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REU Report 2003

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PITA PROJECTS PIT-282-01 & PIT-322-03

FINAL REPORT

2003 RESEARCH EXPERIENCES FOR UNDERGRADUATES

by

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The authors would like to thank the ATLSS Center for support of the projects. In particular, we would like to thank Frank Stokes, Peter Brian, and John Hoffner and his staff for student guidance over the summer. The authors would also like to thank Figg Engineers, Driscoll Hunt & Ewing Cole Cherry Brott Engineers, and High Steel for tours of their ongoing projects.

Table of Contents

1. Program Summary

In coordination with the Center for Advanced Technology for Large Structural Systems (ATLSS) and the Pennsylvania Infrastructure Technology alliance (PITA) a Research Experience for Undergraduates (REU) program in Structural Engineering at Lehigh University was conducted. The program exposed six students from a wide range of academic institutions to applied research (Figure 1). Students were given the opportunity to work in one of eight different thrust areas: earthquake hazard mitigation, explosive effects on structures, bridge field monitoring, fatigue and fracture, bridge systems, building systems, advanced materials, and advanced sensors. Practical research experiences were conducted with faculty advisors and graduate student mentors. In addition to the focused research experience, the program incorporated student development workshops and field trips (Figure 2). Workshops focused on developing the research abilities of the students. Site visits to various fabricators and construction projects were used to illustrate application of research concepts and the future careers available in engineering.

Figure 1: 2003 PITA – ATLSS REU Participants and PI

The goal of the program was to expose under-represented students and minorities to the field of Civil Engineering and the pursuit of advanced degrees in Structural Engineering. The program was conducted over a 10-week period from June to August (Table 1) and was coordinated by the principal investigator Clay Naito, co-principal investigator Robert Connor, and Mr. Robert Alpago the associate director of the ATLSS Center. Research supervisors included Professor Richard Sause, Emeritus Professor Ben Yen, Professor James Ricles, Assistant Professor Yunfeng Zhang, as well as the PI and Co-PI. Financial support from the ATLSS center and the Pennsylvania Infrastructure and Technology Alliance (PITA) was provided to assist with successful operation of the program.

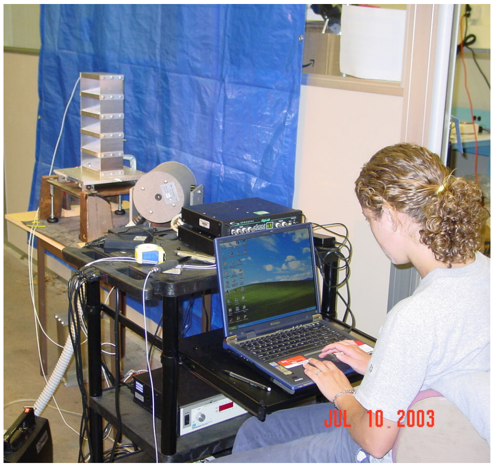

Figure 2: REU student presentations and applied research

Table 1: Program schedule	
Date	Event
2-Jun	ATLSS REU Program Start - Introductions and Welcome ATLSS Laboratory Tour Basic Safety Training Workshop
6-Jun	Advanced Safety Presentation
12-Jun	Library Search Techniques Workshop
13-Jun	REU Lunch Meeting
16-Jun	Structural Instrumentation and Testing Methods Workshop
18-Jun	Trip to High Steel Bridge Production Plant
30-Jun	PowerPoint Workshop
2-Jul	REU Lunch Meeting
9-Jul	REU Lunch Meeting
11-Jul	Preliminary Presentations
12-Jul	Group Social Activity
16-Jul	Construction site visit to Victory Bridge Replacement Project – Figg Engineers - New Jersey
23-Jul	REU Lunch Meeting
30-Jul	Philadelphia Phillies Ballpark Construction – Driscoll Hunt & Ewing Cole Cherry Brott Engineers
6-Aug	REU Final Presentations and Picnic

1.1. Nature of Student Activities

Undergraduate training revolved around applied research projects, a series of workshops, field trips, and social activities. The students worked with a graduate student mentor and faculty advisor on an ongoing research project. The students also participated in a number of activities outside of their applied research project. To improve their writing, research, and presentation skills workshops were held on library search techniques, effective presentation methods, research report writing, and experimental methods. Trips to High Steel Bridge Fabrication Plant, the Philadelphia Phillies Baseball Stadium construction site, and the Victory Bridge Precast Construction Project were integrated to give the students experience with real world applications of their research. Tours were given by contactors, state department of transportation officials, and structural designers which allowed the students to interact with a wide scope of different Pennsylvania practicing engineers. The program concluded with a 30 minute presentation of their work and submission of a final report. These reports are compiled in sections 3 to 8.

1.1.1. Applied Research Activities

The ATLSS Center is devoted to the research and construction of large-structure industries, and education of students on technology issues affecting these industries in design, fabrication, construction, inspection, and protection. REU students were given the opportunity to participate in research projects in one of the areas of expertise of the center's faculty. These areas included:

- Earthquake hazard mitigation
- Explosive effects on structures
- Bridge field monitoring
- Fatigue and fracture

- Bridge systems
- Building systems
- Advanced materials
- Advanced sensors

The 2003 REU research projects included:

- Performance Of A Shake Table And Dynamic Characteristics Of A 5-Story Model Structure
- Development Of Seismic Guidelines For Deep Column To Beam Moment Connections
- Fatigue Of Stainless Steel For Navy Ships And Other Structural Applications
- Evaluation Of Weathered Oriented Strand Board And Plywood Shear Wall Capacity
- Distribution Of Live Loads On The Riegelsville Bridge
- Structural Monitoring Of Bridges On The Northeast Extension

These projects incorporate both experimental and analytical research techniques to provide a balanced exposure to structural engineering. A typical project included laboratory work such as concrete casting, steel erection, strain gauging and instrumentation, and assembly of test setups. Analytical work included modeling and prediction of the experimental behavior, and evaluation of material properties. The summer research experience varied based on the student's abilities and research topic.

To facilitate student exposure to graduate studies and the academic environment each intern worked with a graduate student mentor and faculty advisor in an area of interest. Advisors were encouraged to include student researchers in group meetings. Proper attention was made to match students with projects that are of interest (see application in Appendix B).

1.1.2. Student Development Workshops

The program incorporated workshops covering basic skills needed for research in structural engineering. Workshops focused on both research methods and presentation techniques, they included::

- Library Search Techniques
- Presentation Organization and Delivery
- Introduction to Microsoft PowerPoint
- Preliminary Presentations
- Technical Paper Writing
- Preliminary Report Workshop
- Laboratory Instrumentation and Testing Methods

Library search techniques were taught early in the program to educate students on the techniques used to conduct literature reviews. The workshop covered the process of how to use library search systems to find information on various research topics. In addition, the students were familiarized with standard research sources such as engineering journal publications and conference proceedings.

The program provided tailored workshops on presentation organization, and effective methods of delivery for engineers. In addition, the students were given the opportunity to present their research to an engineering audience. Preliminary presentations were conducted midway through the program to assess and give feedback to the students on their presentation techniques.

The PI's, senior personnel, and other Lehigh University department staff conducted a brief workshop on technical paper writing. The focus was placed on developing research reports and culminated in a final report by each REU student. The students were guided through the program by scheduled submissions of outlines and preliminary reports at appropriate times in the program.

A research methods workshop was included in the program. Methods of conducting structural engineering experiments and techniques of instrumentation were presented. This included strain gage application, displacement and force measurement methods, and data acquisition systems. Laboratory testing was discussed and presented using research projects currently underway at the center.

1.1.3. Field Trip / Site Visits

An important part of structural engineering is the application of concepts. This includes the construction and design of bridge and building systems, repairing older systems, examining the mistakes made in construction, and understanding the degradation that occurs in the built environment. To provide a balance with the research focus of the REU program a number of application oriented engineering field trips were conducted. The trips included visits to fabricators, construction sites, and engineering offices. A balance between building and bridge projects were conducted.

The ATLSS center's strong association with a large number of regional contractors and engineers allowed for a variation of trips over the program duration. The trips included a visit to High Steel Bridge Fabrication Plant, the largest bridge manufacturer in the nation. Students viewed first hand how large structural bridge systems are produced from raw plate material to fully built bridges. A tour of the Victory Bridge Replacement Project was held by Figg Engineers (Figure 3). Students were given the opportunity to talk to the site engineer and experience first-hand the design and repair issues associated with the aging infrastructure. Dr. Robert Connor took the REU students on a tour of the Clark Summit Bridge on the Northeast Extension (Figure 4). The students were able to discuss the field work conducted by one of REU students Bridget Webb in detail (see Section 7). The final tour was of the New Philadelphia Phillies Ballpark (Figure 5). They spoke with the lead structural engineer on the project and discussed the construction management difficulties and the unusual structural design requirements that were faced.

Figure 3: PITA REU students visit Victory Bridge Replacement

Figure 4: Bridget, Kaysi and Nathan under the Clark Summit Bridge on the PA NE Extension

Figure 5: PITA REU students visit the new Philadelphia Phillies Stadium

1.1.4. Social Activities

Due to the large number of students attending the program from other Universities, social activities were undertaken to acclimate the students to Lehigh University. Student luncheons, dinner outings, and social field trips were incorporated. Wednesday luncheons were held over the duration of the program to assess the progress of the students. Luncheons were alternated between group luncheons (including all project participants) and student luncheons (including REU students and principal investigators). The group luncheons provided a casual forum to share research experiences and discuss social activities. The student luncheons provided an opportunity for the REU students to share any concerns or questions that came up with regards to the research advisors and mentors. An ATLSS Center picnic was held at the end of the program following the final presentations.

2. REU Students, Projects, and Administrators

The REU was organized and operated by Assistant Professor Clay Naito, Dr. Robert Connor, and ATLSS Associate Director Robert Alpago. Ms. Phyllis Pagel was the Accounts Manager for the project and for all ATLSS Center projects. Advertisements were distributed in early 2003 with a request for application by March of 2003. The advertisement is attached in Appendix A for reference.

Eleven students from nine different Universities applied to the program. This included Notre Dame, Lehigh University, Purdue University, Morgan State University, Auburn University, Colorado School of Mines, Lafayette College, Penn State University, and the University of Puerto Rico (UPRM). The program announcement was successful in attracting highly qualified and under represented minority students. Six students were admitted to the program; they include Kaysi-Ann Spence, Bridget Webb, Lauren Haney, Nathan Tyson, Irene LaBarca, and Carian Serrano. As a requirement of the program, the students produced detailed research reports on their work. Summary reports are included in the following sections. Details of the project organization are presented in Figure 6.

