Alexander Scordelis Memorial Session: Thin Shell Concrete Structures

Note: Video is available for the Keynote Lectures by Billington, Meyer and Willam

Session Organizers: Maria GARLOCK (Princeton University), John ABEL (Cornell Univ.)

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Alexander Scordelis: Friend, colleague and mentor
David P. BILLINGTON (Princeton University)

Keynote Lecture and Video
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Christian MEYER (Columbia University)

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Alexander C. Scordelis: Legacy in finite element analysis of reinforced concrete
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Alexander Scordelis: Friend, colleague and mentor

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Abstract

It was at a conference at Berkeley in 1957 that began the transformation of my professional life. Led by T.Y. Lin and devoted to Prestressed Concrete, the real organizational work was done by Alexander Scordelis. Thanks to a curious circumstance I became part of the inner circle at that event and thus met Alex and T.Y. The surprising fact was that I brought to the conference six representatives from the Soviet Union. Because of the so called thaw after Stalin’s death, the new leaders apparently had agreed to allow a small delegation of specialists to attend the Berkeley conference and the organizers wanted someone to meet then in New York, show them around, and accompany the group to San Francisco. They needed a Russian speaking structural engineer coming to the conference and they asked Anton Tedesko to find such a unique person. He knew I was attending and asked my advice. I suggested that between my Russian-fluent brother then a Harvard History Professor and myself, we could provide the New York hosting. We did that, and then I flew with the six Russians to the conference. Because this became a central feature of the events, I was included and thereby got to know Alex for the first time. We quickly became friends that developed soon into a unique intimacy unlike any other in my professional life.

This friendship would gradually come to include a collegiality based upon the common passion for prestressed concrete and thin-shell concrete structures. But most of all and later on after many discussions with Alex did I come to feel his presence in all the unusual activities that I wandered into, a presence that was strongly felt even over the many months that we did not see each other. This became for me the continual experience of being mentored in absentia.

Characteristic of our friendship was the time some ten years after our first meeting when at a conference in West Virginia, we were roommates, and, like boys in a boarding school, in twin beds we talked together well into the dark morning hours. He told me in some detail about his wartime experiences in Europe; he was intensely proud to have served his country and yet there was no hint of bravado. He spoke in a simple narrative manner and he was curious to learn of my past as I was to hear of his. We did not at times like that talk at all about professional life. Moreover our friendship did not depend upon parties, vacations together, ski trips, or even mutual hobbies or cultural events. It was strictly personal with a remarkable sensitivity on his part to learn as much about me as I was discovering about him. But this friendship nearly always coincided with professional meetings.

Although we had first met over prestressing, our closest collaboration came with thin shells. This story begins at the same 1957 Berkeley conference where, for the first time, I met Norman Sollenberger, then Professor of Civil Engineering at Princeton. We were together for much of the conference and that began another lifetime friendship, collegiality, and mentoring. Although unaware of his real purpose, I was to learn soon thereafter that he had been commissioned by his Dean and his chairman to get me into academic life. Three years later, I became a faculty member and in searching for a research director took up thin shells. Alex had done the same and our principal collegiality sprung from that relatively new research area.

In the 1960's we became fellow members of the ACI Committee on Concrete Shell Design and Construction, of which Alex became chairman in the late 1960's. This was a greatly stimulating group of engineers engaging in debating the intellectual value of studying, academic research, design ideas, and the performance of built shells. In 1973 Alex retired as chairman and I was named his successor. Immediately, one key member of the committee resigned, telling Alex that he could not work with me. It was true that we did not get along well together and considering that problem to be as much my fault as his, I was willing to go on without him. But not Alex.

He came to my hotel room and in his quiet, firm way gave me his view of this well known (to the Committee at least) split. He was not angry and he never confronted me with any criticism. I believe he had been the central figure in making me the new chairman, but he was also a close friend of the resigning member, whom I did not relish having to deal with once I learned of his open discontent. It was a long evening, beginning more as a debate and continuing then as a discussion of the committee's future. Alex, I realized later, had diverted the agenda away from argument and on to agreement, not about the unhappy member but about the plans for forthcoming meetings, reports, and conference sessions. This was not crafty manipulation but rather an unruffled manner of getting me to focus on content over conflict. The very fact that Alex honored both me and my critic had a deep impression on me partially completed that evening when I finally and reluctantly agreed to meet with that member and encourage him to remain on the committee. That impression was completed only some years later as I began (more slowly than I like now to admit) to recognize the true interest of the displeased member, who did agree to remain and even later on became a chairman himself.

It was a fundamental fact of our intimate friendship that Alex and I were of quite different temperaments and talents and yet our professional work at its core was the same; we were both inveterate teaches and writers, with a passion for structural engineering. For each one of us, our professional work centered almost exclusively in one academic content: The University of California at Berkeley and Princeton University, schools where we both were undergraduates and for our entire academic lives faculty members. We were also of that rare breed who did not have the calling card of research universities, a PhD. Given these similarities, it is still our differences that brought us together with such a profound and I believe such a unique intimacy.

I began to sense differences during one of our long after-conference two-person symposia, when Alex explained to me the origin of Berkeley's dominate position as the premier school for civil engineering after World War II, centered around an unusual collection of young engineering professors, Ray Clough, T.Y. Lin, Igor Popov, and Boris Bresler. Alex gave me a picture of collegiality largely absent from most research universities. There Berkeley professors were developing new research ideas in such fields as finite element analysis and prestressed concrete and were giving brand new courses coming directly from that research. Most intriguing to me, these colleagues were regularly taking each other's pioneering courses and they looked outward as consultants as well. Alex pictured an engaging "college" reflecting the British definition of "a self-governing society of scholars for study and instruction incorporated within a University" or the even older Latin meaning of "partnership."

Alex was not someone who outwardly emoted in conversation but in describing Berkeley's structural engineering during the period when I first met him he seemed to sparkle as he spoke of these happy years. All these men are now dead except for Ray Clough who has left California, and as one just on the rim of that academic generation, I feel the need to ask what the legacy is of such a "college." It is too early for me to make such a judgment even if I could but I believe I can try to express the specific legacy that one member has left for an outsider from the other side of the continent .

As I had begun to write and lecture about the history of structures, structures as Art, European bridges and shells, Alex would quietly ask questions directed toward the scholarly bases for such digressions; but one incident best characterized his influence. I had organized an exhibit in the Princeton University Art Museum on the Bridges of Christian Menn, the great Swiss bridge designer. I was eager to have this collection of large color photos travel around the county and suggested to Alex that Berkeley might be interested. He quickly responded that such an exhibition would be welcomed if I would agree to allow it to be accompanied at his University by a parallel exhibition of California bridges, something he was proud of. He spoke to me of the high aesthetic quality of his state's bridges and he implied that I should study them as well as those from Switzerland. Of course I could not reject an offer like that, and the two exhibitions were mounted together and even later traveled to Boston were they were also partners. The California State Bridge engineers put together a slender catalogue of the same size and shape and color as our Menn document and afterwards I proceeded to travel around California with the state's chief bridge engineer and some of his designers to make a study of these fine works about which I now lecture. His legacy to me in this instance was to study the works in my own country even if I still believed that the Swiss works of Menn and Maillart are unsurpassed anywhere.

When asked to give a keynote speech on Alex at an ACI session in Puerto Rico in 1992, I contacted some of his former students to ask about his teaching. There was agreement among them that he was remarkably well organized and clear in his presentations of analysis as well as his insistence on numerical results to explain actual behavior of structures.

They also emphasized the influence of his strong teaching on their professional works after leaving Berkeley. But one of them used a strange and unusual word – after the many rather standard words of praise – a word that at first seemed to me out of place but one which I have thought about afterwards. In rereading my notes for the 1992 talk in Puerto Rico, I found that word appearing on every page of outlines made to create a concise yet clear picture of the legacy of Alexander Scordelis.

That word was “haunting.” The student spoke of being haunted by the Scordelis teaching and emphatically stated that the word was one of praise at a high level not to be construed as something fearful. Reflecting about the Scordelis influence on my professional life I cannot escape that unusual word and the realization of its power. Something unseen, even unspoken that has kept me from veering too far away from out mutual mooring of structural engineering.

The obituaries for Alexander Scordelis stress his research, his teaching, his care for students, his consulting, and his integrity. They correctly picture an exemplarily engineering professor but what is missing is what should follow later and that is a proper biographical sketch to begin forming a library of personalities worthy of giving generations of engineers a true sense of their tradition. Modern engineering has been taught as if it is merely applied science – devoid of individual people, individual works, and individual ideas. This does not mean we need to engage in uncritical hero worship. No such proper biographical writing will be taken seriously if the peculiarities of the personality are fit into an ideal paragon. When thinking of the major figures in our committee on thin shells I realize that such writings could be and should be written about Anton Tedesko, Jack Christenson, and of course Alexander Scordelis. These people haunt me, not because there were faultless, but because they were representative of my profession who deserve to be kept alive for my students, for Alex’s students and for the entire profession for as long as it exists in the United States.

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Alexander C. Scordelis and concrete shells

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Abstract

The Civil Engineering Department of UC Berkeley had owed its decades-long prominence to the simultaneous presence of several giants. Alex Scordelis' contributions to structural engineering in general and thin concrete shells in particular must be seen in this context, namely in the mutually beneficial interactions and cross fertilization with colleagues such as Ray Clough, Ed Wilson, Egor Popov, T.Y. Lin, and Boris Bresler.

Alex Scordelis is generally credited with having been the first to apply the newly developed finite element method to the nonlinear analysis of a reinforced concrete beam. His first paper, coauthored with his student De Ngo, has become a classic. Soon thereafter he realized the potential of this powerful analytical tool to analyze more complex reinforced concrete members and structures, ably assisted by his doctoral students Art Nielsen and Andrew Franklin. The next logical step was the application to thin concrete shells, for which he introduced the interesting concept of a layered finite element, which he worked out in detail with a succession of students.

