Evaluation of AASHTO Design Specifications for Cast-In-Place Continuous Bridge Deck Using Remote Sensing Technique

by

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“NOTE TO READER”

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EVALUATION OF AASHTO DESIGN SPECIFICATIONS FOR CAST-IN-PLACE CONTINUOUS BRIDGE DECK USING REMOTE SENSING TECHNIQUE

Ebrahim Mehranipornejad

ABSTRACT

This research project concerns the construction, testing, and remote health monitoring of the first smart bridge structure in Florida, the East Bay bridge in Gibsonton, Hillsborough County. The East Bay Bridge is a four span, continuous, deck-type structure with a total length of 120’ and width of 55’. The superstructure consists of an 18” cast-in-place reinforced concrete slab, and is supported on pre-stressed pile bents, each consisting of 5 piles. The smart sensors used for remote health monitoring are the newly emerged Fabry –Perot (FP) Fiber Optic Sensors, and are both surface-mounted and embedded in the concrete deck.

Static and Dynamic testing of the bridge were performed using loaded SU-4 trucks, and a finite element model for the bridge was developed for the test cases using commercial software packages. In addition, the smart sensors were connected to a data acquisition system permanently installed on-site. This system could be accessed through regular phone lines, which permits the evaluation of the bridge behavior under live traffic loads.
Currently, these live structural data under traffic loading are transmitted to Hillsborough County’s bridge maintenance office to assist in the health evaluation and maintenance of the bridge.

AASHTO LRFD Design Code has been investigated using analytical and laboratory test but no attempt has been made to verify its relative outlook with respect to Allowable Strength Design (ASD) and AASHTO Standard Specifications (LFD) in a real field test. The likely reason for could have been the lack of accurate and reliable sensing systems.

The data collected as well as the analytical studies through out this research, suggest that current LRFD design specifications for deck-type bridges are conservative. The technology developed under this work will enable practical, cost-effective, and reliable systematic maintenance of bridge structures, and the study will provide a unique opportunity for future growth of this technology in the state of Florida and in other states and finally, long term collected data can be used to keep the design codes in check.
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CHAPTER 1. INTRODUCTION

1.1 Introductory Background

In 1993, American Association of State Highway and Transportation Officials (AASHTO 1989) Subcommittee on Bridges and Structures responded to interest in developing new-updated AASHTO bridge specifications with accompanying commentary. The goal was to develop more comprehensive specifications that would eliminate any gaps and inconsistencies in the Load Factor Design-based format (AASHTO 1973) of standard specifications by incorporating the latest in bridge research and technology. The decision was made to develop these specifications in a Load and Resistance Factor Design-based format (AASHTO 1993) which takes the variability of the structural elements into account through the application of statistical methods. The LRFD specifications were approved by AASHTO for use as alternative specifications to the AASHTO Standard Specifications for highway Bridges (LFD). The AASHTO LRFD was evolved based on perception of gaps and inconsistencies, non-uniform margin of safety and less reliability in LFD design specifications across a wide variety of structures.

To validate these downside issues raised about LFD design standard specifications and verify acclaimed outlook of AASHTO LRFD design method,
research of literature in related topic, laboratory experiment and actual field load test of bridges designed by LRFD-based format are necessary.

Further continuous monitoring of bridges designed by LRFD-based format deserve closer attention. The merit of this process is to generate sufficient data for analysis of structural behavior of the bridge subject to long term various truck loading conditions, stresses induced by large temperature change and extreme natural events such as hurricanes in Florida and earthquakes elsewhere.

The existing East Bay Road Bridge in Gibsonton, Hillsborough County, Florida was candidate for replacement with a new four continuous span concrete bridge. Hillsborough County provided funds to install 16 fiber optic sensors (FOS), ten of which were embedded in the concrete during construction and four were surface mounted on the underside of bridge deck after completion of construction. It was decided to continuously monitor, observe and record behavior of the bridge under the effect of the traffic and environment for two years. Periodic monitoring at the time of two years inspection cycle will generate a history on structural behavior of the bridge. At the completion of construction, six surface mount strain sensors were installed on the bridge.

Prior to opening the bridge to daily traffic, the bridge was subject to a series of static load tests. The static load test resulted in the strain values that were used to investigate and evaluate the design of the bridge under AASHTO LRFD (AASHTO 1994) design specifications and AASHTO LFD (AASHO 1931) standard specifications. This is the primary objective of this dissertation. The results of field static test were compared with an analytical model of the structure
to define the degree of reliability and conservative state of the bridge design by AASHTO LRFD design specifications and AASHTO LFD standard specifications.

In addition to static and dynamic truck load test, continuous monitoring of the bridge will be performed to obtain real live data (strain values) to compare with the design strain values described in chapter 3. This comparison will help the bridge engineers and facilities management to understand the actual condition of the bridge and its level of performance.

Continuous health monitoring of bridge structures is a new area that has been driven by the necessity of efficient structural condition assessment. Presently, repair and replacement decisions for the bridges are based on highly subjective visual observations (Van Daveer 1975; Chase and Washer 1997). According to Aktan et al. (1996), subjective or inaccurate condition assessment has been identified as the most critical technical barrier to the effective management of bridges, which results in annual $3 billion maintenance cost in the US (Chase and Washer 1997). Nevertheless, an earlier study (Catbas et al. 1998) has confirmed that more than 40 per cent of the bridges in the U.S. are functionally obsolete or structurally deficient due to corrosion, scour, or subjective and inaccurate observations and data collection. In addition, several bridges have experienced major damage or collapse recently due to extreme events (e.g. earthquakes, hurricanes). Often, inaccurate structural condition assessment has lead to unfounded decisions to replace numerous reinforced concrete bridges possessing significantly large number of remaining safe operating service life.
With the advent of today’s new technologies, existing and new structures can now be instrumented for evolution and verification of the code that they have been designed with. The measuring and monitoring systems can be conveniently operated and controlled from a remote central monitoring station that is located several miles away from the field. Sensors are placed at several critical locations along the structure, and send structural information (e.g. strains, stresses, accelerations) to the central station. The structure is thus thought of as a smart system that is capable of sending information that can be used in evaluation and verification of design code and specifications while at the same time would be providing warnings before any major failure.

Several types of advanced sensors are used for remote monitoring and damage detection. Fiber Optic strain Sensors (FOS) are the most commonly used, especially in Canada by the ISIS center (2001). The so-called WiMMS accelerometers have been developed at the Blume Earthquake Engineering Center at Stanford University (Straser and Kiremidjian 1998). In addition, miniature micro-electro-mechanical systems (MEMS) or smart dust accelerometers have been also used.

1.2 Problem Statement

AASHTO LRFD Design Code (AASHTO 1944) has been investigated (Shahawy 1996) using analytical and laboratory test but no attempt has been made to verify its relative outlook with respect to Allowable Strength Design, ASD (AASHO 1931) and AASHTO Standard Specifications, (AASHO 1931) in a real
field test. The likely reason for that is the lack of accurate and reliable measuring systems. Literature has noted mixed opinions regarding vague interpretation, difficult, time consuming calculations, which lead to excessively conservative results for LRFD Design code in comparison with analytical models, laboratory test and prediction by AASHTO Standard Specifications, LFD (Shahawy 1996).

The effective repair and rehabilitation of a bridge depends on understanding of its structural condition. This understanding begins with bridge inspection (Haque 1997). Scheduled periodic bridge inspections are tailored to detect and assess structural damages for the purpose of maintenance and replacement. Inspection and damage assessment based on visual observations are highly objective (Van Daveer 1975; Chase and Washer 1997). Collected data on the condition of bridges are used to determine needs for repair or replacement, and to form models of future needs. Numerical Condition Ratings (NCR) assigned to structural elements during visual inspection are qualitative condition ratings and determine the level of need for repair and rehabilitation. Condition ratings are imprecise, and the ratings are only a subset of the information collected during a bridge inspection (Hearn and Shim 1997). To have a better understanding of bridge condition and verify visual observations, various methods of nondestructive evaluations (NDE) have been developed to detect the extent of deterioration and damage to the bridge elements. One such NDE is a static load test to determine structural strength and load carrying capacity of the bridge. However, neither periodic visual inspection nor random nondestructive evaluation can detect the initiation and propagation of
deterioration in structural elements until the damages are serious and often not repairable. Continuous monitoring of a bridge is known to instantly detect the onset of damage in a bridge; associated with over stress by heavy load, corrosion and structural elements section losses.

1.3 Objectives and Scope of Work

There are two objectives of this research. First objective is the short-term application of newly emerged sensor as a tool for evaluation of AASHTO Design guidelines, and that is, to investigate the new bridge design method of LRFD and old design method of LFD, evaluate and verify the assumptions and parameters considered in design of the East Bay Road Bridge. We then compare the design of the bridge with the data obtained from the sensors installed in the bridge during construction. This data is also used to investigate and verify the results of LFD method of bridge load capacity rating.

