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GATEWAY PARK PROJECT: CEMENTITIOUS UNDERLAYMENT & INFORMATION EXCHANGE

A Major Qualifying Project:

Submitted to the Faculty of the

Worcester Polytechnic Institute

In Partial Fulfillment of the Requirements for the

Degree of Bachelor of Science

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ABSTRACT

Two aspects of the construction of the Gateway Park Project were studied: the cracking of the cementitious underlayment of the floors at 68 Prescott Street and information exchange between owners and construction managers. An investigation into the behavior of timber/concrete floors was conducted by testing a composite and non-composite system, as well as performing a cost analysis. Information exchange was evaluated through use of surveys and meeting checklists.

ACKNOWLEDGEMENTS

We would like to thank Worcester Polytechnic Institute and Worcester Business Development Corporation representatives as well as Consigli and Gilbane construction companies' project managers for completing the information exchange questionnaire. We would like to thank Mr. Steve Johnson, Mr. Nathan Adams, and Mr. Brian Hamilton of Consigli Construction for allowing us to attend their weekly meetings and visit the biotechnology center, and answering our questions both electronically and on paper. Furthermore, we would like to thank Al Abdella of Gilbane for giving us a tour of the garage and Mr. Neil Benner for allowing us to attend their weekly meetings. We would like to thank Bill Carkin of WBDC for giving us an overview of the entire project at Gateway Park and Mr. Blais of WBDC for answering our questions. We would like to especially thank lab manager Mr. Don Pelligrino, lab technician Mr. Dean Deigneault and Professor El Korchi for all of their help that enabled us to do all of our testings for the capstone design. We would like to also thank Ismail Aktas to help us set up our survey website and Professor Declan DePoar for his help for understanding the cracks. We would sincerely like to thank Emily A. Lescak for her critical eye. We would like to thank our advisors, Professor Leonard Albano and Professor Guillermo Salazar, for their continuous support, suggestions and patience through the duration of this project. Overall, we would like to thank everyone who has helped to make this project successful.

CAPSTONE DESIGN STATEMENT

The capstone design was satisfied through improving the serviceability of the floors of the original building by designing a flooring system that would be able to act as a composite structure. The floors of the original building are not acting as a composite structure and have experienced major cracking. The intention of this report is to understand the mechanics behind the behavior of the existing floors of the original building of the Gateway Park Complex and of a composite floor system that could have been used.

Investigative site visits and lab tests were conducted during the project to add a hands-on aspect to act in conjunction with the manual calculations to form a hypothesis. The initial inquiries included mapping cracks in the floors, as well as extensive literary research into the behavior of wood-concrete composite systems. Preliminary calculations were conducted to develop a hypothesis and testing procedures. The research culminated with a testing of a timber/concrete floor system that was intended to help understand the fundamental mechanics of a timber-concrete composite (and non-composite) floor system. From these tests, the feasibility of a timber/concrete composite as a solution to the cracking of the underlayment was assessed.

Four of the eight “realistic constraints” set fourth by the ASCE commentary are met through the completion of this project: economic, sustainability, manufacturability and health and safety. The treatment of each constraint is outlined below.

Economic

If the capstone design was to be used by the Consigli Construction Co., it would have a resulting impact on the economic forecasting of the Gateway Park Project. The additional costs come from the additional materials necessary to construct the system. One of the objectives of the project was to find a system of flooring that was effective and economical to the owners. This objective was achieved by comparing the different options for composite floor systems with their effectiveness. The overall intention of the project was to find a cost effective method of producing a timber-concrete composite floor system.

Sustainability

There are significant advantages to using a composite floor system due to the environmental benefits of wood. Wood is the only primary building material that comes from a renewable source and also requires the lowest amount of energy to manufacture and use (Clouston, 2005). Also, there is the benefit of preserving an existing building instead of building a new one. Restoring a building preserves building materials and minimizes waste produced on the job site.

Manufacturability

The design of the project must be able to be reproduced easily on-site. A necessary aspect of the project was to find a way for the wood and concrete to act composite while using the least amount of labor and materials. The original solution was more labor effective due to the utilization of self-leveling cement. Self-leveling cement requires no additional labor once it is pumped to the desired floor; there was no way to improve on this. However, there are multiple options that can be chosen from to form a composite floor. The intention was to find a solution that balanced the cost and effectiveness of the connector. The criteria that were considered when determining the method of construction included labor intensity, cost of materials, cost of equipment and application effectiveness.

Health and Safety

At the time of completion of the report, upper floors at 68 Prescott Street of the Gateway Park Project were noticeably “bouncy” as a result of foot travel within the building. The deflections of the floors could make a person feel insecure about the quality of the building and uneasy about occupying the structure. The composite flooring system performance could act to increase the serviceability of the structure by stiffening the floors and reducing the bounce.

AUTHORSHIP PAGE

This report has been contributed to equally by both members involved. Equal contributions were given to the research for the timber-concrete design. The majority of the timber-concrete design write-up was completed by Peter Bellino and edited by Mehmet Hergunsel. The majority of the construction project management section was researched by Mehmet Hergunsel and edited by Peter Bellino. Results and conclusions for both capstone design and construction project management were completed equally.

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1 INTRODUCTION

Gateway Park is a biotechnology research complex intended to stimulate the growth of biology and life science research in the Worcester Area. The project is a joint-owner multi-structure venture between the Worcester Businesses Development Corporation (WBDC) and Worcester Polytechnic Institute (WPI). Two of the buildings being constructed (at the time of the project) were the biotechnology center (BTC) at 60 & 68 Prescott Street and the parking garage. The other buildings, still in the design phase, included apartments for graduate students and additional research facilities.

This report studied two unique aspects of the Gateway Park Project. The first aspect investigated the floors of the BTC at 68 Prescott Street and the cracks in the cementitious underlayment. The second part of the report looked into the information exchange between the owners and the construction management firms for both the BTC and the garage. As a result, an alternative solution or suggestion was developed for each part.

The BTC consists of two buildings connected by a glass walkway. The glass walkway connects a newly constructed four-story building to a pre-existing masonry factory building which dates back to Worcester's heyday as a leader in the industrial processes. The flooring system of the original building of the Gateway Park biotechnology building is unique in that it consists of a cementitious underlayment resting on a pre-existing timber floor.

A common practice in Europe (Clouston, 2006) when renovating an older building is to place concrete on top of the existing floors to allow for a durable and even surface for other flooring options, such as rug and tile, to be placed. At 68 Prescott Street, cracking of the cementitious underlayment occurred to a more severe extent than anticipated. This has caused problems due to the desire to have the cracks patched. These patches yielded success, only after a lengthy (and costly) trial and error process.

One objective of this report was to improve the serviceability of the floors at 68 Prescott Street by providing an alternative design consisting of a timber/concrete composite (TCC) floor. An increase in serviceability would make a building that feels better to the occupants by reducing the deflections of the floors. The objective was accomplished through an investigation into the mechanics of the current non-composite floor system and a composite floor system tested in the laboratory.

In addition to the facilities at 60 & 68 Prescott Street, a six-story, 500-car parking garage was erected on site to provide parking for the residents of the park. This was unique in that the construction management firm was different from the one contracted to build and fit-out the BTC. Both construction management firms were on site working towards a similar completion date (BTC – April '07; Garage – August '07). An investigation into how the owners, Worcester Polytechnic Institute (WPI) and Worcester Businesses Development Cooperation (WBDC), exchange information and coordinate with the two construction management firms was done by distributing questionnaires and attending weekly meetings.

The report is broken up into four main sections. Section 2: BACKGROUND presents information regarding the history and the major participants in the Gateway Park project, meeting preparation, why and how the project was being developed, and the overall issues with the floors and research into TCC mechanics.

Section 3 discusses the means by which data was collected for the capstone design and the construction project management (CPM) portions of the report. The timber-concrete design section, 3.1, describes the initial hypothesis, design solution approach and testing procedures used as well as the approach used for analyzing the data. The meeting section presents the "meeting checklist," and questionnaire used to gather data about the weekly meetings and communication process.

Section 4: RESULTS presents the data that was gathered. The timber-concrete design proposed solution in addition to a cost and a schedule analysis of the solution. Meeting section provided analysis of the "meeting checklist", the meeting checklist and the questionnaire.

Section 5: Conclusions, uses the results section data to recommend what measures could have been taken to prevent the problems with the floors in 68 Prescott Street. In addition to this the CPM section recommends how the management could be coordinated to facilitate more efficient project meetings and relay of information.

2 BACKGROUND

The Gateway Park site has had a long history, starting with the industrialization of New England and is currently being transformed into a biotechnology center.

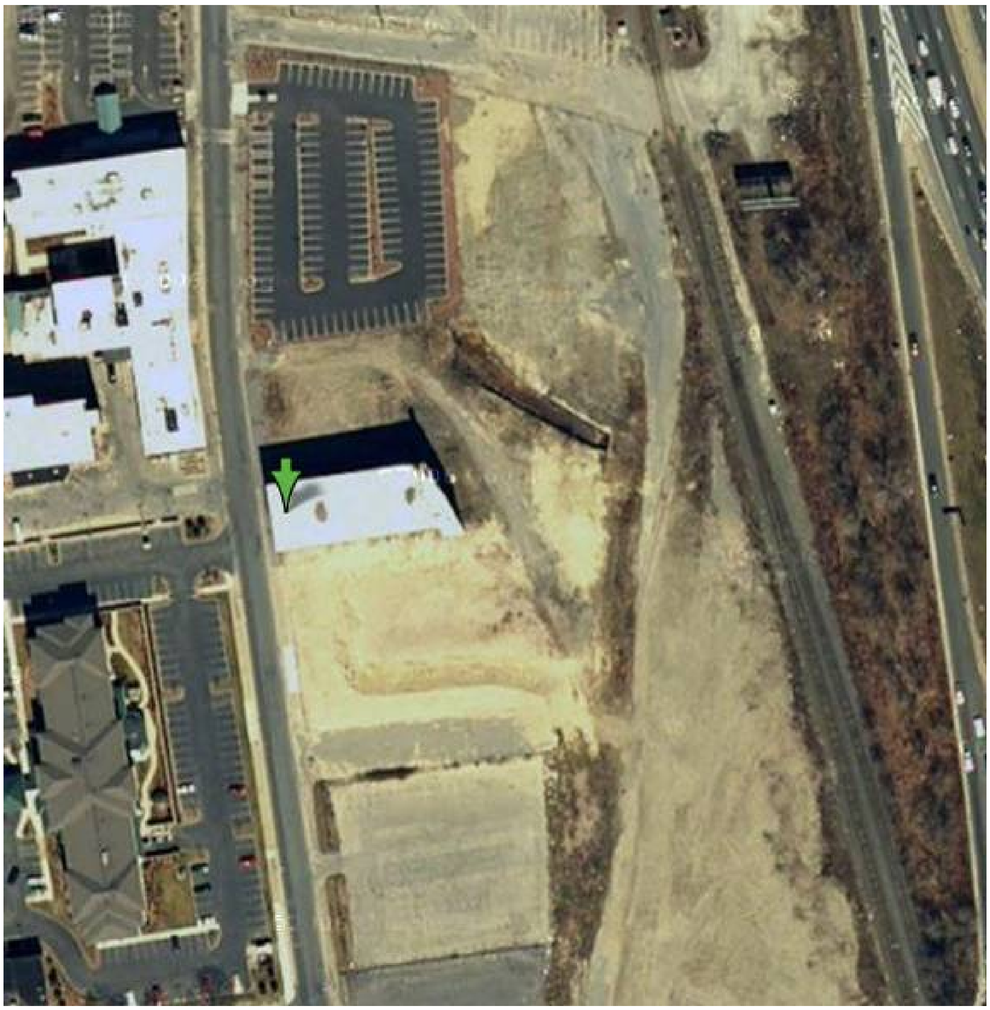
2.1 GATEWAY PARK HISTORY

Through the 1960's Worcester was a heavily industrialized city and developed slowly into a very prosperous business district. The population reached 175,000 people; however, with the decline of the industry and the factories in this country, the job market in Worcester has declined tremendously.

In an effort to revitalize the economy of Worcester, the Gateway Park project emphasizes investment in the business districts along with the expansion of the WPI Campus. Part of the Revitalization of Worcester Program (Armand 2006), is restoring one of the few remaining factory buildings while establishing new job opportunities in bioengineering research areas. The former Mayor of Worcester Timothy Murray underscored the prominence of this project indicating that: "Certainly, through this initiative, WPI reasserted itself to the role it has historically played as an innovator and incubator of jobs to sustain the community. Gateway Park will be the foundation to sustain Worcester for the next 100 years." (Killough-Miller, Joan 2004)

Due to contaminations and environmental hazards at the old factory site of Gateway Park, the WBDC demolished the majority of the original factory buildings. The redevelopment plan, known as City Square, which included Gateway Park, was established to increase new jobs in Worcester from approximately 1675 to 3100.

Additionally, 650 apartments will be constructed in order to meet the needs of the county's real estate market. The housing will be constructed across the road from the BTC. This investment in return will contribute to the revitalization of the Worcester's business district. Picture 1 below shows a birds-eye view of 68 Prescott St. and the surrounding brownfield sites as it was before construction started.



Picture 1: 68 Prescott Street and surrounding brownfields (Google Maps, c.1990)

Gateway Park is part of the fifty-five acre Gateway Redevelopment District. It is a crucial project for the Gateway Redevelopment and will provide at least 300 of the City Square Project jobs to the city of Worcester. Gateway Park is set on an eleven acre brownfield and will “feature between 500,000 and one million square feet of mixed-use space to attract academic and corporate collaborators, and make a significant impact on the economic development of the region” (Worcester Polytechnic Institute (WPI) and Massachusetts Biomedical Initiatives (MBI) to Announce Incubator Facility).

The purpose of the Gateway Park Project were to clean up and redevelop brownfields, establish WPI Technology Transfer, attract bioengineering and biotechnology companies to Worcester, create jobs, expand Worcester’s tax base, and develop a mixed use community which would include housing, offices and retail outlets.

2.2 MAJOR PARTICIPANTS OF GATEWAY PARK LLC.

There are four major participants involved in the Gateway Park Project. WBDC and WPI are dual owners of the BTC and the parking garage. WBDC financed the exterior shell of the biotechnology building while WPI financed the interior fit outs. The organizational breakdown structure is shown in Figure 1 with WPI and WBDC acting as co-owners being responsible for the project.

The two construction management firms involved with the project were Consigli Construction Company and Gilbane Building Company. Consigli was responsible for the completion of the BTC and Gilbane was responsible for the completion of the parking garage.

Consigli Construction Co. was required to keep track of two separate accounts simultaneously during construction because of the tax status of an academic institution (WPI) and a corporation (WBDC).

Picture 2: Overview of Gateway Park

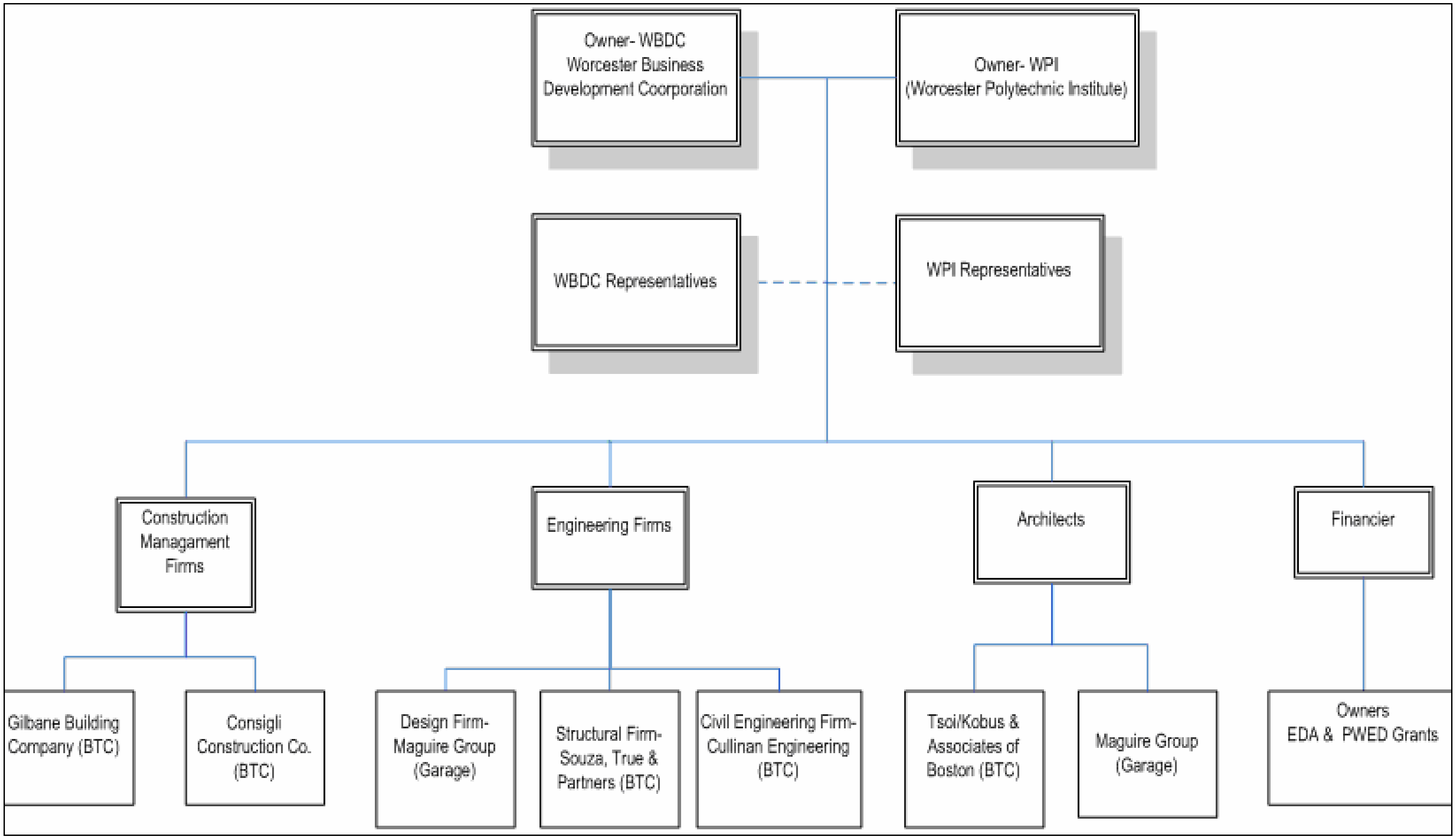


Figure 1: OBS for Gateway Park

2.2.1 Consigli Construction Co.

Consigli Construction Company is a fourth-generation construction firm based out of Milford Massachusetts. Consigli provides construction management, design/build, general contracting and pre-construction planning services for clients. They are ranked among the top 400 construction firms by *Engineering News Record*, grossing more than $125 million annually. They have extensive experience in academic, corporate, health science, and institutional buildings, with many projects for area universities and municipalities (www.Consigli.com).

For the BTC, Consigli's construction management team was comprised of: Project Manager Steve Johnson, Project Manager Brian Hamilton, Supervisor Michael Codianne, Project Executive Michael Walker, Project Manager (for MBI fit out) Sen Lin and Project Engineer Nathan Adams.

Consigli was the general contractor in addition to being the construction manager for the project. Some of their in-house activities included: installation of doors, frames and hardware (supplied by TCI); installation of the kitchen casework and millwork and anything else unrelated to labs (supplied by PadCo); expansion joint installations (supplied by Metro Sales); and masonry restoration. Consigli sub-contracted the remainder of the activities to approximately fifty subcontractors. All major subcontracted activities and subcontractors for the BTC are listed in Table 1.

Table 1: BTC Subcontractors

Activity	Subcontractor	Project Manager
HVAC & Plumbing	William F. Lynch	Don Martneau
Electrical	Ostrow Electrical	Stephanie O'brien
Electrical for MBI	Del Signore	Louis Del Signore
Drywall, Framing, ceilings and fireproofing	H. Carr & Son	Bolo Connoni
Structural Steel	Novel Iron	Sean Wingston
Masonry	Prunier	Steve Prunier
Curtain walls and windows	Modern Glass and Aluminum	Jeff Johnson
Lab Casework	Gibson Associates	Ken Yeager
Sprinkler	Cannistraro	Michael Cray
Elevators	Shindler	Jeff Sherman
Underlayment	Northwest	Jim Dietrich
Flooring	Kesseli Moirse	Jeff Peglioni
Roofing	Greenwood Industries	John D'Etia
Waterproofing/caulking	Superior Waterproofing	James Shaw
Miscellaneous Metals	Soucy Industries	Ryan Ayotte

The project consists of 87,200 sq. ft. of new construction (60 Prescott Street) and the 35,000 sq. ft. renovation of the adjoining building (68 Prescott Street). Picture 3 shows the architectural rendering of the completed facility.

Picture 3: 60 & 68 Prescott St.

WBDC awarded the BTC construction contract to Consigli Construction Co. through competitive bidding. Worcester Polytechnic Institute then hired Consigli to fit out the building as a center for biology and life science graduate research. Consigli agreed to a guaranteed maximum price bid of $14 million for the exterior shell core and

$13 million for the interior fit out. These prices were eventually revised as change orders came in from WPI and WBDC which included $1 million of second floor fit-out. The total price tag for the construction of the building came to $31 million dollars. The final punchlist was started on March 1st 2007 for the hand over date of April 2nd 2007. The only exception is the second floor fit out that was completed for occupancy for WPI by the end of May.

2.2.2 Gilbane Building Co.

Gilbane is one of the country's largest family-owned construction firms with 25 offices across America. The company operates solely as a project management firm and brings in approximately $2.5 billion dollars a year in annual revenues. Gilbane has extensive experience in academic buildings and recently had been contracted by WPI to complete their admissions building. Gilbane was selected to provide construction management for the six-story 500-car parking garage.

Gilbane's construction management team was comprised of Project Manager (in office) Neil Benner, Project Manager (on site) Al Abdullah, Project Executive Bill Kearny, and Accountant/Project Engineer Travis Savoie. The main subcontractor groups are listed in Table 2:

Table 2: Parking Garage Subcontractors

Activity	Subcontractor	Project Manager
Plumbing/Mechanical	William F. Lynch	Glenn Knott
Electrical	Coughlin	Jim Chapdelaine
Sitework	Marois Brothers	Joe May
Miscellaneous Metals	Roman Ironworks	Michael Abruzzese
Concrete	Francis Harvey & Sons	Dan Baird
Prestressed Concrete	Blakeslee Prestress	James Coyle
Curtain Wall	Ipswich Bay Glass	Charley Moniz

With the mild winter of 2006, the erection of the pre-cast concrete structure was completed one month ahead of schedule. The original completion date for winterizing the garage was January 25, 2007, however this critical path activity was finished on the fifteenth of December 2006. The final completion date of the project was set to be August 6, 2007. The project is anticipated to be completed earlier on June 1, 2007 with a cost of $10 million.

2.2.3 Worcester Polytechnic Institute

Established in 1865, Worcester Polytechnic Institute is founded on the bases of technology and science and continues to propel forward with this mission. Gateway Park Project is a major milestone for WPI. It will allow the university to be at the forefront of innovation and advancement in graduate biological and life science research programs. Dr. Dennis Berkey, the president of WPI, made the following statement as to reflect the importance of the revitalization of the Gateway Park:

> "Life science is the new economy for Massachusetts, and WPI—through our core strengths in engineering and science—can make great contributions to the economic development of our region and to improvements in the quality of life. We are thus investing in this area by building the new WPI Life Sciences and Bioengineering Center at Gateway Park, by adding relevant faculty, by deepening our collaborations with the University of Massachusetts Medical School in research and graduate education, and by strengthening our capabilities for technology transfer and commercialization" (http://www.wpi.edu/News/Transformations/2006Spring/creatingsynergy.html).

2.2.4 Worcester Business Development Corporation

The Worcester Business Development Corporation's (WBDC) intentions for investing in the Gateway Park Complex are to bring Worcester to the head of the biomedical research fields. Gateway Park LLC is considered another step in the renovation of downtown Worcester, and the 11-acre project should be able to provide 1675 to 3100 jobs for the area, 300 of which are in biological research. WBDC will control approximately twenty-five percent of the interior of the building, which they will lease to firms. Craig Blais oversees the entire project. Bill Carkin oversees grading and overall coordination. Mike Lavana is the owner's representative for parking garage. Brent Arthaud is the owner's representative for 60 & 68 Prescott Street (WPI News & Events).

2.2.5 Architect & Engineering Firms

The architect for the Gateway Park biology and life science building was Tsoi/Kobus & Associates of Boston. The structural engineering firm was Souza, True & Partners, Inc. The Civil Engineering firm was Cullinan Engineering. The M/E/P/FP firm was vanZelm Hayward & Shadford. The Landscaping firm was Crosby Schlessinger Smallridge LLC. Maguire Group, designer and the engineer for the roadwork projects for the Gateway Park. Led by project manager Harold Morsilli, the Maguire Group designed road, master grading, and parking grading.

2.3 FLOORS

The floors of the existing building of the Gateway Park Complex consisted of multiple layers of wood, metal and concrete. The system is shown in Figure 2 - Figure 4. The only parts added by Consigli were the 1.75 lb galvanized metal lath, Portland based concrete underlayment and the grout and sealant. The remaining wood floor was all previously installed.

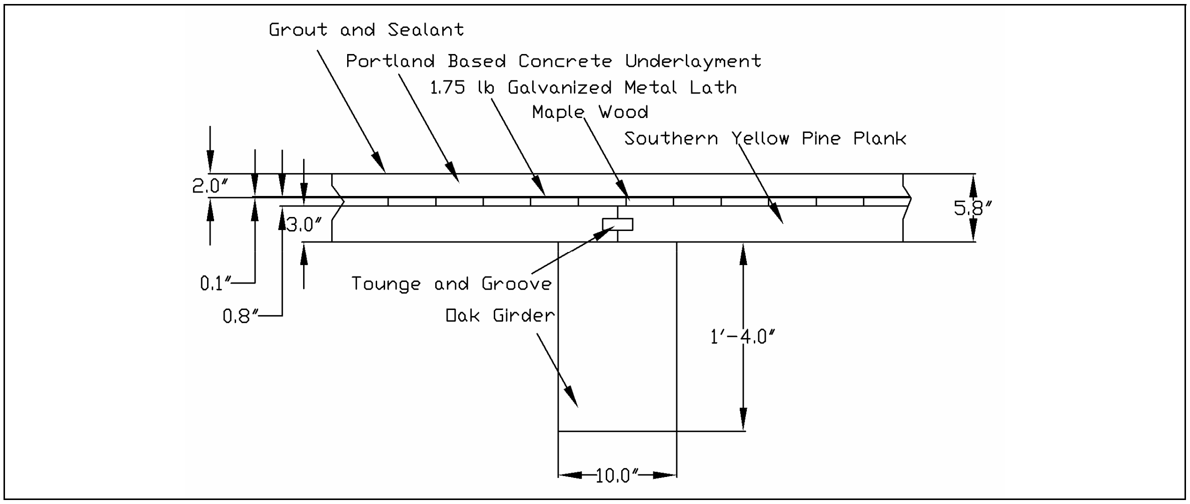


Figure 2: Girder Cross-Section

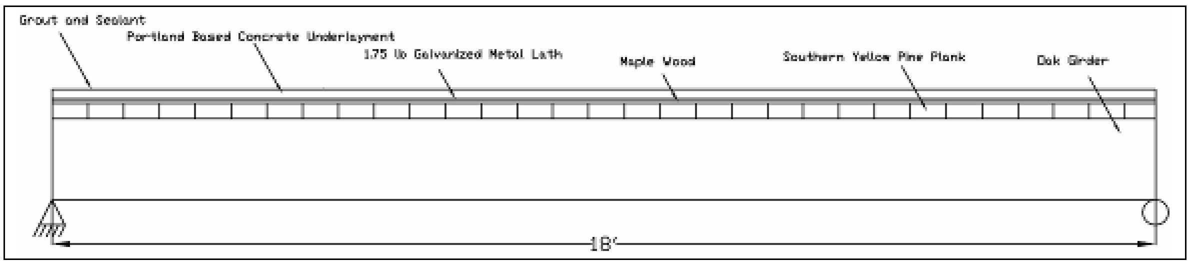


Figure 3: Girder

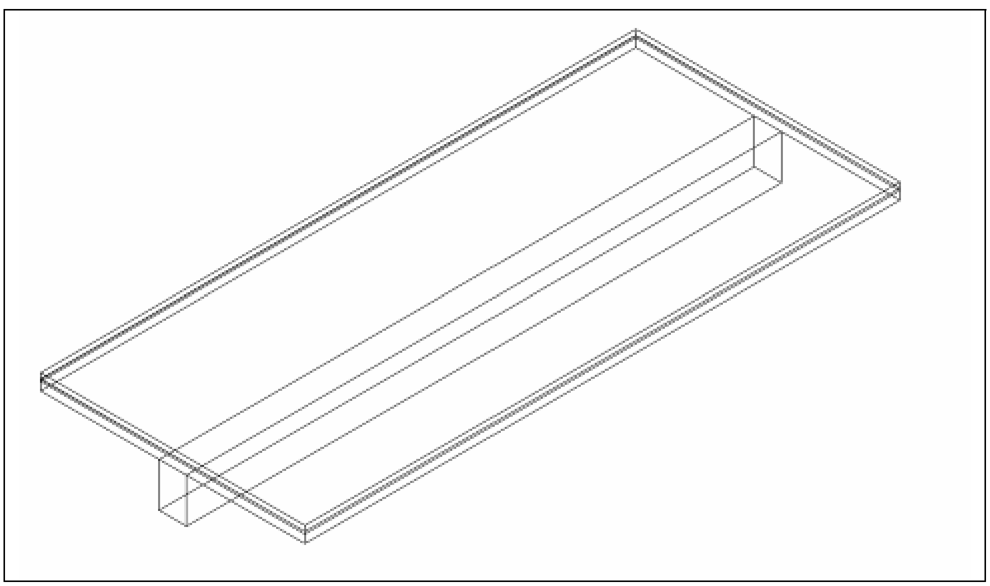


Figure 4: 3-D Slab 8' x 18'

The girders are made of ten by sixteen inch oak timbers, as seen in Picture 4, resting on top of oak columns.

Picture 4: Oak Girder & Southern Yellow Pine Planks

Picture 4 also shows the underside of the 3”x7” full dimensional southern yellow pine planks that are connected to the top of the girders by large nails. These planks are connected together by a tongue and groove joint which are spaced every eight or sixteen feet.

On top of these planks are 3" x $^{3}/_{4}$" full dimension maple planks connected to the southern yellow pine planks by nails on each end laid onto of the southern yellow pine later in the factories life. The maple planks are spaced perpendicular to the southern yellow pine plank and are either nine feet or eighteen feet long. All the wood is original and was not placed by Consigli. Most of this wood is over one hundred years old and has experienced some initial sagging due to previous dead and live loads.

The renovation process for the floors started by removing as much of the existing oil stains as possible and coating the floors with a water-proofing sealer. After the sealant was dry, 1’x4’ sheets of 1.75 lb galvanized metal lath were stapled across the entire floor as seen in Picture 5.


Picture 5: Metal Lath

Next the concrete was placed starting on February 23rd, 2006 with half of the fourth floor. The concrete for the rest of the floors was placed on February 27th, 2006 and March 9th, 2006. The concrete was a self-leveling Portland based cement underlayment. The particular type of concrete required no extra curing procedures after it was poured due to its self-leveling and curing nature. Reference Picture 6 for application of concrete.


Picture 6: Application of Concrete

2.3.1 Serviceability

Serviceability refers to the usefulness of an object. For structures, it refers to the level of usefulness a building possesses to its occupants. It does not refer to the structural integrity of building but more the practical application it provides to its occupants. If a building is considered to have a high degree of serviceability, it not only stands up to a high degree of structural integrity, but is considered exceptional by its occupants. In contrast, a building with a low degree of serviceability is one that meets a high degree of structural integrity but has one or more flaws that causes it to be an undesirable place to occupy. An example of this is a skyscraper that sways in the wind to an extent that causes its occupants to be sick. Therefore, buildings need to be designed to withstand not only the given loads but an allowance for occupancy comfort.

2.3.2 Concrete Cracking

Cracking of concrete is viewed as unsightly and unwanted in a new building. Cracking of new floors gives the impression of shoddy workmanship and causes feelings of ill-content on behalf of the owners and contractors in addition to lowering the serviceability of a building by creating hollow sounding floors, buildings that sway excessively or cracking in plaster and floors. It is necessary avoid or reduce circumstances where concrete may crack and to control the cracking that does occur.

Tensile stresses, shears and bending moments can cause different types of structural related cracking: direct tension cracks, bending with or without axial load cracks, torsion and shear cracking, bond cracks, and bearing and compression cracks due to a concentrated load. Cracks can also occur due to temperature change, shrinkage, and settlement of material. (Macgregor 393-94). There are a few main crack types which were considered for the report.

- **Tension Compression**

Exposing concrete to load stresses in either tension or compression before it has gained sufficient strength will cause cracks that reduce the structure's ability to handle its required load. (Ropke 70)

- **Shifting form cracks**

Solid form construction is necessary for the prevention or minimization of cracks. The form does not need to give way completely to cause a cracking problem. Any shifting due to timber expansion or the loosening of a nail or clamps can cause cracking. These cracks have no particular pattern. (Ropke 71)

Picture 7: Tension Crack on Fourth Floor

- **Thermal**

A temperature gradient within concrete of 35º F within 1 ft is usually considered sufficient to cause cracking. However, within 24-hours of placement, the internal temperature of the concrete can reach anywhere from 20º to 50º F hotter than ambient temperatures resulting in a temperature gradient. This gradient will cause tension within the concrete due to contraction of the slab. This tension then cracks the newly hardened concrete. (Concrete Network)

- **Plastic Shrinkage**

Plastic shrinkages are caused by evaporation of water from the surface and can be fixed by applying water to the surface. Typically this type of crack is wider than shrinkage cracks and can penetrate through the depth of the concrete. Plastic cracks will most often occur on dry windy days, parallel to one another and perpendicular to the wind. (Ropke 71)

- **Drying/Shrinkage**

Drying/shrinkage develops about the time the water sheen disappears from the surface of the concrete. Usually these cracks are random, yet straight hairline cracks that extend to the perimeter of the slab (Ropke 71).

Picture 8: Second Floor Shrinkage

Table 3: Crack Characteristics

Crack Types	Characteristics	Observed cracking
Tension Compression	Cracking along high points or low points	Crowning over Girders and at low point in Floors
Shifting Form	Caused by movement of the slab; could run diagonal to walls	none observed
Thermal	cracking spaced evenly across a slab due to temperature gradient	evenly spaced cracking along second floor due to thin slab
Plastic Shrinkage	Often Occur outdoors due to dry windy conditions; wider than shrink cracks	none observed
Drying Shrinkage	Occurs when concrete is under-wetted; cracks appear to be random	Second Floor cracking Patterns
Desiccation Cracking	Cracking caused by an excessive load, charecterized by circular patterns	Observed on Second Floor

2.3.3 Wood/Concrete Flooring Systems

A composite structure is a combination of two or more materials that maintain their original properties while acting as one (Hibbeler, 2005). According to the materials' behavior for withstanding loads, composite structures can be formed to optimize the load capacity and increase stiffness, which in return can reduce deflections, and vibrations. Concrete-steel composite beam-and-slab systems are quite common in the USA. Concrete takes the compression forces and steel takes the tension forces which maximizes the serviceability of the floor.

The mechanics of a composite structure involves a transformation factor which is an analytical device for creating an equivalent cross-section of one material out of another. To solve for fully composite structures (concrete and steel) the forces in steel and concrete must be equated so they produce the same moment around the axis. Transformation factor used for this purpose ($n = \frac{E_2}{E_1} =$ transformation factor) indicates that a material with a width b on the original beam, must be decreased in width to $b_1 = nb$ when $E_1 \rangle E_2$. That means less of the stiffer material is needed to support a given moment, n acts as a multiplier (Hibbeler 325-6).

Composite concrete and wood structures are generally used in Europe, however, they are still considered novel in the USA (Clouston, 2006). With the recent interest in the renovation of old mill buildings, concrete and wood composite structures are becoming more and more common. UMASS Amherst is one of the few schools in the USA leading the research that's dealing with the design and performance of composite concrete and wood structures. It has been understood through their studies and European experiences that concrete and wood systems do not act fully composite. However, it is the intention of the members of the civil engineering community to improve composite performance (Clouston, 2006).

There are four common types of shear connectors for wood-concrete composite structures: dowel, sheet metal, concrete shear key with anchor and glued-in-metal plate. Dowel and glued-in-metal connectors are easier to install than sheet metal and concrete shear key with anchors since they only would need drilling and/or gluing. Sheet metals and concrete shear key with anchor is harder to install due to incising of the wood. Wood and concrete with glued-in metal plate is the most composite structure of all, however there is still researches done on the durability of the glue. The least composite structure is the one with the dowel, example *a*. Currently, the ASD method is used for the design calculations of the timber composite structures. LFRD method cannot be used at this time. (Clouston, 2006).

Figure 5 depicts the four different shear connector systems. The two connectors chosen for further study in this report were the dowel and the glued-in metal plate. The dowel connector consists of a metal rod (nail or screw typically) penetrated to a depth, *d,* and allows for a discontinuous transfer of shear between the two materials. The glued-in metal plate consists of a thin metal member adhered to a notch cut into the beam and allows for a continuous transfer of shear.

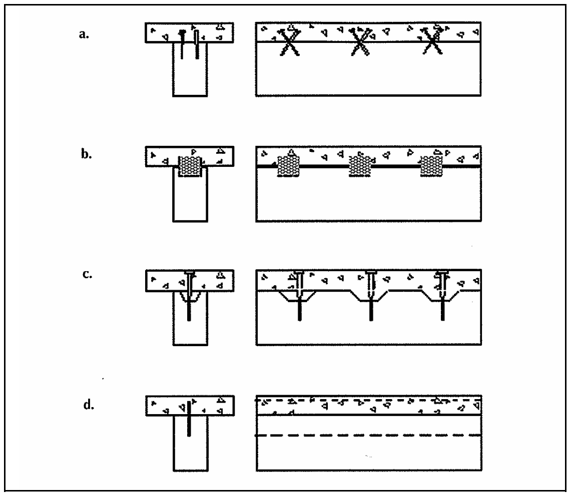


Figure 5: Types of Composite Floor Connections
a) Dowel b) Sheet Metal c) Concrete Shear Key w/ Anchor d) Glued-in Metal Plates (Clouston, 2006)

2.4 MEETINGS

Meetings are a neccesary tool to facilitate information exchange between parties of a project. Meetings allow for a time when the members of a project are able to ask questions directly and not need to wait for e-mail responses or phone-calls. It is very important to have effective meetings in order to facilitate a smooth flow of information and allow the goals of the project to be meet as easily as possible.

2.4.1 Effective Meetings

Effective meetings can be accomplished through adequate preparation, proper facility layout, efficient recording of data, methods of discussion, critical evaluation of the meeting, and eventually a follow-up on the discussed subjects. Effective meeting criteria can be broken down into three subsections: preparation before the meeting, readiness during the meeting, and follow-up after the meeting (http://www.casanet.org/program-management/your-board/meeting.htm).

2.4.1.1 Before the Meeting

The purpose of the meeting shall be clearly determined in advance targeting of all the participants. Information before the meetings must be shared between the staff members to prevent miscommunication. The most appropriate format should be picked (Reference for a list of meeting formats: Table 4). If appropriate, meeting minutes for the previous meeting should be sent out to the participants before the upcoming meeting. A pre-meeting agenda can also be sent out to enhance the inputs of the participants. Meeting site and time frame if not known already, should be established and a confirmation should be sent. If there are any problems from the previous week, they should be resolved before the upcoming meeting. If there are potential problems that could arise during the meeting, a plan should be formulated to address that issue.

Table 4An agenda along with the informational packages to be distributed during the meeting shall be prepared to meet the objectives of the meeting. Furthermore, the meeting facility shall be accessible and should assure that the participants feel comfortable. Meeting logistics including room, materials, refreshments and equipment, should be arranged before the meeting. For instance, the facility layout should accommodate the format, purpose, participants of the meeting. To have a town-hall style meeting in a boardroom would not facilitate optimum communication and transfer of ideas between the necessary parties. Also, it would be ideal for the meeting coordinator to review and rehearse the meeting before the meeting (Public Meeting Preparation and Management).

If appropriate, meeting minutes for the previous meeting should be sent out to the participants before the upcoming meeting. A pre-meeting agenda can also be sent out to enhance the inputs of the participants. Meeting site and time frame if not known already, should be established and a confirmation should be sent. If there are any problems from the previous week, they should be resolved before the upcoming meeting. If there are potential problems that could arise during the meeting, a plan should be formulated to address that issue.

Table 4: Meeting Matrix

Meeting Format Matrix			
Purpose	Participants	Format	Location
Introduce a project	Individuals	**One-on-One Meeting**	Community Centers
Provide project update	Elected/appointed officials	General to specific agenda	Churches
Resolve conflict		Space for exhibits	Project Office/Trailer
Build consensus		Neutral location	
Improve community relations			
Identify project issues			
Evaluate project alternatives			
Develop alternative solutions			
Introduce a project	Special interest groups	**Small Group Meeting**	Community Centers
Provide project update	Agency representatives	General agenda	Libraries
Improve community relations	Elected/appointed officials	Space for exhibits	Schools
Identify project issues	General public	Facilitator	Churches
			Project Office/Trailer
Introduce a project	General public	**Open House/ Transportation Fair**	Shopping malls
Provide project update	Special interest groups	No agenda	County fairs
Improve community relations	Agency representatives	Large open space	Neighborhood events
Identify project issues	Elected/appointed officials	Greeting/comment table	School fairs
		Exhibits	Church socials
Resolve conflict	Special interest groups	**Working Session**	Community Centers
Build consensus	Agency representatives	Specific agenda	Libraries
Evaluate project alternatives	Elected/appointed officials	Seating around a table	Schools
Develop alternative solutions		Space for exhibits	Project Office/Trailer
		Facilitator	
Brainstorm project ideas	Special interest groups	**Charrette**	Community Centers
Develop alternative solutions	Agency representatives	Specific agenda	Schools
	Elected/appointed officials	Layout table space	
		Facilitator	
Resolve conflict	General public	**Open Meeting**	Community Centers
Build consensus	Special interest groups	Specific agenda	Libraries
Evaluate project alternatives	Agency representatives	Break-out areas	Schools
Develop alternative solutions	Elected/appointed officials	Greeting/comment table	Churches
		Space for exhibits	
		Facilitator	
Present prefered	General public	**Public Hearing**	Community Centers
program/plan/project alternative(s)	Special interest groups	Formal agenda	Schools
Satisfy legal madates	Agency representatives	Formal seating	City hall
for public involvement	Elected/appointed officials	Greeting/comment table	Commissionc chambers
	Individuals	Microphone	
		Space for displays	

The purpose, participants, format, location, and timing of the meeting needs to be addressed before proceeding with the meeting. The five basic questions (Why? Who? What? Where? When?) of a meeting need to be answered to set up a meeting properly. The Meeting Format Matrix should be helpful to answer these questions (Public Meeting Preparation and Management).

2.4.1.2 During the Meeting

Meetings should start on time. It should follow the agenda or the meeting minutes. Side conversations should be kept to a minimum. All reference documents shall be presented either electronically or on paper. Direction of the meeting should stray as little as possible from the meeting minutes. This would enable the meeting to be run the meeting efficiently and keep everyone on track.

When group conversation strays from the meeting topic, it is the duty of the meeting coordinator to politely direct the meeting back on track. The main goal of the meeting should be to provide the necessary information in the shortest period of time. To accomplish this efficiently, the agenda must be followed, information input must be relevant to the discussion topic, present pertinent information to resolve discussed issues, and eventually information output shall result in the best alternative being agreed upon.

Sometimes, the best solutions are products of brainstorming during the meeting. To facilitate brainstorming, there needs to be lots of input from participants who are willing to think freely and voice their ideas. This leads to a fast-paced ideas meeting that potentially would come up with the best possible decision.