As he was developing and refining the analytical tools to realistically analyze concrete shells, he never lost his interest in the design and construction of actual shell structures and hanging roofs. The best known examples are St. Mary's Cathedral in San Francisco, designed by Luigi Nervi, and the San Juan Municipal Coliseum in Puerto Rico and Oklahoma City Stadium, on which he cooperated with his lifelong friend T.Y. Lin.

The second part of this presentation addresses the question why concrete shell structures do not seem to enjoy the same popularity they had in the 1950s and 1960s. For a moment I will try to place myself into the position that Alex Scordelis might have taken when confronted with the questions: Yes, why do we no longer build any major structures with thin concrete shell roofs? Given the vast improvement in computational tools to analyze such shells both in the linear and nonlinear domain, and the previously unimaginable advances in concrete technology and construction techniques, don't concrete thin shells deserve another look?

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Alexander C. Scordelis: Legacy in finite element analysis of reinforced concrete

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Abstract

This presentation is dedicated to Alexander C. Scordelis, professor emeritus of structural engineering at the University of California, Berkeley, where he inspired generations of students since joining the faculty in 1949.

Monday, August 27, 2007 he passed away at the age of 83 after battling a long illness at the Alta Bates Summit Medical Center in Berkeley. With his death, the structural engineering profession lost one of the world's most influential experts on long-span bridges and prestressed concrete. His research accomplishments fall into three areas of structural engineering:

(a) Analysis and design of concrete thin shells,

(b) Analysis and design of skewed and curved box girder bridges, and

(c) Nonlinear finite element analysis of reinforced concrete structures.

His contributions to research and education, and his direct involvement in the structural engineering profession left a lasting impression among all of his students. His Socratic style of teaching in large early morning classes made sure that every student was wide awake paying full attention. His demanding teaching stems from his active service in World War II when he was awarded the Bronze Star for meritorious achievement and a Purple Heart for wounds sustained in combat. His leadership by example, and his disciplined educational approach was the hallmark of a very distinguished teaching career.

This paper focuses on the seminal contributions of Alex Scordelis to the development of Finite Element Analysis of Reinforced Concrete Structures, which he pioneered with a ground breaking publication in 1967. Since then this area of investigation has advanced structural analysis to the design of record breaking and unique structural systems enriching the academic and professional lives of many of us who are privileged and proud to carry on his legacy.

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Alex C. Scordelis´ great achievements in bridge engineering - From computer programs to the Golden-Gate-Bridge retrofit

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1. Preliminary Remark

Probably the famous Scordelis-Lo roof shell is the most cited subject on Alex Scordelis´ research [1]; it became a benchmark for Finite Element Technology up to the present time, even for nonlinear analyses although originally defined for linear response. Three years later in 1967 another well-known example was published, namely the first application of the FEM to cracked reinforced concrete beams, thus applying the method to a nonlinear problem. These two papers demonstrate how advanced the research of Alex Scordelis was already at that time. They indicate the principal topics of his research: Finite Element modeling and analyses of reinforced and prestressed concrete, analysis and design of shell roofs and the development of computational models and computer programs for folded plates and box girder bridges. All subjects are in a way related to each other; for example cylindrical roof shells and long-span box girder bridges idealized as folded plates can be based on the same mechanical and numerical model, using a finite strip or a finite element formulation. That is the reason that Alex Scordelis often describes the developments of his methods and programs under concrete shells or box girder bridges, depending in what environment the publication appeared.

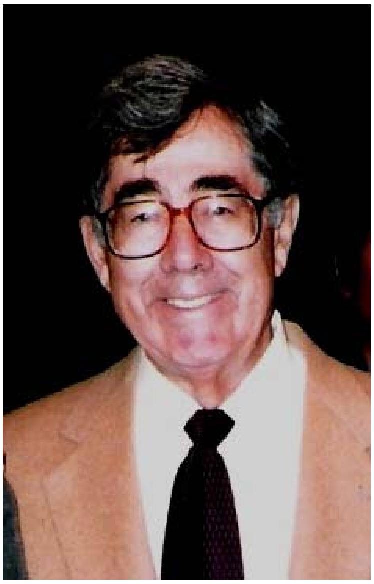

Figure 1: Alexander C. Scordelis at IASS Conference 1996 in Stuttgart

In those times it was very common in the reinforced concrete community to distinguish between the numerical analyst and the designer; Alex was one of the few who worked in between or to express it even better he was an expert in both disciplines. One of his trademarks was his deep insight in the behavior of structures and the ability to interpret the analytical results. It comes without saying that a person of this expertise was not only involved in academia but had also a deep interest in practical work. It all started in the 1950s as a close collaboration with his Berkeley colleague T.Y. Lin. After becoming an authority in reinforced and prestressed concrete structures he was asked to serve as consultant in major projects all over the world.

2. The Family of Computer Programs for Bridge and Shell Analyses

Inspired by the rapid development of the finite element analysis of structures in the late 1950s and early 1960s Alex Scordelis started to develop suitable structural models and computer programs for reinforced and prestressed concrete structures. It began with the analysis of folded plates by the so-called ordinary theory and the theory of elasticity in 1963. These programs were extended to more general loading, to prismatic shells and folded plates reinforced by edge beams. Using the force method in combination with the direct stiffness harmonic analysis continuous prismatic structures with interior diaphragms and general end and interior support conditions could be analyzed. A series of programs were written for special shell geometries like domes, HPs and groined vaults. Besides straight also curved prestressed cellular folded plate structures have been investigated. Research on the nonlinear analysis of concrete shell and folded plate structures began in Berkeley in 1973 including cracking, inelastic and ultimate analyses, temperature changes, time-dependent effects like creep and shrinkage as well as large deformations. The most general computer code was the analysis and design FE program NASHL, allowing to include all mentioned effects and to investigate the structural response up to complete failure.

APPENDIX A: SPECIAL PURPOSE COMPUTER PROGRAMS FOR REINFORCED CONCRETE SHELLS DEVELOPED AT THE UNIVERSITY OF CALIFORNIA AT BERKELEY

1. FOLPLOR [1963] Analysis of Folded Plates by Ordinary Theory.
2. FOLPLEL [1963] Analysis of Folded Plates by Elasticity Theory.
3. MULEL [1963] Analysis of Multiple Cylindrical Shells and Folded Plates with Arbitrary Edge Beams.
4. MULTPL [1965] Analysis of Simply Supported Cellular Folded Plate Structures.
5. PRECYL [1965] Analysis of Simply Supported Prestressed Circular Cylindrical Shells under Uniform Vertical and Dead Load.
6. MUPDI [1966] Analysis of Folded Plates Simply Supported at the Ends with Interior Rigid Diaphragms or Supports.
7. DOME [1968] Analysis of Spherical Domes with or without Edge Ring Beams under Axisymmetric Loading.
8. AXISB [1968] Analysis of Axisymmetric Thin Shells with or without Edge Beams under Axisymmetric Loads.
9. HYPARP [1968] Analysis of Membrane Stresses in Hyperbolic Paraboloid Shells having a Parallelogram Shape in Plan.
10. HYPARQ [1968] Analysis of Membrane Stresses in Hyperbolic Paraboloid Shells having an Arbitrary Quadrilateral Shape in Plan.
11. GROINV [1970] Analysis of Membrane Stresses in Groined Hyperbolic Paraboloid Vaults.
12. CURSTR [1970] Analysis of Curved Folded Plate Structures using the Finite Strip Method.
13. FINPLA [1970] Analysis of Prismatic Folded Plates with Plate and Beam Elements.
14. CELL [1970] Analysis of Cellular Structures of Arbitrary Plan Geometry.
15. MUPDI3 [1971] Analysis of Folded Plates Simply Supported at the Ends with Interior Flexible Diaphragms or Planar Rigid Frame Support Bents.
16. FINPLA2 [1971] Analysis of Nonprismatic Folded Plates with Plate and Beam Elements.
17. MULELR [1971] Analysis of Multiple Cylindrical Shells and Folded Plates with Edge Beams and under Uniform or Partial Loads.
18. MULDI [1971] Analysis of Multiple Shell, Plate and Beam Element Systems under Uniform or Partial Loads with Interior Rigid Diaphragms.
19. NARCS [1973] Nonlinear Analysis of Reinforced Concrete Slabs and Shells.
20. CURDI [1974] Linear Elastic Analysis of Curved Bridges on Flexible Bents.
21. DOMCYL [1975] Analysis of Dome-Ring Beam-Cylinder Systems under Axisymmetric Loading.
22. NOTACS [1976] Nonlinear Analysis of Reinforced Concrete Panels, Slabs and Shells for Time-Dependent Effects.
23. NOPARC [1979] Nonlinear Material, Geometric and Time-Dependent Analysis of Reinforced and Prestressed Concrete Slabs and Panels.
24. LSGNV [1979] Analysis of Membrane Stresses in Hyperbolic Paraboloid, Elliptical Paraboloid and Cylindrical Groined Vaults.
25. NASHL [1982] Nonlinear Analysis of Reinforced Concrete Shells with Edge Beams.
26. MUPDI4 [1985] Elastic Analysis of Straight, Prismatic, Prestressed Cellular Folded Plate Structures with Optional Intermediate Diaphragms and Supports.
27. CURDI4 [1985] Elastic Analysis of Circularly Curved, Prismatic, Prestressed Cellular Folded Plate Structures with Optional Intermediate Diaphragms and Supports.
28. CELL4 [1985] Elastic Finite Element Analysis of Prestressed Cellular Structures of Arbitrary Plan Geometry and Constant Depth.

