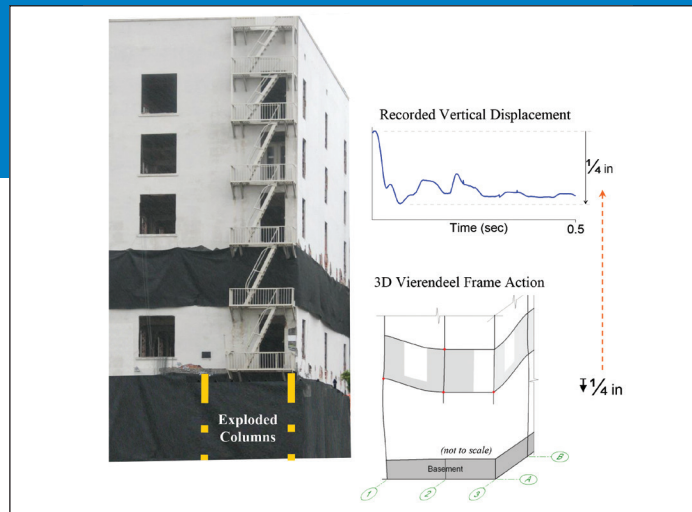


An ACI Technical Publication

SYMPOSIUM VOLUME



Structural Integrity and Resilience

SP-309

Editors:
Mehrdad Sasani and Sarah Orton



American Concrete Institute
Always advancing



Documents
Document Sharing Hub

Structural Integrity and Resilience

Editors:
Mehrdad Sasani
and Sarah Orton



American Concrete Institute

Always advancing

SP-309

First printing, June 2016

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 © 2016
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: 978-1-944487-04-0

Preface

The need for structural integrity has been recognized ever since the 1968 failure of the Ronan Point Apartment building. Improvements to the ACI code in 1989 required additional reinforcement for structural integrity, however those requirements were based on generally good building practices with little research or analysis to support them. However, since the disproportionate failure of the Murrah Federal building in Oklahoma City, these requirements have received renewed interest and new research conducted. More recently, and primarily due to the aftermath of natural and manmade disasters, the need for designing buildings that are resilient against various hazards has been recognized.

While most of the latest research does not directly analyze the efficacy of the structural integrity requirements, it does consider the overall collapse resistance and robustness of reinforced concrete buildings. Research using field experiments conducted in the last decade indicates that reinforced concrete structures are generally robust against local damage like single column removal. Although structural integrity requirements have been included in ACI 318 since 1989, there still exists areas of improvement. For example, recent laboratory experiments show that flat plate structures may still be vulnerable due to the high likelihood of progressive punching shear failures. Furthermore, for structures designed and built without structural integrity provisions, new research highlights ways to improve their robustness and collapse resistance. Finally, improved analysis models and predictions on the likelihood of collapse lead to better assessment of the risks of collapse.

ACI Committee 377 sponsored two sessions during the Fall 2014 ACI convention in Washington, DC to highlight the importance of structural integrity and resilience of reinforced concrete and precast/prestressed structures subjected to extreme loading conditions. The sessions sought papers on topics including improving the structural integrity of structures, minimum level of required integrity, integrity of precast/prestressed structures, performance-based structural integrity and resilience, infrastructure resilience, issues and new developments in modeling, and assessment of existing structures. Both experimental and analytical investigations were presented. The sessions presented 10 papers covering the design of reinforced concrete buildings against progressive collapse, evaluation of NYC code provisions, analysis and experimental testing of post-tensioned and precast/prestressed structures, methods to improve collapse resistance, and probabilistic analysis of collapse.

This special publication includes eight papers that were presented during the sessions. The papers are alphabetically ordered based on the last names of the first authors.

Editors

Sasani, M., and Orton, S.

TABLE OF CONTENTS

SP-309—1

A Simple Method of Enhancing the Robustness of R/C Frame Structures 1.
Authors: Yihai Bao, H.S. Lew, Joseph A. Main and Fahim Sadek

SP-309—2

New York City Building Code Flat Plate Integrity Provisions.....2.
Authors: Ramon Gilsanz, Karl Rubenacker, and Jennifer Lan

SP-309—3

Response of a Post-Tensioned Floor Following a Column Loss 3.
Authors: Leila Keyvani and Mehrdad Sasani

SP-309—4

Uncertainties in Predicting Structural Disproportionate Collapse 4.
Authors: Shalva Marjanishvili and Serdar Astarlioglu

SP-309—5

Post-Punching Capacity of Flat-Plate Floor Systems 5.
Authors: Sarah Orton, Ying Tian, Zhonghua Peng, and Jinrong Liu

SP-309—6

Recent Progress in Understanding of Load Resisting Mechanisms 6.
for Mitigating Progressive Collapse
Authors: Kai Qian, Bing Li, and Ying Tian

SP-309—7

Assessment of Urban Building Complexes Subjected to Natural and 7.
Man-Made Hazards
Authors: Malte von Ramin, Alexander Stolz, Oliver Millon, and Tassilo Rinder

SP-309—8

Risk Assessment of Reinforced Concrete Buildings Against 8.
Progressive Collapse
Authors: Bing Xue and Jia-Liang Le

A SIMPLE METHOD OF ENHANCING THE ROBUSTNESS OF R/C FRAME STRUCTURES

By: Yihai Bao, H.S. Lew, Fahim Sadek, and Joseph A. Main

Synopsis: A simple debonding technique was proposed to reduce strain localization in reinforcing bars in the region of wide flexural cracks in reinforced concrete (R/C) beams, in order to enhance the resistance of R/C buildings to disproportionate collapse. Debonding was achieved by heat-shrinking polyolefin tube over the reinforcing bar. Results from testing of a No. 8 reinforcing bar showed that with an 8 in (203 mm) debonding length on both sides of a $\frac{1}{4}$ in (6.35 mm) wide gap, simulating a wide flexural crack, the elongation of the reinforcing bar prior to fracture was about 38 % more than for the case without the debonding technique. This observation demonstrated that the debonding method could effectively reduce strain localization, thereby delaying the fracture of reinforcing bars. To analyze the effects of debonding, detailed finite-element models of the test specimens were developed, which adequately captured the experimental results. R/C frame structures were analyzed by applying the debonding model under a column removal scenario. The results indicated that the debonding method could enhance the development of catenary action in the beams of R/C frame structures.

Keywords: disproportionate collapse, reinforced concrete, debonding, catenary action, nonlinear finite element analysis

Biography: Yihai Bao is an IPA researcher at the Engineering Laboratory (EL), the National Institute of Standards and Technology (NIST) and a project scientist at the Department of Civil and Environmental Engineering, the University of California - Davis. He received his BS and MS degrees from Tongji University of China and PhD from the University of California - Davis, Davis, CA. His research interests include nonlinear modeling of structural behavior under extreme loading conditions, large-scale experimental methods and advanced scientific computational methods.

ACI Honorary Member **H. S. Lew** is a Senior Research Engineer at the Engineering Laboratory (EL), the National Institute of Standards and Technology (NIST). He received his PhD from the University of Texas at Austin, Austin, TX. He has served on the ACI Board of Direction, and a member of standard and technical committees. His research interests include performance of structural members and systems, construction safety, failure investigations, and earthquake engineering.

Fahim Sadek is Leader of the Structures Group at the Engineering Laboratory (EL), the National Institute of Standards and Technology (NIST). He received his PhD from the Southern Methodist University, and is a licensed professional engineer. His research interests include vulnerability of structures subjected to multi-hazard conditions, response of structures to dynamic and impulse loading and structural health monitoring.

Joseph A. Main is a Research Structural Engineer at the Engineering Laboratory (EL), the National Institute of Standards and Technology (NIST). He received his PhD from Johns Hopkins University. His research interests relate to the computational analysis of structural performance under extreme loads, wind, fire, and disproportionate collapse.

INTRODUCTION

Recent experimental studies¹⁻⁴ on reinforced concrete (R/C) frame structures under column removal scenarios have identified the failure mechanisms that occurred when they underwent large deflections. In the tests of two full-scale frame assemblies¹, it was noted that the beam-end rotations under monotonic loads were approximately 7 to 8 times as large as those based on seismic tests under cyclic loads. However, the rotational capacity can be further increased if more ductility is achieved in the reinforcing bars at critical locations, such as at beam ends near column faces. The ductility of reinforcing bars is adversely affected by strain localization, which may occur at a wide flexural crack at a critical beam section, such as the one shown in **Fig. 1**. The exposed bar segment at the crack developed larger plastic strain than the embedded portions adjacent to the crack, due to the bond of surrounding concrete to the reinforcing bar. Premature fracture of reinforcing bars limits the further development of catenary forces in the beams.

The existence of strain localization was observed in the numerical analysis of two reinforced concrete frame assemblies, where the numerical models have been validated against experimental results⁵. **Fig. 2** shows the calculated plastic strain distribution of the beam bottom reinforcing bar near the unsupported center column just before the fracture. A steep strain increase in the bottom reinforcing bar was observed at the beam section near the unsupported column, where a wide flexural crack occurred during the test. To relieve the strain localization, a debonding concept is applied and illustrated in **Fig 3**. By debonding the reinforcing bar from the surrounding concrete at the potential crack zone, a more uniform strain distribution can be achieved, reducing the strain localization. To demonstrate the effectiveness of this concept, a detailed finite element model was created for one of the above-mentioned frame assemblies. The numerical model considered the debonding of the beam-bottom reinforcing bars near both sides of the unsupported center column with the debonding length equal to the beam depth. The plastic strain distribution of the bottom reinforcing bar is plotted in **Fig. 2**. A relatively uniform distribution was observed as a result of detaching the reinforcing bar from the surrounding concrete. The reduction of strain localization delayed the bar fracture, which resulted in a 14 % increase in the ultimate vertical load capacity and a 10 % increase in the corresponding vertical deflection of the center column as presented subsequently in Table 1 and **Fig. 14(a)**. Larger increases in capacity and deflection at failure were achieved with a larger length of debonding.

A simple approach is proposed herein to accomplish debonding between reinforcing bars and concrete. The effectiveness of this method is demonstrated through an experimental study. The experimental data are also compared with computational models, which are later used to investigate the influence of debonding length and evaluate the behavior of frame structures under seismic loads.

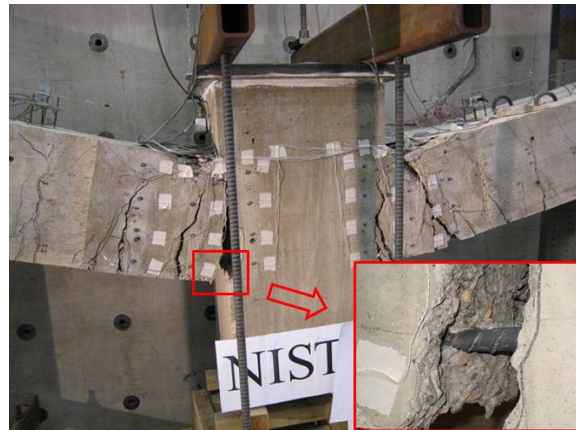


Fig. 1–Reinforcing bar fracture at a wide flexural crack.

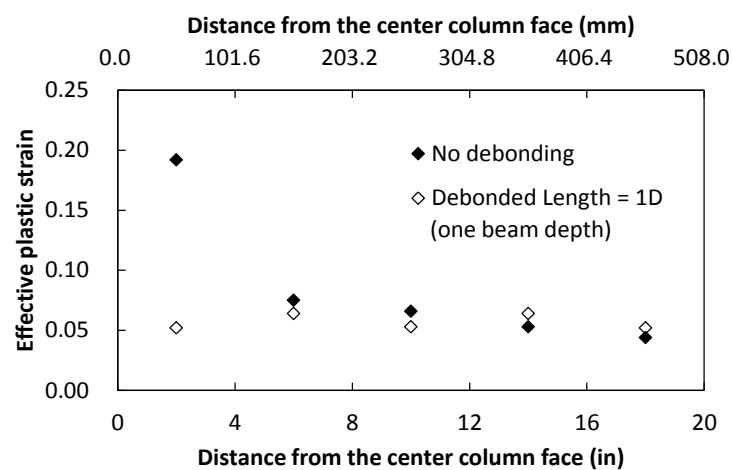


Fig. 2–Computed strain distribution in beam bottom reinforcing bar.

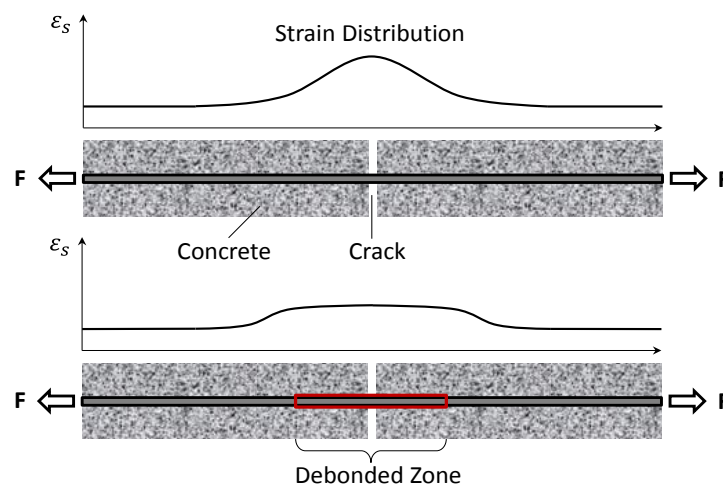


Fig. 3–Debonding concept.