Decisions must be made "rather than deferring or avoiding controversial items" (McCurley). The important topics will never go away, and it is ideal to solve them immediately. Menial topics are not worth the time to spend. Either way, good decision making is important during meetings. These decisions must be recorded accurately. The responsible parties for the decision made, as in who will take certain action to resolve the issues, should be determined, and a timeframe should be set as to how long the particular action will be taking place. The status of the actions should be reported at the next meeting. While decisions are made, it is important to open up discussions to receive everyone's point of view and input. At the end of each meeting, a time, date, and location for the next meeting should be presented to make the next meeting most convenient for everyone.

2.4.1.3 After the Meeting

Afterwards, the meeting shall be evaluated by the meeting leader to increase the performance of the next meeting. The meeting should be evaluated on the basis of efficiency, effectiveness and overall feeling of the meeting. Additionally, follow-ups to the discussion topics must be made from each responsible party to get ready for the next meeting. The follow-ups then develop into the next meetings preparation as the information is distributed to the necessary parties. The exchange of information outside of the meeting requires strong collaboration to take place.

2.4.2 Collaboration

Without effective collaboration among party members, meeting preparation and meeting follow-ups would be impossible and all together pointless. If members are preparing for the meeting using out-dated information, the questions and concerns brought into the meeting would of no use to them. This in turn would form a highly ineffective meeting and waste a large amount of time correcting the information. It is the responsibility of all party members to actively collaborate with each other to prove the most accurate information possible. Collaboration can help solve issues or problems on a construction site quickly and effectively.

Like any construction project issues arose on the jobsite that were unforeseeable before the project began and required the cooperation and collaboration of the owners and construction managers to accomplish the desired outcome. One instance of coordination issues was that WPI purchased approximately 75% of the inside of the building from the WBDC, and then contracted Consigli for the interior finish construction. This required Consigli to draft a new set of plans for the interior finished construction to the specification set by WPI. Consigli now was required to manage the coordination of two sets of plans to make sure nothing that WPI wanted in their building was overlooked in the transition from the WBDC plans. This was accomplished by having another set of plans drawn up and the architect forming a new set of specifications that were adapted to include the new fit-out.

A more minor problem involving the restoration of the original building was the mortar color for the bricks. Since the mortar was placed over a hundred years ago the recipe has been long forgotten. Consigli wanted to match the color and went as far as to have the mortar chemically analyzed to identify what additives would produce the desired color. In the end it was the skilled eye of the mason that was able to recreate the mortar color almost exactly through his own blend of additives.

Another problem that was predicted but not entirely anticipated was the cracking of the cementitious underlayment in 68 Prescott Street. Northwest Systems, the contractor hired to place the slab, had informed Consigli that the floors would crack due to the natural curing and settling of the floors. Consigli relayed this concern to the owners and agreed that since the flooring surface was not critical to the structural integrity of the building, there was no need to rethink the floor choice.

What Consigli had not expected was the severity with which the floors did crack. As the concrete cured and settled, cracks began to appear as expected. However, it was not until a few months had passed that a series of major fractures appeared, running lengthwise down the building. These cracks were most prominent on the fourth and third floors; the second floor showed cracking but mostly as hairline fractures.

Consigli called Northwest Systems to confirm that these cracks were the anticipated cracks. In response, Northwest sent an agent out who determined that the cracks on all floors should be sealed with an epoxy-based solution and the major cracks on the third and fourth floors would be filled with mortar instead of epoxy.

The new building had its fair share of problems as well. The MRI basement needed special fiberglass rebar because metal rebar would have interfered with its operations. This was not known in the original price forecasting because it was not indicated on the drawings or specifications. The required change caused a substantial price overrun.

The ordering and storage of the lab casework was a concern of the WBDC because it was brought to the site earlier than anticipated. At the time of the delivery, the building was not sealed off and the HVAC was not fully installed. The WBDC felt that the casework was at risk of being ruined due to exposure and wanted it returned. Consigli assured the WBDC that everything would be alright concerning the equipment and that due to the time necessary to install the casework it needed to be delivered as scheduled. Consigli assured the WBDC that the storage area for the casework was being carefully monitored for temperature and humidity, and that any major problems would be addressed.

Each of these issues were resolved through team collaboration to develop the best solution. With out an effective way to collaborate and exchange information, the solution process could have seriously delayed the progress of the project. The meetings allowed for a time when all party members could develop solutions and share information effectively through face-to-face communication.

2.4.3 Communication

Communication is vital to the success of any project. Without clear and concise communication techniques, project errors and delays can occur. Notwithstanding the costs associated with errors and delays they can also undermine the synergy that is essential in construction between the owners and the management and the management and the workers. Communication can be oral, written or electronic and it is the project manager’s responsibility to use all the communication tools necessary to coordinate the jobsite and to formulate and execute the most efficient system of construction.

Communication needs to be directed to one focal point to minimize the chances of confusion. Owners, subcontractors and in-house laborers, via the superintendent, depend on the project manager as a means for accurately providing the knowledge and the direction for the project. It is the project manager's responsibility to establish a continuous and comprehensive communication exchange between the owners and the workers. Though the exchange of the majority of information relevant to a project is done through informal means, it is essential for the project manager to make a record of these transactions by documenting them as requests for information (RFI), change orders and punchlists. These exchanges can greatly impact the final outcome of the project. For instance, as a fire chief makes an inspection of a building, he/she will comment on where they feel exit signs, fire extinguishers, fire alarms, etc. should be located through a verbal discourse. It is the project manager who must keep note of these suggestions in order to keep the project from being delayed due to inadequate fire protection. (Oberlender, 2005)

In the not-so-distant past, project managers relied on pencils and paper for record keeping. Though this is the most common way of recording information, as the power of technology increases, the pencil is slowly being traded in for a keyboard and the paper for an LCD screen. The shift to digital communication allows project managers more opportunities to utilize methods of recording and exchanging information in ways that facilitate an easy transfer of data. These new methods allow for easier accessing, storage and displaying of data than previous methods of paper and pencils. One such method that is growing in popularity is the concept of project websites. Some such website include Prolog, AutoDesk and Buzzsaw, were companies are able to track progress through live updates. However, it must be noted that the effectiveness of these tools are only a good as the willingness people who chose to use them.

Websites and other related construction management software (Timberline, and Primavera) aside from websites, allow the project manager to keep up-to-the-minute postings on the progress of the project as well as any issues that may arise through the course of the day. Consigli uses Timberline for contract submittals and RFIs and uses Suretrack to keep of the project schedule.

Consigli and Gilbane's main form of facilitating communication with the owners is a weekly owners/architects/construction managers meeting. These meetings were agreed upon as a good idea, due to the complexity of the project, as a time when the owners would be able to formally meet with the architect and answer questions. However, these meetings were not a contractual agreement. These meetings were held on site outside of the weekly meetings, or if the subcontractors need to give an over-all understanding of their work to the meeting participants. Financial and design consultants, were invited to the meetings to assist the owners in making decisions for the project.

3 METHODOLOGY

3.1 TIMBER/CONCRETE COMPOSITE DESIGN

3.1.1 Statement of Design Problem

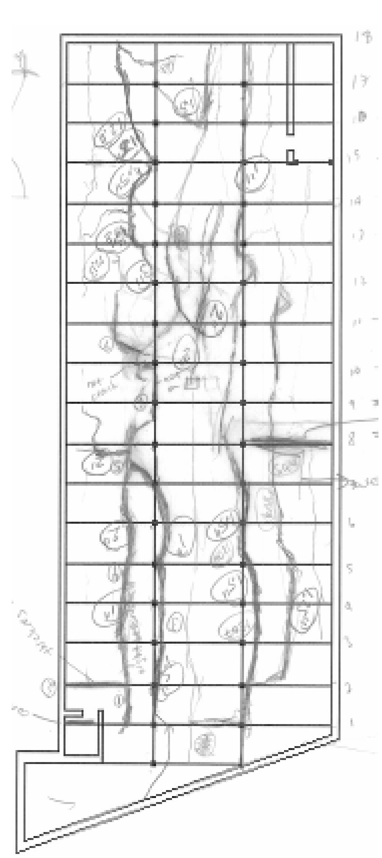
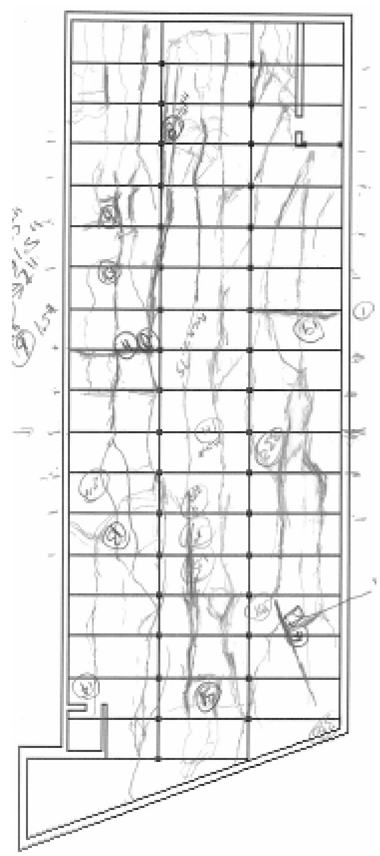
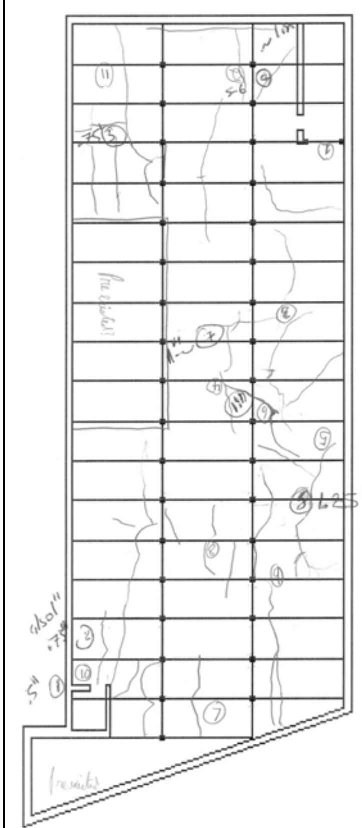
The floors of the original building of Gateway Park experienced an unexpected severity of cracking approximately two months after the concrete was placed.

3.1.2 Design Solution Approach

One or more of the problems explained in Section 2.4 may have caused the cracking of the concrete floor topping. A significant portion of this report involved examining one possible cause of the cracking in the floor slab at Gateway Park, determining an appropriate alternative solution, and developing a cost and schedule analysis of the design.

The first step in determining the cause of the slab failure was to record where the cracks occurred. This was done by creating a “crack map” as seen in, Figures 6-8. The figures show the cracks on fourth, third and second floors respectively.

The majority of the major cracks run lengthwise down the long axis of the building, with very few in the transverse direction. The dark colored lines indicate a wide crack while faint lines indicate a thin crack; the darker the line is, the wider it is. These dark lines are considered to be the major cracks and the cause of the concern on behalf of the owners and construction managers. The fourth floor experienced the most severe cracking and the second floor had very few cracks of substantial width.

 Figure 6: Crack Map of Fourth Floor	 Figure 7: Crack Map of Third Floor	 Figure 8: Crack Map of Second Floor

The variation of crack width could be due to the variation in slab thickness. According to Consigli the second floor has an average slab thickness half that of the third and fourth floors. The second floor experienced numerous hairline cracks as seen in Picture 8 on page 176, but very few major cracks. A large amount of material had been stored on the second floor, making it impossible to obtain a complete map as seen in Picture 12.

After reviewing the results of the crack maps, it seemed that one possible cause for the wide cracks could be tension inside the slab, caused by an excessive amount of deflection of the oak girders spanning left-to-right across the floor layout as depicted on figures 4-6. This deflection then caused the slab to deflect beyond its critical bending capacity, resulting in cracks due to the crowning over the girders spanning from top-to-bottom. The cracks could have resulted from tension in the top portion of the slab, as well as in points where the slab was least supported by the girder, as seen in Figure 9.

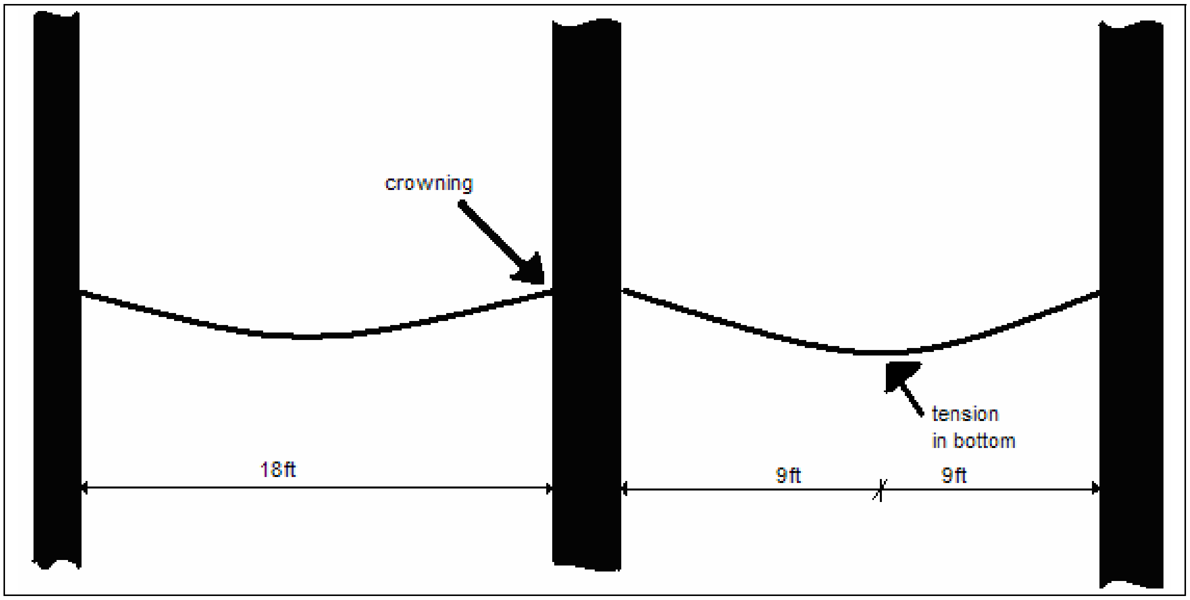


Figure 9: crack spacing

The next step was to determine if the loads on the girders were sufficient to produce flexural cracking of the slab. Bending moment and deflection calculations were performed on the girder using loading criteria obtained from Consigli at the time of cracking. These calculations determined that the oak girder could have deflected 0.6 inches from the dead and live loads on the building at the time of cracking.

Using the known compressive strength of the concrete, obtained from Consigli, the maximum allowable bending stress was found to be 530 pounds per square inch. However, the bending stress that would result from the 0.6 inch deflection of the slab was found to be 534 pounds per square inch. Therefore, the concrete slab was physically unable to withstand the bending of the system due to the weight of the concrete and the superimposing loading. (For detailed calculations, please see Section 8.1 DETAILED DESIGN SOLUTION APPROACH).

3.1.3 Hypothesis

The hypothesis states that the major cracks in the third and fourth floors of the 68 Prescott Street occurred because the loads on the system were sufficient to produce tensile stresses in the concrete in excess of the concrete's tensile limit. By changing the performance of the system by forming a composite floor system in which the concrete is principally in compression and the wood is in tension, through the use of common nails, the propagation of major cracks will be reduced.

3.1.4 Point Connector Testing

For purposes of comparison, the effective modulus of elasticity was investigated for five configurations: two composite systems, a non-composite system, a girder/plank system, and a girder. The modulus of elasticity was determined through manipulation of $\Delta = \frac{Pa}{24EI}\left(3L^2 - 4a^2\right)$, where Δ is the deflection due to the applied load, P is the applied load, a is the distance from the end support to the load support, L is the length of the beam from support to support, E is the modulus of elasticity, and I is the moment of inertia. Load and deflection values were obtained at the midpoint through the use of the Instron Machine's data acquisition program. Additional deflection points were obtained at the quarter length supports by using dial gauges. This data was then graphed as a stress-strain diagram.

The stress, σ, was determined through a manipulation of the bending stress equation $\sigma_b = \frac{Mc}{I}$, where $M = \frac{PL}{8}$ and c was equal to the neutral axis of the cross section. The neutral axis, c, was determined by using the equation $\bar{y} = \frac{\Sigma \bar{y} A}{\Sigma A}$, where $\bar{y}$ is the neutral axis of a specific section of the beam and A is the area of that section. The strain, ε, was found by manipulating the equation $E = \frac{\sigma}{\varepsilon}$, where E and σ are given by the previous calculations. To account for the different materials in the system, a transformation factor was used as presented in Section 2.2.3. An identical approach was used for the composite systems.

3.1.4.1 Testing Procedures

As stated previously, 68 Prescott Street does not utilize a composite floor system; however, it serves as a good comparison for the behavior of non-composite concrete/wood floor systems. The original intent of the project was to recreate the flooring system found at 68 Prescott St. and test it in a lab using composite and a non-composite systems. After a preliminary inquiry into the feasibility of making a scale model of the floor system, it became apparent that there were far too many constraints to build and study scale models appropriate for extrapolation to the in-situ conditions. The constraints included limitations of lab space, limitations of information and availability of material. Therefore, the revised purpose of the tests was not to mimic the floors at 68 Prescott St., but rather to develop an understanding of composite systems and provide a base for design of an alternative solution.

Girder Selection

The first step in testing the composite floor system was to determine the appropriate girder to be used. Materials similar to those at 68 Prescott Street were intended to be tested. However, after an initial inquiry into the procurement of oak girders, it was decided that such girders were too expensive due to their limited availability, and thus impractical to use for testing. No. 2 Douglas-Fir was the most readily available material at the local lumber yard and was therefore selected for this project.

The next step was to determine the size of the test specimens. Due to size constraints of the Instron machine, a full size 18’ x 8’ girder/slab section could not be used. The base of the Instron machine allows for a 5’ x 2 ½’ system to be easily tested without the need for extra supporting devices. However, if necessary, the floor system could be as large as 12’ x 2½’, but then this would cause additional problems with storage and transportation of the samples. Also, larger floor samples would require additional supports to prevent extraneous deflection and consequential cracking. Initially, a trial floor system size was determined by trying to scale down the length of the in-situ girder to a manageable size. The decision was made to scale the girder down to one-third of the original length, which was a six-foot beam.

3-Point Bend Test

The preliminary hypothetical 3-point bend test calculations provided test results that supported the testing hypothesis. However, two issues needed to be resolved before the physical testing procedure could be finalized. First, the size constraints of the Instron machine had to be addressed because a six-foot beam was still too large to fit on the

Instron's testing platform. A support system made up of I-beams would have to be constructed, resulting in the issue of the supports' deflection potentially influencing the deflection measurements of the floor system. To make the testing as simple as possible, a maximum length of five feet was agreed upon. The second issue that needed to be addressed was the 3-point bend test's effect on the concrete. The application of a concentrated load on the thin concrete slab was likely to cause localized cracking, thereby skewing the results of the test.

4-Point Bend Test

A 4-point bend test addressed the second issue by minimizing the effects of local cracking by halving the amount of load at the supports. More importantly, a 4-point bend test would allow for an area of pure shear and an area of pure bending on the beam, which enabled exploration of the composite systems' behavior under two different stress states. Figure 10 shows a rough sketch of the 4-point bend test. An I-beam was used to transfer the vertical load from the Instron Machine into the test specimen. The supports on which the I-beam rested on were placed L/4 the distance from the ends of the girder.

A new set of calculations needed to be performed to scale down the beam, given the constraints of the Instron Machine and the change to a 4-point bend test. As in the 3-point bend test, the deflection in the girder was checked to compare the deflection in the test samples with the deflection in the floors at 68 Prescott Street to see if it would be possible to correlate the data; it was not.

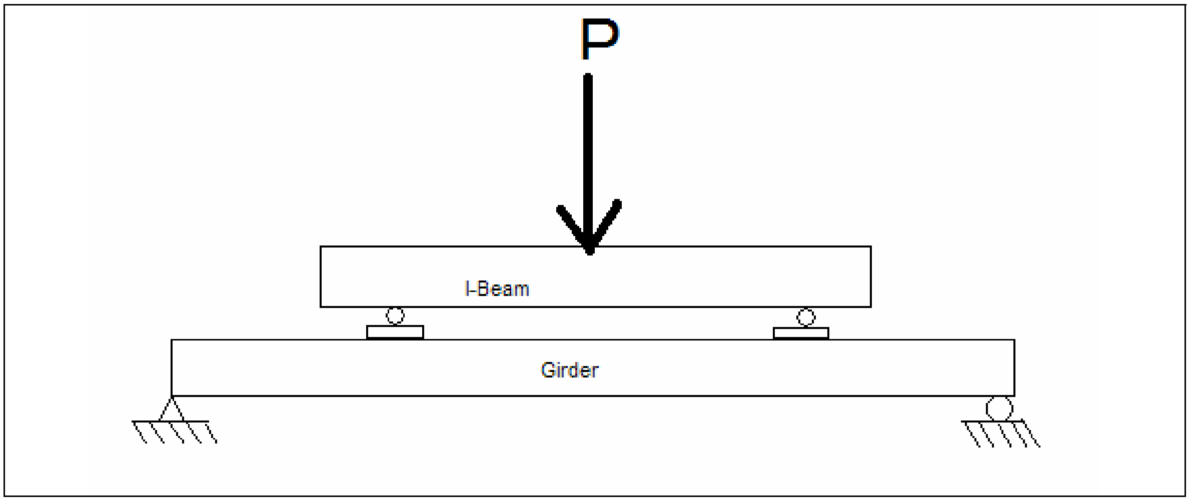


Figure 10: 4-Point Bend Test

The final dimensions of the test system comprised of a No. 2 Douglas Fir girder with a 4" x 6" cross-section cut to four-foot ten-inch sections. The floor planks became 1 inch plywood cut to the dimension of 8.25" x 3.25", totaling eighteen sections per beam. The concrete was 1.8" high and 8.25" wide totaling half a cubic foot of concrete per timber concrete beam.

Shear Studs

The next step in forming the test sections involved the determination of the type of shear connectors to be used. Nails were decided to be the easiest type of connector because they required the least amount of labor to install. Two separate methods were used to determine the number of shear studs needed. One method was based on first principles and the other was based on the method outlined in Peggi Clouston's 2006 article, *Wood Concrete Composites: A Structurally Efficient Material Option.* This method determines effective flexural rigidity $(EI)_{ef}$ of the composite wood-concrete system, using the equation, $(EI)_{ef} = \sum_{i=1}^{2}(E_i I_i + \gamma_i E_i A_i a_i^2)$ (Clouston, 2006, pg. 10), where

E is the modulus of elasticity, *I* is the moment of inertia, γ is the shear connection reduction factor, *A* is the cross-sectional area and *a* is the distance from the centroid of the respected component to the overall neutral axis. The amount of shear between the wood and the concrete was determined to be 189 *lb/ft* when loaded to 8000 pounds using the standard equation for internal shear within a beam section, $\tau = \frac{VQ}{It}$. The amount of shear that one shear stud (3.25 inch spiral-shank 12 penny nail) could withstand was found to be 163 *lb/ft* when calculated using NDS design specifications.

Based on the given calculations, only two nails were needed to withstand the shear between the wood and the concrete when an 8000 lb force 4000 lbs of shear was applied. This required a total of four shear studs to resist the total load of 8000 lbs. The results obtained from the first principles were very different from the results found by using the calculations in the Clouston (2006) article. The Clouston Method determined that a shear connector every quarter inch was necessary to transfer the load from the concrete to the beam in order to resist the bending stresses caused by an 8000 pound load. This resulted in a total of 120 nails as opposed to four. The large discrepancies in the two approaches led to a decision that both sets of solutions would be tested.

Pull Out Test

The value of the shear connector reduction factor is dependent on the slip modulus (K). To determine the slip modulus, a pull-out test was performed on the selected nail size at varying penetration depths. The slip modulus is then taken as the slope of the curve. Penetration depths of 15 times the diameter (d), 17d, 19d, and 21d were tested to see which depth would be used in the test systems. In a report written by

Ahmadi and Saka (1993), the suggested penetration depth was 11d. This depth was considered to leave too much nail exposed above the wood and might not leave enough concrete above the connector.

Table 5 shows the results of the pull-out test for various nail depths. The apparent variability in the data may have been a result of wood being a heterogeneous and anisotropic material. Thus, the amount of force required to pull the nail out is dependent on where the nail is set in the wood. In the same respect, since the nails were set relatively close to each other at the start of each test, diameters apart, perhaps a weakening of the wood block occurred and resulted in skewed test data. To achieve the largest possible slip modulus, the depth 21d was the final value agreed upon for the floor systems' shear studs. The final slip modulus was taken as 7502 psi/in.

Table 5: Pull-Out Test

Nail d (in)	0.133			
Penetration (in)	test 1 (lb)	Test 2 (lb)	test 3 (lb)	test 4 (lb)
15d	253.1	255.6		
17d	453.1	377	320.4	395.5
19d	289.3	270		
21d	573	556.5		

The effective flexural rigidity (*EI)* value depends on the slip modulus and the spaces between connectors. According to Clouston's 2006 article, the average composite reduction factor, $\gamma_{1,}$ of nails is between 0.1 and 0.4 as shown. The closer the value is to one, the higher the composite behavior will be. A connector spacing of a quarter inch was required to resist the loading of 8000 pounds. The application of so many nails was not deemed practical. A connector spacing of three inches was chosen to allow for some composite behavior to be observed, while at the same time making it easier to construct.

Given the slip modulus of 7502 psi/in and a spacing of three inches, the resulting γ_1 equaled 0.014. The composite reduction factor found in Clouston's 2006 article is

$$\gamma_1 = \frac{1}{1 + \frac{\pi^2 EAs}{KL^2}}$$

where *s* is the spacing of the connectors and *K* is the slip modulus.

Testing Justification

The shear connector used in Clouston's 2006 article was a continuous, expanded metal lath glued into a notch cut along the top of the surface of the beam. According to the data from the article, this continuous shear connector drastically out-performed point shear connectors, obtaining approximately ninety-seven percent of the maximum $(EI)_{ef}$. This method was not chosen for the project due to the expected degree of labor-intensive work it would require to construct, not only in the lab but also in the field. This appeared to be much too complex for the original intentions of the project, which was to find an effective TCC floor system for 68 Prescott Street. Instead, it was thought that nails would be far easier and more cost effective. Plus, due to the relatively low levels of loads, for which an office building is designed, a method such as the one developed by Clouston did not seem practical. Hence, all the preliminary calculations and brainstorming were completed with the intention of the hypothetical application in 68 Prescott Street. The intention of the testing was to compare two composite systems, the first principle system and the composite equation system, to a non-composite system to see what failed and why. Therefore, no additional changes were made to the selection of shear connectors and the least involved method was chosen.

Construction of Tests

Once the calculations were completed and a test girder of four-feet ten-inches was chosen, construction of the flooring systems was necessary. The beams and floor planking were cut down to size with the help of the CEE Department lab coordinators. The floor planking was cut to 8.25” x 3” sections; the typical effective flange width was 4.75 inches. Once the cutting was completed, the floor planking was nailed to all but one beam. This was necessary to understand how the girder behaves uninhibited by other material. Each plank had two nails to fasten it to the girder. The nails were adequately spaced to ensure that no slipping would occur between the planking and the girder. The shear studs were added at a penetration depth of 21d (2.8 inches) and spaced according to the specific calculations, two systems at three inch spacing (Clouston Method) and two systems at fourteen-and-a-half inch spacing (First Principles Method). Once complete, the formwork was attached using the excess floor planking material. The concrete was mixed in two batches, placed on eight of the beams, and allowed to cure at room temperature. Three 3” x 6” cylinders were filled for each batch for compression tests. Rags were soaked in water and placed on top of the concrete slabs, then covered with a tarp. The concern was that if the floor systems were placed in a curing room, the formwork would become saturated, expand and cause cracking in the slab once the formwork dried. Two additional slabs of similar dimensions were placed on top of flooring. These slabs were left uncovered and were allowed to cure at room temperature for observation of concrete that cures under imperfect conditions, such as those in the field.

The Test

The actual test consisted of a 4-point bend test. The maximum deflection of the floor system and the maximum loading were recorded using the Instron Machine. Dial gauges were also used to record additional deflections at the point of the load support. Two of each type of floor system were tested to allow for a comparison of results: non-composite floor systems, composite systems, girders, girder/planks and stand-alone floor slabs. To determine the strength of concrete, the cylinders were subjected to a compression test.

3.1.5 Biotechnology Center at 68 Prescott Street

To determine an appropriate alternative for the floors at 68 Prescott Street, a comparison was made of the quality, cost, and construction schedule of a point connector versus a continuous connector. The quality was determined through a deflection analysis where the maximum target deflection was half of the maximum deflection of 0.6 inches (0.3 inches). In order to ensure that the floors would not crack, the decision was made to limit the deflection to half the maximum deflection value. This would also allow for additional loads to be applied if needed in the future. A spreadsheet was developed to facilitate the computations of Clouston’s equations using the dimensions of the building’s floor. These equations were used to evaluate the tensile and compression limits. The cost and schedule implications were compared with those for the actual underlayment used for the building using information verified by the construction manager of the BTC.

3.1.5.1 Point Connector

The same 12 penny nail and the three inch connector spacing values from the laboratory testing were used to evaluate the deflection of the floors. A two-inch spacing was also used to determine if the reduction of nail spacing would result in a significant reduction in deflection and tensile forces.

3.1.5.2 Continuous Connector

Clouston (2006) employed a continuous expanded wire mesh adhered to a notch cut into the beam, as seen in Figure 11, to form a composite system between a glulam beam and a concrete slab. According to Clouston's study, this method performed significantly better than a conventional point connector because it was capable of reaching nearly 100% of the effective modulus of elasticity. The present study also compared the previously described method with point connectors by using developed spreadsheets and the slip modulus data from the 2006 Clouston article.

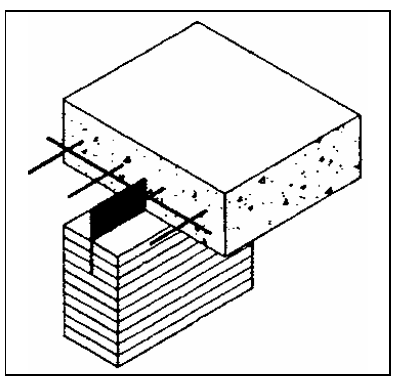


Figure 11: Expanded Wire Mesh Shear Stud (Clouston 2005)

3.1.5.3 Cost and Schedule Analysis

Cost and schedule analyses for the point and continuous connectors were also made in order to decide which system would be most appropriate. The cost analysis accounted for the cost of materials and labor based on information provided by Consigli. The construction schedule was analyzed to determine if application of the connectors would fall onto the critical path of the project. A spreadsheet was developed to analyze the costs and schedules of the three systems for comparison against the actual system. The cost included labor, equipment and materials whereas the construction schedule consisted of floor preparation, laying the connector, placing the concrete and curing. These schedules were also verified by meeting with the Consigli project manager.

3.2 MEETINGS

The goal of the communication section was to understand how the meetings were run and what other forms of communication were happening outside of the weekly meetings. This was accomplished through attending meetings and administering a survey to the meeting participants.

3.2.1 Hypothesis

Efficient meetings require a proper set of established goals as well as communication that provides necessary information exchange relevant to meeting those goals.

3.2.2 Testing Procedure

The hypothesis was tested through forming meeting forms while observing meetings and administering questionnaires to the participants of each meeting. Meeting forms were completed to provide insight into the weekly meetings. Questionnaires were administered to understand the information exchange and usage outside of weekly meetings. This information exchange and usage completes the flow of information that would be valuable in the preparation for weekly meetings.

3.2.2.1 Meeting Forms

Owner/architect/project manager meetings were attended with the intent to become familiar with the participants of the project and to evaluate communication techniques and an overall conduction of the meeting. Parking garage meetings were held bi-weekly by Gilbane Building Co. and biotechnology center meetings were held weekly by Consigli Construction Co. Though as many meeting as possible were attended, only the observations recorded at last three garage meetings and the last five biotechnology center weekly meetings were used for evaluation. The evaluation, which took the form of a "meeting checklist", quantified the quality of communication by means of following topics: interruptions (phone calls, unrelated talks, extra talks), technology (email, PDA, speaker phone), and meeting materials (meeting minutes, RFIs, change orders, site plans, materials samples, calendars, punchlists, personal binders, milestones, hand drawings). Furthermore, the "meeting checklist" recognized the meeting leader for each meeting. Mr. Steve Johnson of Consigli Construction Co. led the BTC meetings whereas Mr. Neil Benner of Gilbane Building Co. was in charge of the parking garage meetings. They provided the goals of the meeting which was done to fully understand the expected goal of each weekly meeting.

3.2.2.2 Questionnare

The construction managers, architect, engineer and owners were surveyed to better understand how the communication was handled outside of the meetings. First, a questionnaire was prepared in both an electronic and a paper format. Questions 1 through 4 were geared towards identifying the participant's background and their experience in construction. Question 5 discovered the preferred communication tools used outside of weekly meetings. Question 6 observed the level of volume information received outside of weekly meetings. Question 7 tried to understand the participants' use of received information outside of weekly meetings. The intention of Question 8 was to discover the comfort or interest level with computers or new technologies. Figure 12 shows the actual questionnaire and the rating system used.

With the permission of the project managers, the leaders of the weekly meetings, the paper version of the questionnaire was distributed. It was either filled out right away or brought back in the following week. To make the data collection even faster, the web link of the electronic questionnaire was sent out to the participants who were unable to attend the meeting or to those who felt it was easier to complete the questionnaire electronically. The electronic version gave the participants the flexibility to fill out at their own convenience as well as allowing the surveyor to receive the data promptly. Seventeen out of the expected eighteen questionnaires were filled out: thirteen on paper and four as electronic. Each one of the answers to the questionnaires was entered into a database. The outliers, which were participants of the weekly meetings who were not actively involved outside of weekly meetings, were not included in the finalized data. The finalized data provided results for the project.

Gateway Park Project Questionnaire

Name:

Email:

This survey is designed to better understand the communication that is taking place between the four major parties involved in the construction of the Gateway Park Project: two owners (WBDC and WPI) and two construction firms (Gilbane and Consigli). Thank you in advance for taking the time to complete this questionnaire.

1) Which one of the major parties are you representing at the Gateway Park meetings?

WPI WBDC Gilbane Consigli Other

2) Which one of the projects are you associated with?

60 & 68 Prescott Street Garage Both

3) At what stage of construction did you become part of this project?

Pre-Design Design Construction Other

4) How many similar construction projects of this magnitude (i.e. organizational complexity) have you been part of?

New experience 1 or 2 other projects 3 or more

5) Besides the weekly meetings, how frequently do you communicate with the other parties using the following methods?

Weekly frequency	0	1-5	6-10	11 or more
Face-to-Face				
Video Conference				
Phone Call				
Conference Call				
Voice-mail				
E-mail				
Mail				
Fax				
Online Collaboration				
Chat room / message board				
Site visits				
Memo				
Other				

Gateway Park Project Questionnaire

6) Do you receive additional information on the following topics outside of the weekly meetings?

Documents	Always (4)	Often (3)	Occasionally (2)	Rarely (1)	Never(0)
RFI					
Submittal					
Change Order					
Meeting Minutes					
Drawings					
Specifications					
Budget					

7) How often do you use these documents outside of the weekly meetings?

Documents	Always (4)	Often (3)	Occasionally (2)	Rarely (1)	Never(0)
RFI					
Submittal					
Change Order					
Meeting Minutes					
Drawings					
Specifications					
Budget					

8) Please check the boxes that characterize your comfort/interest level with computers in the work environment:

	Very High (5)	High (4)	Neutral (3)	Low (2)	Very Low (1)
Basic (MS Windows, MS Office, Internet)					
Construction software (AUTOCAD, TIMBERLINE, PROLOG, PRIMAVERA)					
New technology (Web-based Collaboration)					

9)Please provide us any other information that you feel might be relevant

Figure 12: The Questionnaire

4 RESULTS

4.1 TIMBER/CONCRETE COMPOSITE

As previously stated, the hypothesis for this section was:

Major cracks in the third and fourth floors of 68 Prescott Street occurred because the loads on the system were sufficient to produce tensile stresses in the concrete in excess of the concrete's tensile limit. By changing the performance of the system by forming a composite floor system in which the concrete is principally in compression and the wood is in tension, through the use of common nails, the propagation of major cracks will be reduced.

The hypothesis was verified by developing two sets of calculations. The purpose of the first calculations was to determine if the loading on the floors was adequate to cause cracking in the cementitious underlayment at 68 Prescott Street. The second set of calculations was used to develop a composite system using two different equation sets.

Lab tests were conducted to understand the behavior of timber-concrete designs. The tests consisted of 4-point bend tests of two composite systems based on the calculations as well as a non-composite system, a girder, and a girder-plank combination. Load deflection graphs were created through the use of data from the Instron Machine. In addition to this data, dial gauges were placed under the supports to allow for additional information to be collected for use if needed during the comparisons. Picture 9 shows the composite sections in 4-point bending with the dial gauges placed underneath the load supports.

Picture 9: Testing Photos

The flexural rigidity was calculated and graphed against the composite reduction factor, allowing for the percentage of composite behavior to be determined.

After the test data had been understood, the Clouston Method was utilized to compare the effectiveness of a nail connector versus an expanded wire mesh connector. With this data, a cost analysis based on 68 Prescott Street, was completed to determine the best alternative design.

4.1.1 Testing

Testing was conducted over the course of a week, with the beams typically subjected to a maximum load of 16,000 pounds before the test was stopped. With the composite sections, the girders were never taken to failure out of a concern of damaging the dial gauges. The dead load on each system included its own weight plus the weight of the two supports and the I-beam. The system weighed approximately 150 lbs and the supports and I-beam weighed 180 lbs.

4.1.1.1 Testing Observations

The cracking in the concrete occurred at approximately 8,000 lbs. The cracks, as seen highlighted in Picture 10, started at the supports at L/4 and propagated upwards as the load was increased. This is consistent with the moment diagrams of 4-point bend tests, where the maximum moment is experienced at L/4, shown in Figure 13. From an initial observation, it appeared that there was no variation between the three timber/concrete systems.

Picture 10: Typical Pattern Cracking

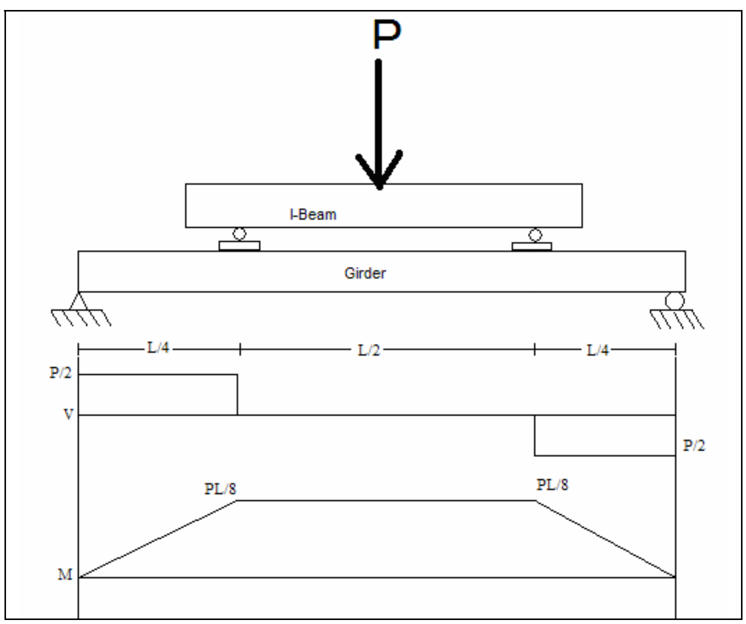


Figure 13: Shear and Moment Diagram for 4-point Bend Test

4.1.1.2 Test Results

The test results were tabulated using the data collected from the Instron Machine and a manipulation of fundamental material properties formulas.

Modulus of Elasticity Analysis

After determining the three most ideal beams, the modulus of elasticity was determined through the manipulation of the 4-point deflection equation $\Delta = \frac{Pa}{24EI}\left(3L^2 - 4a^2\right)$ into $E = \frac{(P_2 - P_1)a}{24(\Delta_2 - \Delta_1)I}\left(3L^2 - 4a^2\right)$. The moment of inertia (*I*) was determined by applying a transformation factor to the composite system. This allowed for the system to be converted to one *E* value and the subsequent moment of inertia to be calculated. Table 6 shows the modulus of elasticity values for the test specimen chosen for further analysis. A trend was observed that as more material was added to the girder, the stiffness increased.

Table 6: Modulus of Elasticity

Test Specimen	E (psi)
Girder #2	3.46E+05
Girder-Plank #4	3.52E+05
Non #1	3.69E+05
1st #2	4.85E+05
Clouston #2	5.69E+05

Stress-Strain Analysis

Figure 14 is a graphical analysis of the stress and strain of each system. The resulting figure gives evidence to the observation in Table 6 that the stiffness of the system increased along with the addition of material.

The girder, girder/plank and the non-composite section all show similar stress-strain relationships, as evidenced by their close proximity on the graph. There was a noticeable increase in the stiffness of the First Principles Method from the non-composite, which can be attributed to composite behavior. The increase in the modulus of elasticity from the First Principles Method to the Clouston Method was not as large as anticipated considering that fourteen nails were added.

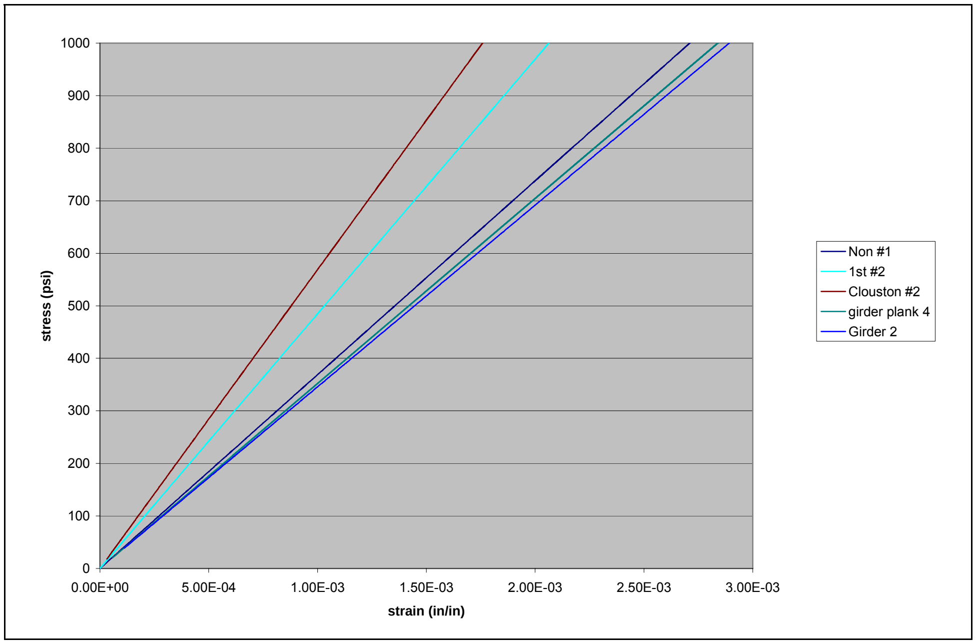


Figure 14: Stress-Strain

Normalized Data

The effective *(EI)* value was obtained at a number of composite reduction factors using the Clouston equations. Figure 15 shows the overall slope of the normalized EI values. Figure 16 shows a close-up of the area of the graph of the Clouston Method's and the First Principles Method's composite reduction factors. The composite reduction factors for both methods were found using the composite reduction factor found in Clouston's 2006 article. The composite reduction factors for the Clouston Method and the First Principles Method were 0.014 and 0.003 respectively. These factors correspond to an $(EI)_{ef.}$ of approximately 17% for the Clouston method and 16.5% for the First

Principles Method. The composite beam designed using the Clouston Method would therefore be able to resist a total load of 2575 pounds before cracking given 4000 psi concrete and the maximum tensile stress equation, $\sigma_t = \sigma_c - \sigma_b = \frac{M}{(EI)_{ef}}\gamma E a - \frac{M}{(EI)_{ef}}\frac{h}{2}E$, where M equals the maximum moment, and h is the height (Clouston, 2006). In contrast, the use of First Principles Method predicted that the composite beam would be able to resist a load of 2350 lbs.