<table>
<tr><th colspan="6">Figure 6: Program organization</th></tr>
<tr><td colspan="2">
Principal Investigator
Clay Naito
Assistant Professor</td><td colspan="2">
Co-Principal Investigator
Dr. Robert Connor
Research Engineer</td><td colspan="2">
Robert Alpago
ATLSS Associate Director</td></tr>
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Notre Dame University</td><td>
Irene LaBarca
Lehigh University</td><td>
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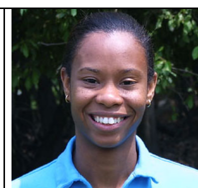
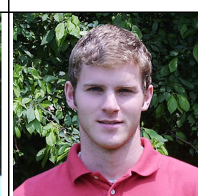
University of Puerto Rico</td><td>
Kaysi-Ann Spence
Morgan State</td><td>
Nathan Tyson
Lafayette College</td><td>
Bridget Webb
Purdue University</td></tr>
<tr><td>Research Advisor:
Yunfeng Zhang</td><td>Research Advisor:
James Ricles</td><td>Research Advisor:
Ben Yen</td><td>Research Advisor:
Clay Naito</td><td>Research Advisor:
Richard Sause</td><td>Research Advisors: Robert Connor and Ian Hodgson</td></tr>
<tr><td>Student Advisor:
Jian Li</td><td>Student Advisor:
Seoksoon Lee</td><td>Student Advisor:
Duncan Paterson</td><td>Student Advisor:
Fatih Cetisli</td><td>Student Advisor:
DaMing Yu</td><td>Student Advisor:
John Hall</td></tr>
<tr><td>Project:
Performance Of A Shake Table And Dynamic Characteristics Of A 5-Story Model Structure</td><td>Project:
Development Of Seismic Guidelines For Deep Column To Beam Moment Connections</td><td>Project:
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Distribution Of Live Loads On The Riegelsville Bridge</td><td>Project:
Structural Monitoring Of Bridges On The Northeast Extension</td></tr>
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2.1. Exit Interview and Recommendations

An exit interview was held with the REU participants to assess the weaknesses and strong points of the program. The overall opinion was that the program was well organized, educational, and enjoyable. Including projects in both building and bridge construction provided a good first hand experience of the different fields available to structural engineers. Also, discussions with practitioners were very rewarding. One of the students has gone as far as applying for a position at one of the engineering companies visited on the program.

The exit interview also indicated that the program was very beneficial in helping the students decide a career path. Five of the participants are firm in their plans to apply to graduate school with three of the students indicating that they will apply to Lehigh University.

For future programs it was strongly recommended that the responsibilities of the REU students be clearly detailed prior to their arrival. This would allow the students to focus directly on their topic from the first day. To accomplish this it is recommended that faculty advisors be required to submit a document describing the project and rough outline of the expected achievements prior to being assigned a student.

It is also recommended to organize weekly group meetings to discuss the ongoing REU projects. The format could be a brownbag lunch meeting attended by the PI but conducted by the students. This would foster more interaction between the students as well as a greater understanding of the other research being conducted. It is strongly recommended that future programs alter the weekly formal lunch meetings currently instituted to the less formal meeting style suggested by the students.

2.2. Previous Participants

A review of the 2002 REU participants was conducted to evaluate the progress that past participants have achieved. Participants in the 2002 REU Program at the ATLSS Center have all continued in the area of Engineering. Four of the five are pursuing advanced degrees in Engineering. Justin McCarthy is now completing a Masters Degree at Lehigh University in the area of structural engineering. Michael Minicozzi is pursuing a graduate degree in Material Science at Lehigh University. Fyiad Constantine is pursuing a Ph.D. at Johns Hopkins University in the area of structural engineering. Greg Parent worked as a design engineer and will be returning to Lehigh University in Spring 2004 to pursue a graduate degree in structural engineering. Edward Regnier is currently a senior at Lehigh University and will be completing his degree in May 2004.

3. REU Report - Dynamic Characterization of Structural Systems and Response to Seismic Simulation Control Measures

By Lauren M. Haney, ATLSS Undergraduate Researcher

Research Advisor: Professor Yunfeng Zhang, Graduate Student Mentor: Jian Li

3.1. Abstract

Despite extensive research efforts, earthquakes still pose a major threat to many urban and rural areas across the world. In order to reduce the property damage and fatalities caused by earthquakes, it is essential to understand the dynamic behavior of structures subjected to earthquake loading. Only then can seismic mitigation measures be tested and applied to structures located in high-seismic-risk areas. The main focus of this project is to characterize typical structural behavior and evaluate the potential benefit of using structural control devices such as nickel titanium (NiTi) superelastic wire.

The primary objectives of this project are to construct a small-scale shake table and two model structures, characterize the behavior of the model structures under seismic simulation, apply additional damping to the structures and assess the impact of these measures.

Project testing led to the conclusion that the superelastic NiTi wire did provide additional damping and energy dissipation, thus increasing the system's natural damping ratio and preventing greater displacements.

3.2. Background

3.2.1. Design and Construction of Test System

In order to understand the behavior of structures subjected to earthquake forces, it was necessary to use a shake table and model structures. In project testing, the model structures were secured to the top plate of the shake table. The shake table was either fixed to test the free vibration of a structure alone or the top plate of the shake table was connected to a vibration exciter and therefore could move relative to the bottom plate.

The shake table design underwent various revisions both before and after assembly in order to decrease assembly time and increase simplicity and precision. The final shake table design included a steel plate, measuring fourteen inches by fourteen inches by one half inch, to serve as the bottom base plate. A top plate, made of aluminum, measured twelve inches by twelve inches by one half inch. The design also included two ten-inch long shafts, each positioned on the outer edges of the bottom plate. These shafts are connected to the aluminum plate by pillow-block bearings and to the bottom plate by stainless steel angles. The pillow block bearings were manufactured to increase precision. Figure 1 shows the final design of the shake table (left), with the aluminum plate removed to emphasize detail, and a digital picture (right) of the completed shake table.

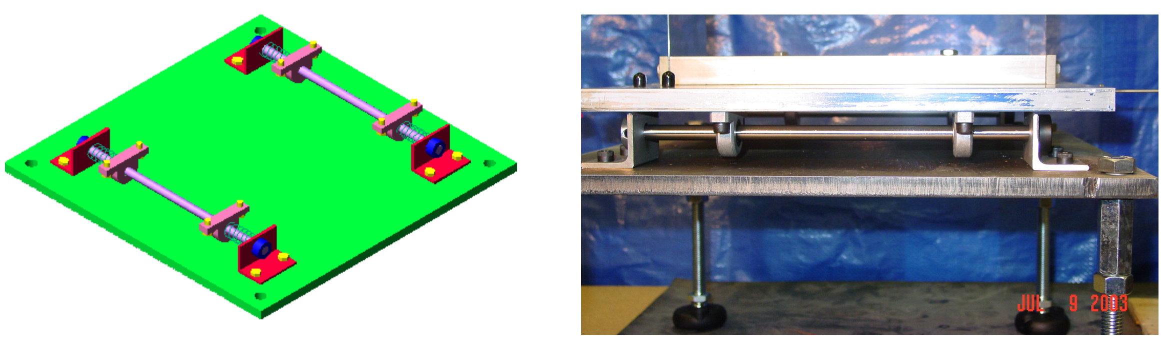

Figure 1: Final Shake Table

Two model structures were designed to represent a one-story structure and a five-story structure. The horizontal levels, or floors, of both model structures are aluminum plates. The thin sheets serving as the columns of the one-story structure are stainless steel strips. The columns of the five-story structure were changed from stainless steel to polycarbonate to reduce rigidity. Figure 2 shows the completed models.

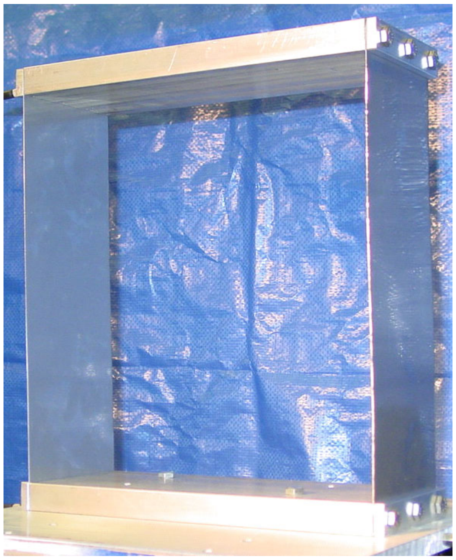


Figure 2: 1-Story and 5-Story Model Structures

3.2.2. Theoretical Predictions

Before beginning the testing, the behavior of the model structures was predicted theoretically. Ideal models of behavior were analyzed using the values of stiffness calculated to estimate the model structures' frequency. The results of experimental testing, discussed shortly, verified the theoretical predictions.

3.2.3. Shape Memory Alloys

NiTi wire, composed of nickel and titanium, belongs to a special grouping of metal alloys known as Shape Memory Alloys, well known for their ability to return to a predetermined shape or position when heated above a particular temperature. Testing was performed on three samples of NiTi wire. The wire was loaded, then unloaded and then reloaded to a higher stress continuously until failure. After each period of loading the NiTi wire returned to its original position. The wire demonstrated superelasticity because when unloaded it returned to its original shape and contained no permanent strains, even though it was stressed into a plastic region and underwent a phase change.

3.3. Overview

3.3.1. Structure Tests

The model structures were attached to the top plate of the shake table and accelerometers were placed on various levels of the structure and the top plate of the shake table. The accelerometers were connected to a digital signal analyzer so that the data from testing could be recorded and displayed.

The first set of tests required the use of an impact hammer, also connected to the digital signal analyzer, so that the impact force could be recorded. The top and bottom plates of the shake table were clamped together so that only motion of the structure was allowed. Upon striking the top level of the structures, the structures' free vibrations were recorded. The natural period and frequency of each structure was determined.

The next set of tests further verified the natural frequency of the structures. The model structures remained bolted to the table and the table remained clamped to prevent undesirable vibration. The top level of each model structure was displaced by hand and then released. The accelerometers recorded the behavior of the structure and this data further supported the previously calculated natural frequencies.

Further testing was performed on the one-story structure. The response of the structure to a range of frequencies was tested by connecting the top plate of the shake table to the vibration exciter, while also removing the table clamp. Over twenty tests were performed, each with the vibration exciter imposing a different frequency to the shake table and structure. The exciting frequencies ranged from 1 to 20 Hz. The results of the 2 Hz frequency test confirmed the calculation of the structure's natural frequency of approximately 2 Hz. As the structure and shake table were vibrated the structure responded to the vibration with a resonance frequency of 2 Hz and the acceleration was seen to increase gradually.

3.3.2. NiTi Wire Damping Tests

NiTi wire was applied solely to the one-story structure and not the five-story structure because of complexity. One long strand of NiTi wire was wrapped tightly around screws at each corner of the structure to create a diagonal bracing system across one face of the one-story structure. The structure's bottom floor was then fixed and free vibration response and impact hammer tests were repeated. Figure 3 illustrates the difference in the behavior of the original system and the system with the NiTi wire during a free vibration response test.