RESEARCH SIGNIFICANCE

Few research studies on enhancing the disproportionate collapse resistance of reinforced concrete frame structures have been conducted to date. Based on the previous experimental studies on the failure mechanisms of reinforced concrete frame structures under column loss scenarios, this paper presents an innovative yet simple method which has the potential to significantly enhance the structural performance under column loss scenarios. The feasibility and effectiveness of the proposed method are demonstrated experimentally and numerically. These studies provide a solid basis for further study on this method and its practical application in reinforced concrete structures.

EXPERIMENTAL INVESTIGATION

A series of tensile tests of an embedded reinforcing bar in concrete with a simulated wide crack was carried out to demonstrate (1) the effectiveness of the technique used in this study to debond the reinforcing bar from the surrounding concrete and (2) the efficacy of the debonding concept to delay the fracture of the embedded bar at a simulated crack location. Two different debonding lengths were considered, as well as a specimen with no debonding, in order to investigate the influence of debonding length on the efficacy of the debonding technique. A key objective of the embedded bar tensile tests with different debonding lengths was to provide experimental data for comparison with computational models. As is discussed subsequently, the computational models were used to further investigate the influence of debonding length on the performance of an R/C frame. Seismic loading was considered as well as column loss, because it is important that the debonding technique should not negatively impact the overall performance of an R/C frame.

Design and fabrication of specimens

A total of three specimens were tested in this study. Each specimen consisted of two 48 in (1219 mm) long 10 in (254 mm) diameter concrete cylinders with one No. 8 reinforcing bar embedded in the center of both cylinders (**Fig. 4**). A $\frac{1}{4}$ in (6.4 mm) gap was preset between the two cylinders, simulating a wide crack opening. The only parameter varied between specimens was the debonding length on each side of the gap. The No. 8 bar was fully embedded (no debonding) for the first specimen (SN). The second (SD-2) and third specimens (SD-8) were designed with 2 in (51 mm) and 8 in (203 mm) debonding lengths on each side of the gap, respectively. During testing, the specimen was pulled in the longitudinal direction until bar fracture occurred. To create a similar loading condition as a beam-bottom bar in tension seen in the assembly tests¹, the external force was applied through a steel loading fixture at each end of the specimen, which included a 2 in (51 mm) thick circular steel plate that was bolted to the specimen using four $\frac{7}{8}$ in (22 mm) diameter high-strength threaded bars embedded in the concrete cylinders.

The polyolefin shrink tube was used to debond the reinforcing bars from the surrounding concrete at the desired locations. When heated, the polyolefin tube shrinks in the radial direction to fit tightly over the reinforcing bar. **Fig. 5** shows tube segments on reinforcing bars before and after exposure to a heat source. Circular high-strength paper tubes were used as formwork. Self-consolidating concrete was placed in the forms through circular holes on the top surface of the forms (**Fig. 6**).

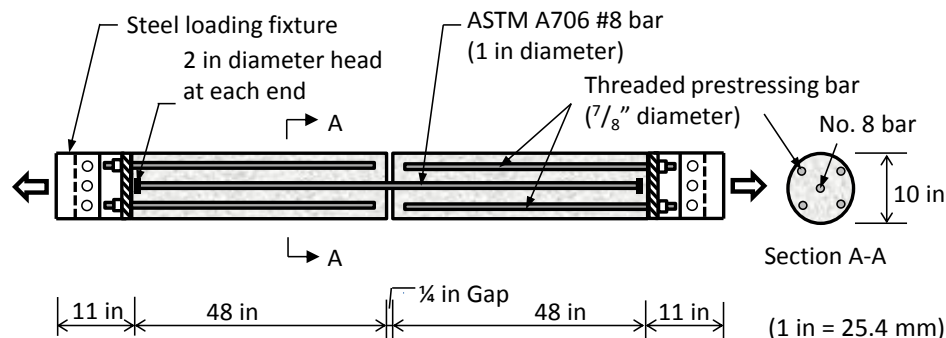


Fig. 4—Test specimen.



Fig. 5–Polyolefin shrink tube before (right) and after (left) heat treatment.



Fig. 6–Concrete placement in cylindrical forms.

Material properties

The specimens were designed with self-consolidating concrete having a nominal compressive strength of 6 ksi (41.4 MPa) and ASTM A706-Grade 60 reinforcing bars with a minimum specified yield strength of 60 ksi (414 MPa). An average compressive strength of concrete from three 6 in × 12 in (152 mm × 305 mm) cylinders at the time of testing was 12 ksi (83 MPa). The measured yield and ultimate strengths of the No. 8 bar were 70 ksi (483 MPa) and 103 ksi (710 MPa), respectively. The yield strain was 0.265 % and the elongation at fracture was 16 % with a gage length of 8 in (203 mm).

Test program

The test specimens were placed in the universal testing machine with the two ends connected to the loading fixtures as shown in **Fig. 7**. The load was applied under displacement control at a rate of 0.0025 in/sec (0.0635 mm/sec) until failure occurred, representing quasi-static loading conditions. Structural deflections following the loss of a column or other load-bearing element do not typically result in strain rates that are high enough to achieve significant strength enhancement, so the conservative use of quasi-static loading conditions is prudent for this application. Two displacement transducers were used to measure the relative movement of the concrete cylinders near the gap. The gage length for the displacement measurement was 6.25 in (159 mm), extending a length of 3 in (76 mm) on each side of the gap (**Fig. 8**). Strain gages were placed on the surface of the No. 8 bar over a length of 8 in (203 mm) on each side of the gap to measure the strain distribution along the bar. During the tests, all measured data were collected and recorded through a data acquisition system. The uncertainty in the measured data from the load cell, displacement transducers, and strain gages was within ±1%.

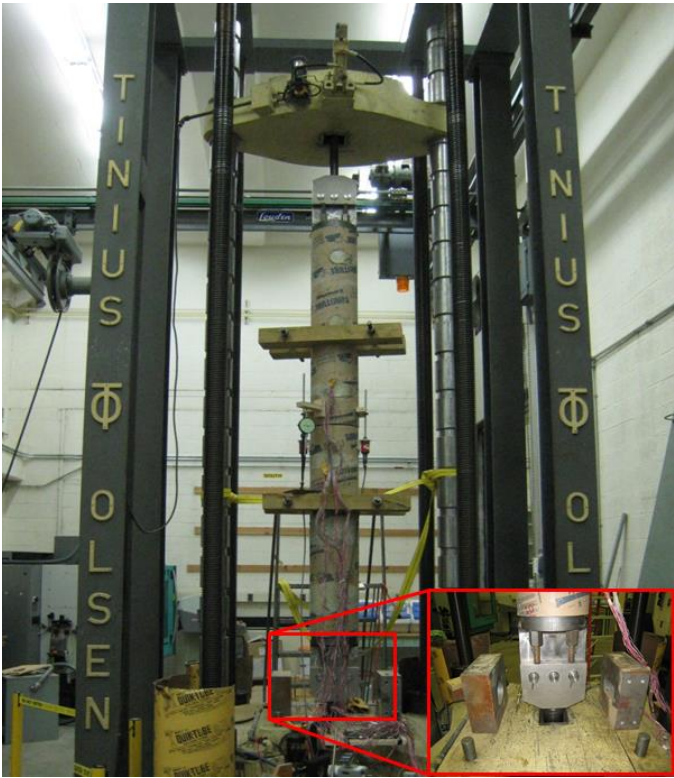


Fig. 7–Specimen in testing machine.

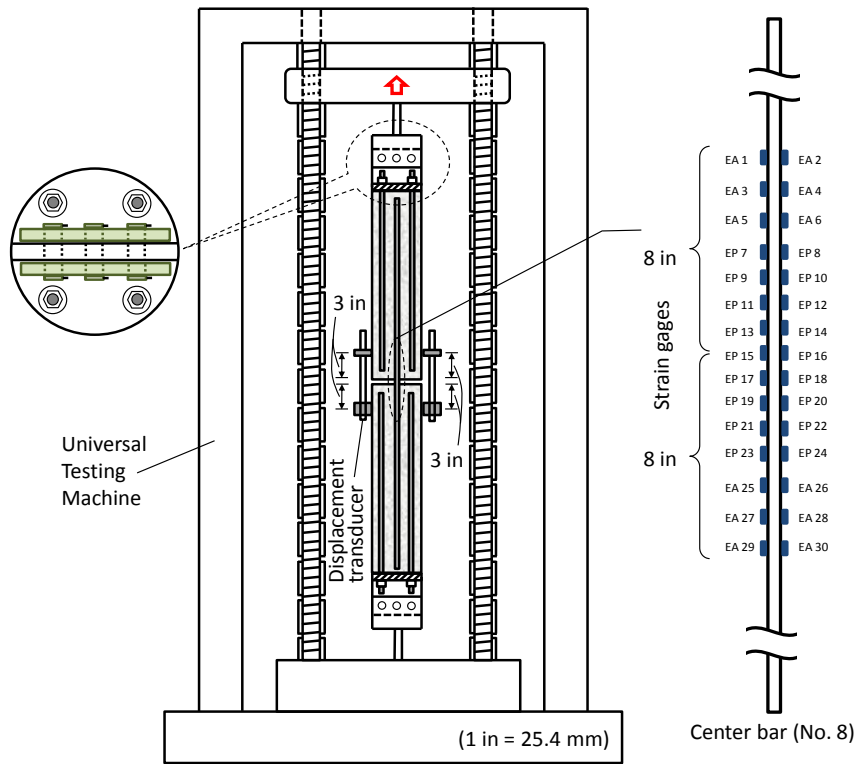


Fig. 8–Schematic of instrumentation plan.

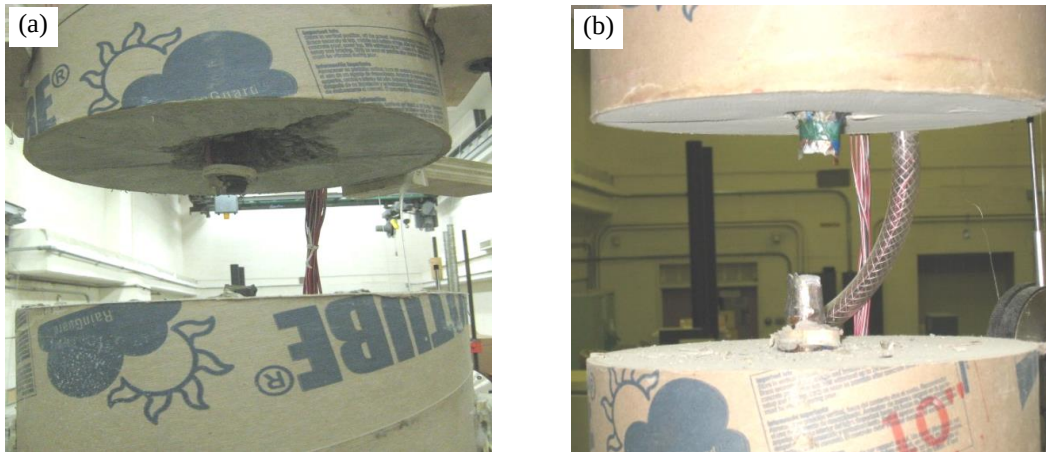


Fig. 9 – Final damage of (a) specimen without debonding and (b) specimen with debonding.

Experimental Results

For all three specimens, failure was characterized by the fracture of the central reinforcing bar within the preset gap. However, for the specimen SN (without debonding), some damage was observed in the concrete before the bar fracture. A typical cone-shape concrete failure emanating from the central reinforcing bar to the gap surfaces occurred during testing. The damaged concrete block was eventually separated from the cylinder before the fracture of the central reinforcing bar [Fig. 9(a)]. However, no significant concrete damage at the gap surfaces was observed for specimens SD-2 and SD-8 (with debonding) during the entire test [Fig. 9(b)]. In the debonded zones, no traction was applied to the surrounding concrete and the central bar could elongate freely.

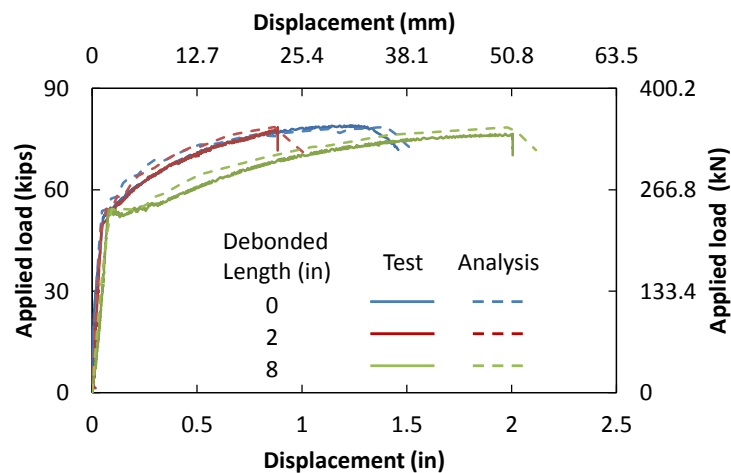


Fig. 10 – Load-displacement curves from testing and numerical analysis

The applied load-versus-displacement curves from the three specimens are plotted in Fig. 10. Although the peak loads are almost identical for the three specimens, significant differences were observed among the displacements of the three specimens prior to fracture. Specimen SD-8, with a debonding length of 8 in (203 mm) on each side of the gap, had the most ductile behavior. The fracture displacement of specimen SD-8 was approximately 2 in (51 mm), which is 38 % greater than the fracture displacement of 1.45 in (37 mm) for the specimen without debonding (SN). Specimen SD-2, with a debonding length of 2 in (51 mm) on each side of the gap, showed a fracture displacement of 0.9 in (23 mm), which is less than that of the Specimen SN. The extensive concrete damage observed on the gap surfaces of the Specimen SN may contribute to this counterintuitive result. Due to the separation of the damaged concrete, a *de facto* debonding was created for specimen SN. Because the debonding length of specimen SD-2 was less than the extent of the damage zone for specimen SN, earlier bar fracture occurred. These results show that debonding over a very short length can actually result in less desirable behavior.