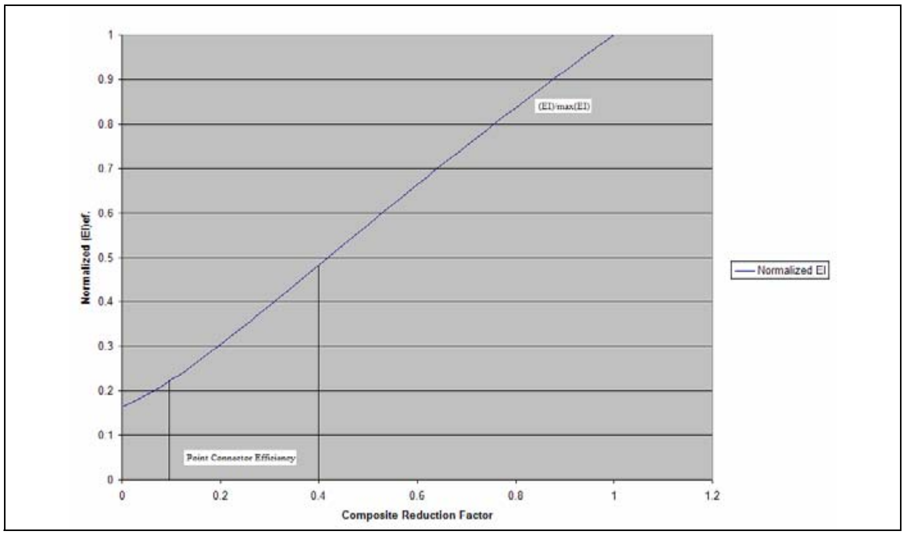


Figure 15: Normalized EI

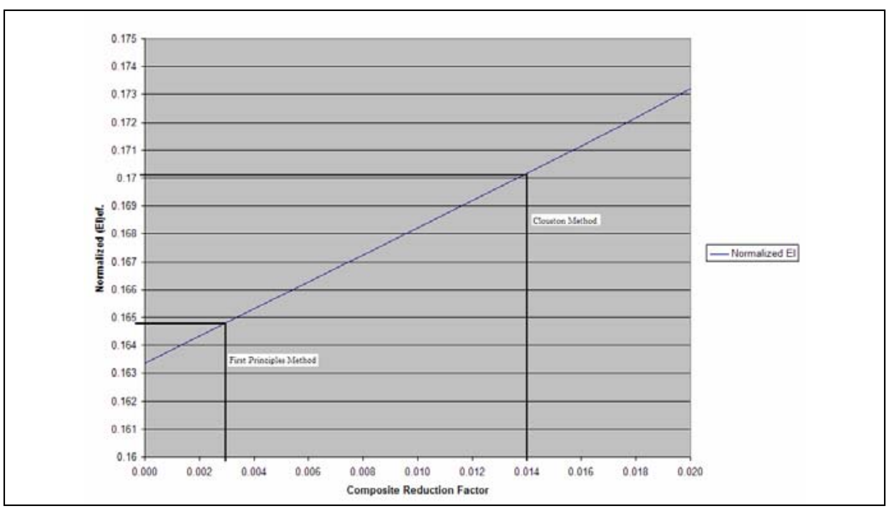


Figure 16: Close-up of Normalized EI

4.1.1.3 Testing Analysis

The difference in composite behavior is minimal between the results obtained through use of the First Principles Method and the Clouston Method. Figure 17 shows the stress distribution and Table 7 displays the values of stresses for the Clouston Method test system.

Table 7: Clouston Method Stress Distribution

Definition	Symbol	Clouston
Normal Compressive stress in the concrete due to force couple in the composite section	σ_C	59
Tensile stress in the wood due to the force couple in the composite section	σ_T	69
Maximum compressive bending in the concrete due to force couple about the concrete section	$\sigma_{b,C}$	1382
Maximum tensile bending stress in the wood due to the force couple about the wood section	$\sigma_{b,T}$	2613
Maximum compressive stress in the concrete	$\sigma_{c,C}$	1441
Maximum tensile stress in the concrete	$\sigma_{c,T}$	1323
Maximum compressive stress in the wood	$\sigma_{w,C}$	2544
Maximum tensile stress in the wood	$\sigma_{w,T}$	2682

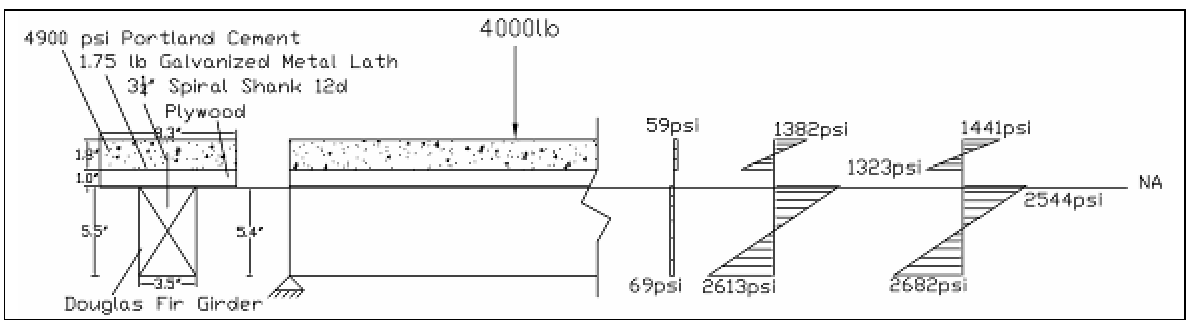


Figure 17: Clouston Method Test Distribution

Figure 18 shows the stress distribution and Table 8 show the values for the First Principles Method test system based on the predicted values from the Clouston Method spread sheet.

Table 8: First Principles Stress Distribution

Definition	Symbol	Clouston
Normal Compressive stress in the concrete due to force couple in the composite section	σ_C	13
Tensile stress in the wood due to the force couple in the composite section	σ_T	15
Maximum compressive bending in the concrete due to force couple about the concrete section	$\sigma_{b,C}$	1439
Maximum tensile bending stress in the wood due to the force couple about the wood section	$\sigma_{b,T}$	2721
Maximum compressive stress in the concrete	$\sigma_{c,C}$	1452
Maximum tensile stress in the concrete	$\sigma_{c,T}$	1426
Maximum compressive stress in the wood	$\sigma_{w,C}$	2706
Maximum tensile stress in the wood	$\sigma_{w,T}$	2736

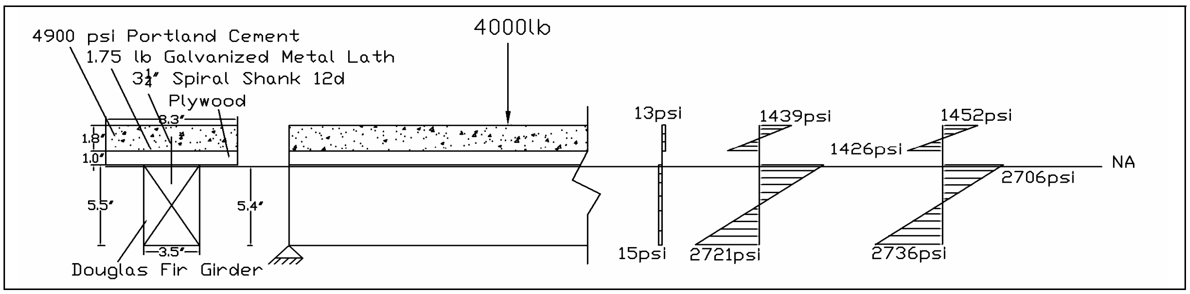


Figure 18: First Principles Test Distribution

4.1.2 Biotechnology Center at 68 Prescott Street

In order to find an alternative design for the floors, the test data were analyzed to determine the behavior of composite designs using the dimensions of the floors. Though point connectors were determined to be capable of transferring low levels of load, they were unable to withstand high loads. The continuous connector used in Clouston's 2006 article was able to withstand loads the point connectors could not handle. Since the point connectors are more cost effective, their use was not disregarded. Therefore, both connectors were analyzed as a potential alternative design. The applied loading for this analysis, seen in Table 9, was 1058 lb/ft. This load level accounted for all dead and live load values in situ at the time of cracking. These values were verified by Consigli. The connectors were analyzed assuming that 4000 psi concrete was used in place of the self-leveling cementitious underlayment currently used in the building. The assumption that the flooring and the girder act as a fully composite system is critical to the analysis of the connectors.

Table 9: Loading

Type of loading	Loads
$w_{concrete}$ (pcf)	105
w_{plank} (psf)	48
$w_{layerofwood}$ (psf)	15
w_{girder} (pcf)	45
$w_{metallath}$ (psf)	0.58
w_{hvac} (psf)	5
$w_{partition}$ (psf)	20
$w_{construction}$ (psf)	20
w_{snow} (psf)	35
$w_{roof\ deck}$ (plf)	434
$w_{roof\ rubber}$ (pcin)	0.04696
w_{wind} (psf)	24

4.1.2.1 Connectors

Both the point connectors and the continuous connectors were analyzed using the building's dimensions in the Clouston equations spreadsheet in order to determine which system would be most applicable for the BTC. The overall deflection of the floors and the maximum tensile stresses formed the basis for determining applicability.

Point Connector Analysis

The point connectors were analyzed at one, two and three inch spacings. The deflections of each system were 0.23, 0.33, and 0.39 respectively. Table 10 shows the values for the stress distributions, as well as the symbol and definition of the symbol. Figure 19 shows the stress distribution of the one inch spacing in the BTC floors, Figure 20 shows the stress distribution of the two-inch spacing and Figure 21 shows the stress distribution for the three-inch spacing. The deflection and the maximum tensile stress in the concrete were both less than the maximum allowable values.

Table 10: Stress Distribution Values for 1 inch, 2 inch and 3 inch spacing (Clouston, 2006)

Definition	Symbol	1 inch (psi)	2 inch (psi)	3 inch (psi)
Normal compressive stress in the concrete due to force couple in the composite section	σ_C	11	5	3
Tensile stress in the wood due to the force couple in the composite section	σ_T	72	62	53
Maximum compressive bending in the concrete due to force couple about the concrete section	$\sigma_{b,C}$	120	187	231
Maximum tensile bending stress in the wood due to the force couple about the wood section	$\sigma_{b,T}$	321	500	616
Maximum compressive stress in the concrete	$\sigma_{c,C}$	131	192	234
Maximum tensile stress in the concrete	$\sigma_{c,T}$	109	182	228
Maximum compressive stress in the wood	$\sigma_{w,C}$	249	438	563
Maximum tensile stress in the wood	$\sigma_{w,T}$	393	562	669

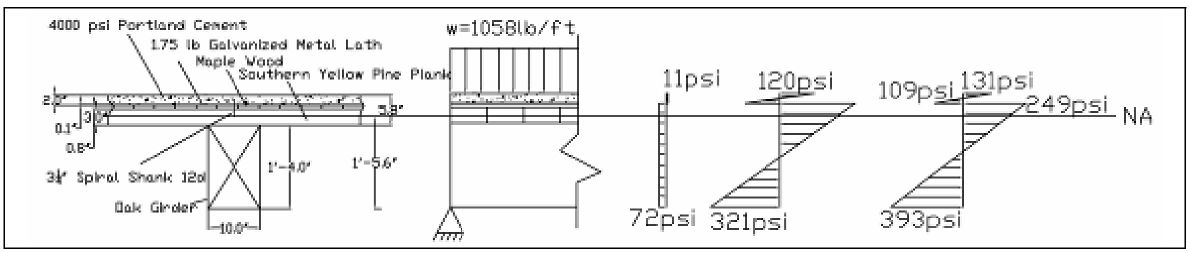


Figure 19: Point Connector 1" Spacing

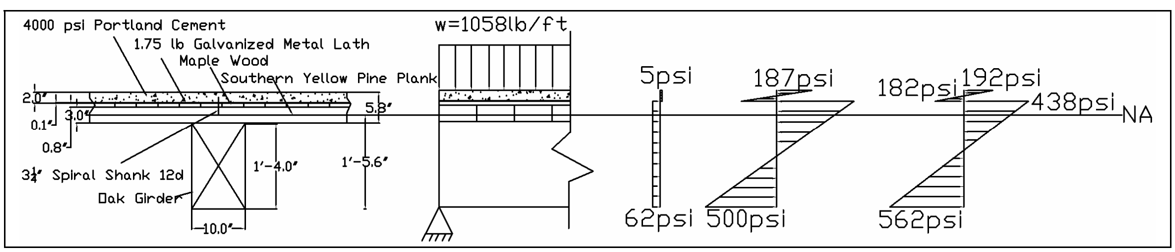


Figure 20: Point Connector 2" Spacing

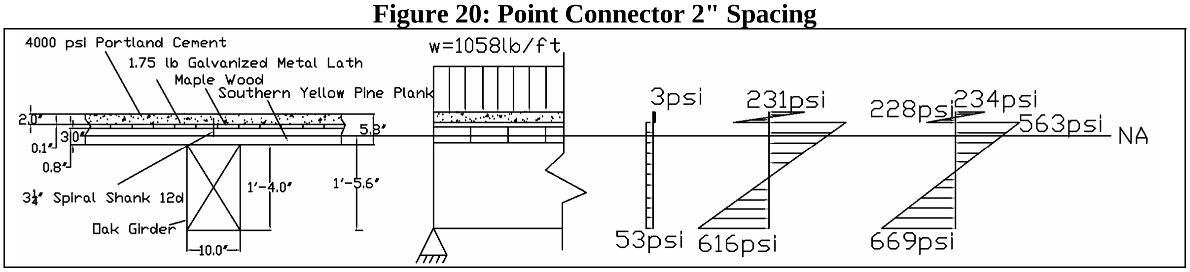


Figure 21: Point Connector 3" spacing

Continuous Connector Analysis

The continuous connector had a slip modulus of 151,000 psi/in for a one inch segment (Clouston, 2006) and the resulting deflection was 0.02 inches. Table 11 describes the stress distribution values seen in Figure 22. These stress distributions were found using the spreadsheets developed for the Clouston equations.

Table 11: Continuous Shear Distribution Values (Clouston, 2006)

Definition	Symbol	Continuous
Normal Compressive stress in the concrete due to force couple in the composite section	σ_C	59
Tensile stress in the wood due to the force couple in the composite section	σ_T	38
Maximum compressive bending in the concrete due to force couple about the concrete section	$\sigma_{b,C}$	15
Maximum tensile bending stress in the wood due to the force couple about the wood section	$\sigma_{b,T}$	40
Maximum compressive stress in the concrete	$\sigma_{c,C}$	73
Maximum tensile stress in the concrete	$\sigma_{c,T}$	44
Maximum compressive stress in the wood	$\sigma_{w,C}$	
Maximum tensile stress in the wood	$\sigma_{w,T}$	78

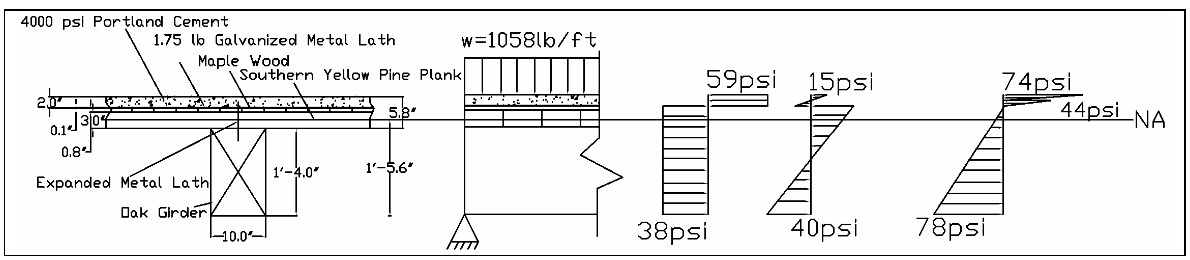


Figure 22: Continuous Connector

Normalized Data

Figure 23 and Figure 24 show the normalized effective EI values for the building. The one-inch spacing connector is approximately five percent composite in comparison to the continuous connector, which is fifty percent composite. These numbers were obtained using the Clouston equations.

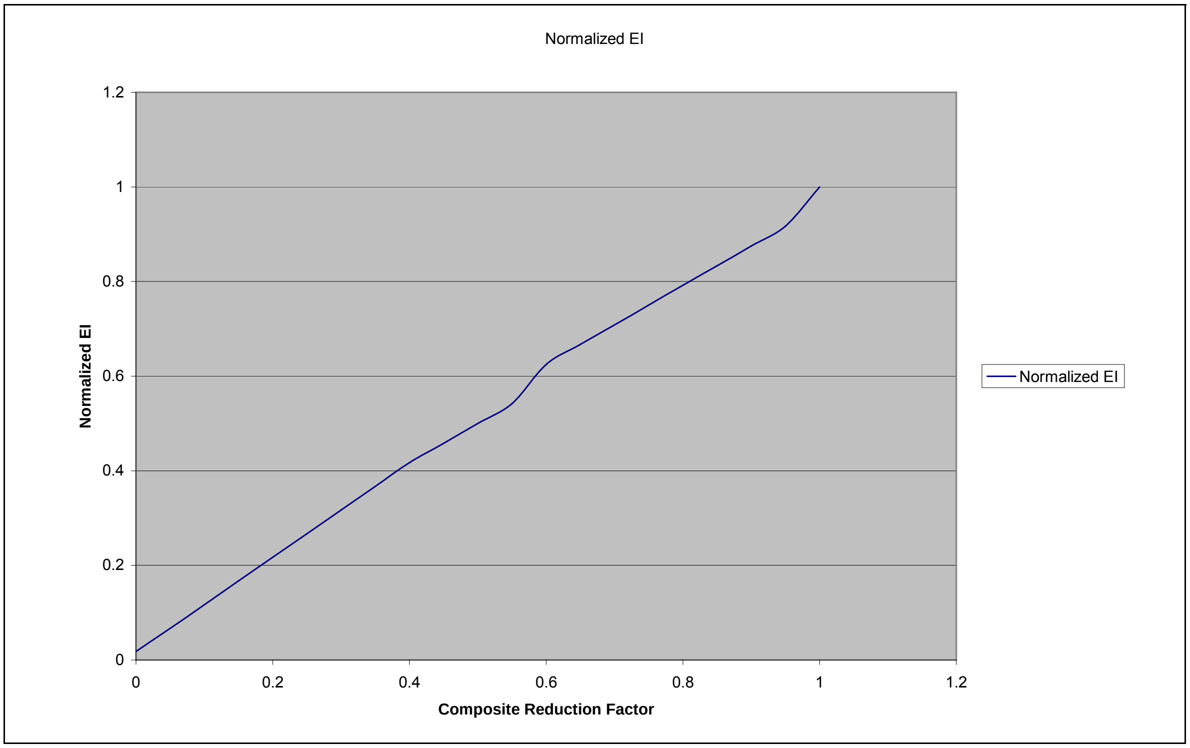


Figure 23: Normalized Graph for BTC

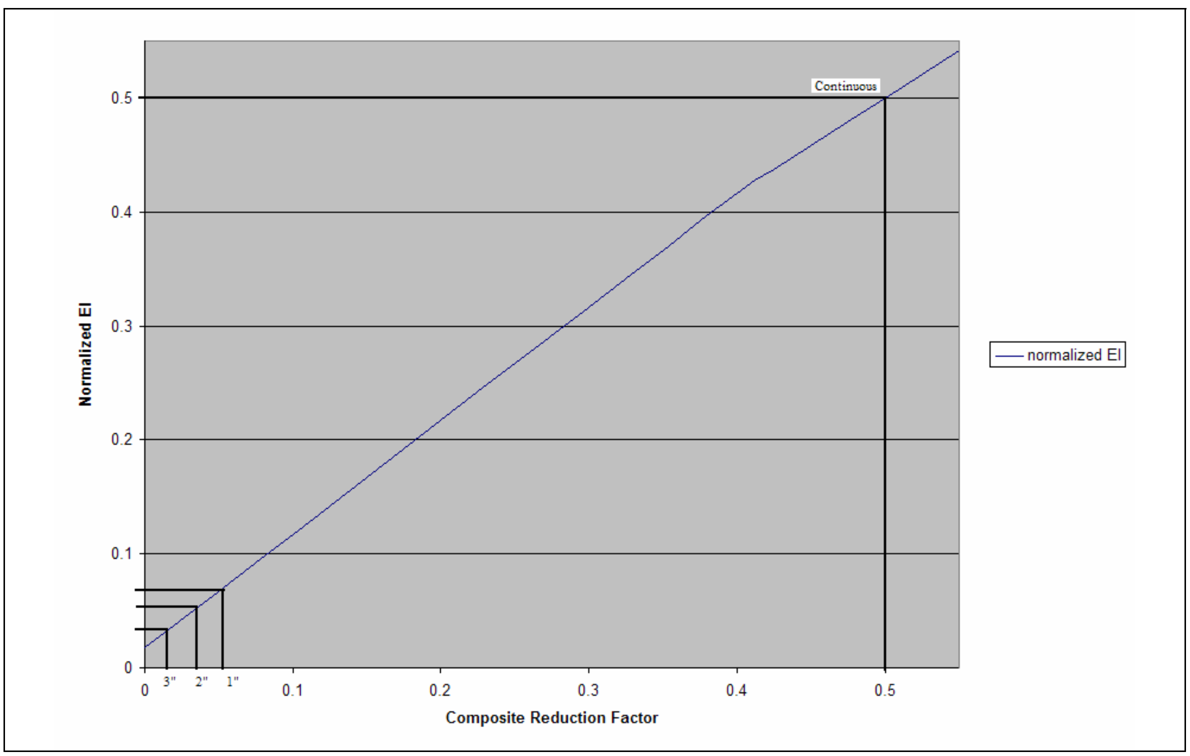


Figure 24: Close-up of Normalized Data for BTC

4.1.2.2 Cost Analysis and Scheduling

Cost and schedule analyses were done on the two different connector types to determine which system would be most economical as compared to the underlayment system. Figure 25 shows the total cost of each flooring option. The point connectors were the most economical when compared to both the continuous connectors and the current underlayment system. However, the underlayment was most efficient in terms of time because it eliminated the installation of shear connectors. The wire mesh, concrete, and preparation costs remain constant for all three floor options.

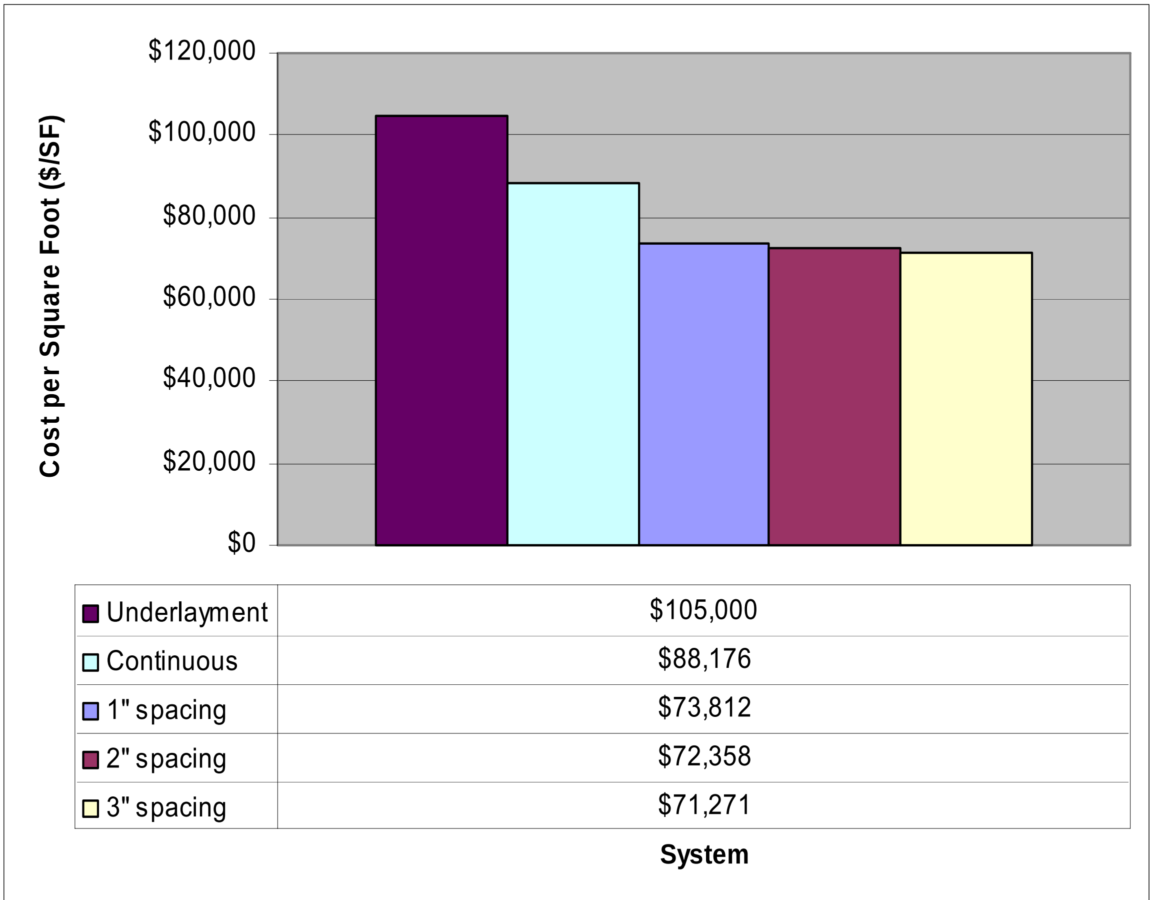
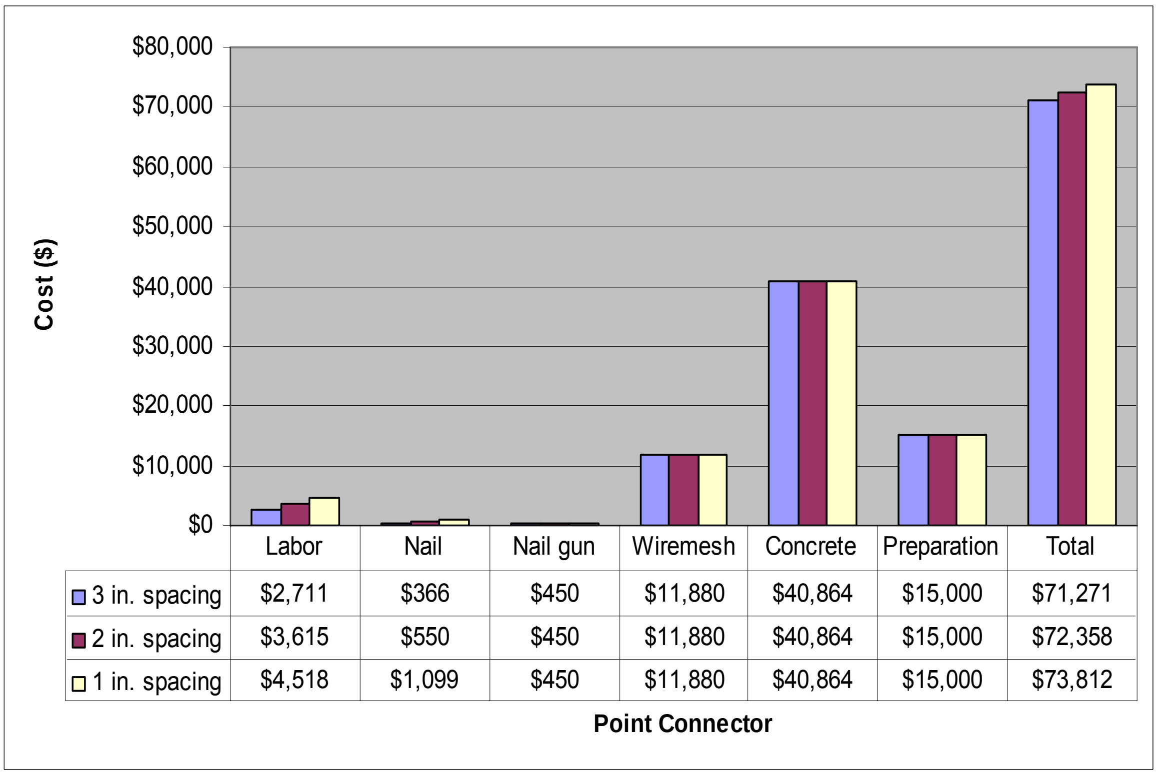


Figure 25: Total Cost for Flooring Options

Cost Analysis

Figure 26 shows a breakdown of the costs associated with the point connectors. Nails and labor are the two variables that differ amongst the three spacings. As the spacing between the nails increases, the cost of labor and nails decreases. The rest of the variables are constants because they do not change as a result of spacing between the nails.



	Labor	Nail	Nail gun	Wiremesh	Concrete	Preparation	Total
3 in. spacing	$2,711	$366	$450	$11,880	$40,864	$15,000	$71,271
2 in. spacing	$3,615	$550	$450	$11,880	$40,864	$15,000	$72,358
1 in. spacing	$4,518	$1,099	$450	$11,880	$40,864	$15,000	$73,812

Figure 26: Point Connector Cost

Figure 27 is a breakdown of the costs associated with the continuous connectors. Installment of continuous connectors is a labor intensive process because a notch or groove needs to be cut into the girder with the use of a circular saw, the notch needs to be cleaned, glue needs to be applied and the connector must be inserted into the notch.

The underlayment option costs $105,000, not including the cost of crack patching, which adds an additional $12,000 dollars. The total cost includes the preparation, wire mesh and concrete. The increased cost is due to the special cement used which was self-leveling and self-curing (Johnson, 2007).

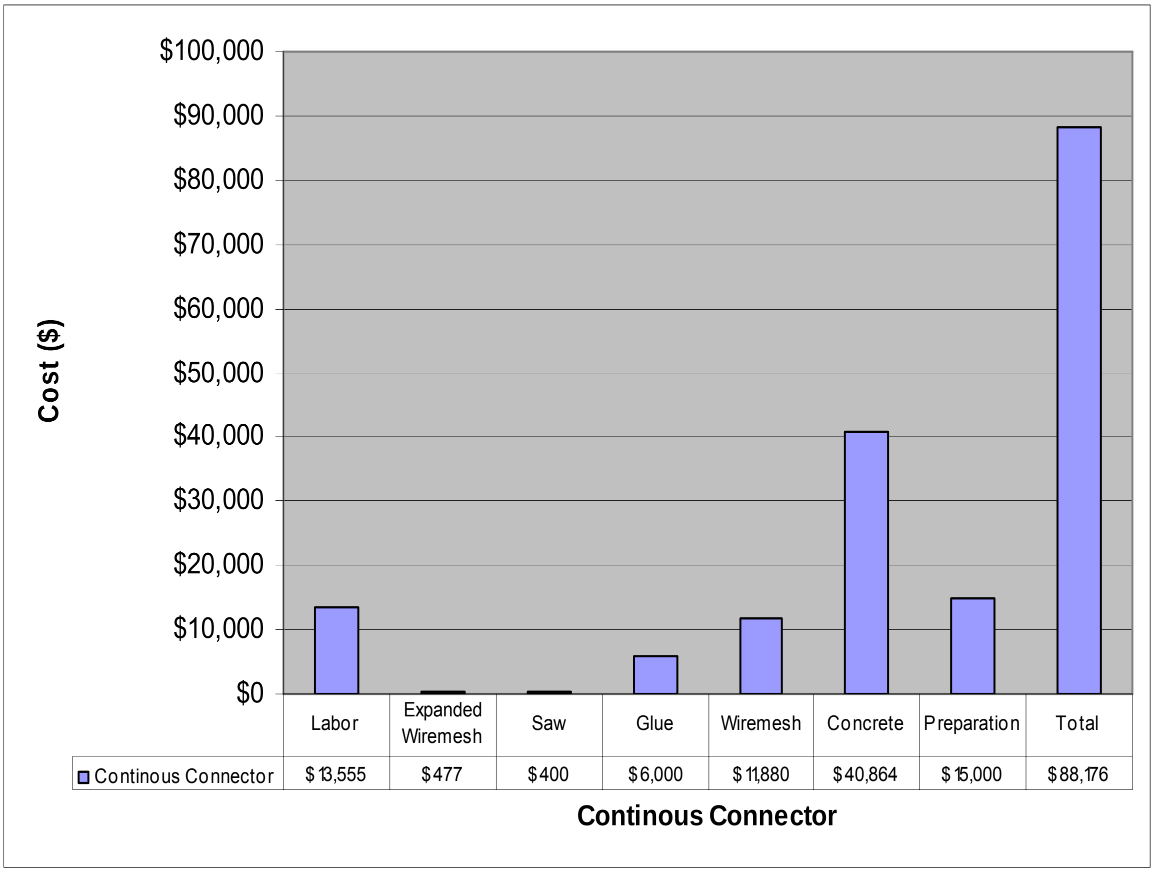


Figure 27: Continuous Connector Cost

Scheduling

Figure 28 shows a breakdown of the days required to apply each floor option. The only variation in the data is the time required to install the connectors. The underlayment required no connectors and therefore is the most time efficient option. Continuous connectors require the largest time commitment, making them least time efficient.

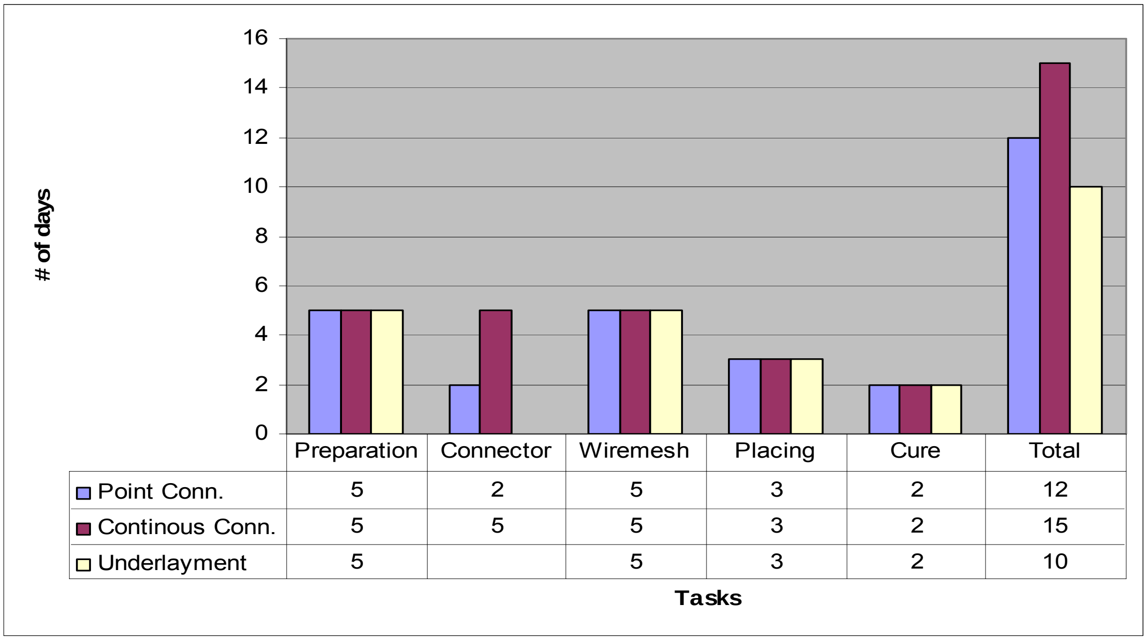


Figure 28: Construction Time

4.2 MEETINGS

The hypothesis for this section is that an efficient meeting required a proper set of goals as well as communication that provides the necessary information exchange relevant to meeting those goals.

The results to the meeting checklist and the questionnaire are provided as measures of meeting effectiveness and communication efficiency. Furthermore, the goals of the meetings are studied in depth and the meeting observations are discussed. Results for meeting checklists identify different communication tools used during the meetings, as well as time lost due to interruptions. The questionnaire leads to an understanding of the usefulness of the communication tools received and used between parties outside of weekly meetings. Overall, this section analyzes the cycle of information from meeting preparation to the meeting itself to follow-up after the meeting.

4.2.1 Meeting Observations

The BTCmeetings were held weekly at the Consigli Construction Co. trailer at eight o'clock every Monday morning. These meetings were not contractually agreed upon; rather they were deemed a necessity by Consigli Construction Co. due to the complexity of the project. At the time of observations, the BTC was in its final months of fit-out. The meeting participants at the BTC meeting typically included four representatives from WPI, three representatives from WBDC, two members from Consigli and the TKA architect.

The garage meetings were held at the Gilbane Building Co. site office at noon every other Wednesday, preceded by lunch. The garage was being erected at the beginning of the observations, and observations finished with discussions of when the power would be connected to the building. The meeting attendees for the garage meeting typically included one representative from WPI, three members from Gilbane, four members from WBDC and the Maguire Group architect.

The BTC had numerous sub-contractors and therefore the meetings were more complex and required the coordination of many more parties. This resulted in a meeting that was more an open forum, geared towards resolving issues and coordinating when and how the best course of action for a given task would take place. The garage had fewer subcontractors and therefore the meetings required less coordination; forty percent of the subcontracted work was from the pre-cast concrete. The meetings were much more informational, with the agenda being completed in less than an hour. In the garage meetings, only one major issue was needed to be resolved and it was taken care of as the first order of business.

WPI representatives played various types of roles during each meeting. These representatives included head of plant services, two consultants, and the vice-president of WPI. The two consultants and vice-president attended each BTC meeting, whereas the head of plant services attended both the garage and the BTC meeting. The roles each participant played varied from highly active to highly passive. The head of plant services was highly involved in the decision making process for both of the projects, the vice-president attended each meeting to observe the progress of BTC project, and the consultants provided inputs to the BTC project when necessary.

WBDC representatives were actively involved in both the garage and the BTC meetings. The WBDC representatives asked well informed questions and presented pertinent information to each meeting, leading to the assumption that WBDC was up to date on the current progress of each project.

The architects' opinions were highly regarded among the meeting participants. Due to their professional knowledge and experience, they were able to provide answers to questions when necessary. They understood the aesthetics and the functionality of each part of the buildings. The TKA architect was highly involved in the BTC meeting discussions compared to every other participant, whereas the Maguire Group architect provided his input to the garage meetings when necessary. For garage meetings, the site projector manager played an integral part in discussions, instead of the architect.

Project managers led and coordinated the weekly meetings, while also serving as a link between the owners and architects. Project Manager Steve Johnson of Consigli Construction Co., leader of biotechnology center meetings, believed that there were three main goals to weekly meetings of the biotechnology center:

- “Update the team and keep them informed on progress and schedule.
- Get questions answered from the architect.
- Discuss any open issues that require input from the team.”

He elaborated on the importance of weekly construction meetings and how to enhance the productivity of these meetings:

> “You can get a lot more accomplished by getting everyone together in the same room rather than sending emails or having conference calls. People are more accountable and productive if they know they have to face a room full of people each week. Everyone at the table is expected to provide any input that is relevant to the discussions/issues, follow-up on their open issues from the previous week, and help resolve any conflicts.”

Project Manager Neil Benner of Gilbane Building Co., leader of garage bi-weekly meetings, believed the goal of the garage project meeting was to “maintain a formal channel for information flow to keep owner up to date on current issues/budgets.” He continued to offer his opinion on the intention of a weekly meeting:

> “Gilbane discusses construction issues. Maguire discusses design issues. WBDC discusses owner issues. These discussions prompt information flow with often unexpected results (which is largely the purpose of the meetings.) There would be no point in meeting if everyone knew what the results would be. This information exchange enables progress to be made on site.”

Project Executive Bill Kearney of Gilbane Building Co. and Project Executive Brian Hamilton of Consigli Construction Co. both agree that the main goals of an Owner/Architect/Engineer meeting are to:

- Ensure communication flow
- Review outstanding Submittals, RFI, Change Orders, Schedule, Budget and other issues that are in the meeting minutes
- Discuss issues of quality control
- Bring new items
- Set completion dates for tasks and follow up
- Make decisions
- Solve Issues
- Discuss Safety

- At 50% completion, add job close-out to the agenda

The minutes provided the agenda for the weekly meeting by outlining the order in which the topics of discussions would be addressed. The progress of the project, submittals, and change orders were reviewed. From an owner representative's perspective, weekly meetings were the time when concerns were brought up and the progress of the project was tracked. The architect and the engineer provided additional guidance to the owners to make necessary changes or choices. Such choices included light switch locations, carpet color, and guardrail options. Weekly meetings served as informative sessions and time to resolve issues between the owners, construction managers, the engineer, and the architect. If there were any issues with the subcontractors, then they were asked to attend weekly meetings to provide their input in order to resolve issues on the table.

There was a large volume of information exchange in the form of paper documents during the meetings. These paper documents were mainly the change orders, submittals, meeting minutes, and RFIs. These documents provided a need for each participant to develop a method of archiving all this data, mainly through the use of heavy duty binders.

Typical interruptions took the form of phone calls, unrelated talks and extra talks during the meetings. Phone calls took away from the meeting because they disrupted the harmony of the meeting topic, and would force one individual to be indisposed for a period of time. "Unrelated talks" was when the group conversation strayed from the meeting minutes, causing delays and inefficiency to meeting flow. "Extra talks" occurred when parallel conversations were conducted in the form of two or more participants discussing a separate issue between themselves while the meeting was in progress. Generally, these conversations happened when topics unrelated to them were being discussed. However, these individuals could potentially miss an important bit of information.

Figure 29 and Figure 30 depict the average number and duration of different interruptions. Phone calls and unrelated talks did not create major disturbances during the meetings since they occurred infrequently over the course of the data collection period. However, one garage meeting skewed the results with a twenty nine minute divergence, since no major issues were needed to be resolved. Extra talks were fairly low; generally two members of the same party would discuss an item to be presented to the team. Biotechnology Center (BTC) meetings experienced more extra talks due to the complexity of the project. BTC project required additional attention to be paid to items that were to be addressed at the meeting. This required team members to double check each other on the topic that was to be discussed.