Figure 2: Computer Program Development at UC Berkeley by Alex C. Scordelis [4]

This amazing development of 28 computer programs, all related to single PhD students or visiting scholars, is briefly documented in [4], see also [3],[5] and [6]. The list shown in Figure 2 is taken from the Appendix of Ref. [4]. It shows a unique series of methods, developed step by step, from early simple linear elastic models including more and more capabilities, ending up in the late 1980s with quite sophisticated geometrically and materially nonlinear time-dependent analyses. The strict consequence in this development is an impressive characteristic of Alex Scordelis´ way to conduct research, extremely successful, not only for himself, but also for his students. Although none of the programs is probably being used today anymore they represent an important phase in the evolution of the related methods.

3. Consultant on California Bridge Projects

Professor Scordelis was well-regarded by the authorities as expert in structural engineering. In 1989 he was appointed to the Governor´s Board of Inquiry into the Loma Prieta Earthquake, which submitted a report on the earthquake´s impact on California infrastructure in 1990. It also recommended forming a Seismic Advisory Board for the California Department of Transportation (Caltrans); Alex Scordelis was a longtime member of this board. He also was the chairman of the Golden Gate Bridge Seismic Instrumentation Advisory Panel and served as a consultant on the Golden Gate Bridge seismic retrofit project, cooperating with District Engineer Mervin C Giacomini. Scordelis´expertise was also asked for in the advisory panel for the design of the new eastern span of the San Francisco-Oakland Bay Bridge.

Figure 2: Alex C. Scordelis at Golden Gate Bridge in San Francisco August 1995

4. Personal Remark

I met Professor Alex Scordelis in 1972 when I was a visiting scholar at UC Berkeley. From the first moment I was taken with his personality. His interest in structural behavior caught me most and seemed to me that his visions were based on a perfect mixture of traditional concepts and modern numerical analyses. I was curious to attend his courses and immediately recognized that he is an ideal teacher, a prototype of an excellent university professor. Later on we often talked about teaching concepts and agreed upon that modeling and interpretation of results should be substantial ingredients in courses on structural mechanics in addition to theory and analysis. It was always a pleasure listing to his plenary lectures, very well prepared, not overloaded with technicalities, precisely coming to the point. Over the years we became friends, I met Alex in Berkeley or on IASS

Conferences; most often he was accompanied by his wife Georgia, like her husband always caring and thoughtful. Alex did know that I was interested in the history of the Golden Gate Bridge and supplied me with much information on this unique structure over the years. In 1995 he arranged a visit to the Bridge, where we were allowed to visit the top of the southern tower, an inimitable experience and for me the start of a close relationship to the bridge in the subsequent years.

I am proud and grateful to have met this outstanding person with a great charisma.

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3-D pushover analysis of a collapsed reinforced concrete chimney

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Abstract

During the Izmit (Kocaeli) Earthquake of August 17, 1999, a 115 m. high reinforced concrete chimney or heater stack, located at the Tüpras Refinery, collapsed. This stack was designed and constructed according to international standards and is representative of similar structures at refineries throughout the world, including those in earthquake-prone regions. This structure is of particular interest because several similar chimneys at the site survived the shock with only moderate damage. The particular distinction of this chimney appears to be an unusually larger rectangular opening, located about 1/3 of the height above the base, which appeared to be the region of collapse initiation. The main focus of the research is the dynamic response of the stack due to an earthquake motion recorded at a nearby site. A new 3-D pushover analysis procedure is proposed in this paper and the results will be compared with those of a nonlinear dynamic analysis. Results are presented that show the importance of the 3-D interaction effects in the dynamic response of the stack. The results also confirm that the stack could readily fail under the considered earthquake and are consistent with the debris pattern.

1. Introduction and objectives

This study is focused on a 115 meter high reinforced concrete chimney, which collapsed during the 1999 Kocaeli earthquake. This earthquake caused great damage to inhabited structures and the regional transportation system that has been well documented. The coincident damage to industrial facilities did not produce a high death toll, but the economic repercussions were enormous. Furthermore, many of these facilities were designed and constructed to international standards and provide information that is readily transferable to other developed countries.

The reinforced concrete chimney shown near the center of Figure 1 collapsed during the earthquake. The debris cut many lines, which fueled fires that shut down the refinery for months.

Figure 1. Heater Stacks before Earthquake

Figure 2. Heater Stacks after Earthquake

This structure is of particular interest because several similar chimneys at the site survived the shock with only moderate damage. As shown in Figure 1, the collapsed heater stack is shown next to a similar structure that survived. The particular distinction of this chimney appears to be an unusually larger rectangular opening, located about 1/3 of the height above the base, which appeared to be the region of collapse initiation. The remnants of the stack are shown in Figure 2.

The overall objectives of the study are four fold:

- To evaluate the original design of the collapsed chimney, known as the Tüpras stack, using current analysis techniques.
- To evaluate the design of a similar size chimney representative of U. S. practice.
- To explain why the single stack in question did indeed collapse while several similar structures in the same vicinity survived with minimal damage through the use of advanced seismic evaluation tools.
- To extend the pushover analysis procedure for chimney structures by taking into account the higher modes and the three dimensional interaction effects.

The first two objectives were addressed earlier [1, 2, and 3] by a response spectrum analysis based on the unsmoothed YPT record as well as the UBC 97 design spectrum.

2. 3-D pushover analysis

2.1 A new 3-D pusohver analysis procedure

In this study, a new 3-D pushover analysis method is proposed to extend the traditional 2-D pushover procedure for the analysis of the asymmetric Tüpras stack. The validity of the proposed method will be assessed by comparing the results with those from an "exact" 3-D step-by-step nonlinear dynamic analysis. The basic procedure is as follows:

1. Carry out a three dimensional modal analysis using a finite element model with the initial geometry and material properties. Obtain the natural frequencies and fundamental modes for each direction.
2. Now, two types of lateral load patterns may be selected based on the basic load patterns. One type is a fundamental mode, usually Mode 1, and the other type may be one of the patterns on Figure 6 other than Mode 1.
3. For a lateral load pattern other than the fundamental mode patterns, apply the lateral forces to the structure, and perform the pushover analysis for each direction. Plot the pushover curves in the spectral displacement vs. spectral acceleration domain (ADRS). The equivalent SDF period for the lateral load pattern in each direction is then taken as the initial secant for the pushover curve before yielding.
4. For each direction, given the fundamental frequencies for the fundamental modes and equivalent SDF system frequencies for the other load patterns, locate the corresponding spectral acceleration values from the response spectrum in each direction (In this case, the longitudinal and transverse directions of the YPT spectrum).
5. Apply two directional lateral forces for each load pattern to the structure, as illustrated in Figure 3, proportional to the spectral acceleration values obtained from Step 3.
6. For each load pattern, perform the 3-D pushover analysis using the lateral load forces described in Step 4, and plot the capacity curve for each direction.
7. Compare the capacity curves with the smoothed mean demand curves of the spectra for each direction to obtain the target displacement of the structure for different load patterns.
8. Determine the response over the height of the structure using the 3-D pushover analysis results for the different patterns at the respective target displacements.

The validity of the proposed method will be assessed by comparing the results with the three dimensional nonlinear dynamic analysis of the stack.

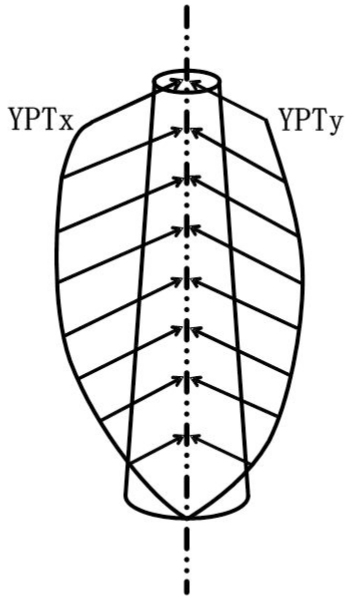

Figure 3. 3-D Pushover Load Pattern

2.2 3-D pusohver analysis results

First, the 3-D pushover procedure is applied to the model without an opening and compared to 3-D nonlinear dynamic analysis with two directional inputs. Then the procedure is extended to predict the failure of the model with the opening. The failure analysis for the model with the opening is carried out by 3-D nonlinear dynamic analysis as well.

2.2.1 Model without an opening

Similar to the traditional 2-D pushover analysis, target displacements are calculated by the proposed 3-D pushover analysis, based on different lateral load patterns as well as the MPA procedure. These target displacements using 3-D and 2-D pushover analysis, given in Table 1, are the magnitudes of the displacement of the stack at the top. As shown in the table, 3-D pushover analysis results are based on the YPT earthquake input in both directions while 2-D results are based on YPTy earthquake input in one direction. They are calculated by combining the results of target displacements for each direction. The errors are obtained by comparison to the nonlinear response history analysis results (NL RHA).

Table 1. Target Displacements for the Model without an Opening

Method	3-D Pushover		2-D Pushover	
	Disp. (m)	Error (%)	Disp. (m)	Error (%)
Mode 1	0.522	-13.1	0.498	-17.1
Uniform	0.416	-30.8	0.316	-47.4
ELF	0.680	13.1	0.492	-18.1
Triangle	0.701	16.6	0.476	-20.8
SRSS	0.662	10.1	0.629	4.7
MPA	0.565	-6.0	0.528	-12.2
NL RHA	0.601	0	-	-

2.2.2 Model with the opening

Since the stack failed in an earthquake having different lateral loading components acting simultaneously, it is appropriate to analyze the structure in multiple directions. The failure displacement and the cracking pattern recorded at the failure point from 3-D nonlinear dynamic analysis will be used to validate the 3-D pushover procedure.

Table 2. 3-D Failure Displacements for the Model with the Opening

Pattern	YPT Mean	
	Failure Disp. (m)	Error (%)
Mode 1	0.667	27.0
Uniform	0.745	41.8
ELF	0.646	23.1
Triangle	0.645	22.8
SRSS	0.597	13.8
NL RHA	0.525	0

As shown in Table 2, where results taking into account higher mode effects in both directions are summarized, the SRSS distribution provides the best prediction, with less than 14% error.

