

An ACI Technical Publication

SYMPOSIUM VOLUME



Fracture Mechanics Applications in Concrete

SP-300



American Concrete Institute
Always advancing



Documents

Document Sharing Hub

Fracture Mechanics Applications in Concrete

Editors:

Cristián Gaedicke, PhD, PE
Amanda Bordelon, PhD, PE



American Concrete Institute

Always advancing

SP-300

First printing, February 2015

Discussion is welcomed for all materials published in this issue and will appear ten months from this journal's date if the discussion is received within four months of the paper's print publication. Discussion of material received after specified dates will be considered individually for publication or private response. ACI Standards published in ACI Journals for public comment have discussion due dates printed with the Standard.

The Institute is not responsible for the statements or opinions expressed in its publications. Institute publications are not able to, nor intended to, supplant individual training, responsibility, or judgment of the user, or the supplier, of the information presented.

The papers in this volume have been reviewed under Institute publication procedures by individuals expert in the subject areas of the papers.

Copyright © 2015
AMERICAN CONCRETE INSTITUTE
38800 Country Club Dr.
Farmington Hills, Michigan 48331

All rights reserved, including rights of reproduction and use in any form or by any means, including the making of copies by any photo process, or by any electronic or mechanical device, printed or written or oral, or recording for sound or visual reproduction or for use in any knowledge or retrieval system or device, unless permission in writing is obtained from the copyright proprietors.

Printed in the United States of America

Editorial production: Aimee Kahaian

ISBN-13: 978-1-942727-05-7

Preface

Fracture Mechanics Applications in Concrete

Over the last decade, new developments in fracture mechanics and the increase of computational capacity have improved the accuracy of modeling the quasi-brittle nature of concrete. This enhanced modeling capacity has facilitated the application of fracture mechanics to solving full-scale design problems and evaluating new concrete and cementitious materials. The papers presented in this special publication focus on the implementation of fracture mechanics techniques in fiber-reinforced concrete, fiber-reinforced polymers, bonding, large structures, beam shear, pavements, and concrete deterioration. Where applicable, the papers compare modeling results with experimental tests. The objective of this publication is to present these recent developments in a manner that facilitates both further research by faculty and students and implementation by civil engineering practitioners. This special publication of the ACI Committee 446 contains ten papers on applications of fracture mechanics in concrete. Many of these papers were originally collected and presented at the Spring Convention in 2012 in Phoenix, AZ. In the first paper, Saouma presents an overview of his 35 years of experience in applying fracture mechanics to concrete structures, providing several fracture mechanics examples in many civil engineering applications as well as a list of future desires for implementation of fracture mechanics by the concrete industry. Next, Mu, Vandenbossche, and Janssen address the use of fracture mechanics to model the bond between Portland cement concrete pavement overlays and existing asphalt pavements. Tompkins, Khazanovich, Bolander, and Stolarski suggest the implementation of fracture information into a lattice finite element model to describe the interface of two composite pavement layers and their likelihood of debonding, while Kim and Bordelon highlight the use of total fracture energy to analyze the fiber component contribution in the fracture behavior of fiber-reinforced concrete. Mobasher, Bonakdar, and Bakhshi propose a back-calculation procedure to determine fracture properties of fiber-reinforced concrete from cyclical flexural fracture tests. Le and Bazant review the problems of accounting for probability in the mechanics-based modeling of strength of concrete, demonstrating the importance of size effect in the reliability analysis of large concrete structures through an example analysis of the Malpasset Dam. Wendner, Strauss, and Novak present an overview of the role of fracture mechanics in the reliability analysis of structures, and Yu and Bazant present a review of recent work on shear failure tests and size effect related to shear for reinforced concrete beams. Rao and Sundaresan use a strut-and-tie model to perform a size-dependent prediction of the ultimate shear strength of reinforced concrete deep beams. Finally, Riveros and Gopalaratnam further analyze the shear response of reinforced concrete deep beams and validate the fracture-mechanics-based numerical model with experiments. We would like to thank the authors who contributed to this special issue as well as the reviewers for their valuable feedback.

Cristián Gaedicke, PhD, PE
Amanda Bordelon, PhD, PE
Editors, ACI 446 SP

TABLE OF CONTENTS

SP-300—1

Applications of Fracture Mechanics to Cementitious Materials:

A Personal Perspective..... 1.1

Authors: Victor E. Saouma

SP-300—2

Quantifying the Mode I Energy Release Rate for interface fracture of Portland

Cement Concrete bonded to Asphalt..... 2.1

Authors: Feng Mu, Julie M. Vandebossche, and Donald J. Janssen

SP-300—3

Simulated fracture in concrete composite pavements using a Couple Lattice
and finite element model 3.1

Authors: D. Tompkins, L. Khazanovich, J.E. Bolander, and H.K. Stolarski

SP-300—4

Determination of Total Fracture Energy for Fiber-Reinforced Concrete 4.1

Authors: Min Ook Kim and Amanda Bordelon

SP-300—5

Back-calculation Procedure for Cyclic Flexural Fracture Tests in

Fiber Reinforced Concrete 5.1

Authors: B. Mobasher, A. Bonakdar, M. Bakhshi

SP-300—6

Mechanics-Based Modeling of Strength Distribution of Concrete Structures:

Problems and Progress 6.1

Authors: Jia-Liang Le and Zdenek P. Bazant

SP-300—7

The role of Fracture Mechanics in Reliability Analyses..... 7.1

Authors: Roman Wendner, Alfred Strauss, and Drahomír Novák

SP-300—8

Shear Strength of Reinforced Concrete Beams: Size Effect and Its

Fracture-Mechanics Basis..... 8.1

Authors: Qiang Yu and Zdenek P. Bazant

SP-300—9

Size dependent prediction of Ultimate Shear Strength of RC Deep Beams using

Refined Strut-and-Tie Model 9.1

Authors: G. Appa Rao and R. Sundaresan

SP-300—10

Shear Response of Reinforced Concrete Deep Beam: Validating Fracture

Mechanics Based Numerical Modelling With Experiments 10.1

Authors: Guillermo A. Riveros, Vellore Gopalaratnam

Paper 01:
Applications of Fracture Mechanics to Cementitious Materials
A Personal Perspective

Victor E. Saouma

Synopsis

It has been well over thirty years since Hillerborg and Bažant presented their landmark papers (cohesive crack and size effect models respectively), and thirty years since the author submitted his Ph.D. dissertation on the application of fracture mechanics to concrete, (Saouma, 1980). Yet, since then, the practical applications of fracture mechanics to concrete structures have been few and far in between.

In this paper, the author shares his experience in trying to apply fracture mechanics not only to concrete structures, but also to other “neighboring” materials such as polymers and ceramics, and he argues for improved collaboration with adjacent disciplines. The underpinnings (experimental, computational) of reported applications will be briefly highlighted.

Finally, the paper concludes with a personal assessment of the current of state in the application of fracture mechanics to concrete structures and venture in some recommendations.

Victor Saouma is a Professor at the University of Colorado in Boulder, and president of the International Association for Fracture Mechanics of Concrete and Concrete Structures (IA-FraMCoS). He received his PhD from Cornell (under the supervision of Anthony Ingraffea). He has multiple areas of research interests: fracture mechanics, deterioration of concrete (he is currently completing a book on *Numerical Modeling of AAR*), numerical modeling of concrete dams and nuclear reactors, real time hybrid simulation, innovative testing procedures.

His research is characterized by an attempt to start from either laboratory work or first principles, followed by model developments, implementation in finite element codes, validation and then application.

His research has been primarily funded by industry, the Electric Power Research Institute (EPRI), and then the Tokyo Electric Power Company (TEPCO) through which he developed over more than 20 years a finite element code Merlin. He has taught (and had research collaboration) with France, Switzerland, Spain, Italy, and Japan.

He has been a consultant on the nonlinear dynamic analysis of very high arch dams in seismic zones, on the fracture mechanics analysis of the delamination of a nuclear power plant, on the prognosis for a massive reinforced concrete structures suffering from AAR, and most recently on the development of Performance based Earthquake Engineering for concrete dams abroad.

1 INTRODUCTION

It has been over thirty years since Hillerborg et al. (1976) and Bažant, Z.P. (1976) (simultaneously) published their landmark papers on the cohesive crack model and the size effect respectively, and thirty years since the author submitted his Ph.D. dissertation on the application of fracture mechanics to concrete, (Saouma, 1980). Since then, there has been countless publications, as well as eight FraMCoS conferences focusing on the fracture of concrete.

Discarding case studies where one performs a numerical simulation of a laboratory test, it is blatantly clear that there are few, very few, reported cases of practical applications of fracture mechanics. From a computational side, this issue was addressed by Emery et al. (2007).

This is indeed a matter of concern, as our infrastructure is aging, and most failure result in micro (under compression), (Hsu et al., 1963) or macro cracks. Whether cracks are the cause failure or are the consequence of (another) failure (mechanism), is another fundamental issues often neglected. Hence, one has to remain hopeful that ultimately fracture mechanics will play a more prominent role in the safety assessment of existing structures, or in forensic studies (which often require a nonlinear analysis) than for the design of new ones¹.

Whereas academic papers emphasize theoretical/experimental aspects and terminate with some sort of an application, this one will start with applications, and conclude with an overview of the theoretical/experimental underpinnings which made these applications possible and credible.

2 Applications

Trying to put things into perspective, the author's PhD thesis, (Saouma, 1980) was probably one of the first doctoral dissertations focusing on the fracture mechanics of concrete structures. Largely inspired by the earlier work of his mentor, (Ingraffea, 1977), it developed a computer program (Finite Element Fracture Analysis Program) with adaptive remeshing for discrete crack propagation. Fig. 1 is an illustrative example of the analysis of the “mythical” reinforced concrete beam widely known as OA-1 tested by Bresler and Scordelis (1961) who were investigating the shear strength of reinforced concrete beams.

Fig. 1(a) is a snap-shot of the mesh after 13 crack propagation increments. The (crude) automatic remeshing is evident, yet such an early analysis (where the only nonlinearity was caused by the nucleation/propagation of a discrete crack governed by linear elastic fracture mechanics) yielded already a nonlinear load-displacement curve, Fig. 1(c). Finally, a realistic crack profile, consistent with ACI-318 prediction was obtained, Fig. 1(b).

Though this program was used for a couple of subsequent years, when the opportunity came (through the financial support of EPRI), an entirely new program (Merlin) was developed (Saouma et al., 2010) and all results presented in this paper are based on it. In the following

¹Though ironically, design is now based on “limit state”, while the analysis remains linear elastic

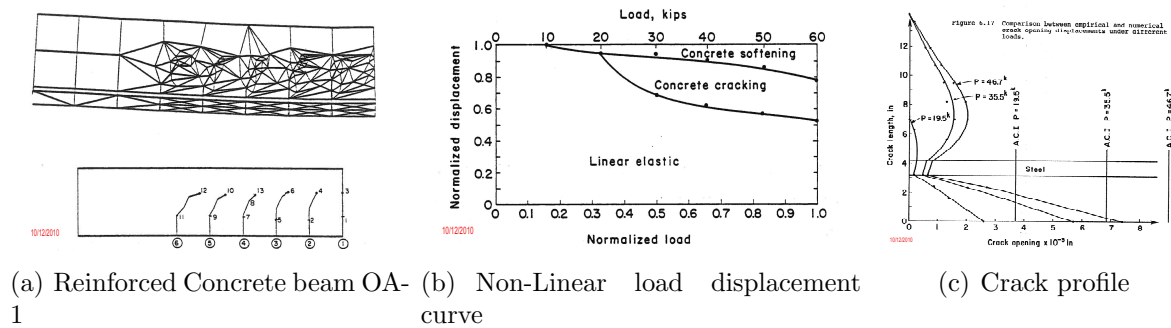


Figure 1: Original figures from the author's PhD thesis in 1982

pages, practical examples of fracture mechanics applications will be shown. The author has been involved in all those analysis, and for obvious reasons structure names, locations, and specific details will be omitted.

2.1 Civil Engineering

Fracture mechanics may be implicitly or explicitly used in concrete structures. Implicitly through code equations which account for the size effect law (such as in the shear strength equation). Unfortunately the most widely referenced design code worldwide (ACI-318) has not yet recognized the need for such a consideration. This aberration can only be attributed to the inability of the research community and the practicing world in this country to communicate and understand each other.

What is more pertinent to the topic of this paper are the explicit applications of fracture mechanics in real structures. Such applications have recently been made possible through the consideration of the cohesive crack model in commercial codes such as those developed by Cervenka Consulting (2010), Dassaut Systemes (2010), or TNO Diana (2010), to various degrees of sophistication. Description of these applications is better left to others for presentation.

A daunting question is the true relevance of fracture mechanics (i.e. to which extent a 10% change in the fracture energy G_F would alter final results) to reinforced concrete structures where cracks are by definition anticipated to occur (in order to mobilize the load carrying capacity of the reinforcement). Whereas this may ultimately limit the applications of fracture mechanics, there is no doubt that massive unreinforced concrete structures are most vulnerable to cracks and indeed should benefit from a fracture mechanics based consideration. Indeed, it is widely believed that the failure of Sleipner A concrete offshore structure was caused by fracturing of the caissons which were not reinforced (an oversight of the finite element analysis), (Selby et al., 1997).

2.1.1 Dams

Dams have historically been catalyst for structural analysis innovations. If many of us are aware that the first reported application of the finite element in civil engineering was precisely

the fracture analysis of a concrete dam by Sims et al. (1964), few know that one of the first applications of the finite differences was to also masonry dams, Richardson (1911).

In the US, energy producing dams safety assessment is governed by the (Federal Energy Regulatory Commission, 2006) which requires identification of the detailed modes of failure, understanding of the progressive behavior and mechanisms of the dam as failure develops, and determination of the “reserve strength” that may be available beyond the nominal limit state strength of the dam. Clearly a simple linear elastic analysis will no longer suffice, and potential failure modes are undoubtedly associated with crack formation and development. This “Potential Failure Mode” is conceptually identical to “Performance Based Design” which is also a new safety paradigm recently introduced by FEMA-349 (2000). Both paradigms would allow engineers to consider spending more resources to achieve quantifiable higher performance thereby reducing risk. Hence, the structural performance of critical structures such as dams, nuclear containment vessels, and offshore structures, could be improved through appropriate non-linear analysis provided that an appropriate model can be used.

Ironically, the *need to perform a fracture mechanics based investigation prior to major rehabilitation* was already recognized (in great part through the author’s work) by the US Army Corps of Engineers in one of its engineering technical letters, Army Corps of Engineers (1993), however this requirement seems to have been rescinded.

Finally, putting things in perspective, it is important to realize that with dams one is dealing with very large concrete structures, that no major distinction is made between cracks and joints (horizontal or vertical) if both are modeled by a fracture mechanics based zero thickness interface such as Cervenka et al. (1998), and that “Achile’s heel” is the joint between rock and concrete. Hence, whereas we assume (in design) that those joints are perfectly closed, they constitute potential cracks once they open.

Seismic Safety Assessment of a Buttress Dam This first example, is an old buttress dam in Japan, Fig. 2(a). Though a relatively small one, there was some concern about its vulnerability to lateral seismic excitation, Fig. 2(b), specially that cracks were already present, Fig. 2(c). To mitigate possible damage, a strengthening plan was put together, Fig. 2(d). However, the safety assessment of the dam did not account for the presence of mild steel reinforcement in the buttresses which could have provided shear strength through dowel effect. At this point, the author got involved in this project.

Modeling reinforcement across the multiple joints was problematic, however joint modeling through interface elements (Sect. 3.1.1) was simple. As such, an experimental program was put together (described below in Sect. 3.2.2) to determine a set of interface element properties equivalent to the one of a joint with reinforcement. Once determined, those properties were assigned to all the joints and thus circumvented the need to model the reinforcements dowel effects.

Finally, a 3D transient nonlinear fracture mechanics was undertaken, Fig. 2(e) and the dam was found to be sufficiently strong to withstand the earthquake, and rehabilitation plans shelved. This was an excellent example of synergy between industry and research in which a reasonable investment in laboratory tests and advanced analysis, yielded substantially larger

savings in unnecessary rehabilitation costs.