Second objective is to develop a new methodology for damage detection and cost life cycle evaluation of bridges. While the primary purpose of fitting the East Bay Road Bridge with sensors was to investigate AASHTO LRFD design specifications, the strain measuring system was permanently left in the structure to provide an opportunity for a long term monitoring of the bridge condition and to develop a new methodology for damage detection and life cycle evaluation of the bridges. To achieve these goals, the long-term application of sensors to build strain history of the bridge by continuous or periodic monitoring and evaluation of collected data is necessary. Needless to say that the analysis of collected data
over a long period of time would be a valuable tool for diagnostic measures such as safety assessment, damage detection and rehabilitation of existing bridge.

Verification of both objectives, the field load testing of East Bay Bridge in addition to damage detection and evaluation of bridge condition are presented in the following chapters.

1.4 Overview of Following Sections

Section 1.4.1 describes the need for a new more advanced bridge design code AASHTO LRFD, the history and development of AASHTO LRFD and technical papers written on the topic. Section 1.4.2 describes sensing technology encompassing development of different sensors in a chronological order, e.g., from early basic sensors to more advanced fiber optic sensors leading to the three most commonly used sensors; Fabry Perot Interferometer, Fiber Brag Grating and Long gauge sensors. Section 1.4.3 describes the application of sensors through literature review on related topics.

1.4.1 History of AASHTO Standard Specifications and AASHTO LRFD Code

AASHO, American Association of State Highway Officials, the “standard specifications” was formed in December 12, 1914. In 1921, AASHO organized the bridge and structures committee to develop and compile design specifications until the first edition of standard specifications, published in 1931 and followed by 1935, 1941, 1944, 1949, 1953, 1957, 1961, 1965, 1969, 1973,

In 1993, AASHTO adopted the Load and Resistance Factor design (LRFD) specifications for bridge design and published the first edition of design specifications in 1994. AASHTO approved the LRFD specification to be used as an alternative specification to the AASHTO Standard Specifications (LFD) for Highway Bridges. Additional versions (editions) were developed and latest appeared in 2005.

The methodology and philosophy of AASHTO LRFD Design specifications and AASHTO Standard Specifications, LFD are presented in Chapter 3.

1.4.2 Sensing Technology

With the emergence of measuring technology, the use of traditional measuring and monitoring devices and systems have been gradually phasing out. These systems were not fully capable of continuous measuring stresses and monitoring of structures. Some of these systems consisted of several parts and components and were time consuming and difficult to handle during installation. Some systems required more than one specialized person for equipment installation and setup. Frequent monitoring of structure with these types of measuring systems was not economically feasible. Amongst these systems include triangulation, water level, vibrating string, dial gages, invar wires, and mechanical extensometers, Base-line system, global positioning system,
strain gage-base system, linear variable displacement transducer (LVDT), accelerometer, etc. This review would briefly describe a few of the systems that are still occasionally implemented in monitoring of some specific situation.

1.4.2.1 Electrical Resistance Strain Gauge

Electric resistance sensor is a device whose electrical resistance varies in proportion to the amount of strain in the device. The most widely used gauge is the bonded metallic strain gauge. The metallic strain gauge consists of a very fine wire or, more commonly, metallic foil arranged in a grid pattern. The grid pattern maximizes the amount of metallic wire or foil subject to strain in the parallel direction. The cross sectional area of the grid is minimized to reduce the effect of shear strain and Poisson Strain. The grid is bonded to a thin backing, called the carrier, which is attached directly to the test specimen. Therefore, the strain experienced by the test specimen is transferred directly to the strain gauge, which responds with a linear change in electrical resistance. Strain gauges are available commercially with nominal resistance values from 30 to 3000 Ω, with 120, 350, and 1000 Ω being the most common values.

1.4.2.2 Base-Line System

Base-line system consists of high strength piano wire, pulley and weight. This system is only capable of measuring deflection due to static load. This system is not suited for dynamic monitoring since the vibration of piano wire and constant tension weight would prevent accurate deflection measurements. Digital
Calipers and Linear Variable Displacement Transducer (LVDT) are used to measure the vertical deflection relative to Base Line. This system was used as deflection monitoring system in H-3 North Halawa Valley (Lee, 1995).

1.4.2.3 Global Positioning System, (GPS)

Deflection (deformation) of Bridge structural elements have been monitored using strain gages. Strain gage capability is limited to measuring deflection due to static load. In this system, in addition to the sensors installed on structure, one sensor must be located on a fixed and stable reference point near the structure. All sensors including the reference sensor must have antenna and communicate with at least four GPS satellites. This system can only process one reading in every ten seconds therefore, it is not recommended for dynamic and seismic applications, (Celebi 2002)

1.4.2.4 Hydrostatic Leveling System (HLS)

The hydrostatic leveling system is based on the classical physical law of “connected vessels”. The vessels are made of calibrated glass beakers connected with transparent plastic tubes. Since the water level within the tubes always remains on a horizontal plane, vertical displacements can be deduced from the difference of the water levels between the deformed and the initial position of the structure. Vibration generated by the traffic does not influence measurements because of the great inertia of the HLS. The error made on deflections for the overall system is about ±0.5 mm.
1.4.2.5 Linear Variable Displacement Transducers

Linear Variable Displacement Transducers, LVDTs are used to measure high frequency of relative displacement between two points on a bridge. They are capable of measuring deflection of bridge elements but require a fixed and stable reference point. They are not recommended for seismic application since a fixed object on the ground would not remain stable during a seismic activity.

1.4.2.6 Accelerometers

Accelerometers are used to measure deflection in structural members subject to dynamic loading. Deflection values are obtained by double numerical integration of acceleration. Literatures have noted unreliability in deflection results due to integration process and undetected anomalies in the sensors recorded values, (Celibi and Sanli 2002).

1.4.3 Fiber Optic Sensors

The new generation of high tech sensors render the aforementioned sensing systems obsolete. These new families of sensing sensors are technologically highly complex and expensive to manufacture. But, their high cost is quickly offset by their physical simplicity to handling, versatility and easy installation. These sensors are known as fiber optic sensors. Several different fiber optic sensors have been developed in recent years, from a simplest form of measuring an on-off state to highly complex sensors capable of measuring a wide range of wavelengths.
Fiber Optic strain Sensors are in general better suited for structural health monitoring of the bridges than accelerometers, LVDTs, HLS, GPS, etc., since they can be easily bonded to reinforcing bars and embedded in the structure, and they can provide a complete strain history including strains from concrete curing, construction loads and in-situ service loads, and creep and thermal changes. FOS sensors have proven to be accurate, inexpensive, and easy to use.

Fiber optic sensors have numerous advantages: small size, lightweight, long-term stability, large selection of gauge length, corrosion-resistance, wide variety of packaging for surface mounting and embedment in the structure, distributed capability, immunity to electromagnetic and radio frequency interference, and multiplexing capabilities among others. Their main advantage though lies in their remote sensing capabilities.

Fiber optic sensors are manufactured either as discrete or distributed type. Discrete sensors come as short and long gauges. Fiber Bragg-grating and SOFO are examples of discrete short-gauge and distributed long-gauge sensors, respectively. Discrete sensors detect changes at locations where they are installed while distributed sensors detect changes at several locations in the structure. Short-gauge sensors are highly influenced by presence of cracks related to their locations in structure (local stress) and thus do not represent global behavior of the structure (e.g., deflection).
1.4.4 Fiber Optic Sensors’ Time Scale

Fiber optic, in a very basic form but yet revolutionary, was developed in 1950. The system was basically light confinement within two layers of glass. In 1960, the laser light source was introduced into the system. The refinement of optical fiber manufacturing methods and use of LED (light emitting diode) as a light source became practical in 1970. In 1980, optical fiber was widely used in telecommunication systems. In 1990, optical fiber was used in instrumentation and commercially available sensors. In 1995, the application of optical fiber on site in highway bridges became possible.

The following Figure 1.1 through Figure 1.9 have been reproduced and recreated with permission from RocTest, Canada.

An optical fiber consists of three principal elements, arranged concentrically:

Coating / Buffer: This is the first non-optical layer around the cladding, typically consists of one or more layers of a polymer that protects the silica structure against physical or environmental damages.
Cladding: This is the first optical layer around the core. The cladding creates an optical wave-guide that confines the light. Cladding is usually made of silica.

Core: This is the central section made of silica. It is the high transmitting region of the fiber.

A Simplex cable is a tight-buffered Optical Fiber Glass reinforced with Kevlar fiber strands and then covered with a PVC outer jacket.

![Diagram of Optical Fiber Cable Components](image)

**Figure 1.2 Components of Optical Fiber Cable Used with Sensors**

Fiber optics are manufactured in singlemode and multimode fiber, Figure 1.3(a) and Figure 1.3(b). Each one has different light signal transmission and properties. In single-mode fiber, only the fundamental mode is propagated, it travels straight through the fiber without reflection at the core-cladding boundary. It has higher bandwidth, 5 to 10 microns core diameter and 125 microns cladding diameter.
In multimode fiber, higher-order modes are propagated in addition to the fundamental. The different modes travel in curved, wavelike paths. It has lower bandwidth, 50 to 100 microns core diameter and 125 microns cladding diameter.

