15] (Figure 2). Lately, carbon rods, glued into grooves in the top deck, were used to resist the negative moment due to live loads [24]. Upgraded slabs were then subjected to full-scale tests to demonstrate their upgraded capacity (Figure 3).

NEW CONSTRUCTION

Piles

Over the last few years, several FRP piles have entered the market, both for fendering and bearing applications. NFESC has tested fender piles of various types: (1) recycled plastic with glass FRP reinforcing rods, (2) pultruded glass or carbon shells (filled or not with concrete), (3) filament wound and scrimp shells, (4) FRP structural shapes (Figure 4). Some Navy activities have adopted some of these piles for secondary fendering, the main fendering typically consisting of foam-filled fenders resting against partially prestressed reinforced concrete piles, also developed at NCEL [34]. Some prestressed concrete bearing piles were also constructed at the South Dakota School of Mines and Technology using CFRP tendons and spirals, and driven in the Port Hueneme harbor [7,9].

Slabs

Standard Navy 18-inch deep concrete slabs prestressed exclusively with CFRP tendons were used in a demonstration pier deck in 1995 [7,9]. Ongoing test & evaluation is determining the suitability of CFRP tendons for reinforcement purposes by assessing their bond capacity. An all-composite deck

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composed of glass FRP tubular sections was completed [7,9], and single large pultruded sections are currently being investigated.

Pier Concepts

Under a recent project, NFESC is looking at the feasibility of a new generation of piers that would take even more advantage of the new materials. Concepts being investigated include double-deck pile-supported and floating piers, incorporating CFRP prestressed/reinforced decks and hulls, as well as pultruded FRP sections and appurtenances.

DURABILITY ISSUES

Sustained Stresses

Glass FRP (GFRP) composites are not being used in applications where sustained, long term, or cyclic stresses are present. GFRP are appropriate for pile wrapping, where the composites are subjected only to occasional large live loads, but not so much for deck retrofits where dead loads, or highly cyclic loads, can be present. Concrete beams reinforced with single CFRP sheets and loaded to 50% of the CFRP strength have been exposed to the inter-tidal zone in the Hueneme harbor for about 4 years without failure.

Adhesives

The long term durability of the resins used to apply FRP composites to concrete is still being investigated. This is especially important for deck retrofits, where the whole retrofit is dependent on the adhesion of the CFRP to the concrete. Short-term effects of temperature, humidity, and chloride content were addressed [26,27]. Long-term freeze-thaw effects showed only a slight effect on CFRP load carrying capacity after 1000 cycles between -30°F and 130°F [24].

ADVANCED COMPOSITES FOR CONVENTIONAL MATERIALS

Epoxy Coatings

While epoxy coating of steel rebars has been used for many years, sometimes with mixed results, NFESC recently participated in the development of ASTM A934 epoxy coatings which show better adhesion (they are fusion bonded), have less defects and exhibit increased UV resistance. ASTM A934 epoxy coatings have been used recently in the Ford Island Bridge in Hawaii [29], as well as other commercial applications [29]. Coated and uncoated bars should not be used side by side.

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Engineered Wood

Within this category, the Navy is looking into materials composed of at least 50% wood flour by weight. The remainder of the mixture includes either high density polyethylene (HDPE) or polyvinylchloride (PVC), and several additives. Moduli of rupture in the order of 3000 psi (21 MPa) and elastic moduli in excess of 500 ksi (3.4 GPa) can be obtained, and these materials are currently being considered for chock, wale and deck replacements. For pile applications, these materials may need to be reinforced, and the inclusion of continuous carbon tows in both pultrusion and extrusion processes is being investigated.

CONCLUSIONS

Advanced composites have already replaced conventional materials in many applications. In the infrastructure, composites are competitive with steel in the upgrade of columns or piles, and provide economical alternatives for structural upgrade of decks. It is expected that the use of these materials will keep increasing, both for upgrade and new construction applications.

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Figure 1. Retrofitted piles, NAVSTA San Diego.

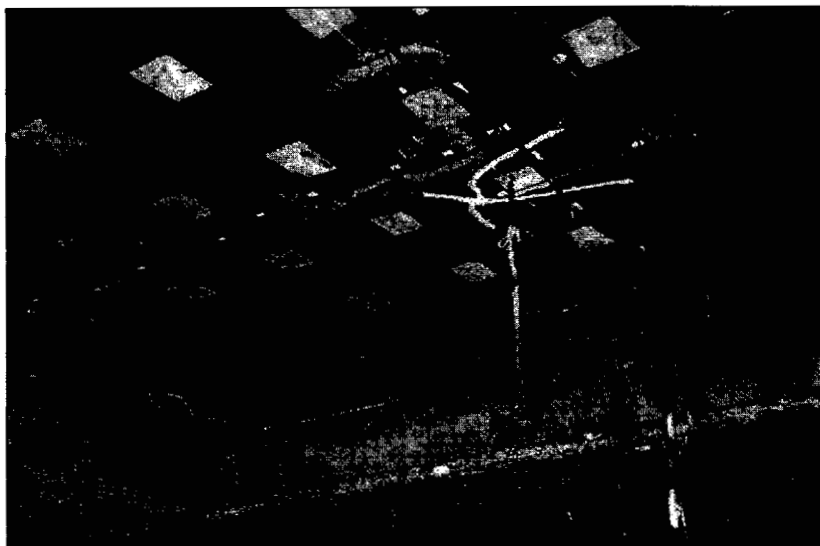


Figure 2. CFRP punching shear retrofit, NAVSTA Norfolk.



Figure 3. Full scale testing of CFRP upgraded section NAVSTA San Diego.