3. Conclusions

A new 3-D pushover analysis procedure was proposed and applied to models of chimneys with and without an opening. Various lateral load patterns were considered. For the target displacement of the model without the opening, the error from the Uniform distribution was the largest, while the Mode 1 distribution, ELF distribution, and Triangle distribution provided somewhat better estimates. The SRSS distribution gave a good prediction, with an error around 10% and the error from the MPA procedure was even less than 10%. As to the peak deflections, the MPA procedure and SRSS distribution provided the best estimates, while the Uniform distribution underestimated the total response by up to 30%. The Mode 1 distribution, ELF distribution, and Triangle distribution gave similar estimates. Compared to a 2-D pushover analysis, the new 3-D pushover analysis procedure provides a better estimation for target displacements.

The 3-D nonlinear dynamic analysis results confirmed that the Tüpras stack could not survive the YPT earthquake inputs under both directions.

Acknowledgement

The authors wish to thank the United States National Science Foundation for the support of this study under grant CMS-0084737.

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Optimization of concrete hyperbolic paraboloid umbrella shells

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Abstract
Concrete hyperbolic paraboloid umbrella shells such as those designed and built by Felix Candela have proven to be durable, efficient and economical structures. Candela’s umbrella shells represent his optimization of the hyperbolic paraboloid form through full-scale experimentation and analysis. This project re-creates Candela’s experimentation and progression of the geometry of that form through computational optimization and structural analysis. Finite element models representing five of his umbrella shells were analyzed in order to understand Candela’s development of the form.

1. Introduction
During the 1950’s and 1960’s, Felix Candela designed and built many thin shell concrete structures in the Mexico City area. Early in his career, Candela discovered the benefits of the hyperbolic paraboloid (hypar) as the geometric form for these shells. The hyperbolic paraboloid not only distributes forces efficiently, but is also formed from straight lines. This geometric curiosity means that the formwork of hypar shells can consist of straight members, thus saving the expense of custom curved formwork. The straight-edged umbrella type hypar was a particularly economical form for him. He was able to cover large spaces with a series of repeated umbrella shells, thus sheltering a number of warehouses, factories, and markets (Figure 1).

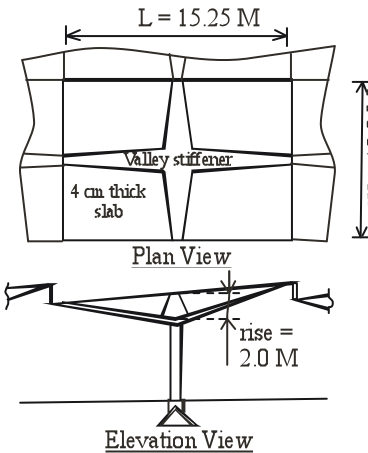

Figure 1: Umbrella shells of Rio’s Warehouse, Mexico City, designed and built by Felix Candela in 1954

Most of Candela’s structures in the Mexico City area are still standing and are in good shape. Despite their enduring benefits, though, hypar concrete shells are rarely designed or built today. One by-product of the dearth of current shell construction is an erosion of the technical expertise needed to properly understand their behavior (Meyer and Sheer [7]). As part of an investigation into the contemporary viability of shell structures, this project sought to re-create Candela’s evolution of designs of umbrella shells as a way of better understanding hypar concrete shell design. Finite element models of some of his representative structures were created, analyzed, and modified to follow Candela's design evolution and better understand his optimization of the form.

2. Candela's design progression

During the 1950s and 1960s, Candela and his Mexico City construction firm, Cubiertas Ala, designed and built many (perhaps hundreds) umbrella type hyperbolic paraboloid concrete shells around Latin America. The dimensions of five of his earliest for which we were able to obtain dimensions are shown in Table 1. Figure 1 defines the area (length (L) times width (W), or projected area) and the rise. Although Candela would eventually become known for his dramatic, curved-edge hypars, the straight-edged umbrella form came to be considered the firm's "bread and butter," as its straightforward construction process and repeatability made it an economical, versatile solution for a wide variety of applications.

Table 1: Dimensions for five of Candela's early umbrella shells (Faber [4] and Avery [1])

Year	Umbrella	Length, m	Width, m	L/W	Area, m^2	Rise, m	Thickness, cm	Rise/Area, m^{-1}	Max defl., cm
1952	Prototype	10.1	10.1	1.0	101.2	0.9	3.81	0.009	-1.57
1953	Second Experiment	7.9	7.9	1.0	62.8	0.6	8.255	0.010	-0.58
1953	Novedades House No. 5	7.6	7.6	1.0	58.1	1.1	n/a*	0.018	-0.42
1954	Rio's Warehouse	15.3	10.0	1.5	152.5	2.0	4	0.013	-1.42
1958	Mercado Chiclayo	15.0	14.0	1.1	210.0	3.2	4	0.015	-1.55

*The thickness of the 1953 Novedades House No. 5 was assumed to be 4 cm.

The changes shown by the five umbrellas in Table 1 demonstrate Candela's optimization of the form. We see, for example, that the ratio of rise to area converges on a value of 0.015 (m^{-1}). Candela eventually revealed that this value was, in fact, his "little formula" (Garay [6]). He likewise eventually determined that the optimal area of an umbrella should be 186 square meters (2000 square feet) based on restricting the height which can lead to larger volume (space) to heat or air condition (Candela [2]). By determining these optimal values, Candela was able to eventually construct large efficient umbrellas.

3. Finite element analysis of Candela's umbrella shells

Finite element models of the five concrete hypar umbrella shells were created and analyzed using the finite element program SAP2000 (Computers and Structures [3]). The models were composed of thin shell elements, and were analyzed under the effect of their own dead load. Results of the finite element analyses are shown in Table 1. Maximum deflection at the edge was used as the criterion for comparison. In all cases, deflections are kept quite low (all below 2 centimeters).

4. Optimization of umbrella shells

Following the finite element analyses of the structures as built, the models of each umbrella shell were modified to follow Candela's path of optimization. Models were created and analyzed to evaluate the effect of (a) ratio of rise to area, (b) ratio of side lengths, and (c) thickness.

4.1 Optimization of ratio of rise to area

The rise of each of the five umbrella models was modified to make the ratio of rise to area equal to Candela's optimum, 0.015 (m^{-1}). This resulted in a decrease in deflection for most of the models. The exceptions were the structures that had ratios of rise-to-area greater than 0.015. Thus, an increase in the ratio of rise-to-area is associated with a decrease in deflection. The ratio of rise-to-area for all of the models was increased further, then, to ratios of 0.020, 0.025, and 0.050. The results of these analyses are shown in Figure 2.

The results show that deflections decrease as the ratio of rise-to-area increases. This raises the question of why Candela settled on an optimal ratio of rise-to-area of 0.015. The designer could, after all, increase the ratio to decrease deflections even further. As a builder, Candela understood that increasing the rise also causes a corresponding increase in volume of concrete required. A rise that is too steep can also add complexity to the construction process. Candela therefore combined an engineer's understanding of the structural behavior with a builder's concern with economy to arrive at an optimized ratio of rise-to-area.

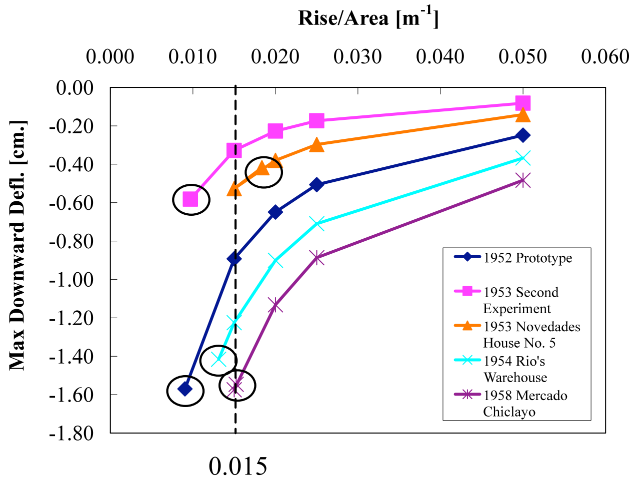

Figure 2: Graph of ratio of rise-to-area versus maximum deflection. Values representing built structures are circled. Other data points represent a re-design.

4.2 Optimization of ratio of side lengths

This project also sought to investigate the differences between square umbrellas and those of unequal side lengths. Table 1 shows that Candela's early umbrellas were square, but later he altered the lengths of the sides. Finite element models for ten different umbrellas of varying ratio of side lengths, from 1 (square) to 10 were created and analyzed under the effects of dead load. All models had a ratio of rise-to-area of 0.015 and a constant thickness of 4 centimeters (1.6 inches). Results of the analyses are shown in Figure 3.

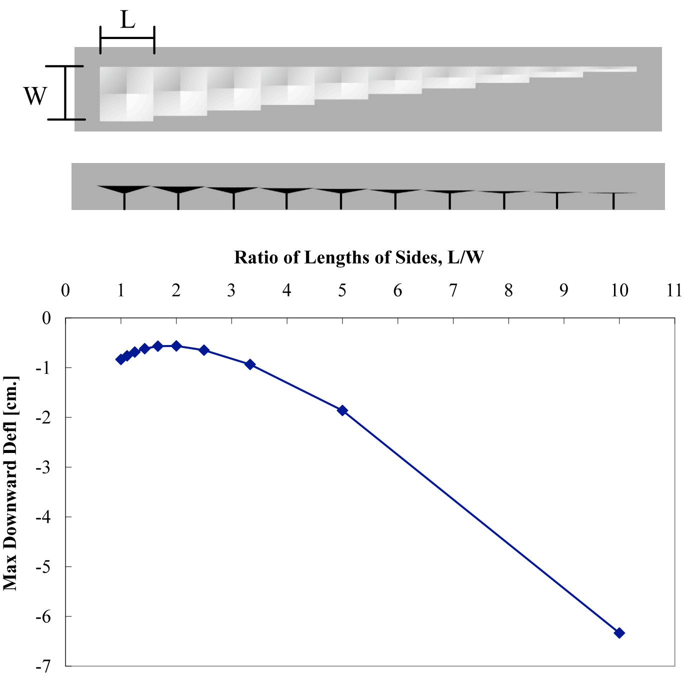

Figure 3: Graph of ratio of side lengths versus maximum deflection (models shown above graph) with a constant rise/area = 0.015.