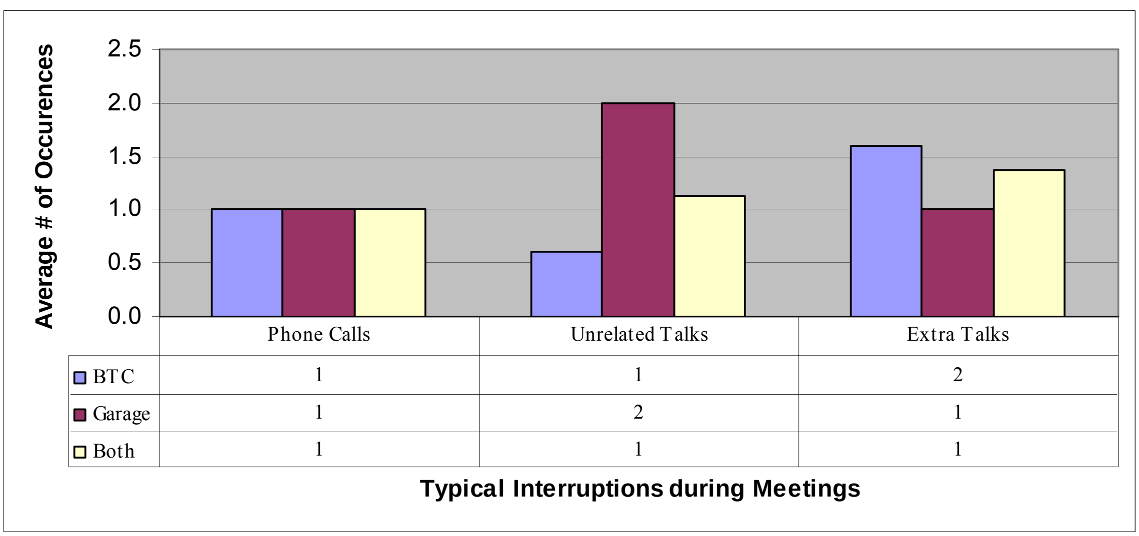


Figure 29: Typical Interruption during the Meetings

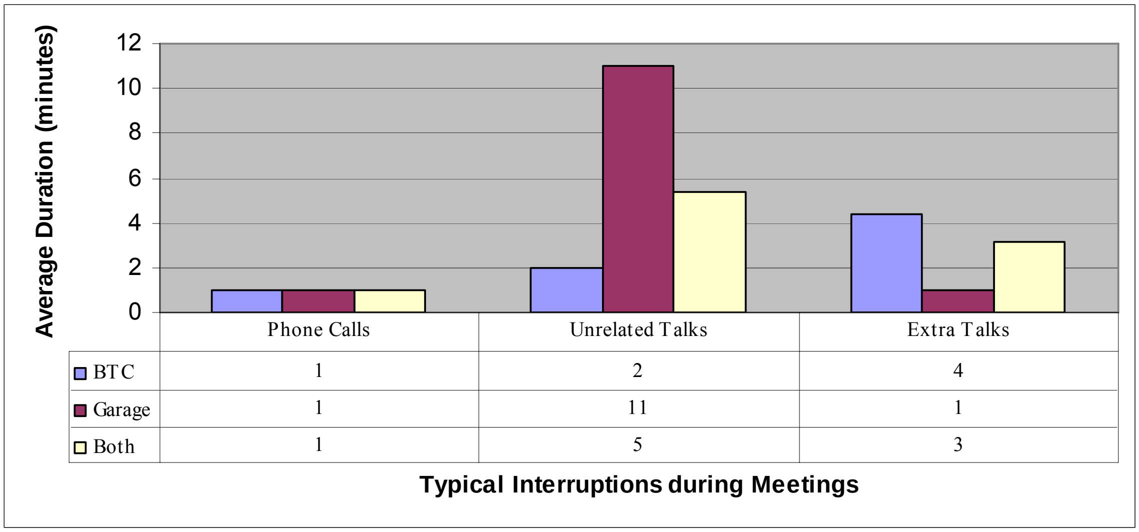


Figure 30: Average Duration of Interruption during Meetings

During the course of the meeting, visual references, in the form of site plans, hand drawings, and material samples, were used to complement a discussion. Often times, these items were brought up briefly and used to demonstrate to the other members of the meeting what was being specifically discussed. Material samples were also used to present the owner with choices for their wants and needs.

Figure 31 shows the frequency of visual references used during each meeting. For example, tiles to line the exterior of the elevator shafts were displayed during the one of the garage meetings. Carpet choices were displayed as paper copies of the particular color and texture options during the BTC meeting; however no material samples were observed. Site plans were commonly used, typically after the meetings, to resolve any outstanding issues. Hand drawings were frequently mimed out in the air at the garage meetings. In one of the observed BTC meetings, the architect presented his idea on paper as a sketch.

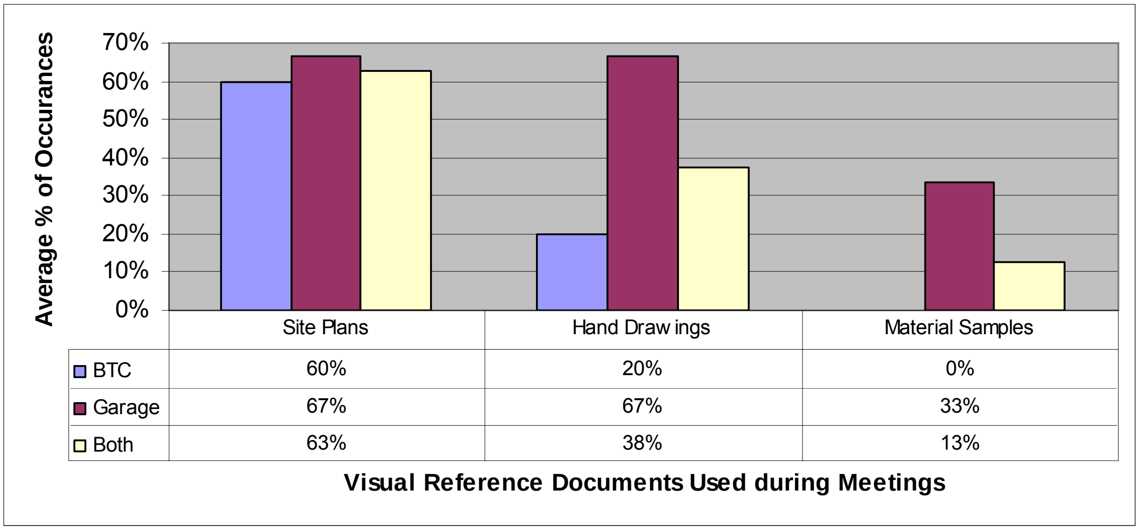
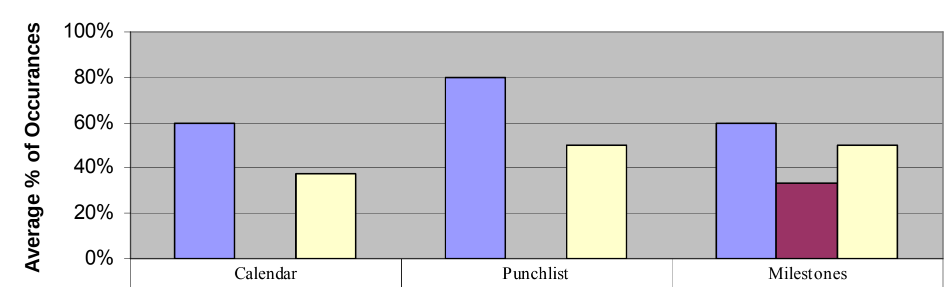


Figure 31: Visual References Used During Meetings

Scheduling tools, in the form of calendars, punchlists, and milestones, were used during the course of the meeting to help facilitate the progress of the project and to give the participants a well established timeline. The BTC which was in the close-out stage by the time the majority of the "meeting checklists" were filled out showed that the punchlists and milestones were extremely important for the continuation of the project. Calendars were used to coordinate numerous safety inspections and site visits as a result of BTC's phase and complexity of the project.

Figure 32 shows the frequency of use of scheduling tools used during the meetings. The parking garage was fully erected at the time of the "meeting checklists" were completed. Once the pre-cast concrete erection was finalized, the project began to increase the complexity; more subcontractors were needed to provide the final requirements.



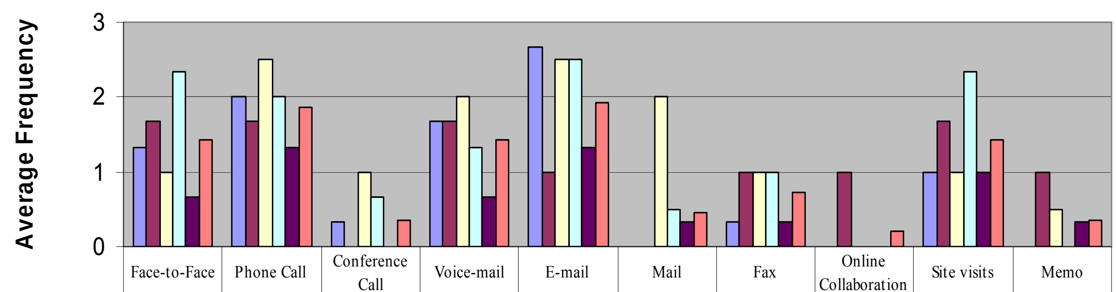
	Calendar	Punchlist	Milestones
BTC	60%	80%	60%
Garage	0%	0%	33%
Both	38%	50%	50%

Figure 32: Scheduling Tools Used during Meetings

4.2.2 Questionnaire

Fourteen participants: three members of Consigli (project executive, project manager and the project engineer), three members of Gilbane (the project executive and two project managers), three WBDC representatives, three WPI representatives (Head of Plant Services and two consultants), TKA architect and Maguire Group engineer were surveyed and their answers were compiled to form the questionnaire data. The participants with lesser involvement from WPI and WBDC, as well as the subcontractor, were disregarded from the finalized questionnaire data. Although the vice-presidents of WPI and WBDC attended the meetings, they were not included in the questionnaire data because their positions were more of observation during the weekly activities. These individuals were important to the project, providing support to the members of the project if needed.

Figure 33 depicts the tools used to communicate between parties outside of weekly meetings. The average frequency responses ranged from 0 to 3 (where 3 is "11 or more", 2 is "6 to 10", 1 is "1 to 5" and 0 is "0". WBDC was most active outside of weekly meetings. Site visits and face-to-face discussion were the preferred method of communication, providing ease of collaboration. There was a lot of communication through emails especially for the BTC meeting. Phone calls usage proved to be more evenly distributed among the parties involved in the project. Voice mail received an average amount of use compared to frequency of email and phone call usage. Online collaboration, mail, fax, memo, and conference calls were the least utilized communication tools. WPI representatives did not use the communication tools as often as WBDC had. This could be due to WPI participants' role in the project. WPI representatives were not concerned with particulars of the overall project. They were more concerned with specific task such as AJB consultants were solely concerned with the lab equipment and related items.

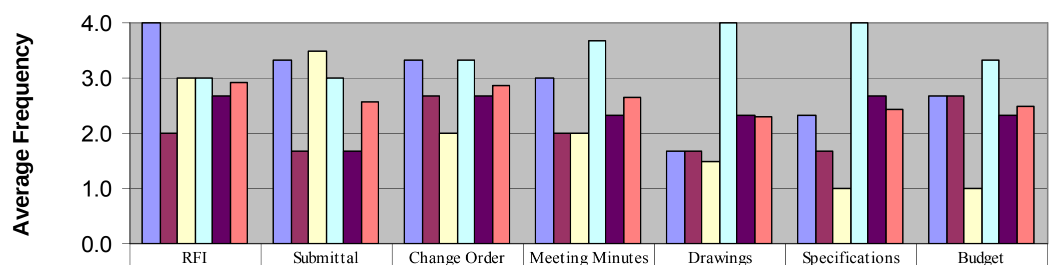


	Face-to-Face	Phone Call	Conference Call	Voice-mail	E-mail	Mail	Fax	Online Collaboration	Site visits	Memo
Consigli	1.33	2.0	0.3	1.7	2.7	0.0	0.3	0.0	1.0	0.0
Gilbane	1.67	1.7	0.0	1.7	1.0	0.0	1.0	1.0	1.7	1.0
Other	1.00	2.5	1.0	2.0	2.5	2.0	1.0	0.0	1.0	0.5
WBDC	2.33	2.0	0.7	1.3	2.5	0.5	1.0	0.0	2.3	0.0
WPI	0.67	1.3	0.0	0.7	1.3	0.3	0.3	0.0	1.0	0.3
All	1.43	1.9	0.4	1.4	1.9	0.5	0.7	0.2	1.4	0.4

Tools Used to Communicate between Parties Outside of Weekly Meetings

Figure 33: Tools Used Outside of Meeting

Figure 34 shows the additional information received outside of weekly meetings. The average frequency responses ranged from 0 to 4 (where 4 is "always", 3 is "often", 2 is "occasionally", 1 is "rarely", and 0 is "never"). WBDC received the most amount of information outside of weekly meetings. This could have been due to the fact that they were more actively involved in the City Square Project. Consigli Construction Co. received the most amount of information outside of the weekly meetings such as RFIs. This was probably as a result of the complexity of the project that Consigli Construction Co. was involved in. WPI received low amount of information outside of weekly meetings. Between the owners and the construction managers there was a good flow of information. The “others” (architect and engineer) received a lot of RFIs and submittals which was expected since they were required to sign off or provide responses. However, the remaining information was not as often received because both the garage and the BTC designs had already been completed. This meant drawings and specifications were finalized. Change orders and budgets were evenly distributed to the four major parties due to the fact that they were essentially very important to the owners and the construction managers.

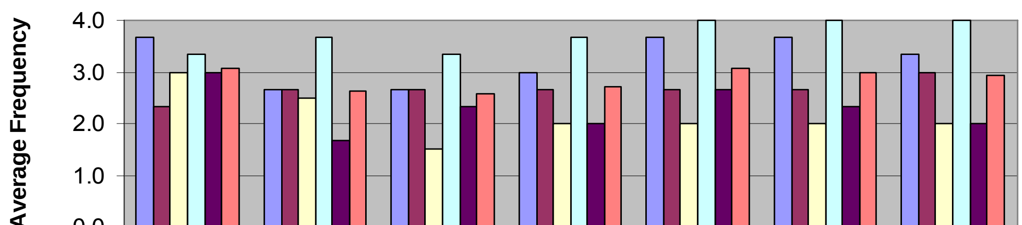


	RFI	Submittal	Change Order	Meeting Minutes	Drawings	Specifications	Budget
Consigli	4.0	3.3	3.3	3.0	1.7	2.3	2.7
Gilbane	2.0	1.7	2.7	2.0	1.7	1.7	2.7
Other	3.0	3.5	2.0	2.0	1.5	1.0	1.0
WBDC	3.0	3.0	3.3	3.7	4.0	4.0	3.3
WPI	2.7	1.7	2.7	2.3	2.3	2.7	2.3
All	2.9	2.6	2.9	2.6	2.3	2.4	2.5

Additional Information Received Outside of Weekly Meetings

Figure 34: Additional Information Received outside of Weekly Meetings

Figure 35 shows the tools used to communicate between parties outside of weekly meetings. The average frequency responses ranged from 0 to 4 (where 4 is “always”, 3 is “often”, 2 is “occasionally”, 1 is “rarely”, and 0 is “never”). There was a good chance that the material used outside of weekly meetings had a direct correlation to the usage of these materials due to the direct willingness of the participants to obtain the information and then use it. However, these materials could have been received during the weekly meetings or the participants had received the information, and not used it as often as they received it. With this being said, there is a high correlation for both WBDC and Consigli between the materials received and materials used. Again, this was due to complexity of BTC and WBDC's total involvement in the project. Drawings and specifications were not often received by Consigli, however, they were always used. There was a large amount of information outflow on behalf of Gilbane, and a low inflow of information to them. This was likely due to the simplicity and phase the project was in. Therefore, Gilbane's job was to keep parties involved on the project's progress through distribution of information, meaning that more information was sent than received. Architect and Engineer occasionally used the information outside of weekly meetings. WPI did not use the information as frequently as anticipated for being the partial owner of the BTC and garage.

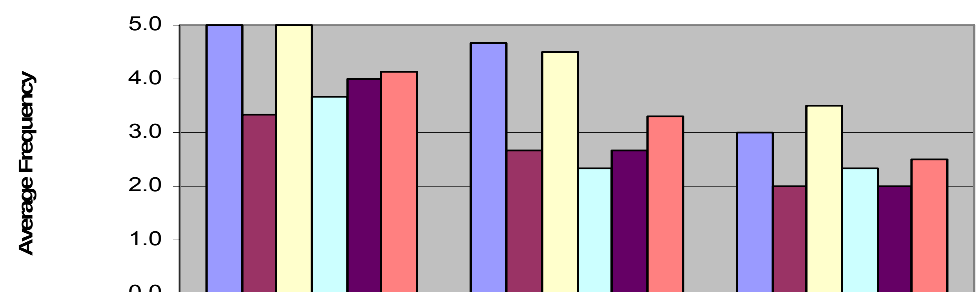


	RFI	Submittal	Change Order	Meeting Minutes	Drawings	Specifications	Budget
Consigli	3.7	2.7	2.7	3.0	3.7	3.7	3.3
Gilbane	2.3	2.7	2.7	2.7	2.7	2.7	3.0
Other	3.0	2.5	1.5	2.0	2.0	2.0	2.0
WBDC	3.3	3.7	3.3	3.7	4.0	4.0	4.0
WPI	3.0	1.7	2.3	2.0	2.7	2.3	2.0
All	3.1	2.6	2.6	2.7	3.1	3.0	2.9

Use of Documents Outside of Weekly Meetings

Figure 35: Use of Documents Outside of Weekly Meetings

Figure 36 shows the comfort or interest level with computers. The average frequency responses ranged from 1 to 5 (where 5 is “very high”, 4 is “high”, 3 is “neutral”, 2 is “low”, and 1 is “very low”). Basic computer comfort or interest level was fairly high for all participants of the project. Consigli and “others” (TKA architect and Maguire Group engineer) were very interested or comfortable with construction software tools. TKA Architects and Maguire Group were also interested in new technologies. Overall, new technologies were less desired and comfort or interest level for construction software was average.



	Basic	Construction Software	New Technology
Consigli	5.0	4.7	3.0
Gilbane	3.3	2.7	2.0
Other	5.0	4.5	3.5
WBDC	3.7	2.3	2.3
WPI	4.0	2.7	2.0
All	4.1	3.3	2.5

Comfort/Interest Level with Computers

Figure 36: Comfort/Interest Level with Computer

After analysis of the questionnaires and “meeting forms”, the results could be explained in terms of WBDC being the most involved in the project. This was likely due to WBDC complete involvement in all aspects of the project which included interior and exterior concerns, landscaping, and site grading project. WPI’s consultant’s responses were low compared to WPI’s Head of Plant Services John Miller’s responses, resulting in overall low information exchange ratings. This means the consultants attended the meetings to observe and offer advice rather than control the project.

The information flow and the intention of the meetings were based on the complexity and the stage of the project for the construction managers both during and outside of weekly meetings. BTC meetings were used to clear up any outstanding problems and present new solutions whereas garage meetings were more of informative meetings to present updates. The architect played more of an integral role in BTC meetings and was vital in the final decision process for the unresolved issues.

5 CONCLUSION

Alternative solutions were proposed for the construction of the floor slabs and the increase in quality of meetings. The suggestions were based on the results of testing and analysis of meetings, for each part of the Gateway Park Project evaluated.

5.1 TIMBER/CONCRETE COMPOSITE DESIGN

After careful study of the results, the best option for the floors at 68 Prescott Street was determined to be the point connectors spaced at one inch. The 1" spaced point connector best balances the quality, as determined by the overall tensile force and deflections of the floors; cost, by the cost per square foot; and schedule, in terms of the number of days required for installation. Figure 37 shows the stress distribution in the floors at the BTC using a one inch nail spacing. The maximum design tensile force in the building (109 psi, seen in Figure 37), based on the load factors, would be less than the maximum allowable tensile force of 475 psi. The maximum allowable tensile force was found using 4000 psi concrete and the ACI equation $7.5 \rightarrow 10\sqrt{f_c'}$. Since the maximum design tensile forces are less than the maximum allowable tensile forces the concrete floors would not crack.

The deflection of this system would be 0.23 inches, well below the critical bending of 0.6 inches. The cost per square foot, though slightly higher than the other two point connector options, is much less than the continuous connector or the current underlayment system. For scheduling purposes, the one-inch point connector is only slightly more time consuming than the original underlayment option, as seen in Figure 38.

The addition of two days does not cause the construction of floors to fall on the critical path of the project. However, the assumption is that as one floor is fitted with shear studs and concrete is placed, work is able to progress on the other floors. Given that the concrete is able to be walked on after one to two days, the concrete curing would not slow down the building's progress. As Project Manager for the BTC, Steve Johnson said, "the addition of five days on a year-and-a-half long project will not cause any serious delays in the project schedule." (Johnson, Steve. personal interview)

In summary, the one-inch spacing was chosen as an alternative design for the floors at 68 Prescott Street because:

- Tensile force and deflection in the concrete are sufficiently reduced to prevent cracking;
- The cost of the system is economically advantageous when compared to the other two options considered (continuous and underlayment); and
- The scheduling does not disrupt the critical path

Table 12: Comparison of Floor Options

System	σ_t in Concrete (psi)	Deflection (in)	Cost ($/SF)	Schedule (days)
3" Point Connector	228	0.39	$3.00	12
2" Point Connector	182	0.33	$3.05	12
1" Point Connector	**109**	**0.23**	**$3.11**	**12**
Continuous Connector	-44	0.03	$3.71	15
Underlayment	< 475	< 0.60	$4.42	10

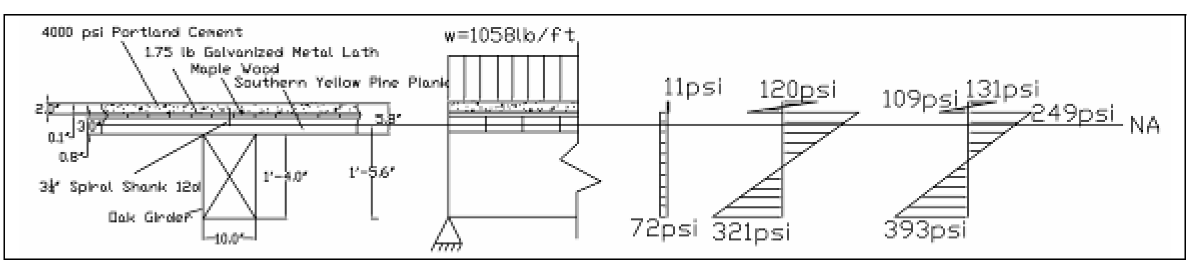


Figure 37: 1" Stress Distribution in BTC

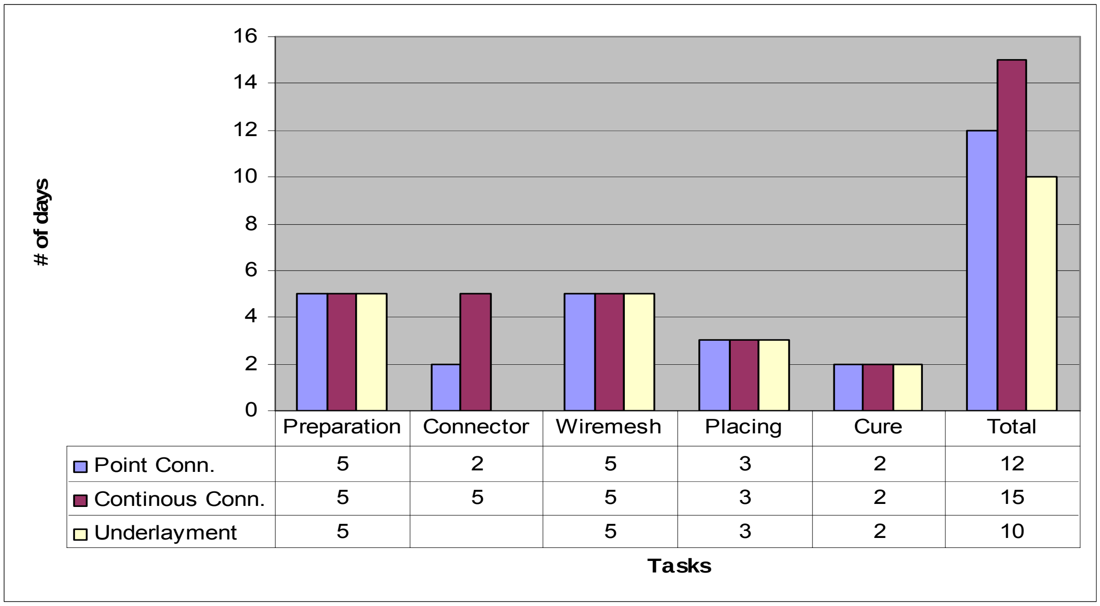


Figure 38: Schedule Breakdown

5.2 COMMUNICATION

The main goals of a weekly meeting remain the same, regardless of the project. However, the specific intentions of a project differed depending on its phase and complexity. The main objectives of the weekly meetings were to keep participants informed about the project and resolve issues. However, the major difference in the agenda of the BTC meetings was the addition of a question and answer session between the architect and the owner. The established goals of the meetings provided the optimal environment for face-to-face communication to occur by enabling each participant to exchange information in the most effective manner. However, the goals of the meetings would be better met without interruptions and extraneous conversations

The atomic model (Figure 39) shows the essential fundamental process for collaboration to meet the established goals of the project. As the model demonstrates, each party is linked to the meeting through the information they provide. Each observed meeting had participants supplying information and requesting information from other participants, thus representing the flow of information. .

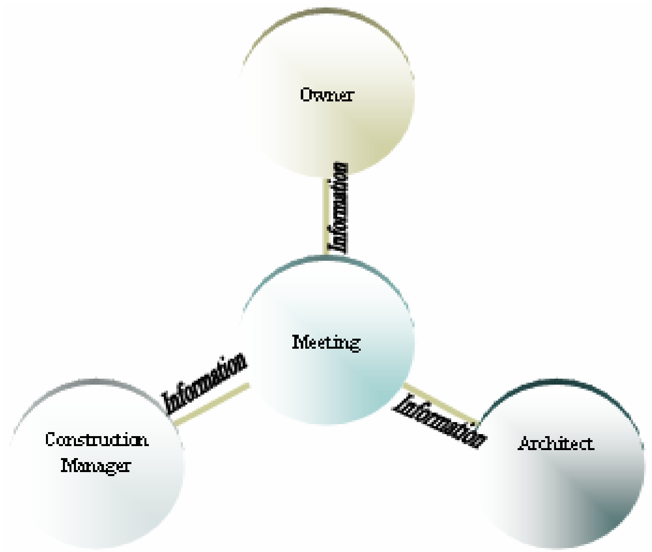


Figure 39: Atomic Model

Efficient meetings require a symbiotic flow of information from one step to another. The cyclone model (40) below shows the direct flow of information exchange between the various meeting processes. The information flow received through follow-ups and preparations enables the updated information to be exchanged during the weekly meetings.

In this project, the information was shared through means of different communication tools, as shown in figure Figure 33 and Figure 34. During the meetings, all the participants received the same amount of information. Outside of weekly meetings, participants had the flexibility to use the available communication tools to exchange information over the course of week. However, the tools, including technology applications, were only as valuable as participants' involvement in the project. Some participants frequently exchanged information while others did not. For example, WBDC representatives showed a lot of interest in the information exchange and used it frequently. This preparation was reflected at the meetings by asking timely questions regarding the progress of the project. WPI did not use the exchange information as frequently outside of weekly meetings. Subsequently, their participation during weekly meetings was low. This showed a general trend that the more information used outside of weekly meetings, the more involved the participants would be during the weekly meetings, thus supporting the hypothesis that communication leads to effective meetings.

The more complex the project gets, the more information exchange takes place during and outside of the meetings. Depending on the stage of the project, the type of information that is provided and asked for during and outside of meetings varies. The complexity and stage of the BTC project highly impacted Consigli Construction Co.. and the TKA Architect's involvement in regards to the exchanged information. Gilbane Building Co. and Maguire Group's involvement was not as frequent in comparison. The construction management firms and architects did an excellent job in providing the necessary communication to efficiently run the meetings. As a result, the hypothesis once again is supported.

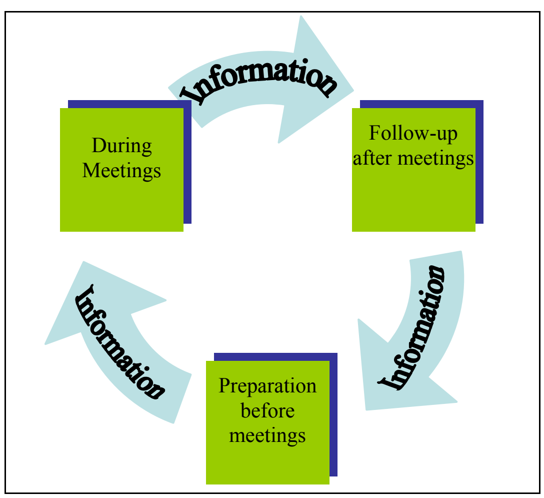


Figure 40: Cyclone Model

After observing the meetings and administering the questionnaires, two major improvements are being proposed for the weekly owner/architect/CM meeting: increase use of electronic media in meetings and decrease interruptions and distractions due to extraneous conversations and phone calls. Even though the current participants are either uninterested or uncomfortable with the introduction of technology, it should be utilized in the weekly meeting environment to help streamline the exchange of information. Interruptions and extraneous conversations can only be improved upon by clearly defining a standard of etiquette at the beginning of the meeting series.

Weekly presentations should utilize more electronic-presentations to provide better visualization and coordination during the meeting. This electronic presentation can be complementary to a building information model (BIM). BIM would not only enable 3D visualization at the meetings, but would provide information exchange to take place efficiently between meetings. BIM can provide clash detection, scheduling and cost estimating integration, substantial reduction of RFIs and elimination of change orders. Furthermore, BIM and e-presentation will cut down on the information in the form of paper which is inefficient, costly, and not sustainable. The Bull's Eye Model (Figure 41) depicts information exchange using communication tools such as BIM and e-presentations. The chart does not consider change orders and submittals due to BIM's capabilities to eliminate them. The information starts in the weekly meeting and through collaboration it is filtered out to the follow-up and preparation by using communication tools as means of exchanging information. The updated information is then returned to weekly meetings. As John McDermott of TKA-Architects stated in his questionnaire, "BIM is coming".

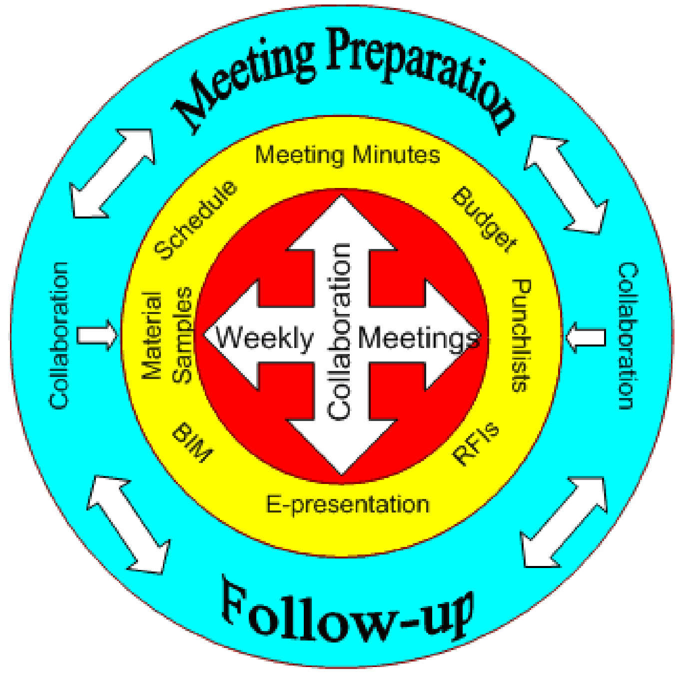


Figure 41: "Bull's Eye" Collaboration Model

The project manager is responsible for the order of the meeting and it is his or her duty to regulate the distractions and use of time during the meeting. In much the same way safety standards are enforced on a jobsite, meeting standards to eliminate distractions can be set. A request to turn off cell phones and other electronic devices can be made at the start of each meeting to remind participants of protocol. The project manager also should remind participants that the point of the meeting is to discuss the current topics and not to jump ahead in the agenda. Careful preparation must be made by each party to ensure that all necessary materials are present at the meeting to reduce any last minute photocopying or printing. Ultimately, it is the responsibility of the meeting leader to uphold the standards of the meeting, though the success of such standards heavily depends on the willingness of the participants.

In summary, four key points to this case study are as follows:

- The established goals of the meetings were sufficient; however the extra and unrelated talks as well as phone calls should be eliminated during the course of a meeting to keep interruptions to a minimum.
- Face-to-face communication in the form of weekly meetings should be the corner stone of communication since it provides the most direct and complete information flow between the participants.
- Communication tools should be used to complement this direct communication.
- Technology (communication tools) is only as good as the participants' willingness to use it; however, they should at the same time recognize important technologies such as BIM.

6 FUTURE RESEARCH

After a retrospective analysis of the testing processes was conducted, a few suggestions as to how to improve the results and take the analysis further were thought of:

- Use of a glulam beam would be ideal because it is an engineered wood that has little to no flaws and allows for a similar modulus of elasticity as the oak girders in the BTC. Glulam would not only help to recreate the scenario in the floors but increase the reliability of the results.
- An examination into the mechanics of a continuous connector would be helpful in the assessment of it as a viable alternative for 68 Prescott Street.
- Strain gauges would be very helpful in determining the location of the neutral axis for the composite sections and calculating the effective modulus of rigidity, EI.

- More testing of each system would allow for a better sample size and increase the validity of results.
- Dynamic testing would demonstrate the effectiveness of the composite systems under repeated loading and unloading.

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8 APPENDICES

8.1 DETAILED DESIGN SOLUTION APPROACH

8.1.1 Crack Mapping

The basement floor and the first floor of the original building did not experience any visible cracking because they were placed on solid ground. Both floors are placed slab-on-grade except a small portion of the first floor where the basement is. A majority of these slabs had been placed before Consigli started working as part of the original factory building. Only a small section on the first floor needed to be replaced by Consigli and are not part of the renovation of the floors.

The second floor had numerous minor hairline cracks throughout the floor when compared to the third and the fourth floors. A major difference between the second floor and that of the third and fourth was the amount of concrete placed. The second floor required about half as much as the other two due to their level of unevenness of the pre-existing wood floors. There was a large variation in the depth of the cracks between the three floors.

Table 13: Average Maximum Crack Depth

Floor	Crack (in.)
2	0.875
3	1.66
4	1.66
Total avg.	1.5

The third floor and the forth floor were similar in their crack patterns; having major cracks on either side of the pillars as well as half of the distance between the pillars and the walls. The major cracks run from end to end and are the primary concern of Consigli. The average crack depth was about 1.66 inches for the third and forth floors and the cracks are spaced roughly 18 feet apart. Table 13 shows the average crack depth across the three floors. The variation in the crack depth could be due to the difference in the average slab thickness between these two floors.

Bending moment and deflection calculations were performed to see if the cause of the cracking was due to the tension caused by the sagging of the girders. The cracks in question are perpendicular to the girder span.

8.1.2 Bending Moment & Deflection Calculations

Calculations were performed assuming the concrete acts as a simply supported beam with no additional reinforcement from the metal lath. The metal lath was stapled directly to the floor to mechanically fasten the underlayment. However, according to Consigli, the amount of concrete under the metal lath was limited because of the way the metal lath was fastened to the floor and therefore, the concrete can be assumed to span across the underlying floor with minimal support from the underlying floor.

The values of the loads, which consist of the floor planks, layer of wood flooring, the oak girder and snow and wind loads, were found in the Massachusetts Building Code which was available online at <http://www.mass.gov/bbrs/newcode.html>. The cementacious underlayment and the metal lath properties came from the Portland cement-based underlayment mix specification document provided by Northwest systems who were subcontracted by Consigli to place the underlayment. The elastic modulus of wood value was attained from the 2005 edition of the NDS supplement. The unshored construction live load was assumed to be twenty pounds per square foot. These loads are a result of small equipment and materials being stored and transported on the floors and were confirmed, verbally, by Consigli. The HVAC and roof decking loads were assumed with the aid of Consigli.

The deflection of the floor planking was not considered as a possible cause for the cracking because of the direction the cracks run through the building. Calculated deflections of the third and fourth floors can be seen in Tables 14-15

Table 14: Fourth Floor Deflection

Distance from Girder (ft)	Deflection (in)
2	0.278
4	0.3245
6	0.334
8	0.495
10	0.892
12	0.803
14	0.582
16	0.414
18	0.128

Table 15: Third Floor Deflection

Distance from Girder (ft)	Deflection (in)
2	0.135
4	3172
6	0.236
8	0.225
10	0.214
12	0.27
14	0.267
16	0.242
18	0.385

Some similar calculations were performed to determine if the floor planks deflected enough causing cracking and the results were negative. If the cracks had occurred parallel to the girders then the likely cause of the cracks would be the deflection. Since the cracks are running perpendicular to the girders it can be assumed the most likely cause is the deflection of the girders.

The first step in finding the deflection values for the concrete deck was to calculate the total load (dead + live) on the beam. The only dead load the concrete deck had was its own weight and the live loads consisted of the partition walls, which were being constructed at the time of cracking, and the construction loads.

Next the moment was found using the values for the total load. The moment of inertia of the deck was found using the standard equation of inertia for a rectangle. The centroid of the concrete deck was found by taking the distance from neutral axis to the most extreme fiber, being a rectangle, it is in the middle. Using the calculated moment half-way through the span of the beam, nine feet, inertia, and centroid of the bending stress of the beam was found using $^{MC}/_{I}$. The allowable compression stress of the concrete was provided by Consigli. To find the allowable tensile stress the equation $5 \rightarrow 7.5\sqrt{f'c}$ was used. 7.5 was used as the multiplier to keep the calculations conservative and allow for the maximum allowable load to be considered. If the bending stress of the concrete was greater than the allowable tensile stress, then the concrete would crack. The deflection value was also calculated halfway through the concrete deck using the E, I, w, moment, and length of the beam values.

Table 16: Concrete Bending and Deflection Values

Girder	Concrete
b (in)	96
h (in)	2
E (ksi)	4,250
w(plf)	460.00
M (lbft)	18,630.0
c (in)	1
I (in4)	64
σ (psi)	3,493.1
f'c (psi)	5,000.0
f't (psi)	530.3

Δ (in)	-3.994

The inertia and load values for oak girder were calculated similarly. The oak girder has more dead load values than the concrete deck. In addition to its own weight, the girder will have the floor planks, layer of wood flooring, metal lath, concrete and HVAC dead loads. From the wood design manual a conservative elastic modulus for the oak girder was chosen to be 1,200 kilo-pounds per square inch (kips) found in the boundary conditions specified in the NDS manual. The wood load was 45 pounds per cubic feet found in table G-8 of the CMR-780 Massachusetts building code. The live loads were the same as the concrete beam: partition and construction live loads. Using the obtained values of elastic modulus, total loads (w), length of the beam, and inertia the deflection of the oak girder can be calculated. Table 17 shows the calculated deflection values for both the concrete and oak.

Table 17: Deflection of Oak Girder

Girder	Oak
b (in)	10
h (in)	16
E (ksi)	1,200
w(plf)	1058.67
M (lbft)	42876
I (in4)	3,413
Δ (in)	-0.610

Minimum Bending Stress Capacity for the crack:

The deflection in the girder is needed to be at least equal to the deflection of the concrete at its minimum critical bending stress to cause cracking. The minimum critical bending stress at which cracking occurs can be calculated by using the deflection value of the girder oak. This can be used to calculate the total load needed to reach the critical bending stress value. The critical stress value must be greater than the tensile allowable stress value.

Minimum Bending Stress to Cause Enough Deflection for Cracking:

To find the minimum bending stress to crack the concrete, the moment was calculated by rearranging the bending equation to solve for the moment. Next the total load was determined. This allowed the deflection equation to be satisfied by finding the total deflection for the minimum bending stress to crack as -0.607 inches which is less than the deflection value for the oak girder.

Table 18: Deflection values

$\sigma_{minconcrete}$(psi)	531.0
M (lbft)	2,832.0
w (lb/ft)	69.9
$\Delta_{concrete}$ (in)	-0.60721
$\Delta_{concrete}$ (in) <	-0.61048

The results confirmed that the concrete would crack given the correct combination of live and dead loads. E values for the concrete was the only variable that was not treated as a conservative value. As a result an elastic modulus 4250 ksi or less was found to be capable of cracking under the given loading conditions.

8.1.3 Reinforced Concrete:

Moment and Deflection Calculations were run using the metal lath as a reinforcing material. Since the metal lath was not intended to be used as reinforcement for the concrete it seemed unlikely that it would aid in the prevention of cracks. According to a cylinder test conducted by an outside contractor for Consigli the concrete without wire mesh had a compression value of 5000 psi and the cylinder with the wire mesh had a compressive strength of 5300 psi. The amount of strength the reinforcement added was minimal and therefore was not considered to be essential to the prevention of cracks. The calculations were run by comparing the cracking moment in the concrete to the nominal moment strength. The cracking moment was determined by multiplying the inertia of the beam by the flexural cracking moment and dividing the product by its centroid. The flexural cracking moment is the square root of the compressive strength of the concrete multiplied by 7.5.

Table 19: Cracking Values

specified compressive strength of concrete-f'r (psi)	5,300.0
Distance to Centroid-yt (in.)	1.0
Inertia (in4)	64.0
Flexural Cracking Moment-fr (psi)	546.0
Cracking Moment-Cr (lb ft)	2,912.0

The nominal moment strength was calculated by multiplying the tension in the member by the difference between the height of the concrete with out the lath and half the thickness of the lath. The tension was found by multiplying the tensile strength of the lath by its area. The nominal strength moment needed to be reduced because of its ability to act as a reinforced structure so the nominal strength moment was multiplied by 0.9. The nominal strength moment then added to the cracking moment with out the metal lath giving the total moment.

Table 20: Nominal and Total Moment Values

fy (tensile) (ksi)	58000
A (in2)	0.247646
T (lb)	14363.47
f'c	5300
b (in)	96
a (in)	0.033212
d (in)	0.96875
Mn (inlb)	13676.09
piMn (lbft)	1025.707
total Moment	3,937.8

To determine if the moment in the beam is large enough to surpass the total moment the positive beam moment was calculated. The load used was the total weight of the concrete plus the office live load weight. The office live load was used because the fully reinforced concrete would not crack with the construction live load. To determine just how effective the metal lath is in preventing cracks the office live load was used to see if the concrete would crack once the occupants moved in. Using a length of one bay, eighteen feet, the positive moment came out to be 28,539 lb-ft. This value far exceeds the cracking moment of the concrete.

Table 21: Positive Moment Values

w (lb/ft)	704.7
L (ft)	18.0
Positive moment (lbft)	28,539.0

With the total moment known, the deflection of the concrete with the wire mesh was calculated. These were calculated the same way as the deflection was calculated with out the reinforcing. This led to a value that, under the same loading conditions as the un-reinforced concrete, would not crack. In order for the concrete to deflect past its maximum allowable deflection the construction load of 20 psi needed to be changed to an office load of 50 psi. This led to an E value of 4,700 ksi capable of cracking the concrete.

Table 22: Calculated Values plus the determined E value

Girder	Concrete	Oak
b (in)	96	10
h (in)	2	16
E (ksi)	4,700	1,200
w(plf)	700.00	1298.67
M (lbft)	28,350.0	52596
c (in)	1	8
I (in4)	64	3,413
σ (psi)	5,315.6	1,479.3
f'c (psi)	5,000.0	
f't (psi)	530.3	

Δ (in)	-5.497	-0.749

Table 23: Deflection of Reinforced Concrete

$\sigma_{minconcrete}$(psi)	531.0
M (lbft)	3,937.8
w (lb/ft)	97.2
$\Delta_{concrete}$ (in)	-0.76346
$\Delta_{concrete}$ (in) <	-0.74888

8.1.4 RISA-2D

RISA-2D was used to determine where the largest moments and deflections occurred in the building for both the oak girder and concrete deck. Fixed connections were used at all the exterior wall and foundation connections and pinned connections were used at all the interior column/girder connections. After an initial run through of the calculations RISA-2D determined that the concrete would not crack when subjected to the maximum moment value. As seen in Table 24 the critical concrete deflection is much greater than that of the oak girder meaning that the oak girder limits how much deflection occurs. The values for Table 24 where taken from Table 42: Deflection in Concrete - w/wind, Table 44: Member Forces in Concrete - w/ wind, Table 48: Deflection in Girder - w/ wind and Table 50: Member forces in Oak Girder - w/ wind.

Table 24: Concrete and Oak Deflection with Corresponding Moment Values

Concrete Deflection (in.)	-0.802	Concrete Moment (ksi)	6.216
Oak Deflection (in.)	-0.133	Oak Moment (ksi)	14.753

To determine if the concrete would crack when acting as a simply supported beam the maximum moment for the concrete was used to determine if the bending in the beam would exceed the maximum allowable tensile stress. Table 25 shows that the bending stress is far greater than the tensile stress thus cracking the concrete.

Table 25: Tensile Stress vs. Bending in Concrete Beam using Max. Moment from RISA-2D

w (lb/ft)	70.3
M (lbft)	6,216.0

$\sigma_{concrete}$(psi)	1,165.5
$\sigma_{concrete}$ (psi) >	530.3

Similar calculations were run at different beams across the structure. Table 25 refers to Member 23 which is in the top left corner of the building. Member 19 and member 21 were chosen as the members to be calculated. Table 27 shows that the concrete would crack.