Figure 1.3 Types of Fiber Optic Cables

In the following section 1.4.4.1, 1.4.4.2 and 1.4.4.3, the general configuration, working principle and application of these three commonly used sensors are presented. The most commonly used (FOS) for health monitoring of bridge structures are: (1) the Fabry Perot Interferometer (FPI), (2) the Fiber Bragg Grating (FBG), and (3) the long-gauge sensors.
1.4.4.1 Fabry-Perot Interferometer

Figure 1.4 is the schematic of Fabry-Perot fiber optic sensor depicting components of sensor, direction of signal and the light source. The light hits the mirror and reflects back to the readout unit (e.g., DMI-16). The principle of interferometer is a unique feature to Fabry-Perot sensor. Interferometer is an optical instrument that allows two beams of light derived from a single source (and thus of the same frequency and in phase at identical distances from the source) to traverse paths whose difference in length determines the nature of the interference pattern obtained when the beams are allowed to interfere. The wavelength of light can be measured if the path length difference is known, and vice versa.

Figure 1.4 Schematic Presentation of Fabry-Perot Sensor’s Components

Figure 1.5 is the schematic of an encapsulated Fabry-Perot fiber optic sensor. In this figure, the actual components of the Fabry-Perot are shown in a 10mm micro capillary tube. The reflected light is traveling toward the readout
unit. The magnitude of strain is function of ratio of change in cavity length and gage length.

\[ Strain(\varepsilon) = \frac{\Delta d}{L_g} \]

Measurement is achieved by measuring the Fabry-Perot cavity length using white light interferometer.

![Figure 1.5 Fabry-Perot Sensor Encapsulated in Micro Capillary Tube](image)

Fabry-Pérot interferometer (FPI) manufactured by Roctest is basically consisted of two multimode optical fibers, 50 to 125 microns thick facing each other. The two fibers are placed inside a 200 microns diameter glass micro-capillary. The tips of fiber ends facing each other are coated with Semi-reflective coating acting as mirrored reflectors. The space separating the two mirrors is called the cavity length. Light from a broadband source is aimed at one arm of a 2 x 2 coupler and directed toward the Fabry-Pérot gauge along an incoming multi-mode optical fiber. Light reflected in the FPI is wavelength-modulated in accordance with the cavity length. The reflected light signal travel through the fiber into a read-out unit. At this point, the light travels through a white-light cross-
correlator (Fizeau Interferometer), and detected by a linear Charged-Coupled Device (CCD) array with a pixel arrangement that allows for 1:10,000 resolution. Finally, the incoming fiber that transports light to the gauge is mechanically decoupled or isolated from the strain sensing fiber (Figure 1.5). The Fabry-Pérot interferometer (FPI) gauge converts strain into cavity length variations measurements (Figure 1.6). Fabry-Pérot has been used to monitor the behavior of several structures such as Morristown bridge in Vermont (Benmokrane, et al. 2003), and the Joffre bridge in Sherbrooke, Canada (Choquet et al. 2000) among others. The principle of this measuring system is shown in Figure 1.7. Fabry-Perot (FP) sensor has the following unique characteristic:

While a calibration process is required for each sensor, in-line FP sensors provide low thermal sensitivity because the cavity is in air, combined with a well-defined gauge length and relatively high strength.

Since the FP sensor is decoupled from the surrounding micro-capillary, it avoids creep that might arise from the use of adhesives. Sensed information is the Fabry-Perot cavity length, which is also an absolute parameter.

The output does not depend directly on the total light intensity levels, losses in the connecting fibers and couplers, or recalibration or re-initialization of the system. Fabry-Perot strain gage uses a multi-mode fiber instead of a single mode fiber.

Fabry-Perot sensors are easier to splice, repair and connect. Their transducers would lose less light when subjected to bending.
1.4.4.2 Fiber Bragg Grating Sensor

A fiber Bragg Grating (FBG) can be fabricated from a continuous germanium doped fiber core, surrounded by germanium-doped silica. The grating portion consists of a modulation in the index of refraction along a length of continuous fiber core. A change in length of the grating is due to mechanical or thermal strain in the host material. The change in the length of grating is detected as a shift in the wavelength of the reflected light. Bragg grating measures strain based on wavelength shift.
Figure 1.7 Fabry-Perot Sensor

The light source can be either a broadband light emitting diode or a tunable laser over a specified wavelength range.

Bragg gratings are supplied with a section of the coating around the grating removed to allow for installation and bonding. The grating itself may appear as a barely-perceptible optical fiber difficult to see with a naked eye. To create the Bragg grating sensor, ultraviolet (UV) light is directed perpendicular to the core of the fiber periodically, along a defined section of the fiber optic cable. The process is referred to as “wiring” FOS gratings (ISIS Design Manual 1, 2001.)
Photonics Research Ontario (PRO) Center of Excellence and E-TEK

Electro Photonics Solutions are the manufacturer of some Fiber Bragg Grating fiber optic sensors. Fiber Bragg Grating (FBG) Sensors have been installed on several structures such as the Commodore Barry Bridge in Philadelphia (Aktan et al. 2000) and the Taylor bridge in Manitoba (ISIS design manual I, 2001). Figures 1.8 and 1.9 show a general working principle of Fiber Bragg Grating (FBG) Sensors. Fiber Bragg Grating sensor has the following unique characteristic.

Sensed information is encoded directly into optical wavelength, which is an absolute parameter. Therefore, the output does not depend directly on the total light intensity levels, losses in the connecting fibers and couplers, or recalibration or re-initialization of the system. Fiber Bragg Grating sensor is also capable of handling wavelength division multiplexing by the fabrication of each grating at a slightly different frequency within the broadband source spectrum on a single fiber. In FBG, mirrors are inscribed inside the fibers. The Bragg Wavelength (\(\Lambda\)) is function of the spacing (\(\Lambda\)) and the refractive index \(n\) of the core.
1.4.4.3 Long Gauge Fiber Optic Sensor

Long gauge sensing system comes in two types. One method involves using conventional telecom optical fibers of arbitrary length configured from 2 inches to about 300 feet. This type of long-gauge can be bonded to a structure or embedded in concrete. The distance between two mirrors on the fiber optic leads defines the gauge length of the system. This type of sensor measures the change in path distance between the mirrors while bonded to the host structure or material. The system demodulates the light signals returning from the mirrors by the principle of low coherence interferometry. The obtained deformation is the average values taken over the gauge length.
Figure 1.9 Fiber Bragg Grating Sensor System Components

The second method, Brillouin scattering also involves using conventional telecom optical fibers and can be used to measure strains due to thermal or mechanical loading. Brillouin scattering is a distributed sensor that can take readings at various points along the optical fiber over a large distance in magnitude of ~ 1000’s feet. The resolution of this system can be about 4 to 8 inches and strain values are the average values taken over the gauge length.

Although expensive, long-gauge sensors were used to monitor the behavior of several bridges such as the Rio Puerco bridge in New Mexico (Idriss, Kersey and Davis 1997), Highway 401 in Toronto (ISIS 2001) and Lutrive twin bridges between Lausanne and Vevey in Switzerland (Inaudi et al. to be published). Either types of long-gauge system is suitable for the applications
where deformation or strain is required in small or very large diameter cylinders or bridge piers due to thermal or mechanical loading is required. Another application of these long-gauge systems is to find the strain in deteriorating bridge pilings and piers wrapped with composite sheets of fiber-reinforced polymer, FRP. Long gauge sensors are not capable of high frequency monitoring and thus are not suitable of monitoring structures subject to high frequency dynamic loads.

System of long-gage, strain sensors, SOFO (surveillance d’Ouvrages par Fiber Optiques or monitoring of Structures by Fiber Optic Sensors) (Inaudi and Vurpillot 1998) have been used in several bridges in Switzerland for monitoring the effect of temperature fluctuation and stresses due to static and dynamic loading on the structures. This system is best suited to determine the deflection profile of a beam type structures such as bridge, frame, etc.

The sensor consists of a pair of single mode fibers installed in the structure. One of the fibers, the measurement fiber would be in mechanical contact with structural member subject to measurement and the other, the reference fiber, is placed loose nearby the structure. Deformation of the structure will then result in a change of the length difference between these two fibers (Inaudi et al. 1977)

SMARTEC, a Swiss company installed 30- six-meter long sensors along the length of fourth span on Lutrive Highway Bridge, a box girder bridge in Switzerland [Inaudi, 1999]. These sensors were used to monitor the effect of temperature variation on curvature. A double integration of curvature will result
in deflection. SOFO system is not capable of high frequency strain monitoring therefore, it is not recommended for seismic monitoring application.