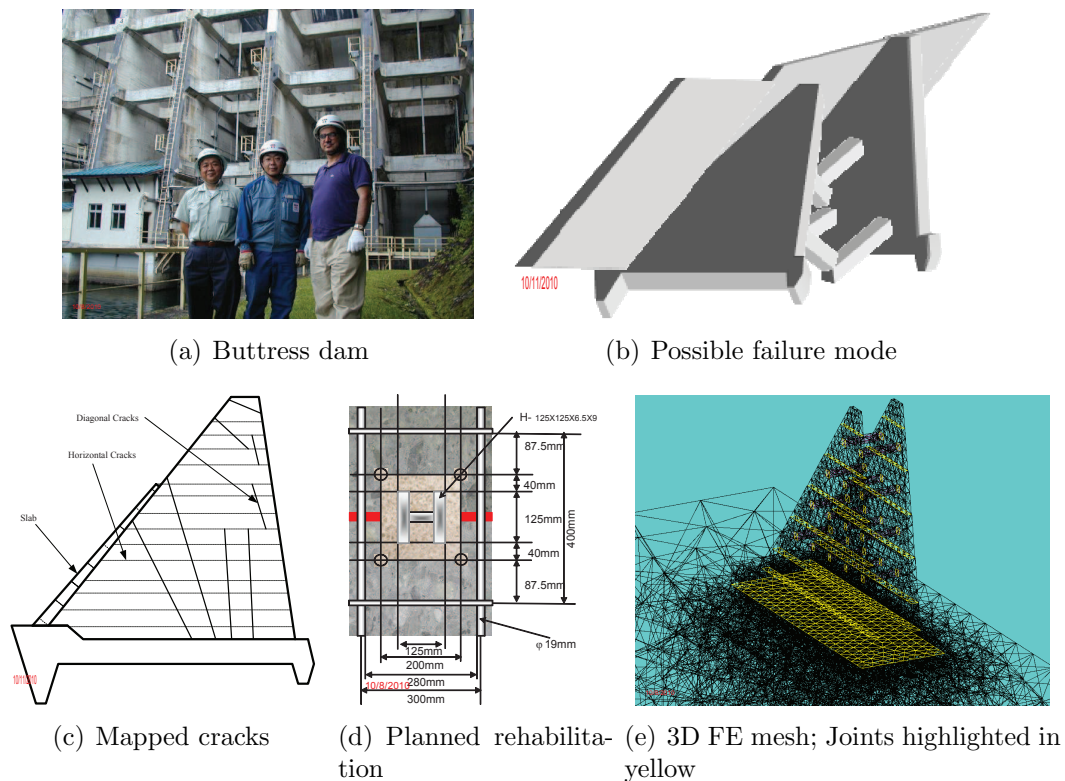


Figure 2: Buttress Dam

Seismic Safety of an Arch Dam What is most striking in the following analysis, Fig. 3(a) is the extensive use of fracture mechanics based interface elements to model vertical joints, and the rock-concrete interface, Fig. 3(b). At first those joints are “zipped” together, and they open or slide when the hyperbolic failure criteria, Eq. 1 (accounting for combined tensile and shear failures) shown in Fig. 14(b) is met. It should be noted that there is no need for adaptive remeshing in these analysis as (most often) it is unlikely that new cracks will develop in the concrete, since the joints constitute “fuses” which would open first and redistribute the stresses.

Safety Assessment of a Complex Gravity Dam This old gravity dam, Fig. 4(a), raised much concern to the surrounding population in light of its apparent deterioration. Much silt had accumulated, it was overtopped numerous times, and simple 2D rigid body analysis failed to yield a sufficiently high factor of safety against sliding. At that point, the author was approached by a consulting firm to perform a 3D fracture mechanics based analysis.

A particularly challenging aspect was the complex geometry: the dam had a shear key, is partially built under existing rock, and has complex geometry which includes a stiff wall at one end.

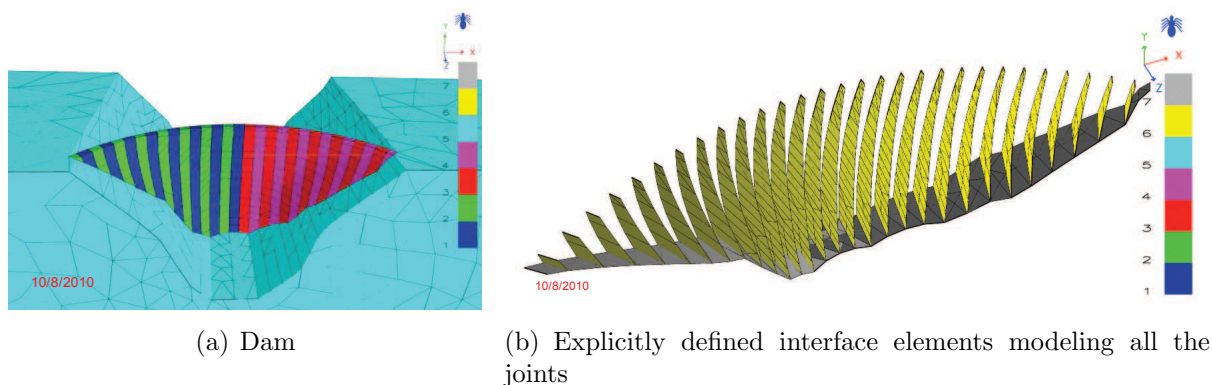


Figure 3: Nagawado Dam

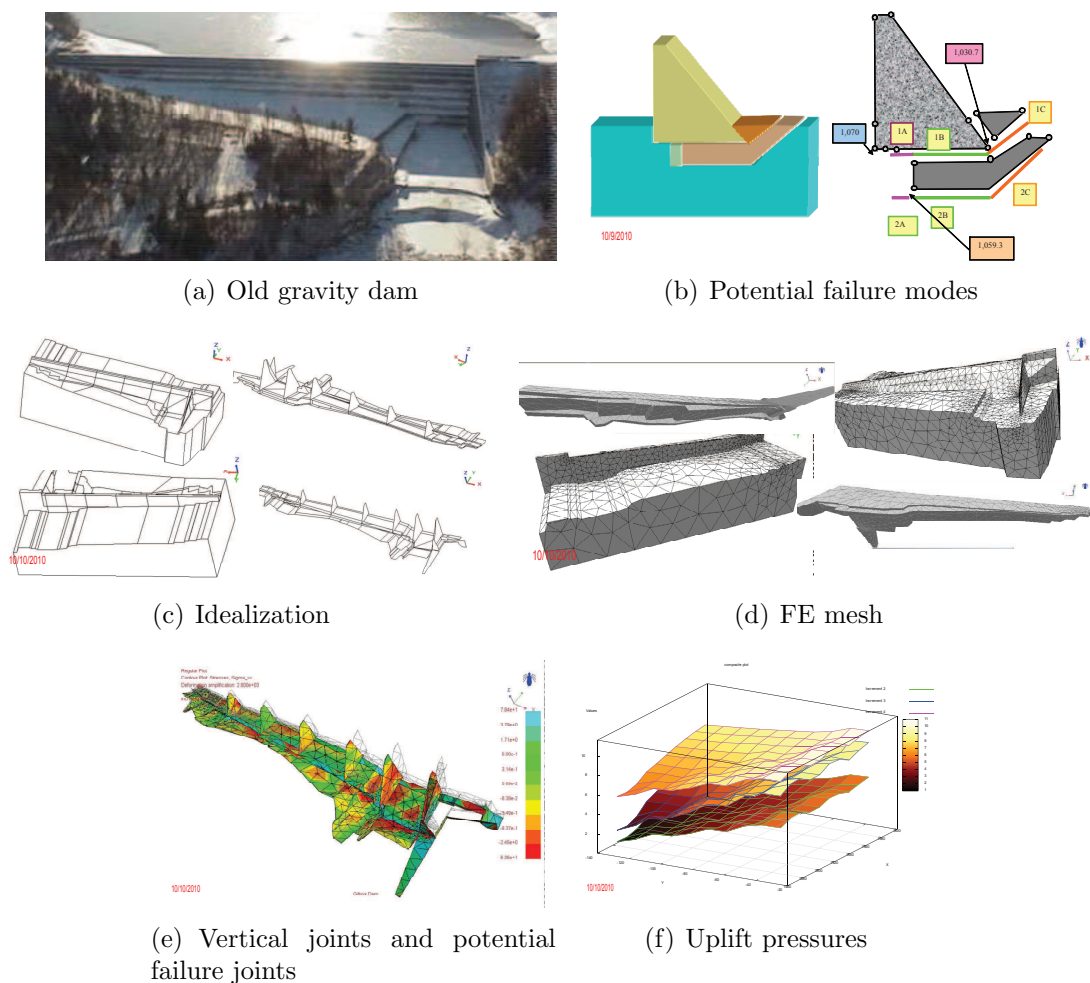


Figure 4: Old gravity dam

Hence, the first task was to identify potential failure modes. Fig. 4(b) is an idealization of the two most likely one. The first cuts across the shear key, goes along the rock/concrete interface and then daylight at approximately 45 degrees. The second assumes a strong

shear key, and failure initiates below it, propagates horizontally, and then again daylight at about 45 degrees. To further complicate matters, this was a 3D analysis with not only varying longitudinal geometry, but also spatially varying rock properties (cohesion, angle of friction). Idealization is shown in Fig. 4(c), while the actual mesh is shown in Fig. 4(d), and the joints (deformed shapes) are in Fig. 4(e).

Given the complex geometry, it was critical to ensure that the mesh was indeed well put together. This was critical as if two nodes across a potential failure modes are not connected by an interface element (i.e. tightly coupled), then no failure would occur (albeit close examination of the deformed shape and stress contour would reveal an irregularity). This was ensured by applying a sufficiently high hydrostatic load upstream to trigger “failure”, and thus ascertain that the potential failure modes can be captured.

Another complexity is the ability to adjust uplift pressures automatically as the crack develop, and most importantly translate the results of a finite element analysis into a safety factor against sliding. The first was addressed by the computer program, and Fig. 4(f) is a graphical representation of the 3D uplift distribution between two parallel vertical joints in terms of the hydrostatic pressure. The second resulted in a slight modification of our computer program to compute the safety factor. In 2D hand calculations, the safety factor can be easily determined from $SF = cL_{uncr} + \Sigma F_v \tan \phi / \Sigma F_h$. In the 3D analysis, this was far more complex to determine, as cohesion, angle of frictions, total horizontal and vertical forces all varied spatially and the vertical had to also include the effects of the uplift pressures. However the inclusion of the interface elements along the failure path greatly facilitated this task as all the required quantities were state variables of the code. Nevertheless, another question was to differentiate between localize (single Gauss point) safety factor *versus* the average one based on the integration over a certain area (such as in between two adjacent vertical joints).

Dam with AAR Many old concrete structures suffer from alkali aggregate reactions (AAR) which result in volumetric expansion. Whereas this may be merely a nuisance for small structures, it is of major concern in massive concrete ones as the constrained expansion is likely to result in structural cracking, inability to operate spillway gates, or worst yet misalignment of the turbines.

AAR is a very slow thermodynamically driven reaction, and many years (over 20) may pass before the dam instrumentation can with certainty indicate that we are in presence of an irreversible deformation (typically upstream, and a crest elevation). Though nothing can stop this reaction, it is of paramount importance for dam owner to assess the evolution (kinetics) of the swelling with appropriate models, (Saouma and Perotti, 2006).

Fig. 5(a) is an arch dam suspected of suffering from AAR (it turned out not to be AAR but another chemically induced reaction in the concrete which results in a volumetric expansion too). As briefly described in Section 3.1.2, the AAR model is in terms of three critical material properties, maximum AAR induced volumetric strain, latency and characteristic times. However whereas those parameters can not be determined from laboratory testing for an existing structure, and since all we have is the historical record of the crest displacement, a system identification procedure must be used. Fig. 5(b) is the graphical user interface of a

Matlab based program which automatically seeks to determine the three material properties by minimizing the square of the error between computed and recorded displacements. It should be noted that each analysis entails a full 3D, 2 years simulations of the dam with 3 months time increments. Fig. 5(c) is the finite element mesh used. It should be noted that vertical joints were not modeled, however it was deemed indispensable to model the rock concrete interfaces with interface elements. Fig. 5(d) shows the internal stresses, and we note the zone of high tensile stresses in the center which may result in some hidden cracks. Indeed a crack was observed along the gallery, and a borehole drilled to determine its extent, Fig. 5(e), as it had not “daylighted” on the downstream face (yet). More details can be found in (Saouma et al., 2007).

It should be noted that this analysis was preceded by a transient thermal analysis to determine internal temperatures at three months interval. Subsequently, input data entailed a 20 years simulation in which temporal variation of thermal load, pool elevation, and uplift pressures had to be accounted for.

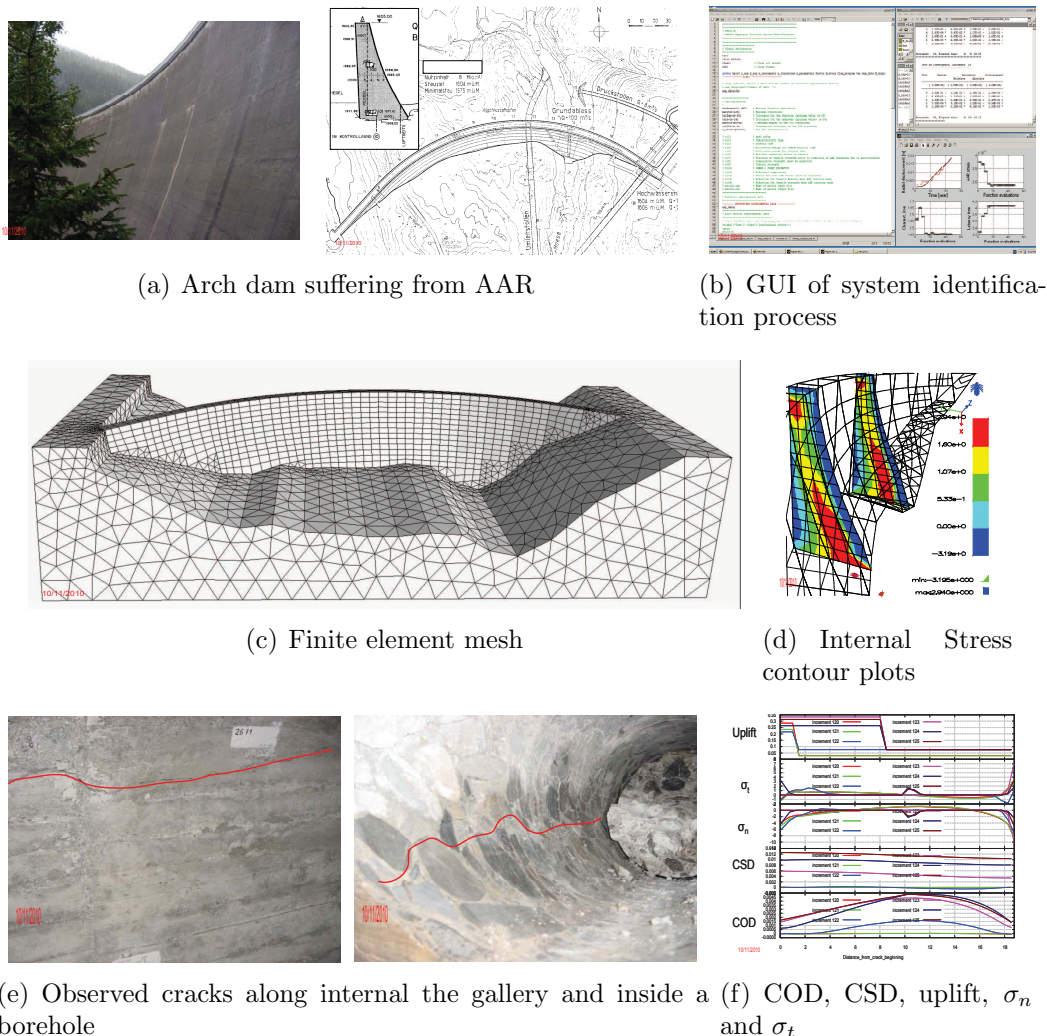


Figure 5: Arch gravity dam suffering from AAR

2.1.2 Deterioration of R/C Decks

Reinforced concrete structures subjected to either saline environment (close to sea), or trenched with de-icing agents are likely to have chloride penetration. By itself, chloride is not detrimental to concrete, however if present in sufficient quantity (as free chloride), it could eventually depassivate the reinforcement. In other words, the high cement Ph ($\simeq 13$) which prevents steel corrosion drops to a critical value, at which point cells are formed and corrosion starts, (Hansen and Saouma, 1999a,b). In these analysis, the chloride diffusion was first modeled, then the corrosion process on the basis of Faraday's law, and finally, based on the volume of rust formed on the cathode, a volumetric expansion of the rebar induced crack nucleation and propagation. Due to the high stress concentration, a nonlinear continuum model is used around the rebar (‘‘smeared crack model’’), while a nonlinear discrete crack one is used for the three macro-cracks formed. Acceleration of the crack propagation due to the ingress of water and its 9% volumetric expansion as it freezes was also accounted for. This analysis proved the viability of a fracture mechanics based solution to improved

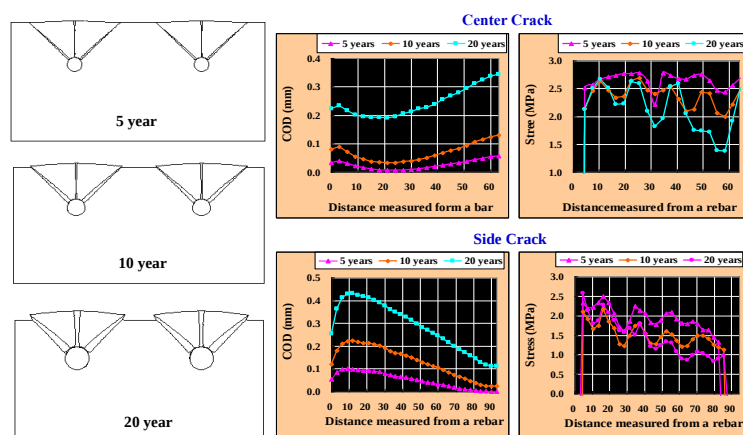
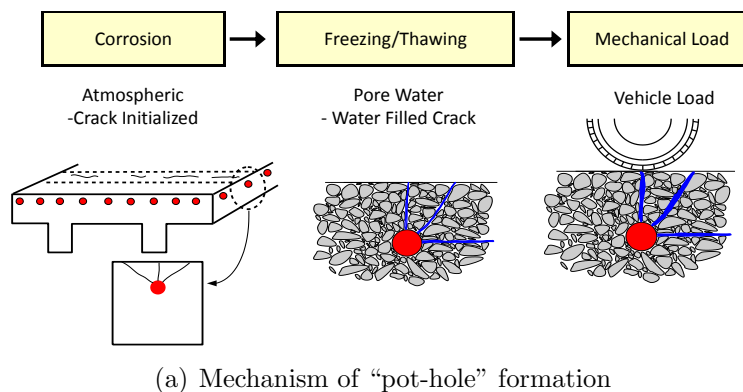


Figure 6: Concrete deterioration due to chloride ingress

understanding of the spalling of the concrete following a long period of ‘‘gestation’’ during which a hair line crack may indeed show on the surface, but for a fully developed one to form and a pothole to form, time is needed.