Table 26: Member 21 Bending vs. Tensile Stress

w (lb/ft)	70.3
M (lbft)	6207.0
$\sigma_{concrete}$(psi)	1163.8
$\sigma_{concrete}$ (psi) >	530.3

Table 27: Member 19 Bending vs. Tensile Stress

w (lb/ft)	70.3
M (lbft)	5038.0
$\sigma_{concrete}$(psi)	944.6
$\sigma_{concrete}$ (psi) >	530.3

The RISA-2D calculations came out to be lower than anticipated. This could be to do the fact that all preliminary calculations were done assuming a simply supported beam. RISA-2D is takes in many more factors one of the most influential being that the ends of the beams are taken as fixed ends, opposed the hand calculations done for this paper, which assumed simply supported beams. The earthquake loads were not taken into account because between the times the concrete was placed until there was cracking no earthquakes occurred.

8.1.5 Vibrations Theory

Wind and Seismic vibrations can adversely affect a building. These vibrations, depending on their magnitude can affect people, crack structures and damage sensitive machines. In order to prevent vibrations, vibration engineers and other specialists can be hired to provide an adequate solution to the problem by devising an isolator. This typically involves installing a spring system in the foundation while the building is built. Another approach would be to place insulators on the floors or to use higher tensile structural steel to resist the materials tendency to crack. This approach is usually entailed towards solving vibration problems in terms of sensitive machines (Waller 10-44)

There will not be any sensitive lab equipment in 68 Prescott Street. Therefore the issues of having vibrations in the floor will only affect the people in the building. Since the vibrations coming from the passing traffic is low and the trains pass infrequently, it is assumed that the cracks in the floors were not caused by vibrations at all but rather through the way the materials acted. It must be noted that the vibrations in the floors are still noticeable, just not problematic.

Waller states that “vibration will have a direct effect on people’s health and ability to function efficiently while carrying out their normal activities at work” (Waller 16). The author is mainly dealing with major types of vibrations as indicated earlier; however the extent to which the floors will act and the noises it will cause each time an employee walks on the floor might become a main concern. It would be ideal to reduce the vibrations by using stiffer materials in the floor to reduce the vibration levels and improve the overall serviceability of the building.

8.1.6 Deflection Measurements

To support the theory that the deflection in the girders was causing cracking in the cement measurements needed to be taken to see how closely the calculated deflection came to the actual deflection. A level was placed at one column and then positioned until level. Using a caliper a reading was taken from the base of the level to the floor. The fourth floor deflected much more than expected being nearly an inch in the middle. Such a large deflection value might also be due to slight unconformities in the concrete because of the self leveling properties of the concrete. Since the concrete was self leveling no one needed to smooth the concrete out but this could lead to small buildups due to an uneven rate of curing. The third floor did not perform as expected. The deflections were much closer to the calculated deflections but there was a high spot in the floor where there was no visible build up of material. The second floor deflection was unable to be determined due to previous loading around the test area.

8.1.7 Testing

Table 28 is a comparison of the buildings values to the test specimen's values. Similar scaled comparisons are conducted for the slab and the wood planking and can be seen in Table 54.

Table 28: Three Point Bend Test Values

3POINT	Oak Girder	Douglas Fir Girder
Δ=PL^3/48EI	Building	Prototype
P (lb)	19056.06	1983.33
L (ft)	18	6
E (ksi)	1200	1300
b (in)	10	5.5
h (in)	16.00	5.5
I (in^4)	3413.33	76.26
Δ (in)	0.977	0.16
$\Delta_{allowable}$ (in)	0.9	0.3

Table 29: Stress Check on 3-point bend test

P	301.0
M (lbft)	225.8
σconcrete(psi)	450.2
σconcrete (psi) >	410.8

σminconcrete(psi)	410.8
M (lbft)	206.0
P (lb)	274.7
Δconcrete (in)	0.14
Δconcrete (in) <	0.16

The modulus of elasticity increased because lumber yard only had No. 2 Grade Douglas-Fir in stock.

Table 30: 4-Point Bend Test

4 POINT	Oak Girder	Douglas Fir Girder
$\Delta=11PL^3/768EI$	Building	Prototype
P (lb)	19056.06	1900.00
L (ft)	18.00	4.83
E (ksi)	1200	1600
b (in)	10	3.5
h (in)	16	5.5
I (in^4)	3413.3	48.5
Δ (in)	0.67	0.07
$\Delta_{allowable}$ (in)	0.9	0.242

Table 31 shows that the bending stress and the deflection are consistent with the hypothesis. Similar calculations are performed for the concrete slab and wood flooring and can be found in the appendix in Table 59.

Table 31: 4-Point Bend Test Check

P	294.4
M (lbft)	177.8
σconcrete(psi)	479.0
σconcrete (psi) >	443.7
σminconcrete(psi)	443.7
M (lbft)	164.7
P (lb)	272.7
Δconcrete (in)	0.06
Δconcrete (in) <	0.07
σdouglesfirgirder(psi)	875.0
M (lbft)	1286.7
P (lb)	2129.7

Table 32: Internal Shear Calculations

E_{wood}	2500	(ksi)
$E_{conc.}$	3990	(ksi)
E_{ply}	1800	(ksi)
h_{wood}	5.5	(in)
$h_{conc.}$	1.8	(in)
h_{ply}	1.0	(in)
h_{total}	8.3	(in)
b_{wood}	3.5	(in)
b_{conc}	8.25	(in)
b_{ply}	8.25	(in)
L	54	(in)
V	8000	(lb)
$n_{w/c}$	1.60	
adjusted $b_{conc.}$	13.2	(in)
$n_{p/w}$	0.72	
adjusted b_{ply}	5.94	
y_{bar}	5.3990	(in)
I_{NA}	287.5	(in4)
Q	47.4	(in3)
τ_{max}	377.0	(psi)

Table 33: Nail Shear Values

Single Shear Yeild Limit	
Mode Im	995.050
Mode Is	770.746
Mode II	392.788
Mode IIIm	389.264
Mode IIIs	223.779
Mode IV psi	162.682

Table 34: Composite Calculations

E_w	2.5E+06	(psi)
E_c	4.0E+06	(psi)
h_w	5.5	(in)
h_c	1.8	(in)

b_w	3.5	(in)
b_c	8.3	(in)
L	55.0	(in)
K	7502.0	(psi/in)
spacing	3.0	(in)
M	55000.0	(lb-in)
P	8000	(lb)
a	13.75	(in)

I_w	48.5	
I_c	4.0	(in4)

γ_1	0.013
γ_2	1.0

a_1	3.0
a_2	0.07

$(EI)_{ef}$	1.4E+08	Effective EI
Δ	0.2668	Deflection in Beam (in)
$\sigma_{w,t}$	2683.21	Max. Tensile stress - wood (psi)
$\sigma_{c,C}$	1440.76	Max. Compresive Stress - Concrete (psi)
σc,T	-1323.107	Max. Tensile stress - Concrete (psi)
q	63.5	Max Beam Shear Stress
fv	1045.6	Shear Flow in Connector

Checks were done to determine if the hypothesis still holds true for this test. The actual bending in the concrete exceeds the allowable bending and therefore should crack. Also, with the resultant bending, the deflection of the concrete does not exceed the deflection of the girder meaning that the girder controls the amount of deflection in the concrete. This check is consistent with the hypothesis.

8.2 PRELIMINARY CALCULATIONS

Table 35: Dimensions of Girders

L (ft)	18
trib width (ft)	8

Table 36: Types of Loading and Respective Values on un-reinforced concrete

Type of loading	Loads	Sources
$w_{concrete}$ (pcf)	105	Cementatious underlayment Doc.
w_{plank} (psf)	48	Southern Yellow Pine CMR 780 Table G-8
$w_{layerofwood}$ (psf)	15	Maple Hardwood CMR 780 Table G-4
w_{girder} (pcf)	45	Oak CMR 780 Table G-8
$w_{metallath}$ (psf)	0.58	Cementatious underlayment Doc.
w_{hvac} (psf)	5	Assumed
$w_{partition}$ (psf)	20	CMR1605.3 (zone 3)
$w_{construction}$ (psf)	20	Consigli assumed equipment load
w_{snow} (psf)	35	CMR 1610.2
$w_{roof\ deck}$ (psf)	40	Assumed
w_{wind} (psf)	24	CMR Table 1611.4 (zone 2 exp. B)

Table 37: Concrete Acting as a Girder

Loads (klf)	Floor 2	Floor 3	Floor 4	Roof
w_{dead}	0.053	0.140	0.140	0.840
w_{live}	0.320	0.320	0.320	
w_{snow}				0.280
$w_{total(vertical)}$	0.373	0.460	0.460	1.120
w_{wind} (kips)	2.616	2.632	2.744	1.432

Table 38: Wood Acting as a Girder

Loads (klf)	Floor 2	Floor 3	Floor 4	Roof
w_{dead}	0.651	0.739	0.739	0.840
w_{live}	0.320	0.320	0.320	
w_{snow}				0.280
$w_{total(vertical)}$	0.971	1.059	1.059	1.120
w_{wind}	2.616	2.632	2.744	1.432

Table 39: Joint Reactions in Concrete - no wind

No Wind			
Joint Label	X (k)	Y (k)	MZ (k-ft)
A	0.415	18.146	-1.954

B	0	0	0
C	0	0	0
D	-0.415	18.146	1.954
F	0.648	6.715	0
G	-0.648	6.715	0
J	-0.446	8.282	0
K	0.446	8.282	0
N	0.969	8.283	0
O	-0.969	8.283	0
R	-1.587	13.843	0
S	1.587	13.843	0
Totals:	0	110.538	0

Table 40: Joint Reaction in Concrete - w/ wind

Wind			
Joint Label	X (k)	Y (k)	MZ (k-ft)
A	0.333	18.144	-1.255
B	0	0	0
C	0	0	0
D	-0.415	18.146	1.954
F	-1.853	6.715	0
G	-0.648	6.715	0
J	-3.108	8.282	0
K	0.446	8.282	0
N	-1.782	8.283	0
O	-0.97	8.283	0
R	-3.013	13.846	0
S	1.587	13.843	0
Totals:	-9.424	110.538	0.7

Table 41: Deflection in Concrete - no wind

No Wind			
Member Label	Sec	x(in)	y(in)
M1	1	0	0
	2	0	0
	3	0	0
	4	-0.001	0
	5	-0.002	0
M2	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0
M3	1	0	0
	2	0	0
	3	0	0
	4	0	0

	5	0	0
M4	1	0	0
	2	0	0
	3	0	0
	4	-0.001	0
	5	-0.002	0
M5	1	-0.002	0
	2	-0.002	0
	3	-0.002	0
	4	-0.002	0
	5	-0.003	0
M6	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0
M7	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0
M8	1	-0.002	0
	2	-0.002	0
	3	-0.002	0
	4	-0.002	0
	5	-0.003	0
M9	1	-0.003	0
	2	-0.003	0
	3	-0.003	0
	4	-0.004	0
	5	-0.004	0
M10	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0
M11	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0
M12	1	-0.003	0
	2	-0.003	0
	3	-0.003	0
	4	-0.004	0
	5	-0.004	0
M13	1	-0.004	0

	2	-0.004	0.005
	3	-0.005	0.015
	4	-0.005	0.017
	5	-0.005	-0.002
M14	1	0	0
	2	0	-0.003
	3	0	-0.006
	4	0	-0.007
	5	0	0
M15	1	0	0
	2	0	0.003
	3	0	0.006
	4	0	0.007
	5	0	0
M16	1	-0.004	0
	2	-0.004	-0.005
	3	-0.005	-0.015
	4	-0.005	-0.017
	5	-0.005	0.002
M17	1	0	-0.002
	2	0	-0.366
	3	0	-0.649
	4	0	-0.365
	5	0	0
M18	1	0	0
	2	0	-0.364
	3	0	-0.648
	4	0	-0.364
	5	0	0
M19	1	0	0
	2	0	-0.365
	3	0	-0.649
	4	0	-0.366
	5	0	-0.002
M20	1	0	-0.003
	2	0	-0.453
	3	0	-0.802
	4	0	-0.451
	5	0	0
M21	1	0	0
	2	0	-0.449
	3	0	-0.798
	4	0	-0.449
	5	0	0
M22	1	0	0
	2	0	-0.451
	3	0	-0.802

	4	0	-0.453
	5	0	-0.003
M23	1	0	-0.004
	2	0	-0.454
	3	0	-0.801
	4	0	-0.45
	5	0	0
M24	1	0	0
	2	0	-0.45
	3	0	-0.8
	4	0	-0.45
	5	0	0
M25	1	0	0
	2	0	-0.45
	3	0	-0.801
	4	0	-0.454
	5	0	-0.004
M26	1	0.002	-0.005
	2	0.001	-0.078
	3	0	-0.116
	4	0	-0.065
	5	0	0
M27	1	0	0
	2	0	-0.039
	3	0	-0.073
	4	0	-0.039
	5	0	0
M28	1	0	0
	2	0	-0.065
	3	0	-0.116
	4	-0.001	-0.078
	5	-0.002	-0.005

Table 42: Deflection in Concrete - w/wind

Wind			
Member Label	Sec	x(in)	y(in)
M1	1	0	0
	2	0	0
	3	0	0
	4	-0.001	0
	5	-0.002	0.00E+00
M2	1	0	0.00E+00
	2	0	0.00E+00
	3	0	0.00E+00
	4	0	0.00E+00
	5	0	0.00E+00
M3	1	0	0.00E+00

	2	0	0.00E+00
	3	0	0.00E+00
	4	0	0.00E+00
	5	0	0.00E+00
M4	1	0	0.00E+00
	2	0	0.00E+00
	3	0	0.00E+00
	4	-0.001	0.00E+00
	5	-0.002	0.00E+00
M5	1	-0.002	0
	2	-0.002	0
	3	-0.002	0
	4	-0.002	0
	5	-0.003	0
M6	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0
M7	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0
M8	1	-0.002	0
	2	-0.002	0
	3	-0.002	0
	4	-0.002	0
	5	-0.003	0
M9	1	-0.003	0
	2	-0.003	-0.001
	3	-0.003	0
	4	-0.004	0
	5	-0.004	0
M10	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0
M11	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0
M12	1	-0.003	0
	2	-0.003	0
	3	-0.003	0

	4	-0.004	0
	5	-0.004	0
M13	1	-0.004	0
	2	-0.004	0.005
	3	-0.005	0.014
	4	-0.005	0.016
	5	-0.005	-0.003
M14	1	0	0
	2	0	-0.003
	3	0	-0.006
	4	0	-0.007
	5	0	0
M15	1	0	0
	2	0	0.003
	3	0	0.006
	4	0	0.007
	5	0	0
M16	1	-0.004	0
	2	-0.004	-0.005
	3	-0.005	-0.015
	4	-0.005	-0.017
	5	-0.005	0.002
M17	1	0	-0.002
	2	0	-0.366
	3	0	-0.649
	4	0	-0.365
	5	0	0
M18	1	0	0
	2	0	-0.364
	3	0	-0.648
	4	0	-0.364
	5	0	0
M19	1	0	0
	2	0	-0.365
	3	0	-0.649
	4	0	-0.366
	5	0	-0.002
M20	1	0	-0.003
	2	0	-0.453
	3	0	-0.802
	4	0	-0.451
	5	0	0
M21	1	0	0
	2	0	-0.449
	3	0	-0.798
	4	0	-0.449
	5	0	0

M22	1	0	0
	2	0	-0.451
	3	0	-0.802
	4	0	-0.453
	5	0	-0.003
M23	1	0	-0.004
	2	0	-0.454
	3	0	-0.801
	4	0	-0.45
	5	0	0
M24	1	0	0
	2	0	-0.45
	3	0	-0.8
	4	0	-0.45
	5	0	0
M25	1	0	0
	2	0	-0.45
	3	0	-0.801
	4	0	-0.454
	5	0	-0.004
M26	1	0.003	-0.005
	2	0.003	-0.078
	3	0.002	-0.116
	4	0	-0.065
	5	0	0
M27	1	0	0
	2	0	-0.038
	3	0	-0.073
	4	0	-0.038
	5	0	0
M28	1	0	0
	2	0	-0.065
	3	0	-0.116
	4	-0.001	-0.078
	5	-0.002	-0.005

Table 43: Member Forces in Concrete – no wind

No Wind				
Member Label	Sec	Axial (k)	Shear (k)	Moment (kft)
M1	1	18.146	-0.415	1.954
	2	18.146	-0.415	0.553
	3	18.146	-0.415	-0.849
	4	18.146	-0.415	-2.251
	5	18.146	-0.415	-3.652
M2	1	0	0	0
	2	0	0	0
	3	0	0	0

	4	0	0	0.001
	5	0	0	0.002
M3	1	0	0	0
	2	0	0	0
	3	0	0	0
	4	0	0	-0.001
	5	0	0	-0.002
M4	1	18.146	0.415	-1.954
	2	18.146	0.415	-0.553
	3	18.146	0.415	0.849
	4	18.146	0.415	2.251
	5	18.146	0.415	3.652
M5	1	14.79	-1.064	6.406
	2	14.79	-1.064	2.748
	3	14.79	-1.064	-0.911
	4	14.79	-1.064	-4.569
	5	14.79	-1.064	-8.228
M6	1	0	0	-0.004
	2	0	0	-0.002
	3	0	0	0
	4	0	0	0.004
	5	0	0	0.006
M7	1	0	0	0.004
	2	0	0	0.002
	3	0	0	0
	4	0	0	-0.004
	5	0	0	-0.006
M8	1	14.79	1.064	-6.406
	2	14.79	1.064	-2.748
	3	14.79	1.064	0.911
	4	14.79	1.064	4.569
	5	14.79	1.064	8.228
M9	1	10.652	-0.617	4.168
	2	10.652	-0.617	2.17
	3	10.652	-0.617	0.171
	4	10.652	-0.617	-1.827
	5	10.652	-0.617	-3.826
M10	1	0	0	-0.002
	2	0	0	-0.003
	3	0	0	-0.003
	4	0	0	-0.004
	5	0	0	-0.004
M11	1	0	0	0.002
	2	0	0	0.003
	3	0	0	0.003
	4	0	0	0.004
	5	0	0	0.004

M12	1	10.652	0.617	-4.168
	2	10.652	0.617	-2.17
	3	10.652	0.617	-0.171
	4	10.652	0.617	1.827
	5	10.652	0.617	3.826
M13	1	6.515	-1.594	8.565
	2	6.515	-1.594	2.332
	3	6.515	-1.594	-3.9
	4	6.515	-1.594	-10.132
	5	6.515	-1.594	-16.364
M14	1	0	0.007	-0.035
	2	0	0.007	-0.006
	3	0	0.007	0.022
	4	0	0.007	0.05
	5	0	0.007	0.079
M15	1	0	-0.007	0.035
	2	0	-0.007	0.006
	3	0	-0.007	-0.022
	4	0	-0.007	-0.05
	5	0	-0.007	-0.079
M16	1	6.515	1.594	-8.565
	2	6.515	1.594	-2.332
	3	6.515	1.594	3.9
	4	6.515	1.594	10.132
	5	6.515	1.594	16.364
M17	1	-0.649	3.356	-10.059
	2	-0.649	1.677	1.266
	3	-0.649	-0.001	5.038
	4	-0.649	-1.68	1.257
	5	-0.649	-3.358	-10.078
M18	1	0	3.357	-10.072
	2	0	1.678	1.258
	3	0	0	5.035
	4	0	-1.678	1.258
	5	0	-3.357	-10.072
M19	1	-0.649	3.358	-10.078
	2	-0.649	1.68	1.257
	3	-0.649	0.001	5.038
	4	-0.649	-1.677	1.266
	5	-0.649	-3.356	-10.059
M20	1	0.447	4.138	-12.396
	2	0.447	2.068	1.567
	3	0.447	-0.002	6.216
	4	0.447	-2.072	1.55
	5	0.447	-4.142	-12.431
M21	1	0	4.14	-12.423
	2	0	2.07	1.549

	3	0	0	6.207
	4	0	-2.07	1.549
	5	0	-4.14	-12.423
M22	1	0.447	4.142	-12.431
	2	0.447	2.072	1.55
	3	0.447	0.002	6.216
	4	0.447	-2.068	1.567
	5	0.447	-4.138	-12.396
M23	1	-0.977	4.137	-12.39
	2	-0.977	2.067	1.568
	3	-0.977	-0.003	6.212
	4	-0.977	-2.073	1.54
	5	-0.977	-4.143	-12.446
M24	1	0	4.14	-12.416
	2	0	2.07	1.556
	3	0	0	6.214
	4	0	-2.07	1.556
	5	0	-4.14	-12.416
M25	1	-0.977	4.143	-12.446
	2	-0.977	2.073	1.54
	3	-0.977	0.003	6.212
	4	-0.977	-2.067	1.568
	5	-0.977	-4.137	-12.39
M26	1	1.594	6.515	-16.364
	2	1.594	3.122	5.317
	3	1.594	-0.271	11.73
	4	1.594	-3.664	2.875
	5	1.594	-7.057	-21.249
M27	1	0	6.786	-21.17
	2	0	3.393	1.733
	3	0	0	9.367
	4	0	-3.393	1.733
	5	0	-6.786	-21.17
M28	1	1.594	7.057	-21.249
	2	1.594	3.664	2.875
	3	1.594	0.271	11.73
	4	1.594	-3.122	5.317
	5	1.594	-6.515	-16.364

Table 44: Member Forces in Concrete - w/ wind

Wind				
Member Label	Sec	Axial (k)	Shear (k)	Moment (kft)
M1	1	18.144	-0.333	1.255
	2	18.144	-0.333	0.131
	3	18.144	-0.333	-0.993
	4	18.144	-0.333	-2.117
	5	18.144	-0.333	-3.241

M2	1	0	0	0
	2	0	0	0
	3	0	0	0
	4	0	0	0.001
	5	0	0	0.002
M3	1	0	0	0
	2	0	0	0
	3	0	0	0
	4	0	0	-0.001
	5	0	0	-0.002
M4	1	18.146	0.415	-1.954
	2	18.146	0.415	-0.553
	3	18.146	0.415	0.849
	4	18.146	0.415	2.251
	5	18.146	0.415	3.652
M5	1	14.788	-1.097	6.817
	2	14.788	-1.097	3.047
	3	14.788	-1.097	-0.723
	4	14.788	-1.097	-4.493
	5	14.788	-1.097	-8.262
M6	1	0	0	-0.004
	2	0	0	-0.002
	3	0	0	0
	4	0	0	0.003
	5	0	0	0.006
M7	1	0	0	0.004
	2	0	0	0.002
	3	0	0	0
	4	0	0	-0.004
	5	0	0	-0.006
M8	1	14.79	1.064	-6.406
	2	14.79	1.064	-2.748
	3	14.79	1.064	0.911
	4	14.79	1.064	4.569
	5	14.79	1.064	8.228
M9	1	10.65	-0.62	4.135
	2	10.65	-0.62	2.129
	3	10.65	-0.62	0.123
	4	10.65	-0.62	-1.882
	5	10.65	-0.62	-3.888
M10	1	0	0	-0.002
	2	0	0	-0.003
	3	0	0	-0.003
	4	0	0	-0.004
	5	0	0	-0.004
M11	1	0	0	0.002
	2	0	0	0.003

	3	0	0	0.003
	4	0	0	0.004
	5	0	0	0.004
M12	1	10.652	0.617	-4.168
	2	10.652	0.617	-2.17
	3	10.652	0.617	-0.171
	4	10.652	0.617	1.827
	5	10.652	0.617	3.826
M13	1	6.513	-1.589	8.502
	2	6.513	-1.589	2.292
	3	6.513	-1.589	-3.918
	4	6.513	-1.589	-10.128
	5	6.513	-1.589	-16.337
M14	1	0	0.007	-0.035
	2	0	0.007	-0.007
	3	0	0.007	0.022
	4	0	0.007	0.05
	5	0	0.007	0.079
M15	1	0	-0.007	0.035
	2	0	-0.007	0.006
	3	0	-0.007	-0.022
	4	0	-0.007	-0.05
	5	0	-0.007	-0.079
M16	1	6.515	1.594	-8.565
	2	6.515	1.594	-2.332
	3	6.515	1.594	3.9
	4	6.515	1.594	10.133
	5	6.515	1.594	16.365
M17	1	1.852	3.356	-10.057
	2	1.852	1.677	1.267
	3	1.852	-0.001	5.039
	4	1.852	-1.68	1.257
	5	1.852	-3.358	-10.078
M18	1	0	3.357	-10.072
	2	0	1.679	1.258
	3	0	0	5.034
	4	0	-1.678	1.258
	5	0	-3.357	-10.072
M19	1	-0.649	3.358	-10.078
	2	-0.649	1.68	1.257
	3	-0.649	0.001	5.038
	4	-0.649	-1.677	1.266
	5	-0.649	-3.356	-10.059
M20	1	3.109	4.138	-12.397
	2	3.109	2.068	1.567
	3	3.109	-0.002	6.216
	4	3.109	-2.072	1.55

	5	3.109	-4.142	-12.431
M21	1	0	4.14	-12.423
	2	0	2.07	1.549
	3	0	0	6.207
	4	0	-2.07	1.549
	5	0	-4.14	-12.423
M22	1	0.447	4.142	-12.431
	2	0.447	2.072	1.55
	3	0.447	0.002	6.216
	4	0.447	-2.068	1.567
	5	0.447	-4.138	-12.396
M23	1	1.775	4.137	-12.39
	2	1.775	2.067	1.568
	3	1.775	-0.003	6.212
	4	1.775	-2.073	1.54
	5	1.775	-4.143	-12.447
M24	1	0	4.14	-12.416
	2	0	2.07	1.556
	3	0	0	6.214
	4	0	-2.07	1.556
	5	0	-4.14	-12.416
M25	1	-0.977	4.143	-12.446
	2	-0.977	2.073	1.54
	3	-0.977	0.003	6.212
	4	-0.977	-2.067	1.568
	5	-0.977	-4.137	-12.39
M26	1	3.021	6.513	-16.337
	2	3.021	3.12	5.336
	3	3.021	-0.273	11.74
	4	3.021	-3.666	2.876
	5	3.021	-7.059	-21.256
M27	1	0	6.787	-21.177
	2	0	3.394	1.728
	3	0	0	9.364
	4	0	-3.392	1.732
	5	0	-6.785	-21.168
M28	1	1.594	7.057	-21.247
	2	1.594	3.664	2.876
	3	1.594	0.271	11.731
	4	1.594	-3.122	5.317
	5	1.594	-6.515	-16.365

Table 45: Joint Reactions in Girder - no wind

No Wind			
Joint Label	X (k)	Y (k)	MZ (k-ft)
A	1.315	34.166	-6.431
B	-0.001	0	0.006

C	0.001	0	-0.006
D	-1.315	34.166	6.431
F	0.916	17.508	0
G	-0.916	17.508	0
J	-0.116	19.106	0
K	0.116	19.106	0
N	-0.314	19.156	0
O	0.314	19.156	0
R	-1.799	13.825	0
S	1.799	13.825	0
Totals:	0	207.522	0

Table 46: Joint Reactions in Girder - w/ wind

Wind			
Joint Label	X (k)	Y (k)	MZ (k-ft)
A	1.021	34.165	-3.868
B	-0.001	0	0.006
C	0.001	0	-0.006
D	-1.315	34.166	6.432
F	-1.302	17.515	0
G	-0.916	17.506	0
J	-2.869	19.106	0
K	0.116	19.107	0
N	-3.031	19.155	0
O	0.314	19.156	0
R	-3.242	13.821	0
S	1.799	13.826	0
Totals:	-9.424	207.522	2.564

Table 47: Deflection in Girder - no wind

No Wind			
Member Label	Sec	x(in)	y(in)
M1	1	0	0
	2	0	0
	3	-0.001	0.002
	4	-0.002	0.002
	5	-0.003	0.001
M2	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0
M3	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0

M4	1	0	0
	2	0	0
	3	-0.001	-0.002
	4	-0.002	-0.002
	5	-0.003	-0.001
M5	1	-0.003	0.001
	2	-0.003	0
	3	-0.004	0
	4	-0.004	0.002
	5	-0.005	0
M6	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0
M7	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0
M8	1	-0.003	-0.001
	2	-0.003	0
	3	-0.004	0
	4	-0.004	-0.002
	5	-0.005	0
M9	1	-0.005	0
	2	-0.005	0
	3	-0.006	0.002
	4	-0.006	0.004
	5	-0.007	0
M10	1	0	0
	2	0	0
	3	0	0
	4	0	-0.002
	5	0	0
M11	1	0	0
	2	0	0
	3	0	0
	4	0	0.002
	5	0	0
M12	1	-0.005	0
	2	-0.005	0
	3	-0.006	-0.002
	4	-0.006	-0.004
	5	-0.007	0
M13	1	-0.007	0
	2	-0.007	0

	3	-0.007	0.009
	4	-0.008	0.014
	5	-0.008	-0.002
M14	1	0	0
	2	0	0
	3	0	-0.004
	4	0	-0.006
	5	0	0
M15	1	0	0
	2	0	0
	3	0	0.004
	4	0	0.006
	5	0	0
M16	1	-0.007	0
	2	-0.007	0
	3	-0.007	-0.009
	4	-0.008	-0.014
	5	-0.008	0.002
M17	1	-0.001	-0.003
	2	0	-0.068
	3	0	-0.116
	4	0	-0.065
	5	0	0
M18	1	0	0
	2	0	-0.062
	3	0	-0.11
	4	0	-0.062
	5	0	0
M19	1	0	0
	2	0	-0.065
	3	0	-0.116
	4	0	-0.068
	5	0.001	-0.003
M20	1	0	-0.005
	2	0	-0.076
	3	0	-0.128
	4	0	-0.072
	5	0	0
M21	1	0	0
	2	0	-0.067
	3	0	-0.12
	4	0	-0.067
	5	0	0
M22	1	0	0
	2	0	-0.072
	3	0	-0.128
	4	0	-0.076

	5	0	-0.005
M23	1	0	-0.007
	2	0	-0.081
	3	0	-0.133
	4	0	-0.074
	5	0	0
M24	1	0	0
	2	0	-0.065
	3	0	-0.117
	4	0	-0.065
	5	0	0
M25	1	0	0
	2	0	-0.074
	3	0	-0.133
	4	0	-0.081
	5	0	-0.007
M26	1	0.002	-0.008
	2	0.002	-0.078
	3	0.001	-0.115
	4	0	-0.064
	5	0	0
M27	1	0	0
	2	0	-0.039
	3	0	-0.074
	4	0	-0.039
	5	0	0
M28	1	0	0
	2	0	-0.064
	3	-0.001	-0.115
	4	-0.002	-0.078
	5	-0.002	-0.008

Table 48: Deflection in Girder - w/ wind

Wind			
Member Label	Sec	x(in)	y(in)
M1	1	0	0
	2	0	0
	3	-0.001	0
	4	-0.002	0
	5	-0.003	-0.001
M2	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0
M3	1	0	0
	2	0	0

	3	0	0
	4	0	0
	5	0	0
M4	1	0	0
	2	0	0
	3	-0.001	-0.002
	4	-0.002	-0.002
	5	-0.003	-0.001
M5	1	-0.003	-0.001
	2	-0.003	-0.003
	3	-0.004	-0.002
	4	-0.004	-0.002
	5	-0.005	-0.003
M6	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0
M7	1	0	0
	2	0	0
	3	0	0
	4	0	0
	5	0	0
M8	1	-0.003	-0.001
	2	-0.003	0
	3	-0.004	0
	4	-0.004	-0.002
	5	-0.005	0
M9	1	-0.005	-0.003
	2	-0.005	-0.004
	3	-0.006	0
	4	-0.006	0
	5	-0.007	-0.003
M10	1	0	0
	2	0	0
	3	0	0
	4	0	-0.002
	5	0	0
M11	1	0	0
	2	0	0
	3	0	0
	4	0	0.002
	5	0	0
M12	1	-0.005	0
	2	-0.005	0
	3	-0.006	-0.002
	4	-0.006	-0.004

	5	-0.007	0
M13	1	-0.007	-0.003
	2	-0.007	-0.003
	3	-0.007	0.007
	4	-0.008	0.012
	5	-0.008	-0.004
M14	1	0	0
	2	0	0
	3	0	-0.004
	4	0	-0.006
	5	0	0
M15	1	0	0
	2	0	0
	3	0	0.004
	4	0	0.006
	5	0	0
M16	1	-0.007	0
	2	-0.007	0
	3	-0.007	-0.009
	4	-0.008	-0.014
	5	-0.008	0.002
M17	1	0.001	-0.003
	2	0.001	-0.068
	3	0	-0.116
	4	0	-0.065
	5	0	0
M18	1	0	0
	2	0	-0.062
	3	0	-0.11
	4	0	-0.062
	5	0	0
M19	1	0	0
	2	0	-0.065
	3	0	-0.116
	4	0	-0.068
	5	0.001	-0.003
M20	1	0.003	-0.005
	2	0.002	-0.076
	3	0.002	-0.128
	4	0	-0.072
	5	0	0
M21	1	0	0
	2	0	-0.067
	3	0	-0.12
	4	0	-0.067
	5	0	0
M22	1	0	0

	2	0	-0.072
	3	0	-0.128
	4	0	-0.076
	5	0	-0.005
M23	1	0.003	-0.007
	2	0.003	-0.081
	3	0.002	-0.133
	4	0	-0.074
	5	0	0
M24	1	0	0
	2	0	-0.065
	3	0	-0.117
	4	0	-0.065
	5	0	0
M25	1	0	0
	2	0	-0.074
	3	0	-0.133
	4	0	-0.081
	5	0	-0.007
M26	1	0.004	-0.008
	2	0.003	-0.078
	3	0.002	-0.115
	4	0	-0.064
	5	0	0
M27	1	0	0
	2	0	-0.039
	3	0	-0.074
	4	0	-0.039
	5	0	0
M28	1	0	0
	2	0	-0.064
	3	-0.001	-0.115
	4	-0.002	-0.078
	5	-0.002	-0.008

Table 49: Member Forces in Girder - no wind

No Wind				
Member Label	Sec	Axial (k)	Shear (k)	Moment (kft)
M1	1	34.166	-1.315	6.431
	2	34.166	-1.315	1.993
	3	34.166	-1.315	-2.446
	4	34.166	-1.315	-6.884
	5	34.166	-1.315	-11.323
M2	1	0	0.001	-0.006
	2	0	0.001	-0.001
	3	0	0.001	0.003
	4	0	0.001	0.007

	5	0	0.001	0.011
M3	1	0	-0.001	0.006
	2	0	-0.001	0.001
	3	0	-0.001	-0.003
	4	0	-0.001	-0.007
	5	0	-0.001	-0.011
M4	1	34.166	1.315	-6.431
	2	34.166	1.315	-1.993
	3	34.166	1.315	2.446
	4	34.166	1.315	6.884
	5	34.166	1.315	11.323
M5	1	25.457	-2.233	14.48
	2	25.457	-2.233	6.806
	3	25.457	-2.233	-0.869
	4	25.457	-2.233	-8.544
	5	25.457	-2.233	-16.219
M6	1	0	0.003	-0.019
	2	0	0.003	-0.009
	3	0	0.003	0.001
	4	0	0.003	0.011
	5	0	0.003	0.022
M7	1	0	-0.003	0.019
	2	0	-0.003	0.009
	3	0	-0.003	-0.001
	4	0	-0.003	-0.011
	5	0	-0.003	-0.022
M8	1	25.457	2.233	-14.48
	2	25.457	2.233	-6.806
	3	25.457	2.233	0.869
	4	25.457	2.233	8.544
	5	25.457	2.233	16.219
M9	1	15.97	-2.119	11.773
	2	15.97	-2.119	4.913
	3	15.97	-2.119	-1.946
	4	15.97	-2.119	-8.806
	5	15.97	-2.119	-15.665
M10	1	0	0.006	-0.034
	2	0	0.006	-0.015
	3	0	0.006	0.004
	4	0	0.006	0.023
	5	0	0.006	0.042
M11	1	0	-0.006	0.034
	2	0	-0.006	0.015
	3	0	-0.006	-0.004
	4	0	-0.006	-0.023
	5	0	-0.006	-0.042
M12	1	15.97	2.119	-11.773

	2	15.97	2.119	-4.913
	3	15.97	2.119	1.946
	4	15.97	2.119	8.806
	5	15.97	2.119	15.665
M13	1	6.533	-1.809	11.622
	2	6.533	-1.809	4.55
	3	6.533	-1.809	-2.523
	4	6.533	-1.809	-9.596
	5	6.533	-1.809	-16.669
M14	1	0	0.01	-0.066
	2	0	0.01	-0.027
	3	0	0.01	0.012
	4	0	0.01	0.052
	5	0	0.01	0.091
M15	1	0	-0.01	0.066
	2	0	-0.01	0.027
	3	0	-0.01	-0.012
	4	0	-0.01	-0.052
	5	0	-0.01	-0.091
M16	1	6.533	1.809	-11.622
	2	6.533	1.809	-4.55
	3	6.533	1.809	2.523
	4	6.533	1.809	9.596
	5	6.533	1.809	16.669
M17	1	-0.918	8.709	-25.803
	2	-0.918	4.34	3.556
	3	-0.918	-0.03	13.252
	4	-0.918	-4.399	3.286
	5	-0.918	-8.769	-26.343
M18	1	0	8.739	-26.313
	2	0	4.369	3.181
	3	0	0	13.013
	4	0	-4.37	3.181
	5	0	-8.739	-26.313
M19	1	-0.918	8.769	-26.343
	2	-0.918	4.399	3.286
	3	-0.918	0.03	13.252
	4	-0.918	-4.34	3.556
	5	-0.918	-8.709	-25.803
M20	1	0.113	9.487	-27.992
	2	0.113	4.721	3.976
	3	0.113	-0.044	14.498
	4	0.113	-4.81	3.576
	5	0.113	-9.575	-28.791
M21	1	0	9.531	-28.735
	2	0	4.765	3.432
	3	0	0	14.154

	4	0	-4.766	3.432
	5	0	-9.531	-28.735
M22	1	0.113	9.575	-28.791
	2	0.113	4.81	3.576
	3	0.113	0.044	14.498
	4	0.113	-4.721	3.976
	5	0.113	-9.487	-27.992
M23	1	0.31	9.437	-27.288
	2	0.31	4.672	4.457
	3	0.31	-0.094	14.756
	4	0.31	-4.859	3.611
	5	0.31	-9.625	-28.979
M24	1	0	9.531	-28.87
	2	0	4.765	3.297
	3	0	0	14.019
	4	0	-4.766	3.297
	5	0	-9.531	-28.87
M25	1	0.31	9.625	-28.979
	2	0.31	4.859	3.611
	3	0.31	0.094	14.756
	4	0.31	-4.672	4.457
	5	0.31	-9.437	-27.288
M26	1	1.809	6.533	-16.669
	2	1.809	3.14	5.096
	3	1.809	-0.253	11.592
	4	1.809	-3.646	2.82
	5	1.809	-7.039	-21.22
M27	1	0	6.786	-21.129
	2	0	3.393	1.774
	3	0	0	9.408
	4	0	-3.393	1.774
	5	0	-6.786	-21.129
M28	1	1.809	7.039	-21.22
	2	1.809	3.646	2.82
	3	1.809	0.253	11.592
	4	1.809	-3.14	5.096
	5	1.809	-6.533	-16.669

Table 50: Member forces in Oak Girder - w/ wind

Wind				
Member Label	Sec	Axial (k)	Shear (k)	Moment (kft)
M1	1	34.165	-1.021	3.868
	2	34.165	-1.021	0.422
	3	34.165	-1.021	-3.025
	4	34.165	-1.021	-6.472
	5	34.165	-1.021	-9.918
M2	1	0	0.001	-0.006

	2	0	0.001	-0.002
	3	0	0.001	0.003
	4	0	0.001	0.008
	5	0	0.001	0.012
M3	1	0	-0.001	0.006
	2	0	-0.001	0.001
	3	0	-0.001	-0.003
	4	0	-0.001	-0.007
	5	0	-0.001	-0.011
M4	1	34.166	1.315	-6.432
	2	34.166	1.315	-1.993
	3	34.166	1.315	2.446
	4	34.166	1.315	6.885
	5	34.166	1.315	11.324
M5	1	25.461	-2.337	15.806
	2	25.461	-2.337	7.774
	3	25.461	-2.337	-0.258
	4	25.461	-2.337	-8.29
	5	25.461	-2.337	-16.322
M6	1	0	0.003	-0.02
	2	0	0.003	-0.01
	3	0	0.003	0
	4	0	0.003	0.012
	5	0	0.003	0.022
M7	1	0	-0.003	0.019
	2	0	-0.003	0.009
	3	0	-0.003	-0.001
	4	0	-0.003	-0.012
	5	0	-0.003	-0.022
M8	1	25.456	2.233	-14.482
	2	25.456	2.233	-6.806
	3	25.456	2.233	0.869
	4	25.456	2.233	8.544
	5	25.456	2.233	16.219
M9	1	15.974	-2.103	11.675
	2	15.974	-2.103	4.869
	3	15.974	-2.103	-1.937
	4	15.974	-2.103	-8.743
	5	15.974	-2.103	-15.55
M10	1	0	0.006	-0.034
	2	0	0.006	-0.015
	3	0	0.006	0.004
	4	0	0.006	0.023
	5	0	0.006	0.042
M11	1	0	-0.006	0.034
	2	0	-0.006	0.015
	3	0	-0.006	-0.004

	4	0	-0.006	-0.023
	5	0	-0.006	-0.042
M12	1	15.97	2.119	-11.772
	2	15.97	2.119	-4.913
	3	15.97	2.119	1.947
	4	15.97	2.119	8.806
	5	15.97	2.119	15.666
M13	1	6.536	-1.82	11.747
	2	6.536	-1.82	4.632
	3	6.536	-1.82	-2.484
	4	6.536	-1.82	-9.6
	5	6.536	-1.82	-16.716
M14	1	0	0.01	-0.066
	2	0	0.01	-0.027
	3	0	0.01	0.012
	4	0	0.01	0.051
	5	0	0.01	0.091
M15	1	0	-0.01	0.066
	2	0	-0.01	0.027
	3	0	-0.01	-0.012
	4	0	-0.01	-0.052
	5	0	-0.01	-0.091
M16	1	6.533	1.809	-11.622
	2	6.533	1.809	-4.55
	3	6.533	1.809	2.523
	4	6.533	1.809	9.595
	5	6.533	1.809	16.667
M17	1	1.301	8.703	-25.724
	2	1.301	4.334	3.61
	3	1.301	-0.036	13.281
	4	1.301	-4.405	3.289
	5	1.301	-8.775	-26.365
M18	1	0	8.74	-26.333
	2	0	4.371	3.168
	3	0	0.001	13.006
	4	0	-4.368	3.181
	5	0	-8.738	-26.307
M19	1	-0.918	8.769	-26.337
	2	-0.918	4.399	3.29
	3	-0.918	0.03	13.254
	4	-0.918	-4.34	3.555
	5	-0.918	-8.709	-25.806
M20	1	2.866	9.487	-27.998
	2	2.866	4.722	3.972
	3	2.866	-0.044	14.496
	4	2.866	-4.809	3.576
	5	2.866	-9.575	-28.789

M21	1	0	9.531	-28.733
	2	0	4.765	3.433
	3	0	0	14.155
	4	0	-4.766	3.432
	5	0	-9.531	-28.735
M22	1	0.114	9.575	-28.791
	2	0.114	4.81	3.576
	3	0.114	0.044	14.498
	4	0.114	-4.721	3.976
	5	0.114	-9.487	-27.992
M23	1	3.026	9.438	-27.297
	2	3.026	4.672	4.45
	3	3.026	-0.093	14.753
	4	3.026	-4.859	3.611
	5	3.026	-9.624	-28.976
M24	1	0	9.531	-28.868
	2	0	4.765	3.298
	3	0	0	14.02
	4	0	-4.766	3.297
	5	0	-9.531	-28.871
M25	1	0.31	9.625	-28.979
	2	0.31	4.859	3.611
	3	0.31	0.094	14.756
	4	0.31	-4.672	4.457
	5	0.31	-9.437	-27.288
M26	1	3.252	6.536	-16.716
	2	3.252	3.143	5.064
	3	3.252	-0.25	11.575
	4	3.252	-3.643	2.818
	5	3.252	-7.036	-21.207
M27	1	0	6.785	-21.117
	2	0	3.392	1.782
	3	0	0	9.412
	4	0	-3.394	1.774
	5	0	-6.787	-21.132
M28	1	1.809	7.039	-21.224
	2	1.809	3.646	2.818
	3	1.809	0.253	11.592
	4	1.809	-3.14	5.096
	5	1.809	-6.533	-16.667