The experimental results demonstrate the effectiveness of the proposed technique using the polyolefin shrink tube to debond the reinforcing bar from the surrounding concrete. The results also indicate that the debonding concept, when applied over a sufficient length, can effectively delay the fracture of a reinforcing bar.

Computational analysis

Detailed finite element models of the three test specimens were developed in LS-DYNA⁶, a general-purpose finite element program. Eight-node solid elements were used to model the concrete cylinders, and two-node beam elements were used to represent the reinforcing bars [Fig. 11(a)]. Hourglass modes in solid elements were suppressed through a stiffness-based control algorithm. The bond between concrete and reinforcing bars was modeled by introducing a one-dimensional contact between the concrete elements and steel elements. This contact interface allowed relative displacement of the beam elements in the longitudinal direction based on a predefined bond-slip relationship, while preventing penetration through the surrounding concrete elements normal to the bar. Debonding was modeled by specifying a negligibly small value for the bond modulus (the ratio of bond stress to slip) within the debonded region, so that free axial elongation of the bar could occur in this region while transverse bar displacements were restrained by the one-dimensional contact interface.

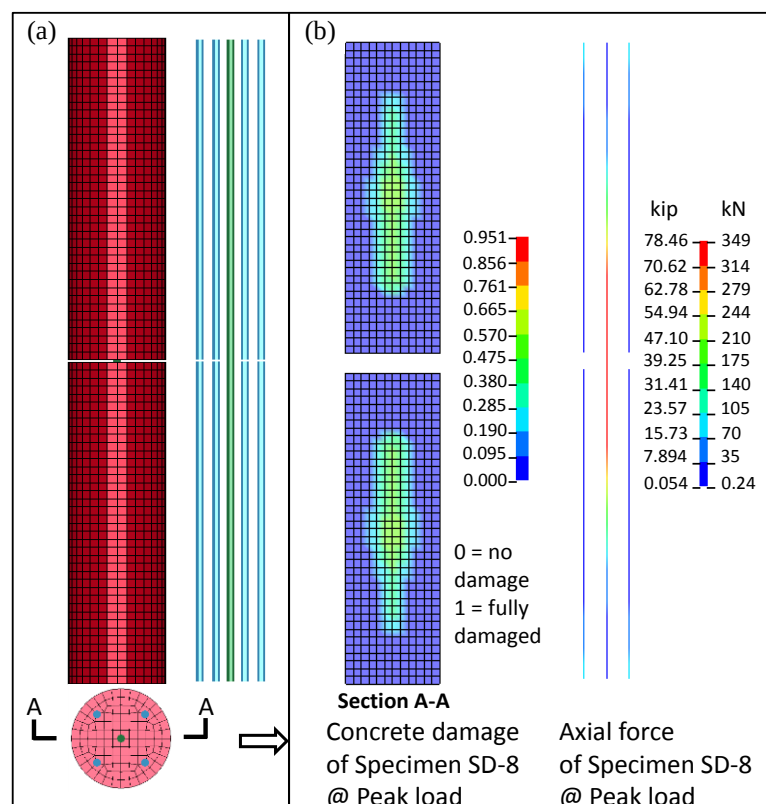


Fig. 11 – (a) Detailed finite element model of test specimens and (b) concrete damage contours and bar force distribution for specimen SD-8.

The concrete behavior was modeled by a continuous surface cap model with the following main features: isotropic constitutive equations, a yield surface formulated in terms of three stress invariants with translation for pre-peak hardening, a hardening cap that expands and contracts, and damage-based softening with erosion and modulus reduction⁷. The reinforcing steel was modeled using an isotropic elastic-plastic model, in which the nonlinear behavior of steel was defined through a true stress versus effective plastic strain curve. A failure strain was specified, and once reached, the corresponding element was eroded to simulate bar fracture. The average mesh size of the solid elements was approximately 1 in (25 mm) and that of the beam elements was 2 in (51 mm). A mesh sensitivity study showed that this discretization was adequate to capture the state of stress in the concrete and in the steel reinforcing bar, and that further refinement of the mesh would not significantly affect the final results. This modeling approach was able to successfully capture previous full-scale tests of R/C

moment-frame assemblies under a column removal scenario⁵, in which the effects of bond slip on bar fracture were significant.

The analysis results of the three specimens are plotted in **Fig. 10** along with the experimental results. This comparison shows that the modeling techniques used in this study adequately captured the experimental results. **Fig. 11(b)** shows contours of the concrete damage index along a radial section of specimen SD-8 under the peak load, where a damage index of zero indicates no damage and a damage index of unity indicates complete damage. Within the debonded zone, no clear concrete damage is observed and a constant axial force is seen in the central bar. The axial force in the bar gradually decreases with increased distance from the debonded zone because of the bond between the surrounding concrete and the reinforcing bar. **Fig. 12(a)** shows the strain distribution of the central bar for specimen SN (without debonding) within a region of 8 in (203 mm) from the center of the gap. The strains were measured at an applied load of 20 kip (89 kN). A peak in the strain values is evident at the gap location, with a sharp drop in strain at a distance of 4 in (102 mm) away from the gap. **Fig. 12(b)** depicts the strain distribution of the central bar for specimen SD-8 [with 8 in (203 mm) debonding on each side] under the same applied load magnitude. A more uniform strain distribution is observed, confirming the successful elimination of strain localization at the simulated crack location. It is noted that the number of data points is smaller in **Fig. 12(a)** than in **Fig. 12(b)** because of failure of some of the strain gauges on specimen SN. Computational results for these two cases are also plotted in **Fig. 12** and show reasonable agreement with the measured data.

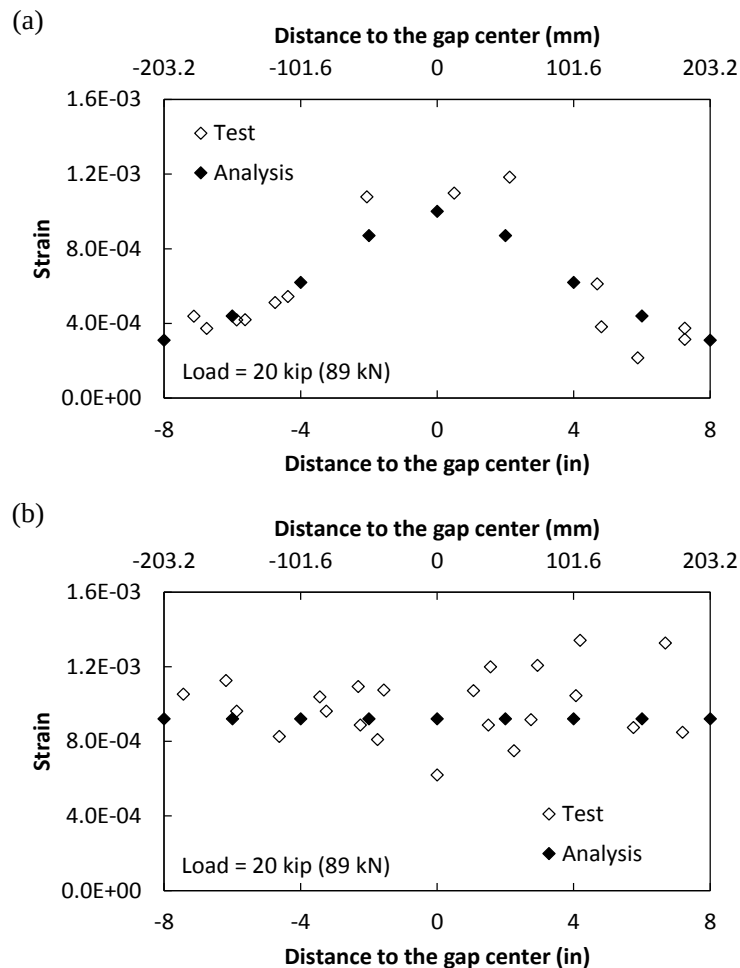


Fig. 12 – Strain distribution along the central reinforcing bar (a) without debonding and (b) with 8 in (203 mm) debonding on each side of central gap.

APPLICATION TO R/C FRAME

The above sections have demonstrated the effectiveness of the debonding concept experimentally and numerically under uniaxial tensile loading. The purpose of this section is to consider application of the debonding concept to reinforced concrete frame structures. This section includes discussion on the influence of the debonding location and length, and the evaluation of the effect of debonding on seismic performance. A three-dimensional frame structure is analyzed under a center column removal scenario to show the effectiveness of the proposed debonding approach to enhance the structural robustness.

Influence of debonding location and length

According to experimental observations¹, the ideal debonding location would be the bottom reinforcing bars at beam ends near the column face. Although the top reinforcing bars at beam ends could also experience tensile forces under negative moment, the bar fracture would more likely occur first at the bottom reinforcing bars in tension because (1) the positive moment capacity is usually smaller than the negative moment capacity at beam ends and (2) the maximum positive moment at the removed column is usually larger than the maximum negative moments at the adjacent columns due to the finite stiffness provided by the adjacent frame members. If beam-slab composite action is considered, the above conclusion may be reversed, and bar fracture may first occur in the top reinforcing bars at the beam ends with negative moment. In this case, the moment capacities should be calculated based on the effective T-section. In general, only beam members that have the potential to develop catenary forces under column removal scenarios will be selected for the debonding application. Debonding of both top and bottom reinforcing bars is not recommended, because that could negatively impact the seismic performance. The final debonding location can be determined by comparing the moment demand capacity ratios (DCRs) of beam-end sections. Since the slab is excluded in the example frame structures investigated in this study, the debonding location is determined to be the bottom reinforcing bars at beam ends.

Table 1 – Peak load and displacement under various debonding lengths.

	Debonding schemes	Peak load kip (kN)	Peak displacement in (mm)
IMF	No debonding	$P_1=120$ (534)	$d_1=41$ (1041)
	Debonded length = $1D$	$1.14 P_1$	$1.10 d_1$
	Debonded length = $2D$	$1.34 P_1$	$1.32 d_1$
	Debonded length = $3D$	$1.37 P_1$	$1.34 d_1$
SMF	No debonding	$P_2=270$ (1201)	$d_2=48$ (1219)
	Debonded length = $1D$	$1.03 P_2$	$1.02 d_2$
	Debonded length = $2D$	$1.32 P_2$	$1.35 d_2$
	Debonded length = $3D$	$1.38 P_2$	$1.39 d_2$

A parametric study was carried out to investigate the influence of debonding length using two tested full-scale beam-column assemblies¹, which included an intermediate moment frame (IMF) assembly and a special moment frame (SMF) assembly. The design of the two frame assemblies was in accordance with ACI 318-02⁸. The dimensions and section details of the tested assemblies are shown in **Fig. 13**. Debonding of the beam bottom reinforcing bars near the center column stub was considered with various debonding lengths: $1D$, $2D$, and $3D$ (where D is the depth of the beam). Detailed finite element models were developed to investigate the influence of debonding length on the peak load achieved prior to the fracture of reinforcing bars. The same modeling techniques as presented in the previous section were used to model the debonding between the reinforcing bars and the surrounding concrete. Further modeling details were described in the cited reference⁵. **Fig. 14** depicts the applied vertical load versus the vertical displacement of the center column, based on analyses with different debonding lengths. For both assemblies, the case with $3D$ debonding length gives the best performance in terms of the peak applied load. However, because the difference in the peak loads between the cases with $2D$ and $3D$ debonding lengths is very small, it seems prudent to select $2D$ as the recommend debonding length, in order to reduce the potential negative impact of debonding on the stiffness of the

connection. Table 1 shows the peak loads and the corresponding displacements at the center column for both assemblies under the center column loss scenario with various debonding lengths, along with the case of no debonding. These results indicate that an improvement of more than 30 % in the peak load for both the IMF and SMF assemblies is achieved with a debonding length of two times the beam depth.

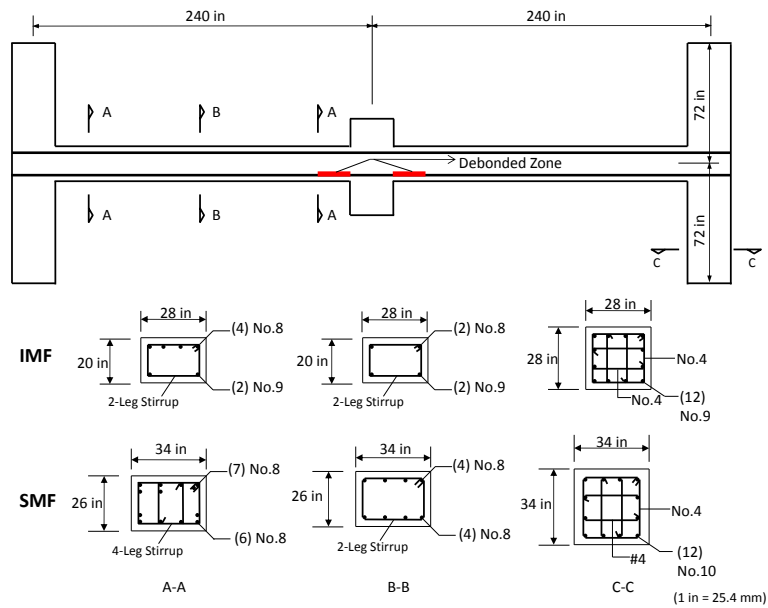


Fig. 13 – Dimensions and section details of the IMF and SMF assemblies.