2.1.3 Nuclear Reactor Containment Vessels

Another major civil infrastructure of which cracking we should be concerned for obvious reasons are nuclear reactor containers. Indeed, the major design constraint is not strength but serviceability, i.e. no leakage should occur, and thus “no cracks allowed” This is ensured through occasional Internal Pressurization Tests where the inside of the container is pressurized up to around 60 psi, and possible leaks identified.

Another major concern with nuclear reactor is the often unanticipated cost of decommissioning. As such, many utility companies find it more cost effective to seek NRC’s approval for life extension from the usual 25-30 years life span to well over 50 years. A major cost associated with this life extension is the replacement of massive steam generator (SGR) as those become so embrittled through radiation exposure, that repair is no longer an option, and full replacement is a must.

Crack in a ring There is a class of French reactors without inner liners. The analysis of the pressurization of these containers has never accounted for the crack pressurization, and the question which begs for an answer is “how bad is it if we neglect this effect”. Such an analysis, was facilitated by the fact that Merlin supports various uplift models for dams, uplift which can be defined as a function of crack opening. Henceforth, the following analysis addresses precisely this problem, (Hansen and Saouma, 2003).

The nature of this problem allowed us to focus on a ring of a fictitious (but realistic) nuclear reactor vessel, (Ghavamian, 2001). The ring has an internal radius of 22,500 mm, and an external one of 23,400 mm. The prestressing tendon is at a radius of 23,150 mm. There are two opposite ribs which house the tendon anchors. The concrete has an elastic modulus E of 31,000 MPa, Poisson’s ratio of 0.22, mass density of 2,500 Kg/m³, compressive strength f'_c of 38.3 MPa, and a tensile strength of 4 MPa. The steel reinforcement consists of four 25 mm diameter rebars with an elastic modulus of 190,000 MPa, Poisson’s ratio of 0.3, mass density of 7,850 Kg/m³, and a yield stress f_y equal to 500 MPa. The prestressing steel consists of two tendons, A_{ps} equal to 5,143 mm² each, initial prestressing force of 4.4 MN, elastic modulus of 190,000 MPa, Poisson’s ratio of 0.3, mass density of 7,850 Kg/m³, and a yield stress f_y equal to 1,814 MPa. Finally, the vertical prestressing is 8.5 MPa.

The ring cross section (away from the anchor zones) is shown in Fig. 7(a), the deformed shape in 7(b),

For this problem, the following simplifying assumptions were made:

1. Perfect bond between steel and concrete (bond slip could indirectly be accounted for by reducing the steel elastic modulus), and reinforcement is composed of 3D truss elements.
2. Initial prestressing is accounted for by specifying a coefficient of thermal expansion α equal to unity for the prestressing steel, and a temperature drop equal to the prestressing force divided by the steel’s Young modulus.
3. In all analyses, the first load increment applies the prestressing force (through nodal equivalent temperature), and vertical prestressing as surface traction.

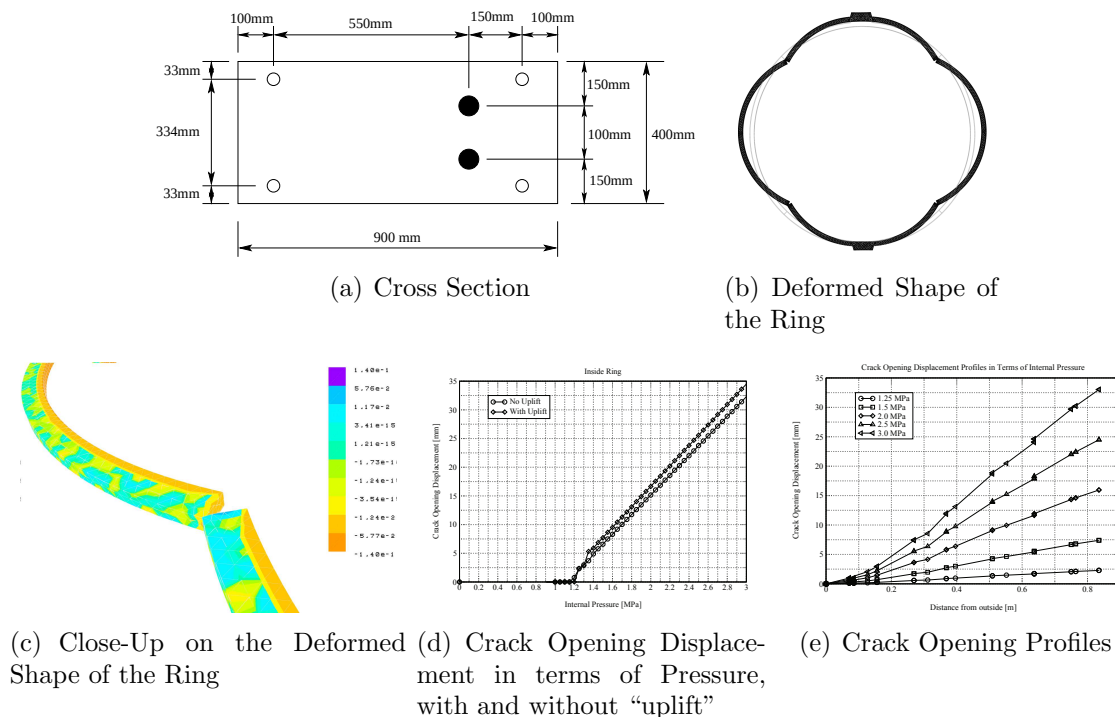


Figure 7: Analysis of a nuclear reactor container ring

- In all subsequent increments, internal pressurization is gradually increased. The vertical tractions are reduced in relation to the increase in internal pressure to simulate the effect of the closed container cap.
- Correctness of load application and boundary conditions is (partially) confirmed through the graphical displays of the deformed mesh (symmetric).

A closer look at the ring, Fig. 7(c) highlights the kinematics of the ring under internal pressurization, crack opening from the inside, and closed crack on the outside.

Since the location of the discrete cracks and the deformation of the ring are symmetric, insight into the performance of the ring in relation to crack opening can be obtained by studying only one of the cracks. If gas leakage to the outside of the container is to be avoided, then one benchmark for "failure" of the ring is the point where the crack opening provides a clear path for gasses to travel from inside the ring to the outside. Crack opening versus internal pressure is plotted in Fig. 7(d) for both analysis cases, whereas the crack profile is shown in Fig. 7(e).

Finally Fig. 7(e) illustrates the deformed shape of the interface element by itself. The small overlap on the exterior of the ring is caused by the finite (yet large) stiffness given to the interface element.

From this analysis, it was determined that accounting for crack pressurization did not substantially increase the crack openings. On the other hand, the discrete nature of the crack model, did provide with a more realistic estimate for the crack opening displacement than a

smear crack model. As mentioned earlier, this estimate of the COD is critical as it provides required information to assess the container permeability to gas leak.

Delamination of a Container Wall As mentioned earlier, the life extension of nuclear reactor may require the replacement of the steam generator. When this is needed, and the circular hatch opening (present in most container) can not be used, then the only alternative is to cut the container wall. Such an operation entails multiple steps which must be carefully planned for:

1. Detensioning of the horizontal and vertical prestressing tendons (which are usually not bonded).
2. Removal of the outer layer of steel reinforcement.
3. Mechanical cut of the concrete walls through the entire wall thickness and liner.
4. Plugging the concrete hole.
5. Retensioning of the cables.

A critical issue is the order in which tendons are detensioned/retensioned, whether tendons in the immediate vicinity of the cut are also to be released, and commensurate nonlinear finite element analysis to assess the safety of such an operation. Analysis which should account for creep, aging, and potential cracking of the wall.

Whereas many such operations were safely performed, it is a matter of public record, (Danielson, 2009), that in one particular case a major delamination crack was observed on the plane defined by the horizontal posttensioning sleeves, Fig. 8(c). This incident has resulted in multi-million dollars costs, (Danielson, 2010).

If such an analysis was to be undertaken using a nonlinear fracture mechanics approach, the key question is how to model the container. Since in this particular case the crack/delamination plane is known, a discrete crack is the most natural candidate. Hence, one would model the concrete as a linear (visco)elastic continuum, however interface elements would be placed along the plane of the delamination crack, Fig. 8(b). To account for the presence of the tendons, discontinuities will be introduced, and those in turn will be subjected to the tendon hoop stresses. Hence, as shown in Fig. 8(d) interface elements (shown in black) will interlace discontinuities along which posttensioning force could be applied.

One would have to model the all too important creep occurring in the concrete, the proper sequence of cable detensioning, and the removal of the concrete. All of this while accounting for the complex geometry which would require different set of material properties in accordance with the internal reinforcement (not explicitly modeled but resulting in modified effective Young modulus).

Indeed, such an analysis did capture what may be the failure mode, Fig. 8(e) where at some stage delamination initiates in the upper/lower part of the cut as shear cracks “coerced” to realign themselves vertically by the closely spaced discontinuities due to the post-tensioning sleeves. It is a matter of public record that additional cracks formed during repair (entailing further cable detensioning), and finally, this structure was deemed unsalvageable and will be decommissioned. Another example of a multi-million dollar loss which *may* have been prevented through better application of the fracture mechanics of concrete.

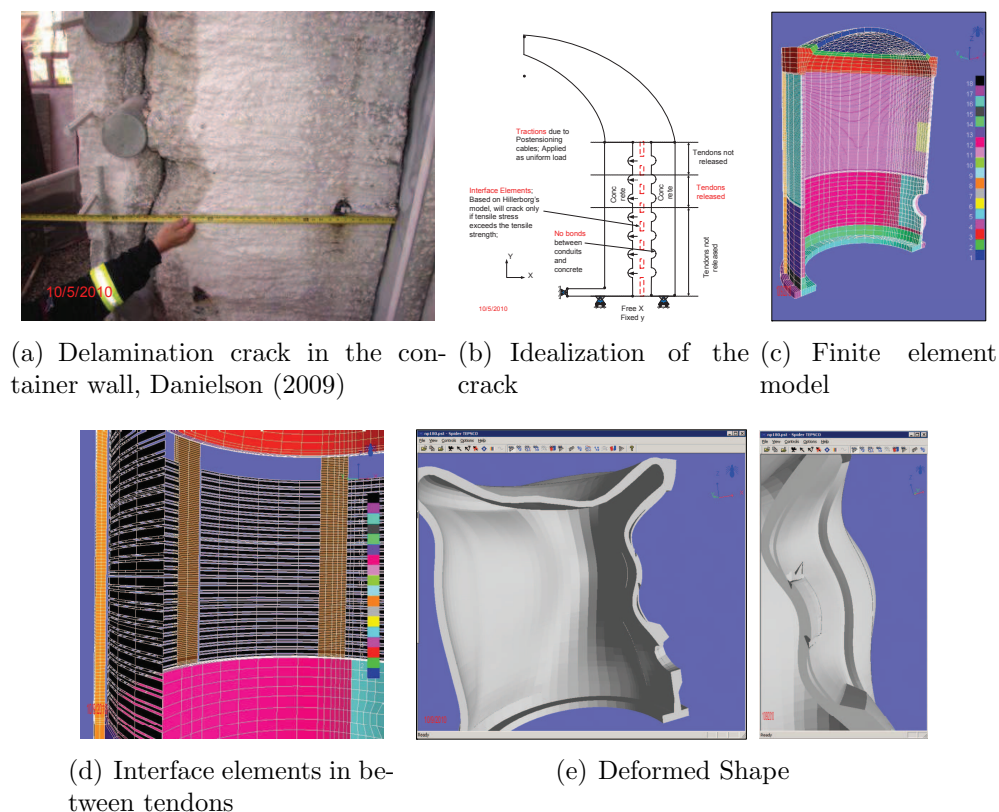


Figure 8: Analysis of a cracked nuclear reactor wall container

2.1.4 Massive R/C Support

Another example of a critical structure subjected to AAR is a massive reinforced concrete support of an electric transmission tower, (Saouma et al., 2007). It was discovered that a segment (painted in white in Fig. 9(a)) was undergoing such a reaction as evidenced by extensive cracking. Since AAR results also in a degradation of the elastic modulus and the tensile strength, the main concern was whether such a structure could resist a strong earthquake following years of deterioration.

This was addressed by first putting together a finite element mesh which accounted for the heavy reinforcement shown in Fig. 9(b) and winker springs to account for the soil structure interaction. Various models were considered, and it was determined that modeling the full structure with localized AAR, Fig. 9(d) yielded the best (and excellent) strain correlation with those measured *in-situ*. In this analysis, the smeared crack model of Merlin was used, it is based on the fracture plastic model developed by Červenka and Červenka (1999). *in-situ* cracks have been monitored for many years and thoroughly mapped. Average strain was determined by averaging the ratio of crack width over multiple length scale.

Finally, the code was modified to allow a dynamic restart following the 20 years AAR simulation to perform a seismic evaluation. The restart was essential, as it was critical for the dynamic simulation to begin with the existing static stress field, and corresponding

damaged elastic modulus and tensile strength.

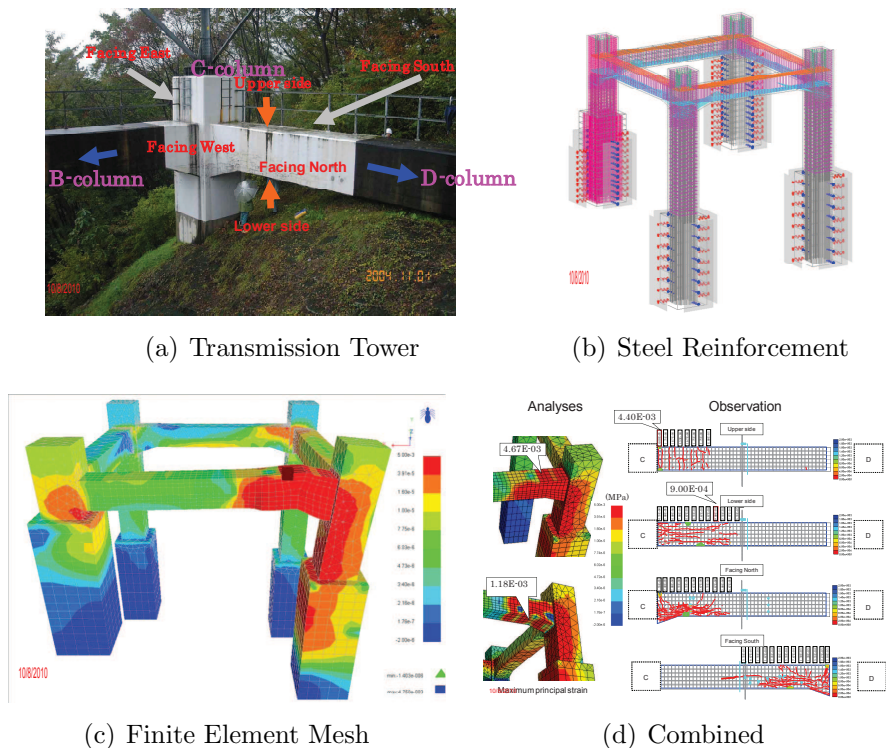


Figure 9: Transmission Tower

2.1.5 Dynamic Analysis

A potential field of application for fracture mechanics is the seismically induced cracking of reinforced concrete frames. Presently, there are two adjacent communities which do not intersect: earthquake and fracture mechanics of concrete. In the former, a major concern is the resilience of structures, which in turn depend on the response of individual components. Indeed, a current major issue is the strength of r/c column (designed prior to modern design codes) with insufficient shear reinforcement in California. These columns are likely to have two orthogonal shear cracks, followed by an axial failure.

Indeed, the author has performed tests on 1/3 scale column with low shear reinforcement, with three dynamically controlled actuators: constant axial force, zero rotation, and imposed lateral displacement, (Ghannoum et al., 2012) using the setup shown in Fig. 10(a), which in turn resulted in the failure shown in Fig. 10(b). Subsequently Saouma et al. (2012) conducted series of real time hybrid simulation (numerical simulation of most of the frame, concurrent tightly coupled testing of a r/c with low shear resistance), Fig. 11(a) and it was shown that this innovative paradigm could potentially replace shake table tests at a fraction of the cost, hence replace the experimental setup of Fig. 11(b) by the one in Fig. 11(c).