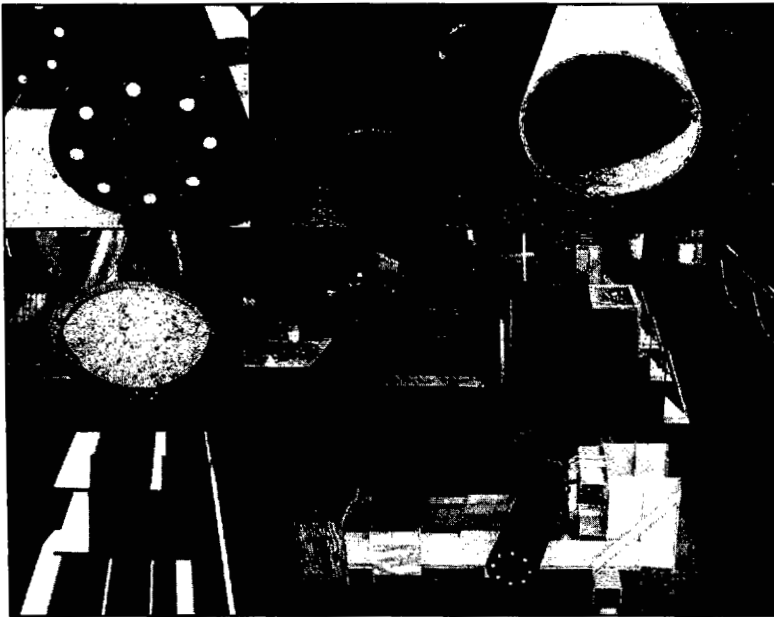


Figure 4. Fender pile testing.

SP 196-3**Comparison of ACI, AISC, and Basic
Methods for CFTs****by B. M. Shahrooz and W. Zhang****Synopsis**

Using experimental data from previous tests and detailed analytical studies, the applicability of ACI and AISC standard techniques for concrete-filled tubular columns (CFTs) is evaluated. The test specimens include short and slender CFTs made with normal and high strength steel tubes filled with normal and high strength concrete. The focus of this paper is on rectangular and square tubes. To gauge the success of the code-based methods, the capacities are also computed by the fiber analysis techniques, along with a member level iteration algorithm for analyzing members with significant length.

The results indicate that the ACI and AISC methods can yield substantially different capacities. In general, the capacities from the ACI method are reasonably close to those obtained from detailed analytical methods so long as normal strength tubes are used. Both the ACI moment magnifier method and AISC method are appropriate for slender CFTs although the ACI method tends to match the analytically calculated capacities more closely. Neither the ACI nor AISC method is applicable for CFTs made with high strength steel tubes as both techniques substantially underestimate the capacity of such columns. For CFTs with high strength steel tubes, it is more appropriate to assume that the steel tube fully yields when the capacity is developed. A revised version of the ACI standard method was developed by incorporating this assumption. The revised ACI method provides a fairly close estimate of the experimentally obtained capacities and those from detailed analysis.

Keywords: columns; high-strength concrete; tests

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INTRODUCTION

The use of concrete-filled tubular columns (CFTs) in high-rise buildings has become more popular in recent years as they provide several advantages over reinforced concrete or steel columns, e.g. formwork and reinforcement are eliminated, high strength and stiffness can be achieved by filling high strength tubes with high strength concrete, concrete is confined rather easily by the steel tube, thinner steel tubes can be used as the concrete improves the stability of the tube, etc. Despite such advantages, unified design codes do not currently exist in the United States, and designers need to choose between the methods available in ACI Building Code (ACI 318-95) and AISC Load and Resistance Factor Design (LRFD, 1994). Using available experimental data and results from relatively detailed analytical studies, these two methods are gauged and required adjustments are recommended. The focus of this paper is on square or rectangular short and long CFTs made with normal or high strength steel and concrete.

SUMMARY OF DESIGN METHODS

(a) ACI Method

Following ACI 318-95, the moment strength is taken as that corresponding to developing a compressive strain of 0.003 in the concrete. The steel is assumed to follow an elastic-perfectly plastic stress-strain relationship, and a uniform stress specified in ACI 318-95 is used for the concrete as shown in Figure 1. However, the ultimate concrete compressive strength is taken as f'_c instead of $0.85 f'_c$ to reflect that concrete inside tubes does not split and environmental effects

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protected. Previous studies by Furlong (1967) and others (Roik and Bergmann, 1992) also suggest that f'_c is more appropriate for CFTs. Following the basic principles, the closed-form expressions summarized in Figure 2 are developed. Note that the equations assume that $F_y \leq 600$ MPa. Using these equations, axial load and bending moment are computed by varying the depth of the neutral axis (c) and using the appropriate set of equations. Hence, the axial load-moment interaction diagram is generated.

For slender CFTs, the moment-magnifier method is followed. The capacity is established by finding the intersection of the loading line (the relationship between the axial load and bending moment for a given eccentricity) from Eq. 1 with the axial load-moment interaction diagram.

$$M = \frac{M_2}{1 - \frac{P}{0.75 P_c}} \quad (1)$$

In this equation, the Euler buckling load (P_c) is computed based on $EI=0.20E_cI_g + E_sI_t$. The radius of gyration (r) is computed from Eq. 2.

$$r = \sqrt{\frac{0.2 E_c I_g + E_s I_t}{0.2 E_c A_g + E_s A_t}} \quad (2)$$

where A_g , I_g , A_t , and I_t are the cross-sectional area and moment of inertia of the concrete infill and steel tube, respectively.

(b) AISC Method

In contrast to ACI, AISC prescribes the interaction diagram shown in Figure 3. Standard equations are followed to calculate the key points of the interaction diagram, i.e., $P_n = A_s F_{cr}$ and $M_n = Z F_y$. However, the equivalent terms summarized in Eq. 3 for the yield strength and modulus of elasticity are to be used. (Note that the expressions in Eq. 3 are based on AISC equations for encased composite columns but the term associated with the area of longitudinal bars drops out as CFTs do not typically have longitudinal reinforcing bars.) As implied by Eq. 3, the AISC method is based on $0.85f'_c$ and not f'_c as assumed for the ACI method.

$$\begin{aligned} F_{my} &= F_y + 0.85 f'_c \left(\frac{A_c}{A_s} \right) \\ E_m &= E_s + 0.4 E_c \left(\frac{A_c}{A_s} \right) \end{aligned} \quad (3)$$

The first-order moment and second-order moment, if applicable, are taken into account similar to the techniques used for steel columns.