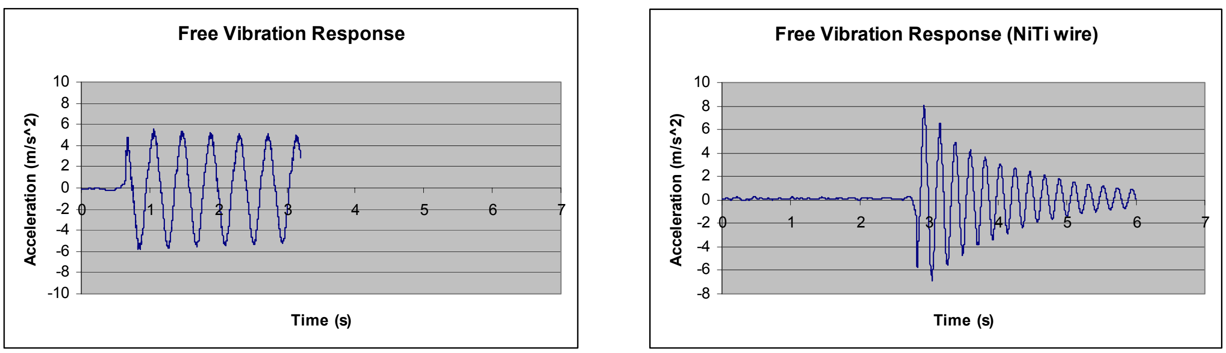

Figure 3: Comparison of System Behavior

The data collected from previous testing on the one-story structure and the data from these damping tests were analyzed to find each system's damping ratio, using the following formula,

$$\zeta = \frac{1}{2\pi j} \ln\left(\frac{\ddot{u}_i}{\ddot{u}_{i+j}}\right),$$

where j represents the number of complete vibration cycles between measurements and $\ddot{u}_i$ and $\ddot{u}_{i+j}$ represent acceleration values at specific peaks in the behavior, separated by a distance of j cycles (Chopra, 50). A comparison of the average damping ratio for each set of tests revealed that the system with the NiTi wire had a greater damping ratio than the original system, as predicted.

3.4. Conclusions

The dynamic behavior of two model structures was determined through vibration testing. The behavior of the original system was then compared to that of the modified system with additional damping measures. The application of NiTi wire damping to the one-story model structure increased the system's damping ratio, thus minimizing the vibration response of the structure. Therefore, NiTi wire may help to increase the damping in large-scale structures as well. The application of NiTi wire damping to large-scale structures will increase the system's damping ratio, exhibit a larger energy dissipation capability, and hopefully prevent damage and fatalities.

3.5. Acknowledgements

I would like to thank everyone who contributed to this project: Dr. Yunfeng Zhang, Jian Li, Joseph Zelinski, Dr. Clay Naito, Dr. Robert Connor, Lehigh University, ATLSS, PITA

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4. REU Report - Analytical Studies of Deep Column Moment-Resisting Building Frame Connections with Reduced Beam Sections

By Irene K. LaBarca, ATLSS Undergraduate Researcher

Faculty Investigator: Dr. J. M. Ricles; Graduate Researcher: Xiaofeng Zhang

4.1. Abstract

Numerous studies have been conducted to investigate the benefits of using reduced beam section beam-to-column moment connections in moment-resisting frames for buildings under high risk of earthquake damage. This type of connection system is often simplified for ease of construction and testing by not including a composite floor slab in the model. The results of this project's analytical studies indicate that including a composite floor slab in analysis and testing is essential to obtain realistic results and propose economical design theories.

4.2. Project Background and Summary

4.2.1. Background

Strong seismic loads have been shown to cause brittle fracture of beam-to-column connection welds in moment-resisting frames (MRFs). Many buildings with regular MRFs suffered severe damage during the Northridge, California earthquake of 1994. Since then, numerous studies of various design and setup have been conducted to determine the effects of using reduced beam section (RBS) connections to lessen the severity of damage to MRFs during strong earthquakes. RBS connections direct failure away from the welded connections so that yielding and local buckling occur in the beam web and flanges. Frame connections are thus more likely to remain stable long enough for safe building evacuation and subsequent inspection.

When severe seismic loads are applied to reduced beam section MRFs, the beam web and flanges yield and locally buckle, resulting in lateral displacement of the beam flanges. Driving forces in the beam flanges thus become eccentric with respect to the column, such that twisting develops in the column. This problem is especially prevalent in connections with deep columns. (Chi & Uang, 2002)

4.2.2. Project Objectives

The purpose of this project is to analyze several moment-resisting connection models with reduced beam sections and deep columns. Evaluations will include development of lateral flange deflection in the beams, development of twist in the columns, and final load capacity of the connections. Using the ABAQUS computer modeling program, two columns will be analyzed with nine W36 beams of varying weight. Each analysis will be studied with and without a composite concrete floor slab laid on top of the beams. Effects of the addition of the floor slab will be evaluated. The results of these analyses will be compared to data from full-scale models tested at Lehigh University's ATLSS civil engineering research center.

4.3. Model Setup and Analysis

4.3.1. Setup and Description

Each model includes two beams welded centerline to centerline to one 156-inch tall column. The ends of the column and beams in the model correspond to midheight and midspan of the members, respectively, where points of inflection are assumed to be located in a prototype MRF. Each beam has top and bottom 50% RBS cuts with centerlines 22.5 inches from the column faces. Supports at the ends of both beams are rollers. The column is pin-connected to the ground. Lateral loads are applied to the top of the column to produce story drift increments corresponding to 1%, 2%, 3%, and 4% of column height.

Two models are studied in each analysis, one with and one without a composite metal-concrete deck laid on the top beam flange. In actual building frames, the deck provides lateral bracing to the top flange of the beams.

4.3.2. Analysis matrix

The analysis matrix is summarized in Table 1. Two columns will be analyzed in this study. A W27x194 column will be modeled with W36 beams ranging in weight from 135 to 210 pounds per linear foot (Analysis Set A). A W36x230 column will be modeled with W36 beams ranging in weight from 135 to 236 pounds per linear foot (Analysis Set B). Beams studied with each column are selected based on weak beam-strong column criteria from the American Institute of Steel Construction Seismic Provisions (AISC, 2002). Each set of analyses will be modeled with and without a composite floor slab.

	Analysis Set			
	A		B	
Columns	**W27x194**		**W36x230**	
Beams	1	W36x135	1	W36x135
	2	W36x150	2	W36x150
	3	W36x160	3	W36x160
	4	W36x170	4	W36x170
	5	W36x182	5	W36x182
	6	W36x194	6	W36x194
	7	W36x210	7	W36x210
			8	W36x232
			9	W36x256

Table 1 Analysis Matrix

4.4. Results and Discussion

4.4.1. Flange Deflection

Lateral movement of points on each beam is monitored at the end of each drift increment. Deflection measured is orthogonal to the applied load direction. The magnitude of nodal deflection increases with drift, resulting in an increase in beam flange driving force eccentricity due to larger seismic loads. Figure 1 shows lateral flange deflection at the RBS centerline of the bottom beam flange at 4% story drift for each analysis. All data reported in this study is from the 4% story drift increment. This amount of drift corresponds to relatively strong seismic loading.

4.4.2. Column Twist

Lateral deflection is measured at points on the column, and twist about its longitudinal axis is subsequently calculated. Column twist increases with each drift increment and generally becomes appreciable (greater than one degree) at 3% drift. Figure 2 shows twist about each column's longitudinal axis at 4% drift.

4.4.3. Lateral Resisting Force

As the drift increment is increased, the frame exerts a greater resisting force in response. In other words, a greater force is required to impose a greater lateral deflection to the frame's column. In each analysis, the connection's final lateral resisting force was recorded at 4% story drift. Figure 3 shows the data for final lateral resisting force at 4% story drift.

4.4.4. Plastic Strength Deterioration

At a certain drift increment, each connection frame enters a plastic mode, and its strength and stiffness change. Local web and/or flange buckling occurs in the beams. The measured resisting force decreases, and deterioration in the frame stiffness occurs. The deterioration in strength generally occurs between 3% and 4% drift. For models that include a concrete deck, little or no strength deterioration occurs. Figure 4 compares lateral load-drift history curves for one case with and without the addition of a composite floor slab. The case shown is from Analysis Set A: W27x194 column and W36x170 beam. As the drift increment increases past 3% for the case without a composite floor slab, the connection's resisting force decreases as the connection enters a plastic mode. Very little deterioration occurs in the comparison case with a composite floor slab.