Table 51: Deflection in Concrete Using the Largest Positive Moment from RISA-2D

$\sigma_{minconcrete}$(psi)	531.0
M (lbft)	6,216.0
w (lb/ft)	153.5
$\Delta_{concrete}$ (in)	-1.33278
$\Delta_{concrete}$ (in) <	-0.61048

Table 52: Complete 3-point bend test comparison

3POINT	Oak Girder			Douglas Fir Girder	
$\Delta=PL^3/48EI$	Building	Full Scale	Prototype	Prototype	Prototype
P (lb)	19056.06	11900.00	1983.33	1983.33	1983.33
L (ft)	18	18	6.00	6.00	6
E (ksi)	1200	1200	1200	1500	1300
b (in)	10	10	4	3.5	5.5
h (in)	16.00	16.00	6	5.5	5.5
I (in^4)	3413.33	3413.33	72	48.53	76.26
Δ (in)	0.977	0.610	0.18	0.21	0.16
$\Delta_{allowable}$ (in)	0.9	0.9	0.3	0.3	0.3

Table 53: Girder Multiplication factors for 3-point bend test

Values	Girder Mult. Factor
P (lb)	6
L (ft)	3
E (ksi)	0.923076923
b (in)	1.82
h (in)	2.91
I (in^4)	44.76
Δ (in)	3.92
$\Delta_{allowable}$ (in)	3.92

Table 54: 3-point bend test comparison for slab and planking

3POINT	Concrete Slab			Southern Y.P.Plank	
$\Delta=PL^3/48EI$	Building	Full Scale	Prototype	Building	Prototype
P (lb)	8280.00	5150.00	1983.33	629.22	1983.33
L (ft)	18	18	6	8	1.083333333
E (ksi)	4250	4250	3000	1000	1000
b (in)	96	96	13	7	5.538461538
h (in)	2	2	1.67	3	1
I (in^4)	64	64	5.02	15.75	0.461538462
Δ (in)	6.39	3.98	1.02	0.74	0.20
Δ/L	0.03	0.02	0.01		
<L/depth	108	108	43.2		

Table 55: Multiplication factors for Concrete slab

Values	Concrete Mult. Factor
P (lb)	2.60
L (ft)	3.00
E (ksi)	1.42
b (in)	7.38

h (in)	1.20
I (in^4)	12.76
Δ (in)	3.88
$\Delta_{allowable}$ (in)	3.88

Table 56: Multiplication Factors for Planking

Values	Planking Mult. Factor
P (lb)	0.32
L (ft)	7.38
E (ksi)	1.00
b (in)	1.26
h (in)	3.00
I (in^4)	34.13
Δ (in)	3.74
$\Delta_{allowable}$ (in)	3.74

Table 57: Complete 4-Point Bend Test

4 POINT		Oak Girder		Douglas Fir Girder	
$\Delta=11PL^3/768EI$	Building	Full Scale	Prototype	Prototype	Prototype
P (lb)	19056.06	17300.00	2162.50	2162.50	1900.00
L (ft)	18.00	18.00	6.00	6.00	4.83
E (ksi)	1200	1200	1200	1500	1600
b (in)	10	10	4	3.5	3.5
h (in)	16	16	6	5.5	5.5
I (in^4)	3413.33	3413.33	72.00	48.53	48.53
Δ (in)	0.67	0.61	0.13	0.16	0.07
$\Delta_{allowable}$ (in)	0.9	0.9	0.3	0.3	0.241666667

Table 58: Multiplication Factors for the Girder 4-point bend test

Values	Girder Mult. Factor
P (lb)	9.105263158
L (ft)	3.724137931
E (ksi)	0.75
b (in)	2.857142857
h (in)	2.909090909
I (in^4)	70.34023827
Δ (in)	8.91
$\Delta_{allowable}$ (in)	8.91

Table 59: Slab and Planking Scaling

4 POINT	Concrete Slab			Southern Y.P.Plank	
$\Delta=11PL^3/768EI$	Building	Full Scale	Prototype	Building	Prototype

P (lb)	8280.00	7525.00	1900.00	629.22	1900.00
L (ft)	18.00	18.00	4.83	8.00	0.69
E (ksi)	4250	4250	3000	1000	1000
b (in)	96	96	8.25	7	3.222222222
h (in)	2	2	1.80	3	1
I (in^4)	64.00	64.00	4.01	15.75	0.27
Δ (in)	4.39	3.99	0.44	0.51	0.06
Δ/L	0.020342316	0.018487431	0.007610787		
<L/depth	108.000	108.000	32.222		

Table 60: Multiplication Factors for Concrete Slab

Values	Slab Mult. Factor
P (lb)	3.960526316
L (ft)	3.724137931
E (ksi)	1.416666667
b (in)	11.63636364
h (in)	1.111111111
I (in^4)	15.96209004
Δ (in)	9.05
$\Delta_{allowable}$ (in)	9.05

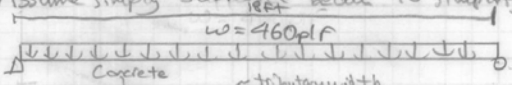
Table 61: Multiplication Factors for Planking

Values	Plank Mult. Factor
P (lb)	0.331169591
L (ft)	11.63636364
E (ksi)	1
b (in)	2.172413793
h (in)	3
I (in^4)	58.65517241
Δ (in)	8.90
$\Delta_{allowable}$ (in)	8.90

Deflection Calculations

Concrete acting itself as a girders (Floors 3+4)

Assume simply supported beam to simplify our calculations:



$L = 18ft$

~tributary width

$W_{dead} = (105pcf)(8ft)(2/12) = 140\ plf$

$b = 96in$, 2in

$W_{live} = W_{partition} + W_{construction}$

$W_{live} = (20psf)(8ft) + (20psf)(8ft) = 320\ plf$

$w = W_{dead} + W_{live} = 140plf + 320plf = 460\ plf$

base = 96in   h = 2in   assume: $E(ksi) = 4000$ (can range between 3000–4000 ksi)

solve for the moment in the middle of the concrete girder:

$$M = \frac{wL^2}{8} = \frac{(460plf)(18ft)^2}{8} = 18630\ lbft$$

$c = 1$

$$I = \frac{bh^3}{12} + \cancel{Ad^2}^{\,0} = \frac{(96'')(2'')^3}{12} = 64in^4$$

$$\text{bending stress} = \frac{MC}{I} = \frac{18630lbft}{64in^4} = 3493.1psi > 531\ psi$$

cracks

$f'c = 5000$ (according to compression test)

$f_t = (5\ \text{to}\ 7.5)(\sqrt{f'c})$ ACI eqtn

use 7.5 → higher value more conservative

$f_t = 7.5\sqrt{5000} = 530.3\ psi$

Minimum bending stress to crack = 531

$$M = \frac{(\sigma)(I)}{(c)} = \frac{(531psi)(64in^4)(1ft)}{(1in)(12in)} = 2832\ lbft$$

$$w = \frac{8\,(M)}{L^2} = \frac{(8)(2832lb/ft)}{(18ft)^2} = 69.9\ lb/ft$$

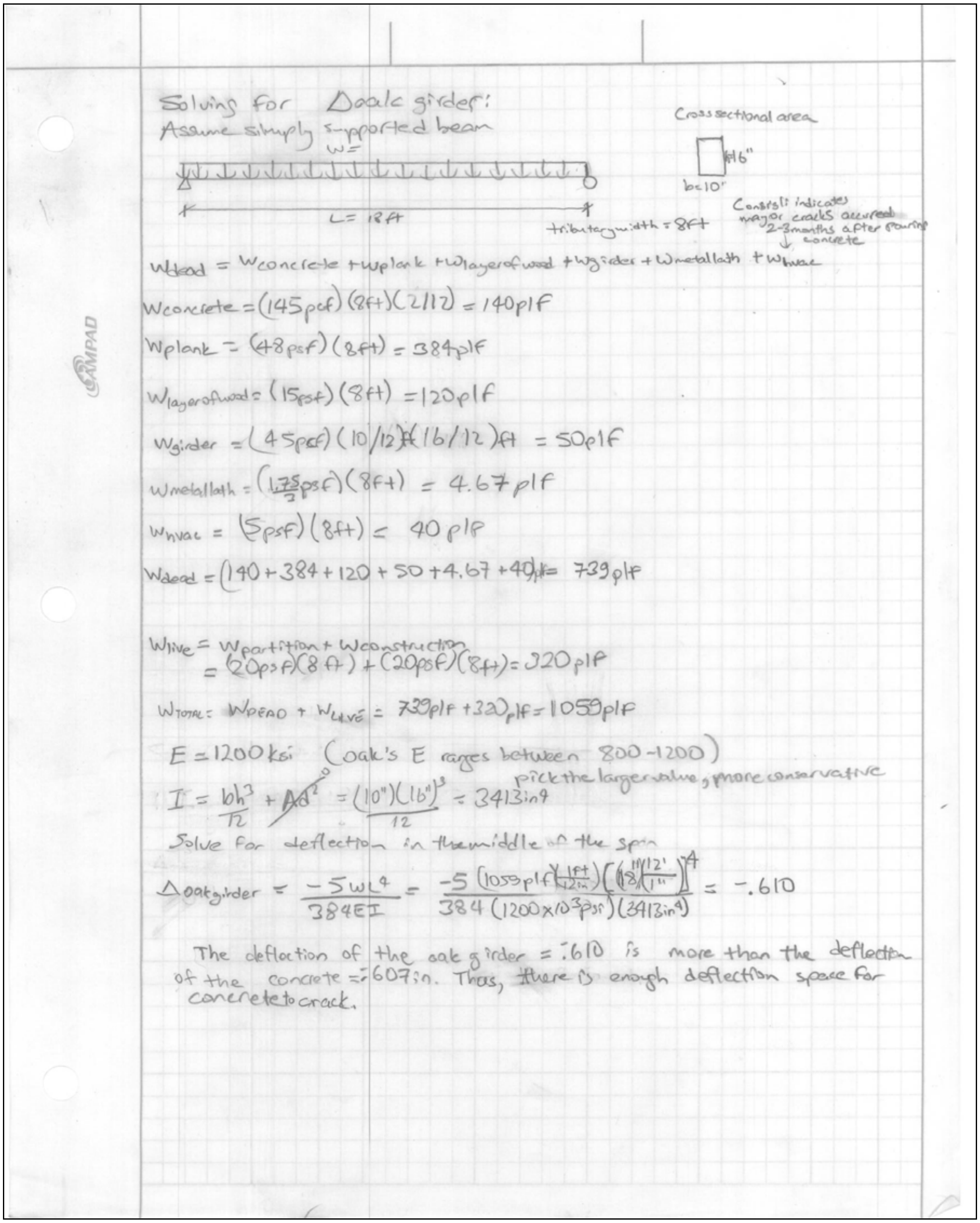
$E_{concrete} = 3000 - 5000\ ksi$, assume $E = 4250ksi$

Solve for deflection in the middle of the span with σ_{min}

$$\Delta_{conc} = -\frac{5wL^4}{384EI} = \frac{-5(69.9lb/ft)\left(\frac{1ft}{12in}\right)\left[(18ft)\left(\frac{12in}{ft}\right)\right]^4}{384(4250\times10^3psi)(64in^4)} = -.607in$$

$\Delta_{concrete} = -.607in$ must be less than the deflection of the oak girder. Solve for $\Delta_{oak\,girder}$:

Figure 42: Concrete Deflection Calculations



Solving for Δoak girder:
Assume simply supported beam

$w =$

$L = 18ft$

tributary width $= 8ft$

Cross sectional area

$h = 16"$

$b = 10"$

Consoli indicates major cracks occurred 2-3 months after pouring concrete

$w_{dead} = w_{concrete} + w_{plank} + w_{layer of wood} + w_{girder} + w_{metal lath} + w_{hvac}$

$w_{concrete} = (145pcf)(8ft)(2/12) = 140plf$

$w_{plank} = (48psf)(8ft) = 384plf$

$w_{layer of wood} = (15psf)(8ft) = 120plf$

$w_{girder} = (45pcf)(10/12)ft(16/12)ft = 50plf$

$w_{metal lath} = (\frac{1.75}{2}psf)(8ft) = 4.67plf$

$w_{hvac} = (5psf)(8ft) = 40plf$

$w_{dead} = (140 + 384 + 120 + 50 + 4.67 + 40)plf = 739plf$

$w_{live} = w_{partition} + w_{construction}$
$= (20psf)(8ft) + (20psf)(8ft) = 320plf$

$w_{TOTAL} = w_{DEAD} + w_{LIVE} = 739plf + 320plf = 1059plf$

$E = 1200ksi$ (oak's E ranges between 800-1200)
pick the larger value, more conservative

$I = \frac{bh^3}{12} + \cancel{Ad^2} = \frac{(10")(16")^3}{12} = 3413in^4$

Solve for deflection in the middle of the span

$$\Delta_{oak girder} = \frac{-5wL^4}{384EI} = \frac{-5(1059plf)(\frac{1ft}{12in})\left((18')(\frac{12"}{1'})\right)^4}{384(1200\times10^3psi)(3413in^4)} = -.610$$

The deflection of the oak girder $= -.610$ is more than the deflection of the concrete $= -.607in$. Thus, there is enough deflection space for concrete to crack.

Figure 43: Oak Girder Deflection Calculations

Another approach to find concrete cracking:

use $\Delta_{oak\ girder} = -.610\text{in}$ to find w to find $\sigma_{concrete}$ to check with f'_t

$$\Delta = \frac{-5wL^4}{384EI} \qquad E = 4250\text{ksi} \quad L = 18\text{ft} \quad I = 64\text{in}$$

$$w = \frac{\Delta 384EI}{-5L^4} = \frac{-384(-.610\text{in})(4250\times10^3\text{psi})(64\text{in})}{-5((18)(12))^4} = 5.85\text{lb}\cdot\text{in} = 70.3\text{lbft}$$

$$M = \frac{wL^2}{8} = \frac{(70.3\text{lbft})(18\text{ft})^2}{8} = 2847.2\text{lbft}$$

$$\sigma_{concrete} = \frac{Mc}{I} = \frac{(2847.2\text{lbft})(12\frac{\text{in}}{\text{ft}})(1\text{in})}{(64\text{in})} = 533.9\text{psi} > f'_t = 530.3\text{psi}$$

$\therefore$ cracks

Figure 44: Another Method of Calculating Concrete Cracking

Moment of reinforced Concrete pg 1

compressive strength of Concrete - $f'_c = \boxed{5300\ psi}$

Distance to centroid - $y_t = \boxed{1.0\ in}$

$I_{matrix} = \dfrac{96 \times 2^3}{12} = \boxed{64\ in^4}$

flexural cracking moment - $f_r = 7.5\sqrt{5300} = \boxed{546\ psi}$

Cracking moment - $C_r = \dfrac{64 \times 546}{1.0} \times \dfrac{1}{12} = \boxed{2912\ lb\,ft}$

$W = \boxed{464.7\ lb/ft}$

$L = 18\ ft$

positive moment $= \dfrac{464.7 \times 18^2}{8} = \boxed{18{,}819\ lb\text{-}ft}$

F_y (tensile) $= \boxed{58{,}000\ psi}$

$A = \dfrac{.015 \times 2}{.5} = .06 \qquad \left[\left(\dfrac{.132}{.5}\right) \times .06\right] + \left[(1 - .06)\left(\dfrac{.01}{.5}\right)\right] = .035$

$\dfrac{(.035 + .01)}{2} = .022 \qquad (.95 \times 2 \times .035 \times 2) + \left[\left(\dfrac{1}{.539}\right) \times .05 \times .178)\right] = .111$

$.022 \times 8 \times 12 \times .111 = \boxed{.239\ in^2}$

$T = 58{,}000 \times .239 = \boxed{13859\ lb}$

$f'c = \boxed{5300\ psi}$

$b = 8 \times 12 = \boxed{96\ in}$

$a = \dfrac{(.239 \times 58000)}{(.85 \times 5300 \times 96)} = \boxed{.032\ in}$

$d = \dfrac{31}{32} = \boxed{.969}\ in$

$M_n = .239 \times 58000 \times \left(.969 \times \dfrac{.032}{2}\right) = \boxed{13203}\ lb\,ft$

Figure 45: Bending Moment Calculations for Concrete and Metal Lath

reinforced Concrete — pg 2

$\phi M_n = 13203 \times .9 \div 2 = \boxed{990 \text{ lb ft}}$

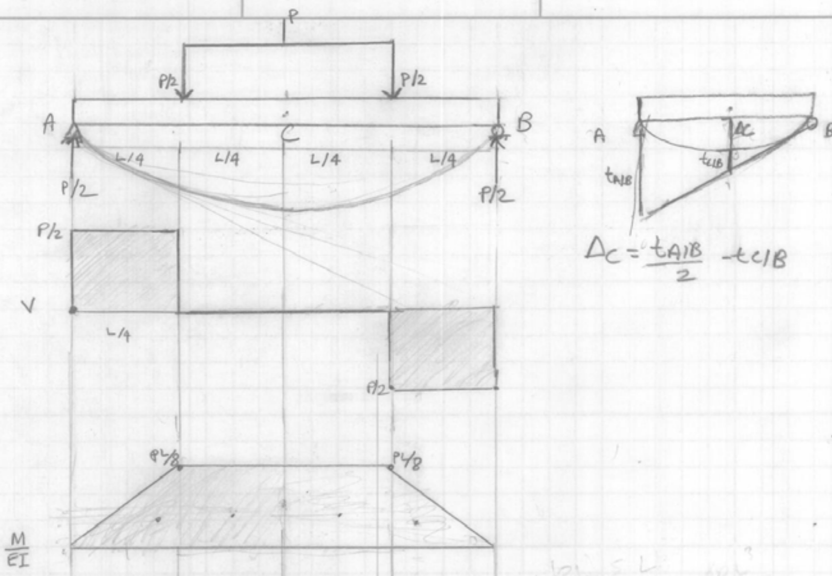
$\phi M_n + M_{cr} = 990 + 2912 = 3902 \text{ lb-ft}$

Positive moment > $\phi M_n + M_{cr}$

$18{,}819 > 3902$

∴ cracking will occur regardless of reinforcement

Figure 46: Moment Calculation for Concrete and Metal Lath pg 2



$$t_{A/B} = \left[\left(\frac{1}{2}\right)\left(\frac{PL}{8}\right)\left(\frac{L}{4}\right)\left(\frac{L}{4}\right)\left(\frac{2}{3}\right) + \left(\frac{L}{2}\right)\left(\frac{PL}{8}\right)\left(\frac{L}{2}\right) + \left(\frac{1}{2}\right)\left(\frac{PL}{8}\right)\left(\frac{3L}{4} + \left(\frac{1}{3}\right)\frac{L}{4}\right)\left(\frac{L}{4}\right)\right] \Big/ EI$$

$$= \left[\left(\frac{PL^3}{384} + \frac{PL^3}{32} + \frac{PL^2}{64}\left(\frac{10L}{12}\right)\right)\right] \Big/ EI$$

$$= \left[\frac{2PL^3}{768} + \frac{24PL^3}{768} + \frac{10PL^2}{768}\right] \Big/ EI$$

$$= \frac{36L^3}{768EI}$$

$$t_{C/B} = \left[\left(\frac{L}{4}\right)\left(\frac{PL}{8}\right)\left(\frac{1}{2}\right)\left(\frac{L}{4}\right) + \left(\frac{1}{2}\right)\left(\frac{PL}{8}\right)\left(\frac{L}{4}\right)\left[\left(\frac{1}{3}\right)\left(\frac{L}{4}\right) + \left(\frac{L}{4}\right)\right]\right]$$

$$= \frac{PL^3}{256} + \frac{PL^2}{64}\left(\frac{4L}{12}\right)$$

$$= \frac{3PL^3 + 4L}{768EI}$$

$$= \frac{7PL^3}{768EI}$$

$$\Delta_C = \left(\frac{\overset{18}{36}L^3}{768EI}\right)\left(\frac{1}{2}\right) - \frac{7PL^3}{768EI} = \boxed{\frac{11PL^3}{768EI}}$$

$\frac{16}{768}$

Figure 47: 4-Point Bend Test

8.3 MEETING MINUTES

MQP MEETINGS

August 28, 2006

- Meet to discuss possible project topics
- General guidelines of how MQP is run is laid out

September 18, 2006

- Capstone idea discussed
- Proposal first presented
- Discussion of how to improve proposal took place

September 25, 2006

- Testing possibilities are discussed in detail
- Testing proposal shown to advisors
- Discussion of how testing proposal needed to be improved

October 2, 2006

- First copy of MQP presented to advisors
- Discussion related to the organization of the TOC took place
    - Rearrangement of items
    - Clarity of thoughts through TOC organization was needed
- CPM aspect of project was discussed

October 9, 2006

- Second Copy of the MQP was presented to advisors
- Discussion on progress of paper and clarity of TOC proceeded
- Further talks on the CPM aspect of project took place
    - Information Systems

Ocotober 31, 2006

- Tuesday October, 31st Meeting Minutes
- Discussion on exact intentions of CPM portion of project resulted in a paragraph that was submitted on Nov. 1st
- We were told to check into the crack situation at Gateway because problems with the epoxy filler have occurred.
- Making a “diary” of the crack was suggested along with the CPM of the cracks
- Suggestions on how to go about modeling the beams were made

November 7, 2006

- By Tuesday, November 21 a schedule and methodology of testing is needed
- Professors requested more specifics on our testing
- Discussed a means for gathering data during Consigli’s weekly meetings

November 14, 2006

Calculations

- Scaling

    - Deflections
    - Shear studs

Testing Scope

- Performance- composite vs. noncompoiste
    - Deflections
    - Bending/Shear (Four point test)

Testing Order

- Girder
    - Test bending limits of girder to allow for a comparison to the rest of the tests
- Girder + Flooring
    - To determine if assuming the flooring and girder act as a single beam is justified.
- Girder + Flooring + Concrete (wire mesh + priming) => Noncomposite
    - To see how the concrete behave when there are no shear studs.
- Girder + Flooring + Concrete (wire mesh + priming) + Shear plate (on girder)
    - This is the supposedly the highest perfoming form of a shear stud (peggi clauston)
- Girder + Flooring + Concrete (wire mesh + priming) + Nails (on girder + flooring)
    - This is the most economically efficient for of shear studs.
- Girder + Flooring + Concrete (wire mesh + priming) + Shear plate (on girder) Nails (on flooring)
    - If none of the two types of shear studs prove to be providing the desired results, then a combination of the two will be done to see if this will allow for the required loading

Materials

- 18 cubic feet of concrete – six cubic feet per beam
- 56 feet of 6x8 oak cut into six 9 foot sections
- 156 feet of 3x7 southern yellow pine planking cut into 2 foot sections
- 270 feet of .75x4 maple flooring cut into 2 foot sections
- Wire mesh
- priming
- Nails and glue to attach flooring and planking to girder
- Nails (20d 7 gauge spiral shank) to use as shear studs
- Wire mesh to use for shear studs
- Adhesive

November 21, 2006

Capstone:

- Calculations
- Materials
- Cost

CPM:

- Rationale

November 28, 2006

CPM

- What to look for checklist:
  - Technology used during the meeting
  - Number of phone calls received during the meeting
  - Presentation methods used
  - Number of times group conversation drifts from original topic
  - Number of times outlier conversations are had
  - Other time delays
- The Gilbane and Consigli checklists will be compared

Testing Facilities and Materials

- Talked to Howe Lumber and confirmed materials were instock
- Talked to Don about obtaining materials
  - P.O is ready to be submitted
- Talked to Dean who agreed to build a connector
  - Says he will order all materials once the final P values are known

Testing Calculations

Three Point Bend Test:

- Tried to match distributed load deflection values for the three point bend test
  - By applying a smaller load we were able to come up with a full scale oak girder deflection model.
  - There is not enough lab space for full scale modeling
  - Three different prototypes were considered with a length of six feet
    - 4x6 oak girder
    - 3.5x5.5 Douglas Fir Girder
    - 5.5x5.5 Douglas Fir Girder
  - 5.5x5.5 Douglas Fir Girder was the most available and cost effective girder for testing
  - Multiplication Factors for P,L,E,b,h,I and Deflection were found
  - Deflection Values for 5.5x5.5 Douglas fir girder, concrete, and southern yellow pine plank were reduced by a factor of three compared to the original design.
  - We checked to make sure the deflection values were below the allowable deflection values
  - We checked to make sure the bending stress was adequate to crack

Four Point Bend Test:

- Because a three point bend test has the potential to cause localized cracking we choose to use a four point bend test.
- The same procedure as above was followed to check values

Composite Beam:

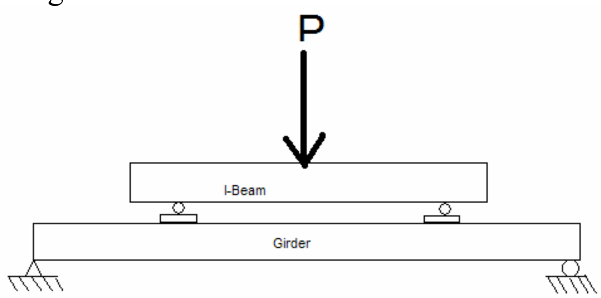
- Using Equations given by Clouston's articles the following values were found
  - Shear Connector Reduction Factor – γ_1
  - Effective EI - $(EI)_{ef}$
  - Deflection of Beam – Δ
  - Total Normal Stress in Wood – $\sigma_{w,T}$
  - Total normal stress in concrete - $\sigma_{c,C}$

- Shear Flow through the connector - f_v
- Maximum Beam Shear Stress – q

December 5, 2006

Capstone:

- Wood has been ordered and due to arrive within the next few days.
- Final design for 4-point bend test
    - Girder Length Limitation: 59 inches
    - Simpler and more effective design: instead of a frame we are using an I-beam with two supports
    - Following is a graphical figure as to how the four point bend test will be done using Instron machine.



Schedule for Testing:

- Test nail to find slip modulus to finalize the number of nails required (Friday December 8).
- Test the girder itself, and also girder with the planking (Friday December 8 or 15)
- Cutting plywoods, Nailing, making forms, providing adequate support will be completed (Friday December 15- Tuesday December 19)
- Pour Concrete (Wednesday, December 20)
- Test the beams within the first two weeks of C-term.

CPM:

- Finalized Checklist
- Gilbane Meeting
    - Pretty smooth; not complex. Good Weather ☺
    - Confidence and smiles
    - Abdallah from Gilbane Co. gave us a tour of the site.
        - Talked about the coordination of the labor, and jobsite
        - Provide trust -> Get the most out of your workers. Ex. $2000 worth of supplies (gloves etc.). Get in return $25000 worth of labor.
        - Jobsite: precast construction

Consigle Meeting

- December 4: Pretty smooth and finished quickly. Extra talks (. 2 phone calls
- Afterwards, Mr. Carkin Gilbane construction, who is coordinating the overall pavement of the Gateway Park, gave a tour the site while talking about the coordination issues of the Gateway Park.
    - Gateway Park consists of four different contracts:

    - Master Grading plan (Dirt to balance out)
    - Public Works Economic Development (PWED) (road and parking lots)
    - Parking Garage
    - 66 & 68 Prescott Street (Consigli)
  - Major coordination problem between Consigli and Gilbane
    - 66 Prescott Streee opening for the movement of the materials
    - Gilbane wanted it somewhere else.
    - Consigli has to work around Gilbane's construction schedule.
  - One of the parking lots will be paved before winter.
    - Currently used for double T beam onsite transportation
    - The space will be available for paving after double Ts are assembled
    - Underground electrical placed
    - Consigli will have parking space: an area easy to plow for winter and storage.

B-term Submittal will contain:

- Corrections for A term submittal
- Explanation of testing methodology, critical path and calculations
- Background for CPM
  - Construction companies: Gilbane & Consigli
  - Coordination issue at Gateway Park
  - Information on communication
- CPM Methodology
  - Checklist
  - Observations from weekly meetings.
  - Questionnaire on Technology use (C-term)

December 12, 2006

1) Observe electronic and non-electronic communication for each particular entity (WPI, WBDC, Consigli & Gilbane) on the project.
   a. meetings
      i. Identify, through observation, incidences in meetings where time is spent away from the order of meeting.
      ii. Take a time recording of minutes spent, diverged from the designated order of the meeting. (phone calls, unrelated conversations)
      iii. Find incidences where the meetings would be run more efficiently if a different means of presenting was available.
      iv. Keep track of communication tools used during the meetings
   b. Questionnaire
      i. Communication (electronic & non-electronic) between parties in addition to weekly meetings
         1. electronic – email, fax, phone calls, web-based information – exchange program (prolog)

   2. non-electronic – mail, face-to-face conversation, additional meetings (subs meeting, walk throughs)

2) Make suggestion based on observations, questionnaire and background information

Capstone:

Testing:

- Nails purchased
    - Home Depot
    - Elwood Adams
- Supplies for pull out test purchased
- Wood Delivered
- Pull-out test performed

Nail d (in)	0.133			
Penetration (in)	test 1 (lb)	test 2 (lb)	test 3 (lb)	test 4 (lb)
15d	253.1	255.6		
17d	453.1	377	320.4	395.5
19d	289.3	270		
21d	573	556.5		

Table 62: Pull Out Test Results

- 15d, 17d, 19d, 21d, pull out test
- 19d test looks inaccurate
- 21d best – will use 21d with a longer nail (3.5")

What is Next?

B-term Submittal:

- Include methodology of capstone design to the A term submittal
- More detailed background on owners and construction firms
    - OBS

Capstone Test work:

- Order concrete and wire mesh
- Test the girder
- Test the girder + plank
- Form the molds
- Pour Concrete

January 16, 2007

B-term submittal reviewed

- Composite beam section needs to be clear and very concise
- Communication section needs to be expanded.
- Table of Contents is sufficient
- First submittal of the C-term is February 7, 2006

Composite beam testing

- Loading- observe cracks
- Global behavior
- Deflections- dial gauge? Strain gauge?

Communication issues

- Organized meetings- What needs to be improved? Adequate preparation?
- At least one solid observation to focus on

January 23, 2007

Capstone Design

- Discussion of testing (pictures, videos etc)
- Dynamic vs static testing
- Design solution approach is handed back

CPM

- Feedbacks on the questionnaire (Question #6)
- Distribution intentions on the questionnaire
    - Public Relation Approach: Contact Steve Johnson and Neil Banner

January 30th, 2007

- Went over revisions for the submittal
- Testing 1 presented
- Questionnaires presented
- Discussed submittal methods
    - Hardcopy for Professor Salazar
    - Cover page, Abstract, Authorship, CAPSTONE and the rest of it on CD
    - Electronic submittal to registrar.

February 6th, 2007

Testing began on Tuesday, January 30, and continued through February 6th, 2007.

Testing Schedule:

- Tuesday January 30, 2007
    - 1 non-composite
- Wednesday February 31, 2007
    - 1 first principles
    - 1 Peggie-Clouston
- Friday February 2, 2007
    - 1 non-composite
    - 1 first principles
    - 1 Peggie-Clouston
- Tuesday February 6, 2007
    - 1 girder
    - 1 girder with planking

Testing Approach:

- Tested deflection vs. Loading

Meetings & Questionnaires

- Monday January 30, 2007
    - Attended Consigle Meeting
    - 3 surveys completed
    - 1 more survey received within the same week.
- Wednesday February 31, 2007- Gilbane Meeting
    - 6 surveys completed
- Monday February 5, 2007

- Attended Consigli Meeting
- 1 survey completed
- 1 survey electronically submitted

• Tuesday February 6, 2007
- 1 survey electronically submitted
- 1 survey received in mail box

• Total of 14 questionnaires received and waiting on 4 more questionnaires.

February 13, 2007

General questions were asked to the advisors regarding the background and the analysis and blank spaces on the report.

Timber Concrete Design sections includes:

Results Section
- – Key Test
- – General Trend of Data
- – Important Issues w/ Testing
  - • Mishaps and Mistakes?

Conclusion
- – Final Design
- – Cost of Design
- – Final Recommendations
- – What was learned?

17 out of 18 questionnaires were returned.

Tuesday February 20, 2007

The hypothesis, the goal, and the results of the meeting section were discussed.

Tuesday February 28, 2007

The results section for timber composite beams will include stress strain diagram and cost analysis. Statistical analysis was discussed, however it was not found suitable for the limited testing.

BTC & GARAGE MEETINGS

9/18/06 (BTC)

Insulated manhole cover finalized

Progress on Railing? Talk to Suzy on Friday, make it continuous, sketch at the shop and get a price

Rubbing of Foundations

Sign- location? Backlighting? City zoning? Signage on top of the building as well

Weather conditions -> concrete delays

Smoking area- closer or away from the entrance

Casework-humidity levels/fluctuations?

If lack of airflow, manufacturers can run away

Humidity 30%-60%

Heat (once epoxy use) not below 65°F not over 80°F

Gas inspector- boilers started?

Enclosing- no fresh air – too hot, too cold, too humid

WBDC- “risk is small but significant”

End of October for enclosure-> 3-4 months to see if something goes wrong.

Critical path-> Plumbing needed for cases
National Grid (electricity)-> 2 weeks delay, give them a week to do their task (for power and heat)
Ceiling inspections- grid must be fixed, do a punch list. Document to WBDC what is found.

09/25/06 (BTC)

Talks about general issues took place. Consigli informed WPI and the WBDC of the events that had taken place during the previous week as well as what was scheduled to be completed for the week to come. A reading through the "change orders" was done and any questions the owners had were answered by Consigli.

10/02/06 (BTC)

Landscaping-> Value engineering Prices? Choices?
$35000 gap -> what was on design vs. value eng.
View from Prescott Street -> More brick?
Check on chillers, heat exchangers. Chill water even if power goes down?
90° Glass Returns – small amount of glass remains to be delivered
Roof Screen -> 2nd week of October
A/C-> Wednesday meeting- laid but not confirmed 3 to 4 weeks from now, check rotation/fill system/go from there.
Temp at 50s-60s, Humidity is consistent
MRI Roof – grass? – ground cover?- exhaust fan, MRI related pipes, heating/cooling systems better when they are point onto roof. Access for maintenance? Durability issue? Warranty?
Exterior wall assembly testing- new windows, existing doors, curtain walls, metal panels

10/30/06 (BTC)

Talk directed around benches
Confusion on where benches will be placed
Times wasted passing papers out
Time wasted searching for items on page
Fume hoods, bathrooms, pouring of ramp walls, phone placement and 4th floor casework discussed
Punchlist detailed
Drilling and cutting of casework discussed
Questions about last meeting events
Repointing of building questioned

11/6/06 (BTC)

Storage, lighting, trash receptacles (bottom of stairs, northside) issues are discussed.
Cracks in the walls?- put joints?
Tapering off -> asthetic reasons, then a retuning wall, grade at roof to walk
Return wall or sandblast?
Architect draws on the marker board.

11/13/06 (BTC)

Schedule was revised using primavera
Progress of construction questioned heavily
Site map used to show where trash cans would be placed

Architect tries to free hand sketch
Retainage- give half back to masonry after punchlish
The competitives did not say good things about Northwest
Nortwest will crackfill the cracks
Ardex guy months ago hammered the Northwest.

11/27/06 (BTC)

Talks about vanzelm
Delays occur, people enter office - less than one minutes
One off track conversation about meshing leads to other questions to arise
RFI's started in middle of minutes

11/29/06 (Garage)

Change orders are agreed upon
Can't have gas (regulations for fireproof, fume, and exhaust), has to be diesel (less toxic)
Landspacing- guardrails (trees in?) -> Not worth to go to change board. Thus, leave it in.
Good winter- ahead of schedule
14 tractor trailers -> Permit issue problem at another state. Might have to wait until January 2nd

12/4/06 (BTC)

Type of carpeting to be used discussed and shown on paper.
Architect and Project engineer discussion regarding fine finishing gravel vs. regular gravel
Start freight elevator?
Different types of steams were discussed: clean steam, process steam
Short meeting compared to usual

12/11/06 (BTC)

Rubbing the side of the building- WBDC thought it was too much money
WPI upset about the price of the door swipes
Consigli will try to carpet the 4th floor by the end of the week.
Walk through for both WPI and WBDC is scheduled.

01/17/07 (Garage)

- Scott Farrar of National Gril mentions that cables will be installed but the service will not be energized until easement is completed. Ms. Lynch asked Alita on a phone conference to see if it can be considered a public way. The answer was positive. That would allow National Grid to proceed.
- Guardrails of the parking lot were discussed. (metal on wood, wood on metal, etc) Metal poles with wood rails were recommended because metal on wood was agreed upon at the previous meeting. The fact that they were able to get wood on wood was arbitrary since the final decision remained the same.

01/22/07 (BTC)

- Vanzelm was charging Gateway Park extra money for items overlooked in their original estimate.
- Underlayment of 68 was crossed of the meeting minutes. It was considered a non-issue.
- The exit signs will put up by Consigli before fire department walks through.

01/29/07 (BTC)

60 & 68 Prescott Street Meeting led by Consigli:

- Going over Vanzelm's change orders and RFI to make sure Vanzelm is all set for completion of the project.

December 4, 2006 (a typical meeting checklist)

Description	Number	Minutes
Technology		
e-mail		
PDF		
Type of Presentation		
Computer Presentation		
Slide Show		
no formal speaker		
Dominant Speaker		
Meeting Leader		
Meeting Materials		
Meeting Minutes		
RFI		
Change Orders		
Site Plans		
Material Samples		
Calendar	X	
Phone Calls (during meeting)	X	
	X	
Unrelated Talks (when group conversation astrays)		
Extra Talks (when two people talk or when they go of tangents)	X	4
Others (Waste of Time)		
Photocopy during Meeting		
Two people left meeting		
Nate & J. McDermott talk (delay)		5

December 11, 2006 (a typical meeting checklist)

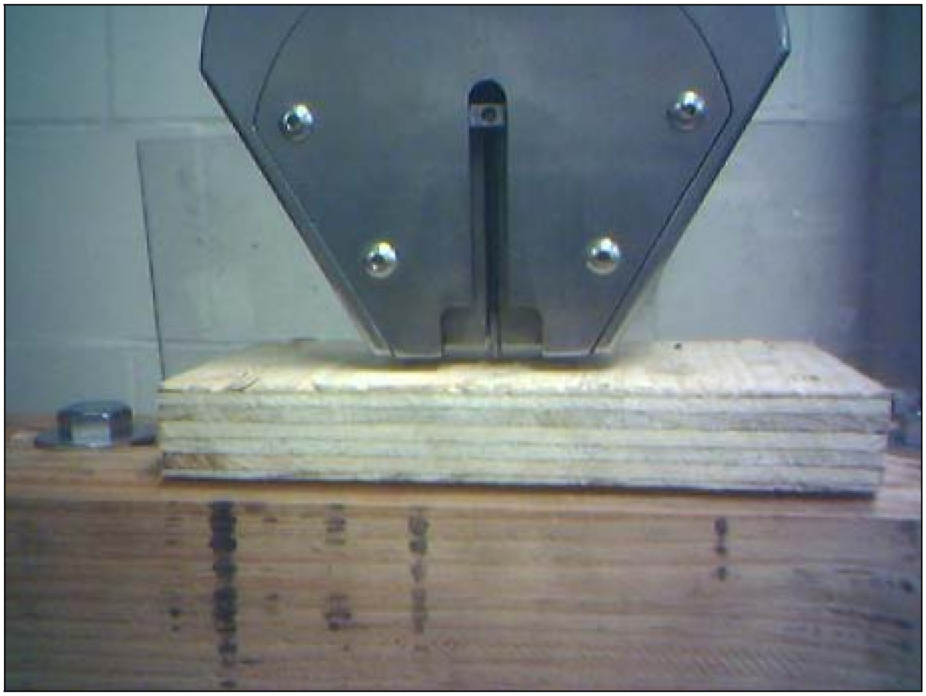
Description	Number	Minutes
Technology		
e-mail		
PDF		
PDA	X - meeting	
Type of Presentation		
Computer Presentation		
Slide Show		
Meeting Leader		
no formal speaker		
Dominant Speaker	X - Arch. & PM	
Meeting Materials		
Meeting Minutes	X	
RFI	X	
Change Orders	X	
Site Plans		
Material Samples		
Calendar		
Punch List	X	
Personal Binders	X	
Fit Out Schedule	X	
Phone Calls (during meeting)		
Unrelated Talks (when group conversation astrays)	X	3
	X	6
Extra Talks (when two people talk or when they go of tangents)		
	X	5
	X	2
	X	3
Others (Waste of Time)		
	Pasteries arrive late	

The entire results of tables can be found in the "meeting checklist" and questionnaire section of the Appendix.