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SUMMARY OF ANALYTICAL TECHNIQUES

The capacity of short columns is computed based on the moment-curvature response which is established by fiber analysis. In fiber analysis, a section is divided into a sufficiently large number of layers. Knowing concrete and steel stress-strain relationships, compatibility and equilibrium requirements of the fibers are used to establish cross-sectional responses such as moment-curvature relationships. In this study, slip at the concrete-steel interface was ignored, and accordingly a linear strain distribution through the section was assumed. A cubic equation proposed by Saenz (1964) was used to model the post-yield behavior of steel. The behavior of confined concrete inside steel tubes was represented by a model proposed by Tomii (1991). As seen from Figure 4, the ascending part of the stress-strain curve up to a strain of 0.002 is represented by a parabolic equation. In the reported study, the relationship proposed by Hognestad (1951) was implemented. The post-peak response, beyond concrete strain of 0.005, depends on B/t as indicated in Figure 4. A linear interpolation may be used for values of B/t other than those shown. The concrete is assumed to reach a constant level of stress after strain of 0.015.

For slender columns, the second-order moments are taken into account by numerical integration of analytically derived moment-curvature response in conjunction with iterative second-order analysis procedure. The procedure is described below, and is shown schematically in Figure 5 for one particular loading arrangement. (Note that the basic procedure is applicable to other cases with different loading arrangement and boundary conditions. In this method, a member is divided into N segments.)

- Divide the member into N segments. The length of each segment is typically identical so that the resulting nodes are spaced equally.
- For a given load increment, the following steps are followed.
 - Step a: Ignoring second-order moments, obtain the values of moments at each node, i.e., $M_1, M_2, \dots, M_i, \dots$
 - Step b: Generate the member moment-curvature relationship for a given axial load. For each moment $M_1, M_2, \dots, M_i, \dots$, obtain the corresponding curvatures $\phi_1, \phi_2, \dots, \phi_i, \dots$ based on the $M-\phi$ relationship.
 - Step c: Integrate the curvature distribution from step (b) to compute rotation at each node.
 - Step d: Integrate the curvature distribution from step (b) twice to compute transverse displacement at each node.
 - Step e: Incorporate second-order moments. Add the primary and

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second-order moments to compute the total moment at each node. Repeat steps (b) and (c) with the revised moments computed in this step.

- Step f: Repeat step (d) with the revised curvature distribution obtained in step (e). If the displacements from this step and step (d) are within a specified tolerance, a solution has been obtained for this load step. Otherwise, go to step (e) with the displacements found in this step.
- Increase the applied load and repeat steps (a) to (f). These steps are followed until the full load-deflection and/or moment-rotation relationships are constructed.

Using the aforementioned method, the load-deflection curves for three concrete-filled tubular columns (Zhang and Shahrooz, 1999) were calculated. Three specimens were tested as shown in Figure 6. The main test variable was the level of imposed axial load which was varied between 0 and $0.29P_o$. The calculated applied load-midspan deflection for the specimen with axial load equal to $0.18P_o$ is shown in Figure 7. A reasonably good correlation of the experimental data is evident. A similar trend was observed for the other two specimens (Zhang and Shahrooz, 1999). The strength is matched very closely. The computed response tends to lose stiffness at a higher load. This behavior is attributed to softening and hysteretic behavior of steel and concrete due to the load-unload cycles which were not taken into account in the analyses. Therefore, the outlined iterative method can reliably incorporate material and geometric nonlinearity of slender CFTs.

SUMMARY OF EXPERIMENTAL PROGRAM

The experimental data from a relatively large number of specimens were considered in this study. The specimens included those tested by Bridge (1976), Furlong (1967 and 1968), Knowles and Park (1969 and 1970), Shakir-Khalil and Zeghiche (1989), Tomii and Sakino (1979), researchers at the Building Contractors Society (BCS) and Building Research Institute (BRI) in Japan (Fujimoto et al., 1995), and Zhang and Shahrooz (1997).

As evident from Table 1, the test specimens covered CFTs made with normal strength tubes (yield strength less than 600 MPa) or high strength tubes (yield strength above 600 MPa) filled with normal concrete (compressive strength less than 46 MPa) or high strength concrete (compressive strength above 46 MPa). The tubes were relatively stocky; the maximum tube width to thickness ratio (B/t) was 50. Most of the specimens meet the minimum tube thickness as required by

ACI or AISC, i.e., $B = \sqrt{\frac{F_y}{3E_s}}$. The thickness of the tubes for BRI specimens

ER6-D-4-10 and ER6-D-4-30 was 65% of that required by ACI or AISC. Local

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buckling of the steel tubes was not reported for these specimens prior to developing the capacities reported in Table 1 (Fujimoto et al., 1995). The length of the specimens tested by Bridge (1976), Knowles and Park (1969 and 1970), Shakir-Khalil and Zeghiche (1989), and Zhang and Shahrooz (1997) was significant, i.e., these columns would be classified as slender. The remaining test specimens, on the other hand, provided data regarding the behavior and capacity of short CFTs. The specimens in the database were tested differently as seen from Figure 8 in which the various test setups are shown schematically.