1.4.5 Sensing Systems

Miniature sensors represent another technology used for remote monitoring of structures. An attempt to apply this technology for monitoring civil structural systems was performed at the John A. Blume Earthquake Engineering Center at Stanford University in collaboration with Los Alamos National Laboratory. The team developed the so-called WiMMS (Wireless, Modular Monitoring System) for remote damage detection. The data acquisition system is moved to the sensor unit, where the computation is performed. Sensors located at different locations in the structure, send the information wirelessly to a centralized data storage system. WiMMS sensors are battery-operated accelerometers aimed at monitoring the vibration characteristics of structural elements. Advanced micro-electro-mechanical (MEMS) wireless accelerometers have also been used for structural monitoring. These devices, also called Macro Motes, have been developed at the Berkeley Sensor and Actuator Center (BSAC). These devices incorporate communication, processing, sensing, and batteries into a package about a cubic inch in size.
1.5 Literature Review on Application of Fiber Optic Sensors

1.5.1 Low Coherence Fiber Optic Deformation Sensors

This system was used in Versoix Bridge near Geneva, Switzerland to measure the displacements of the fresh concrete during the setting phase and to monitor its long-term deformations. The measurement technique relies on an array of standard telecommunication optical fibers in mechanical contact with concrete. Any deformation of the host structure results in a change in the optical length of the fibers. Each sensor line consists of two single-mode fibers. One of the fibers, the measurement fiber would be in mechanical contact with structural member and the reference fiber, is placed loose near the other one. Deformation of the structure will then result in a change of the length difference between these two fibers.

1.5.2 Long-Gauge Structural Monitoring of Civil Structures

The fourth span of Lutrive, a 2800 feet twin bridge was fitted with Thirty, 18 feet long SOFO sensors. The sensors were installed in pairs on interior surface of box girder near the top and bottom of bridge web. A series of strains data result in bridge curvature and a double integration of curvature would lead to the vertical displacement. The sensors were used to collect data for quasi-static test under thermal loading and under static load as well as for statistical characterization of the dynamic behavior of the bridge.

The verification of static and dynamic values and their comparison with the analytical model and computation were not presented. A table presenting an
organized collected data is lacking. Conclusions present application and benefits of SOFO monitoring system but the results of test were not clearly conclusive. A system for protecting the sensors was presented. The program associated (material cost of strain measurement system and the labor) cost with respect to total construction cost was not been presented.

1.5.3 Use of Fiber Reinforced Polymer Reinforcement Integrated with Fiber Optic Sensors for Concrete Bridge Deck Slab Construction

The bridge concrete deck and girders in Joffre Bridge, built in 1950, in Sherbrooke, Quebec, Canada over the St. Francois River were severely deteriorated due to heavy corrosion activity (Inaudi et al. 1988)]. The Ministry of Transportation of Quebec, determined to replace the bridge deck and girders to satisfy the serviceability requirements. A part of concrete deck, a section traffic barrier and sidewalk were reinforced with fiber-reinforced polymer (FRP), carbon fiber reinforced polymer (CFRP) and Glass fiber reinforced polymer (GFRP). During the construction, in addition to a variety of different sensors, some Fiber optic sensors were installed within these elements. Strain values were recorded and mentioned however, there was no comparison between the sensors strain values and analytical results to indicate whether the strain values were high, low or in agreement. Without such an indicator, the accuracy and reliability of strain values may be questionable. A table depicting these analytical and experimental results for the purpose of comparison was lacking. The number of sensors and method of installation is not described.
There was no explanation as to how the sensors were attached to FRP, CFRP and (GFRP). Were they bonded, loosely attached or just placed next to the member? A system for protecting the sensors was not presented. The strain values from the sensors were not compared with the analytical results derived from the code for the verification. No long term remote monitoring was presented. Literature has ignored to present the program associated (cost of strain measurement system and the labor) cost of equipment and labor.

1.5.4 Test Model for the First Canadian Smart Highway Bridge

Carbon fiber reinforced plastic tendons (CFRP) were used for the first time in to pretension six girders of a concrete highway bridge, built in the City of Calgary, Alberta (Reference).

This paper summarizes an experimental program conducted at the university of Manitoba to examine the behavior of four pretension concrete beams similar to the bridge girders pre-stressed with CFRP tendons. Four pre-stressed concrete T-beams were examine for various limit state behaviors, ultimate capacities, and failure modes. The experimental pre-stressed concrete T-beams were 21 feet long and 13 inch deep with overall span-depth ratio as similar to the Calgary bridge girders and scale of 1:3.3. These beams were fitted with fiber optic sensor for monitoring the strain induced due to static and dynamic loads.

The experiment concluded that the section curvature at failure of beams pre-stressed by CFRP was much less than that for beams pre-stressed by steel
strands. However, by increasing the reinforcement ratio beyond 0.56 percent, the section curvature of the beams pre-stressed with CFRP fairly matched the behavior of the beams pre-stressed with steel.

No comparison is made between the sensors strains values and analytical results to indicate whether the experimental strain values were high, low or in agreement with strain readings of sensors. Without such an indicator, the accuracy and reliability of strain values may be questionable. A table depicting these analytical and experimental results for the purpose of comparison was lacking. The type and number of sensors and method of installation were not presented. There is no explanation as to how the sensors were attached to CFRP. A system for protecting the sensors was not presented. The experimental strain values from the sensors were not compared with the analytical results derived from the code for the verification to discuss the code values. The data acquisition system and analysis software were not presented. No long term remote monitoring was presented. The Literature has ignored to present the program estimated associated (cost of strain measurement system and the labor) cost of equipment and labor.
1.5.5 Using Fiber Bragg Grating Sensors to Monitor Pavement Structures

The objective of this research was to develop an innovative fiber-optic sensing system to evaluate pavement materials or monitor pavement infrastructure. The sensor was developed and designed to measure simultaneously pavement temperatures and strains (Wang and Tang et al. 2005).

The reliability and long-term stability tests for this sensor were examined by mounting it on the surface of two types of specimens, asphalt and concrete. The paper mentions the shortcoming of simultaneous measurement of strain and temperature and suggests a possible solution. Experiment was conducted on two specimens, one concrete and an asphalt pavement in a laboratory setting. Sensors were surface mounted. The results of readings between the two specimens theoretical values were compared. The application of FBG for pavement condition assessment was verified.

No reference was made to any field experiment on asphalt, either surface mount or embedded. The asphalt and concrete pavements surface mount sensors would not be able to resist the impact of vehicular traffic. The Literature has failed to present the program associated cost of strain measurement system and the labor and overall cost of laboratory and fieldwork.

1.5.6 Using Sensors for Remote Field Test

Fabry-Perot sensors were used to perform field testing of University Drive bridge in Jacksonville, Florida for FDOT in collaboration with University of Florida.
was not successful. A laptop was used to remotely collect data from surface mount installed data. The program did not work and it was abandoned.

1.6 Summary of Research Work and Implementation of the Objectives

The study is related to the application of Fiber Optic Sensors (FOS) to investigate the AASHTO LRFD bridge specifications to determine its level of reliability. A total of sixteen Fabry-Perot FOS sensors were installed on the East Bay bridge, in Hillsborough County, Florida. The bridge is a 4-span continuous reinforced concrete deck-type structure. The bridge is considered the first smart structure in the State of Florida. The FP sensors were both bonded to the longitudinal reinforcing bars and surface-mounted to the concrete deck. Detailed step-by-step description of the installation process is presented. Static and dynamic tests of the bridge under SU4 trucks were conducted. A finite element model was developed, and its output was compared to the experimental data obtained from the truck tests. The results confirmed the accuracy of FP sensors in evaluating the bridge behavior under traffic loads. A remote communication system was established through phone lines in order to connect the acquisition system to the Internet. This technique enables live traffic monitoring from a central station located in the county maintenance office. Live traffic data are currently being collected and stored on PC hard drive and CD. These data will be used to (a), evaluate current AASHTO specifications for deck type bridges and (b), facilitate the bridge maintenance process, receive early warnings regarding possible structural deficiencies, and assist in decision-making
processes regarding functionality of bridges. The proposed remote health monitoring technique with FOS sensors proved to be practical, cost-effective, and efficient, providing skillful installation.