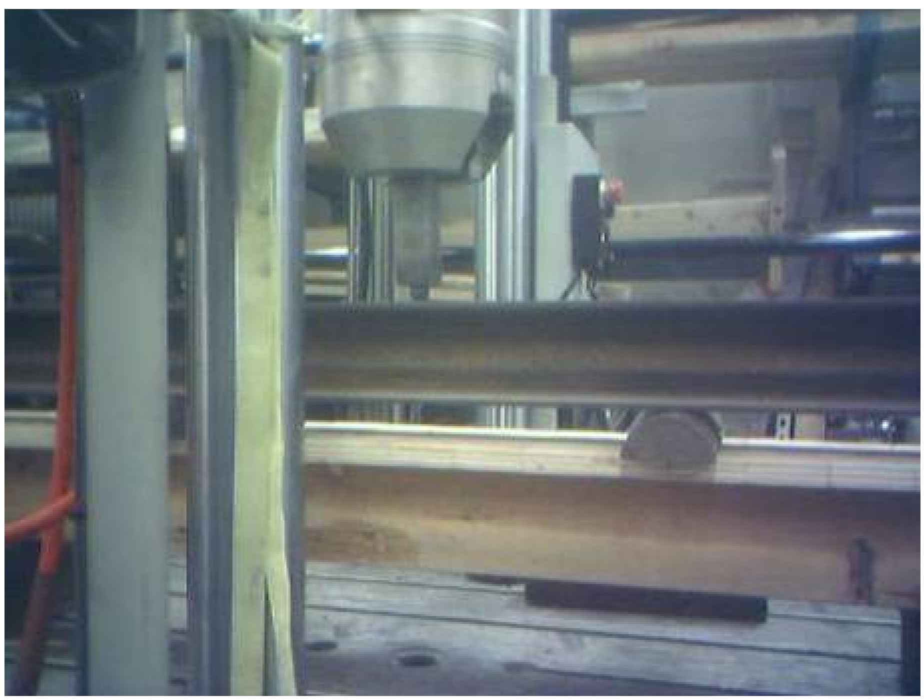
8.4 PICTURES

Picture 11: Tension Crack and Wood Floor

Picture 12: storage on second floor during crack mapping



Picture 13: Pullout Test



Picture 14: Bend Test – Girder & Plank

Picture 15: Girder Plank Bend Test

8.5 FIGURES

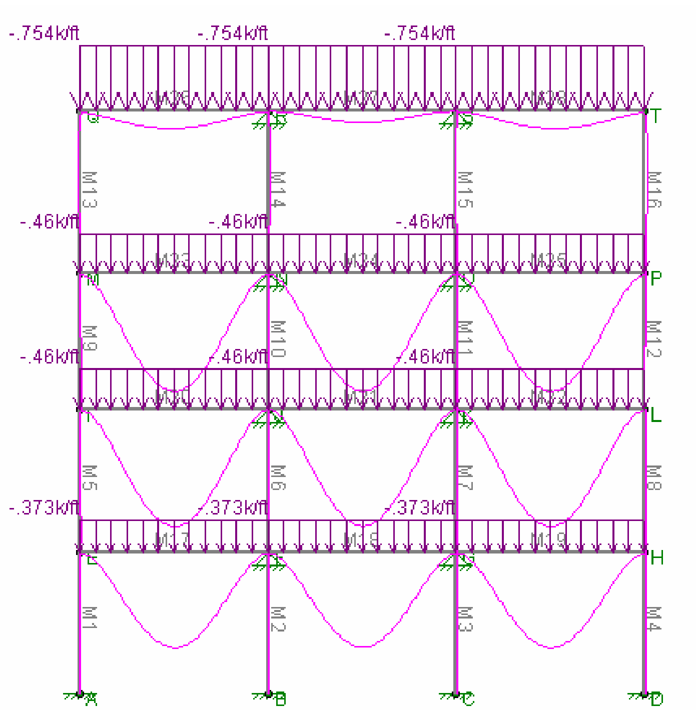


Figure 48: Deflection of Concrete - no wind

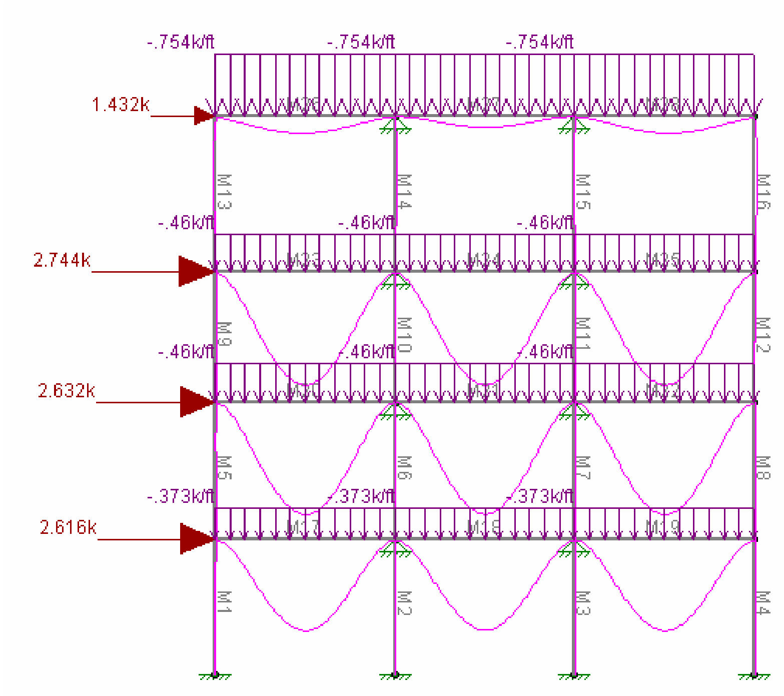


Figure 49: Deflection of Concrete - w/ wind

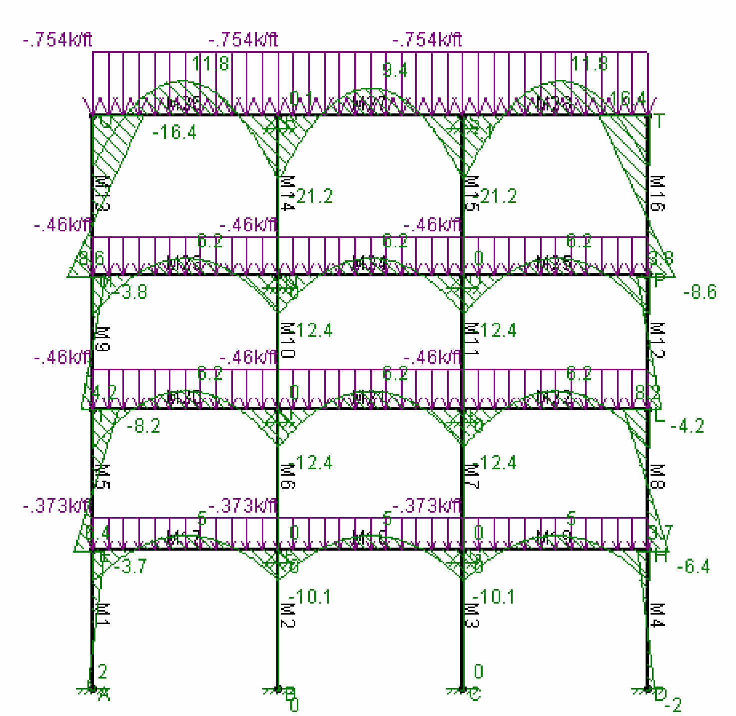


Figure 50: Moment in Concrete - no wind

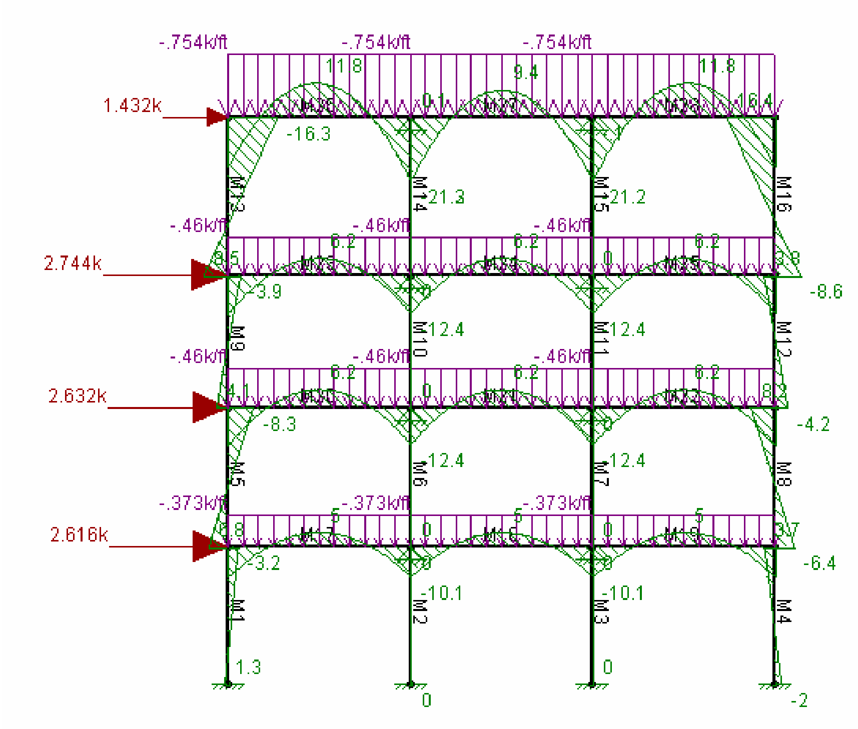


Figure 51: Moment in Concrete - w/ wind

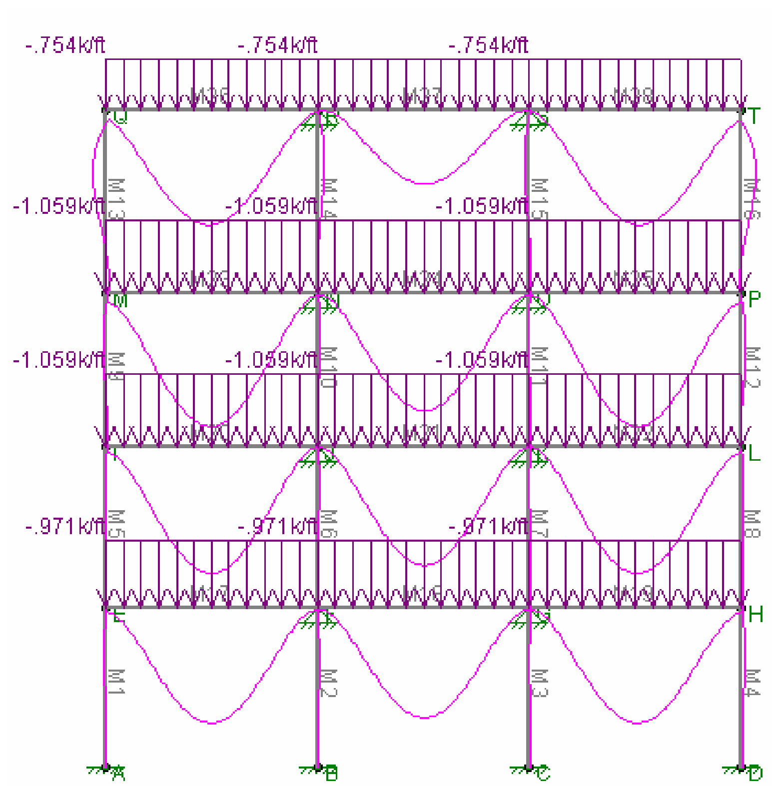


Figure 52: Deflection in Girder - no wind

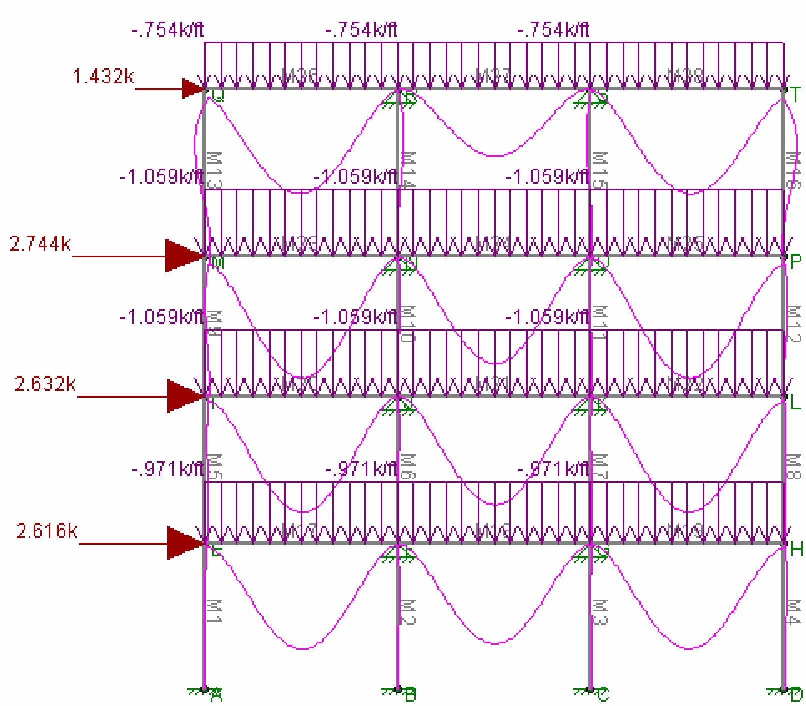


Figure 53: Deflection in Girder - w/ wind

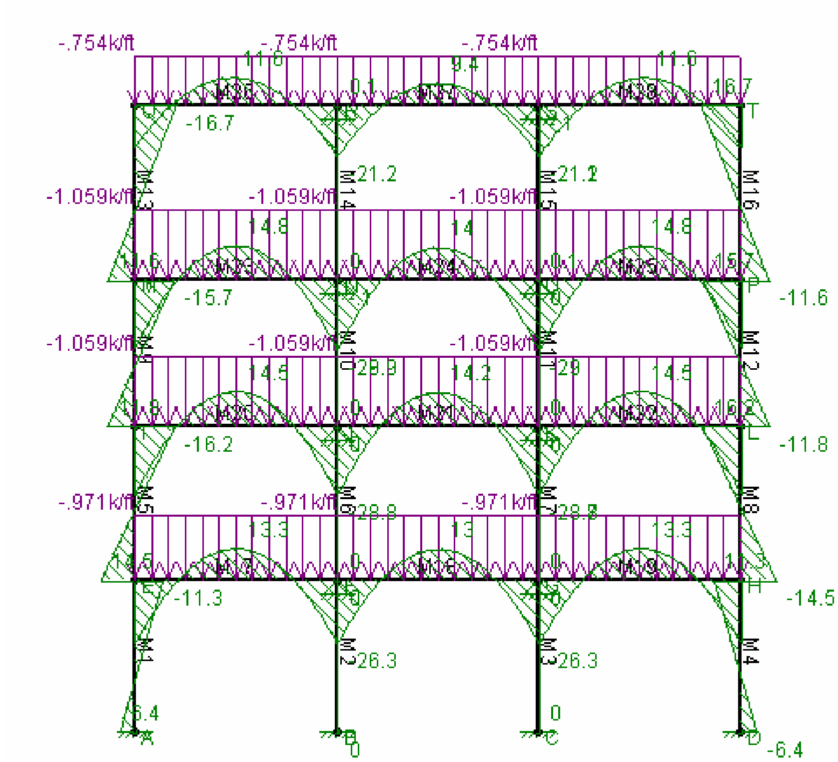


Figure 54: Moment in Girder - no wind

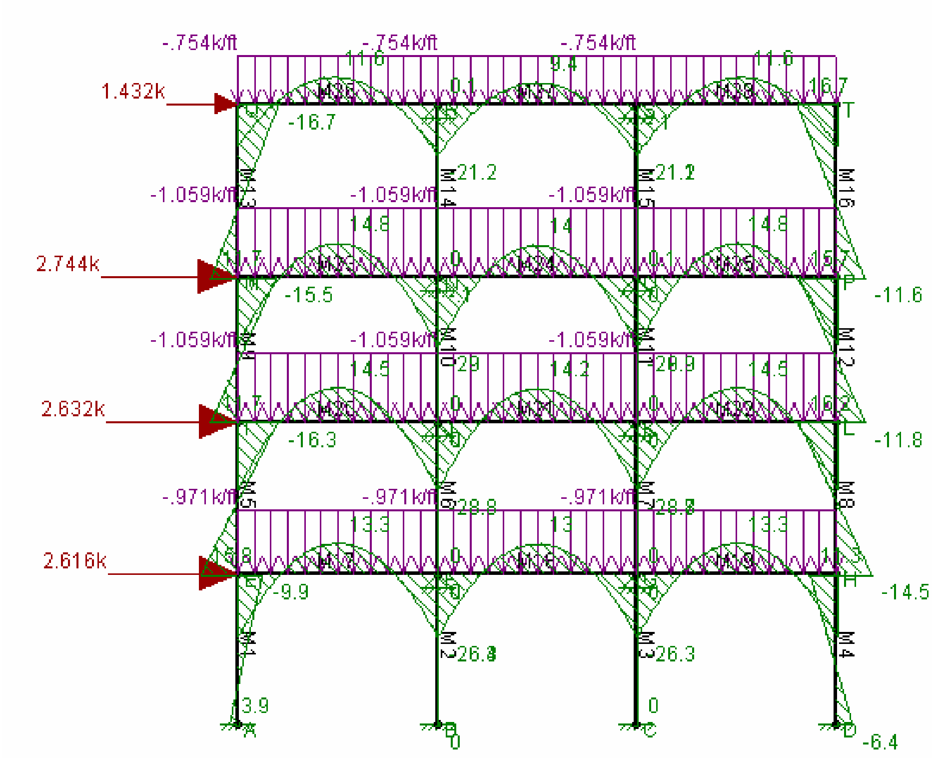


Figure 55: Moment in Girder - w/ wind

8.6 E-MAILS

Pete,

The purpose of the metal lath was to prevent the cementitous underlayment from separating and lifting off the existing wood floors. We poured half of the fourth floor on 2/23/06, and then completed the rest of the floors between 2/27 and 3/9. We had some water pressure issues that we had to deal with, so there was a gap between the pours. We typically started the pours mid-morning and finished mid-afternoon. The second half of the fourth floor and half the third floor were poured on the same day. The second half of the third floor and the entire second floor were poured in one day.

As discussed yesterday, we did have the compressive strength tested. The tests were performed by Thompson and Lichtner Company in early July and the method they used was ASTM C-109-06. The average result was approximately 5040 psi without the metal lath and 5363 psi with the metal lath.

I will forward your question regarding the sensitive equipment to the Structural Engineer and let you know when I hear from him.

Steve

E-Mail 1: Steve Johnson - October 3rd, 2006

Mehmet,

The beams in the existing building are 16" high and 10" wide. The southern yellow pine planks on the top of the beams are 3" thick and 7" wide. The layer of wood over the planks is 3/4" thick and was made out of maple in some areas.

Regarding the lath...I do not have the specs on the lath, but the attached picture might be able to help.

Steve

E-Mail 2: Steve Johnson - October 10th, 2006

Pete,

Sorry I missed you guys yesterday. We had a few issue pop up all in a row.

Regarding the roof...I can get the weight of the two rooftop units, but have no idea about the decking. We just ripped off the old roof and replaced it with a new rubber roof. We did not touch the decking.

The HVAC in the building...I could get a rough idea from our HVAC subcontractor.

I do not know what kind of brick was used on the original building, but there are a couple of places in the building where you can get a good look at the old brick (all layers in a cross section) if that will help.

I am at our main office today, but I will look into the HVAC and rooftop
info tomorrow.

Steve

E-Mail 3: Steve Johnson - October 17th, 2006

Peter:

Let me see if I can give you a few details, but feel free to call me at 508-831-5612 if you want more or just want to talk.

WPI had looked at the property down there as far back as probably 1986. We were looking for additional land for either new labs and classrooms or parking. We walked away from it then because of our concerns about environmental clean-up exposure. But the laws changed in 1998 and then in December of 1999 we got together with the WBDC and formed the LLC known as Gateway Park LLC which bought the first parcel of land down there for $2,700,000. The significant change in the law was that the buyer was no longer legally or financially responsible for the sins of previous owners. However, as owners we still had to clean up the site so we could develop it, and we used available public programs to fund that program, but at least there was no tail to the purchase where we would have liability for what others had done over the years. We continued to buy parcels in the area over succeeding years to round out the 11.5 acres we now own. I believe we did the master plan in 2000-2001 and the zoning change with the city which moved us to the BG-6 category was probably completed in 2002. That zoning change was a key because it is the most flexible zoning category in the city and it gives us great flexibility as we develop the area.

Does that help? Questions? I'll be on campus until Thursday noon, then unavailable the rest of the week, but available by either email or cell phone (508-889-3028) next week.

Good luck and if I don't talk with you, best wishes for Happy Holidays.

Steve

E-Mail 4: Steve Hebert - December 12th, 2006

8.7 TEST RESULTS

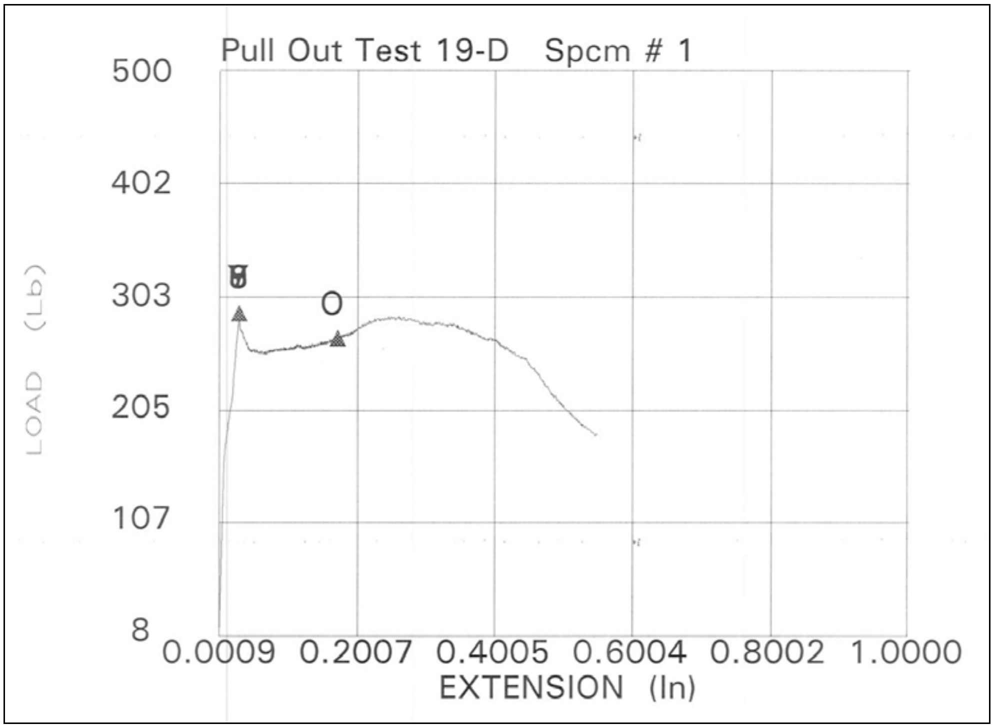


Figure 56: Pullout test-19d; 289.3 lb

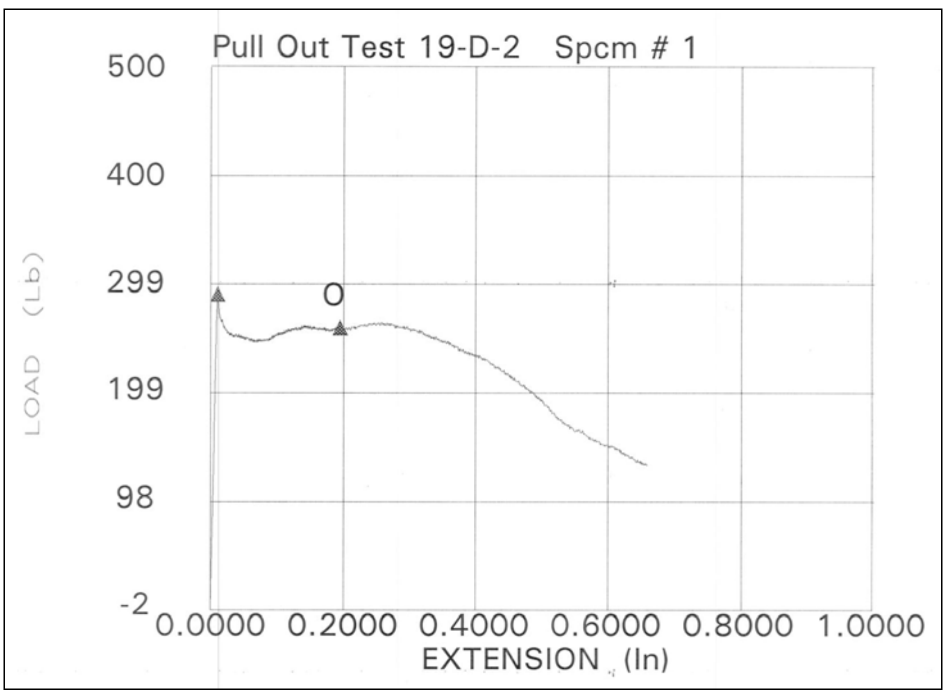


Figure 57: Pullout Test - 19d; 270 lb

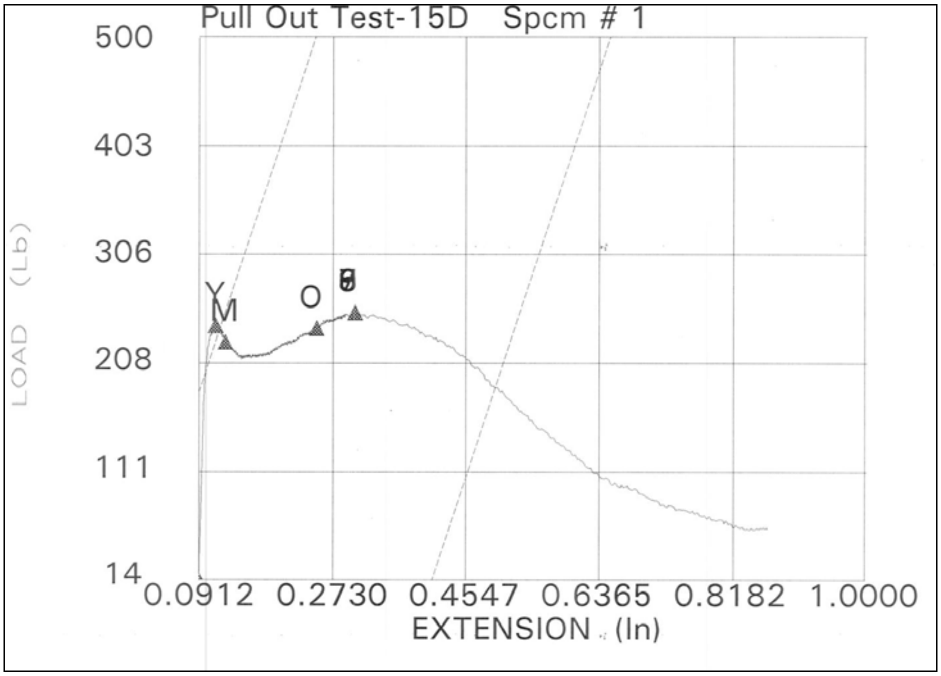


Figure 58: Pullout Test - 15d; 253.1 lb

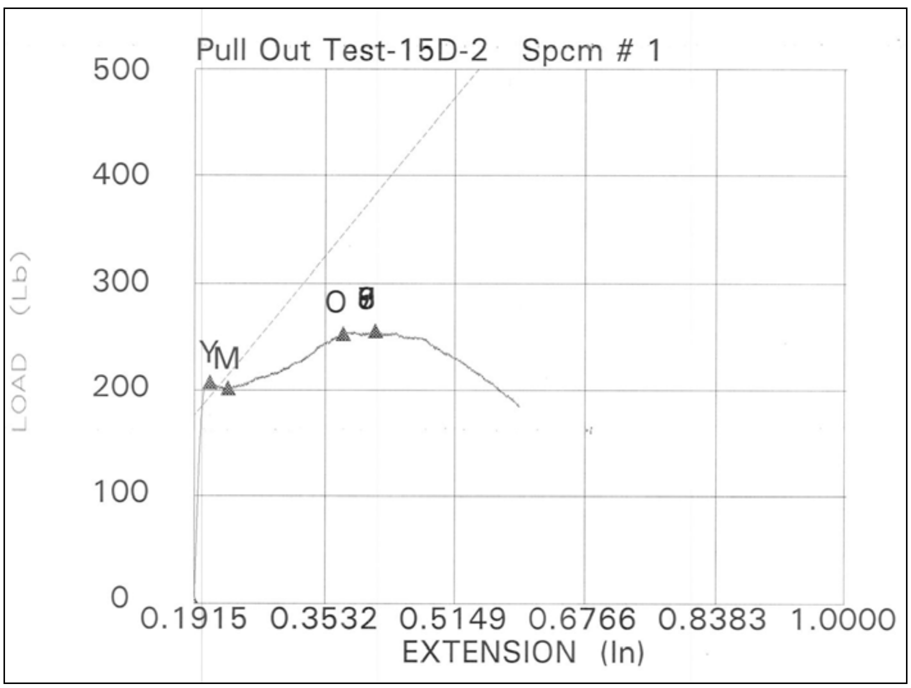


Figure 59: Pullout Test - 15d; 255.6 lb

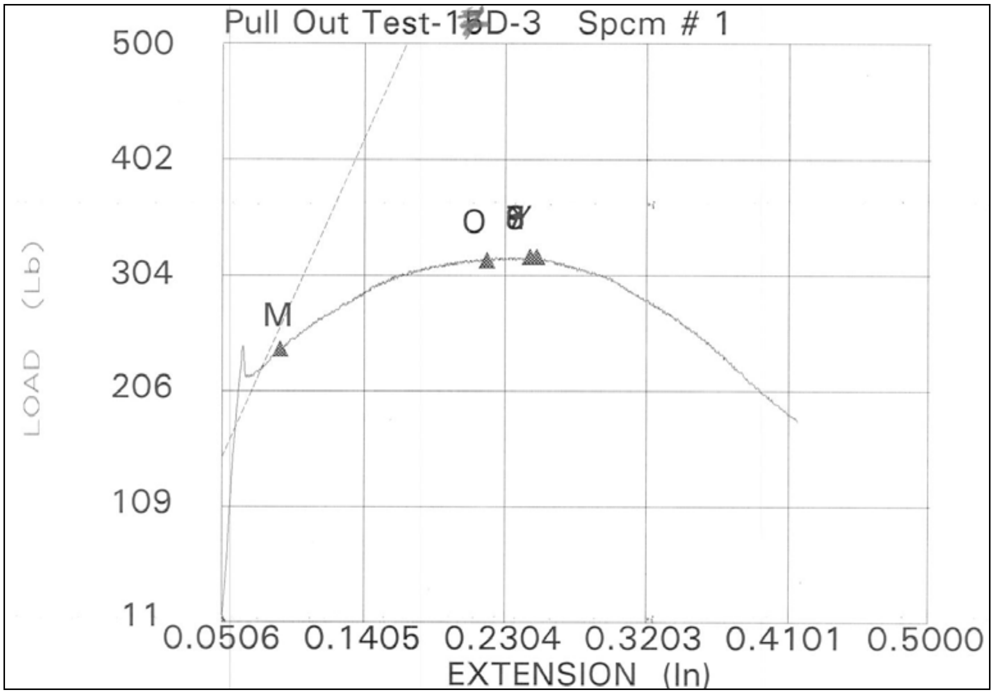


Figure 60: Pullout Test - 17d; 320.4 lb

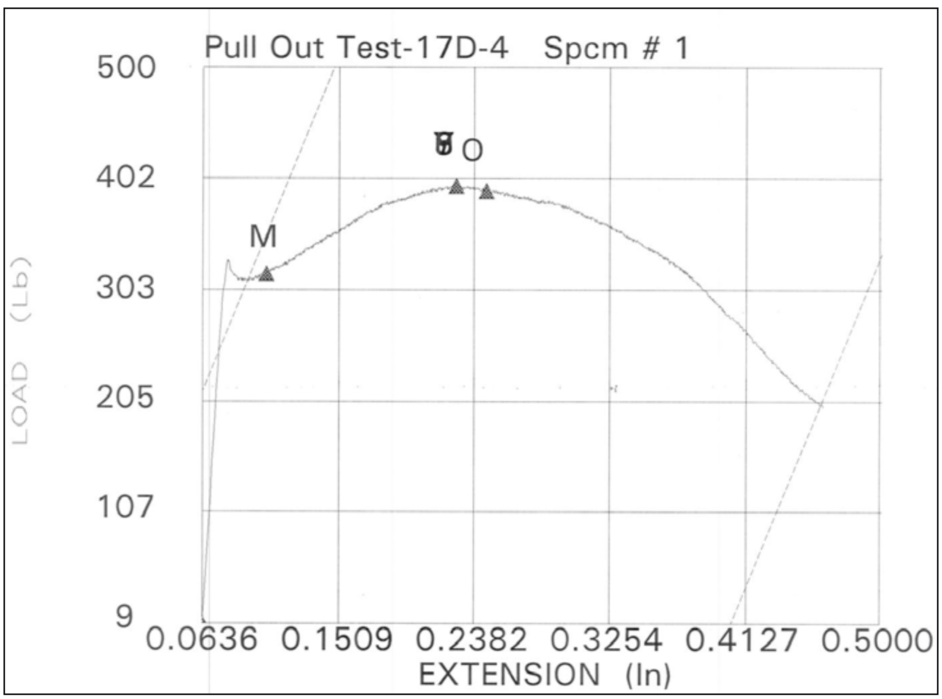


Figure 61: Pullout Test - 17d; 395.5 lb

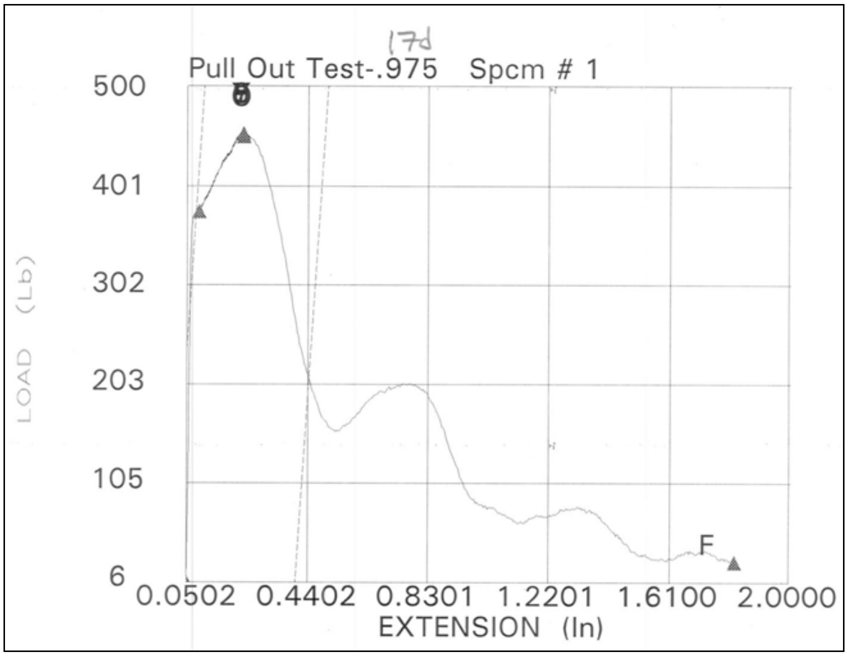


Figure 62: Pullout Test - 17d; 453.1 lb

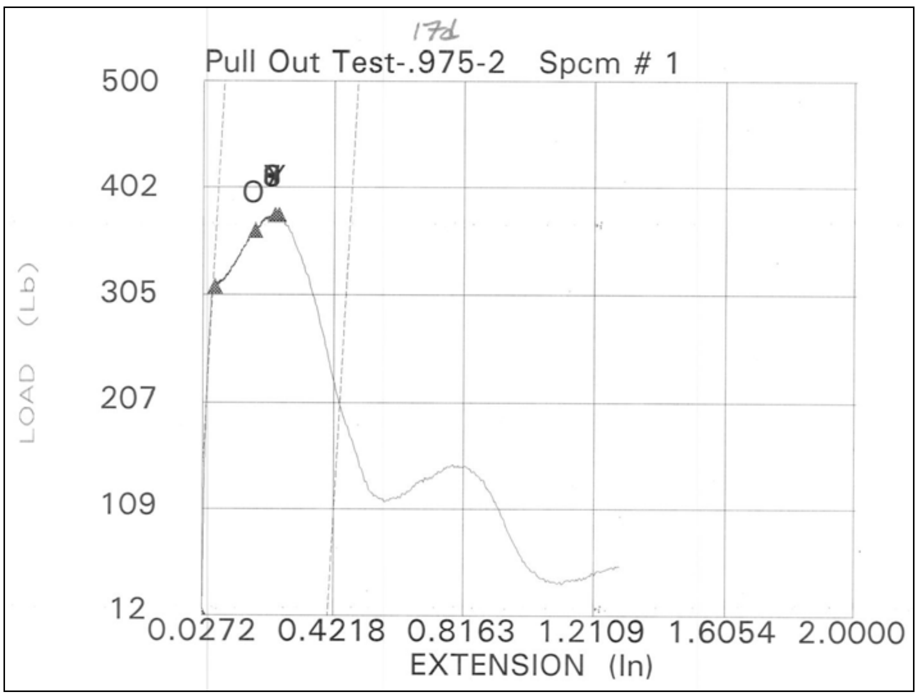


Figure 63: Pullout Test - 17d; 377 lb

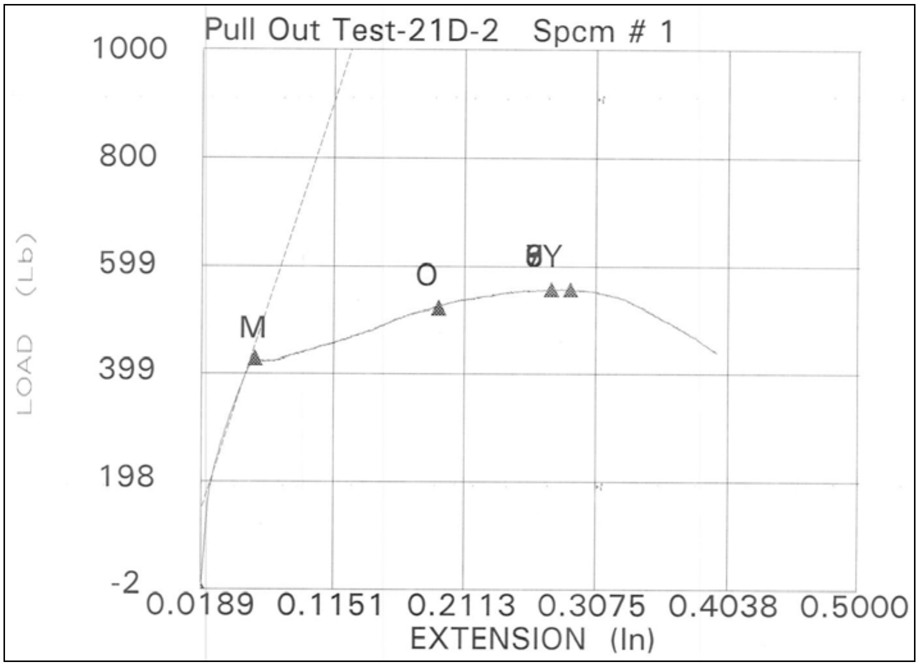


Figure 64: Pullout Test - 21d; 556.5 lb

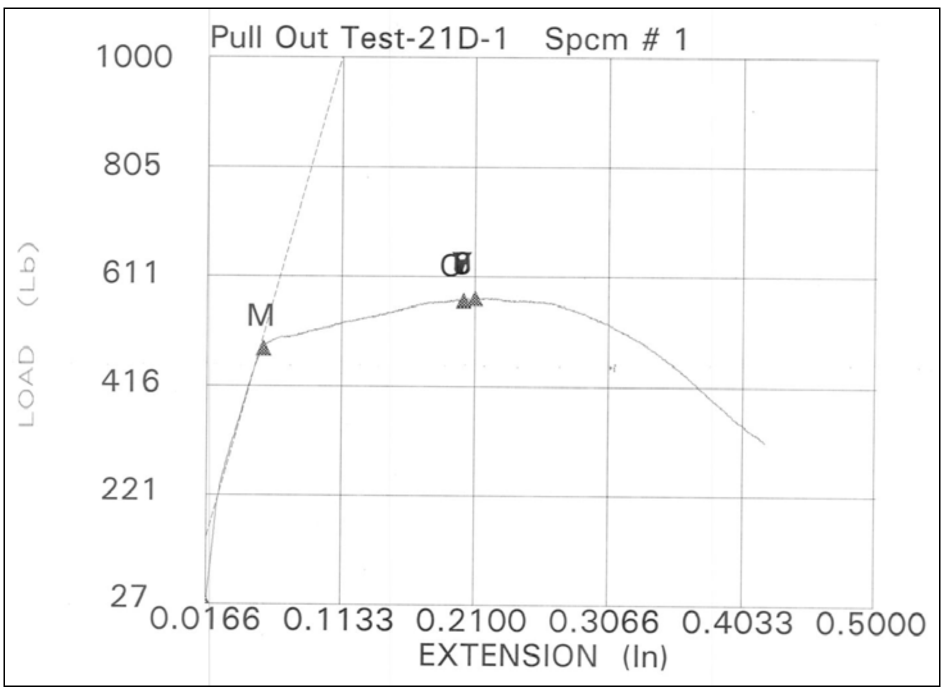


Figure 65: Pullout Test - 21d; 573 lb

$\frac{650\ lb}{27\ Cu.F} = \frac{x}{2.2}$ Mehmet0.5.xls 1 cu yd = 27 cu Ft

Unit Volume 1yd $\frac{2.2}{27}$ = lb/2.2 CFF

high silica fume for high strength concrete

Mix Date		24-Jan-05				
w/c		0.5				
silica fume(%C)		0%				
slump		5-6 "				
Air		6.00%				
Course Aggregate Properties						
Sp.Gr.		2.7				
Max Agg Size(in)		3/8 in				
Dry U.W.(lb)		100				
M.C.(%)		0.00%				
Absorption(%)		0.90%		7 X 1/2	= 3 1/2	Cu. FT.
Fine Aggregate Properties						
Sp. Gr.		2.645				
F.M.		2.7				
Absorption(%)		2.50%				
M.C. (%)		0.00%		1 Cy 6X12	= 0.2	Cu. FT.
Mix Design Calculations						
Water (T9.5), lbs		325				
Cement		650.00		Cylinder = 3	plus	
Silica Fume		0.00				
CA Vol% (T9.4)		61.00%				
CA wt dry		1,647.00				
Volume Calculations						
Cement Sp. Gr.						
Vol Water (cf)		5.21				
Vol Cement (cf)		3.31				
Vol Silica Fume		0.00	2 Batches			
Vol C.A. (cf)		9.78				
Vol Air (cf)		1.62			no more then 3	
Subtotol Vol (cf)		19.91				
Vol F.A. (cf)		7.09				
Wt F.A.(dry), lbs		1,170.05	Laboratory Mix		lbs	2.5
Corrected Weights		lb/y3	2.2cf/batch	CU FT		
Cement (lbs)		650.00	52.96	POUNDS	.53	Cement 560.19
Silica Fume		0.00	0.00	POUNDS		0.00
CA (lbs)		1,647.00	134.20	POUNDS	134 C.A.	→ 152.50
FA (lbs)		1,170.05	95.34	POUNDS	95.5 F.A	sand → 108.34
Water		369.07	30.07	POUNDS	30 H_2O	→ 34.17
AEA(oz/100 # cwt)		→ 2	0.16	POUNDS		.19
AEA (ml)		384.41	31.32	ML air	Entrainment	→ 35.59mL
H2O Added (lb)				POUNDS		
WR (Oz/100# cwt)		1	0.08	POUNDS		.09
WR (ml)		192.21	15.66	ML water reducer.		← if slump hi 17.80mL
Quality Control Data						
Unit Wt(pcf)						
Air Fresh (%)	Pressuremeter					skip
Air Fresh (%)	Chase meter					WR
Slump (in)						
Temp C to (F)						
Observations	Workability	Cohessiveness				

lb/y3

y3

look don't use

Mehmet0.5.xls ce3026 12/19/06

Figure 66: Concrete Mix Design

Worc. Polytechnic Inst.

I00 Institute Road

Worc. Mass. 01609

Company:	M&P Concrete	Name:	
Lab name:	WPI	Number of specimens:	1
Operator ID:		Temperature:	
Test date:	2/2/07	Humidity:	
		Speed 1:	0.05 in/min

Note 1: Clouston-2 *

Note 2:

Note 3:

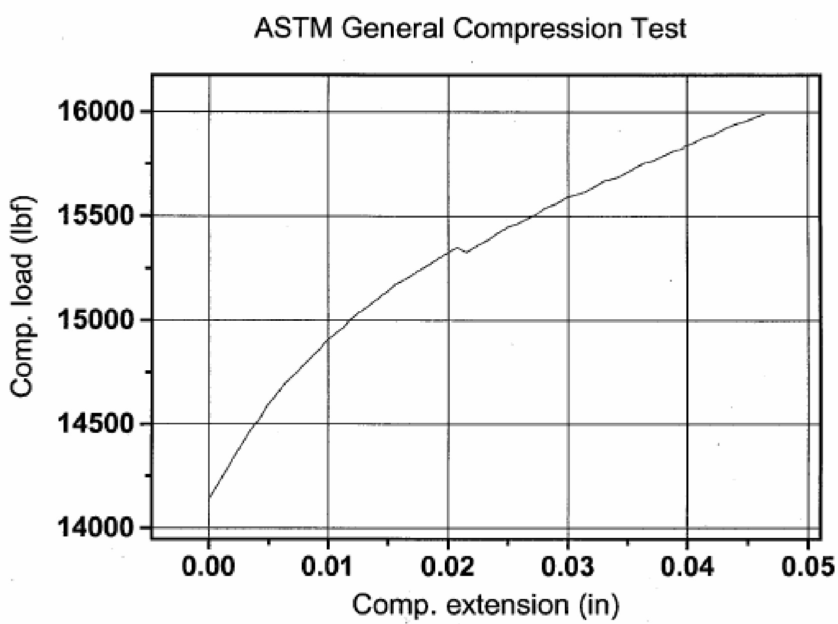


Figure 67: Clouston test

Worc. Polytechnic Inst.