EVALUATION OF DESIGN METHODS

The ratios of the measured to computed capacity, as obtained from the ACI and AISC methods as well as from detailed analytical method, are plotted in Figures 9 and 10 for the short and slender test specimens, respectively. The classification of the specimens into short and slender columns is based on ACI criteria. The test setup was similar for all the specimens except for those tested by Tomii and Sakino (1979) and those tested at BCS (Fujimoto et al., 1995). The calculations based on the AISC method were, therefore, identical for the short or slender specimens except for Tomii-Sakino's and BCS's specimens. In all the calculations, the strength reduction factors recommended by ACI or AISC were taken as 1 because (a) the material properties, load levels, and loading conditions were known, i.e., there was no uncertainty about the parameters affecting the strength; and (b) the main purpose of the study was to examine the basic differences between the methods and not the reliability of the methods as influenced by the reliability indexes used in ACI or AISC. The success of each method is gauged for the short and slender columns separately in the following paragraphs.

(a) Short CFTs

The ACI method provides a very good estimate of the measured strength of the specimens tested by Tomii and Sakino. The results from the ACI method are generally comparable to those calculated from fiber analysis which match the measured capacities very closely. For the same specimens, the AISC procedure is not as successful. Neither ACI nor AISC method is as successful in calculating the capacities measured by Furlong (1967). Fiber model offers a slightly improved result although the computed capacities tend to be larger than the measured values for most of the specimens. These differences are attributed to the fact that the reported material properties for a number of the specimens were apparently the nominal and not measured values. For the specimens tested at BRI, the results as computed by the ACI method are reasonable, and generally are similar to those obtained from fiber analysis. The AISC procedure, on the other hand, tends to consistently and appreciably overestimate the capacity. It is unclear why the AISC method is unconservative for the specimens tested at BRI and BCS. A similar trend has been observed by others (Aho and Leon, 1997). Note that for some of the specimens (those in ER6 series), the measured capacities are generally higher than their counterparts computed by the ACI method.

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specimens, high strength steel tubes ($F_y = 618 \text{ MPa}$) had been used, and the results suggest that current design methods need to be adjusted. The correlation of the data for the specimens tested at BCS is particularly bad. While the ACI method tends to underestimate the capacities, the AISC method overestimates the capacities. Such large underestimation of the measured capacities by the ACI method was not observed for the specimens tested by Tomii-Sakino, Furlong, and most of those tested at BRI. As summarized in Table 1, the BCS tubes had a significantly higher yield strength than those used in the other test programs. This observation also confirms that current design methods have to be modified for CFTs made with high strength steel tubes.

On the average, both ACI and AISC methods produce comparable values (the average value of the measured to computed capacity is 1.169 and 1.174 for the AISC and ACI method, respectively). However, the scatter of the results from the AISC method is significantly higher than that for the ACI method (the standard deviation of the measured to computed capacity is 0.672 for the AISC method versus 0.330 for the ACI method). Both methods are clearly inappropriate for cases in which high strength steel tubes are used, and need to be adjusted as described later in this paper. The results show that fiber analysis can reasonably replicate the measured capacities.

(b) Slender Columns

The ratio of the measured to computed capacity is plotted in Figure 10. The ACI method tends to overestimate the reported capacities; on the other hand, the capacities from AISC are smaller than the measured values. While the capacities from ACI are, on the average, about 8% larger than the measured values, the AISC method calculates capacities which are, on the average, about 8% less than the experimental data. The scatter of the computed capacities from either method is essentially the same.

The capacities were also computed from detailed second-order analysis. Due to unavailability of measured post-yield material properties, the steel was assumed to follow an elastic-perfectly plastic stress-strain relationship for the specimens tested by Bridge (1976), Shakir-Khalil and Zeghiche (1989), and Knowles and Park (1969 and 1970). It is unclear why the measured capacities are less than their analytical counterparts. Note that the analytical method was much more successful for the specimens tested by the authors for which the complete material properties were available.

The results as computed by the ACI procedure are fairly close to those computed from detailed techniques. The capacities from the ACI and AISC method are, on the average, 99% and 116% of the corresponding values from the second order analysis. In view of the aforementioned results, it appears that the ACI moment magnifier technique can reasonably provide a simple and reliable estimate of capacity of slender CFTs. The AISC method is also reasonable; however, it tends to underestimate the capacity.

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REVISED METHOD FOR HIGH-STRENGTH STEEL TUBES

The above comparisons and discussions reaffirm that the ACI standard procedure can reliably be used for CFTs so long as normal strength steel tubes are used. However, this method tends to significantly underestimate the capacity of CFTs made with high strength tubes, and is not appropriate. In a revised method, referred to as "revised ACI" herein, the steel tube is assumed to have fully yielded when the cross-sectional capacity is developed, as shown in Figure 11. Similar to the standard assumptions used in the ACI method, strain hardening of the steel is ignored. The necessary equations to construct the interaction diagram based on this method are summarized in Figure 12.

Using this model, the capacities were recalculated. As seen from Figure 9, the revised model does not appreciably change the results for the specimens tested by Tomii-Sakino, Furlong, and BRI specimens, which were made with normal strength tubes. This trend is expected as at concrete strain of 0.003, at which the ACI standard method assumes that the moment capacity is developed, a large portion of a normal strength tube would be at or above yield strain. The distribution of steel stress in the ACI standard method is, therefore, very close to that taken in the revised model, and hence the two methods produce similar results. On the other hand, the measured capacities of the BCS specimens and BRI specimens with high strength steel are correlated more closely with the revised method. Considering the results from all the specimens, the measured capacities are just 2% over those computed by the revised ACI method. The assumption regarding full yielding of the tube significantly enhances the interaction diagram of CFTs with high strength tubes, and hence the moment capacity is calculated more reliably.

SUMMARY AND CONCLUSIONS

Using available experimental data for short and slender concrete-filled tubular columns (CFTs), the success of the ACI and AISC standard techniques for computing the capacity of such composite columns was examined. Columns with normal and high strength steel tubes filled with normal and high strength concrete were considered. For comparison purposes, the capacities were also calculated by reasonably detailed analytical techniques. The calculated capacities from the ACI and AISC standard methods can vary significantly. However, the results from the ACI method tend to match those from the detailed analytical models for short CFTs made with normal strength steel tubes. Either method is reasonable for slender CFTs made with normal strength steel tubes. Neither the ACI method nor the AISC method is applicable for cases in which high strength steel tubes are used. For such CFTs, it is more appropriate to assume that the steel tube fully yields when the capacity is developed. The results based on this assumption are fairly close to those from detailed analysis.