1.7 An Overview of Dissertation

The use of fiber optic sensors to investigate AASHTO LRFD Design Specifications, AASHTO LFD Standard Specifications, LFD Bridge Rating, Real time Remote monitoring of bridge condition and literature review on needs, development and use of fiber optic sensors to investigate the structural behavior of bridges have been presented in this chapter. Chapter 2 describes the experimental program portion of the dissertation. The experimental program consists of laboratory examination of two concrete beams using FOS to verify the beams cracking state subject to four point static load, field test depicting project tasks (construction sequences) coordination and installation of sensors and monitoring system. The challenge and duration for installation of electricity and telephone at the bridge site is mentioned. Chapter 3 presents a brief description of the old (replaced) and the new bridge, the summary of Design Code formulas and calculations for the new bridge and application of fiber optic sensors for monitoring of structural behavior. Design and analysis of the new bridge by FDOT software programs using LRFD and LFD are presented in chapter 3. Also, presented in chapter 3 are LFD rating of bridge subject to Florida legal loads and finite element modeling for verification of experimental collected data. Chapter 4 illustrates methodology for data collection, truck load testing data and
organization of plots, graphs and table of maximum strains for a duration of one year. In Chapter 5, the current design specification is compared with the analytical results of Chapter 3 and the real time data collected in Chapter 4. Chapter 6 provides a summary of research findings, conclusions and recommendations.
CHAPTER 2. EXPERIMENTAL PROGRAM

This chapter illustrates the installation of fiber optic strain sensors embedded in the concrete deck and on the underside surface of deck on East Bay Road Bridge to be used as an important tool to satisfy the objectives of this research.

There are two objectives of this research. The first objective is a short-term application of Fiber Optic Sensor (FOS) for evaluation of AASHTO bridge design guidelines to investigate the new bridge design method of LRFD and old design method of LFD, evaluate and verify the assumptions and parameters considered in design of East Bay Road Bridge. We then compare the design of the bridge with the data obtained from Fiber Optic Sensors installed in the bridge during the construction. This data is also used to investigate and verify the results of LFD method of bridge load capacity rating.

The second objective is development of a new methodology for damage detection and life cycle evaluation of bridges. To achieve these goals, the long-term application of FOS is essential to build strain history of the bridge by continuous or periodic monitoring of the bridge and evaluation of collected data for monitoring of structural behavior. Literature on the related topic emphasizes on expert installation of sensors for gathering useful and accurate data. Some literature has shown photographs of installed fiber optic sensors but the process
and procedure of installation have not been clearly presented. The
“instruction manual sensoptic fiber-optic sensors Fabry-Perot strain Gauge FOS
series” by RocTest (2000) is the only available source to be considered during
installation of fiber optic sensors. Due to the absence of such vital information
and difficult encounters during installation of sensors, a great deal of emphasis
has been placed in the step-by-step process of embedded strain sensors
installation in concrete media and surface of structural elements. This chapter
describes material type, property, variables and factors in determining the
procedure for installation of fiber optic strain sensors in laboratory and filed
experiment settings.

2.1 Beams Fabrication for Laboratory Test

Two reinforced concrete beams were fabricated for laboratory experiment.
Beam (1) was a 3.5” x 3.5 “x 36” with 4 #3 deformed grade 60 steel (yield
strength of 60 ksi) placed one at each corner, as indicated in Figure 2.1. Stirrups
were #2 grade 40, smooth steel placed at 4 inches on center. The steel clear
cover was 3/4 inches. The concrete compressive strength was 5000 psi. The
wood forms were lightly covered with oil to provide easy form removal and
prevent damage to the beam. Tapping on the sides of forms with rubber mallet
consolidated the concrete in the forms. Beam (2), was a 3.5” x 3.5” x 36”
specimen, had 1 # 3 rebar placed at the bottom middle of the form to simulate
50% steel section loss in flexure, Figure 2.2.
2.2 Laboratory Test Setup

The purpose of the laboratory experiment was to evaluate a new testing system, known as surface mount sensors (blade). Based on the results of laboratory tests, it will be determined to use this system in field experiment or investigate other types of strain measuring sensors. Due to the budget restraint, we did not purchase equipments and material for this experiment. Some of the material and equipment were available in the laboratory and were fabricated or modified to meet the testing requirements. Two strain sensors were purchased and a data conditioner was rented. The testing framework was a rigid welded frame constructed of 3”x 5” steel tubing (Figures 2.3 and 2.4). A hydraulic pump with a pressure gauge and two 30-ton hydraulic jacks were available in the lab (Figures 2.5 and 2.6).
2.3 Data Acquisition System Components (Hardware)

The rented system consisted of a 32-channel Bus system (data acquisition), two surface-mount Fabry Perot fiber optic sensors with the composite laminates (conveniently called “Blade”) and a desktop computer. A communication serial link cable, RS-232 established a link between the Bus system and the computer (Figures 2.7, 2.8 and 2.9).
The surface of the beam was sanded using a 100 grids sandpaper to plane the surface. All loose material was removed and the surface was sanded again using a 200 grids sand paper to provide a smooth surface. The sanded surface was dusted and wiped off with paper tissues, wet with 75% by volume isopropyl alcohol. The surface was wiped several times, each time with a new tissue and in only one direction to avoid surface contamination. A quicker
alternative cleaning was to wash the surface with water, however, this method would require 24 hours for the surface to dry while it may again collect dust and debris. In comparison, a concrete surface cleaned with alcohol can be used immediately. A straight edge was used to verify the surface flatness. Any gap more than one mm is considered excessive and must be filled with putty. Optional bottom CFRP/GFRP sheets may be installed to provide a primary surface for blade sensors installation for the surfaces larger recessed areas.

The accuracy of collected data from FOS is directly related to proper installation of the sensors. As soon as the surface was prepared, a uniform layer of epoxy was placed on concrete surface. The sensor was placed on epoxy and covered with another coat of epoxy (Figures 2.10, 2.11).

Figure 2.10 Two Components Epoxy
Figure 2.11 FOS-N Installed
2.5 Installation of Beam on Reaction Frame

The beam was suspended from the reaction frame by two brackets. Figures 2.12 through 2.15 illustrate positioning of the concrete beam, reaction frame, brackets supporting the beam and 30-tons hydraulic jacks. The hydraulic jacks were seated on 3.5”x 3.5” x 1/2” steel plates and dense rubber sheets to provide a surface for uniform load transfer over the jack’s seats areas (Figure 2.1). The top plates were selected to provide an area for the top part of the jack. The reaction beam was made from two steel tubes and the jacks were not able to push against the reaction beam. The jacks were positioned at 1/3 points.

![Figure 2.12 Load Assembly](image1)

![Figure 2.13 Suspended Brackets](image2)

In addition to the FP sensor described in Section 2.4, a regular strain gauge was installed on the beam and a digital caliper deflection gauge was placed on the reaction frame to monitor beam deflection (Figures 2.16 and 2.17).
2.6 Laboratory Loading Condition

The load through the hydraulic jacks was gradually applied to the beam at 15-psi increments until the first crack appeared on the specimen as shown in figures 20 through 23. The analytical values of strain are calculated as follows:

\[ \text{Stress: } \sigma = \frac{Mc}{I} = \frac{12P \times 1.75 \times 12}{3.5^4} = 1.68 \text{ksi} \]
Uncracked Stress: $\sigma = 0.5 ksi$ (See Figure 2.18)

$$1.68P = 0.5 ksi \quad \Rightarrow \quad P = \frac{0.5}{1.68} = 0.297 \text{kips} = 297 \text{lbs}$$

Given:

$$f'_c = 5000 \text{ psi}, \text{ concrete compressive strength at 28 days}$$

Modulus of Elasticity:

$$E_c = 57000 \sqrt{f'_c}$$

$$E_c = 57000 \sqrt{5000 \text{psi}} = 4030,5086 \text{psi} \approx 4030 \text{ksi}$$

Uncracked strain:

$$\varepsilon = \frac{\sigma}{E_c} = \frac{0.5}{4030} = 0.000125 = 125 \mu \varepsilon$$

Consider the beam cross-section and cracking load, $P_{cr}$:

Calculate neutral axis,

Let,

$$n = \frac{E_s}{E_c} = \frac{29000}{4000} = 7.25$$

CL = 0.75" typical

Bar diameter = 3/8"

Area of steel, $A_s = 0.22 in^2$
Figure 2.18 Experimental Beam Cross Section

\[ f_y = 60 \text{ksi} \]

\[ \bar{Y} \times 3.5 \times \frac{\bar{Y}}{2} + A_{\text{top-steel}} \times 7.25(1 - \bar{Y}) = A_{\text{bottom-steel}} \times 7.25(2.5 - \bar{Y}) \]

\[ 1.75 \bar{Y}^2 = 3.95 - 1.59 = 2.39 \]

\[ \bar{Y}^2 = \frac{2.36}{1.75} = 1.36 \]

\[ \bar{Y} = 1.17 \text{in} \]

Determine cracking moment of inertia, \( I_{cr} \) and cracking load, \( P_{cr} \)

\[ I_{cr} = \frac{\bar{Y}^3}{3} \times 3.5 + A_{\text{top-steel}} \times 7.25(1 - \bar{Y})^2 + A_{\text{bottom-steel}} \times 7.25(2.5 - \bar{Y}) \]

Substitute for \( A_{\text{top-steel}} \) and \( A_{\text{bottom-steel}} \) in the above equation to obtain \( I_{cr} \)

\[ I_{cr} = \frac{1.17^3}{3} \times 3.5 + 0.22 \times 7.25(1 - 1.17)^2 + 0.22 \times 7.25(2.5 - 1.17)^2 = 3.4 \text{in}^4 \]
\[ \sigma_y = \frac{36}{7.25} = \frac{M_{cr}C}{I_{cr}} = \frac{12P_{cr}C}{3.4} \]

Where, \( C = 3.5-1.17-1 = 1.33'' \)

And substitute for \( C \),

\[ \frac{36}{7.25} = \frac{12P_{cr}1.33}{3.4} = 4.96 \]

Thus,

\[ P_{cr} = \frac{4.69 \times 3.4}{12 \times 1.33} = 1.06 \text{kips} = 1,060 \text{lbs} \]

The results of test for investigation and verification of FOS readings in laboratory setting will be compared with analytical values. Two \( \frac{1}{2}'' \) thick plates with 14 in\(^2\) \((4'' \times 3.5'')\) area were placed on the beam under each hydraulic jack’s round base for support and uniform load distribution.