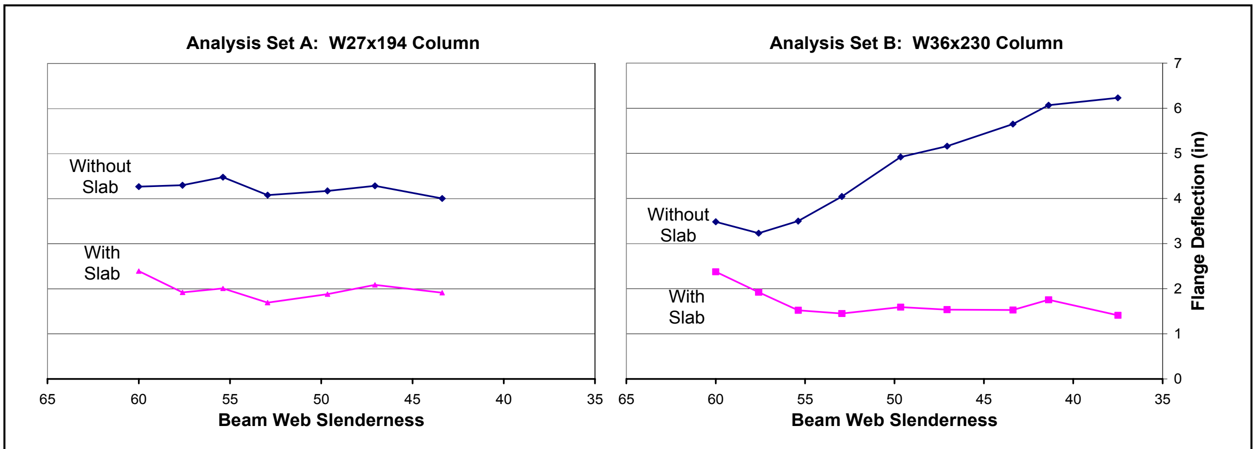

Figure 1 Maximum Flange Deflection, 4% Drift

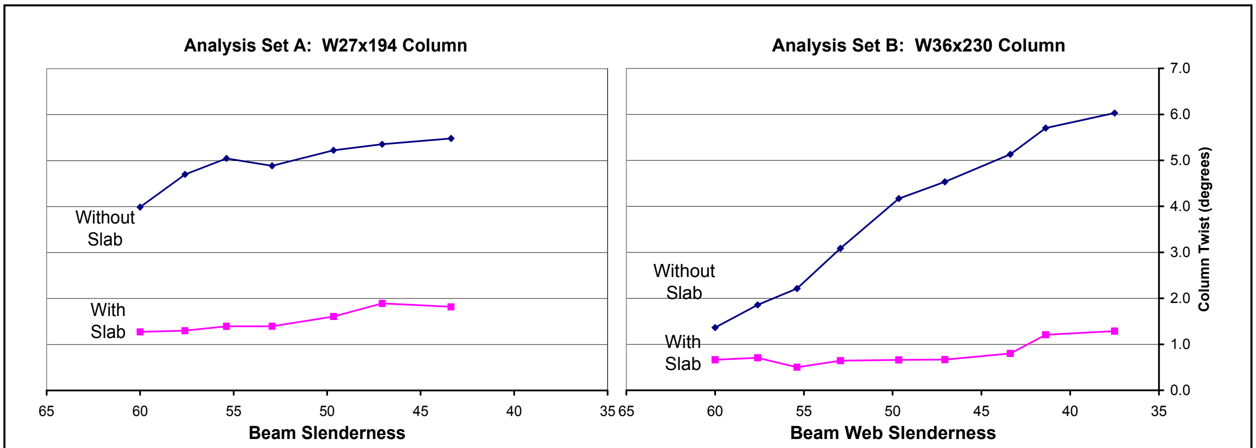

Figure 2: Maximum Column Twist, 4% Drift

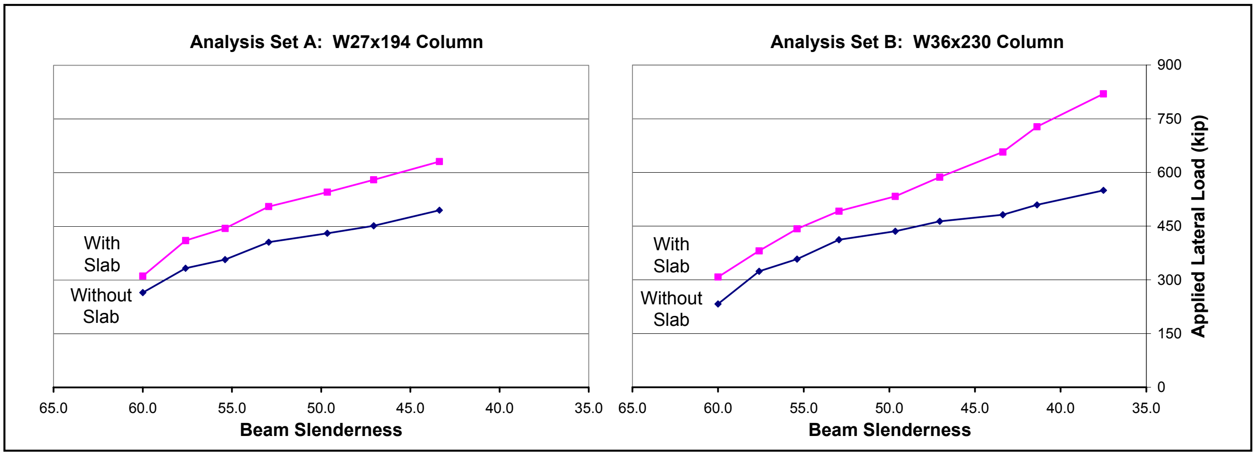

Figure 3: Final Lateral Resisting Force, 4% Drift

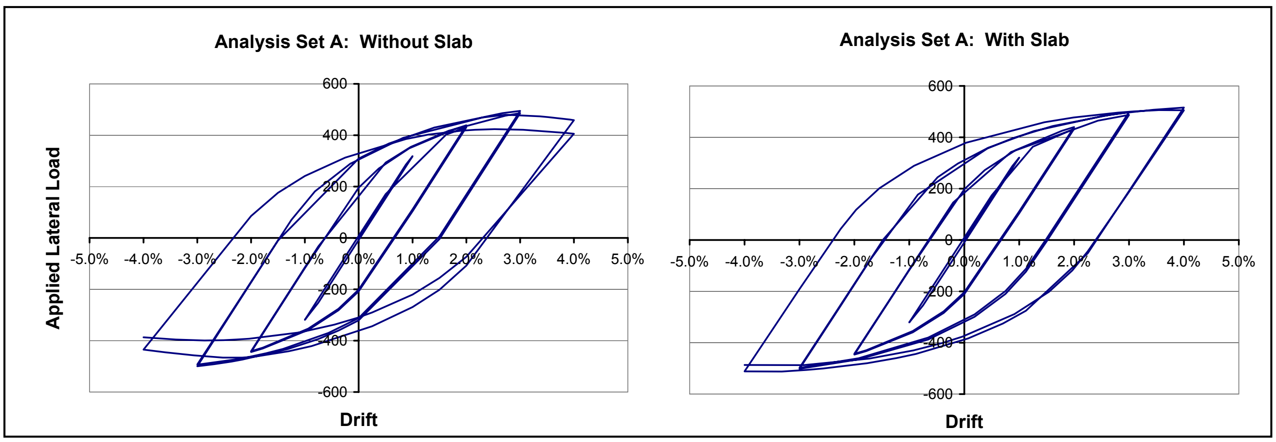

Figure 4: Lateral Load-Drift History Curves With and Without Floor Slab *(W27x194 column, W36x170 beam)*

4.5. Full-Scale Model Tests

4.5.1. Test Description

Two full-scale RBS beam-to-deep column connection specimens have undergone testing at Lehigh University. (Ricles et al., 2002) The first specimen (tested April 4, 2003) consisted of W36x150 beams connected to a W27x194 column. The second specimen (tested May 27, 2003) consisted of W36x150 beams connected to a W36x230 column. Both specimens included a composite metal-concrete deck laid on the top beam flange. Lateral loading was applied to the top of the column using a hydraulic actuator. (Ricles et al., 2002)

4.5.2. Test Results

Results of the full-scale model tests are comparable to data retrieved from the analyses completed in this study. A comparison can be made between the model test data and Analysis Sets A and B using W36x150 beams. The results of this comparison are shown in Table 2. The results of these two studies alone, however, are not enough to draw conclusions and make design recommendations. After the analysis of experimental data from four more planned tests at Lehigh University (scheduled for Fall 2003), more accurate conclusions can be drawn.

	W27x194 Column W36x150 Beam		W36x230 Column W36x150 Beam	
	Analytical	Experimental	Analytical	Experimental
Lateral Flange Deflection (in)	1.9	1.2	1.92	2.6
Column Twist (Degrees)	1.30	0.48	0.71	0.73
Lateral Resisting Force (kip)	410.3	331.5	381.0	285.5

Table 2
Comparison of Analytical and Experimental Test Data, 4% Drift

4.6. Conclusions and Discussion

The addition of a composite concrete floor slab to the model decreases lateral flange deflection. The floor slab stiffens the connection and restrains the top beam flange such that lateral movement is reduced. The results of the W27x194 column analyses show that the connections with the floor slab underwent half the lateral flange deflection of the connections without the floor slab. In the W36x230 column analyses, the lateral flange deflection was two to three times less in cases with a floor slab. (Figure 1)

The addition of a composite concrete floor slab to the model decreases the degree of column twist. The floor slab limits local flange and web buckling in the beam which is the ultimate cause of greater eccentricity in the beam flanges' driving forces. In the W27x194 column analyses, column twist in the connections with the floor slab was two to three times less than the twist in connections using the same beam weight and no floor slab. In the W36x230 analyses, the column twist was four to six times less in the models with the addition of a floor slab. (Figure 2)

The addition of a composite concrete floor slab to the model increases the connection's ultimate lateral resisting force. The floor slab reduces local buckling, limiting development of stresses in the connection and increasing its force resistance at higher story drift increments. This increase in strength is evident in both the W27x194 column analysis and the W36x230 column analysis. (Figure 3)

The addition of a composite concrete floor slab to the model reduces the connection's plastic strength deterioration. The presence of a composite concrete floor slab on top of the beam essentially braces the top beam flange and restrains its movement. This lessens the stress in the upper portion of the beam, thereby limiting or eliminating local flange and/or web buckling in the beam at 4% story drift. As this reduction in local buckling implies, less beam strength has been lost at 4% drift, and the onset of further plastic behavior is delayed.

Excluding the "W36x230 Without Floor Slab" scenario, lateral flange deflection remains constant as the W36 beam weight increases. For the three scenarios other than "W36x230 Without Floor Slab," variation in lateral flange movement is less than one inch as the W36 beam weight increases. This phenomenon can be explained by interactions in beam and flange thicknesses. As beam weight increases, both parameters increase. The increase in beam flange thickness (which increases the area of the flange) causes the driving forces in the flanges to increase, while the increase in beam web thickness causes the beam to become stockier. The increasing stockiness counteracts an increase in lateral flange deflection that an increased beam flange driving force would cause alone, and thus the lateral flange movement of the beam remains constant.

4.7. Acknowledgements

Dr. J. M. Ricles (Primary Advisor, P.I.), Mr. Xiaofeng Zhang (Graduate Advisor), Dr. Clay J. Naito (REU Program Coordinator), Dr. Robert J. Conner (REU Program Coordinator), Center for Advanced Technology for Large Structural Systems (ATLSS), Pennsylvania Infrastructure Technology Alliance (PITA), Lehigh University

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5. REU Report - Painting Fatigue of the Advance Double Hull of AL-6XN Steel
By Carián Rivera, ATLSS Undergraduate Researcher

Research Advisor: Professor Ben T. Yen and Graduate Student Mentor: Duncan Paterson

5.1. Abstract

The need to construct a nonmagnetic stainless steel ship hull that provides a better corrosion resistance and fracture control while lowering the life-cycle cost of materials has necessitated the feasibility study of the of the Advanced Double Hall (ADH) concept for U.S. Navy surface combatant ship. A program coordinated through the ATLSS Engineering Research Center is to study the overall structural behavior.

First, modeling the AL-6XN super-austenitic stainless steel single-box girders in ABAQUS, a finite element modeling program, will help to study the behavior of stresses depending on the initial out-of-flatness (OOF) in tension or compression load. Difficulties in modeling the proper boundary conditions in the ABAQUS model that represents the single-cell test box lead to results that did not match with the experimental test results (previously competed at ATLSS)

Second, measurements of a three-cell box girder initial OOF have been done to make conclusions on a three-cell test compare with the single-cell specimen results. The triple-cell is expected to crack first in the weld toe of the flange-diaphragm junction of the middle cell.

5.2. Background

The advanced double hull (ADH) concept consist of two hull plating, an inner hull and an outer hull, which are connected at intervals by longitudinal members, thus resulting in a cellular hull construction. This contrasts with the conventional single hull design in which an outer hull plate is stiffened on its interior surface by a grid of longitudinal and transverse members. With the introduction of the advanced double hull concept for ship structures using multi-cellular box girder design, it is not uncommon for sides of the box sections to buckle in compression. The flange and the web plates of the boxes deflected repeatedly out-of-plane (panting).

The structural integrity and manufacturing technology required for ADH ships utilizing low-carbon HSLA steels has been demonstrated [1]. Tests on single cell box girders subjected to full stress reversal to assess fatigue crack development from out-of-plane deformation of the box flange plates simulating components of double hull ship structures has been done [5]. Local stresses ranges along the boundaries of flanges plate panels were much higher than the nominal stress ranges and fatigue cracks developed at these locations of higher stress ranges. This behavior of crack appearing at locations of maximum stress has been witnessed in previous studies examining repeated out-of-plane displacements [3, 4]

5.3. Description

5.3.1. FEA Analysis

A simplified model was used to evaluate the feasibility of reducing the analysis to only the flange plate of the specimen. A 36” x 24” x 0.405” plate was modeled with the following boundary conditions: symmetry about the Y axis along the bottom boundary (to represent the full 36” x 48” dimension of the flange plate); no displacement/rotation in the U1, U3, UR1, UR2 UR3 direction along all other sides to simulate a fixed condition; uniform static load in the Y direction in tension and compression along the top boundary to simulate the applied load. The profile of the plate was introduced as an initially deflected sinusoidal curve. Because previous studies of a single-cell box girders have shown that the maximum stresses occurred at the weld toe of the flange-diaphragm junction in the middle-span and this stresses are related with the initial deflection, the analytical procedure involved changing the initial deflection profile of the plate to 0.001”, 0.01”, 0.1”, 0.25”, 0.5” and 0.625” and plot the stress in the (0”,24”) and (18”,0”) coordinate (at the boundaries) vs. the initial deflection. It is expected that the graphs would show that as initial OOF increases, so will the stresses at the boundaries.