Deflections decrease slightly as the ratio of side lengths goes from 1 to 2. This suggests that for a constant ratio of rise-to-area of 0.015, the optimal form for an umbrella shell is not square, but rectangular with a ratio of side lengths of 2.

None of the five Candela shells studied here had a ratio of side lengths of 2, but the last two shells do have a slight variation in side lengths (Table 1). Through his experimentation in the field, Candela perhaps recognized that the optimal form would vary from the square. He most likely again resolved this understanding with considerations of construction.

4.3 Optimization of thickness

Modern computational structural optimization algorithms can also be used to evaluate Candela's optimization of his hypar umbrella shells. In this case, a thickness optimization algorithm developed for use in conjunction with the finite element program Dynaflow was employed to determine an optimal thickness distribution for a model of the 1954 Rio's Warehouse shell (see Holzer et al. [5]). Preliminary results of this analysis are shown in Figure 4 for the case where the maximum thickness was set to 40cm in the program. The results show an optimal thickening towards the center of the shell.

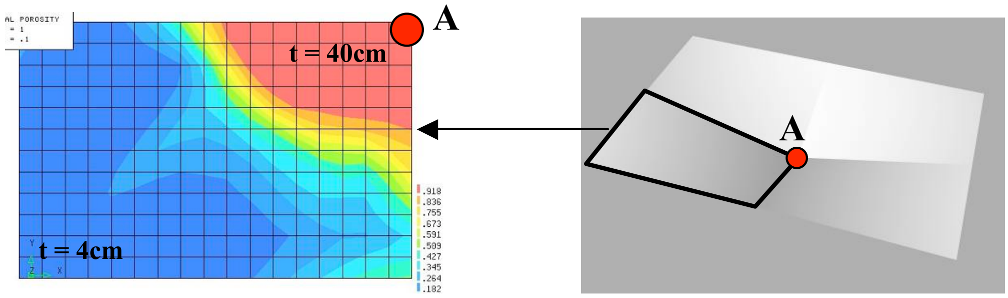

Figure 4: Thickness optimization results for model of a quarter of Rio's Warehouse shell.

5. Conclusion

Five of Felix Candela's earliest umbrella-type hypar shells show a progression towards more efficient structural forms. Candela used his understanding of engineering behavior to optimize the ratio of rise-to-area, the ratio of side lengths, and thickness. As a builder, he also used his experience and intuition to integrate forms of optimal structural behavior into structures that could be built economically. The use of modern computational analytical tools can help us to understand how a master builder such as Felix Candela was able to optimize his hypar umbrella shells. Those tools, combined with a thorough structural understanding, might then in turn allow the contemporary designer to again employ this neglected yet effective structural form.

Acknowledgements

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Delamination in two-layer thin-shell dome with unanticipated construction openings

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Abstract

As Nuclear Power Plants age many require steam generator replacement. There is a nickel alloy in the steam generator tubes that is susceptible to stress cracking and although these cracks can be sealed the generator becomes uneconomical after 10%-15% of the tubes have cracked. The steam generator in a typical nuclear power plant is housed in the containment structure next to the reactor The equipment hatch in such structures is not big enough to facilitate steam generator replacement, thus construction openings must be made. Where both the walls and the dome of the containment structure have been post tensioned such openings are generally made in the walls, and where only the walls are post tensioned it is easier to put the openings in the dome.

This study examines the effects of such openings placed in a dome of a nuclear containment shield building. The concrete structure is made up of a 61 cm thick dome atop 91 cm thick and 47 m high cylindrical walls (radius 20 m) with a tension ring 4.6 m high and 2.4 m thick in between. The dome of the building is cast in two layers; a lower 23 cm layer that serves as the formwork for an upper 38 cm layer. This study concentrates on the stresses through the depth of the dome to investigate the potential for delamination of the two layers when the construction openings are made. The weight of the dome is carried in axial compression along the hoops and meridians of the dome. The openings interrupt the hoops and meridians and thus the weight of the dome must be redistributed around the openings. Without openings, the stresses due to dead load in the structure are very low when compared to the material strength. The impact of the openings is increased compression stresses near the opening. The maximum stresses are between three and four and a half times larger than in the original structure with no openings. In the affected areas there is a significant difference between the compression on the top surface of the dome and that on the bottom surface, leading to shear stresses through the depth of the dome. This study examines those shear stresses at the interface of the two layers of the dome. The shear capacity of this interface is determined by previous empirical studies of composite beams. Where the shear stresses in the dome with openings are larger than the shear strength of the interface delamination can occur.

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Testing, modeling and constructing wood-plastic composite Catalan vaults

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Abstract

This presentation will describe the application of new materials to a traditional building method. An overall goal of our current research is to invigorate the tradition of thin shelled roofs, by reinventing the Catalan laminated vaulting technique using new, ecologically-friendly building materials. In order to encourage architects and structural engineers to design such structures, proof-of-concept studies such as this one must be conducted and published.

1. Introduction: The Composite Material

The material we are using is generally classified as a wood thermoplastic composite and more specifically as wood-plastic composites (WPC). The wood used in WPCs is often a particulate, such as wood flour, but it can also be a mixture of short and/or bundled fibers. **[1]** Current commercial formulations of WPCs can have anywhere from 50% to 70% wood fiber reinforcement. The composite material we used in this study has a plastic matrix of high density polyethylene (HDPE) which can be obtained as virgin material or recycled from waste stream products such as milk jugs. This environmentally-friendly aspect of our material has generated enthusiasm for this project among students and granting agencies. Another interesting ecological aspect of this material is that the wood reinforcement can be harvested from invasive small diameter trees such as Salt Cedar. Or it can be made from sustainable high strength fibers such as kenaf or flax.

By far, the largest current market for WPCs is the outdoor deck market in North America. Consumers and contractors are attracted to its durability, lack of splintering, and low maintenance, and this has fueled a $3.2 billion market for this material. Yet the use of a WPC as a bending member, as one would use in outdoor decks, is not the most structurally efficient configuration. This is true because bending members such as deck boards must be fairly hefty to ensure adequate stiffness. A much more structurally efficient use of this material is to place it as a structural unit in a laminated vault. Such vaulting would not be used for decks, but rather, as a thin-shelled roof, as a stand-alone sunscreen, or as a load bearing component of a larger structure. It is this path that we have been researching in the Architectural Engineering Department at Cal Poly.

2. Catalan Vaulting

Traditional laminated vaults originated in the Catalan region of Spain, yet features of this vaulting technique have been found in other cultures. **[2]** The difference between Catalan vaulting and traditional masonry vaulting is that the tiles are laid flat, with a substantial mortar thickness between layers or courses (See Figure 1).

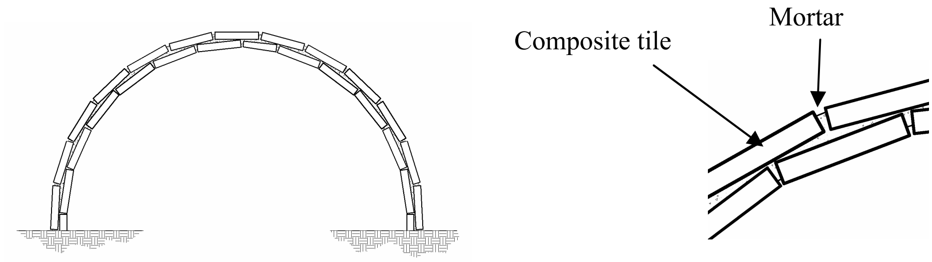

Figure 1: Elevation of Catalan Vaulting and Detail of Mortar and Composite Tile

The result is a thin shell that is essentially monolithic because of the tenacity of the mortar. This vaulting technique was made popular here in the US by the father and son team of Rafael Guastavino and Rafael Jr. Their firm, the Guastavino Company, was involved in over one thousand buildings, some of which are seminal in the history of neo-gothic and neo-classical architecture, such as St. John the Divine Cathedral and the Boston Public Library. A rule of thumb in all good thin shell design is the selection of the proper form. A properly designed thin shell structure will exhibit little or no bending and will carry loads primarily through compression.

3. Testing of the New Material

Unlike Guastavino's Portland Cement mortar, we used a two component, acrylic based high strength adhesive such as Simpson AT. We also added a casting agent, which added some volume to the mortar, allowing us to create the curve shown in Figure 1. This mortar adhered best if the wood-plastic composite was first sanded with a very coarse grit paper.

We did several shear tests and a more extensive compression testing program. The average shear stress we obtained was 15.8 MPa (2300 psi) at failure. Since the laminated vaults will not experience substantial in-plane shear, this shear strength has been deemed adequate. Specimens of the wood-plastic material were tested in compression (See Figure 2). We found an average initial modulus of elasticity to be 2.96 GPa (430,000 psi).

Figure 2: Compression Testing

The stress vs. strain curve is nonlinear as other studies have found. [**3**] A bilinear curve can be used if the material is to be modeled as orthotropic. A multi-linear elastic model can be used in a finite element model if the material is assumed to be isotropic as was done in the present study. Figure 3 shows the multi-linear elastic constitutive model used in the finite element program which was arrived at through testing.