The beam was loaded and loading was increased in 15-psi increment until the first crack appeared under 86.5 psi (1060 lbs). At this time, the loading process was terminated. It was observed that the FOS reading of 260 \( \mu \varepsilon \) for cracking condition was closer to analytical strain value of 245 \( \mu \varepsilon \) than to strain gauge reading of 280 \( \mu \varepsilon \). The FOS strain reading of 130 \( \mu \varepsilon \) for uncracked condition was closer to analytical strain value of 125 \( \mu \varepsilon \) than to gauge reading of 145 \( \mu \varepsilon \).

The analytical deflection of 0.022 inches as shown in Figure 2.19 at cracking condition is close to the gauge reading of 0.026 inches.
The results confirm FOS is more accurate than strain gages. FOS values are closer to analytical values. Strain gauge readings are slightly higher than value of the analytical model.

Figure 2.19 represents cracked and uncracked sections of two experimental specimens with relative yield strength of steel. Stress is 0.5 ksi for uncracked specimen and 1.06 ksi for cracked section respectively.

The beam in the reaction frame was closely examined for the presence and location of any cracks while application of load was in progress. Figures 2.20 through 2.23 illustrate the extent and pattern of cracking under the applied load.
2.7 Conclusions

The entire laboratory testing assembly was performed economically (approximately $3,000.00) and successfully. It was determined that the Fabry Perot composite laminate sensor would be used in the field experiment. The strain values of Fabry Perot are close to the results of analytical strain values of prism. The readings of strain gauge were slightly higher than the strain values of
FOS and analytical strains values. The locations and pattern of cracks were classical and an indication of intended behavior of reinforced concrete beam under applied static load. Fabry Perot sensors will be used in field test.

2.8 Proposed Remote Sensing System

With the emergence of present day technologies, structures can now be monitored remotely from a central monitoring station located several miles away from the field. This remote capability allows continuous monitoring of structures, a condition needed to conduct this research study. Sensors are placed at several critical locations along the structure, and send structural information to the central station. The structure is thus thought of as an intelligent or smart system that is capable of sending information and providing warnings before any major failure.

The proposed remote sensing system follows the above mentioned approach and consists of the following as shown in Figure 2.24.

(a) Fabry-Perot (FP) Fiber Optic Sensors attached to critical locations of the structure.

(b) Fiber Optic Cables to connect the FP sensors to their signal conditioner system.

(c) A signal conditioner system housed in a secured on-site location.

(d) A power supply to charge the signal conditioner provided from nearby power lines.

(e) A phone line connection to connect the signal conditioner to the Internet.
The embedded FP sensors transmit the data to the signal conditioner through Fiber Optic Cables placed in conduits to be protected from the environment. The signal conditioner is connected through a phone line (or DSL connections if available) securely to the Internet, where data could be retrieved and processed easily from the office, with a software program like Lab View.

![Diagram of proposed remote sensing system]

**Figure 2.24 Proposed Remote Sensing System**

The proposed system depicted in Figure 2.24 is currently being installed on the East Bay Road Bridge over Bullfrog Creek in Hillsborough County, Florida as a part of this research project funded by Hillsborough County to monitor the behavior of the bridge under traffic loading. The bridge is considered the first smart structure in the State of Florida. The bridge is a four span, continuous, deck-type structure with a total length of 120’ and width of 55’. The superstructure consists of an 18” cast-in-place reinforced concrete slab, and is supported on pre-stressed pile bents, each consisting of 5 piles as shown in Figure 2.25. FP sensors are bonded to the bottom bars in the mid-spans 2 embedded in the slab,
where the maximum positive bending moments are expected, and bonded to the top bars over the pile bent 3, where the maximum negative bending moments are expected, as shown in Figures 2.26 and 2.27. The signal conditioner system is housed securely to the side of the bridge on a parapet wall. The bridge was opened to traffic in February 2005, and it is expected that live traffic data will be transmitted to the Hillsborough County Bridge maintenance office.

Figure 2.25 Profile of the East Bay Road Bridge

Figure 2.26 FP Sensors Bonded to Reinforcing Steel

Figure 2.27 FP Sensors in Conduits
2.9 Field Experiment

The contractor, “All American Concrete Inc.”, had a construction agreement with Hillsborough County to replace an existing concrete bridge with a cast-in-place reinforced concrete bridge on East Bay Road in Gibsonton, Florida, Figure 2.28. This bridge was a low profile concrete structure built in the early 1970's. This bridge was classified as functionally obsolete due to frequent flooding and its narrow width.

![Figure 2.28 Elevation View of Old East Bay Road Bridge](image)

Coordination of the field experiment effort with the contractor and Hillsborough County project management team was crucial to the success of the experiment. The field experiment was allowed only if it could stay transparent through the construction. Hillsborough County imposes substantial daily penalty on the contractor for any unjustifiable cause of delay in a construction schedule. A field experiment was not considered a justifiable cause of construction delay.
Schedule to set the forms for the superstructure (All components of bridge sitting above the top of the bent cap) and concrete pour was on September 20, 2004 with the completion date of November 1, 2004. The critical time to install the embedded sensors was when placement of reinforcing steel was in progress. Time was of the essence for installation of the sensors since pouring concrete would begin as soon as reinforcing steel was in place. The installation of sensors had to take place parallel to the process of bridge construction.

2.10 Determine Location of Sensors

2.10.1 Transverse Positions of Sensors

The embedded strain sensors were placed under the wheels in a transverse direction. Figures 2.29 show the position of truck wheels. SU4 truck was used to test this bridge under service load. This position configuration meets AASHTO section 3.6 (AASHTO 1994) requirement for trucks occupying 10' of 12' lane.

Figure 2.29 Transverse Positions of Wheels on the Bridge Deck
The shoulder lane was strategically selected for installation of sensors for static load testing of the bridge while open to traffic. The load test of the traffic lane is not practical or safe while the bridge is open to traffic. The bridge must be closed to traffic during the test. The bridge closure process requires a detour route determined by The Hillsborough County Traffic Department and approved by Public Information Services Department. This process is very time consuming and request for bridge closure may not be obtained.

2.10.2 Longitudinal Positions of Axels on the Bridge Deck

The embedded strain sensors for positive moments were bonded to rebars placed at mid-span 2. This is a simplified location very close to the point of maximum positive moment. The SU4 truck axles spacing and weight are shown in Figure 2.30.

![Figure 2.30 Longitudinal Spacing of Axles in SU4 Truck](image)

Figure 2.30 Longitudinal Spacing of Axles in SU4 Truck
2.10.3 Locations of Embedded and Surface Mount Sensors

The positions of sensors were determined on the topside of the bridge deck by measurements taken from the inside face of the traffic barrier. Figure 2.31 illustrates this configuration.

Legends:

(a) Surface mount sensors ASM, BSM, CSM, DSM (FISO-B, Blade) = Sensors bonded with epoxy to the bottom surface of concrete deck.

(b) Surface mount sensors P1 and P2 = these sensors were bonded to concrete, 3/4" below the surface of deck.

(c) Embedded sensors C, D, E, F = Bonded to the bottom surface of rebar on bottom reinforcing steel mat.

(d) Embedded sensors G, H, I, J = Bonded to the bottom surface of rebar on top reinforcing steel mat with epoxy.