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NOTATIONS

A_c : cross-sectional area of concrete infill;
 A_g : cross-sectional area of steel tube;
 A_s : cross-sectional area of steel tube;
 A_t : cross-sectional area of concrete infill;
 B : column width;
 b : column width;
 c : depth of the neutral axis;
 E_c : concrete modulus of elasticity;
 E_m : modified modulus of elasticity;
 E_s : steel modulus of elasticity;
 e_o : eccentricity of axial load;
 f'_c : concrete compressive strength;
 F_y : steel tube yield strength;
 F_{my} : modified yield strength;
 h : column depth;
 I_g : gross moment of inertia of concrete infill about column centroidal axis;
 I_t : moment of inertia of steel tube about column centroidal axis;
 M : bending moment;
 M_p : plastic moment capacity;
 P : axial load;
 P_n : nominal axial load;
 P_c : Euler buckling load;
 P_o : squash load capacity;
 r : radius of gyration
 t : tube thickness;
 y : transverse displacement;
 Z : plastic section modulus;
 β_1 : ratio of the average concrete compressive strength to the maximum stress as defined by ACI 318-95;
 θ : rotation; and
 ϕ : curvature.

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Table 1. Test Variables

(a) Tomii and Sakino (1979)

Specimen	B (mm.)	t (mm.)	f_c (MPa)	f_y (MPa)	E_s (MPa)	P/P _o
I-0	100	2.29	24.0	194	205945	0
I-1	100	2.29	38.2	194	205945	0.13
I-2	100	2.29	38.2	194	205945	0.27
I-3	100	2.29	38.2	194	205945	0.33
I-5	100	2.29	38.2	194	205945	0.46
I-6	100	2.29	36.7	194	205945	0.58
II-0	100	2.27	21.6	305	208887	0
II-1	100	2.27	21.6	305	208887	0.09
II-2	100	2.20	21.6	339	213790	0.18
II-3	100	2.20	21.6	339	213790	0.26
II-4	100	2.22	21.6	289	215752	0.38
II-5	100	2.22	21.6	289	215752	0.48
II-6	100	2.22	21.6	289	215752	0.57
III-0	100	2.98	20.6	289	210848	0
III-1	100	2.98	20.6	289	210848	0.09
III-2	100	2.98	20.6	289	210848	0.19
III-3	100	2.99	20.6	288	205945	0.28
III-4	100	2.99	20.6	288	205945	0.37
III-5	100	2.99	20.6	288	205945	0.47
III-6	100	2.99	20.6	288	205945	0.56
IV-0	100	4.25	18.6	284	225559	0
IV-1	100	4.25	18.6	284	225559	0.10
IV-2	100	4.25	18.6	284	225559	0.19
IV-3	100	4.25	18.6	285	225559	0.29
IV-4	100	4.25	19.8	285	225559	0.38
IV-5	100	4.25	19.8	285	225559	0.48
IV-6	100	4.26	19.8	288	215752	0.57

Table 1. Test Variables (Cont.)

(b) Furlong (1967)

Specimen	B (mm.)	t (mm.)	f_c (MPa)	f_y (MPa)	E_s (MPa)	P/P _o
F-I-1	127	4.80	44.8	485	199955	0.25
F-I-2	127	4.80	44.8	485	199955	0.38
F-I-3	127	4.80	44.8	485	199955	0.38
F-I-4	127	4.80	44.8	485	199955	0.63
F-II-1	102	2.13	23.4	331	199955	0.18
F-II-2	102	2.13	23.4	331	199955	0.18
F-II-3	102	2.13	23.4	331	199955	0.48
F-II-4	102	2.13	23.4	331	199955	0.74
F-II-5	102	2.13	23.4	331	199955	0.74
F-III-0	102	3.18	28.8	331	199955	0
F-III-1	102	3.18	28.8	331	199955	0.06
F-III-2	102	3.18	28.8	331	199955	0.19
F-III-3	102	3.18	28.8	331	199955	0.19
F-III-4	102	3.18	28.8	331	199955	0.39
F-III-5	102	3.18	28.8	331	199955	0.45
F-III-6	102	3.18	28.8	331	199955	0.45
F-III-7	102	3.18	28.8	331	199955	0.65

Note: The value of E_s was assumed to be 199955 MPa.

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Table 1. Test Variables (Cont.)

(c) BRI (Fujimoto et al., 1995)

Specimen	B (mm.)	t (mm.)	f_c (MPa)	f_y (MPa)	E_s (MPa)	P/P _o
ER4-A-4-4.5	149	4.38	41.1	262	209868	4.5
ER4-A-4-20	149	4.38	41.1	262	209868	20
ER4-C-2-6	216	4.38	25.4	262	216732	6
ER4-C-2-20	216	4.38	25.4	262	216732	20
ER4-C-4-6	216	4.38	41.1	262	216732	6
ER4-C-4-10	216	4.38	41.1	262	216732	10
ER4-C-4-20	216	4.38	41.1	262	216732	20
ER4-C-8-6	216	4.38	80.3	262	216732	6
ER4-C-8-10	216	4.38	80.3	262	216732	0
ER4-D-4-6	324	4.38	41.1	262	213790	6
ER4-D-4-20	324	4.38	41.1	262	213790	20
ER6-A-4-4.5	144	6.36	41.1	618	216732	4.5
ER6-A-4-20	144	6.36	41.1	618	216732	20
ER6-C-2-6	210	6.36	25.4	618	221636	6
ER6-C-4-6	210	6.36	41.1	618	221636	6
ER6-C-4-10	210	6.36	41.1	618	221636	10
ER6-C-4-30*	210	6.36	41.1	618	221636	30
ER6-C-8-6	210	6.36	80.3	618	221636	6
ER6-C-8-20	210	6.36	80.3	618	221636	20
ER6-D-4-10	318	6.36	41.1	618	217713	10
ER6-D-4-30*	318	6.36	41.1	618	217713	30

* Weld between CFT and test frame stub failed before developing the maximum load.