5.3.2. OOF Measurements

To measure the initial OOF of the bottom flange of the triple-cell specimen a 2” by 2” grid was draw over a rectangle of 100”x 48” in the middle section of the flange, encompassing the area of interest between the diaphragms. OOF was then measured with a laser level at each point in the grid area. Predictions are made for the behavior of the 3-Cell specimen based on the previous experimental testing of single cell specimens.

5.4. Results discussion

5.4.1. FEA Analysis

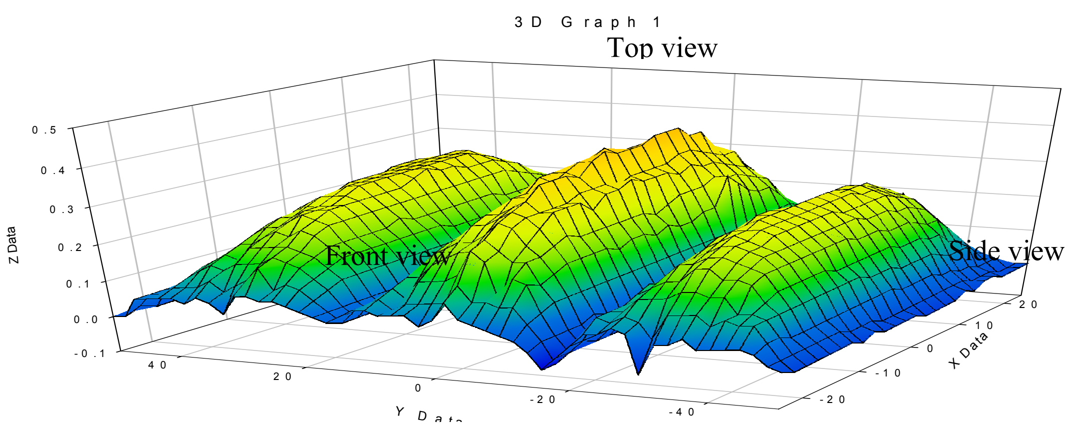
To ensure that the ABAQUS model accurately represents a real cell test specimens it is compared to the single-cell test results of the top flange of Specimen 1C-30 (box girder subjected to a nominal stress of 30 ksi) [5] that has a maximum initial deflection equal to 0.234” with ABAQUS results of the 0.250” initial deflections. In the table, S11 and S22 refer to the stresses in the y and x direction, respectively, while positive and negative refer to the top and bottom plate surfaces, respectively.

STRESS							
Compression		1C-30			ABAQUS		
Top		(0”,24”)	(0”,0”)	(18”,0”)	(0”,24”)	(0”,0”)	(18”,0”)
S11	positive	2.8	-3.8	16	-4.822	15.287	-24.113
	negative				-3.673	-11.910	23.626
S22	positive	3	**-18**	-15.4	-16.072	**0.740**	-24.072
	negative	-34			-12.268	-23.103	-9.724

However, the results do not match. So we need to re-arrange the model. Some factors that could be considered to improve the ABAQUS model are: instead of just modeling the plate, the model should be assembled for the entire box specimen; instead of a uniform load the effects of shear lag should be considered.

5.4.2. OOF Measurements

The profile of the three-cell specimen demonstrates that the cell in the middle of the specimen is more deflected initially. Many factors can contribute to the differences in initial OOF, however for the three cell box, one likely explanation is that it the center cell dissipates heat more slowly than the exterior counterparts.



Since the center cell has greater initial OOF, it is expected to have the higher stress in the weld toe of the flange-diaphragm junction in the center cell of the box girder. So, based in the results of the single cell specimens [5], the center cell it is expected to crack first.

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5.6. Acknowledgements

The author would like to thank Duncan Paterson, Prof. Yen, also to the ATLSS Engineering Center Laboratory staff, John Hoffner, Russ Longenbach, Roger Moyer, Dave Altemus and Paul Weber. Special thanks to Drs. Rob Conner and Clay Naito, Bob Alpago and Betty MacAdam for coordinating the summer REU program.

6. REU Report - Evaluation of Weathered Oriented Strand Board and Plywood Shear Wall Capacity for Use in Woodframe Construction

By Kaysi-Ann Spence, ATLSS Undergraduate Researcher

Research Advisor: Clay J. Naito, Ph.D., P.E.

6.1. Abstract

A study wood frame shear wall sheathing materials subjected to weathering is conducted. Two different sheathing materials are used to examine their respective strength loss after exposed to one year of simulated weathering. The sheathing materials used are oriented strand board and plywood. Five sets of test specimen were fabricated for each material. Three of the sets were exposed to weathering. Of the three sets exposed to weathering one was nailed to the wood stud while the other two only sheathing was exposed. The weathering technique used was a five-day boil and dry variation of the ASTM D 1037. The results show that the strength loss due to weathering occurred in the sheathing and not in the wood stud. The load bearing behavior of the specimens changed due to weathering, specimens showed brittle behavior after they were exposed to weathering. Both quasi-static and rapidly moving tests were done. For the configurations tested both plywood and OSB demonstrated comparable reductions in strength.

6.2. Background

The area of woodframe construction is primarily limited to residential applications such as single-family homes or low-rise housing complexes. Research in this area has been traditionally very minimal. In general, research dollars are not spent in this area until problems arise. The Northridge earthquake in 1994 saw damage being predominated by woodframe construction in all three basic categories of earthquake loss; casualties, property loss and functionality of buildings.

There have been previous studies done to evaluate the strength of shear walls however the area of weathering effects has been untouched. Additionally oriented strand board (OSB) has been realized to be an economical alternative to the more traditional plywood construction and its popularity has been supported by its experimentally proven performance in shear wall tests. The question that remains is whether OSB is a feasible alternative when considering the durability after weathering.

This study sought to facilitate the increase of the knowledge and performance of woodframe construction after exposure to prolonged periods of weathering. Evaluation and comparison of two different types of common sheathing materials- Oriented Strand Board and Plywood are conducted.

6.3. Literature Review

A shear wall is essentially a vertical element of the lateral force resisting system in a building, it has four major components: framing members, sheathing, nails, and hold-downs.

There are two major functions of a shear wall; they are to provide strength to resist shear loads, and stiffness to control side-sway. When the sheathing is properly fastened to the stud wall framing, the shear wall can resist forces directed along the length of the wall. When designed and constructed properly, shear walls will have the strength and stiffness to resist the horizontal forces generated by wind and earthquakes.

6.3.1. OSB and Plywood

As previously mentioned OSB has become increasingly popular and has replaced traditional plywood in many areas of the US. Though both are wood composites there is a great difference in the manufacture of each. Plywood is made by shaving thin strips, or plies, of veneer from logs. After the veneer has been dried and graded, adhesive is applied. Each layer of veneer is oriented at 90 degrees to the layer above and below it. The glued pieces of veneer are then placed in a hot press. The heat and pressure allow the glue to penetrate deeply into the wood fibers, producing a lasting bond. The layering or cross-lamination of the plies is vital, giving the plywood superior strength and stiffness. The cross layering also minimizes expansion and contraction and eliminates splitting.

OSB is made in much the same way, not with large sheets of solid wood veneer but, rather, with thousands of 3-and 4-inch strands of solid wood, which allows younger trees to be used in this product. The strands are oriented so they overlap and interlock at a 90-degree angle. Each strand of wood is completely coated with high-performance resin glue. After the OSB leaves the hot press, the product is strong and durable.

6.3.2. Weathering

Weathering is the result of the exposure to the elements of the environment. It causes mechanical changes and chemical reactions within the wood composite. It has been found that weathering can be simulated in the laboratory for experimental purposes by utilizing accelerated-aging techniques. Two variations of the ASTM D 1037 accelerated-aging test have shown bending strength and stiffness reduction after both the standard and simplified exposures were about the same for the particle panel products (McNatt, McDonald 1993). The accelerated-aging method used in this study will be presented later.

6.4. Experimental Procedure

The study was conducted under two conditions, a quasi-static phase and a dynamic phase. A matrix of the tests performed is shown in Table 1 and Table 2. Weathering was done in accordance to Variation B of the ASTM D 1037. Each exposure cycle (24 hr.) consisted of the steps outlined in Table 3; the 24-hour cycle was repeated four times. After the fourth repetition the wood samples was reconditioned in room temperature (McNatt, McDonald 1993). The test specimen was designed as a variation of the test specimen used by Fonseca, and is shown in Figure 1. The specimens were tested in the 810 Material Test System (MTS); setup is shown in Figure 2.

The wood stud was a 2x4 Douglas Fir-Larch Structural No. 1, the sheathings used were 3/8in. plywood and a 15/32in. OSB. The nails used were 8d common nails (2 ½ in. by 0.131 in diameter) and were driven flush with the sheathing using a Stanley Bostitch pneumatic nail gun.

6.4.1. Test matrix

The OSB (O) and plywood (P) shear specimens are tested in two conditions, unweathered (U) and weathered (W). The specimens were tested monotonically under load displacement control. To account for the inherent variability of wood, three tests of each detail are conducted. The test matrix is shown in Table 1 and 2. OUM1-OUM3 and PUM1-PUM3 were specimens that were unweathered, OWM4-6 and PWM4-PWM6 had both sheathing and wood stud weathered, and OWM7-OWM9 and PWM7-PWM9 had only the sheathing weathered. OUMA-OUMC and PUMA-PUMC are unweathered, while OWMA-OWMC and PWMA-PWMC have the sheathing only exposed to weathering.

OSB	OUM1	OUM2	OUM3						
	OWM1	OWM2	OWM3	OWM4	OWM5	OWM6	OWM7	OWM8	OWM9
Plywood	PUM1	PUM2	PUM3						
	PWM1	PWM2	PWM3	PWM4	PWM5	PWM6	PWM7	PWM8	PWM9

Table 1: Test Matrix- Quasi-static loading

OSB	OUMA	OUMB	OUMC
	OWMA	OWMB	OWMC
Plywood	PUMA	PUMB	PUMC
	PWMA	PWMB	PWMC

Table 2: Test Matrix- Rapid loading

6.4.2. Weathering Technique

Step	Exposure	Time (hr)
1	Water soaking (120°F)	8
2	Dry air heating (210°F)	16

Table 3: Cycle for alternate accelerated-aging exposures (Variation B)

6.4.3. Experimental Setup

For quasi-static testing, a 0.03-in/min rate of loading corresponding to a movement of the testing machine head will be used to failure (ASTM E 72-98). For rapid moving testing, a 4.4-in/min rate was used. The loading protocol was consistent for both weathered and unweathered specimens.