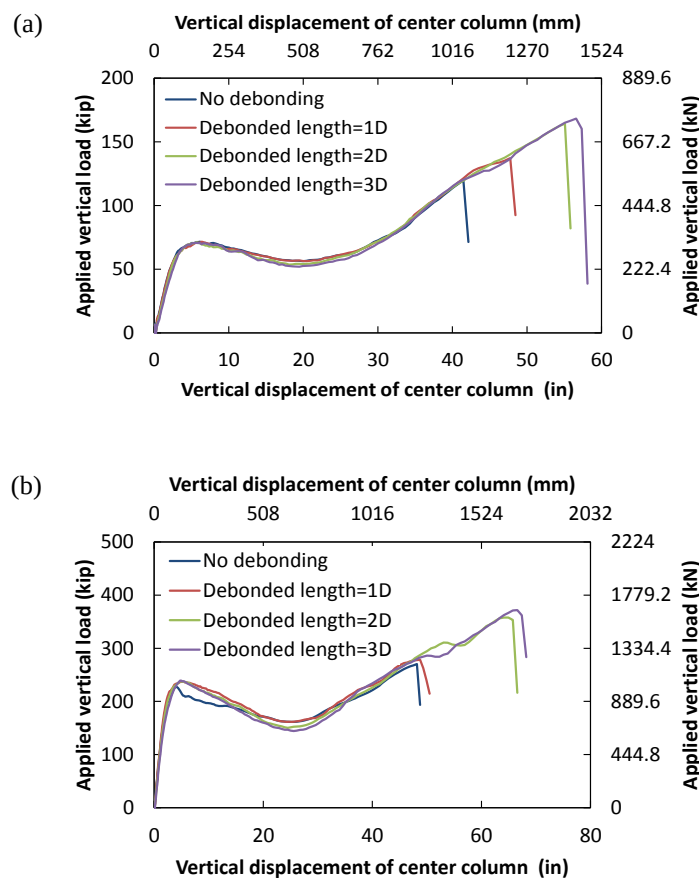


Fig. 14 – Computed vertical load versus vertical displacement of center column: (a) IMF assembly; (b) SMF assembly.

Performance under seismic load

Although the proposed debonding method can effectively enhance the disproportionate collapse resistance by eliminating premature bar fracture due to strain localization, it is important to verify that the debonding approach will not result in significantly degraded performance under other loading conditions, especially seismic loading. To investigate this issue, detailed finite element models of a cruciform beam-column assembly with and without debonding were analyzed under both monotonic and cyclic lateral loading conditions. For the model with debonding, the beam-bottom reinforcing bars near the joint were debonded with a debonding length of $2D$.

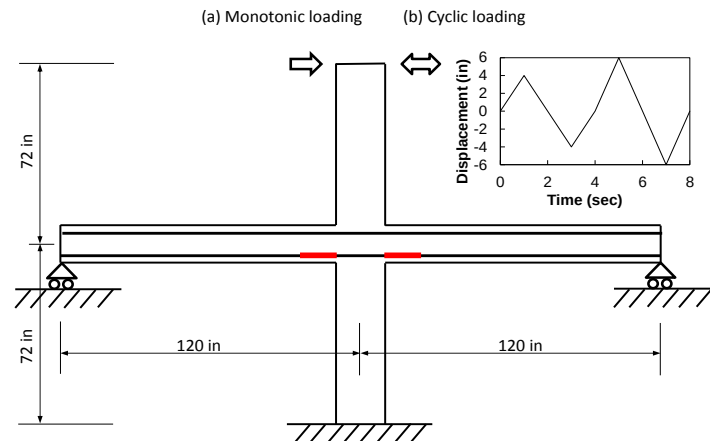


Fig. 15 – Dimensions, boundary conditions and loading schemes of IMF cruciform assembly
(Note: 1 in = 25.4 mm).

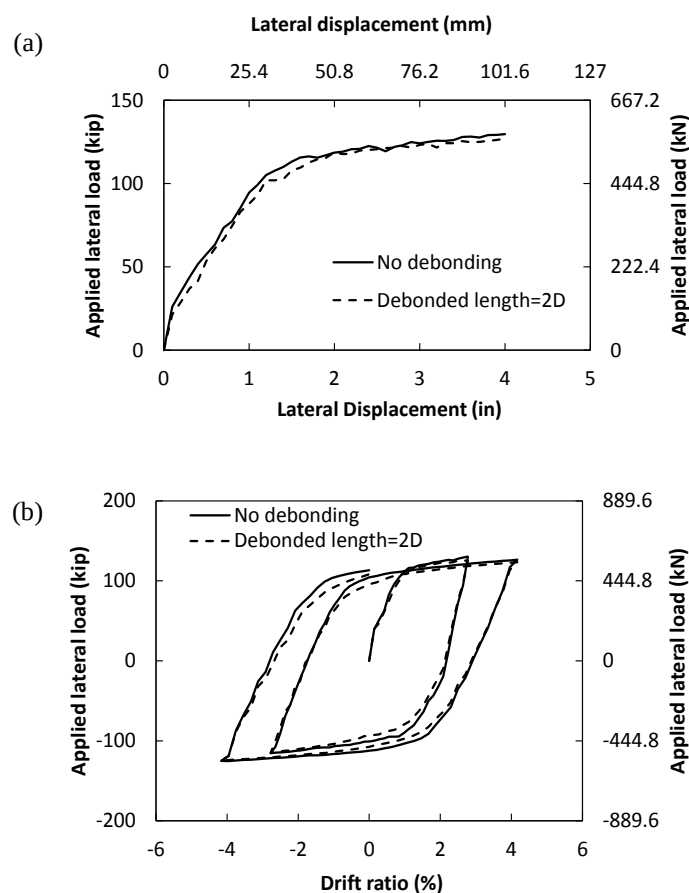


Fig. 16 – Computed seismic behavior of IMF cruciform assembly under (a) push-over loading and
(b) cyclic loading.

The cross-sectional details of the cruciform assembly are same as the IMF assembly presented in the previous section. As shown in **Fig. 15**, the lengths of the upper and lower columns are both 72 in (1829 mm, or half of the story height), and the lengths of the left and right beams are both 120 in (3048 mm, or half of the beam span). The bottom of the column was assumed to be fixed, and the two beam ends were considered to be pinned connections with no restraint of horizontal movement. Displacement-controlled lateral loads were applied at the top of the column under two different loading schemes: 1) a monotonically increasing displacement up to 4 in (102 mm) and 2) two full cycles of reversed loading with the maximum drift ratio up to about 4 %.

The computed push-over curves under the monotonic loading are presented in **Fig. 16(a)** and the load-versus-displacement loops under the cyclic loading are depicted in **Fig. 16(b)**. In both cases, a slight degradation in stiffness was observed in the cruciform with debonding relative to that with no debonding, while no significant strength degradation was observed. The overall responses indicate that debonding the bottom reinforcing bars in the vicinity of the column could have only a minor effect on the seismic resistance of the analyzed frame assembly. It must be noted that the above conclusion is based on the analysis of a beam-column assembly with continuous beam-bottom reinforcing bars. When the bottom reinforcing bars are lap spliced or cutoff within the beam span, a reduction of the recommended debonding length ($2D$) may be necessary to assure a sufficient anchorage length to achieve the full strength of bars. The proposed debonding method is not applicable to cases in which discontinuity of bottom reinforcing bars occurs within or directly adjacent to the joint region.

System level performance

A detailed finite element model was developed to consider application of the proposed debonding method to a three-dimensional frame system. The frame system was developed from a 3-story half-scale reinforced concrete frame building tested under multiple column removal scenarios⁹. The first-floor corner bay of the prototype building was extracted and modified into a 2-bay by 2-bay symmetric one-story system as shown in **Fig. 17**. The exterior columns were fixed at their bases and restrained horizontally at their tops. The load was gradually applied to the top of the center column until complete failure of the frame system. The external beams were included in the model to account for the additional torsional stiffness they provide. Only beam-bottom reinforcing bars near the center column were select for debonding. Due to the cutoff of the beam-bottom bars, the debonding length was reduced to $1D$ to assure a sufficient anchorage length.

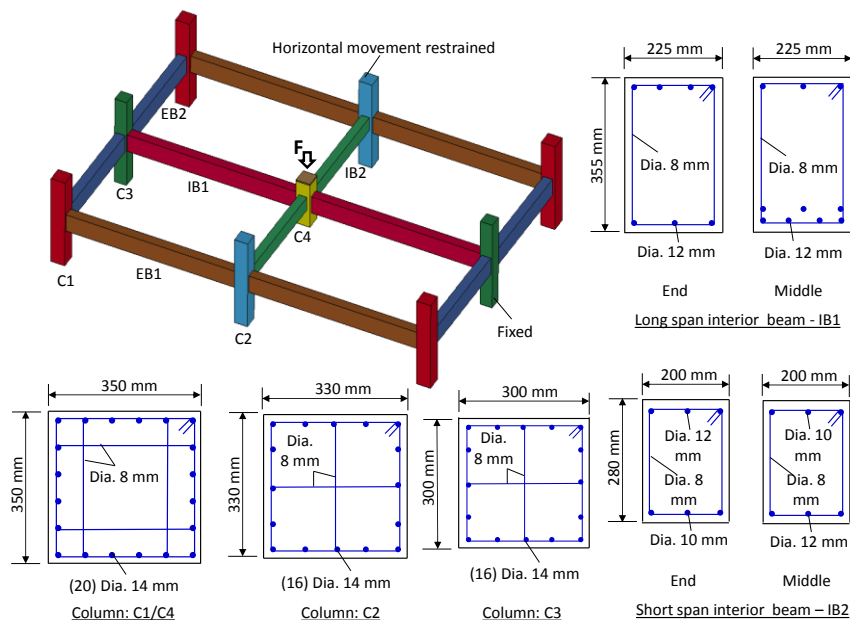


Fig. 17 – 2-bay by 2-bay frame system for analysis under a center column loss scenario.

Fig. 18 shows the computed load-versus-displacement curves for the cases with and without debonding of the beam-bottom reinforcing bars. A similar failure sequence occurred in both cases, with the first sharp drop in load caused by fracture of the bottom reinforcing bars in the short-span beams near the center column and the second sharp drop in load caused by fracture of the bottom reinforcing bars in the long-span beams close to the

center column. After the second drop in load, the frame system retained very little vertical resistance and was considered to be completely failed. Although almost identical responses were seen from the two models until the first bar fracture, the system with debonding was able to sustain a greater load at a greater displacement. The maximum load resistance was increased by almost 20 %, even under a reduced debonding length of $1D$.

The results presented in Figs. 14, 16, and 18 indicate that the proposed debonding technique is promising and warrants further study. While debonding lengths of 2 in (51 mm) and 8 in (203 mm) were tested in this study, future experimental studies should consider larger debonding lengths, since a debonding length of twice the beam depth appeared to be the most appropriate based on the computational results. While this study considered debonding of No. 8 bars, experimental results for different bar sizes would be valuable. Testing of the debonding technique within a moment-frame assembly subject to column loss should also be conducted, to demonstrate the efficacy of the debonding approach when subjected to realistic stress states associated with compressive arching action followed by catenary action. In addition, further research is needed to investigate the influence of floor slabs on the proposed debonding technique, since floor slabs contribute significantly to the overall resistance to disproportionate collapse^{10,11}. The presence of floor slabs could influence the most effective location for debonding in frame members. The debonding technique could potentially be applied to enhance the slab resistance as well, although more research would be needed to investigate the influence of debonding on the failure mechanisms of the slab.

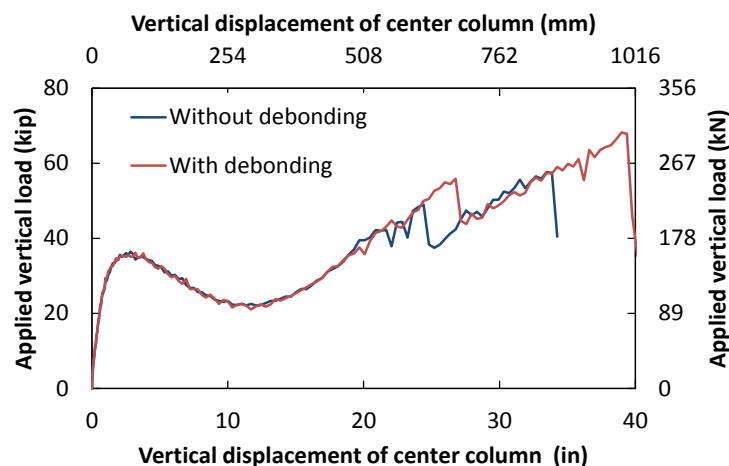


Fig. 18 – Computed vertical load versus vertical displacement of 2-bay by 2-bay frame system with and without debonding.

SUMMARY AND CONCLUSIONS

Based on the results of experimental and numerical investigations, the following conclusions are drawn:

1. Previous experimental studies revealed that the premature fracture of reinforcing bars due to strain localization limits the further development of catenary action – an important mechanism for resisting disproportionate collapse. Debonding of reinforcing bars from the surrounding concrete at locations where a major crack may occur can help to mitigate the strain localization and therefore enhance the disproportionate collapse resistance of reinforced concrete frames.
2. The proposed debonding concept and the effectiveness of a debonding technique were demonstrated through testing of embedded reinforcing bars with and without debonding the bar segment near a simulated wide crack. Experimental results showed that debonding of 8 in (203 mm) on each side of the simulated crack increased the peak displacement by about 38 %, relative to the specimen without debonding. Detailed finite element models of the test specimens were developed that captured the experimental results.
3. Using this modeling approach, parametric studies were conducted to investigate the influence of debonding length on the enhanced performance of structures. The studies showed that debonding the bottom reinforcing bars in beams from surrounding concrete, over a length of two times the beam depth from the column face, could have the following beneficial effects under a column removal scenario: (1)

mitigating or eliminating strain localization in the crack zones; (2) enhancing the beam-end rotational capacity; and (3) delaying the bar failure and improving the ultimate capacity of the frame assemblies. Results indicate that the assembly using the proposed debonding method achieved an improvement of more than 30 % in the ultimate load, with no appreciable degradation of seismic resistance in terms of stiffness or strength.