Henceforth, the point of this note is that there should be a congruence of efforts of those two communities to achieve a more fundamental understanding of the fracture mechanism

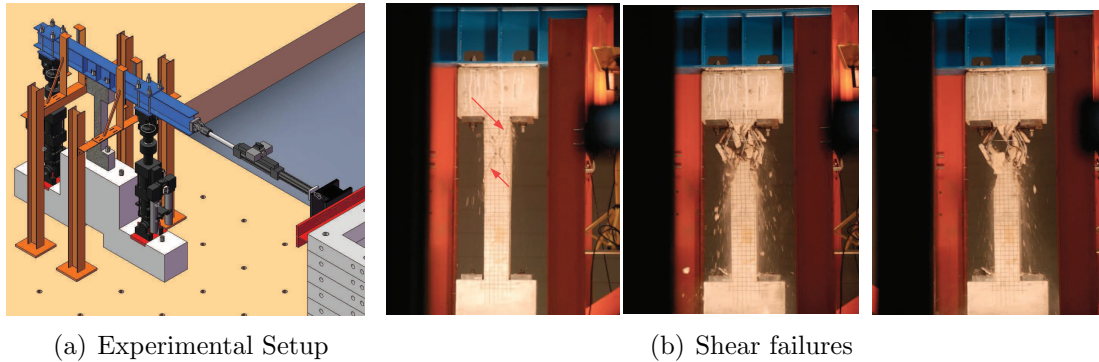


Figure 10: Rate Effects

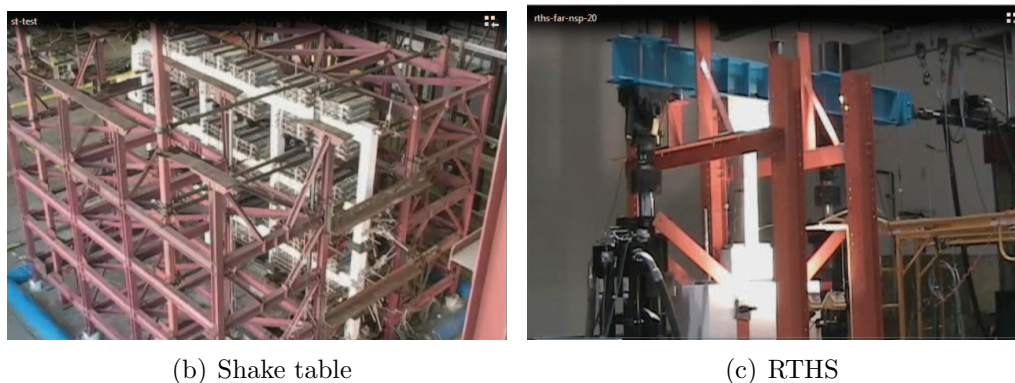
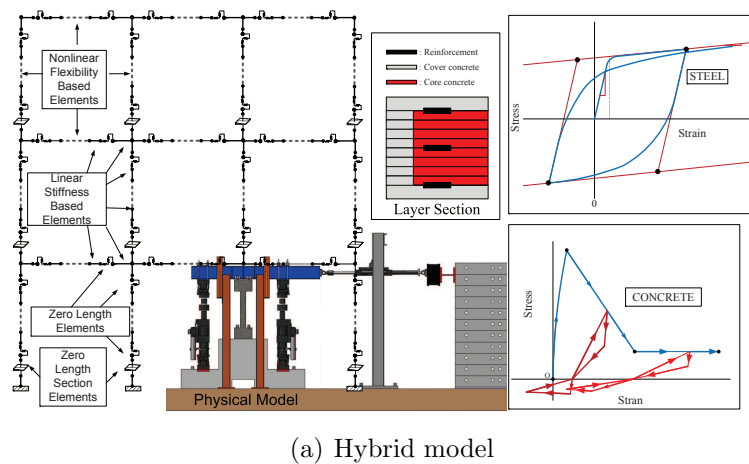


Figure 11: Real time hybrid simulation to capture shear cracking

which ultimately leads to failure.

2.2 Others

2.2.1 Ceramic-Composites

Ceramics are being increasingly used in a number of industrial components, Table 2.2.1. Yet

Application	Size (m)	Loading
Electronics	$10^{-4} - 10^{-3}$	Thermal fatigue, mechanical loading
Cutting Tools	$10^{-3} - 10^{-2}$	High concentrated mechanical and thermal loads, shock loads
Engine components	$10^{-3} - 0.1$	High thermal and mechanical fatigue, shock loads, concentrated loads, stress corrosion
Turbine Blades	$10^{-2} - 1$	Concentrated mechanical loads, thermal and mechanical fatigue, stress corrosion

Table 1: Advanced Ceramics Applications

they suffer from a major drawback: they are inherently brittle, or quasi-brittle. A closer look reveals much similarity with concrete as illustrated in Table 2. An important similarity

Charact.	Ceramics	Concrete
	Heterogeneous	
Specimen prep.	Expensive	Simple
Application	Numerous	Few
Bulk of research	Phenomenological	Theor./Num.
Prevalent Cohesive Stress Model	Dugdale	Hillerborg
Modeling	LEFM “Infancy”	NLFM “Mature”
Energy Dissipation	At Crack tip	Along FPZ
Crack Res.	K_{Ic} ; Pa $\sqrt{\text{m}}$	G_F ; J/m ²

Table 2: Ceramics *versus* Concrete

with concrete is the presence of grains which will confer the ability to transfer cohesive stresses, and potentially exhibit a size effect, Fig. 12(a). Yet, most research in ceramics has limited itself to LEFM, and occasionally at simple plasticity based models around Dugdale’s postulate.

As such, the “concrete community” had much opportunity to develop ever increasingly complex models as there appeared to be few applications. On the other hand, the “ceramic community” is confronted with both manufacturing challenges and multiple applications, and as such may not have developed models of the same sophistication as the one developed for the more mundane concrete material.

Though not specifically addressing a practical application, this section was inserted to highlight the potential cross-fertilization potential between those two “communities”.

Hence, the objective was to explore the possibility of applying concrete models to ceramics, (Saouma et al., 2002). This was accomplished in performing a nonlinear fracture mechanics analysis (based on Hillerborg’s model) of the experimental tests of Sbaizero et al. (1998) who

tested Rectangular bars 3mm x4mm x20mm ($B \times W \times L$) with an a/W of 0.5. Cohesive crack parameters were first adjusted to yield comparable load deformation curves, and then those were frozen and analysis of different specimen sizes performed. This resulted in a crisply defined size effect, Fig. 12(b). This is important as ceramics are used in many different applications of varying sizes, Fig. 12(c) and as such size effect adjusted properties should be used. More recent studies, (Saouma et al., 2006) looked at the presence of residual stresses in

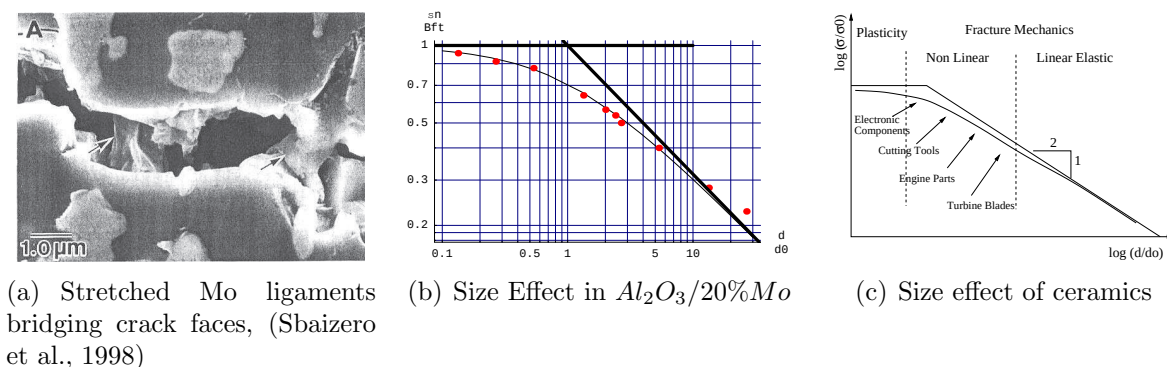


Figure 12: Nonlinear fracture mechanics simulation of ceramics

ceramics, and the effect of physical transformation on increased fracture toughness, (Saouma et al., 2005).

2.2.2 Polymers

The last application focuses on polymers used for solid rocket propellants. A composite solid rocket propellant is generally made of a saline oxidizer, such as ammonium perchlorate, and a rubbery binder, such as Polybutadiene, which acts as a fuel. Most rockets today use AP and Hydroxyl-Terminated PolyButadiene, mixed with other minor but fundamental constituents. The salt is milled in one, two or three different grain sizes, and mixed with the liquid HTPB. Hardener, Bonding Agent and Catalyser are then added, and the propellant is cast in the rocket case, generally made of some high-performance steel alloy or of composite material. A mandrel shapes the propellant surface. The motor is cured at high temperature in an oven, and the propellant solidifies, becoming a rubber-like, filled elastomer. However, a critical step is the extraction of the mandrel, Fig. 13(a) as it may result in microcracks at the bore, Fig. 13(b). In the presence of a crack, binder filaments bridge the crack partially connecting the two sides, the bonding agent anchors the oxidizer particles to the binder with strong bonds, Fig. 13(d) and confers higher toughness to the material, generating crack bridging through binder anchoring at two opposite oxidizer grains. Hence, along a portion of the crack, there is a transfer of stress through the stretched binder, giving rise to a fracture process zone, Fig. 13(e). Another great analogy with concrete, and a prime candidate for nonlinear fracture mechanics.

Should there be a crack, the mechanical/thermal stress may cause crack growth resulting in additional surface area. The risk is that there would be combustion inside the crack, Fig. 13(c) producing pressures much higher than the designed maximum pressure resulting in the

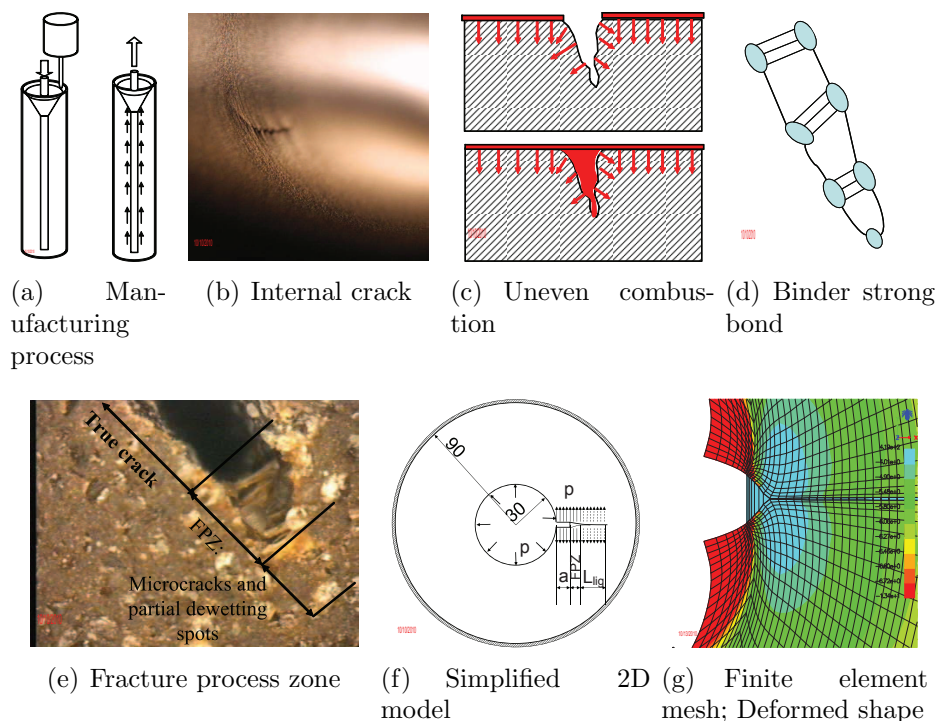


Figure 13: Fracture of polymers

rocket explosion.

This is not an academic exercise, as indeed there has been a motor explosion of a Titan IV PQM, (Chang et al., 1995). Since then, fracture mechanics to predict the service life of a motor and the determination of the critical crack size as an acceptance criterion for high-valued SRM (such as the Titan IV boosters), has been partially published, (Liu, 2003).

Whereas most of the early fracture mechanics applications were driven by LEFM, there were only timid attempts to use the more appropriate cohesive crack model. In light of the above, an experimental program was first set up to determine the fracture energy of solid propellants, Sect. 3.2.3, (Tussiwand et al., 2006), and then numerical simulation using the fictitious crack model were undertaken, Fig. 13(g).

This work is currently being extended to couple the nonlinear fracture mechanics code with the one which simulates the combustion. Hence at each time step, the updated pressure and crack profile are passed to the stress analysis code from the combustion one.

3 Underpinnings

The previously presented applications could not have been achieved by merely using a computer program as a “black box”, and without a good grasp of both the physics of the problem and first principles. Furthermore, in many cases, models had to be based on proper testing, and finally computer models must be validated.

Henceforth, this last section will briefly present the underpinnings of the applications presented.

3.1 Theoretical Supports

3.1.1 Constitutive Models

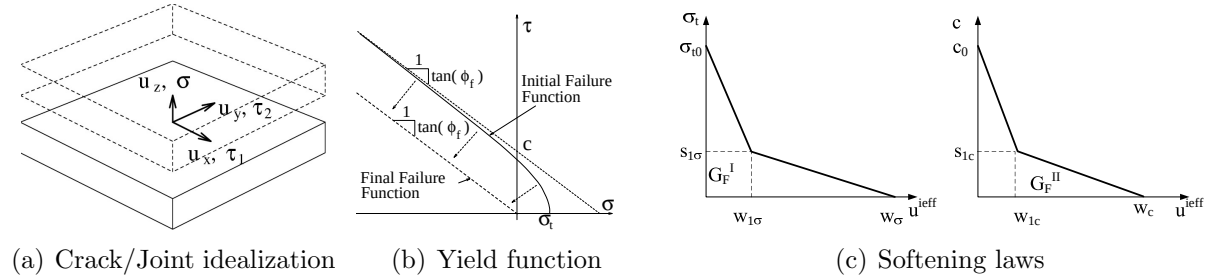


Figure 14: Interface crack model

Most of the reported applications used the so-called discrete crack model. It is based on an extension of the model of Hillerborg et al. (1976) and implemented in an interface element, Fig. 14(a), as pioneered by Goodman, R.E. and Taylor, R.C. and Brekke, T.C. (1968). More details can be found in (Cervenka et al., 1998).

The model is based on the following assumptions: The major premises upon which the model is developed are: 1) Shear strength depends on the normal stress; 2) Softening is present both in shear and tension; 3) There is a residual shear strength due to the friction along the interface, which depends on the compressive normal stress; 4) Reduction in strength, i.e. softening, is caused by crack formation; 5) There is a zero normal and shear stiffness when the interface is totally destroyed; 6) Under compressive normal stresses neither the shear and nor the normal stiffnesses decrease to zero; In addition, should a compressive stress be introduced in the normal direction following a full crack opening, two faces of the interface come to contact, and both tangential and normal stiffnesses become nonzero; 7) Irreversible relative displacements are caused by broken segments of the interface material and by friction between the two crack surfaces; 8) Roughness of the interface causes opening displacements (i.e. dilatancy) when subjected to sliding displacements; 9) The dilatancy vanishes with increasing sliding or opening displacements.

The hyperbolic yield criteria, Fig. 14(b), is given by:

$$F = (\tau_1^2 + \tau_2^2) - 2c \tan(\phi_f)(\sigma_t - \sigma) - \tan^2(\phi_f)(\sigma^2 - \sigma_t^2) = 0 \quad (1)$$

where: c is the cohesion, ϕ_f the angle of friction, σ_t the tensile strength of the interface; τ_1 and τ_2 the two tangential components of the interface traction vector, and σ the normal traction component. Softening laws, Fig. 14(c) for both mode I and II are defined. An extension of this model to account for reverse cyclic load was also developed, (Puntel et al., 2006).