Figures 2.32 and 2.33 depict detailed location of sensors within the concrete slab bonded to reinforcing steel, bonded to the surface of concrete and bonded to concrete slightly below the surface from the top of the deck. Step-by-step procedure and techniques are outlined in the following sections of this chapter illustrating the installation of all sensors in this experiment. Three types of sensor are placed in four categories of installation. Three types of sensors are identified as (a) Surface mount, known as FOS-N (blade), (b) Embedded sensors and (c) Embedded temperature sensors. The layout of the sensors identified as shown in Figure 2.31.
Figure 2.31 Locations of 16 Sensors on Two Northbound Spans
2.11 Field Readiness and Planning

Coordination of effort with the contractor was one of the most important first steps in field experimentation. Contract drawings indicate the bridge deck was heavily reinforced. The top and bottom mats consisted of #9 rebar (1.125" diameter) placed 6" on center. The clear space between the bars was 4.87". Placement of the top mat would have made the access to the bottom mat impossible, particularly if very congested #9 reinforcing bars were tied together.
with tie wire as shown in preceding section 2.8, Figure 2.27. The sensors (C, D, E, F) for flexural stresses had to be bonded to the bottom of the bars. While #9 rebars were placed untied on the form, it was an opportunity to bond these sensors to the top of the bars and then turn (twist) the bars 180 degrees in place and tie them together afterward.

The contractor agreed to begin reinforcing steel placement from span 4 instead of span 1. The strategy was to place bottom and top mats in span 4 and span 3 up to bent 3, and place the bottom mat in bent 2. At this point, the author began installation of G, H, I, J and T2 sensors on top mat over bent 3.

The next step was for the contractor to place reinforcing steel for the bottom and top mats in span 1. This would provide an ample time for the author to install sensors C, D, E, F and T1 on bottom mat bars at mid span 2.

2.12 Methodology and Procedure

Manufacturer of fiber optic sensors (RocTest) has recommended guidelines for installing sensors within and on structural members. However, the quality of installation would be as good as knowledge and experience of the installer. Accuracy and good quality of data is directly related to proper installation of sensors. The author has exercised a great deal of patience and care during each step of every sensor installation. Numerous photographs and detailed descriptions are presented in every step of the sensors and equipment installation process.
2.13 Surface Preparation of Steel Bars

An electric angle grinder connected to an inverter connected to the car's battery (there was no electric power at the bridge site) was used to grind the surface of the steel rebar flat and smooth to install the sensor. An area of approximately 3" long and 3/8" wide on #9 grade 60 rebar (deformed) was grinded flat. A straight edge was used to verify the flatness of the area. Dry abrading was continued with 200 and 300 grit silicon carbide papers to achieve a flat, smooth surface. It was rinsed with M-prep Neutralizer 5A (from Measurement Group) and wiped with paper tissue such as kimwipe wipers. The area of the sensor was wet with M-prep conditioner A and abraded the area with 400 grit silicon carbide paper. The sensor area was checked frequently with a straight edge for flatness and smoothness.

The abraded area was wiped with Isopropyl alcohol and rinsed with M-prep Neutralizer 5A. The area was wiped unidirectional using the wipes, using a new wipe after each wiping to avoid contamination of the sensor area for bonding. The sensor was placed on the rebar and held down with electric tape, one inch away from micro capillary. A very small drop of 5 minutes epoxy was placed on incoming fiber optic, approximately 1/8" from micro capillary. As soon as 5 minutes epoxy was cured, the adhesive was prepared and applied with a linear motion along the entire length of the gage (figures 2.34 to 2.42).
The remainder of adhesive was applied to the optical fiber up to the fiber jacket. For additional protection, M-coat Protective coating was applied to the sensor (Figure 2.38). After this protective coating dried, a rubberized waterproof sheet, nitrite rubber sheet was wrapped around the sensor as shown on Figure 2.39. All material named in this section were purchased from “Measurement Group”. Figures 2.33 through 2.40 are the pictorial presentation of installation of the sensor on reinforcing steel. These figures are used with permission from Roctest Canada.

Immediately after application of adhesive over sensitive region of sensor, a piece of Mylar tape was placed over it to keep the sensor in a good contact with surface of rebar. Figures 2.37 through 2.39 show the application of 5-minutes epoxy to sensitive region of gauge and optical fiber.
Figure 2.35 M-Bond 5 Minutes Adhesive

Figure 2.36 M-Coating and Neutralizer

Figure 2.37 Area of Rebar to Place the Sensor on

Figure 2.38 Sensor Secured on Rebar
Figure 2.39 A Very Small Drop of 5-Minutes Epoxy Placed on Incoming Optical Fiber

Figure 2.40 Correct and Incorrect Procedure for Sensors with Epoxy
Figure 2.41 Mylar Tape was Applied to Sensor to Keep it in Good Contact with Rebar

Figures 2.42 Final Procedural Steps of Sensor Installation
Figures 2.43 and 2.44, illustrate the actual final steps of filled installation of sensor.

Figure 2.43 Placing Mylar Tape on Sensor Optic Fiber

Figure 2.44 Sensor Wrapped in Nitrite Rubber and Placed in Conduit

Soon after sensors were bonded to rebar, FTI-10, a single channel data logger was used to test the sensors and verify bonded condition (Figure 2.45). The readings on the data logger are in nanometer (nm) and verify a successful bond interface between the sensor and rebar.

Figure 2.45 Single Channel Data Logger Reads the Strain of Sensor in nm
The strain of sensor in nm is calculated by dividing FTI-10 reading (gauge zero, reading at no load) by the gauge length. The gauge length of each sensor is unique to that sensor. In this case, the strain in gauge “C” is:

\[
\text{Strain} = \frac{\text{FTI-10}}{\text{gauge length}} = \frac{14993.5}{2.09} = 7456 \text{nm}
\]

The following procedures were proposed by Roctest to interpret the reading of Fabry Parot sensor. The relationship between the length of the cavity \( L_{\text{cavity}} \) and the strain \( \varepsilon \) is determined by the following formula.

\[
\varepsilon = \frac{\Delta L}{L_{\text{gage}}} = \frac{\left( L_{\text{cavity}} - L_0 \right)}{L_{\text{gage}}}
\]

Where:
- \( L_{\text{cavity}} \) = Length of Fabry-Perot cavity, in Nanometers and varies between 8000 and 23000 in nm
- \( L_{\text{gage}} \) = Gage length, the space between fused welding, mm
- \( L_0 \) = Initial length of Fabry-Perot cavity, in nanometer
- \( \varepsilon \) = Total strain measurement, in \( \mu \varepsilon \)

The total strain \( \varepsilon \) is the raw strain obtained directly from FOS readings with readout units after the gage factor has been defined in readout memory and selected.

Therefore:
- \( \varepsilon = \varepsilon_1 - \varepsilon_0 \)

\( \varepsilon \) = Total strain measurement, in \( \mu \varepsilon \)

\( \varepsilon_1 \) = Current strain, in \( \mu \varepsilon \)

\( \varepsilon_0 \) = Initial strain, in \( \mu \varepsilon \)
This total strain includes the mechanical strains and thermal strains in the investigated structure. The real strain induced by the stress due to thermal change can be computed with the following formula:

$$\varepsilon_r = \varepsilon - \beta \cdot (T_1 - T_0)$$

Where:
- $$\varepsilon_r$$ = Real strain, in με
- $$\varepsilon$$ = Total strain reading, με
- $$T_1$$ = Temperature reading of structure, in °C
- $$T_0$$ = Initial temperature reading of structure, in °C
- $$\beta$$ = Thermal expansion factor of structure in $$\mu m / m / °C$$ on which the sensor is fixed. The thermal expansion factor ($$\beta$$) can be obtained from laboratory test. The $$\beta$$ factor range for steel is:

$$10 \text{ } \mu m / m / °C < \beta > 16 \mu m / m / °C$$

A numerical example of this procedure is presented as follows:

Given: $$\varepsilon_0 = 2002.2$$ units, Initial strain (με) reading of FOS with fiber optic readout unit

$$\varepsilon_i = 2407.8$$ current strain (με) reading of FOS with fiber optic readout unit

$$T_0 = 20.2 \text{ °C}$$, initial temperature reading of structure

$$T_1 = 26.2 \text{ °C}$$, current temperature reading of structure

$$E_T = 12 \text{ } \mu m / m / °C$$, thermal expansion factor of structure
Calculate the strain ($\varepsilon$):

$$\varepsilon = \varepsilon_1 - \varepsilon_0 = 2407.8 - 2200.2 = 207.6 \mu\varepsilon$$

and the real strain ($\varepsilon_r$) can be calculated as follows:

$$\varepsilon_r = \varepsilon - \beta \times (T_1 - T_0)$$

Therefore,

$$\varepsilon_r = 207.6 - 12 \times (26.2 - 20.2) = 135.6 \mu\varepsilon$$

In case of sensors for flexural condition, the rebars were turned 180 degrees to place the sensors facing the forms. The successful installation of sensors and fiber optic cables was followed by a well thought protection method to assure their sound condition in the system. The following section represents planning and installation of this protective system.