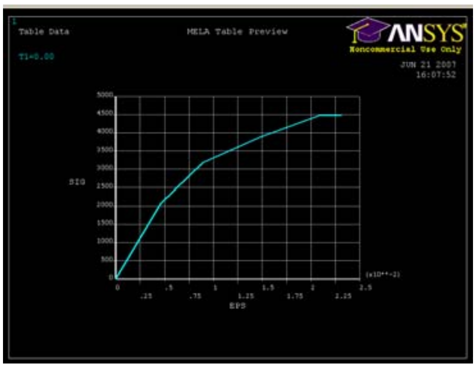

Figure 3: Experimental Compression Data

We used the commercially available finite element software ANSYS to capture the nonlinear material properties as well as the thin shell behavior of the structural units. While such finite element programs are robust, it is often quite tedious to input all of the geometrical properties. To increase the efficiency of the input process and to reduce the errors of manually drawing or entering the geometric data, we devised a means of using the program MATLAB to create the necessary input data which can then be pulled into ANSYS. To test the viability of this method, we focused our attention on spiral staircases. We created a MATLAB program that can automatically generate a staircase of any user-defined inner radius, outer radius, and pitch of the spiral. Such MATLAB generated output is shown in Figure 4. Having the MATLAB generated data file greatly increased our efficiency in exploring a variety of spiral staircase geometries and to do so in an error-free manner.

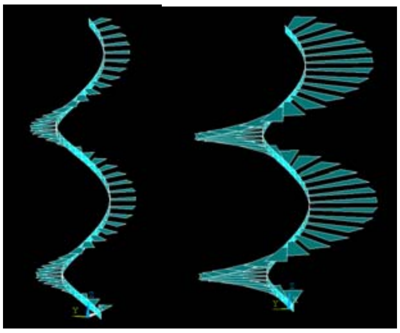

Figure 4: MATLAB Generated Geometry

We chose the spiral staircase as a prototype test case because of its dramatic geometry. It also provides interesting and challenging constructability issues; issues that must be solved at the bench top scale before attempting larger scale Catalan structures. Typical results that can be obtained from such finite element analyses are principal stress data and vertical displacement data. Other information such as collapse mechanisms can also be found. Figure 5 shows such a collapse study.

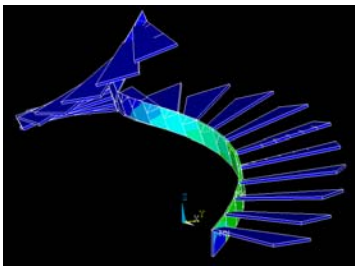

Figure 5: Finite Element Collapse Study

5. Preliminary Model Making

We have constructed several table-top size Catalan vaults made from the composite materials. Figure 6 shows a one meter tall spiral staircase. Other models took advantage of straight line generators to create barrel vaults or hyperbolic paraboloids (Figure 7).

Figure 6: Preliminary One Meter Tall Spiral Staircase

Figure 7: Straight Line Generated Forms

6. Future Work

This research will continue to explore larger physical models. We hope to solve constructability issues that will arise as well build on a larger scale. Ultimately, we will load test these larger structures and compare the finite element response to the physical experiments.

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Concrete vaulting in Imperial Rome: A structural analysis of the Great Hall of Trajan's Markets

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Abstract

The Great Hall of the Trajan's Markets in Rome is the oldest surviving example of a cross vaulted structure built entirely in Roman pozzolanic concrete (*opus caementicium*.) The Finite Element method is used to investigate the structural behavior of the Great Hall under static gravitational loads and with the assumption of Roman concrete behaving as a linearly elastic material. The results of the analysis are in excellent agreement with the observable fracture patterns in the vault and in the lateral arches. The study shows that, contrary to the commonly held view in the literature, the lateral arches do not perform any contrasting action due to incorrect positioning. However, the structural skeleton of the Great Hall, characterized by the innovative use of lateral arches, shear walls, and supporting travertine blocks, can be regarded as the earliest prototype of the structural solution adopted for the design of gigantic concrete vaults.

1. Introduction and objectives

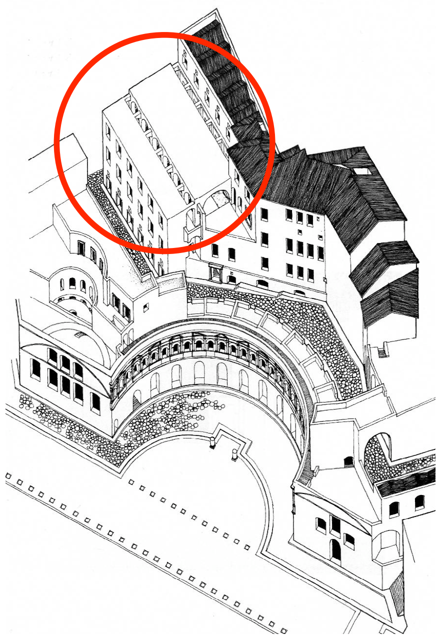

Figure 1: Isometric view of the Trajan's Markets (MacDonald [1]) - the Great Hall is highlighted

The Great Hall occupies a preeminent position within the Trajan's Markets, the imperial building complex on the Quirinal hill overlooking the Forum of Trajan in Rome – Figure 1. Built between the years 98 and 117 AD and essentially intact in all its structural elements, the hall is a vaulted rectangular space 36 meters long and 8.8 meters wide, built entirely in Roman pozzolanic concrete (*opus caementicium*). The vault itself, consisting of a series of six cross vaults resting on travertine blocks and connected to the adjacent structures by lateral arches, can be regarded as the precursor of the gigantic vaults in *opus caementicium* used in Roman imperial baths and basilicas from the second to the fourth century AD.

The Markets of Trajan have been studied extensively by archaeologists and art historians since being brought to light by the 1926-1934 excavations and restorations, with particular attention often reserved for the Great Hall. Because of its series of cross-vaults and corbelled travertine support piers, the Great Hall was soon regarded as an innovative architectural form, made possible by the concrete revolution in Roman Imperial architecture. Beginning with the earliest study (Giovannoni [2]) the lateral arches were identified as structural contrasting elements, necessary to oppose the horizontal thrusts generated by the large concrete vault – Figure 2. Their perceived functional role made them the oldest precursors of the flying buttresses of Gothic cathedrals.

In recent years, a series of in-depth archaeological studies of the Great Hall have shed additional light upon aspects ranging from the construction to the definition of internal components and their qualitative function as a structural whole (Lancaster [3].) Still, perhaps the most striking features of the hall – the physical structures of the vault and its support system – have not been subjected to engineering structural analysis. Thus, the functional identification of the lateral arches as flying buttresses has been reaffirmed over the years by several authors based solely on qualitative reasoning, see, for example, the recent work by Addis [4].

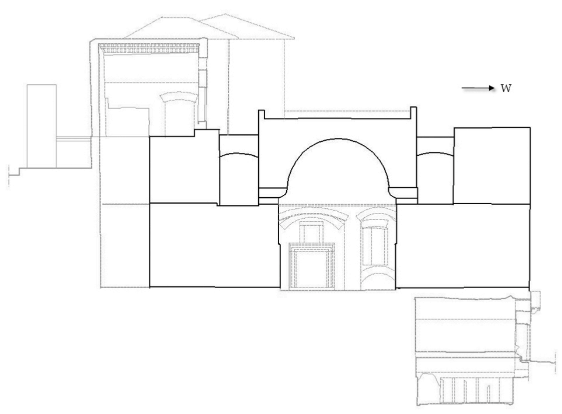

Figure 2: E-W cross section of the Great Hall

As part of an interdisciplinary research on the engineering design of concrete Roman vaults conducted in collaboration with the Museums of the Imperial Fora in Rome and the University "La Sapienza", Rome, we are investigating the structural behavior of the Great Hall through Finite Element (FE) stress analysis. In the present paper, we limit the discussion to the response of the structure under static gravitational loads. The primary objectives are: (1) develop an understanding of the mechanics of deformation of the vault and the supporting system; (2) identify areas subjected to elevated tensile an compressive stresses; (3) determine the functional role of the lateral arches; and (4) attempt an evaluation of the structural design of the Great Hall.

2. Methods

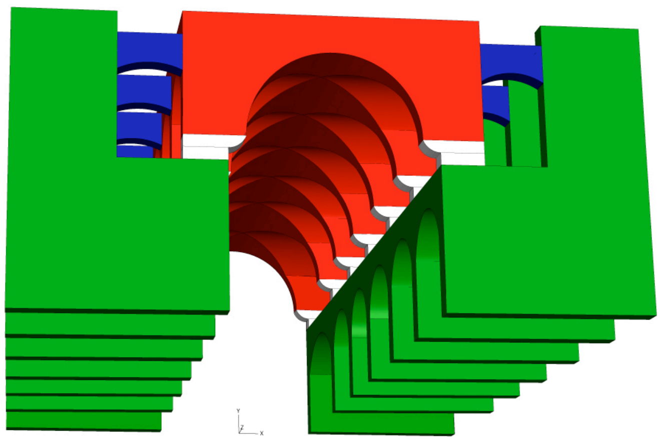

Figure 3: Structural skeleton of the Great Hall

All FE models are based on a three-dimensional solid model derived from a survey of the monument and representing the structural skeleton of the Great Hall isolated from the other buildings in the Trajan's Markets. The model – shown in Figure 3 – is comprised of the concrete vault (red), the supporting travertine blocks (white), the lateral arches (blue), and the concrete transversal shear walls and piers (green). This solid model is used to create a series of local and global 3-D meshes consisting of quadratic tetrahedral elements.

The *opus caementicium* used in the Great Hall is a conglomerate consisting of a coarse aggregate (primarily tuff but also brick pieces) embedded in a pozzolanic mortar. Following the experimental data for a Roman pozzolanic conglomerate of similar composition provided by Samuelli Ferretti [5] for the Basilica of Maxentius, the *opus caementicium* is modeled as a linear elastic isotropic material with modulus of elasticity equal to 3 GPa, mass density 1540 kg/m^3, and Poisson ratio 0.2. The compressive strength of this material is 4 MPa and the tensile strength is conservatively set at 0.1 MPa. All the FE analyses assume infinitesimal strains.