100 Institute Road

Worc. Mass. 01609

Company:	M&P Concrete	Name:		
Lab name:	WPI	Number of specimens:	1	
Operator ID:		Temperature:		
Test date:	2/1/07	Humidity:		
		Speed 1:	0.05	in/min

Note 1: ~~[illegible] 2~~
1st principle

Note 2:

Note 3:

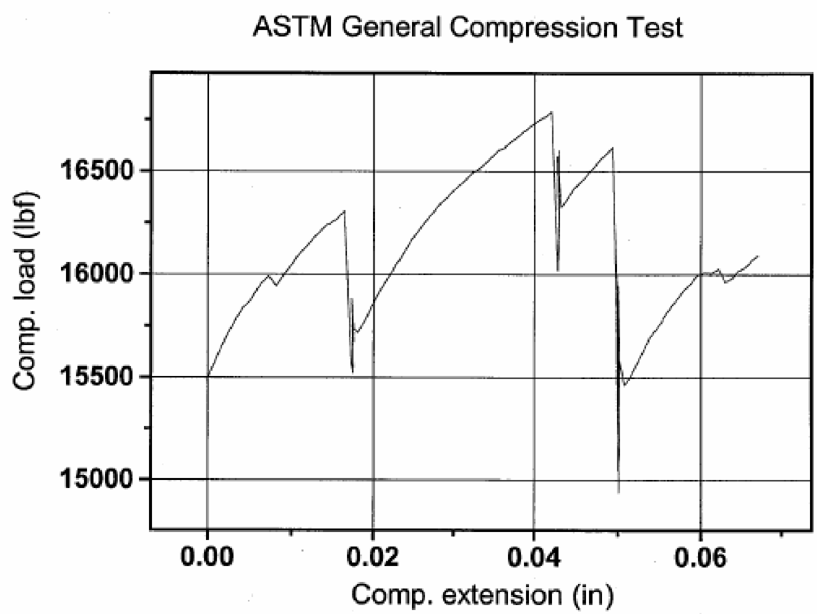


Figure 68: First Principles Test

Worc. Polytechnic Inst.

100 Institute Road

Worc. Mass. 01609

Company:	M&P Concrete	Name:	
Lab name:	WPI	Number of specimens:	1
Operator ID:		Temperature:	
Test date:	1/31/07	Humidity:	
		Speed 1:	0.05 in/min

Note 1: Clouston

Note 2:

Note 3:

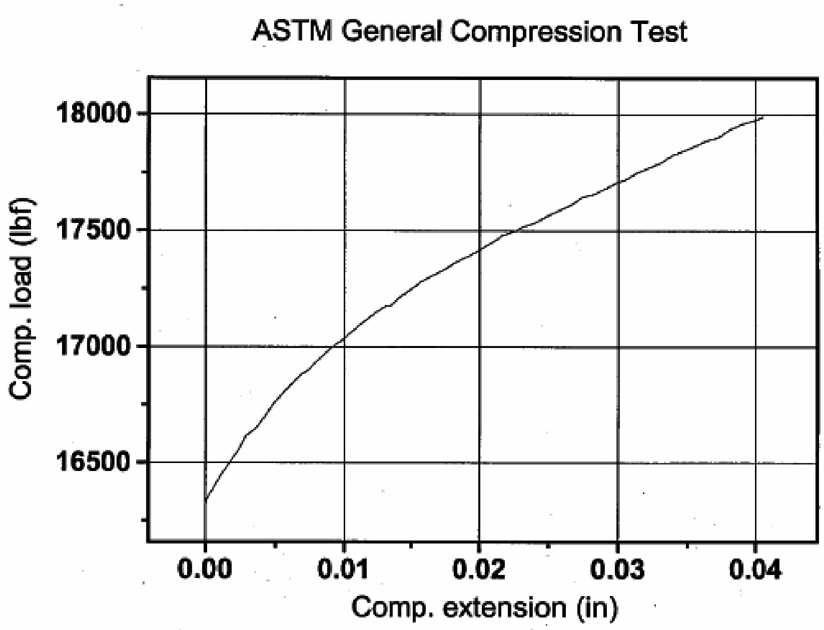
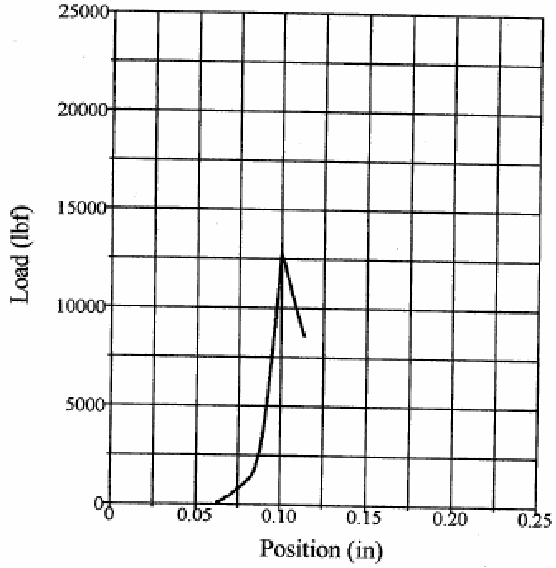


Figure 69: Clouston Test



Test Results

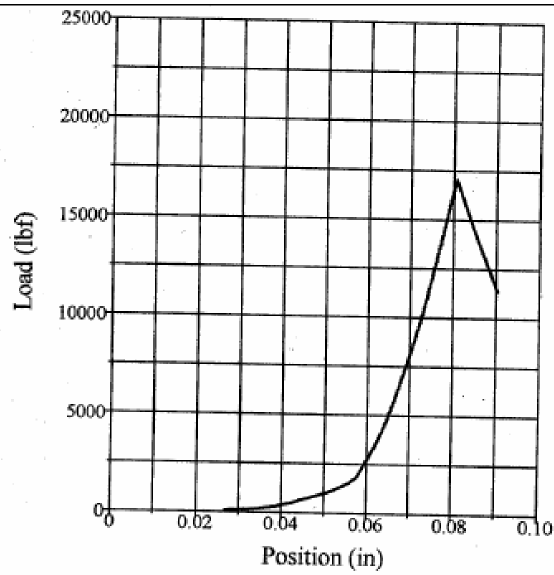
Diameter: **3.0000** in
Area: **7.0686** in^2
Peak Load: **12710** lbf

$E\varepsilon = \sigma_T = \frac{2\times 12710}{\pi \times 6 \times 3} = 449.5$

Test Summary

Counter:	**876**
Elapsed Time:	**00:00:26**
Material:	**Concrete**
Lab:	
Sample:	**Split Tensile-1**
Specimen Identification:	**M&P MQP**
Procedure Name:	**dpel.Concrete Split Tensile**
Start Date:	**02/22/2007**
Start Time:	**3:03:18 PM**
End Date:	**02/22/2007**
End Time:	**3:03:44 PM**
Workstation:	**Worcester Polytechnic Inst**
Tested By:	**default**

Figure 70: Concrete Split Cylinder Test 1



Test Results

Diameter:	**3.0000** in
Area:	**7.0686** in^2
Peak Load:	**17072** lbf

$\frac{11 PL^3}{384EI}$

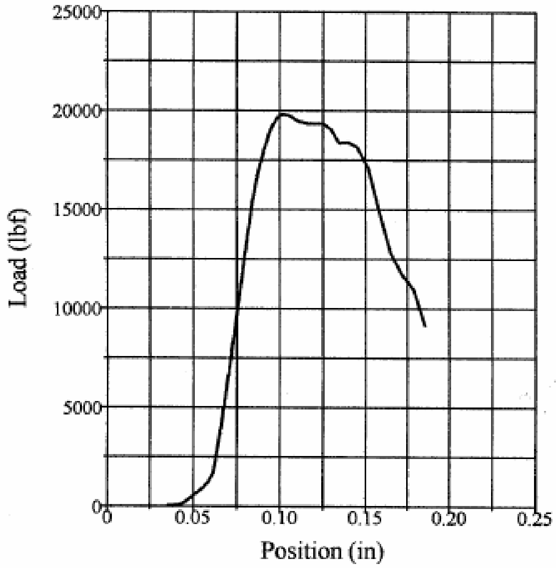
$\sigma_t = \frac{2P}{\pi DL} = \frac{2 \times 17072}{\pi \times 3 \times 6} = 603$ psi

Test Summary

Counter:	**875**
Elapsed Time:	**00:00:34**
Material:	**Concrete**
Lab:	
Sample:	**Split Tensile-2**
Specimen Identification:	**M&P MQP**
Procedure Name:	**dpel.Concrete Split Tensile**
Start Date:	**02/22/2007**
Start Time:	**2:43:11 PM**
End Date:	**02/22/2007**
End Time:	**2:43:45 PM**
Workstation:	**Worcester Polytechnic Inst**
Tested By:	**default**

=

Figure 71: Split Cylinder Test #2



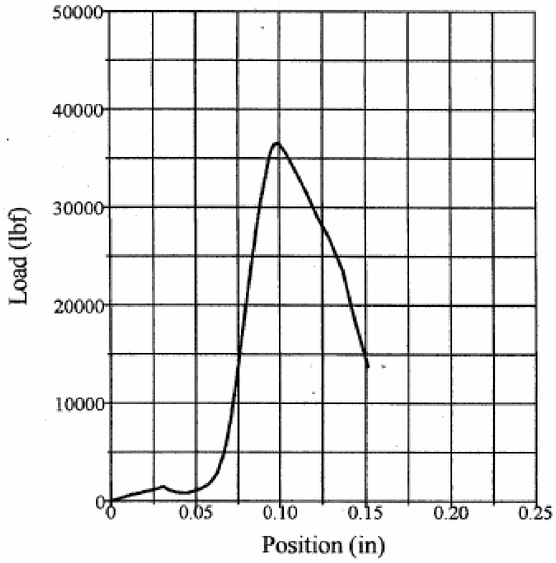
Test Results

Diameter:	**3.0000** in
Area:	**7.0686** in^2
Peak Load:	**19812** lbf
Work:	**1898** in·lb

Test Summary

Counter:	**873**
Elapsed Time:	**00:00:32**
Specimen Identification:	**2-1**
Material:	**Concrete**
Comments:	**M&P MQP**
Procedure Name:	**6x12ConcreteCyl.-1PRC**
Start Date:	**02/22/2007**
Start Time:	**2:30:51 PM**
End Date:	**02/22/2007**
End Time:	**2:31:23 PM**
Workstation:	**Worcester Polytechnic Inst**
Tested By:	**default**

Figure 72: Compression Test 2-1



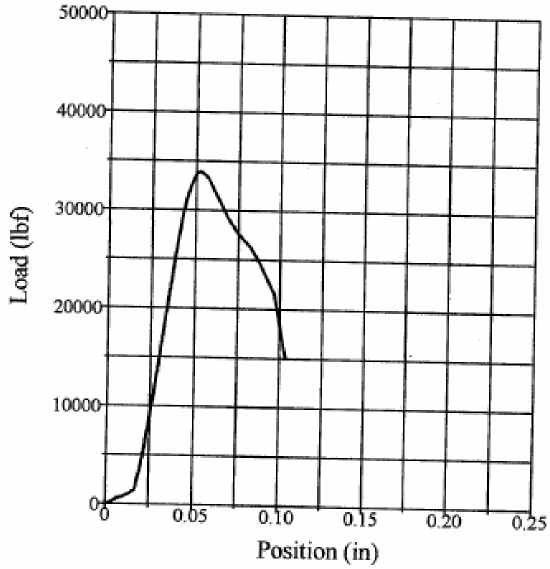
Test Results

Diameter:	**3.0000** in
Area:	**7.0686** in²
Peak Load:	**36547** lbf
Work:	**2242** in·lb

Test Summary

Counter:	**874**
Elapsed Time:	**00:00:57**
Specimen Identification:	**2-2**
Material:	**Concrete**
Comments:	**M&P MQP**
Procedure Name:	**6x12ConcreteCyl.-1PRC**
Start Date:	**02/22/2007**
Start Time:	**2:37:30 PM**
End Date:	**02/22/2007**
End Time:	**2:38:27 PM**
Workstation:	**Worcester Polytechnic Inst**
Tested By:	**default**

Figure 73: Compression Test 2-1



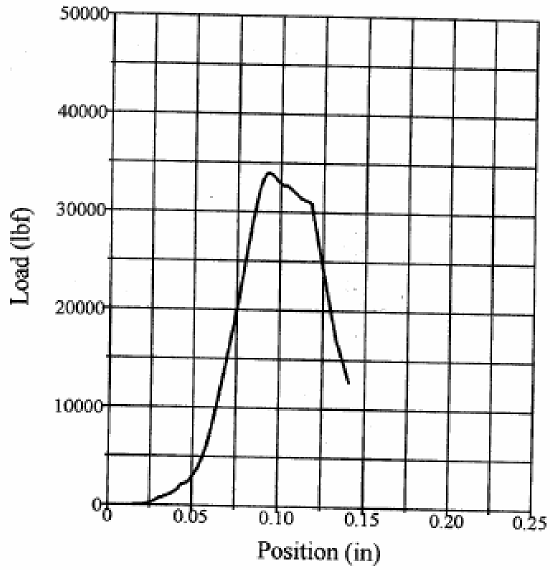
Test Results

Diameter:	**3.0000** in
Area:	**7.0686** in^2
Peak Load:	**33921** lbf
Work:	**2083** in·lb

Test Summary

Counter:	**878**
Elapsed Time:	**00:00:53**
Specimen Identification:	**1-2**
Material:	**Concrete**
Comments:	**M&P MQP**
Procedure Name:	**6x12ConcreteCyl.-1PRC**
Start Date:	**02/22/2007**
Start Time:	**3:21:19 PM**
End Date:	**02/22/2007**
End Time:	**3:22:12 PM**
Workstation:	**Worcester Polytechnic Inst**
Tested By:	**default**

Figure 74: Compression Test 1-2



Test Results

Diameter:	**3.0000** in
Area:	**7.0686** in^2
Peak Load:	**33993** lbf
Work:	**2081** in·lb

Test Summary

Counter:	**877**
Elapsed Time:	**00:00:53**
Specimen Identification:	**1-1**
Material:	**Concrete**
Comments:	**M&P MQP**
Procedure Name:	**6x12ConcreteCyl.-1PRC**
Start Date:	**02/22/2007**
Start Time:	**3:17:33 PM**
End Date:	**02/22/2007**
End Time:	**3:18:26 PM**
Workstation:	**Worcester Polytechnic Inst**
Tested By:	**default**

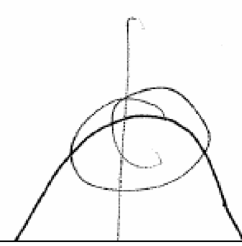
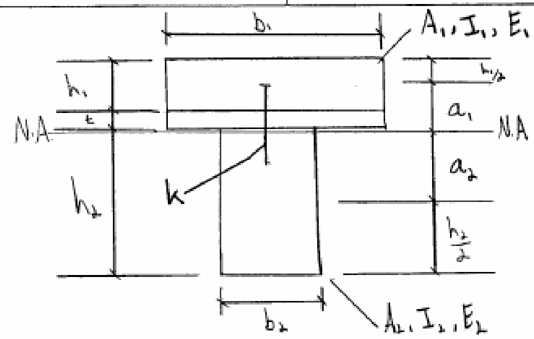


Figure 75: Compression Test 1-1

8.8 DESIGN CALCUTION AND COST ESTIMATE

DESIGN CALCULATION:

Clouston Method



$E_{wood} = 2500$ ksi
$E_{conc} = 3990$ ksi
$h_{wood} = 5.5$ in
$b_{wood} = 3.5$ in
$h_{conc} = 1.8$ in
$b_{conc} = 8.25$ in
$L: 55$ in
$k = 7502$ psi/in
$s = 3$ in

$$(EI)_{eq} = \sum_{i=1}^{2} \left(E_i I_i + \gamma_i E_i A_i a_i^2\right)$$

$$\gamma_1 = \frac{1}{1 + \frac{\pi^2 E_1 A_1 s}{kL^2}} = \frac{1}{1 + \frac{\pi^2 (3990\times10^3)(1.8\times 8.25)(3)}{7502 \times 55^2}} = 0.013$$

$$\gamma_2 = 1$$

$$a_1 = 1$$

$$a_2 = \frac{\gamma_1 E_1 A_1 (h_1 + h_2 + 2t)}{2\sum_{i=1}^{2} \gamma_i E_i A_i} = \frac{0.013(3990\times10^3)(1.8\times8.25)(1.8+5.5+2\times1)}{2\sum_{i=1}^{2}\left(0.013(3990\times10^3)(1.8\times8.25)\right)}$$

$$= 0.07 \text{ in}$$

$$(EI)_{eq} = \left[(3990\times10^3)\left(\frac{1.8^3\times 8.25}{12}\right) + (1)(3990\times10^3)(1.8\times8.25)(1)^2\right] + \left[(2500\times10^3)\left(\frac{3.5\times5.5^3}{12}\right) + (1)(2500\times10^3)(3.5\times5.5)(0.07)^2\right]$$

$$\boxed{= 1.4\times10^8 \text{ lb-in}^2}$$

Figure 76: Clouston Method Calculations

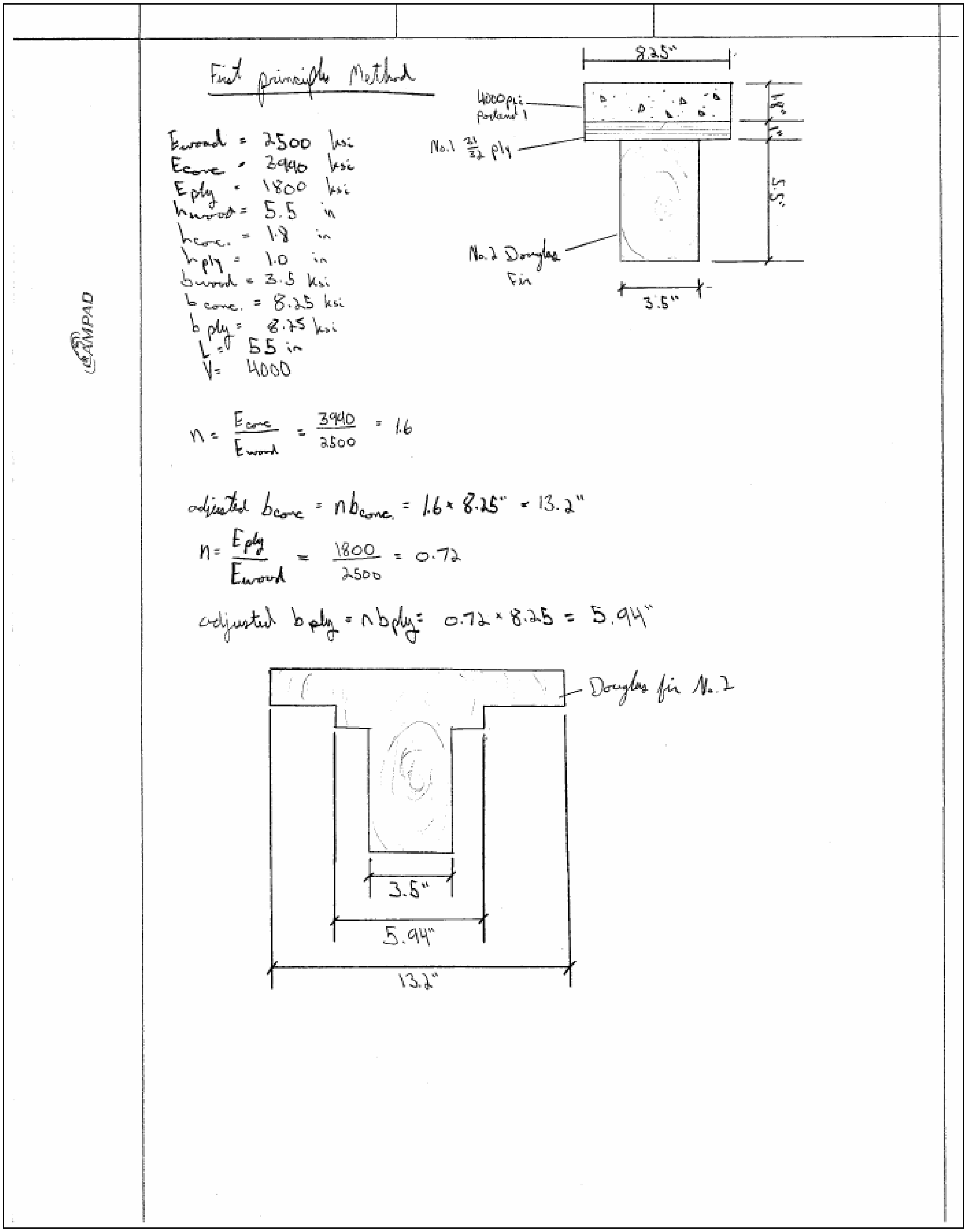


Figure 77: First Principles Hand Calc #1

$$\bar{y} = \frac{\Sigma \bar{y} A}{\Sigma A} = \frac{\left[\left(\frac{h_{wood}}{2}\right)\left(h_{wood} \times b_{wood}\right)\right] + \left[\left(h_{wood} + \frac{h_{ply}}{2}\right)\left(h_{ply} \times b_{ply}\right)\right] + \left[\left(h_{wood} + h_{ply} + \frac{h_{core}}{2}\right)\left(h_{core} \times b_{core}\right)\right]}{\left(b_{wood} \times h_{wood}\right) + \left(b_{ply} \times h_{ply}\right) + \left(b_{core} \times h_{core}\right)}$$

$$= \frac{\left[\left(\frac{5.5}{2}\right)\left(3.5 \times 5.5\right)\right] + \left[\left(5.5 + \frac{1}{2}\right)\left(1 \times 5.92\right)\right] + \left[\left(5.5 + 1 + \frac{1.8}{2}\right)\left(1.8 \times 13.2\right)\right]}{\left(5.5 \times 3.5\right) + \left(1 \times 5.92\right) + \left(1.8 \times 13.2\right)}$$

$$= 5.4''$$

$$I_{NA} = \Sigma\left(\bar{I} + Ad^2\right) = \left[\tfrac{1}{12}(b_{wood})(h_{wood})^3 + (b_{wood})(h_{wood})\left(\bar{y} - \frac{h_{wood}}{2}\right)^2\right]$$
$$+ \left[\tfrac{1}{12}(b_{ply})(h_{ply})^3 + (b_{ply})(h_{ply})\left(\bar{y} - \left(h_{wood} + \frac{h_{ply}}{2}\right)\right)^2\right]$$
$$+ \left[\tfrac{1}{12}(b_{core})(h_{core})^3 + (b_{core})(h_{core})\left(\bar{y} - \left(h_{wood} + h_{ply} + \frac{h_{core}}{2}\right)\right)^2\right]$$

$$= \left[\tfrac{1}{12}(3.5)(5.5)^3 + (3.5)(5.5)\left(5.4 - \frac{5.5}{2}\right)\right] + \left[\tfrac{1}{12}(5.92)(1)^3 + (5.92)(1)\left(\left(5.5 + \tfrac{1}{2}\right) - 5.4\right)^2\right]$$
$$+ \left[\tfrac{1}{12}(13.2)(1.8)^3 + (13.2)(1.8)\left(\left(5.5 + 1 + \frac{1.8}{2}\right) - 5.4\right)^2\right]$$

$$= 284.4 \text{ in}^4$$

$$\tau_{max} = \frac{VQ}{It} = \frac{4000 \text{ lbs} \times 47.1 \text{ in}^3}{284.4 \text{ in}^4 \times 3.5 \text{ in}} = \boxed{189.4 \text{ psi}}$$

$$Q = \bar{y}' A' = \left(h_{total} - \left(\frac{h_{core}}{2}\right) - \bar{y}\right)\left(h_{core} \times b_{core}\right)$$
$$= \left(\left(5.5 + 1 + 1.8\right) - \frac{1.8}{2}\right) - 5.4\right)\left(1.8 \times 13.2\right)$$
$$= 47.1 \text{ in}^3$$

Figure 78: First Principles Calcs. #2

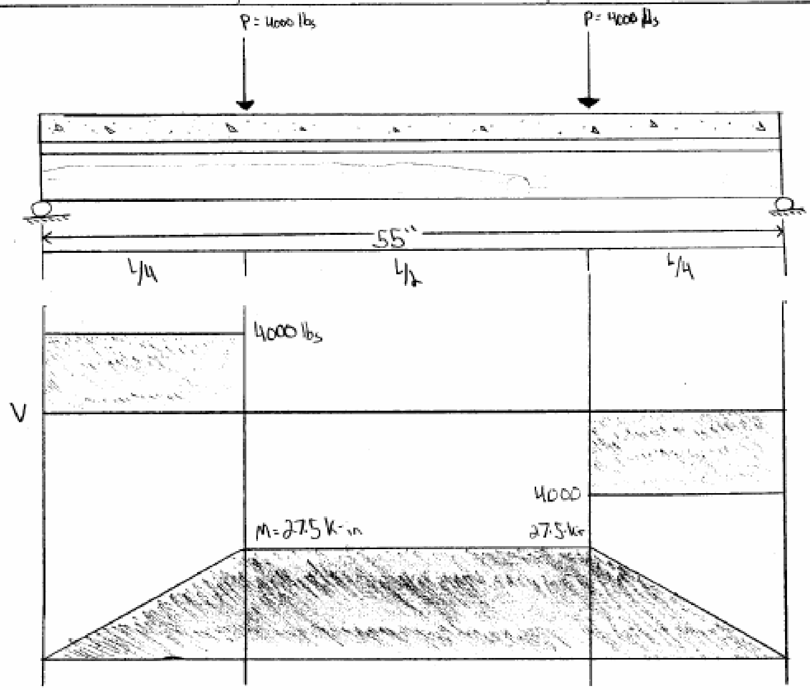


Figure 79: Shear and Moment Diagrams

COST ESTIMATION:

Table 63: Concrete Cost

CONCRETE

Description	**Amount**
Average Thickness (in)	2
Area per floor (ft^2)	7920
Total Area (ft^2)	23760
Concrete Volume (ft^3)	3960
Concrete Volume (CY)	146.7
Description	**Cost**
Concrete ($/CY)	$96
Concrete	$14,080
Description	**Cost**
Labor per Hour	$62
Pumping	$2,976
Placing	$11,904
Finishing	$11,904
Labor (Equip. included)	$26,784
Description	**Cost**
Concrete	$14,080
Labor	$26,784
Total Cost	$40,864

Table 64: Point Connector Cost

Point Connector

NAIL	**3 inches**	**2 inches**	**1 inches**
Description	**Length (in)**	**Length (in)**	**Length (in)**
Girder Size	216	216	216
Nailing Space	3	2	1
Description	**Amount**	**Amount**	**Amount**
Nails per girder	72	108	216
Girders per floor	53	53	53
Nails per floor	3816	5724	11448
Nails for 68 Pres.	11448	17172	34344
10% allowance	1145	1717	3434
Total Nails	12593	18889	37778
Description	**Weight (lb)**	**Weight (lb)**	**Weight (lb)**
Nail	0.01	0.01	0.01

Total Nails	166.57	249.86	499.72
Description	**Cost**	**Cost**	**Cost**
5lb Nail Box	$11	$11	$11
1lb Nail Box	$2	$2	$2
Total Nails	$366	$550	$1,099

LABOR	**3 inches**	**2 inches**	**1 inches**
Description	**Time (min)**	**Time (min)**	**Time (min)**
Nails per girder	15	20	25
Nails per floor	795	1060	1325
Nails for 68 Pres.	2385	3180	3975
10% allowance	238.5	318	397.5
Total Nails	2623.5	3498	4372.5
Description	**Time (hr)**	**Time (hr)**	**Time (hr)**
Labor Hours	44	58	73
Description	**Cost**	**Cost**	**Cost**
Labor/Hour	$62	$62	$62
Total Labor	$2,711	$3,615	$4,518

Description	**Cost (3in)**	**Cost (2in)**	**Cost(1in)**
Labor	$2,711	$3,615	$4,518
Nail	$366	$550	$1,099
Nail gun	$450	$450	$450
Wiremesh	$11,880	$11,880	$11,880
Concrete	$40,864	$40,864	$40,864
Preparation	$15,000	$15,000	$15,000
Total	$71,271	$72,358	$73,812

Table 65: Continuous Connector Cost

Continuous Connector

WIREMESH

Description	**Length (ft)**
Girder per floor	954
Mesh per floor	954
Mesh for 68 Pres.	2862
10% allowance	286
Total Mesh Length	3148
Description	**Cost**
Mesh per girder	$3
Mesh per floor	$159

Mesh for 68 Pres.	$477

LABOR

Description	**Time (min)**
Mesh per girder	75
Cut	40
Vacuum	10
Glue	15
Mesh	10
Mesh per floor	3975
Mesh for 68 Pres.	11925
10% allowance	1192.5
Total Expanded Mesh	13117.5
Description	
Labor Hours	219
Description	**Cost**
Labor/Hour	$62
Total Labor	$13,555

Description	**Cost**
Labor	$13,555
Expanded Wiremesh	$477
Saw	$400
Glue	$6,000
Wiremesh	$11,880
Concrete	$40,864
Preparation	$15,000
Total	$88,176

Table 66: Cost Comparison

STATUS OF FLOORING

Description	**Cost**
UNDERLAYMENT	$90,000
PREPARATION	$15,000
TOTAL	$105,000
PATCHING	$12,000

SAMPLE COST CALCULATIONS

CONCRETE:

Through looking at the floor plans, the area of each floor was calculated to be 7920 ft^2, total area equaling 23760ft^2. With an average of 2 inch thickness, the total volume of concrete would be 146.7 CY (23760ft^2*(2/12)ft=3960CF/27=146.7CY). With a labor of $62/hour, pumping, placing, and finishing costs were assumed and verified with the Consigli PM. The overall cost for concrete turned out to be the total of labor (which assumes to account for equioment) plus the material (which is the concrete itself) equaling $40864 ($14080+$26784).

POINT CONNECTORS:

For Point connectors, nails per girder was found by taking the girder size and dividing it by the nail spacing. For instance, if the spacing is three inches apart, the nails per girder equals 72 (216in/3in), nails per floor equals 3816 (girders per floor*nails per floor=53*72), nails for 68 Prescott Street equals 11448 (nails per floor * number of floors=3816*3).

Ten nails were weighed, and average was found to be .0132 lb. Total weight of the nails turn out to be 166.57 lb (.0132lb/nail*11448nails). A 5 lb nail box costs about $11 at Home Depot, hence a total of 166.57lbs would cost about $366(166.57lb*($11/5lb).

Nailing per girder will roughly take about 15 minutes for 3inch spacing. Nailing would require about 795 min of labor per floor (15min*53girders/floor), about 2385 min of labor for 68 Prescott Street (795min*3floors/building), and with a 10% allowance, total amount of labor would equal 2623.5 minutes which is about 44 hours. With a $62 labor/hr cost, total cost of labor for 3 inch spacing would equal $2711.

The total cost for 3 inch spacing costs $71,271 which includes labor ($2711), nail ($366), nail gun with a compressor ($450), wiremesh for the floors ($11800), concrete ($40864), and preparation work ($15000).

The same method is applied to 2 inch spacing as well as 1 inch spacing.

CONTINOUS CONNECTORS:

Since there are 53 girders per floor with each being 18 ft span, there would be 954 ft (18*53) of continuous mesh needed. 2862 ft (954 ft * 3 floors/girder) of mesh for 68 Prescott is necessary. With a 10% allowance, total mesh length would be 3148ft.

Mesh is about $3 per girder, equaling $159 (53 girders*$3) per floor, $477 (159*3 floors) for 68 Prescott Street.

The labor of mesh per girder would take roughly about 75 minutes which includes cut (40 min), vacuum (10min), gluing (15min), laying the mesh (10min). The labor per floor would equal 3975 min (75 min * 53 girders per floor). The mesh labor for the 68 Prescott Street would equal 11925 minutes and with a 10% allowance, total labor would equal 13,117.5 min which equals 219 hours. With a labor cost of $62/hr, the total cost would be $13,555 (219hours*$62/hr).

The total cost would be $88,176 which consists of labor ($13,555), expanded wiremesh ($477), saw ($400), glue ($6,000), wiremesh ($11800), concrete (40,864) and floor preparation (15,000).

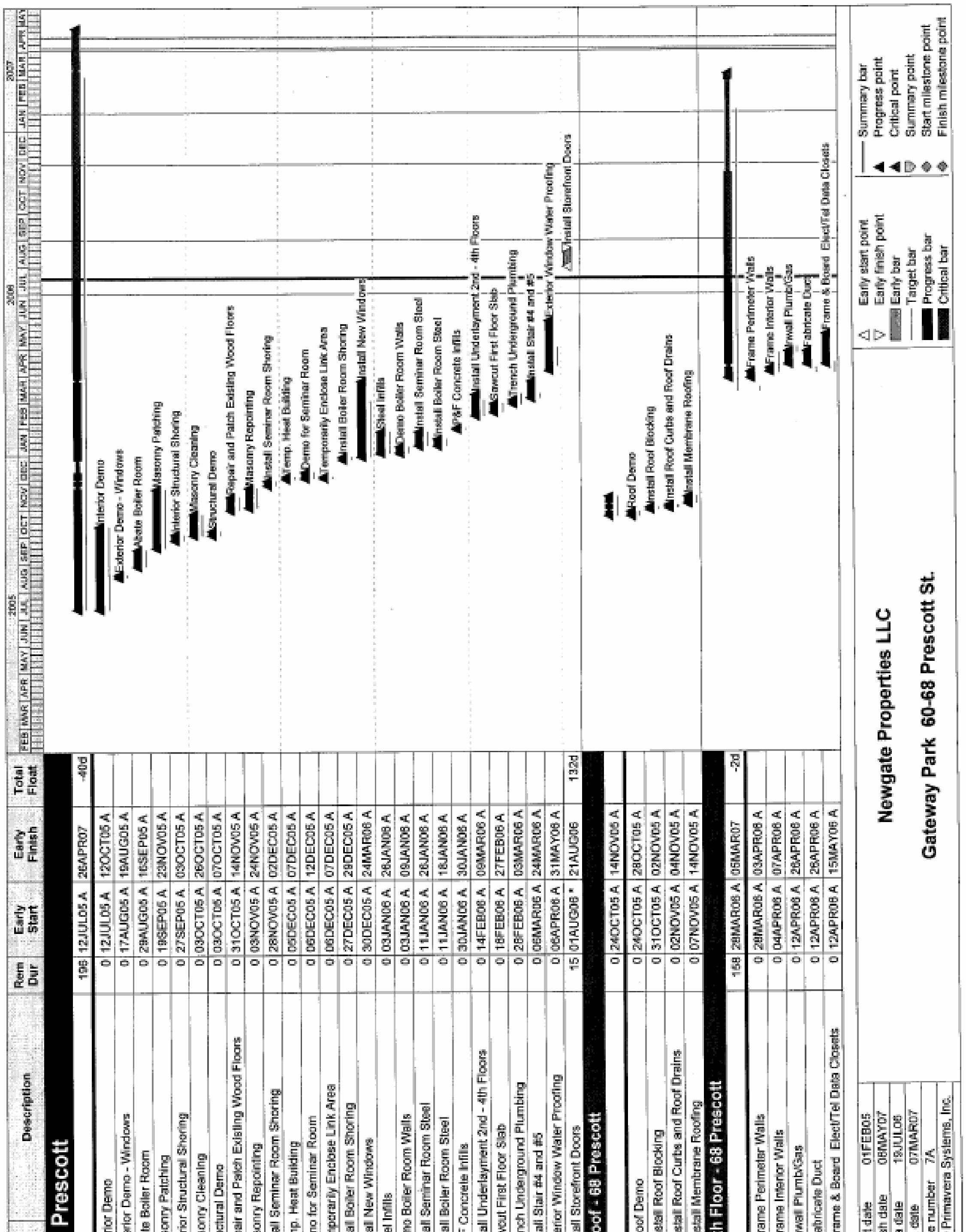


Figure 80: Consigli Schedule for the BTC at 68 Prescott Street

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8.9 MEETING CHECKLIST AND QUESTIONARRE

MEETING CHECKLIST:

Table 67: Meeting Checklist

Meeting	CONSIGLI										GILBANE						Averages					
Date	02/05/06		01/29/06		01/22/06		12/04/06		12/11/06		02/01/06		01/17/06		11/29/06		Consigli		Gilbane		Total	
Description	#	Min.	#	Min.	#	Min.	#	Min.	#	Min.	#	Min.	#	Min.	#	Min.	#	Min.	#	Min.	#	Min.
Phone Calls	0	0	0	0	3	3	2	2	0	0	0	0	3	3	0	0	1	1	1	1	1	1
Unrelated Talks	1	1	0	0	0	0	0	0	2	9	0	0	2	4	4	29	1	2	2	11	1	5
Extra Talks	2	2	0	0	1	1	2	9	3	10	1	1	1	1	1	1	2	4	1	1	1	3
Technology	**If used, 100% If not, 0%**																**f Average Use of Technol**					
E-mail	0%		0%		100%		0%		0%		0%		100%		100%		20%		67%		38%	
PDA	0%		0%		0%		0%		100%		0%		100%		100%		20%		67%		38%	
Speaker Phone	0%		0%		0%		0%		0%		0%		100%		0%		0%		33%		13%	
Meeting Materials	**If used, 100% If not, 0%**																**of Average Use of Materi**					
Meeting Minutes	100%		100%		100%		100%		100%		100%		100%		100%		100%		100%		100%	
RFI	100%		100%		100%		100%		100%		100%		100%		100%		100%		100%		100%	
Change Orders	100%		100%		100%		100%		100%		100%		100%		100%		100%		100%		100%	
Site Plans	100%		100%		100%		0%		0%		100%		100%		0%		60%		67%		63%	
Material Samples	0%		0%		0%		0%		0%		0%		100%		0%		0%		33%		13%	
Calendar	0%		100%		100%		100%		0%		0%		0%		0%		60%		0%		38%	
Punchlist	100%		100%		100%		0%		100%		0%		0%		0%		80%		0%		50%	
Personal Binder	100%		100%		100%		100%		100%		100%		100%		100%		100%		100%		100%	
Milestones	100%		100%		100%		0%		0%		0%		0%		100%		60%		33%		50%	
Hand Drawings	0%		0%		100%		0%		0%		0%		100%		100%		20%		67%		38%	
Meeting Leader	Office PM										Office PM						Office PM					
Dominant Speaker	Architect										Site PM						Arch.		Site PM			
Others			Enter/Ex		Enter/Ex		Photocop		Pastry la													
			Photocop		Photocop																	

QUESTIONNARE:

Table 68: Basic Information

Represent	Project	Const. Stage	Experience
Consigli	BTC	Design	3 or more
Consigli	BTC	Construction	3 or more
Consigli	BTC	Construction	New experience
Gilbane	Garage	Construction	3 or more
Gilbane	Garage	Pre-Design	3 or more
Gilbane	Garage	Design	3 or more
Other	BTC	Construction	3 or more
Other	Both	Pre-Design	3 or more
Other	BTC	Pre-Design	3 or more
WBDC	Both	Construction	1 or 2
WBDC	Both	Pre-Design	1 or 2
WBDC	Garage	Pre-Design	3 or more
WBDC			
WBDC	BTC	Construction	3 or more
WPI	BTC	Construction	3 or more
WPI	Both	Design	3 or more
WPI	Both	Other/ LLC-1999	3 or more
WPI	BTC	Design	3 or more

Table 69: Tools used outside of weekly meetings

Represent	Face-to-Face	Video Conference	Phone Call	Conference Call	Voice-mail	E-mail	Mail	Fax	Online Collaboration	Chat room/message board	Site visits	Memo
Consigli	1	0	2	0	1	2	0	0	0	0	1	0
Consigli	2	0	2	0	2	3	0	1	0	0	1	0
Consigli	1	0	2	1	2	3	0	0	0	0	1	0
Gilbane	1	0	1	0	1	2	0	0	0	0	1	0
Gilbane	3	0	3	0	3	0	0	3	3	0	3	3
Gilbane	1	0	1	0	1	1	0	0	0	0	1	0
Other	1	0	2	0	3	3	0	0	0	0	1	0
Other	1	0	3	1	2	3	3	1	0	0	1	1
Other	1	0	2	1	2	2	1	1	0	0	1	0
WBDC	3	0	1	0	1	2	0	1	0	0	3	0
WBDC	1	0	1		1	1	1	1	0	0	1	0
WBDC	3		3	1	1			1	0	0	3	0
WBDC												
WBDC	1	0	2	1	2	3	1	1	0	0	1	0
WPI	0	0	1	0	1	1	0	0	0	0	0	0
WPI	1	0	2	0	1	2	1	1	0	0	2	1
WPI	2	0	3	1	1	3	0	1	0	0	1	0
WPI	1	0	1	0	0	1	0	0	0	0	1	0

Actual	Representation
0	0
1 to 5	1
6 to 10	2
11 or more	3

Table 70: Addition Information Received

Represent	RFI	Submittal	Change Order	Meeting Minutes	Drawings	Specifications	Budget
Consigli	4	2	3	4	1	1	2
Consigli	4	4	4	4	2	2	3
Consigli	4	4	3	1	2	4	3
Gilbane	0	0	2	0	0	0	2
Gilbane	4	4	4	4	4	4	4
Gilbane	2	1	2	2	1	1	2
Other	4	4	4	4			
Other	3	4	2	2	2	1	1
Other	3	3	2	2	1	1	1
WBDC	1	4	2	3	4	4	4
WBDC	1	1	2	4	2	2	1
WBDC	4	4	4	4	4	4	4
WBDC							
WBDC	4	1	4	4	4	4	2
WPI	3	1	3	3	2	2	3
WPI	3	2	4	4	3	4	4
WPI	1		2	4	1	1	3
WPI	2	2	1	0	2	2	0

Actual	Value
Always	4
Often	3
Occasionally	2
Rarely	1
Never	0

Table 71: Documents used outside of Weekly Meetings

Represent	RFI	Submittal	Change Order	Meeting Minutes	Drawings	Specifications	Budget
Consigli	3	1	2	4	3	3	3
Consigli	4	3	3	2	4	4	4
Consigli	4	4	3	3	4	4	3
Gilbane	2	3	3	3	3	3	3
Gilbane	4	4	4	4	4	4	4
Gilbane	1	1	1	1	1	1	2
Other	3	3	3	2	3	3	
Other	3	2	1	2	1	1	1
Other	3	3	2	2	3	3	3
WBDC	2	3	2	3	4	4	4
WBDC	1	1	2	2	3	2	3
WBDC	4	4	4	4	4	4	4
WBDC							
WBDC	4	4	4	4	4	4	4
WPI	3	1	3	3	2	2	3
WPI	3	2	2	2	3	2	3
WPI	0	0	0	0	0	0	0
WPI	3	2	2	1	3	3	0

Actual	Value
Always	4
Often	3
Occasionally	2
Rarely	1
Never	0

Table 72: Interest /comfort with technology

Represent	Basic	Construction Software	New Technology
Consigli	5	5	4
Consigli	5	4	4
Consigli	5	5	1
Gilbane	5	5	3
Gilbane	1	1	1
Gilbane	4	2	2
Other	4	4	1
Other	5	4	3
Other	5	5	4
WBDC	3	1	1
WBDC	5	2	4
WBDC	3	1	1
WBDC			
WBDC	5	5	5
WPI	5	4	4
WPI	3	1	1
WPI	5	1	3
WPI	4	3	1

Actual	Value
Very high	5
High	4
Neutral	3
Low	2
Very Low	1

Table 73: Optional Question

Represents	Comments
Consigli	
Consigli	
Consigli	There is a lot of information exchanged in short phone calls and hand sketches that through formal means of communication would be extremely inefficient.
Gilbane	
Gilbane	
Gilbane	
Other	
Other	
Other	BIM is coming
WBDC	
WBDC	
WBDC	
WBDC	
WBDC	
WPI	
WPI	
WPI	
WPI	AJB Consultants is working with WPI on Lab Equipment and related items, AJB has been involved since design working between WPI researchers and the construction team to ensure proper electrical, mechanical, plumbing hook-ups to equipment, helped with the procurement of portable lab tables, sterilizers,chemical cabinets, etc.