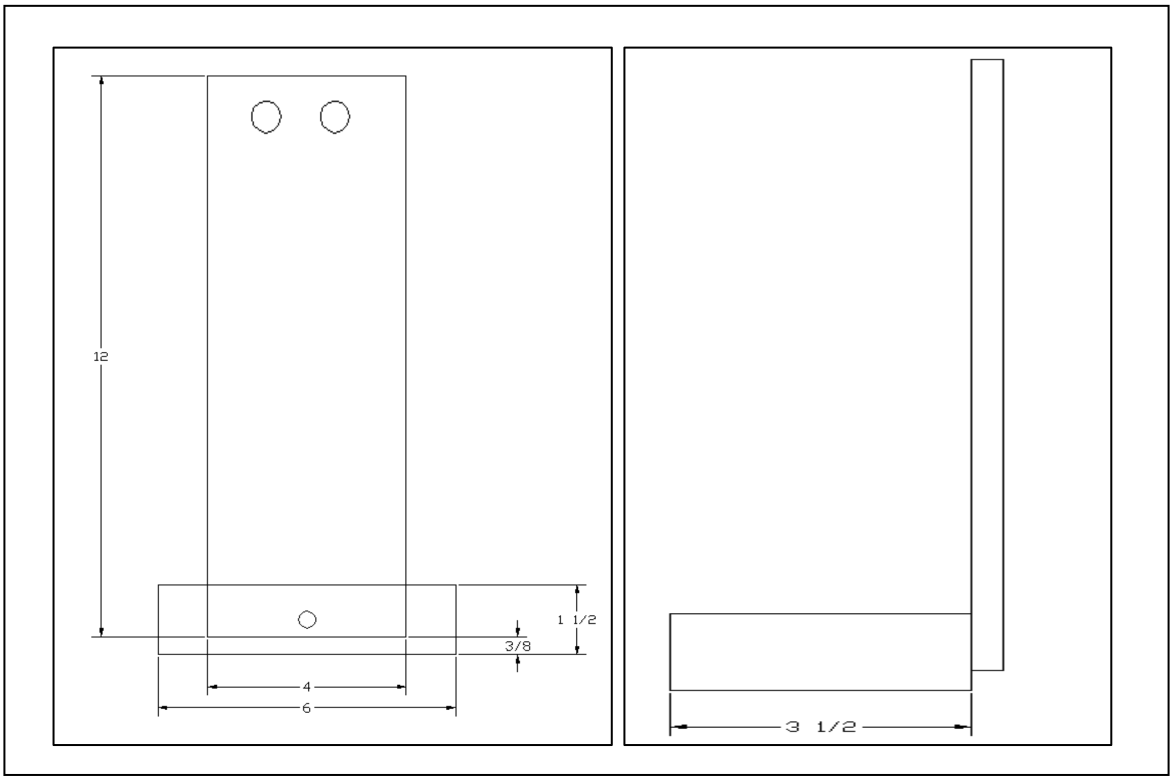

Figure 1: Diagram of set-up (Front and side views)

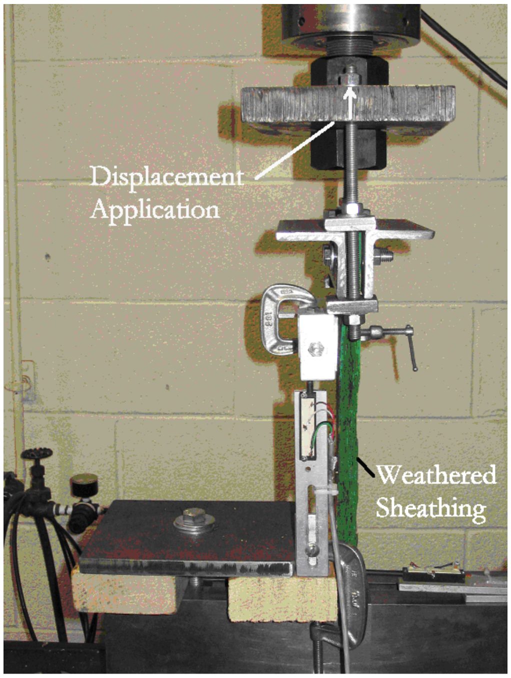

Figure 2: Test set-up in MTS

6.5. Results and Discussion

Figures 3 and 4 show the results of the quasi-static loading. The plywood produced a better consistency between tests than the OSB. There was not much variability with the results for the specimens with both the wood stud and the sheathing weathered and with just the sheathing alone weathered, since hardboard properties are much less affected by the modified aging process (NcNatt, McDonald 1993), this indicates that the changes resulted in the sheathings.

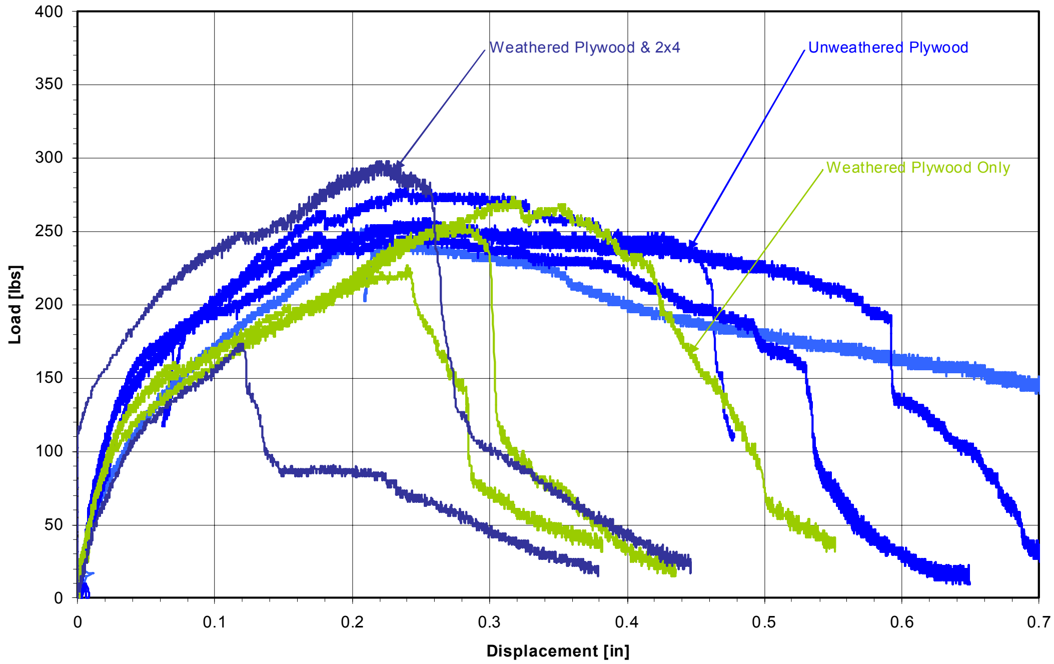

Figure 3: Quasi-static results for Plywood specimen

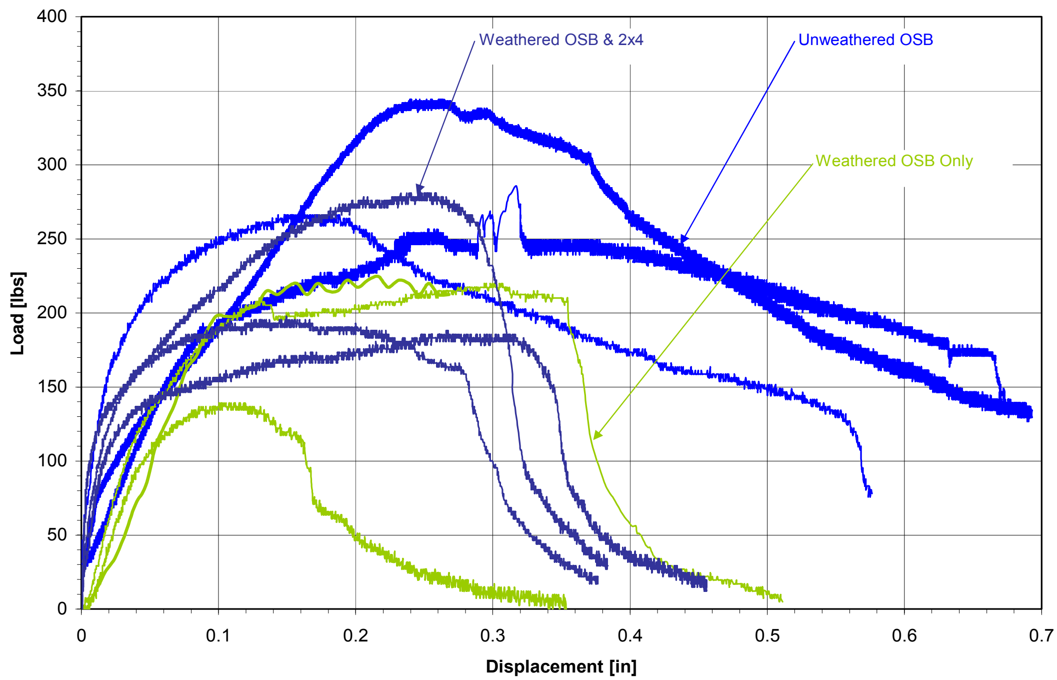

Figure 4: Quasi-static results for OSB specimen

The maximum load capacity for both unweathered and weathered plywood appears at a deflection of approximately 0.24 in. with a slightly lower load capacity seen in the weathered specimens, with OSB maximum load capacity is seen at an average displacement of 0.21 in. for unweathered and 0.17 in. for weathered specimens. The sudden drop after peak load capacity, weathered samples, seen in Figures 3 through 6 indicates a loss of ductility after exposed to weathering.