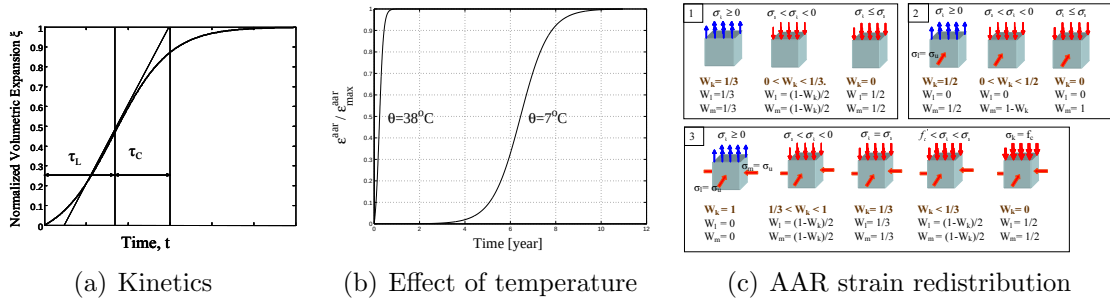


Figure 15: AAR model

3.1.2 AAR

In our model, (Saouma and Perotti, 2006), the AAR expansion is considered to be a volumetric one, which rate is given by the following function

$$\dot{\epsilon}_V^{AAR}(t) = \Gamma_t(f'_t|w_c, \sigma_I|COD_{max})\Gamma_c(\bar{\sigma}, f'_c)g(h)\dot{\xi}(t, \theta) \epsilon^\infty|_{\theta=\theta_0} \quad (2)$$

where ϵ^∞ is the final volumetric expansion as determined from laboratory tests at temperature θ_0 . $0 \leq \Gamma_t \leq 1$ is a parameter which reduces the expansion in the presence of large tensile stresses (macrocracks absorbing the gel), f'_t the tensile strength, and σ_I the major (tensile) principal stress. Similarly, $0 \leq \Gamma_c \leq 1$ is a parameter which accounts for the absorption of the gel due to compressive induced stresses, $\bar{\sigma}$ and f'_c are the hydrostatic stress, and the compressive strength of the concrete. $0 \leq f(h) \leq 1$ is a function of the humidity (set to zero if the humidity is below 80%), $\dot{\xi}(t, \theta)$ the kinetics law given by, Fig. 15(a)

$$\xi(t, \theta) = \frac{1 - e^{-\frac{t}{\tau_c(\theta)}}}{1 + e^{-\frac{(t-\tau_l(\theta))}{\tau_c(\theta)}}}; \quad \tau_i(\theta) = \tau_i(\theta_0) \exp \left[U_i \left(\frac{1}{\theta} - \frac{1}{\theta_0} \right) \right] \quad (3)$$

where τ_i stands for wither τ_l or τ_c and are the latency and characteristic times respectively. The first corresponds to the inflexion point, and the second is defined in terms of the intersection of the tangent at τ_L with the asymptotic unit value of ξ . The latency time τ_l and characteristic times, τ_c are expressed in terms of the absolute temperature ($\theta^\circ K = 273 + T^\circ C$) and the corresponding activation energies. U_l and U_c are the activation energies minimum energy required to trigger the reaction for the latency and characteristic times respectively. The effect of temperature on the rate of expansion is clearly noticeable in , Fig. 15(b).

Once the volumetric AAR strain is determined, it is decomposed into a tensorial strains based on a “weight” determined from the principal stresses: 1) Expansion is prevented if stress exceeds about 8 MPa; 2) restrained expansion is redirected in other directions; in accordance to the three weight factors associated with the principal stresses, Fig. 15(c).

Finally, there is a degradation of the tensile strength and elastic modulus.

3.2 Laboratory Testing

Any complex nonlinear, fracture mechanics based, analysis must rely on proper models (as described above), and those in turn must rely on proper experimental tests. This section

will detail two such set of experiments.

3.2.1 Concrete

First, large scale tests with specimens as large as 5' and 3" maximum size aggregates were tested, Fig. 16(a), to determine the fracture energy of "dam concrete", (Saouma et al., 1991). Direct tension tests on 36x6 inch cross section specimens were also conducted, (Slowik et al., 1996). Since we have at the base of a dam a large shear stress, the potential shear crack propagation was also investigated, Fig. 16(d), (Slowik and V., 1996). Then the interaction of fluid and fracture was investigated, Fig. 16(b), to determine the variation of uplift pressures in terms of crack opening, (Brühwiler and Saouma, 1995a) and (Brühwiler and Saouma, 1995b). Much later, the effect of reverse cyclic loading on the degradation of concrete roughness, Fig. 16(e), was experimentally derived, (Puntel and Saouma, 2008). Finally, as fracture mechanics is likely to be more widely used in the context of structural assessment, one must test concrete cores recovered from site. Fig. 16(c) is such a test on an 8" core inside an environmental chamber at 80°C to accelerate creep fracture.

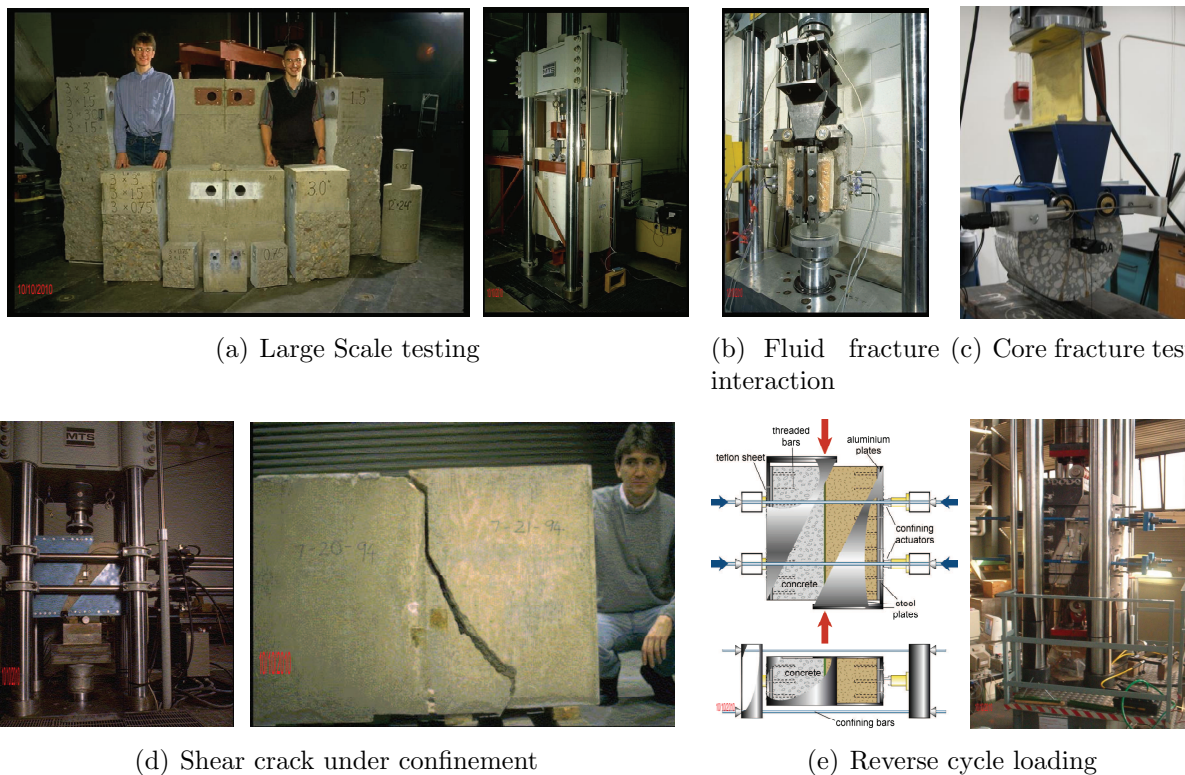


Figure 16: Laboratory testing for concrete

3.2.2 Project Specific Testing

Whereas laboratory tests are most often performed to determine generic models, at times, they can be project specific. In support of the analysis reported in Sect. 2.1.1, laboratory

tests were performed on large jointed concrete blocks crossed by 19 mm smooth rebars (identical to the ones found in the dam). The objective being to determine the global response (which would account for the dowel effect), and then determine a set of interface joint properties which could provide a similar response prior to a full dam analysis. Fig. 17(a) describes the test, the actual specimen is shown in Fig. 17(b), and the corresponding finite element model in 17(c).

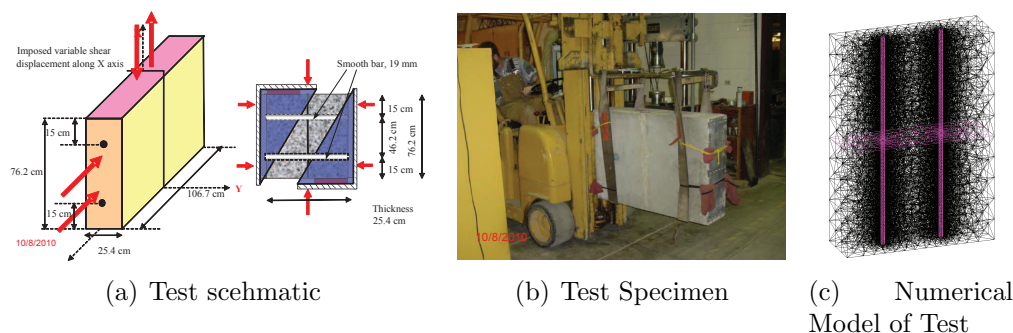


Figure 17: Project specific testing

3.2.3 Polymers

This last test is an illustration of the cross-fertilization between concrete and other material. Fig. 18(a) is a (enclosed) wedge splitting test of a polymer used as a rocket propellant, and the fracture process zone is shown in Fig. 18(a) where image analysis is used to determine (and control) crack openings. Finally Fig. 18(c) is the resulting master curve for the fracture energy (which accounts for rate and temperature of this visco-elastic material).

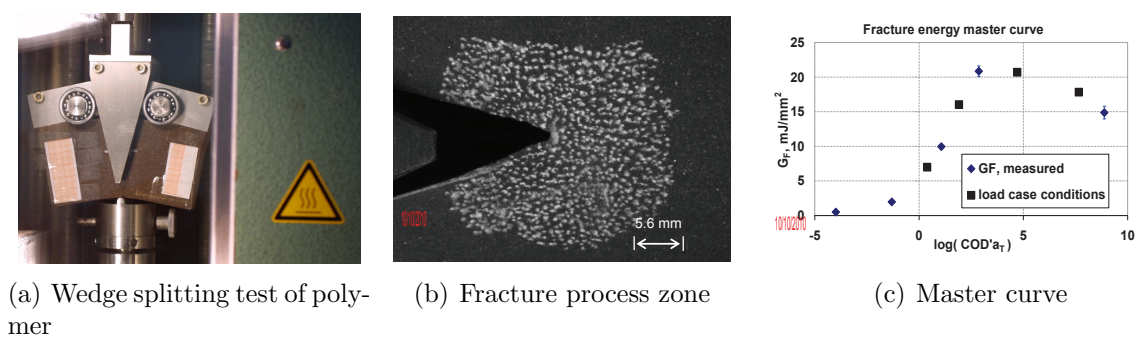


Figure 18: Fracture of polymers

3.3 Centrifuge Based Validations

Finally, any finite element program must be validated. Though it is customary to validate a program by reanalyzing the very same tests from which the constitutive model was (implicitly or explicitly) derived, this is barely acceptable. In our case, some aspects of

Merlin capabilities were assessed through centrifuge tests of a dam model subjected to varying hydrostatic load, (Gillan et al., 2004). Then a more ambitious test was performed by dynamically exciting a dam model inside a centrifuge, (Uchita et al., 2005).

Fig. 19(a) is the large centrifuge of Obayashi Corporation in Tokyo where the test was performed. Fig. 19(b), shows the dam model (1/30th scale). Pre-test calculations were made by Merlin, and Fig. 19(c) is an illustration of the crack and location of some of the instruments. Accelerometers were mounted at the base and the crest. The dam was subjected to a series of six increasing harmonics. Measured and predicted crest accelerations are shown in Fig. 19(d), and the transfer functions in Fig. 19(e). From the last two figures, it was evident that Merlin captured well the nonlinear dynamic response of the dam, and thus provided additional confidence in its use for nonlinear transient fracture mechanics analysis. In this case the noticeable shift in the transfer function response is a clear indication of

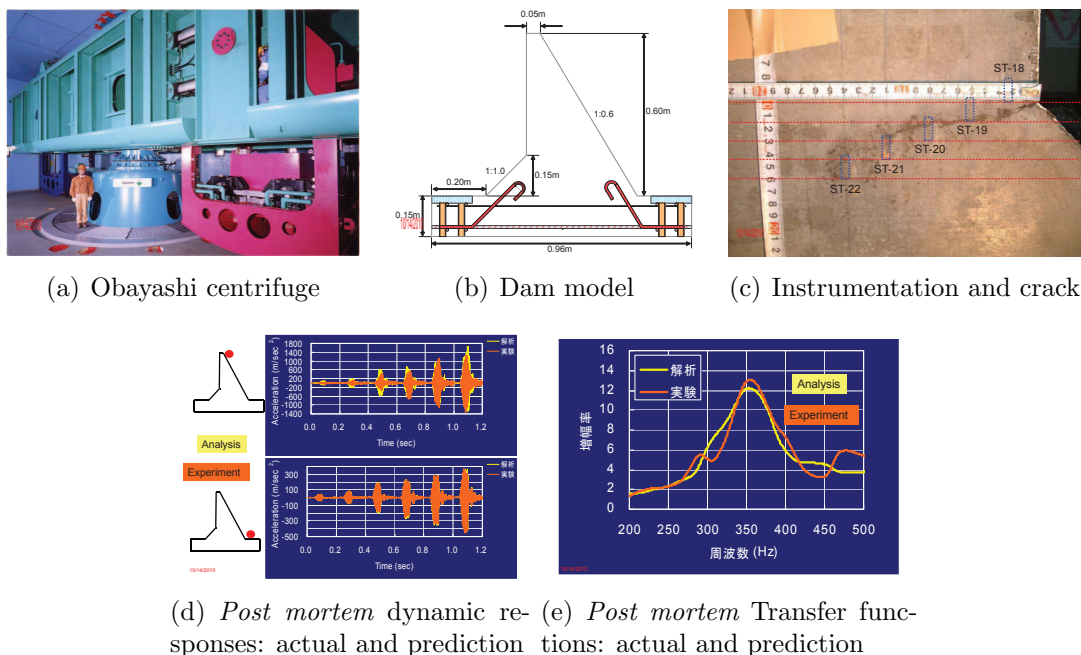


Figure 19: Centrifuge/shake table tests for Merlin validation

internal damage (cracking) which induced a reduction in stiffness, and henceforth a left shift of the transfer function.

4 Needs

Based on the experience gained through this reported work and his international collaborations, the author identifies the following issues as deserving particular attention:

1. Recognition by the practicing engineers of the multitude of talent and expertise in academia waiting to be tapped for the solution of complex problems entailing fracture of concrete (contrarily to the business world which does not hesitate to call upon the expertise of the academic one).

2. Federal agencies tend to over-rely on “big name” companies to task them with complex analysis. From his limited experience, the author found most (but not all) of them to be ill equipped for such tasks (though they are very well versed in large scale linear elastic analyses).
3. Despite numerous attempts, none of the reported research has been funded by the National Science Foundation, the Bureau of Reclamation or the US Army Corps of Engineers. Yet similar research is well funded by government agencies in France or Switzerland. Ultimately, this country is at risk of a serious technological gap in (reinforced) concrete nonlinear modeling. This was already evident by the lack of participation in some of the major international Benchmarks.
4. University professors should be encouraged to broaden their research goals to address complex interdisciplinary, multi-scale practical problems. Fundamental research will always be essential, and should be recognized as such. Yet, it is presumptuous to think that one single narrow engineering discipline (such as fracture mechanics) by itself can help us solve practical problems. Fracture mechanics should be *part of* the solution and not *the* solution.
5. ACI-318 should seriously consider impact of size effect laws on some of its provisions (shear equation in particular).
6. The earthquake engineering community should seek guidance from the concrete fracture mechanics one to achieve improved fundamental understanding of seismically induced fractures/failures.
7. Fracture mechanics is much more relevant to assess the integrity of existing structures than for the design of new ones. Henceforth, core based specimens should be favored over prismatic ones which can not be extracted from existing sites.
8. One critical research need is creep fracture (where the only major experimental research was undertaken about 20 years ago by Zhou (1992)).
9. A certification program to “validate” computer codes with nonlinear fracture mechanics capabilities should be put in place. Such an effort could be spearheaded by NRC, FEMA or EPRI.
10. Students should receive some exposure to the fracture mechanics of concrete (which is very different than the one for metals traditionally taught).

5 Conclusions

Thirty years after his dissertation on the application of fracture mechanics to concrete, this paper was an attempt to summarize some of the practical applications undertaken by the author. Within “traditional” civil engineering, they covered concrete dams, nuclear reactor containment vessels, massively reinforced concrete structures. Other application to ceramics and polymers were also presented.

Subsequently, the underpinnings of the numerical models were briefly presented. More specifically constitutive laws, laboratory testing and validation.

Finally, personal (possibly provocative) remarks on deficiencies and needs were made.

6 Acknowledgments

The author would like to first, and foremost, thank his Ph.D. advisor, Prof. A. Ingraffea, who supervised his dissertation, thirty years ago.

Sincere thanks are also extended to a few “visionary” persons from the practical word who came to accept fracture mechanics. This includes Howard Boggs (Bureau of Reclamation, Ret), Chong Chiu (PII), Doug Morris (EPRI), Giuseppe Tussiwand (Bayern Chemie), Yusidoi Uchita (TEPCO).

Last, but not least special thanks are to my closest collaborators, colleagues who made Merlin a reality, Jan Červenka, Gay Haussmann, Ronald Reich, and Yoshinori Yagome.

Institutionally, the Electric Power Research Institute (EPRI), and the Tokyo Electric Power Company (TEPCO) financial support is gratefully acknowledged.