3 Results

Two sets of boundary conditions are used to simulate the mechanics of the supporting travertine blocks under gravitational loads: either fully constrained (no relative motion between the blocks) or with rollers allowing the upper block to slide in the horizontal plane. A more accurate modeling of the contact conditions, involving both sliding and rotations, requires a nonlinear FE approach and is beyond the scope of the present paper.

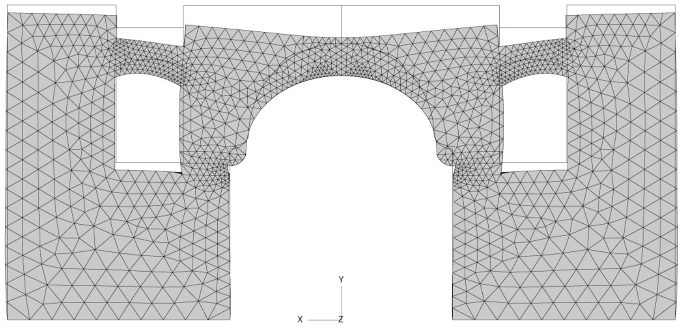

Figure 4: Deformation pattern under static gravitational loads (600x)

3.1 Deformation

The primary deformation mechanism – shown in Figure 4 – consists of a downward sag (bending) at the crown accompanied by an outward displacement at the springing of the vault. There is no outward displacement at the attachment of the lateral arches. Following the motion of the vault, the arch is deflected downward while the attachment section rotates. Sliding of the blocks markedly increases the magnitude of the deformations without significantly altering the deformation pattern. Analyses of the structure with the lateral arches removed indicate that the deformations are not affected by the presence of the arches. The removal of the shear walls, however, is shown to substantially increase the magnitude of the deformations, especially in correspondence of the supporting blocks.

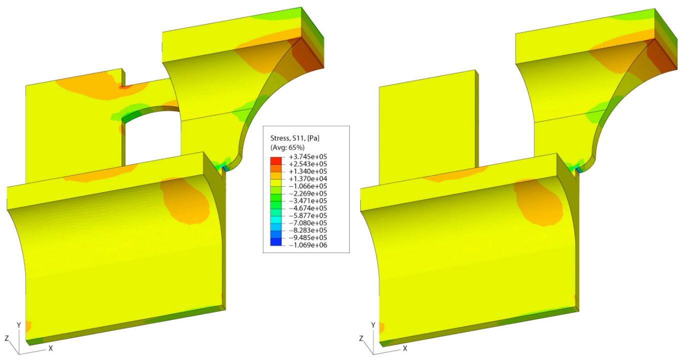

Figure 5: Stress distribution in x direction with lateral arch (left) and without arch (right)

3.2 Stress distribution

The bending deformation at the crown induces tensile stresses at the intrados of the vault, Figure 5, forming a band that extends along the entire length of the vault, Figure 6. Theses stresses exceed the tensile strength of the material. Nuclei of tensile stresses, also exceeding tensile strength, are also present at the attachments of the lateral arches, due to bending of the arches, Figure 5. As for the displacements, the removal of the lateral arches does not affect the stress distribution at the intrados or extrados of the vault, while the presence of the concrete shear walls below the travertine blocks plays a critical role in controlling the magnitude of the tensile stresses in the vault. All compressive stresses are well within the compressive strength of the *opus caementicium.*

4. Discussion

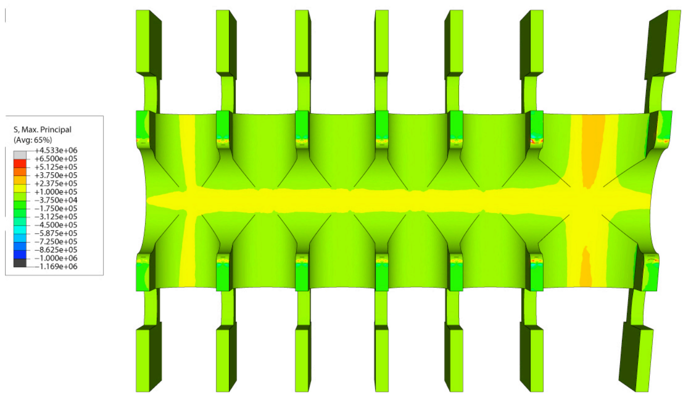

Figure 6: Maximum principal stresses at vault intrados

The numerical results are validated by comparing the computed stress distributions to the pattern of fractures observable in the structural skeleton. We then address the functional role of the arches and finally present a tentative evaluation of the structural design of the Great Hall.

4.1 Validation of the FE results

A longitudinal fracture, visible in pictures taken during the original restoration and subsequently repaired with brick insertions and injections of modern concrete, traverses longitudinally the entire intrados of the main vault, in excellent agreement with the distribution

Figure 7: Intrados of the vault – present state

of tensile stresses predicted by the model – Figure 7. Similar repaired fractures are found in the lateral vaults. Results also suggest that the lateral arches may have been weakened by tensile stresses acting at the attachments. In fact, only few of the original arches have survived, often with major repairs. The predicted tensile stresses match the observable fracture patterns at the attachments of several arches. Arguably, the collapse of the other arches may be taken as a further proof of the accuracy of the predicted stresses.

4.2 Functional evaluation of the lateral arches

The lateral arches do not function as contrasting arches under static gravitational loads. The numerical results, supported by the physical evidence, indicate that the vault does not produce a horizontal thrust at the level of the arches. As shown earlier, the arches are subjected to bending and shear but not compression. Their presence does not contribute in any appreciable way to controlling the deformation, and therefore the stress state, of the vault. Perhaps, considering that this is the earliest free-standing cross vault in *opus caementicium* which has survived, we should regard the lateral arches as an initial, and partially failed, attempt by the Roman engineers to create a stabilizing element for a new type of structural form.

4.3 Structural design of the Great Hall

The design of monumental cross vaults made possible by the opus caementicium set forth new challenges for the Roman structural engineer that can be summarized as follows. The structural form and the intrinsic weakness of concrete to tension produce fracturing at the intrados of the crown accompanied by large thrusts at the springing of the vault with the potential for catastrophic collapse. These challenges were resolved in part by developing a design based on lateral shear walls, contrasting arches, and supporting blocks and in part by reducing the weight of the vault as much as possible. As shown in Figure 3, these elements are all present in the structural skeleton of the Great Hall, albeit at different state of refinement. Thus, while the shear walls are effective in reducing the tensile stresses, the contrasting arches are not properly positioned to counteract the lateral thrust, which, acting at the level of the travertine blocks, may cause these to slide and rotate. The presence of ancient dovetail clamps and the damage on the blocks suggest that these motions may have taken place, worsening the fractured state of the vault. In conclusion, the structural analysis of the Great Hall shows that its designer was moving in the correct direction, formulating a new structural scheme that, once perfected, would have made possible the construction of gigantic vaults such as those at the Baths of Caracalla and Diocletian and at the Basilica of Maxentius.

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Numerical study of steel corrosion in concrete shell members

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Abstract

In this paper, the rate of steel corrosion in concrete shell members is investigated using virtual polarization resistance experiments. The virtual experiments are based on the numerical solution of the Laplace's equation with predefined non-linear polarization boundary conditions and have been designed to establish independent correlations among corrosion rate, temperature, concrete resistivity, and cover thickness for a wide range of possible anode/cathode (A/C) distributions on the reinforcement. The results have been used to develop a closed-form regression model for the prediction of the corrosion rate of steel in concrete shell members. Using the model the effect of cover thickness on steel corrosion is studied. It has been demonstrated that the effect of cover thickness on the corrosion rate of steel is not significant when the concrete cover in reinforced concrete shell member is intact.

1. Introduction

The corrosion of steel in reinforced concrete shell members is an important issue in highly aggressive environments. Factors that can affect the rate of steel corrosion in concrete shells include temperature, moisture content, concrete resistivity, and kinetic parameters of corrosion. Beside these, concrete cover thickness has also been considered as one of the possible factors that may affect the active corrosion rate of steel. Concrete cover above the steel reinforcement is a barrier against the penetration of aggressive ions (e.g. chlorides), moisture, and dissolved oxygen, all of which are essential for the initiation of steel corrosion in concrete. However, the role of concrete cover during the propagation stage of active corrosion is still debated. The goal of this investigation is to carry out a comprehensive numerical study to obtain a closed-form model that relates the corrosion rate of steel and the influential parameters affecting corrosion in reinforced concrete members. Using the developed model the effect of cover thickness on active steel corrosion is further studied.

2. Corrosion modeling

The corrosion rate at any point on the surface of steel in concrete is related to the current density, which can be predicted if the electrical potential distribution around that point is known. Once the potential distribution along the reinforcement is known, the current density at any point on the steel surface can be calculated by (Munn and Devereux [4]):

$$i = -\frac{1}{r}\frac{\partial \phi}{\partial n} \tag{1}$$

where i [A/m^2] is the current density, ϕ [volts] is the potential, r [Ω.m] is the resistivity of the pore solution and n is the direction normal to the bar surface. Assuming electrical charge conservation and isotropic conductivity, the potential distribution can be represented by the Laplace's equation (Munn and Devereux [4]):

$$\nabla^2 \phi = 0 \tag{2}$$

Calculation of the potential distribution around the surface of the steel involves the solution of Eq. (2) subject to prescribed boundary conditions. These boundary conditions comprise the nonlinear relationship between potential and current density for the anodic and cathodic regions as well as prescribed current densities. Further information on the nonlinear boundary conditions can be found in (Stern and Geary [4], Isgor and Razaqpur [3]). These boundary conditions necessitate a non-linear solution of Eq. (2), for which a customized finite element program was previously developed and was verified with experimental data (Ghods et al. [2]). Once the potential distribution is obtained after the solution of Eq. (2), the corrosion current density at any point on the surface of the rebar can be obtained by locally applying the Ohm's law via Eq. (1).