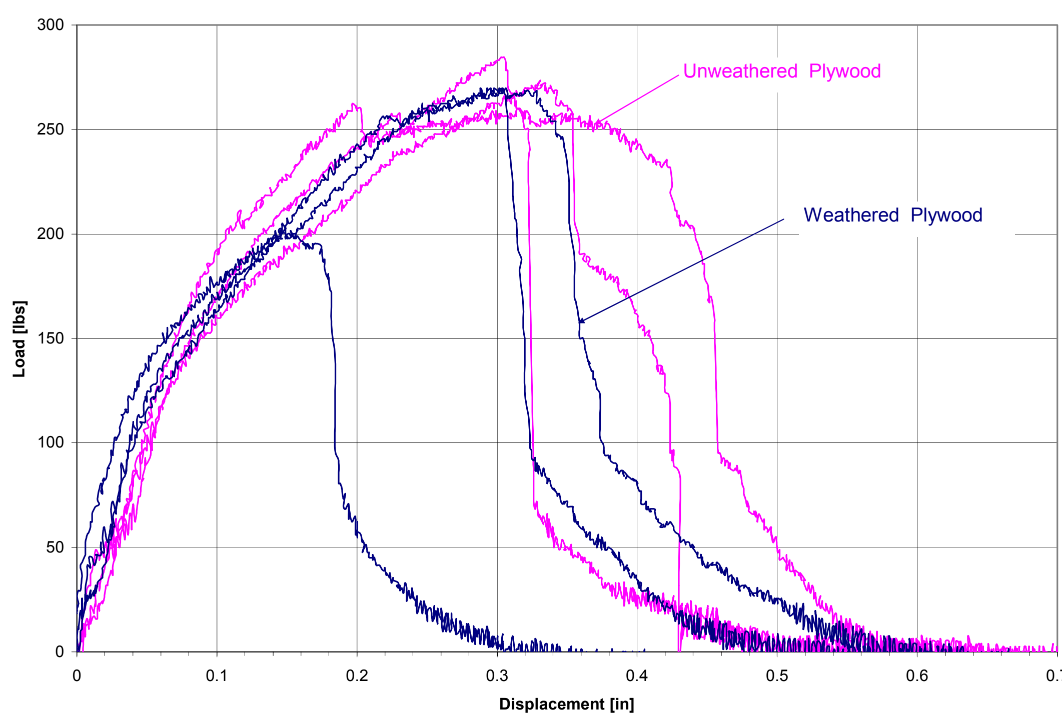

Figure 5: Rapid loading results for Plywood specimen

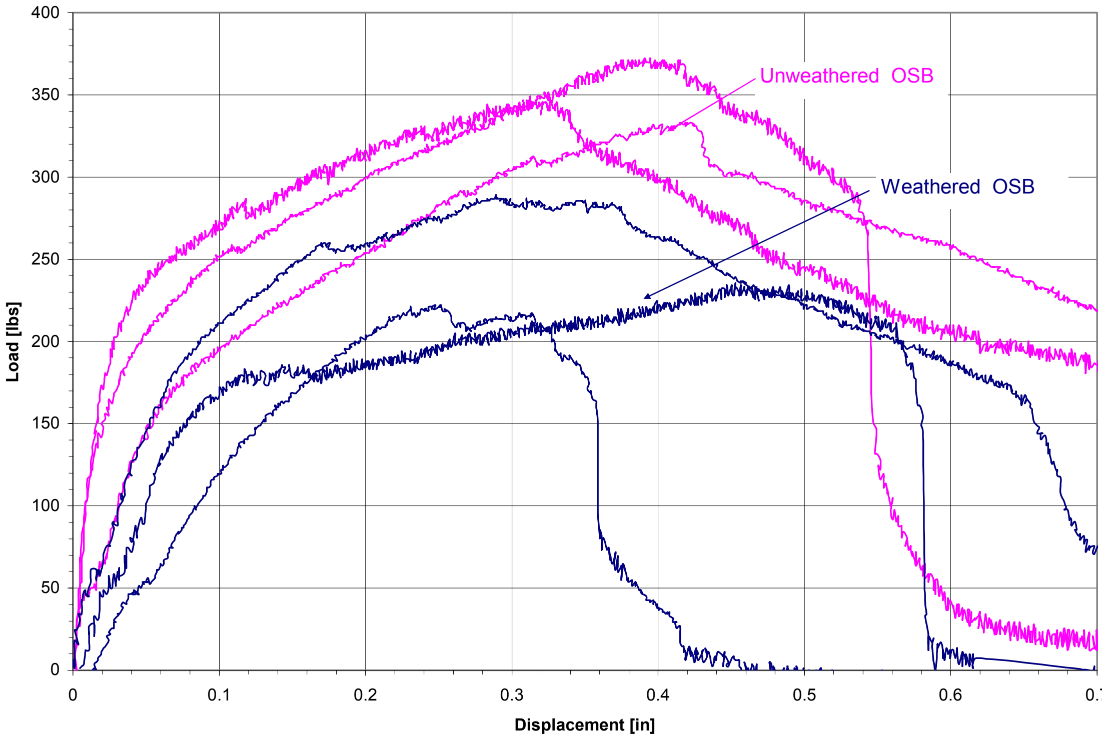

Figure 6: Rapid loading results for OSB specimen

Figures 5 and 6 show the results of the rapid loading tests. Again the variability of the OSB is seen, this could be attributed to the strands of board that overlay to make the OSB. The maximum load capacity for unweathered plywood appears at approximately 0.31 in. with a slightly lower load capacity seen in the weathered specimens, with OSB maximum load capacity is seen at an average displacement of 0.36 in. for unweathered and 0.17 in. for weathered specimens.

The plywood shows a load bearing capacity that is approximately equal for both the unweathered and weathered specimens; with the OSB there is a decrease of around 100 lbs due to weathering. The OSB showed a higher load-displacement capacity than the plywood; however this can be attributed to the differences in the sheathing thickness.

6.6. Summary and Conclusion

The results showed a difference in results for both the plywood and the OSB, in respect to weathering and loading rates. OSB was seen to have a great variability in results, which can be attributed to the strands of the OSB, while the plywood was more consistent. The plywood shows a load bearing capacity that is approximately equal for both the unweathered and weathered specimens; with the OSB however there is a decrease seen due to weathering. In both types of sheathing there was brittleness resulting from exposure to weathering. The faster loading rate caused a greater displacement before reaching the highest load capacity of the sheathings.

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7. REU Report - Experimental and Analytical Study of a Retrofitted Pin and Hanger Bridge

By Bridget Webb, ATLSS Undergraduate Researcher

Research Advisors: Robert Connor, Ian Hodgson, and Carl Bowman

7.1. Abstract

In July 1983, a two-girder, pin and hanger bridge collapsed over the Mianus River in Greenwich, CT when a pin and hanger system fractured [NTSB]. This collapse spurred studies of this bridge type including a PennDOT investigation of all fracture-critical bridges with pin and hanger connections.

The objective of this project was to study the behavior of one such bridge and calibrate a finite element model of the existing bridge to accurately estimate the behavior of the original pin and hanger design. The bridge studied was found to act continuously and compositely. The model for the various section, composite, continuous span bridge was found to accurately predict behavior of the bridge in its existing continuous state. As such, the model of the original various section, composite bridge with pin and hanger connection is assumed to accurately predict behavior of the original pin and hanger bridge.

7.2. Background

The bridge studied in this project was the Clarks Summit Bridge, a two-girder, riveted steel bridge as seen in Figure 1. The total length is 1626'-10" consisting of 8 main spans of 170'-3" and 2 end spans of 132'-5". It has a maximum clearance of 139'-5". The bridge is located near the northern end of the northeast extension of the Pennsylvania Turnpike spanning U.S. Rtes. 6 and 11, D.L. and W.R.R., and L.R. 953.

The Clarks Summit Bridge was originally designed in 1955 with 4 pin and hanger connections. Spans four and seven each contained two connections, one on each girder as shown in Figure 1. The bridge was made continuous in 1991 by replacing each pin and hanger connection with bolt-spliced continuity plates. This was a rare retrofit [Christie].

The bridge type in question is extremely non-redundant as neither the pin and hanger connections nor the two-girder superstructure provide an alternate load path in the event of first member fracture. Thus, the failure of one connection or girder would be expected to result in collapse of the bridge [Ghosn].

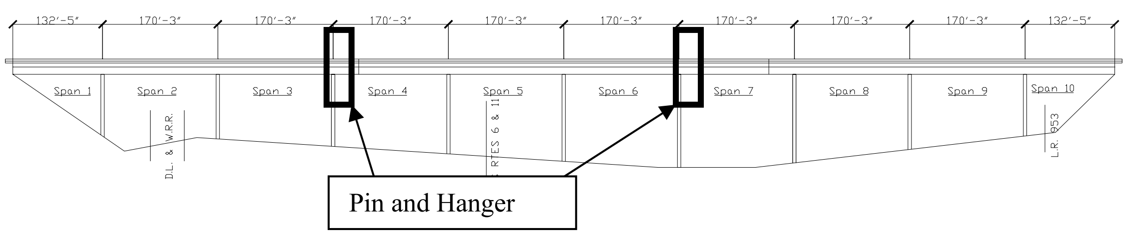

Figure 1. Elevation view of Clarks Summit Bridge

7.3. Finite Element Analysis

Finite element analysis was performed using SAP 2000. Various two-dimensional line-girder models were created for combinations of composite and non-composite sections; variable and constant section properties; and zero and full moment connections at the location of the pin and hanger. All models were loaded with a single one-kip moving load to simulate a truck driving across the bridge

7.4. Analysis of Variable Sections

The girders of the Clarks Summit Bridge contain various section moduli, due to the addition of coverplates along the length, as well as variations in the web plate thickness. Models for both variable and constant section moduli were created and compared for estimation purposes. The moment envelopes of the two models were identical in shape. However, a shift of 15% occurred. Thus, for a quick estimate, it is unnecessary to find the section moduli across the entire length.

7.4.1. Analysis of Composite Girder and Deck

The bridge was initially designed with a non-composite cast-in-place reinforced concrete deck. The bridge was later redecked using pre-cast deck panels assuming non-composite action in 1979. Models were created for both the composite and non-composite cases for each of the variable and constant cases. There was no difference between the composite and non-composite cases for the constant section model. The comparison of constant and variable section models showed that the moment envelope for the constant section model was shifted downward 10% from the moment envelope for the variable section model.

7.5. Analysis of Moment at Former Pin and Hanger Connection

The model simulating the original pin and hanger bridge design exhibited a zero-moment at each connection. All models simulating the retrofitted bridge design exhibited a full-moment reaction where the pin and hanger connections were formerly located.

7.6. Field Instrumentation

Using SAP 2000 and moment envelopes developed by analyzing the variable-section continuous bridge with full moment connections, the gage plan was created. Eighteen uniaxial strain gages were applied in nine locations. Each girder held three locations: the positive moment region, former pin and hanger joint, and negative moment region. The floor beam in the negative moment region held two locations: one at its center, the other near the west girder. The stringer in the negative moment region nearest the west girder held one position midway between floor beams.

7.7. Controlled Load Tests

The test truck was a 4 axle Mack Aerial Axle UB 50 snooper with the fourth axle riding up. The gross vehicle weight of the truck was 62,440 pounds. The truck was provided by the Pennsylvania Turnpike Commission and was labeled as Truck #14-002. No other traffic was permitted on the bridge during testing.

The tests performed included crawl, park and dynamic tests. The crawl and dynamic tests were duplicated to establish variability of the data. The four lanes were numbered with lane 1 over the west girder and increased to lane 4 over the east girder. The crawl tests were performed in each lane twice. The dynamic tests were performed in lane 1. The park tests were performed in lanes 1 and 2 over the instrumented floorbeam and a mid-span floorbeam with the tires of one side of the truck on the instrumented stringer.

7.8. Results of Field Analysis

7.8.1. Variability in Testing

Each crawl and dynamic test was duplicated to determine the level of variability in the test. A 10% maximum variance in the girder response was found in the tests. There was a maximum variance of 15% in the floorbeam for these tests. This is to be expected due to the greater sensitivity of the floorbeam to transverse position of the test truck.

7.8.2. Load Distribution in Girders

Two park tests were used to determine the load distribution between the girders. One test was performed with the truck in the outside southbound lane (lane 1). The other was performed with the test truck located in the inside southbound lane (lane 2). The values of highest normalized stress in the bottom flange at each longitudinal location were compared with the theoretical distribution factors. This comparison can be seen in Tables 1 and 2. The bottom flange gages were used due to their higher response to the test load.