References

- Army Corps of Engineers (1993). Fracture mechanics analysis of gravity lock monolith. ETL 1110-2-344, Department of the Army, US Army Corps of Engineers, Washington, D.C.
- Bazant, Z.P. (1976). Instability, ductility and size effect in strain softening concrete. *Journal of Engng. Mech., ASCE*, 102(2):331–344.
- Bresler, B. and Scordelis, A. (1961). Shear strength of reinforced concrete beams. *Journal of the American Concrete Institute*, 60(1):51–74.
- Brühwiler, E. and Saouma, V. (1995a). Water fracture interaction in concrete; part i fracture properties. *ACI Materials Journal*, 92(3):296–303.
- Brühwiler, E. and Saouma, V. (1995b). Water fracture interaction in concrete; part ii hydrostatic pressures cracks. *ACI Materials Journal*, 92(4):383–390.
- Červenka, J. and Červenka, V. (1999). Three dimensional combined fracture-plastic material model for concrete. In *5th U.S. National Congress on Computational Mechanics*, Boulder, CO.
- Cervenka, J., Chandra, J., and Saouma, V. (1998). Mixed mode fracture of cementitious bimaterial interfaces; part ii: Numerical simulation. *Engineering Fracture Mechanics*, 60(1):95–107.
- Cervenka Consulting (2010). Atena. <http://www.cervenka.cz/products/atenal>.
- Chang, I., Patel, N. R., and Yang, S. (1995). Titan iv motor failure and redesign analyses. *Journal of Spacecraft and Rockets*, 32(4):612–618.
- Danielson, R. (2009). Cracked wall to keep progress energy’s crystal river nuclear plant off-line longer. <http://www.tampabay.com/news/business/energy/cracked-wall-to-keep-progress-energys-crystal-river-nuclear-plant-off-line/1053210>.
- Danielson, R. (2010). Crack at crystal river nuclear power plant explained.
- Dassaut Systemes (2010). Abaqus. http://www.simulia.com/solutions/architecture_buildings.html.
- Emery, J., Hochhalther, J., and Ingraffea, A. (2007). Computational fracture mechanics of concrete structures: A retrospective through multiple lenses. In Carpinteri, A., P., G., Ferro, G., and Plizzari, G., editors, *Proceedings 6th International Conference on Fracture Mechanics of Concrete and Concrete Structures (FraMCoS-VI)*, pages 3–15, Catania. Taylor & Francis.
- Federal Energy Regulatory Commission (2006). Engineering guidelines for the evaluation of hy-

- dropower projects. Technical report, Federal Energy Regulatory Commission, Office of Hydropower Licensing.
- FEMA-349 (2000). Fema 349-action plan for performance based seismic design. Technical report, Federal Emergency Management Agency.
- Ghannoum, W., Saouma, V., Haussmann, G., Polkinghorne, K., and Eck, M. (2012). Experimental investigations of loading rate effects in reinforced concrete columns. *ASCE Journal of Structural Engineering*, pages 1032–1041.
- Ghavamian, S. (2001). Etude “anneau”, tranche courante de l’enceinte de flammaville. Technical report, EDF, Theroetical Mechanics and Applications R& D Division.
- Gillan, C., Saouma, V., and Shimp, T. (2004). Centrifuge tests of concrete dams. *International Journal of Water Power and Dam Construction*, pages 38–41.
- Goodman, R.E. and Taylor, R.C. and Brekke, T.C. (1968). A Model for the Mechanics of Jointed Rocks. *J. of the Soil Mechanics and Foundations Division ASCE*, 94:637–659.
- Hansen, E. and Saouma, V. (1999a). Numerical simulation of concrete deterioration; part i: Chloride diffusion. *ACI Material Journal*, 96(2):173–180.
- Hansen, E. and Saouma, V. (1999b). Numerical simulation of concrete deterioration; part ii: Steel corrosion and concrete fracture. *ACI Material Journal*, 96(3):331–339.
- Hansen, E. and Saouma, V. (2003). 3 d nonlinear finite element/fracture mechanics analysis of a pressurized nuclear reactor container ring. *Nuclear Engineering and Design*, 225:1–10.
- Hillerborg, A., Mod  er, M., and Petersson, P. (1976). Analysis of crack formation and crack growth in concrete by means of fracture mechanics and finite elements. *Cement and Concrete Research*, 6(6):773–782.
- Hsu, T., Slate, F., Sturman, G., and Winter, G. (1963). Micro cracking of plain concrete and the shape of the stress-strain curve. *Journal of the American Concrete Institute*, 60:209–224.
- Ingraffea, A. (1977). *Discrete Fracture Propagation in Rocks: Laboratory Tests and Finite Element Analysis*. PhD thesis, University of Colorado.
- Liu, C. (2003). The application of fracture mechanics to predict the critical initial crack length. Technical Report AFRL-PR-ED-TP-2003-057, Edwards AFB CA Space and Missile Propulsion Div.
- Puntel, E., Bolzon, G., and Saouma, V. (2006). A fracture mechanics based model for joints under cyclic loading. *ASCE J. of Engineering Mechanics*, 132(11):1151–1159.
- Puntel, E. and Saouma, V. (2008). Experimental behavior of concrete joints under cyclic loading. *ASCE J. of Structural Engineering*, 134(9):1558–1568.
- Richardson, L. (1911). The approximate arithmetical solution by finite differences of physical problems involving differential equations, with an application to the stresses in a masonry dam. *Philosophical Transactions of the Royal Society, Series A.*, 210:307–357.
- Saouma, V. (1980). *Finite Element Analysis of Reinforced Concrete; a Fracture Mechanics Approach*. PhD thesis, Cornell University, Department of Structural Engineering.
- Saouma, V., Broz, J., Br  hwiler, E., and Boggs, H. (1991). Effect of aggregate and specimen size on fracture properties of dam concrete. *ASCE, Journal of Materials in Civil Engineering*, 3(3):204–218.
- Saouma, V., Chang, S., and Sbaizero, O. (2005). Numerical simulation of transformation ZrO_2 -toughened Al_2O_3 . *European Journal of Ceramics*. Submitted.
- Saouma, V., Chang, S., and Sbaizero, O. (2006). Numerical simulation of thermal residual stress in mo- and feal⁻-toughened Al_2O_3 . *Composites Part B: Engineering*, 37:550–555.
- Saouma, V., Haussman, G., Kang, D., and Ghannoum, W. (2012). Real time hybrid simulation of a non ductile reinforced concrete frame. *in-print ASCE J. of Structural Engineering*.

- Saouma, V., Natekar, D., and Sbaizero, O. (2002). Nonlinear finite element analysis and size effect study of a metal-reinforced ceramics-composite. *Materials Science and Engineering A*, 323:129–137.
- Saouma, V. and Perotti, L. (2006). Constitutive model for alkali aggregate reactions. *ACI Materials Journal*, 103(3):194–202.
- Saouma, V., Perotti, L., and Shimp, T. (2007). Stress analysis of concrete structures subjected to alkali-aggregate reactions. *ACI Materials Journal*, 104(5):532–541.
- Saouma, V., Červenka, J., and Reich, R. (2010). Merlin finite element user's manual. <http://civil.colorado.edu/~saouma/pdf/users.pdf>.
- Sbaizero, O., Pezzotti, G., and Nishida, T. (1998). Fracture energy and R-curve behavior of Al_2O_3/Mo composites. *Acta. Mater.*, 46(2):681–87.
- Selby, R., Vecchio, F., and Collins, M. (1997). The failure of an offshore platform. *Concrete International*, 19:28–35.
- Sims, F., Rhodes, J., and Clough, R. (1964). Cracking in norfolk dam. *J. of the American Concrete Institute*.
- Slowik, V., Saouma, V., and Thompson, A. (1996). Large scale direct tension test of concrete. *Cement and Concrete Research*, 26(6):949–954.
- Slowik, V. and V., S. (1996). Behavior of rock concrete interfaces in gravity dams. Technical report, Electric Power Research Institute, Palo-Alto, CA. Final Research Report to be Submitted by the University of Colorado.
- TNO Diana (2010). Diana. <http://tnodiana.com/DIANA-concrete>.
- Tussiwand, G., Saouma, V., and Terzenbach, R. (2006). Fracture mechanics of a solid rocket motor using AP-HTPB based composite propellant; part i material testing. *Journal of Propulsion and Power*. Submitted for Publication.
- Uchita, Y., Shimp, T., and Saouma, V. (2005). Dynamic centrifuge tests of concrete dams. *Earthquake Engineering and Structural Dynamics*, 34:1467–1487.
- Zhou, F. (1992). *Time-dependent Crack Growth and Fracture in Concrete*. PhD thesis, Division of Building Materials Lund Institute of Technology.

**QUANTIFYING THE MODE I ENERGY RELEASE RATE FOR INTERFACE FRACTURE OF
PORTLAND CEMENT CONCRETE BONDED TO ASPHALT**

Feng Mu, Julie M. Vandenbossche, and Donald J. Janssen

ABSTRACT

The performance of ultra-thin and thin concrete overlays on existing asphalt pavements, commonly referred to as whitetopping, requires the bond between the two layers to be maintained throughout the service life. Tensile stresses generated at the interface and adjacent to the joints due to slab curvature and the continuous nature of the underlying HMA contribute to the localized debonding of these two layers. A wedge splitting test was employed in this study to quantify the mode I loading induced fracture along the Portland cement concrete/hot mix asphalt interface of specimens designed specifically for this test. An analytical model is developed to characterize the response of the specimen under this loading condition. The model is used to assist in identifying the initiation as well as the growth of the interfacial crack, and for establishing the interfacial energy release rate. Using this model, the initiation as well as the growth of the interfacial crack is predicted for specimens with different surface textures at the interface.

Keywords: **Interface** Bond, Interface fracture, Mode I energy release rate, Whitetopping, Wedge splitting test, Milling

Feng Mu is a pavement engineer. He received his BSCE in the Department of Hydraulic Engineering at Tsinghua University, Beijing, China, his MPhil in Geotechnical Engineering at the University of Hong Kong, and his PhD in the Civil and Environmental Engineering at the University of Pittsburgh. His research interests include the characterization of interfacial composite material behavior and the design, maintenance, preservation and rehabilitation of pavements and other structures.

Julie Vandenbossche is an assistant professor in the Civil and Environmental Engineering Department at University of Pittsburgh, PA. She received her BSCE and her MSCE in the Civil and Environmental Engineering at Michigan State University, MI and PhD in Civil Engineering at the University of Minnesota. Her research interests include material characterization and the analysis, design and rehabilitation of concrete pavements. She is also interested in the instrumentation and modeling of pavement behavior.

Donald Janssen is an associate professor in the Department of Civil Engineering at University of Washington, WA. He received his BSCE, MSCE and PhD in the Civil and Environmental Engineering at the University of Illinois, IL. His research interests are durability and physical properties of Portland cement concrete.

INTRODUCTION

Whitetopping is a rehabilitation technique in which a thin (4-6 in [100-150 mm]) or ultra-thin (4 in [100 mm] or thinner) Portland cement concrete (PCC) overlay is placed to rehabilitate a distressed hot mix asphalt (HMA) pavement. The practice of whitetopping has been in existence in the U.S. for more than 80 years, with the first documented project constructed in Indiana in 1918 (ACPA, 2012). The whitetoppings constructed early on were not intentionally bonded to the underlying HMA and thus the overlay had to be relatively thick (8 in [200 mm] or thicker) to survive the traffic and environmental loads. In 1991, the first bonded ultra-thin whitetopping constructed in Louisville, Kentucky led others to try this technology (Cole and Mohsen, 1993). It has been determined that whitetopping can be much thinner, provided that an effective bond is available for increasing the composite stiffness of the structure and thereby reducing the tensile stress in the overlay. This emerging technology, i.e. thin and ultrathin bonded whitetopping with short (6 ft [3 m] or smaller) joint spacing, quickly gained its popularity. From 1992 to 2001 approximately 300 ultrathin whitetoppings were paved in 35 states and during the 1990s over 50% of the states have constructed whitetopping projects (Rasmussen and Rozycki, 2004). Part of this resurgence can be attributed to the development of procedures for designing these overlays in the mid to late 1990's. Meanwhile, test sections were constructed and instrumented in Minnesota, Iowa, Colorado, and a number of other states (Rasmussen and Rozycki, 2004) to gain a better understanding of the failure mechanisms. Now with over a decade of performance data collected from these sections, the failure mechanism and design procedures for thin and ultra-thin whitetopping are being re-evaluated. As one component of this effort, this study introduces a framework that quantifies the interface debonding for the PCC/HMA interface in terms of the fracture energy release rate in order to account for the debonding mechanism in the design process.

One of the most significant differences of thin and ultra-thin whitetopping relative to conventional concrete overlays is the interface bond. Currently, the way to consider interface debonding during design is to increase the design stress in the overlay by a constant percentage. The adjustment is based on either engineering experience or limited dynamic strain measurements (Mu and Vandenbossche, 2011), which is very empirical and unreliable. However, the failing of this bond is gradual due to repeated traffic and environmental loadings (Vandenbossche and Fagerness, 2002; Chabot et al., 2008), which eventually results in the premature failure of whitetopping.

Interface debonding occurs when the fracture energy release rate exceeds a critical value. The failure can be further broken down into mode I (tensile) and mode II (shear) depending on the nature of the stress contributing to the fracture. Stress contributing to mode I debonding develops when differential deflections occur across the joint between the loaded and unloaded slabs. Differential deflections can be significant for thin and ultrathin whitetopping because they are typically undoweled and the reduced thickness relative to the saw cut at joints provides little aggregate interlock that contributes to load transfer. As the loaded side of the joint deflects downward independently from the unloaded side, the asphalt tends to be peeled away from the concrete overlay. Stresses contributing to mode I debonding can also develop due to the curling/warping of the overlay slabs as a result of temperature/moisture gradients. When the slabs curl/warp up, the continuous asphalt tends to stay flat and thus debonding is induced at the PCC/HMA interface. As a first step in quantitatively modeling the interface debonding, this study focuses on establishing the critical fracture energy release rate only for debonding under mode I loading.

One key factor for the bond strength is the asphalt surface texture prior to the overlay. The observation that milling the HMA prior to the overlay contributed to higher bond strengths was first made for test sections in Iowa (Grove et al., 1996). Milling the HMA created a macrotexture that resulted in higher bond strengths, as measured using the Iowa shear test. The results from a study funded by the Colorado Department of Transportation provided similar results (Tarr et al., 1998). In the study, strain measurements were made for in-service pavements. It was found that the dynamic strains for overlays placed on milled HMA were 25% lower than those with unmilled HMA. Chabot et al. (2008) also found that the interface failure was much rougher when the HMA was shot blasted. In their research, the whitetopping was constructed on a test track and testing cores from the test section revealed that it was tougher to fracture an interface with increased texture.

In this paper, it first provided a description of the test procedure and test specimens developed to characterize the PCC/HMA interface fracture under mode I loading. Then, an analytical model was developed to analyze the load-displacement response from the test. Finally, using the model the fracture energy release rate was calculated and the effect of the HMA surface texture on the interface bond was discussed.

WEDGE SPLITTING TEST

In this study, the wedge splitting test (WST) (Brühwiler & Wittmann, 1990) was employed to produce the PCC/HMA interface fracture under mode I loading. The specimens consist of half HMA and half PCC, as shown in Figure 1. A small square of material is removed from the central portion of the specimen where a clip gage resides for monitoring the crack opening. A notch is cut in the central portion of this region at the interface to establish the location for crack initiation. Furthermore, a guide notch of approximately ¼ in [6 mm] deep is made on both sides of the specimen to ensure that the crack propagates along the interface.

Experiment configuration

The setup of the WST is illustrated in Figure 1.



Figure 1-WST configuration and WST specimen.

A steel cap is placed on each half of the specimen of which round bearings rest. These bearings, together with the wedge, transform the axial load from the actuator to horizontal splitting forces that can be calculated using Equation (1). The WST specimen rests on two linear supports, which are aligned with the bearings as well as the center of gravity of each half specimen. This alignment minimizes the moment induced by the vertical force relative to the interface. A clip gage is instrumented at the mouth of the starter notch to monitor the crack mouth opening displacement (CMOD). The WST specimen is then loaded to failure, with a constant rate of CMOD.

$$F_H = \frac{F_V + P_{wedge}}{2 \tan \theta} \quad (1)$$

where

F_H =splitting force,

F_V =axial load by the actuator,

P_{wedge} =11.8 lbf [0.05 kN], the weight of the wedge used in this study and

θ =15°, the wedge angle used in this study.

Specimen preparation

Each half of the specimen has a geometry of approximately 6 in [150 mm] (L) × 6 in [150 mm] (W) × 3.5~4.5 in [90-110 mm] (T). The HMA was obtained from Route 50 in Bridgeville, PA, which was an in-service pavement being reconstructed. Prior to the removal of the HMA pavement, portions of the HMA surface were milled while other areas were left unmilled. The thickness of the HMA was approximately 3.5 in [90 mm] for the milled sections and 4.5 in [110 mm] for the unmilled sections. The sand patch test (ASTM-E965-96, 2006) was carried out on both the

milled and unmilled HMA specimens to obtain a characteristic depth that correlated to the surface roughness. The difference in the roughness between the milled and unmilled HMA surfaces can be clearly seen in Figure 2.



Figure 2-Surface of (a) milled and (b) unmilled HMA specimen.

The composite specimens were made by first brushing the asphalt to remove free particles and then casting concrete that was wet cured for 28 days. The cleaning prior to the overlay placement was employed to mimic the application of whitetopping overlay in the field. The HMA portion of the specimen was wrapped with tapes during the curing in order to minimize the moisture degradation of the HMA. Before testing, all the specimens were air dried under room temperature.

Material property

PCC property-The concrete tested was cast in two batches. The material properties for both mixtures were measured immediately before the WST was performed and the results are presented in Table 1. Although concrete from Batch 1 appears to be stiffer and stronger than Batch 2, they both present a much higher stiffness than the asphalt.