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Table 1. Test Variables (Cont.)

(d) BCS (Fujimoto et al., 1995)

Specimen	B (mm.)	t (mm.)	f_c (MPa)	f_y (MPa)	E_s (MPa)	P/P _o
ER8-A-4-01*	120	6.47	40.5	835	216732	0.08
ER8-C-2-04	174	6.47	25.4	835	218694	0.39
ER8-C-2-06	174	6.47	25.4	835	218694	0.59
ER8-C-4-025*	174	6.47	40.5	835	218694	0.25
ER8-C-4-04	174	6.47	40.5	835	218694	0.40
ER8-C-4-06	174	6.47	40.5	835	218694	0.59
ER8-C-8-04	174	6.47	77.0	835	218694	0.40
ER8-C-8-06	174	6.47	77.0	835	218694	0.60
ER8-D-4-04	264	6.47	40.5	835	216732	0.41
ER8-D-4-06	264	6.47	40.5	835	216732	0.61
ER6-C-2-025	210	6.36	25.4	618	221636	0.25

* Weld between CFT and test frame stub failed before developing the maximum load.

(e) Bridge (1976)

Specimen	B(mm.)	H(mm.)	t (mm.)	L (mm.)	eo (mm.)	f_c (MPa)	f_y (MPa)	E_s (MPa)
B-1	203.7	203.9	9.96	2130	38	30.2	291	205000
B-2	203.3	202.8	10.03	2130	38	34.5	313	205000
B-3	202.8	203.4	9.88	2130	38	33.1	317	205000
B-4	202.6	203.2	10.01	3050	38	37.8	319	205000
B-5	203.2	202.1	9.78	3050	64	32.1	317	205000
B-6	152.5	152.3	6.48	3050	38	35	254	205000
B-7	152.5	152.3	6.48	3050	64	35	254	205000

(f) Shakir-Khalil and Zeghiche (1989)

Specimen	B(mm.)	H(mm.)	t (mm.)	L (mm.)	eo (mm.)	f_c (MPa)	f_y (MPa)	E_s (MPa)
S-1	80	120	5	3210	24	34	386	205000
S-2	80	120	5	3210	60	34	385	205000
S-3	120	80	5	2940	16	37.4	385	205000
S-4	120	80	5	2940	40	36.5	343	205000
S-5	119.888	80.01	5.0038	2938.78	7.874	35.7161	357.161	205000

Table 1. Test Variables (Cont.)

(g) Knowles and Park (1969 and 1970)

Specimen	B(mm.)	H(mm.)	t(mm.)	L(mm.)	eo(mm.)	fc(MPa)	fy(MPa)	Es(MPa)
K-1	76.2	76.2	3.3274	812.8	7.62	41.37	324.065	213745
K-2	76.2	76.2	3.3274	1422.4	7.62	41.37	324.065	213745
K-3	76.2	76.2	3.3274	812.8	25.4	41.37	324.065	213745
K-4	76.2	76.2	3.3274	1422.4	25.4	41.37	324.065	213745

(h) Zhang and Shahrooz (1997)

Specimen	B(mm.)	H(mm.)	t(mm.)	L(mm.)	P/Po	fc(MPa)	fy(MPa)	Es(MPa)
1	256	256	7.685	4724	0.18	41.7	370	158585
2	256	256	7.685	4267	0	41.7	370	158585
3	256	256	7.685	4724	0.29	41.7	370	158585

Note: Average values of B, H, and t are shown in this table.
Distance between the supports is 3657 mm.
L is the distance between the locations at which axial load is applied.

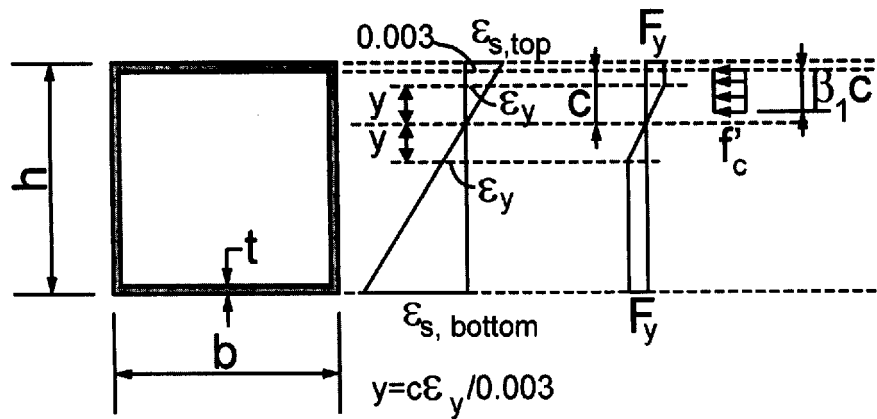


Figure 1. Modeling of CFTs According to ACI Method

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Case 1: $0 \leq c \leq h - 2t$

$$P = \left(t^2 + t(4c - 2th) \right) F_y + \beta_1 c f_c (b - 2t)$$

$$M = \left[-t^2(4c + b) + t(-2c^2 + 2hc - \frac{2}{3}y^2 + bh) \right] F_y + f_c \left[\beta_1^2 c^2 (-0.5b + t) + \beta_1 c (2t^2 - t(h + b) + 0.5hb) \right]$$