Longitudinal Location	Distribution Factor (%)			
	East Girder		West Girder	
	Measured	Theoretical	Measured	Theoretical
Negative Moment	3	10	97	90
Joint	10	10	90	90
Positive Moment	17	10	83	90

Table 1: Bottom flange stress distribution with test truck in lane 1

Longitudinal Location	Distribution Factor (%)			
	East Girder		West Girder	
	Measured	Theoretical	Measured	Theoretical
Negative Moment	33	24	67	76
Joint	40	24	60	76
Positive Moment	38	24	62	76

Table 2: Bottom flange stress distribution with test truck in lane 2

The average stress distribution factor with the truck in lane 1 matches well with the theoretical stress distribution factor, as seen in Table 1. The average distribution factor with the truck in lane 2, with a value of 37%, does not correlate with the theoretical distribution factor, as seen in Table 2.

7.8.3. Effect of Transverse Position of Test Truck

Crawl tests were used to determine the general effect of transverse position of the test truck. The bottom flange stress range responses were compared for the east and west girders at the positive moment, joint, and negative moment locations. The bottom flange responses are presented due to higher response. Table 3 shows the girder response when the test truck was driven in various transverse positions over the bridge.

Gage	Stress Range (ksi)			
	Lane 1	Lane 2	Lane 3	Lane 4
EGBFP	0.7	1.4	2.2	2.5
WGBFP	2.5	2.1	1.4	0.9
EGBFJ	0.4	0.8	1.4	1.8
WGBFJ	1.7	1.3	0.8	0.4
EGBFN	0.3	0.7	1.3	1.5
WGBFN	1.4	1.2	0.7	0.4

Table 3: Bottom flange stress ranges measured along the girders with varying truck position

7.8.4. Composite Action

Initial review of the bridge showed the pre-cast deck panels attached to the girder with multiple spring clamps. Many spring clamps were observed to be broken in the last bay near the southern abutment. The lack of connection between the deck and girder suggests non-composite action between the deck and girder.

Data from the outer lane park test were used to estimate the actual degree of composite action. West girder data were used since it was located directly under the test truck. Varying values for neutral axes and level of composite action were found longitudinally along the bridge. The data reported in Table 4 are the locations of the neutral axis at the highest positive bottom flange west girder stress response. The data suggest that the girders are highly composite.

Longitudinal Girder Location	Calculated Neutral Axis (in)	Theoretical Neutral Axis (in)		Percent Composite (%)
		Composite	Non-composite	

Negative Moment	92 -100	93	67	103
Joint	117 - 122	93	67	128
Positive Moment	85-90	96	67	91

Table 4. Table of neutral axis values, measured from bottom of bottom flange

7.8.5. Analytical Comparison of Moment at former Pin and Hanger Location

The influence line created by SAP to model a various section, composite bridge at the positive moment section was compared with the influence line generated with field data from the west girder bottom flange gage at the same location. It was necessary to scale the model with four factors to obtain a basis for comparison. The scale factors converted moment to stress, from two-dimensional SAP model to one of the girders of the existing bridge, for load distribution, and from a one-kip point load to a 62.4 kip test truck. The results of the scaled influence line correlate very well to the obtained field results. Table 5 shows this correlation.

	Maximum Stress (ksi)	Minimum Stress (ksi)
Field Results	2.006	-0.489
Analytical Model	1.878	-0.572

Table 5: Results of influence line comparison

7.9. Conclusions

The Clarks Summit Bridge was found to act continuously with a full moment reaction at each of the former pin and hanger joint locations. It was also found to behave with a high level of composite action.

The average factor for stress distribution between girders matches well with the theoretical stress distribution factor when the truck is in lane 1, as seen in Table 1. However, when the truck is in lane 2, the average stress distribution factor, at a value of 37%, does not correlate with the theoretical distribution factor, as seen in Table 2. This is likely due to greater load distribution in the longitudinal direction.

Transverse truck position resulted in a mirrored effect in opposite lanes, as seen in Table 3. This is to be expected as the floorbeam should be able to distribute load regardless of the origin of the load.

A comparison between an influence line created analytically with SAP 2000 and an influence line obtained experimentally from the same longitudinal location showed that the pin and hanger model is an accurate representation of the behavior of the bridge before the retrofit of the pin and hanger connections, shown in Table 5.

Finally, the tests were shown to be highly repeatable. Thus, these tests can be taken as valid and the results considered accurate.

7.10. References

American Institute of Steel Construction, Inc. 1959 <u>Moments, Shears and Reactions: Continuous Highway Bridge Tables</u>. New York, NY.

Christie, Scott and John M. Kulicki. Feb. 1991 "New Support for Pin and hanger Bridges." <u>Civil Engineering-ASCE</u>. Vol. 61 no. 2. p.62-64.

Ghosn, Michel and Fred Moses. Aug.1995 "Redundancy in Highway Bridge Superstructures." <u>Fourth International Bridge Engineering Conference.</u> Vol. 2. San Francisco, CA. Transportation Research Board and National Research Council

National Transportation Safety Board. Jul. 1984 "Highway Accident Report – Collapse of a Suspended Span of Interstate Route 95 Highway Bridge over the Mianus River. Greenwich, CT. June 28 1983." Washington, D.C. United States Government.

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7.11. Acknowledgments

Pennsylvania Turnpike Commission, Robert J. Connor, Ian Hodgson, P.E., S.E. , Carl Bowman, Prof. Clay Naito, P.E., PITA, ATLSS

8. Appendix A - REU Advertisement Poster

ATLSS/PITA
REU SUMMER 2003

LEHIGH UNIVERSITY

Summer Research Experience for Undergraduates (REU) in Structures, Materials, and Construction

Program Description:
The Center for Advanced Technology for Large Structural Systems (ATLSS) at Lehigh University, with support from the Pennsylvania Infrastructure Technology Alliance (PITA) is offering summer fellowships for research at the ATLSS Center. The fellowship is for undergraduate students in their junior year or beginning their senior year of college in civil engineering with interests in the areas of structures, materials, or construction.

Program Outline:
Ten-week research experience with a stipend of $3700 (June 2 - August 8). Opportunity to work on an interesting engineering topic in the area of structures, materials, or construction. Work under the mentorship of a faculty member or research engineer and a graduate student. Participate in workshops on how to write technical reports and how to give effective presentations. Visit structural engineering companies and construction projects.

Research Areas:
Field Testing; Laboratory Testing; Structural Design; Structural Analysis; Finite Element Modeling; Innovative Materials; Blast Resistant Engineering; Earthquake Engineering; Sensor Technology, Bridge Systems; Building Systems

Requirements:
Cumulative GPA 3.3
U.S. citizen or permanent resident
Minorities and women strongly encouraged

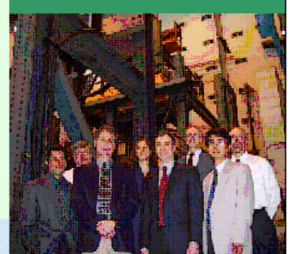
Application:
Applications can be found online at http://www.atlss.lehigh.edu/cjn3/PITAREU/reu.html

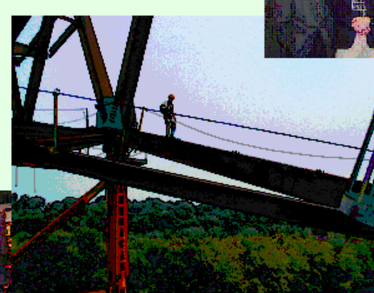
Application Deadline: March 7, 2003
Notification: April 2003

Contact Information:
ATLSS Center
117 ATLSS Drive, IMBT Labs
Bethlehem, PA 18015

Phone: 610-758-3525
Fax: 610-758-5553

Program Managers
Dr. Clay Naito
Dr. Robert Connor





ATLSS

Further information:
www.atlss.lehigh.edu/cjn3/PITAREU/reu.html

PITA Pennsylvania Infrastructure Technology Alliance

9. Appendix B – REU Application

Lehigh University
ATLSS Center
PITA REU PROGRAM
2003
Application
(June 2, 2003 - August 8, 2003)

The application package (which is also available on the PITA REU homepage: www.atlss.lehigh.edu/~cjn3/PITAREU/reu.html), consists of:

1. ☐ Personal information sheet
2. ☐ One page essay
3. ☐ Resume
4. ☐ Transcript
5. ☐ One letter of recommendations

Send the complete package to:

Assistant Professor Clay Naito
ATLSS/PITA REU Program
Department of Civil and Environmental Engineering
Lehigh University
117 ATLSS Drive
Bethlehem, PA 18015

The application package can also be emailed to cjn3@lehigh.edu as an attachment and type in the Subject header: "PITA03".

For additional information contact:

Mr. Robert Alpago, rpa2@lehigh.edu, or 610-758-6107
Dr. Robert Connor, rjc3@lehigh.edu
Or visit the PITA REU homepage: http://www.atlss.lehigh.edu/~cjn3/PITAREU/reu.html

Application Deadline: Friday, March 14, 2003

Application 1 PITA REU 2003

Lehigh University
PITA REU
Application Form
Summer 2003

Personal Information	
Last name:	First name:
Name Institution:	Email:
Department:	GPA (…/4.0):
Degree:	Current status (junior, senior…):
Major:	Recommender Name(s):
Campus address where you can be reached during the academic year: Campus Phone:	Permanent home address: Home Phone:
Post graduate plans: MS: ☐ Ph.D.: ☐ Work: ☐ Not sure yet: ☐	Research Interests (check all that apply): Field Testing ☐; Laboratory Testing ☐; Structural Design: ☐; Structural Analysis ☐; Finite Element Modeling ☐; Innovative Materials ☐; Blast Resistant Engineering ☐; Earthquake Engineering ☐; Sensor Technology ☐, Bridge Systems ☐; Building Systems ☐
☐ U.S. Citizen; ☐ Permanent Resident	Social Security Number: - -
Optional Information: ☐ Male ☐ Female Ethnic background (specify one or more): ☐ African American; ☐ Asian, ☐ Hispanic, Native American, ☐ White	List any honors, prizes or awards:
Package includes: ☐ Statement of purpose; ☐ Resume; ☐ Transcript; ☐ One or more letters of recommendation	

Mail complete application package to: Prof. C. Naito; PITA REU Program; Lehigh University; ATLSS Center; Lehigh University; 117 ATLSS Drive; Bethlehem, PA 18015-4729

Application 2 PITA REU 2003

Statement of Purpose

(Discuss your reasons for applying to the PITA REU program; mention any prior research experience and career goals.)

Enter your name:

Application 3 PITA REU 2003

PITA REU Program
Lehigh University

Recommendation Letter
Due: March 14, 2003

Name of Applicant:

This student has applied for the PITA REU 2003 program whose goal is to provide motivated and bright undergraduate students with hands-on research experience in the general area of structural engineering. Please comment on the student's intellectual qualifications, attitude towards research, and maturity.

A letter can substitute this form.

Admission to PITA REU Program is:

☐ Strongly recommended ☐ Recommended ☐ Recommended with reservations ☐ Not

Recommender's name: Position or title:	Signature Date:

You can give this form to the student in a signed envelope or you can mail it directly to:
Prof. C. Naito, PITA REU, Dept. of Civil and Environmental Engineering, 117 ATLSS Drive, Bethlehem, PA 18015-4729. If you prefer to email this letter, mail to: cjn3@lehigh.edu or fax to 610-758-5902

Application 4 PITA REU 2003