Table 1-PCC material properties.

		Number of samples	Average	Standard deviation
Elastic modulus, 10 ⁶ psi [GPa]	Batch 1	3	4.1 [28.3]	0.03 [0.2]
	Batch 2	3	3.9 [26.8]	0.14 [1.0]
Poisson's ratio	Batch 1	3	0.21	0.01
	Batch 2	3	0.19	0.01
Compressive strength, psi [MPa]	Batch 1	3	4,170 [28.8]	60 [0.4]
	Batch 2	3	3,330 [23.0]	450 [3.1]
Testing day	Batch 1	3	28	
	Batch 2	3	161	

HMA property-A power function is usually used to express the relaxation modulus for a visco-elastic material, as shown in Equation (2).

$$E(t) = E_{ref} t^{-m} \quad (2)$$

where

E_{ref} = reference modulus and

m= relaxation factor that characterizes the viscosity of the material.

Relaxation modulus is rarely determined directly through laboratory testing. Instead, it is usually converted from creep compliance based on the inverse Laplace transform of the Prony series. The unconfined uniaxial creep test (Brown et al., 2001) could be carried out to characterize the visco-elastic behavior of the HMA. In the test, a constant static axial compression is applied at several levels and the resultant strains are recorded. The measured stress-strain data is then fitted into Equation (2) to obtain the m values. In this study, however, the empirical correlation in the AASHTO Mechanistic-Empirical pavement design guide (ARA, 2004) was used to estimate the range of m -value, as shown in Equation (3).

$$m = 1.1628 - 0.00185 \cdot Temp - 0.04596 \cdot Va - 0.01126 \cdot VFA + 0.00247 \cdot Pen77 + 0.001683 \cdot Temp \cdot Pen77^{0.4605} \quad (3)$$

where

$Temp$ =temperature at which the creep compliance is measured, °F,

Va =air voids, %

VFA =voids filled with asphalt, % and $VFA = V_{eff}/(Va + V_{eff})$,

V_{eff} = effective binder content, % and

$Pen77 = 10^{290.5013 - (81177.288 + 257.0694 \cdot 10^{(A + 2.72973 \cdot VTS)})^{0.5}}$ where A and VTS are 9.715 and -3.208

For VFA between 65% and 75% for typical HMA paving mix, the m -value should be from 0.65 to 0.8.

Loading rate

A CMOD rate of 0.02 in/min [0.5 mm/min] was employed in this study. The performance of HMA, and most likely the interface, is highly dependent on the CMOD rate, so it was carefully selected. A few pilot specimens as listed in Table 2 were first loaded under only the wedge weight so that the resultant CMOD rate R_{Wedge} could be obtained. The CMOD rate used for external loading must be significantly higher than the one for only wedge load so that the influence of wedge weight would become negligible. However, on the other hand, the CMOD rate should not be too high to cause abrupt failure. It should be also noted that the influence of the wedge varies with the notch depth. In Table 2, the CMOD rates for specimens with deep notches were much higher than those with shallow notches.

Table 2-CMOD rates under only wedge weight loading.

Specimen	Size (H×W), in [mm]	Normalized notch depth*, %	R_{Wedge} , in/min [mm/min]
1	6×4 [150×100]	37	0.0013 [0.033]
2	6×4 [150×100]	12	0.0003 [0.008]
3	6×6 [150×150]	31	0.0013 [0.033]
4	6×6 [150×150]	10	0.0003 [0.0008]
5	8×8 [200×150]	35	0.0012 [0.0030]
6	8×8 [200×150]	9	0.0001 [0.003]

* normalized by the nominal contact area

WST RESULTS

The WST results in terms of the splitting force vs CMOD curves are presented in Figure 3. For both milled and unmilled specimens, some of the curves presented a distinct peak within a couple of seconds of loading. The other specimens exhibited a plateau before the splitting forces began to decline. After careful examination of the test data, it was found that the peak load depicted for the specimens “Milled-1” and “Unmilled-1” was not the initiation point of the interface debonding. The peak and decline during the first two to three seconds of loading was most likely attributed to the actuator’s incapability in quickly achieving the desired CMOD rate. Evidence supporting this hypothesis was found after examining the CMOD-time history. The CMOD-time history for the “WST2-7”

specimen in Figure 3 was plotted in Figure 4. The peak for the “WST2-7” in Figure 3 occurs at a CMOD of approximately 0.0014 in [0.035 mm], in coincidence with the CMOD at which the loading rate converges to the target loading rate of 0.02 in/min [0.5 mm/min] in Figure 4. Therefore, there seems to be no distinctive feature from Figure 3 that can be used to indicate the initiation of interface debonding. Nevertheless, Figure 3 shows that the milled specimens required a greater splitting force than the unmilled specimens to achieve the same CMOD. This indicates that milling the asphalt enhances the fracture resistance of the interface. Moreover, it was also found that the notch depth played an important role in the interface debonding. In Figure 5, it presents the load-displacement curves for three unmilled specimens. These specimens had the same interface roughness and only varied in their notch depths. It is apparent that the fracture resistance (area under the load-displacement curve) was the greatest for the specimen with the smallest notch.

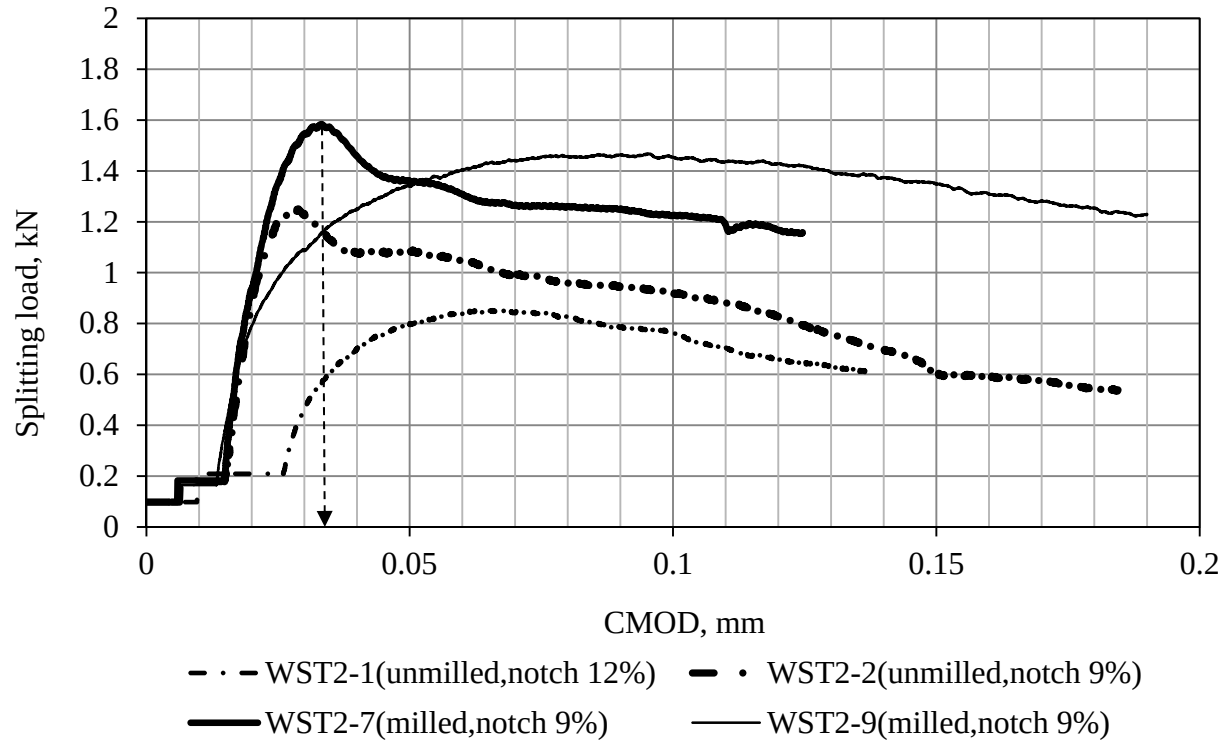


Figure 3-Load-displacement curve tested at room temperature and a CMOD rate=0.02 in/min [0.5 mm/min].

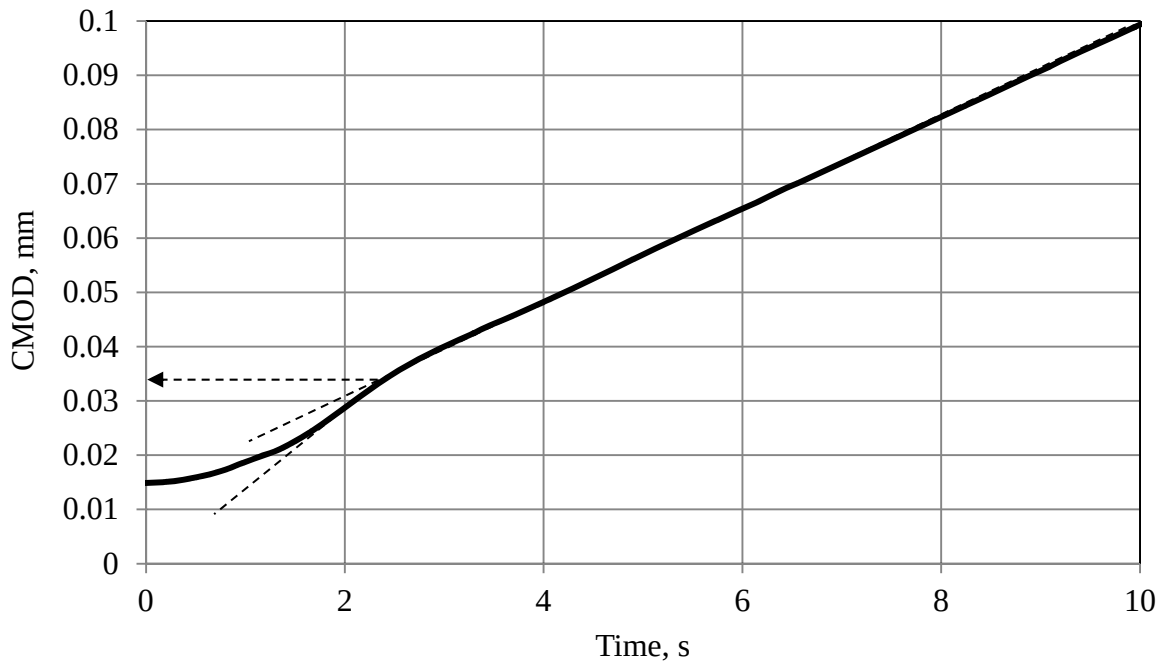


Figure 4-CMOD rate for the WST2-7 specimen with time 0 indicating the initial loading by the actuator.

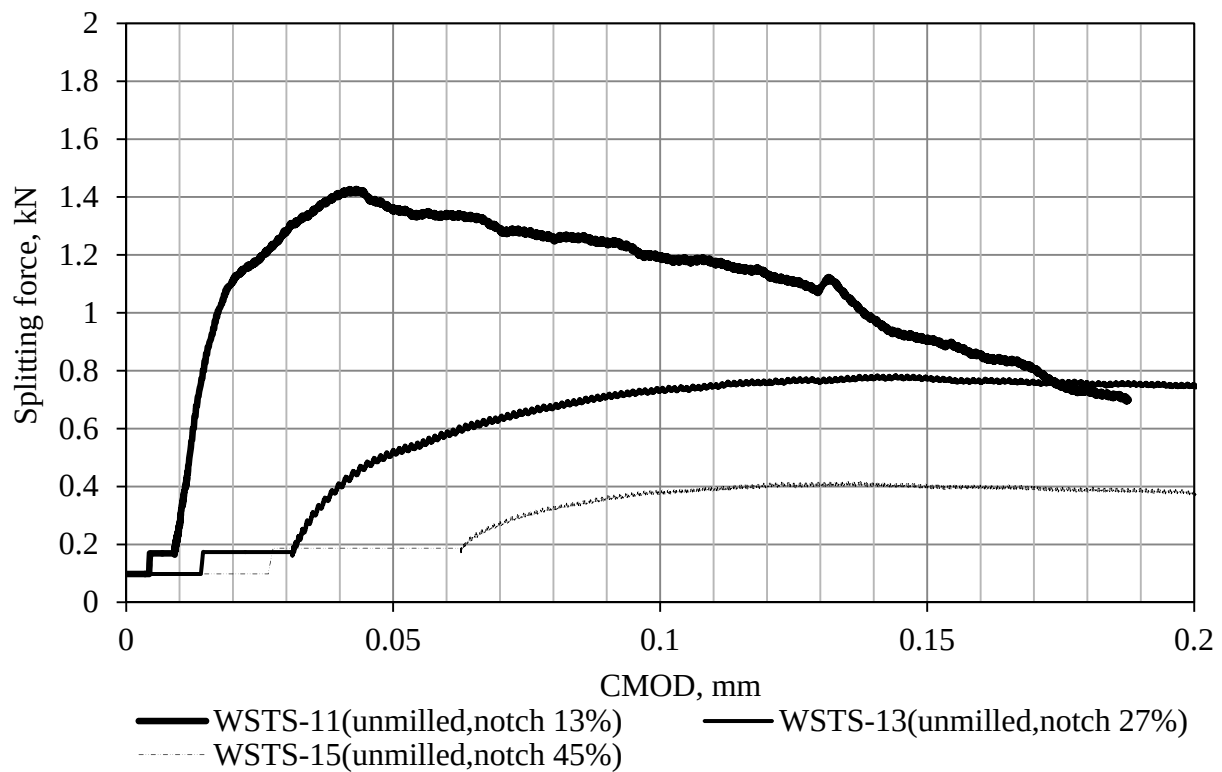


Figure 5-Load-displacement curve for unmilled specimens of the same size but different notch depths, tested at room temperature and CMOD rate=0.02 in/min [0.5 mm/min].

MODEL FOR DETERMINING THE ENERGY RELEASE RATE USING THE WST TEST

Derivation of the model

It can be seen from Figure 6, that the WST configuration resembles the geometry of two adhered cantilever beams. Therefore, the CMOD monitored by the clip gage can be estimated by calculating the bending of the beams. The action points for the splitting forces are considered to be an extension of the WST beams. This is because the extension is significantly thinner than the WST beams resulting from the removal of materials. The sum of the deflections at the end of the WST beams (CMOD) can then be related to the splitting forces after assuming the WST beams are under steady-state equilibrium during bending. The deflection of the concrete beam was computed using Equation (4).

$$\Delta = \Delta_{Load} + \Delta_{Moment} = \frac{F(t)a^3}{3EI} + \frac{M(t)a^2}{2EI} = \frac{F(t)}{EI} \left(\frac{a^3}{3} + \frac{fa^2}{2} \right) \quad (4)$$

where

Δ =CMOD, in [mm],

F =splitting force acting on the bearings, lbf [kN],

a =length of the WST beam, in [mm],

E =elastic modulus of the WST beam, psi [GPa],

I =moment of inertia of the WST beam, in⁴ [mm⁴] and

f =distance between the splitting force (the bearing) and the mouth of the notch as shown in Figure 6, in [mm].

For the HMA beam with visco-elastic response, Equation (4) becomes Equation (5).

$$F(t) = \frac{6I_{HMA}}{2a(t)^3 + 3fa(t)^2} \int_0^t E(t-\tau) \frac{d\Delta_{HMA}(\tau)}{d\tau} d\tau \quad (5)$$

where

$a(t)$ =crack length as a function of time, in [mm],

I_{HMA} =moment of inertia of the HMA beam, in⁴ [mm⁴].

$E(t)$ =relaxation modulus of the HMA, psi [GPa] and

$\Delta_{HMA}(t)$ =CMOD due to the bending of the HMA half of the WST specimen.

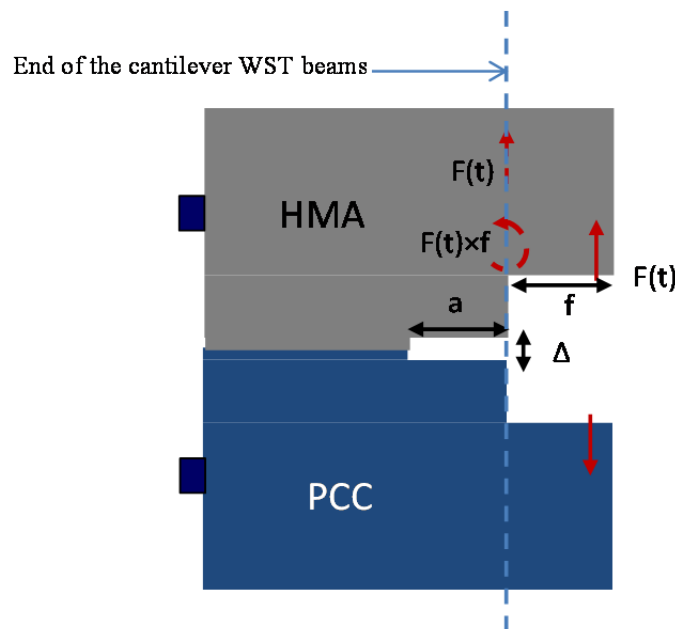


Figure 6-Analogy between WST and cantilever beams.