3. Effect of concrete cover thickness

The effect of cover thickness on the active corrosion of steel during the propagation stage in concrete manifests itself through oxygen diffusion, i.e. through concentration polarization of the cathodes. The limiting current density is the main parameter that controls the relationship between the amount of oxygen available on the cathodic surfaces and the amount of polarization. The common simplified approach to treat this process is to consider cover thickness as a diffusion layer, through which steady-state diffusion of oxygen takes place. With this approach, the limiting current density, i_L (A/m^2), can be calculated as a closed-form equation as a function of concrete cover thickness, d (m), in the following form (Bohni [1]):

$$i_L = n_e F \frac{D_{O_2} C_{O_2}^s}{d} \tag{3}$$

where n_e (=4) is the number of electrons, F (≈96500 C/mole) is the Faraday's constant, D_{O_2} (m^2/s) is the concrete oxygen diffusion coefficient, and $C_{O_2}^s$ (mol/m^3) is the amount of dissolved oxygen in water.

4. Virtual experiments

Virtual experiments are chosen over a comprehensive laboratory study to investigate the effect of cover thickness on the corrosion of steel in concrete and to develop a closed-form model simulating the existing corrosion rate measurement techniques that are based on polarization resistance methods. The virtual experiments are based on the solution of the Laplace's equation for the determination of the electric potential distribution and current densities in the domain of analysis. The domain of the problem is chosen to simulate the common polarization resistance tests which use an external electrode (e.g. a guard ring) covering a representative area of the concrete surface and measure the corrosion rate as an average current over the entire length of the reinforcement underneath, as illustrated in Fig. 1. The polarized length of the rebar in the current investigation is conservatively taken as 300 mm. The domain of the problem is discretized using 5x5 mm rectangular elements, which were found to be the optimum size and shape after a comprehensive sensitivity analysis. It is assumed that the steel under the coverage area is depassivated such that it can be represented by a macrocell consisting of an anode and a cathode that can be characterized by an anode-to-cathode ratio (A/C).

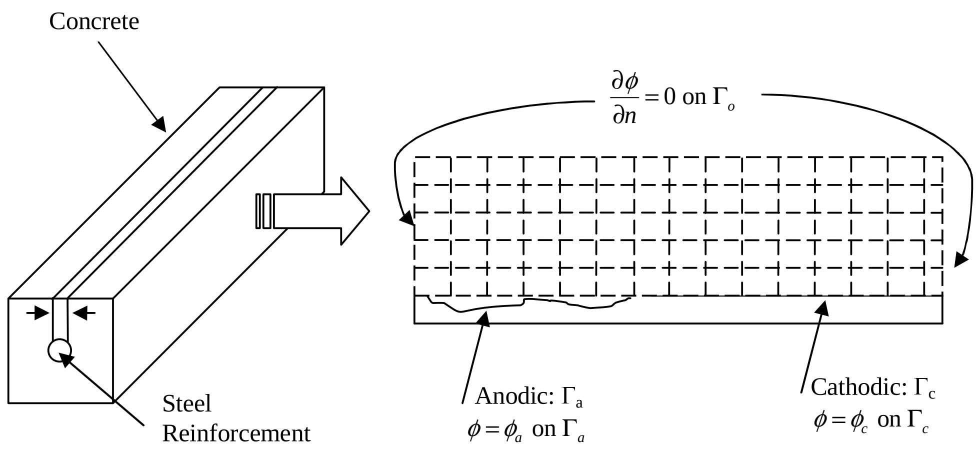


Figure 1: Schematic illustration of the domain and its boundary conditions

Using the model that is briefly described above 16,000 polarization resistance tests were simulated by independently varying kinetic parameters, concrete cover thickness, concrete resistivity and limiting current density for 10 A/C ratios that vary between 0.1 to 1.0. A nonlinear regression analysis on the results was conducted with the help of a statistical analysis software package. Eq. (4) is the resulting closed-form equation, whose coefficients (i.e. $\tau, \gamma, \eta, \kappa, \lambda, \mu, \nu, \varpi, \theta, \vartheta, \chi, \zeta$) are provided in Table 1:

$$i_{corr} = \frac{1}{\tau r^{\gamma}}\left(\eta T d^{\kappa} i_L^{\lambda} + \mu T \nu^{i_L^{\varpi}} + \theta\left(T i_L\right)^{\vartheta} + \chi r^{\gamma} + \zeta\right) \tag{4}$$

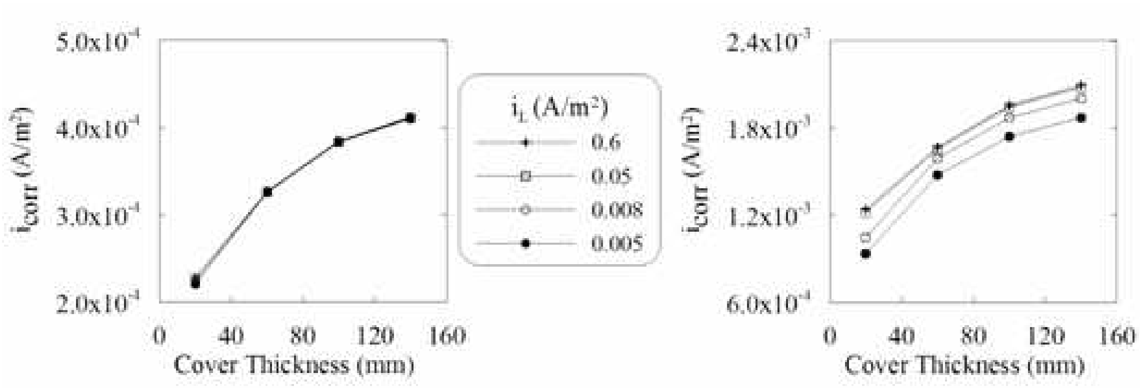
where i_{corr} is the corrosion rate of rebar (A/m^2) and T is the temperature (^{o}K). Using the developed model the effect of cover thickness on the corrosion rate of steel is investigated further.

Table 1: Constants for Eq.(4)

Constant	Value	Constant	Value
τ	1.181102362x10^{-3}	μ	1.23199829 x10^{-11}
η	1.414736274 x10^{-5}	θ	-0.000102886027
ζ	-0.00121155206	ϑ	0.475258097
κ	0.0847693074	χ	5.03368481 x10^{-7}
λ	0.130025167	ν	90487
γ	0.800505851	ϖ	0.0721605536

4. Results and conclusion

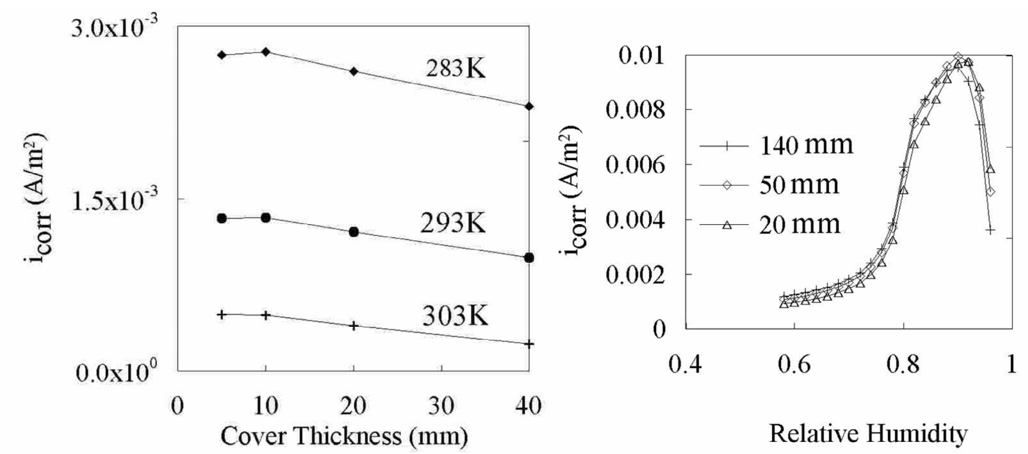
Figure 2 illustrates the effect of cover thickness on the corrosion rate of steel in concrete when it is assumed that the limiting current density is independent from cover thickness; i.e. oxygen availability is not considered in the calculation of corrosion rates. For this assumption, an increase in cover thickness increases the corrosion rates as illustrated in Fig. 2. The increase of corrosion rate with cover thickness is due to better distribution of potential in the bulk of the concrete. As cover thickness increases there is more space available for ions to travel between anodic and cathodic sites, hence, larger length of the anodic and cathodic sections polarize which results in higher corrosion current.



(a) **(b)**

Figure 2: Variation of corrosion rate with cover thickness when the changes in cover thickness are not coupled with limiting current density (at 303^{o}K): (a) r = 5000 Ω.m, (b) r = 650 Ω.m

In reality, since the limiting current density is a function of cover thickness according to Eq. (3), by increasing the cover thickness, the limiting current density should decrease, resulting in a decrease in the corrosion rate. In Fig. 3, the coupled effect of cover thickness and limiting current density (i.e. oxygen availability) on steel corrosion at the different temperatures and relative humidities is illustrated. It can be observed from Fig. 3(a) that the increase of cover thickness actually slightly decreases the corrosion rate. In other words, when the coupled effect is considered, corrosion rate seems not to be significantly affected by the concrete cover thickness (see Fig. 3(a,b)). Although it was shown before that with increase of cover thickness the electric potential resulting from the electrochemical reaction is distributed better in the bulk of the concrete, this effect is compensated by the decreasing limiting current density due to increasing cover thickness. At the end, the two effects cancel each other out. Therefore, when the concrete cover is intact without any large cracks or spalling, cover thickness can be considered not to have a significant effect on the rate of corrosion in concrete shell members.



(a) **(b)**

Figure 3: Variation of corrosion rate with cover thickness when changes in cover thickness are coupled with limiting current density: (a) effect of temperature, (b) effect of relative humidity of concrete

Acknowledgement

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