4. The system performance was evaluated through the analysis of a three-dimensional frame system. A similar failure sequence was observed in the systems with and without debonding. In both cases, the bottom reinforcing bars in the interior beams fractured. The results showed, however, that the ultimate capacity of the debonded frame system was increased by approximately 20 % compared to the system without debonding.

The results of this study indicate that the proposed debonding technique is promising and warrants further study. Future experimental studies should consider larger debonding lengths than were tested in this study, since the recommended debonding length based on the computational results was twice the beam depth. Experimental results for different bar sizes would also be valuable, and the efficacy of the debonding technique within a moment-frame assembly subject to column loss should be investigated experimentally. In addition, further studies are needed to investigate the influence of floor slabs on the proposed debonding technique, since floor slabs contribute significantly to the overall resistance to disproportionate collapse and may influence the most effective location for debonding in frame members.

DISCLAIMER

Trade names or company products are mentioned in the text to describe a procedure adequately. In no case does identification imply recommendation or endorsement by the National Institute of Standards and Technology, nor does it imply the product is the best available for the purpose.

REFERENCES

1. Lew, H. S., Bao, Y., Pujol S. and Sozen, M. A., 2014, "Experimental Study of RC assemblies under a Column Removal Scenario," *ACI Structural Journal*, V. 111, No. 4, Jul.-Aug., pp. 881-892.
2. Su, Y., Tian, Y., and Song, X., 2009, "Progressive Collapse Resistance of Axially-restrained Frame Beams," *ACI Structural Journal*, V. 106, No. 5, Sep.-Oct., pp. 600-607.
3. Yi, W., He, Q., Xiao, Y., and Kunnath, S. K., 2008, "Experimental Study on Progressive Collapse-resistant Behavior of Reinforced Concrete Frame Structures," *ACI Structural Journal*, V. 105, No. 4, Jul.-Aug., pp. 433-439.
4. Yu, J., and Tan, K. H., 2013, "Experimental and Numerical Investigation on Progressive Collapse Resistance of Reinforced Concrete Beam Column Sub-assemblages," *Engineering Structures*, V. 55, Oct., pp. 90-106.
5. Bao, Y., Lew, H.S., and Kunnath, S., 2014, "Modeling of reinforced concrete assemblies under a column removal scenario," *Journal of Structural Engineering*, ASCE, V. 140, No. 1, Jan., 04013026.
6. Hallquist, J., 2007, *LS-DYNA Keyword User's Manual*, Version 971, Livermore Software Technology Corporation, Livermore, CA.
7. Murray, Y.D., Abu-Odeh, A., and Bligh, R. (2007). "Evaluation of LS-DYNA Concrete Material Model 159." *FHWA-HRT-05-062*, Federal Highway Administration, McLean, VA.
8. American Concrete Institute (ACI), 2002, *Building code requirements for structural concrete and commentary*, ACI 318-02 and ACI 318R-02, Farmington Hills, MI.
9. Xiao, Y., Kunnath, S.K., Li, F.W., Zhao, Y.B., Lew, H.S., and Bao, Y., 2015, "Collapse test of three-story half-scale reinforced concrete frame building," *ACI Structural Journal*, V. 112, No. 4, Jul.-Aug., pp. 429-438.
10. Main, J.A., 2014, "Composite floor systems under column loss: collapse resistance and tie force requirements." *Journal of Structural Engineering*, ASCE, 140, SPECIAL ISSUE: Computational Simulation in Structural Engineering, A4014003.
11. Alashker, Y., Li, H., and El-Tawil, S., 2011, "Approximation in progressive collapse modeling." *Journal of Structural Engineering*, ASCE, 137, SPECIAL ISSUE: Commemorating 10 Years of Research since 9/11, 914-924.

NEW YORK CITY BUILDING CODE FLAT PLATE INTEGRITY PROVISIONS

By Ramon Gilsanz, PE, SE, FSEI; Karl Rubenacker, PE, SE, CWI; Jennifer Lan, PE, SE

Synopsis: This paper explains the physics behind some of the reinforced concrete structural integrity provisions in the 2014 New York City Building Code (NYCBC). These provisions were developed from 2003 to 2007 and adopted in the 2008 NYCBC and have been in effect since then. The provisions studied are the ones related to the mitigation of punching shear failure. This paper shows that the additional bars provide resistance to punching shear failure that covers nearly all dead load and reasonable and different amounts of live load. The provisions are not onerous, having been used in NYC for the last six years and the authors believe they represent an improvement over the prescriptive and non-quantified ACI provisions.

Keywords: Column Failure; Concrete; Flat Plate Construction; Integrity Steel; New York City Building Code; Punching Shear; Reinforcement; Slab Failure

Ramon Gilsanz, PE, SE, FSEI, is a founder of Gilsanz Murray Steficek. Ramon has participated on several post-disaster investigative teams. He is chair of the New York City Department of Buildings' Structural Technical Committee, a group responsible for the 2014 revision of the NYC Building Code; he is working on several committees of the Applied Technology Council, Earthquake Engineering Research Institute, American Council of Engineering Companies, Structural Engineers Association of NY, American Society of Civil Engineers, American Concrete Institute, and American Institute of Steel Construction.

Karl Rubenacker, PE, SE, CWI, is a partner at Gilsanz Murray Steficek. Karl has twenty years of structural engineering experience designing a broad array of structures from new construction to renovations, and including high rise, commercial, residential, and educational buildings. He is active in several professional committees of American Council of Engineering Companies, Structural Engineers Association of New York, American Society of Civil Engineers /Structural Engineering Institute, and the NYC Buildings Department.

Jennifer Lan, PE, SE, LEED, is an Associate at Gilsanz Murray Steficek, where she has worked on new construction projects and renovations since 2007. Jennifer holds a Bachelor of Science in Civil Engineering from MIT and a Master of Science in Civil Engineering from Stanford. She is a member of the ASCE 7 Seismic Sub-Committee and the Structural Engineers Association of New York.

INTRODUCTION

Punching shear failure may result in catastrophic failure such as the Sampoong Department Store collapse in Seoul, South Korea in 1995 that had over five hundred deaths and almost one thousand injuries (Lankov 1995). Collapses due to punching shear may occur during construction or after the building is complete due to overload, gas explosions, vehicular impact, airplane collisions and crane collapses. Both the ACI-318 and the current New York City Building Code (NYCBC) specify prescriptive requirements to improve the integrity of the slabs and provide some reserve in case of design and/or construction error. This paper shows that the resistance to punching shear failure provided by the NYCBC provisions covers the dead load and different amounts of live load. The provisions for the additional bars at the columns are not onerous, have been used in NYC for the last six years, and we believe represent an improvement on the ACI-318 provisions

SLAB FAILURE

In New York City, multifamily residential construction frequently utilizes flat-plate slabs, the design of which is often controlled by two types of failures: slab failure and column failure. Slab failure may occur as a result of a local overload of the floor (see figure 1). Column failure may occur as a result of an explosion or impact, say by a truck or a mover's trolley damages the column (see figures 2 and 3). To prevent collapse in the event of a slab failure, the integrity steel should keep the floor slab attached to the column even when the concrete around the column is damaged. To prevent collapse in the event of a column failure, the integrity steel should keep the floor slab attached to the adjacent columns so that further failure of the slab does not occur.

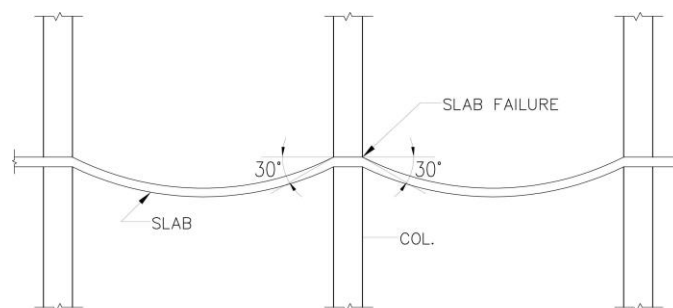


Figure 1 – Schematic section of slab failure

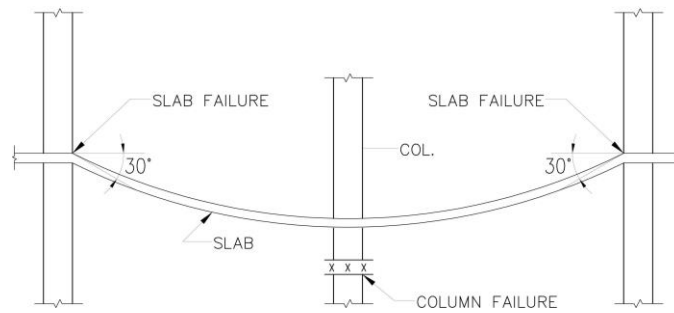


Figure 2 – Schematic section of column failure

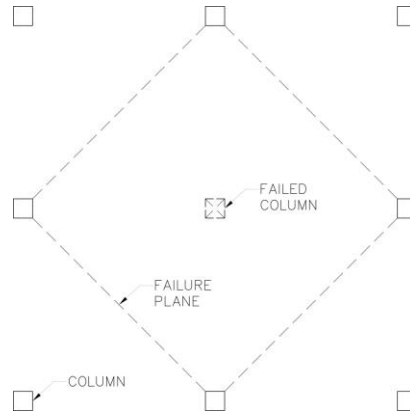


Figure 3 – Plan of tributary area during column failure

Both the ACI-318 and the NYCBC require integrity reinforcing bars go through the column cores (see figure 4, in plan; and figure 5 in section), with the reasoning that if a slab or column fails, the required reinforcing bars will act in tension to support the weight of the separated slab and transfer the load to the column. The calculated integrity steel requirement assumes that when a slab or a column fails, the bars develop an angle of incidence of 30° with respect to the supporting columns (see figure 6). This angle was determined to be a reasonable assumption, supported by a 1968 study of shearhead reinforcement by Corley and Hawkins. In this study, 21 specimens were tested and resulted in inclination of the failure surface to the horizontal of $20\text{--}30^\circ$ without shearheads, $20\text{--}45^\circ$ with over-reinforcing, and 30° with under-reinforcing (Corley and Hawkins 1968).