Case 2: $c > h - 2t$; $\beta_1 c \leq h - 2t$; $y > h - c - t$

$$P = \frac{F_y}{y} \left[-4t^3 + t^2(1.5b - 4c + 4h) + t(-y^2 - h^2 - c^2 + 2cy + by - bh + cb + 2hc) \right] + \beta_1 c f_c (b - 2t)$$

$$M = \frac{-F_y}{12y} \left[16t^4 - t^3(10b + 24h) + t^2(12h^2 - y^2 - c^2) - 6bc + 6yb + 24yc + 15bh + t(4y^3 - 2h^3 - 4c^3 + 6y^2(h - 2c) + 6c^2(h + 2y) - 6h(bh + by + 2cy - cb)) \right] + \frac{c}{2} f_c \left[c\beta_1^2(2t - b) + \beta_1(4t^2 - 2t(h + b) + bh) \right]$$

Case 3: $c > h - 2t$; $\beta_1 c > h - 2t$; $y > h - c - t$

$$P = \frac{F_y}{y} \left[-4t^3 + t^2(1.5b - 4c + 4h) + t(-y^2 - h^2 - c^2 + 2cy + by - bh + cb + 2hc) \right] + f_c (4t^2 + \beta_1 bc - 2th)$$

$$M = \frac{F_y}{12y} \left[16t^4 - t^3(10b + 24h) + t^2(12h^2 - y^2 - c^2) - 6bc + 6yb + 24yc + 15bh + t(4y^3 - 2h^3 - 4c^3 + 6y^2(h - 2c) + 6c^2(h + 2y) - 6h(bh + by + 2cy - cb)) \right] - \frac{c}{2} f_c \left[bc\beta_1^2 + (2bt - hb)\beta_1 \right]$$

Figure 2. Closed Form Expressions for Axial and Moment per ACI Method

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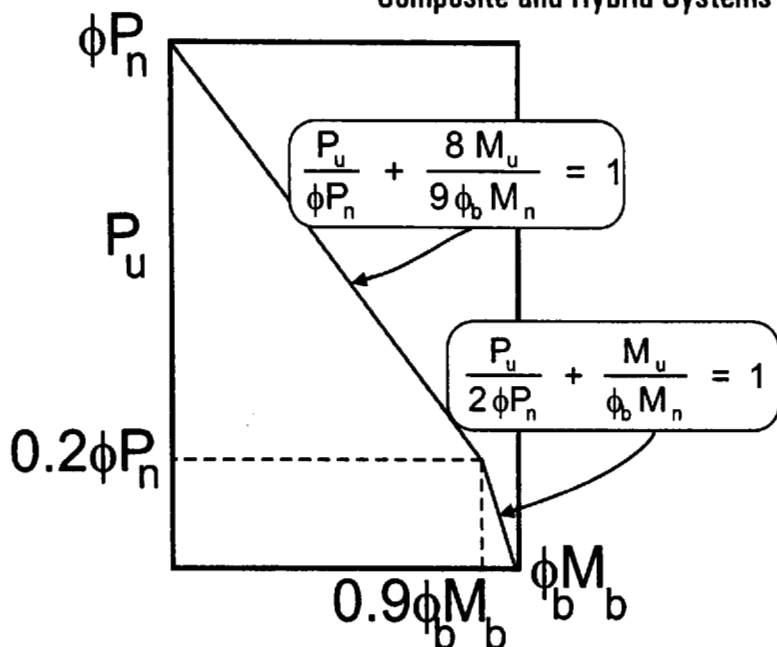