It is reasonable to assume $\Delta_{Total} = \Delta_{HMA}$, because the HMA is substantially less stiff than the PCC. The difference in their stiffness even increases as the HMA creeps during the loading as per Equation (2). Therefore, Equation (6) can be derived after introducing Equation (2) into Equation (5).

$$F(t) = \frac{6I_{HMA}}{2a(t)^3 + 3fa(t)^2} \int_0^t E_{ref} t^{-m} \frac{d\Delta_{Total}(\tau)}{d\tau} d\tau \quad (6)$$

The discrete form of Equation (6) is presented in Equation (7), which facilitates the analysis of discrete data series, i.e. $F(t)$ and $\Delta_{Total}(t)$.

$$F(x) = \frac{6I_{HMA}}{2a(x)^3 + 3fa(x)^2} \sum_0^{y=x} E_{ref}(x-y)^{-m} \cdot \delta\Delta_{Total}(x) \quad (7)$$

where

$\delta\Delta_{Total}(x) = \Delta_{Total}(x+1) - \Delta_{Total}(x)$, the increment of CMOD between two adjacent sampling points, in [mm].

Eventually, the energy release rate can be determined by calculating the energy required to progress a unit length of crack, as shown in Equation (8).

$$G \equiv \frac{W(t)}{Bda(t)} = \frac{F(t)d\Delta_{Total}(t) + F(t)f d\theta_{Total}(t)}{Bda(t)} \quad (8)$$

where

G =energy release rate, lbf·in/in² [kN/mm/mm²],

$\Delta_{Total}(t)$ =total CMOD, in [mm],

$\theta_{Total}(t)$ =total angular displacement at the crack mouth, radian,

B =width of the specimen, in [mm] and

$a(t)$ =crack length, in and $a(0)$ is the initial notch depth.

Based on the elastic beam theory, the relationship between Δ_{Total} and θ_{Total} is determined by Equation (9).

$$d\theta_{Total} = d(\theta_{Load} + \theta_{Moment}) = d\left(\frac{3\Delta_{Load}}{2a} + \frac{2\Delta_{Moment}}{a}\right) = \frac{3(a+2f)}{a(2a+3f)} d\Delta_{Total} \quad (9)$$

Therefore, the energy release rate in Equation (8) can be simplified as Equation (10) after including Equation (9).

$$G = \frac{F(t)d\Delta_{Total}(t)}{Bda(t)} C(t) \quad (10)$$

where

$C(t)$ can be defined as a function of the WST geometry as shown in Equation (11).

$$C(t) = \frac{2a(t)^2 + 6fa(t) + 6f^2}{2a(t)^2 + 3fa(t)} \quad (11)$$

Employment of the model

The history of the horizontal load $F(t)$ can be estimated based on the measured $\Delta_{Total}(t)$ using Equation (7). Before the crack initiation, the growth of $F(t)$ is relatively linear and the crack depth is equal to the notch depth, i.e. $a(t) = a(0)$. Using the data from this period of loading, the coefficients, m and E_{ref} in Equation (7) can be backcalculated by matching the predicted and measured $F(t)$. This initial loading period usually corresponds to CMOD of 0.001-0.002 in [0.025-0.05 mm]. An example of this process is presented in Figure 7. The curve 'Prediction with constant crack length' was based on $m=0.8$ and $E_{ref}=1,410$ psi [9.7 MPa], which was backcalculated assuming a constant crack length, i.e. $a(0)$, up to a CMOD of 0.001 in [0.025 mm]. Using these backcalculated parameters, the predicted $F(t)$ kept in a good agreement with the measured $F(t)$ until CMOD of approximately 0.0016 in [0.04 mm], where the interface debonding began to develop.

The difference between the predicted and measured $F(t)$ for CMOD greater than 0.0016 in [0.04 mm] can be explained by the propagation of interface cracks. In other words, if there was no interface cracking, more energy would have been required to achieve the same deformation. The development of the crack depth $a(t)$ can be estimated by varying $a(t)$ at each loading step until the predicted $F(t)$ agrees with the measured $F(t)$. In such a way, the development of the crack for WST2-7 was determined, i.e. 'Prediction with varying crack length' in Figure 7.

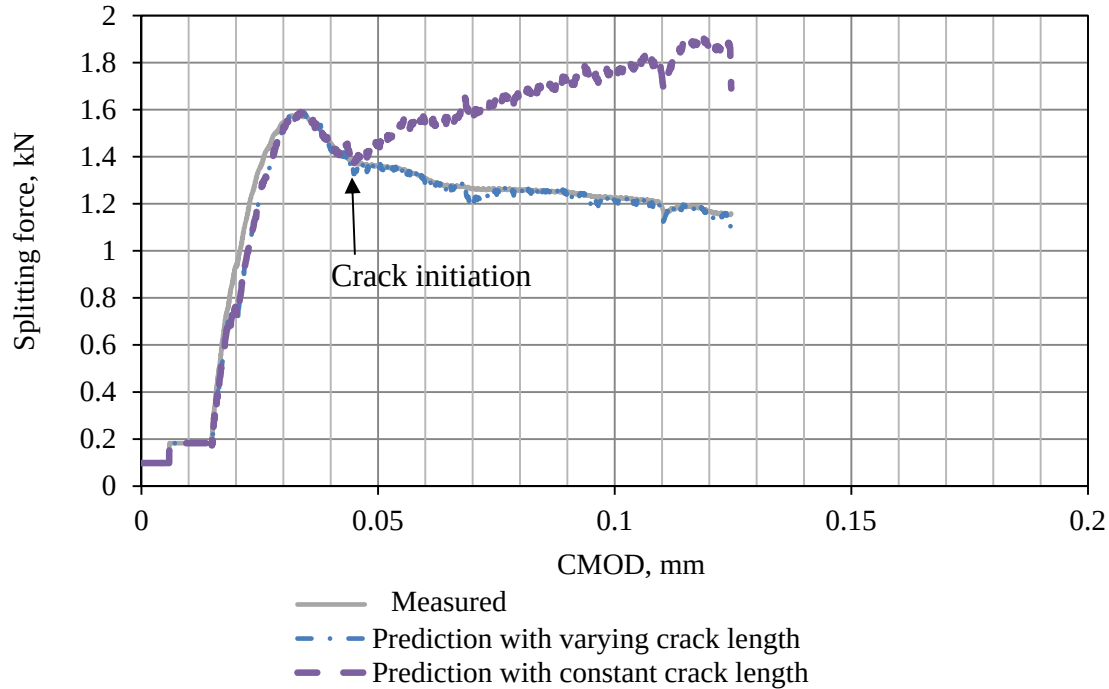


Figure 7-Measured and predicted load-CMOD curves for the WST2-7.

It has been discussed that the peak load that quickly occurred after loading was the result of a retarded CMOD rate, while not the indication of the interface debonding. In Figure 7, the crack initiation for WST2-7 occurred at CMOD= 0.0016 in [0.04 mm] that was after the peak load. In general, for the specimens that presented a loading plateau as the one in Figure 8, the initiation of the crack usually occurred at 60% -90% of the maximum load.

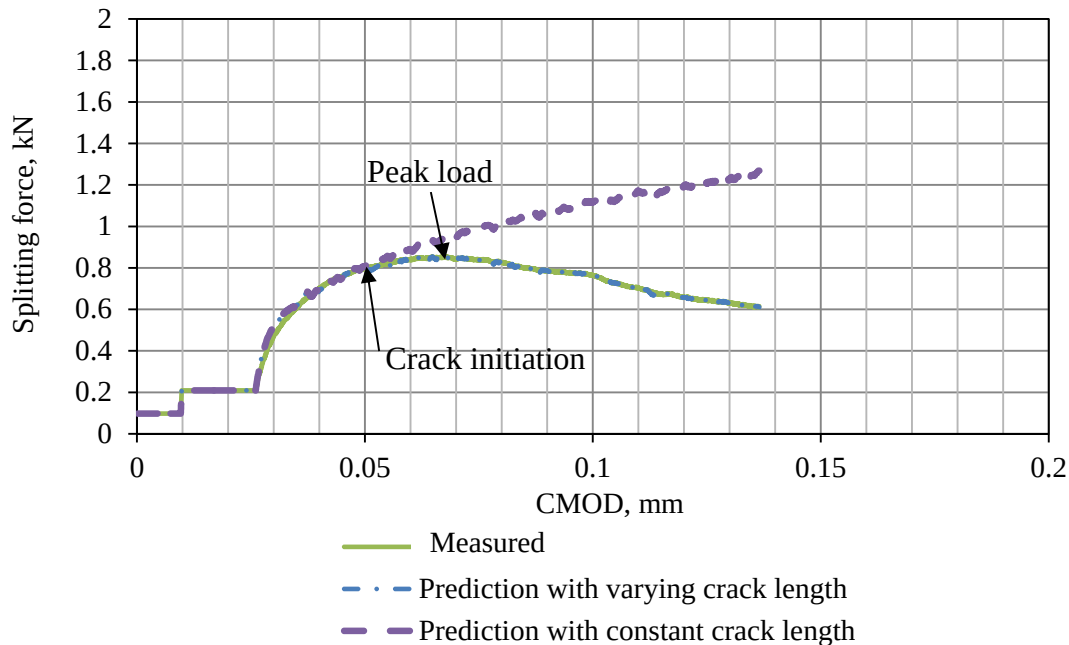


Figure 8-Measured and predicted load-CMOD curves for the WST2-1.

The parameters m and E_{ref} in Equation (2) for all the tested specimens are summarized in Table 3. The testing was performed at two temperatures and moisture conditions. For all the milled specimens, m varies between 0.4 and 0.8 and E_{ref} varies between 1400 psi [9.7 MPa] and 16,000 psi [110 MPa]. For all the unmilled specimens, m varies between 0.5 and 0.7 and E_{ref} varies between 800 psi [6 MPa] and 7,700 psi [53 MPa]. For the milled specimens at room temperature, m varies between 0.4 and 0.8 and E_{ref} varies between 1,400 psi [9.7 MPa] and 16,000 psi [110 MPa]. For the unmilled specimens tested at room-temperature, m varies between 0.5 and 0.7 and E_{ref} varies between 800 psi [6 MPa] and 7,700 psi [53 MPa]. With respect to the specimens tested in a frozen condition, m varies between 0.16 and 0.25 and E_{ref} varies between 28,000 psi [193 MPa] and 84,500 psi [583 MPa] for the milled specimens and m varies between 0.2 and 0.3 and E_{ref} varies between 20,000 psi [138 MPa] and 42,000 psi [290 MPa] for the unmilled specimens.

Table 3- m and E_{ref} for all the WST specimens.

Specimen	Testing temperature, °C	Milled	m	E_{ref} , psi [MPa]
WST2-07	23	Y	0.8	1,410 [10]
WST2-08	23	Y	0.67	1,750 [12]
WST2-09	23	Y	0.7	2,100 [14]
WST2-12	23	Y	0.5	8,500 [59]
WST2-13	23	Y	0.57	7,500 [52]
WST2-15	23	Y	0.58	5,200 [36]
WSTS-01	23	Y	0.4	16,000 [110]
WST2-01	23	N	0.69	1,600 [11]
WST2-02	23	N	0.65	1,080 [7]
WST2-03	23	N	0.7	800 [6]

WSTS-10	23	N	0.55	5,400 [37]
WSTS-11	23	N	0.48	2,070 [14]
WSTS-13	23	N	0.55	5,700 [39]
WSTS-15	23	N	0.55	7,690 [53]
WST2-10	-13	Y	0.16	47,000 [324]
WST2-11	-13	Y	0.16	28,000 [193]
WST2-14	-13	Y	0.25	84,500 [583]
WST2-04	-13	N	0.3	20,000 [138]
WST2-05	-13	N	0.25	420,00 [138]
WST2-06	-13	N	0.2	42,000 [290]

Three conclusions can be made based on the comparison of the abovementioned ranges. First, E_{ref} is generally greater for milled specimens. Second, m depends on the temperature not the milling condition. The deviation of the m value at the same temperature is believed to be due to the variation in the initial loading rate as well as the variation in material properties. Finally, the deviation of m and E_{ref} for the milled specimens is larger than that for the unmilled specimens, as is expected since the milled specimens have more complicated texture and thus present greater variability.

A more important function of the model is to reveal the history of the crack propagation. Since the fracture energy up to any crack depth can be determined by solving the integral of Equation (10), the energy release rate can be obtained by calculating the slope of the fracture energy relative to the crack depth, as shown in Figure 9.

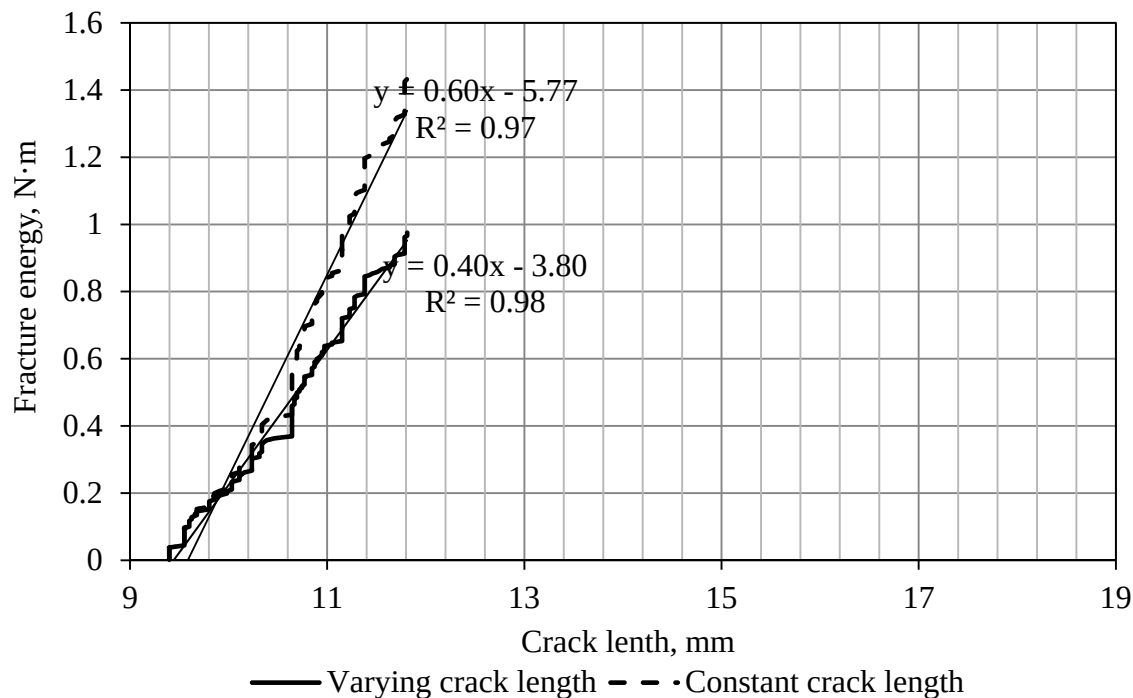


Figure 9-Fracture energy release rate for WST2-7.

The determined crack propagation was validated by comparing the predicted crack depth to the measured crack depth at the end of the test. The results of such comparison are presented in Table 4, where only the specimens having final crack depth measurements available on both sides and not failing abruptly were included. It should be noted that the average of the two surface measurements does not necessarily represent the depth of the crack across the specimen. Nevertheless, the comparison at least confirms the ability of the method in differentiating quick and slow crack development.

Table 4-Comparison between measured and predicted crack lengths.

Specimen	Measured crack length, in [mm]			Predicted crack length, in [mm]
	Front	Back	Average	
WST2-07	0.00 [0]	0.43 [11]	0.22 [6]	0.10 [3]
WST2-12	0.30 [8]	0.64 [16]	0.47 [12]	0.30 [8]

CONCLUSIONS

A wedge splitting test is developed to fracture PCC/HMA specimens at the interface under mode I loading. The test was carried out under CMOD control and the splitting force-CMOD data was recorded. An analytical model was then established for the analysis. The model is based on beam theory. It considers the viscoelasticity of the HMA material and more importantly is able to determine the initiation as well as the growth of the interface crack.

Using the model, the initiation as well as the growth of the interface crack was determined for both milled and unmilled specimens, which enabled the calculation of the energy release rate for each specimen. The viscosity coefficient m in the model appears to be a function of the test temperature and loading frequency. This is indicated by that m was a function of the temperature as well as the displacement rate. m , on the other hand, was not influenced by the surface texture (milling condition). The reference modulus in the model E_{ref} appears to be dependent on the interface roughness, because on average E_{ref} was greater for milled specimens. The milled HMA interface presented a substantially rougher failure surface, indicating a thicker interface layer. The increase in the interface thickness might explain the increase of stiffness for the milled specimens. The laboratory test as well as the model developed in this study can be used to conduct a quick yet mechanistic evaluation of the interface bond under mode I loading. However, further numerical and laboratory studies are still needed for more accurate characterization of the interface bond and a better understanding of the debonding mechanism.

ACKNOWLEDGMENTS

The authors would like to thank the Minnesota, Missouri, Mississippi, New York, Texas and Pennsylvania Departments of Transportation. These states are all participating in the FHWA Pooled Fund Study TPF 5-165 under which this work was performed. The authors would also like to gratefully acknowledge Swank Construction and the Pennsylvania Department of Transportation District 12 office who made it possible to acquire aged asphalt from an in-service pavement. Without their hard work, creative resolutions to challenges and commitment to this project this research would not have been possible.