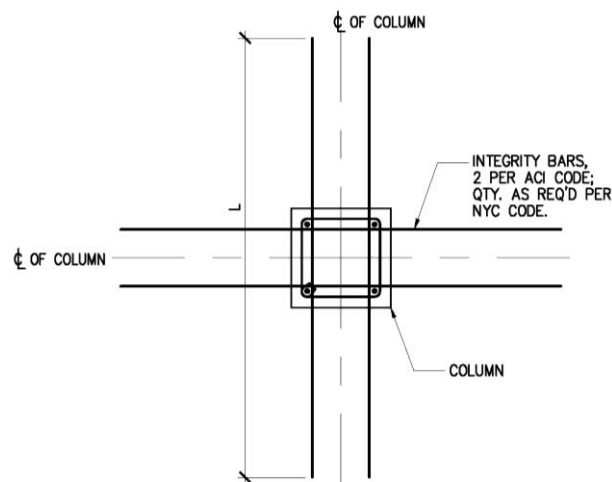


Figure 4 – Plan of typical integrity bar location

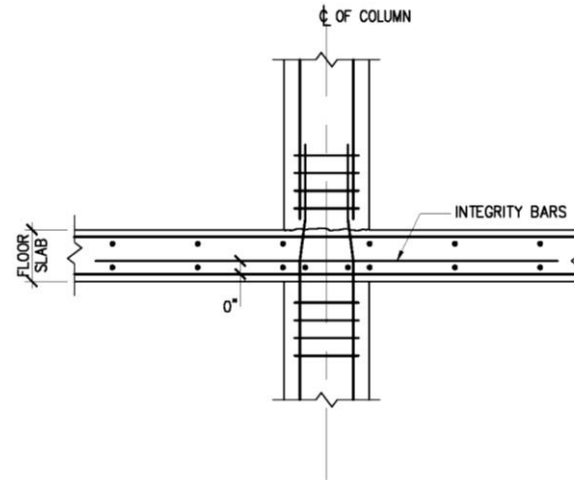


Figure 5 – Section of slab/column reinforcement before failure

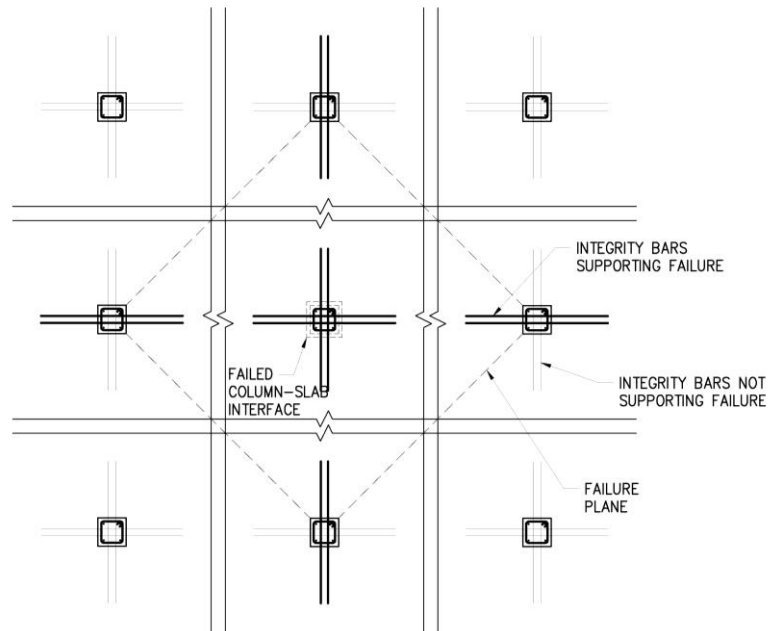


Figure 6 – Plan of integrity bars at neighboring columns after column failure

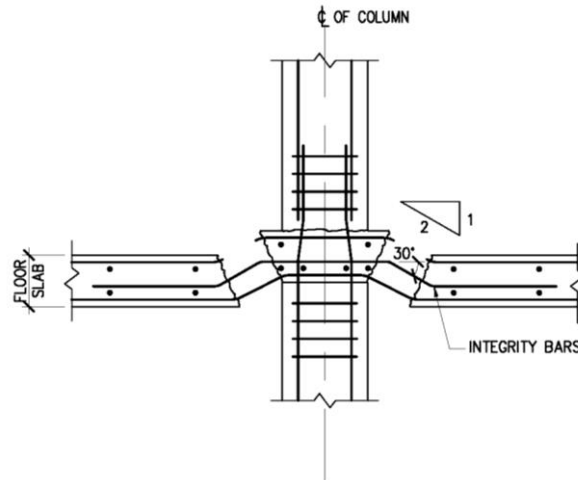


Figure 7 – Section of slab/column reinforcement after failure

In addition, to ensure a continuous load path in the event of a slab or column failure, NYCBC 1917.2.1 mandates that the slabs have a continuous bottom mat in two (approximately) perpendicular directions. In addition, the bottom reinforcement shall be anchored and shall not be less than the steel required for temperature reinforcement. (Some exceptions apply.) In comparison, ACI-318 Section 318-14 8.7.4.2.1 only requires continuous reinforcing at the column strips.

REINFORCEMENT PROVISIONS FOR STRUCTURAL INTEGRITY STEEL THROUGH COLUMN CORE

To ensure the capacity of the integrity steel at each column, NYCBC 1917.2.3 requires that the bottom reinforcement passing through the column core shall develop a tension force equal to the maximum of (1) or (2):

- (1) Three times the load entering the column at that level using a load combination of $1.0 \times$ self-weight of the slab
- (2) One and a half times the load entering the column at that level using the load combination of $(1.2 \times \text{dead load} + 1.6 \times \text{live load})$ or $(1.4 \times \text{dead load})$.

The greater of these two quantities applies. The dead load includes both self-weight and superimposed dead load. In comparison the ACI-318 code does not quantify the amount of reinforcing steel required. ACI 318-14 8.7.4.2.2 states, “At least two of the column strip bottom bars or wires in each direction shall pass within the region bounded by the longitudinal reinforcement of the column and shall be anchored at the exterior supports.”

NYCBC 1917.2.3 provisions one and two define the amount of rebar that must be added to the bottom of slabs at column cores. The first provision, which deals with dead load only, addresses the possibility of floor failure at the time of construction, specifically during the time of concrete placement. The second provision, which incorporates both dead load and live load, addresses the potential floor failure once the building has been occupied.

For both slab failure and column failure, the computation of the required amount of reinforcing steel is independent of the span lengths and of column geometries. The calculation only depends on dead load to live load ratios as well as the size of the tributary area. These provisions make no allowances for impact or dynamic effects. The formulas are based on the understanding that the origination of punching shear failure occurs slowly, with progressive development of cracks over time.

In the case of a failed column, the load is transferred to the adjacent columns. For slabs on a square column grid, the increased load in the adjacent columns is approximately 36% (see figure 7). For slabs on a rectangular column grid with a 2:1 ratio, the load in the adjacent columns increases along the shorter span by 60% (see figure 8). In most

residential buildings, even when the column placement is irregular, the spans between columns are comparable. The square and 2:1 rectangular grid bound the load distribution to adjacent columns.

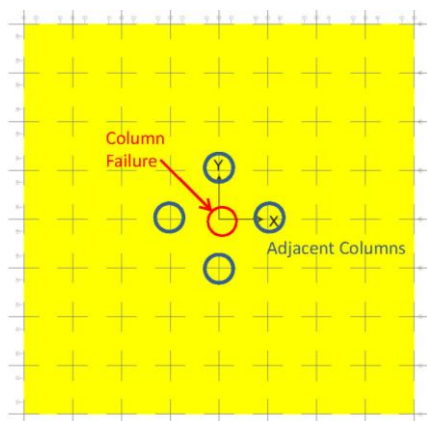


Figure 8 – Adjacent columns on a square grid

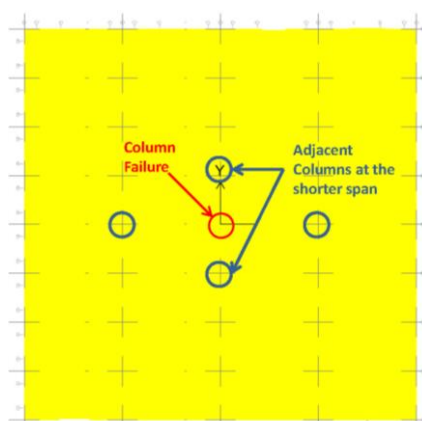


Figure 9 – Adjacent columns on a rectangular grid

APPLYING THE CODE PROVISIONS

The following scenarios illustrate the application of NYCBC's integrity code provisions for different dead load to live load ratios and different types of column grids. These examples are based on an 8" normal weight concrete slab with a dead load of 100 psf, and live loads of 150 psf (light storage), 50 psf (office), and 20 psf (roof). Since superimposed dead loads are usually small compared to the structure's self-weight in residential buildings, it is neglected in the examples below to simplify calculations. Adding in the superimposed dead loads will alter the numbers slightly but will not change the conclusions.

When the live load (LL) is equal to 0.5 times the dead load (DL), the load entering a column with a tributary area "A" is equal to $(DL+LL) \times A$ or $1.5 \times DL \times A$. Since superimposed dead loads are ignored, the dead load is equal to the slab self-weight. The code required capacity of the structural integrity reinforcement is calculated below:

NYCBC 1917.2.3 (1)

$$3 \times 1.0 \times DL = 3 \times DL \times A$$

NYCBC 1917.2.3 (2):

$$1.5 \times (1.2 \times DL + 1.6 \times LL) = 1.5 \times [(1.2 \times DL \times A) + (1.6 \times 0.5 \times DL \times A)] = 3 \times DL \times A$$

In this case the two equations give the same value. As the load entering the column is $1.5 \times DL \times A$ and assuming a 30 degree angle, the actual tension demand on the integrity steel is as follows:

Dead Load Only:

$$\frac{DL \times A}{\sin 30} = 2 \times DL \times A$$

Dead Load + Live Load:

$$\frac{[(DL \times A) + (0.5 \times DL \times A)]}{\sin 30} = 3 \times DL \times A$$

In the event of slab failure the integrity steel can support all of the dead load and live load at the column location of slab failure. Assuming a 1:1 square column grid, the adjacent columns will see a 36% increase in load in the event of a column failure. The demand becomes the larger of the two cases below:

$$\frac{1.36 \times (DL \times A)}{\sin 30} = 2.72 \times DL \times A$$

$$\frac{1.36 \times [(DL \times A) + (0.5 \times DL \times A)]}{\sin 30} = 4.08 \times DL \times A$$

In this case, the integrity steel can support 100% of the dead load and 20% of the live load, as shown below:

$$\frac{1.36 \times [(DL \times A) + (0.5 \times DL \times A \times 0.2)]}{\sin 30} = 3.00 \times DL \times A$$

Similar calculations can be applied to different live loads and column grid layouts. The results are summarized below for a square column grid:

Ratio	Capacity of integrity steel	In case of slab failure, the integrity steel at the column with the failed slab can carry	In case of column failure the integrity steel at the adjacent column can carry
LL=0.2 x DL	3 x DL x A	100% of DL and LL	100% of Dead Load 51% of Live Load
LL=0.5 x DL	3 x DL x A	100% of DL and LL	100% of Dead Load 20% of Live Load
LL=1.5 x DL	5.4 x DL x A	100% of DL and LL	100% of Dead Load 66% of Live Load

For comparison, if the grid is a 2:1 rectangular column grid, the results are:

Ratio	Capacity of integrity steel	In case of slab failure, integrity steel can carry	In case of column failure, integrity steel can carry
LL=0.2 x DL	3 x DL x A	100% of DL and LL	94% of Dead Load 0% of Live Load
LL=0.5 x DL	3 x DL x A	100% of DL and LL	94% of Dead Load 0% of Live Load
LL=1.5 x DL	5.4 x DL x A	100% of DL and LL	100% of Dead Load 46% of Live Load

STEEL QUANTITY

Consider, for example, an 18x18 foot square column grid with a #4@12" each way bottom mat and dead loads of 100 psf and live loads of 40 psf. When comparing the requirements for integrity steel through the column core, we find that the requirements set forth by the NYCBC are not onerous when compared to the ACI-318 requirements.

The differences in steel quantity would be calculated as follows:

$$\begin{aligned}
 3 \times DL &= 300 \text{ psf} \\
 1.5 \times [(1.2 \times DL) + (1.6 \times LL)] &= 276 \text{ psf} \\
 1.5 \times (1.4 \times DL) &= 210 \text{ psf} \\
 \text{Therefore, } 300 \text{ psf} &\text{ governs} \\
 300 \text{ psf} \times 18' \times 18' &= 97.2 \text{ kips} \\
 \text{Requirement} &= 97.2 \text{ kips} / (60 \text{ ksi}) = 1.6 \text{ in}^2 = 8\#4 \text{ Bars extending out from the column face } 1/3 \text{ of the span, or } 4\#4 \\
 &\text{Bars } 14 \text{ foot long (14 foot equals } 18/3 \times 2 + \text{column dimension)} \\
 \text{Steel weight} &= 4 \times 0.668 \text{ \#/ft} \times 14' = 37 \# \\
 37 \# / (18 \times 18) &= 0.11 \text{ psf}
 \end{aligned}$$

Other iterations of this calculation based on the applicable code and the dimensions of the tributary area can be summarized as follows:

Code	Tributary Area (Square Feet)	Integrity Steel Required	
		No. of Bars	#/SF
ACI	324 (18x18)	4#4s	0.12
NYC BC	324 (18x18)	4#4s	0.12
ACI	484 (22x22)	4#4s	0.09
NYC BC	484 (22x22)	4#5s	0.14

With an 18x18 grid, the steel sizes required by NYCBC are consistent with ACI if the designer uses #4 bars in the bottom mat. In the case of a 22x22 grid, NYCBC requires 4#5 bars, whereas ACI would allow designs using either #4 or #5 bars. It should be noted that if the bars from the typical bottom mat pass through the column core, they may be counted as the integrity reinforcing. This further reduces the impact of the NYCBC requirements.

CONCLUSION

This paper explains the rationale of the NYCB integrity provisions. It shows that the additional bars provided for punching shear resistance are adequate for supporting nearly all the dead load and, in most cases, a reasonable portion of the live load. The provisions are not onerous and have been used in NYC for the last six years. The authors believe they represent an improvement over the ACI-318 provisions because they provide a quantification of the amount of steel required relative to the amount of load in order to prevent failure.

REFERENCES

- The New York City Building Code sections cited in this paper reference the 2008 Code. The same provisions appear in the 2014 Code.
- Corley, W. Gene & Hawkins, Niel M. (1968) "Shearhead Reinforcement for slabs." *ACI Journal*, October 1968, 811-824.
- Lankov, Andrei (1995), "Collapse of Sampoong Department Store," *Korea Times*, 29 June 1995.

RESPONSE OF A POST-TENSIONED FLOOR FOLLOWING A COLUMN LOSS

By Leila Keyvani and Mehrdad Sasani

Synopsis: Structural response of post-tensioned floors following a severe event such as a sudden column loss is an important topic, which has not been extensively studied in the past. In this paper, the system-level response and collapse resistance of an actual post-tensioned parking garage is evaluated analytically and verified by experimental data following the sudden loss of a column. The field data demonstrated that the slab resisted collapse with a permanent maximum vertical displacement of about 2.4 in (61 mm). The development of the membrane forces and their effects on the response of the slab, particularly the region in the neighborhood of the lost column which experienced cracks at the bottom of slab, are studied analytically. According to analytical results, which are in good agreement with experimental data, slab flexural strength is increased due to formation of additional compressive membrane forces. The floor system resists the additional compressive membrane force by formation of a peripheral tensile ring.

Keywords: Concrete slabs; Flat slab; Layered shell; Membrane forces; Progressive collapse; Post-tensioned; Tension ring.

Leila Keyvani is a Part-time Lecturer at Northeastern University. She received her PhD (2014) in Structural Engineering from Northeastern University. She has studied the progressive collapse resistance of post-tensioned and reinforced concrete flat plate structures. She focuses on finite element modeling of structures' response following a threat-independent initial damage.

Mehrdad Sasani is an Associate Professor of Civil Engineering at Northeastern University, Boston, MA. He is the chair of ACI Committees 377, Performance-Based Structural Integrity & Resilience of Concrete Structures and a member of ACI Committees 369, Seismic Repair and Rehabilitation; 374, Performance-Based Seismic Design of Concrete Buildings. His research interests include behavior of reinforced concrete structures to extreme natural and man-made hazards and building and community resilience.