Figure 3. Axial Load-Moment Interaction Diagram per AISC Method

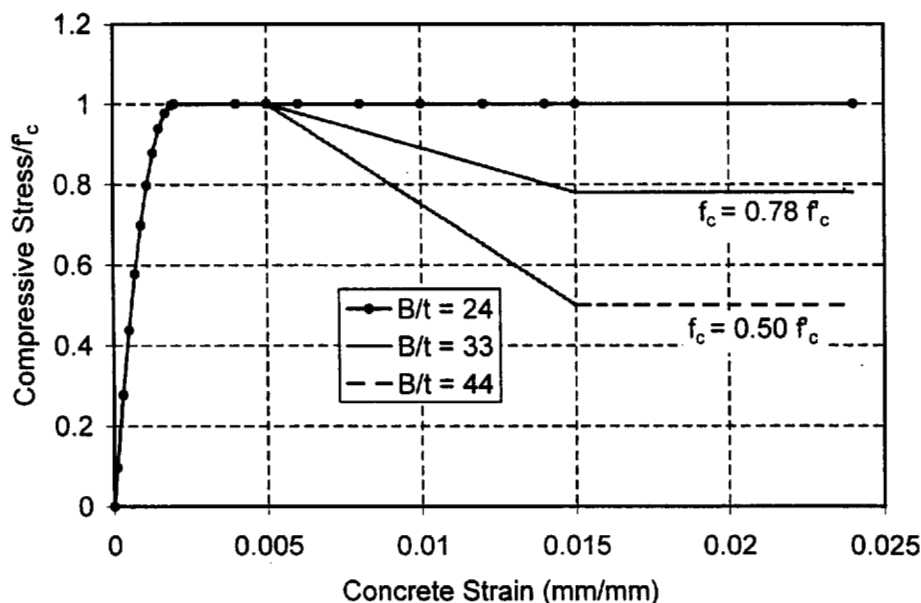


Figure 4. Tomii-Sakino's Concrete Model in Concrete-Filled Tubes

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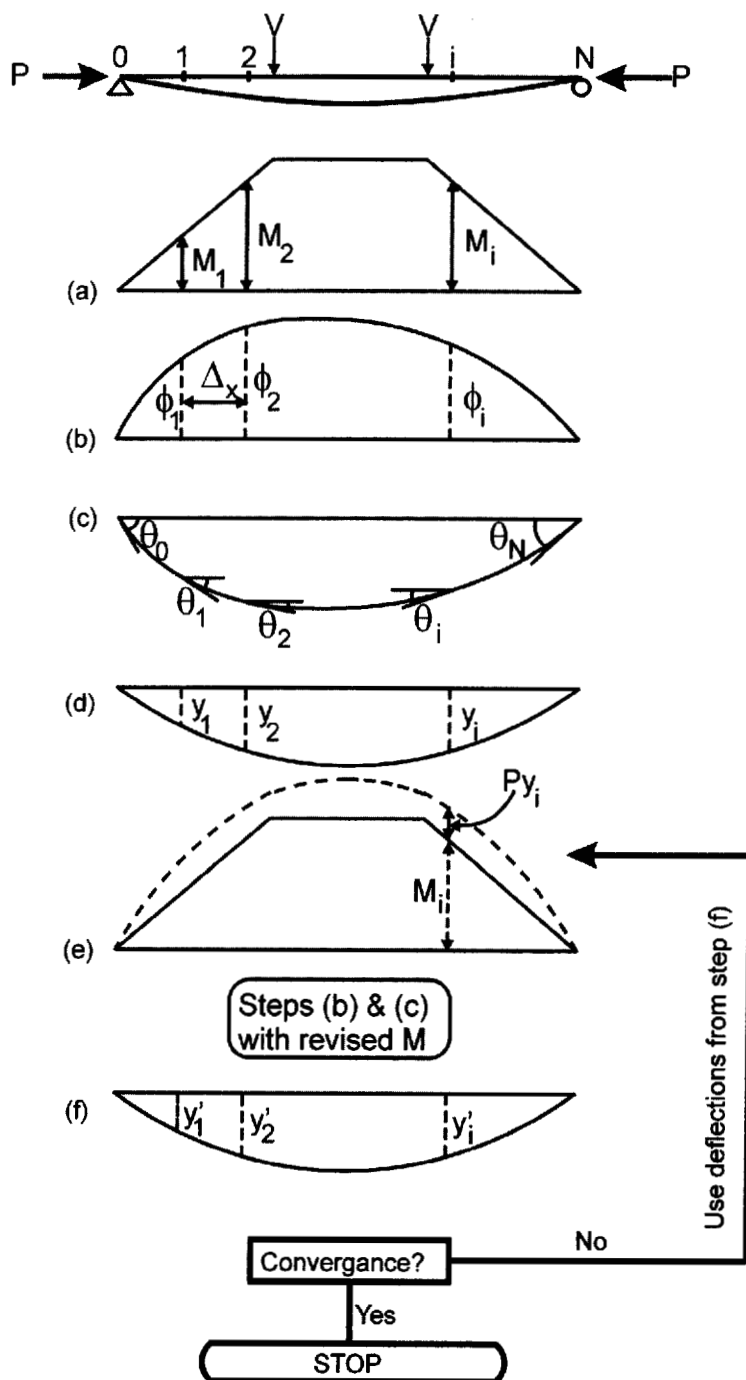


Figure 5. Iterative Solution Technique for Slender Columns

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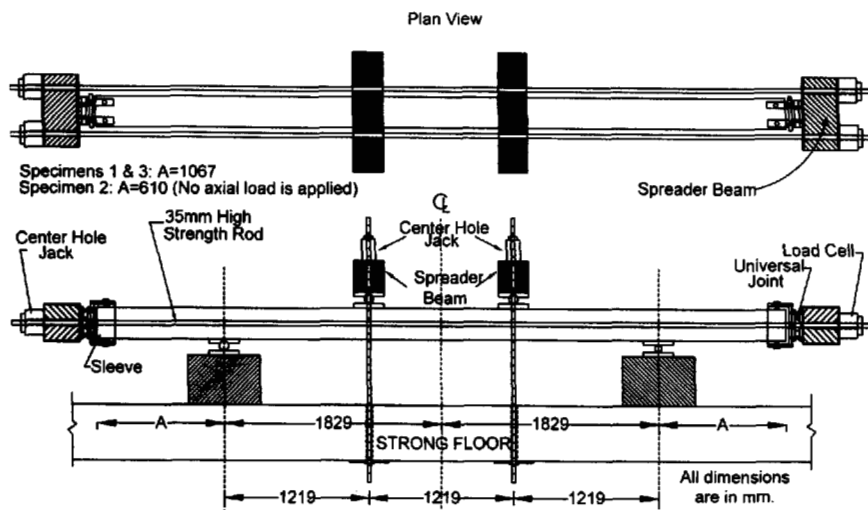


Figure 6. Details of CFT Test Specimens

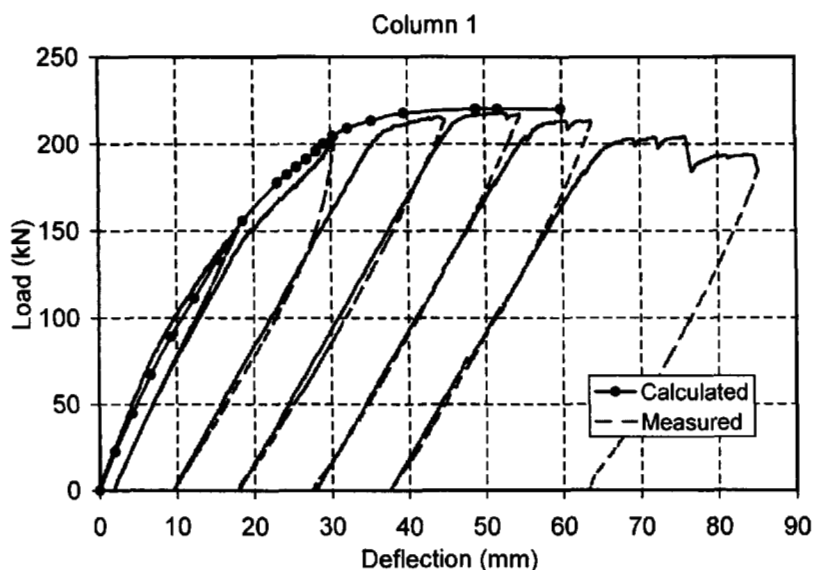


Figure 7. Correlation of CFT Beam Column Specimen