INTRODUCTION

About 1 billion m² (10 billion ft²) of post-tensioned (PT) slabs have been constructed in the United States (Bondy, 2012). Despite their popularity and efficiency in allowing long spans, there is lack of experimental and analytical research on the system-level response of PT structures and the resisting mechanisms to prevent progressive collapse in extreme events such as loss of an interior column. Progressive collapse is the spread of an initial local failure from element to element, which eventually results in the collapse of an entire structure (ASCE/SEI-7, 2010). The Direct Design Method is used in this study to explicitly investigate the ability of the structure to find an alternative load path and arrest the initial damage propagation (DoD, 2010; GSA, 2013). In this threat-independent method, a column is removed and the load redistribution mechanisms are studied.

Depending on the hazard nature and type, different characteristics of the structure become more significant in resisting the demands. Under severe seismic ground motions, it has long been recognized that the deformation capacity of structures are very important in resisting seismic actions (Sozen, 1981 and Sasani and Anderson, 1996). This is primarily due to the fact that the lateral forces applied to the structure during earthquakes are limited by the lateral strength of the structure, i.e. they are not fixed. Under column removal scenarios, however, because the gravity load is constant and needs to be redistributed, the strength of the structure plays a more important role in collapse resistance.

One of the resisting mechanisms of flat slab structures is floor membrane forces. The floor membrane forces have shown to contribute significantly to load transfer after a column removal. Since the 1960s, studies have shown that membrane forces developed in reinforced concrete slabs contribute to both flexural and punching strength enhancement of the slab by formation of compressive membrane forces under small deformations (Ockleston, 1958; Vecchio and Tang, 1990; Salim and Sebastian, 2003; Kai and Li, 2012; Yi et al. 2014; Keyvani et al. 2014) and tensile membrane forces under large deformations (Park, 1964; Bailey, 2001; Foster et al., 2004; Kai and Li, 2012; Dat and Hai, 2013; Dat and Tan, 2014; Yi et al. 2014). Studies conclude that under small deformations, formation of tensile strains at the mid-height of the slab due to concrete cracking and rebar yielding causes the slab to have tendency for expansion. This tendency is constrained by the surrounding slab leading to formation of compressive membrane forces at the deflected region. Experiments by Vecchio and Tang (1990) and Yi et al. (2014) showed the expansion of the slab under small deformations. Keyvani and Sasani (2015) and Keyvani et al. (2014) showed that the additional compressive membrane forces formed in the slab come to equilibrium with tensile membrane forces formed in the regions farther away from the deflected region. At large deformations, however, most of the load is carried by the formation of tensile membrane forces. The membrane forces are found to considerably affect the flexural capacity of the slab.

There are limited experimental and analytical studies on the response of PT floors to a column loss and that model the interaction between the tendon and the slab. Huang et al. (2010) studied two modeling approaches for PT beams and slab-column connections: modeling details of post-tensioning such as the tendons and the tube and a simplified model that uses springs to model the connection between the tendon and slab. The latter was validated by Huang et al. (2010) and is shown to be more efficient. Keyvani and Sasani (2015) modeled the interaction between the tendon and the slab of an actual PT parking garage surrounding the Crowne Plaza Hotel in Houston, TX using link elements available in SAP2000. The floor was modeled using grids of beam elements with discrete plasticity. In the current study, in order to better evaluate the floor response of the PT parking garage surrounding the Crowne Plaza Hotel in Houston, the floor is modeled using layered nonlinear shell elements with distributed plasticity. The use of shell elements is important in describing and characterizing the membrane actions developed in the floor. Keyvani and Sasani (2015) also discussed in general the behavior of the slab under positive moment in the vicinity of the removed column, where

the tendons and the rebars were close to the top of the section. In the current study, after briefly presenting this response using the results of the new model, the behavior of sections in which the moment demand is in the same direction as the primary moment is characterized in detail. More importantly, because the slab is modeled using shell elements, important floor responses and behavior such as the smeared crack pattern, formation of tensile ring and its extent, and membrane forces in vector form are shown that were not available in the previous model with beam elements modeling the floor system. The experimental results from the response of the actual building to an interior column removal, by explosion, is used for validation. The interaction between the tendon and the slab is modeled similar to that adopted by Keyvani and Sasani (2015) and Huang et al. (2010) using link elements. A nonlinear dynamic finite element (FE) analysis is performed which accounts for both material and geometric nonlinearities in SAP2000 (2009). The analytical results show formation of a tensile ring that surrounds the compression zone under small displacements which plays a considerable role in satisfying the horizontal equilibrium in plain concrete slab as one of the main resisting mechanisms. At large displacements, studies show formation of a concrete compression ring in the peripheral form to resist the central tensile region which can maintain stability of the slab and avoid collapse. However, no study has been performed to evaluate this pattern under small deformations.

CROWNE PLAZA HOTEL PARKING GARAGE

Figure 1 is the plan view of a portion of the parking garage of the Crowne Plaza Hotel, located in Houston, TX. Axis 5 shows the symmetry line in the plan. The garage was constructed in 1973 and completely demolished after the experiment. An interior column (column D5) is removed by explosion (see Figure 2a), and the vertical displacement of the top of the column is recorded using the average recording of two diagonal potentiometers connected to the neighboring columns as shown in Figure 2b. The floor was an 8.5 in (216 mm) thick unbonded PT concrete slab. The slab had only #4 top reinforcement at the locations of the columns as shown in Figure 1. Button-headed wire tendons, a common technique for post-tensioning in the 1960s, were used in the garage floor (Krauser, 2006). The PT cable consisted of a group of seven parallel 0.25 in (6 mm) diameter wires bundled together. The average effective post-tensioning force in the slab was 27 kip/ft (36 kN/m).

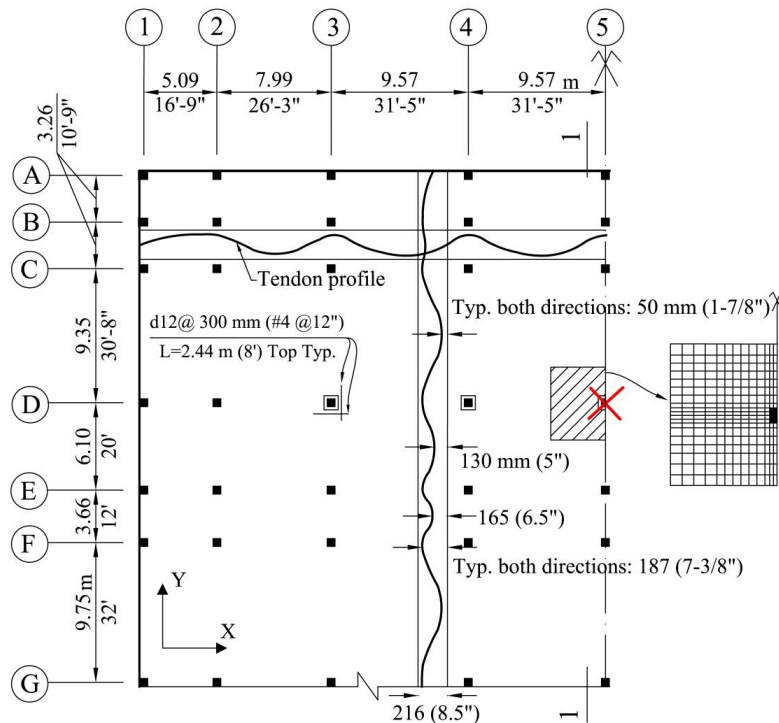


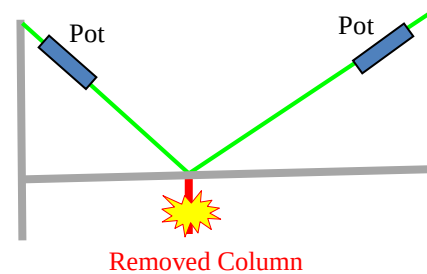
Figure 1 - Plan view of the PT parking garage (adopted from Keyvani and Sasani, 2015)

The floor is modeled using 4-node layered shell elements with coupled membrane and plate behavior. The layered shell elements can have any number of layers with different material properties and locations in the section (SAP2000, 2014). The thick plate formulation for bending is adopted in this method which includes transverse shear deformations. Internal actions are found by using standard Gauss integration procedure over the five integration points in the shell section. The membrane behavior of the shell elements is modeled nonlinearly while the transverse shear behavior is modeled linearly. Floor reinforcement at the top of the slab is modeled as two layers with equivalent thickness to represent the cross sectional area of steel in the section. These layers are modeled using nonlinear membrane behavior in X and Y directions with uniaxial material behavior.

The tendons are modeled by truss elements, and the interaction of these post-tensioning trusses and the slab shell elements is modeled by link elements which can be used for both unbonded and bonded post-tensioning. Figure 3 shows a 3D view of a portion of the PT slab modeled with trusses with links connected to the slab at mid-height. A link element constrains the movement of a truss node to its corresponding node on the slab which follows the mesh layout of the slab. For unbonded tendons, all degrees of freedoms except the horizontal component of the link elements, which is parallel to the tendon line, are rigid. Thus, the tendons are free to deform horizontally with respect to the slab. A similar modeling technique for unbonded post-tensioned slabs has been examined and verified by Huang et al. (2010) against experimental results of Foutch et al. (1990). Post-tensioning of the slab is achieved by reducing the temperature of the truss elements. Since the tendons are constrained horizontally to the slab at the ends (i.e. the location of the anchors), this temperature reduction develops compression in the slab and tension in the truss elements.



a)



b)

Figure 2 - a) Column D5 after explosion and undamaged top portion of column with deflected column reinforcement (adopted from Keyvani and Sasani, 2015), and b) diagonal potentiometers used in experiment to record the column displacement

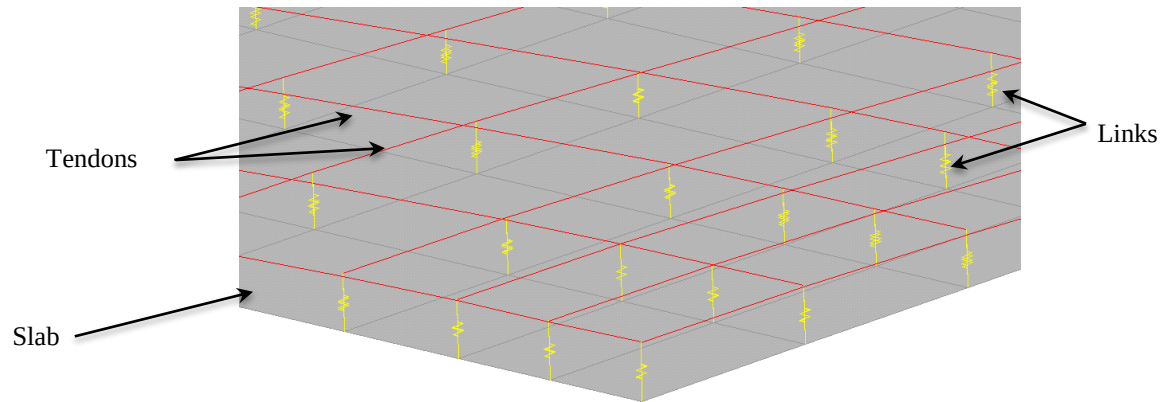


Figure 3 - A 3D view of a portion of PT slab showing tendons connected to slab at mid-height by link elements in two normal directions

Although the top portion of the removed column was undamaged (see Figure 2a) and the reinforcing bars of the lower half of the column remained in place with some lateral deformation, Keyvani and Sasani (2015) showed that the effect of reinforcement on the maximum displacement of the slab is negligible. Thus, analytically the column is removed suddenly and completely within the model. However, the weight of the undamaged portion of the column on the response is accounted for. To model the column removal, first its reactions under gravity loads and post-tensioning effects are found. Then the column is replaced with its reactions. In this case, the results of analysis under the gravity loads and post-tensioning effects are identical for the two models (with and without the column). Then, forces equal but opposite to the reactions plus the weight of the undamaged portion of the column are applied in 2 milliseconds (Sasani and Sagioglu, 2008), and a nonlinear dynamic analysis using time-integration method is performed. Hilbert-Hughes-Taylor (HHT) numerical damping is utilized to dissipate the effects of higher modes of vibration and improve numerical convergence.

Material properties

A concrete compressive strength of 4.6 ksi (32 MPa) is used based on a sample obtained from the first floor of the Crown Plaza Hotel which the garage surrounds. A linear post-peak softening is considered up to a strain of 0.006, with no residual strength left beyond this point for the compressive behavior of the concrete material. The modulus of rupture is found to be 0.5 ksi (3.5 MPa) based on ACI 318-11 recommendation. Similar to the compressive behavior, a linear tensile post-peak softening is considered up to a strain of $3\varepsilon_{cr}$ (Sinha et al., 1964; Bahn and Hsu, 1998) with no residual strength left beyond that point. For hysteretic behavior of the material, the Takeda model is adopted. The yield and ultimate strength of the reinforcement was 70 ksi (480 MPa) and 90 ksi (620 MPa) which was modeled with a bilinear behavior stress-strain relationship beyond the yield point. The tendons had yield strengths of 250 ksi (1700 MPa). The modulus of elasticity of tendons and reinforcement was about 29 000 ksi (200 GPa).

RESULTS AND DISCUSSIONS

Figure 4 shows the history of vertical displacement of the analytical model compared with the experimental data recorded from the building after column removal. Since the initial upward motion did not have significant effect on the global response of the building (Keyvani and Sasani, 2015), it was neglected for this study; therefore it was assumed that the column was completely removed after explosion. The peak and permanent displacements are -3 in (-76 mm) and -3.64 in (-93 mm) for the analytical and experimental results, respectively. In both scenarios, the slab comes to equilibrium without failure.

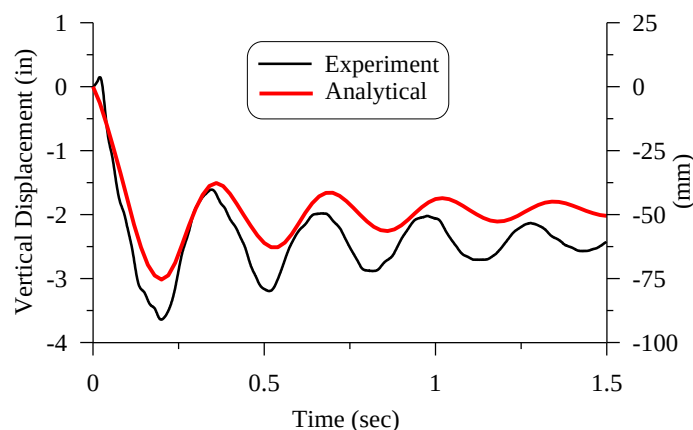


Figure 4 - Vertical displacement history of experiment and analytical model

Membrane force and moment diagrams

After column removal, as the vertical displacement increases, positive moment (tensile stress at the bottom) forms in the neighborhood of the removed column while the negative moment increases at the location of the neighboring columns. Figure 5 shows the moment distribution per unit width of the slab at the peak displacement in two normal directions. As a result of formation of positive moments around the removed column, smeared cracks form at the bottom of the slab.

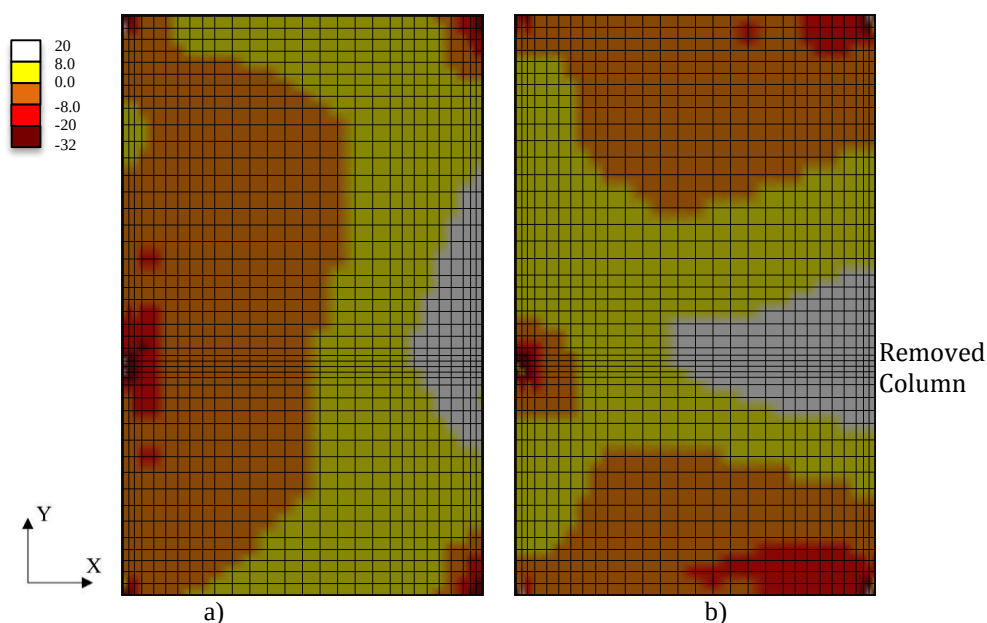


Figure 5 - Moment in two normal direction for slab (kip-ft/ft) between axes 4-5 and C-E at peak displacement
a) in longitudinal direction about Y axis; and b) in transverse direction about X axis

Figure 6a shows the total moment diagrams of the slab in the transverse direction (about axis 5 in Figure 1) before column removal (at $t=0$, i.e. under the gravity and post-tensioning effects), at the peak displacement, and at the permanent displacement. Note that the bending moments are per unit width (one foot or one meter) of the slab and are the average values over a 5 ft (1.5 m) wide strip. The total moment applied to the slab is the moment that the concrete section will experience and is the sum of three components: 1) the primary moment (due to the eccentricity of the post-tensioning force); 2) the secondary moment (due to the effects of the indeterminacy of the structure and in turn the support reactions developed as a result of post-tensioning); and 3) the moments due to the gravity loads (Khan and Williams, 1995).

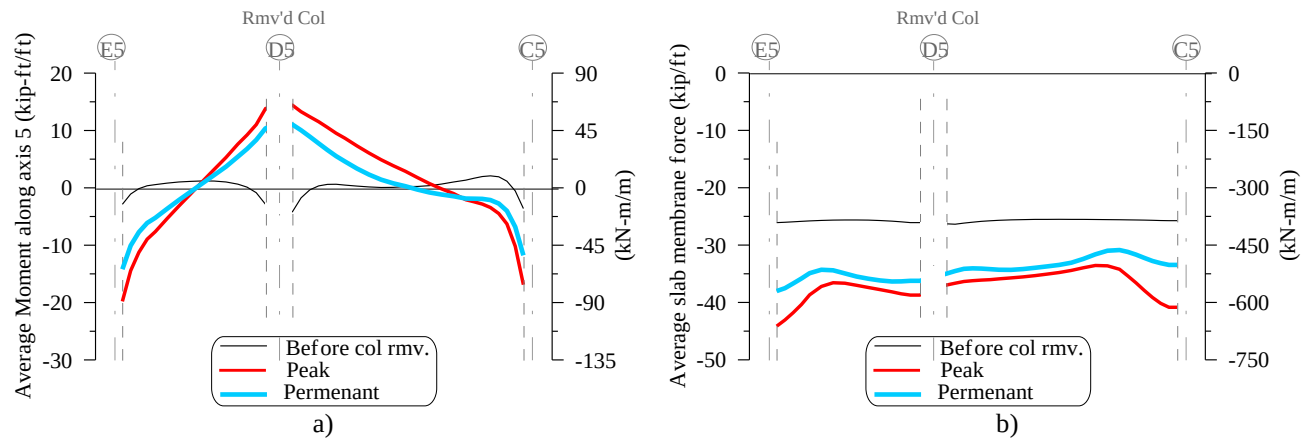


Figure 6 - Average a) total moment about X axis and b) slab membrane forces, both in transverse direction along axis 5 (symmetry line) at t=0 (Before column removal), peak and permanent displacements

At the peak displacement, as can be seen in Figure 6a, the maximum negative total moment (compression at the bottom of the section) is about 20 kip-ft/ft (90 kN-m/m) at the face of column E5. The maximum positive total moment (compression at the top of the section) is about 14 kip-ft/ft (60 kN-m/m) at the face of shear cap D5 (i.e. the location of the removed column). It is important to recognize that at both locations the tendons and the reinforcing bars are placed near the top of the section. In other words, while under negative moment at the face of column E5, the tendon and the rebars are on the *correct* side of the section; their locations within the section at the face of shear cap D5 is *incorrect*. That is, not only the external loads but also the tendons themselves impose tensile stress at the bottom of the section. As a result, deep flexural cracks form at the faces of shear cap D5. In spite of the fact that at E5, not only are the tendons on the right side of the slab's section, but there are also tensile rebars, while under the positive moment there is no tendon or reinforcing bars on the tensile (bottom) side of the section, the maximum negative total moment is only about 43% larger than the maximum positive total moment. In order to better evaluate this behavior, let us consider the post-tensioned cantilever element shown in Figure 7.

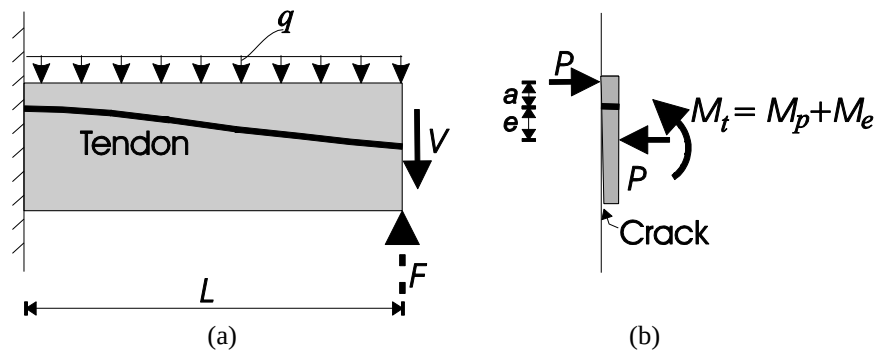


Figure 7 - a) A post-tensioned cantilever beam before cracking and b) an infinitesimally short portion after cracking

The main point here is to characterize section behavior of PT elements where the moment demand is in the same direction as the primary moment that occurs in the vicinity of the removed column. Because such a section behavior can be equally examined for beams and slabs, and for clarity, a beam element is discussed rather than a slab element. The cantilever shown in Figure 7a represents a portion of the slab in the vicinity of shear cap D5 (again, for clarity only the load transfer in one direction is discussed). That is, from the face of the shear cap represented by the fixed support at the left end of the beam to the point of inflection (zero moment) in the slab which is the tip of the cantilever. The distributed gravity load on the slab, q , and the downward shear force at the inflection point, V , are shown. In the slab, following the column removal, the shear cap moves downward compared to the rest of the slab. The dashed force, F , moves the tip of the cantilever upwards (compared to the left support), which represents the

relative displacement between the shear cap and the slab. When a tensile stress develops as a result of the upward force, F , at the bottom of the section at the support equal to the modulus of rupture of concrete, the section starts to crack. Under an increase in the same force, the crack would propagate upwards.

Figure 7b shows an infinitesimally short portion of the element at the face of the support. On the right side, the internal forces (axial and moment) are shown. The total moment, M_t , is the sum of the primary moment, M_p , and the moment due to the external loads, M_e . On the left side the resultant compressive force due to the concrete compressive stress at top of the section is shown. When the condition presented below is reached, the beam will fail.

$$(F - V) L - \frac{qL^2}{2} > P(a + e) - M_p = Pa \quad (1)$$

In Equation (1), P is the horizontal component of the post-tensioning force, e is the eccentricity of the tendon measured from the slab section mid-height, and a is the vertical distance between the location of the tendon at the support and the location of the resulting compressive force. It can be readily shown that the vertical force causing the initiation of cracking satisfies Equation (1); therefore, as soon as the cracking starts, the system will fail.

The response of the statically determinate structure discussed above is in three ways different from the response of the slab. The first difference is that the PT slab is a statically indeterminate structure and has alternative load paths to redistribute the gravity loads. The fact that the structure is statically indeterminate leads to the second difference, which is the development of the secondary moment. The third important difference is related to the tendency of the slab to elongate as a result of cracking (Sasani and Kropelnicki, 2008). Because the surrounding slab imposes in-plane constraint on this growth tendency, additional compressive force, P_{add} , will develop in the slab which could not develop in the statically determinate cantilever. This will increase the resisting moment capacity of the section to:

$$(P + P_{add})(a + e) - M_p = Pa + P_{add}(a + e) \quad (2)$$

It should be noted that considering the additional compressive force, the depth of the compressive zone will increase; therefore the value of a drops accordingly.

Figure 6b shows the axial force diagrams in the transverse direction (parallel to axis 5) before column removal (at $t=0$, i.e. under the gravity and post-tensioning effects), at the peak displacement, and at the permanent displacement. As explained before, the axial forces are per unit width (one foot or one meter) of the slab which are the average values over a 5 ft (1.5 m) wide strip. The axial force on the left side of the shear cap (Figure 6b) up to the peak displacement increases from 26 kip/ft (375 kN/m) to about 39 kip/ft (570 kN/m), i.e. a significant additional compressive force equal to 13 kip/ft (190 kN/m), which is equal to half of the post-tensioning force. If we assumed an equivalent rectangular compressive zone with $0.85f'_c$ compressive stress, the depth of the compressive zone would be $\frac{39}{(0.85)(4.6)(12)} = 0.83$ in (21 mm). Therefore the additional moment resistance of the cracked section due to the additional compressive force is equal to

$$P_{add}(a + e) = (13) \left(\frac{8.5}{2} - \frac{0.83}{2} \right) \left(\frac{1}{12} \right) = 4.2 \text{ kip-ft/ft (19kN-m/m)} \quad (3)$$

In order to better understand the contribution of different parts of the slab on load redistribution, one should consider the effects of the primary moments. This is particularly important after column removal, as at the faces of the adjacent columns the primary moment is in the opposite direction to moment due to the external loads. However, at the faces of the shear cap D5, the moment due to the external loads and the primary moment have the same direction. Assuming that the tendon force is constant during the floor response to column removal (i.e. the change in the tendon force is negligible), the primary moment in the transverse direction is shown in Figure 8a. Figure 8b shows the external moment, which is the total moment minus the primary moment. As can be seen, while the negative external moment at the face of column E5 at the peak displacement is about 26 kip-ft/ft (115 kN-m/m), this moment at the face of shear cap D5 is about 7.5 kip-ft/ft (33 kN-m/m). That is, at the location where the tendon is on the correct side of the slab, the resisting moment is about 3.5 times larger than that at the section where the tendon is on the incorrect side. Furthermore and as mentioned in Equation (3), about 4.2 kip-ft/ft (19 kN-m/m) of the 7.5 kip-ft/ft (33 kN-m/m) resisting moment is due to the additional compressive force. One can approximately calculate the resisting moment at