### 2.13.1 Protecting the Sensors and Optical Fibers in the Slab

The micro capillary section of the sensors is glass and thus is very sensitive to scratch and impact. Also, optical fiber is very sensitive to bends, kinks, sharp curves and impact during the final steps of installation and during the bridge construction. All sensors bonded to rebars were wrapped in a thick nitrite rubber sheet for protection against impact and moisture. Fiber optic cables were inserted into one-inch diameter schedule 40 PVC conduits. Large radius 90 degree elbows (sweep) were used to avoid sharp bends and kinks in the fiber optic cables. The conduits were guided through the crowded rebar mats to the edge of the slab, openings in the conduits were sealed with nitrate rubber sheet and caulking, then the conduits were tightly secured to the rebars with steel wires. Photos in Figures 2.46 through 2.49 illustrate this process.
2.13.2 Protection of Fiber Optic Cables Out of Slab

Conduits containing fiber optic cables were brought unto the forms to the side (edge) of the slab. At this area, sensors and fiber optic cables are the most vulnerable to the construction activities such as worker’s traffic and placing and removing the forms. A 2” diameter hole was drilled to allow the conduit to exit.
the slab (Figures 2.50 through 2.55). Three small boxes (12” × 12” × 12”) were fabricated to house the cables at the exit points.

Other alternatives for the cables to exit the forms were investigated. One option was to exit from the underside of the slab at the bottom. However, removal of the slab forms with heavy equipments would have damaged or severed the cables during the process.

**Figures 2.50 G, H, I and J Sensors in Conduit Exiting the Forms**

**Figure 2.51 C, D, E and F Sensors in Conduit Exiting the Forms**

**Figures 2.52 FO Cables in the Box**

**Figure 2.53 Forms are Removed**
2.13.3 Special Installation of FOS-B, P1 and P2

Typically, surface mount sensors are bonded to the surface of the structural elements subsequent to surface preparation. In this case, it was determined to install two FOS-B sensors on top of the concrete deck over intermediate bent 2. If the sensors were installed on the top of the slab, they would be exposed to traffic and the harsh environmental elements and damage to their integrity would be imminent. Another suggested alternative was to leave the sensors floating in freshly poured concrete. This method was not acceptable since the position of the sensors could not be guaranteed with any degree of certainty during concrete placement.

The last alternative was to place the sensors below the surface of hardened concrete. The positions of two sensors were marked on the surface of the completely cured concrete slab. It was determined to grind the top of the deck, ¾" below the surface to provide a uniform flat area to bond the sensors. An investigation was carried out to find a specialized tool such as a router to cut
a ¾” × 2” × 18” groove into the concrete deck. This type of tool was not found.

As an alternative an electric 4.5” angle grinder and a diamond blade was used to cut and grind the concrete, ¾” below the surface. Two ¾” deep lines were cut in a designated area over bent 2. Side to side motion of angle grinder cut the concrete to the desired depth. The cutout area was cleaned and dusted. The surface of the area was wiped with a piece of clean cloth and isopropyl alcohol. A thin layer of epoxy was placed on the dried and cleaned cutout area to provide a uniform and level bonding surface area for the sensors. This installation procedure assured sensor protection against traffic and groove cuts in the slab for vehicular wheels traction and to avoid the danger of hydroplane action.

Figure 2.56 through 2.68 show the process for this installation.
Figure 2.58 Edge Bedding for P1 and P2

Figure 2.59 Edge Bedding for P1 and P2 with PVC

Figure 2.60 Beddings are Prepared and Ready to Install P1 and P2 Sensors and Optical Cable
Figure 2.61 Sensor P1 is Installed  
Figure 2.62 Sensor P2 is Installed

Figures 2.63 P1 and P2 Sensors  
Figure 2.64 P1 and P2 Sensors out of the Slab at the Edge of the Bridge
Figure 2.65 Material were Used to Install Sensor P1 and P2 on the Deck Over Bent 2

Figure 2.66 Protective Box

Figure 2.67 Final Step, P1, P2 Sensors in the Protective Box
2.13.4 Protecting Fiber Optic Cables in PVC Conduits

Accessibility to specific areas of the bridge for the installation of fiber optic sensors and equipment, installation of a protection system for fiber optic cables and data logger and safety of personnel were primary concerns during the planning and construction. The details of different alternatives were carefully investigated. For example, the two points of exit for conduits and fiber optic sensors were evaluated. One point of exit was the underside of the deck. This option was not practical because, (1) to attach the fiber optic cable protective
housing to the bridge underside required an extensive scaffold setup or a cherry picker. Employment of either technique was not within the budget limit, (2) the method of removal of the forms from the underside of the bridge deck was by sliding the forms out through a narrow space between the forms support and the concrete deck itself. This motion would shear off the protective housing and the fiber optic cables, which were placed within the housing.

The point of exit from the side of bridge deck was a practical alternative since it was accessible and the forms removal would not damage the fiber optic sensors.

The task of installation of the measuring system required detailed coordination with the authorities in Hillsborough County, County Wide Division (Maintenance Headquarter) and the bridge contractor. The Countywide Division had offered assistance, providing scaffold and manpower to install one-inch schedule 40 PVC conduits on the side of the bridge deck. As it is shown in Figure 2.69 and 2.76, this was a very difficult, if not impossible task. The invaluable assistance of County Wide Division made the safe installation of conduits, cables and DMI possible. Access to the side of the bridge deck was neither safe nor practical without a special type of bridge parapet mount scaffold. In Figures 2.69 and 2.70 the difficulties and inaccessibility are shown to the bridge parapet for installation of conduits and the various equipments.
Figures 2.69 Accessibility Problem at the Bridge Edge

Figures 2.70 Accessibility Problem to Install Conduits

Figures 2.71 through 2.76 show the step-by-step process of installing scaffolds to use as a safe platform to install conduits carrying FO cables and equipments on the bridge.

Figures 2.71 Scaffold Installation from the Top

Figure 2.72 Scaffold Installation from the Bottom
The following Figures 2.77 and 2.78 clearly show the potential to damage unprotected fiber optic cables and sensors.
As it has been illustrated in previous figures, sensors were installed successfully (e.g., the signals were transmitted from sensors to FTI-10, data reception was verified) and routed to the eastside edge of the bridge without any incident. The conduits were attached to the side of the bridge. The successful working condition of sensors is owed to protective measures taken during the installation. Installation process of conduits, DMI and connection of fiber optic cables to DMI are shown in Figures 2.79 through 2.84., and installation of telephone and electric power at the bridge are shown in Figures 2.85 and 2.86.

Fiber optic cables were fished through conduits to the DMI unit. The connectors at the end of the cables were cleaned and connected to the ports inside the DMI unit (Figures 2.83 and 2.84).
Figure 2.79 Attaching Conduit to the Bridge

Figure 2.80 Fishing Cables Through Conduit

Figure 2.81 Conduits Entering DMI

Figure 2.82 Conduits are Attached to the Bridge and Connected to the DMI

Figure 2.83 FO Cables are Guided Through Conduit into DMI System

Figure 2.84 FO Cables and Sensors are in DMI Box
2.14 Installation of Electric Power and Telephone on the Bridge

The power lines came down from the electric pole to a hand hole box at the base of the electric pole, about 160 feet from the point of installation on the bridge. The telephone box was also about 160 feet from the bridge located near the electric pole. Two 2” conduits were placed 2’ below the ground surface in a trench. Telephone and electric lines were pulled through the conduits and housed on the bridge (Figures 2.85 and 2.86).

Figure 2.85 Telephone Line is Secured on the Bridge
Figure 2.86 Electric Line is Secured on the Bridge

Telephone cable and electric wires were extended from the telephone box and disconnect box to DMI and were connected to a telephone jack and a receptacle. The purpose of direct electric power to DMI was to have an uninterrupted power supply to DMI. DMI is supplied with rechargeable battery pack, however, recharging the battery was only possible while connected to DMI.
A generator and a power inverter were taken to bridge to charge the battery. This process was inconvenient and took about 6 hours for recharging the battery.

All previously described crucial steps were taken carefully to provide an objective, sound framework for SU4 truck static and dynamic load-test. The process for this task was evaluated in detail prior to commencement and is presented below. Six different positions (cases) were assigned for this static load-test. Figure 2.87 depicts the layout of the plan of action to perform these six different load cases.

---

**Legend:**

- Northbound direction of traffic
- Southbound direction of traffic
- Two-lane truck load-test
- Centerline of span 1
- Centerline of span 2

<table>
<thead>
<tr>
<th>Case</th>
<th>Diagram</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>![Diagram Case 1]</td>
</tr>
<tr>
<td>2</td>
<td>![Diagram Case 2]</td>
</tr>
<tr>
<td>3</td>
<td>![Diagram Case 3]</td>
</tr>
<tr>
<td>4</td>
<td>![Diagram Case 4]</td>
</tr>
<tr>
<td>5</td>
<td>![Diagram Case 5]</td>
</tr>
<tr>
<td>6</td>
<td>![Diagram Case 6]</td>
</tr>
</tbody>
</table>

*Figure 2.87 Six Cases of Static Truck Load-Test*
Section 2.15 describes and illustrates the positioning of the trucks coincident with six load cases shown in figure 2.87.

2.15 Truck Static Load-Test

The locations of the sensors as described in section 2.10.3 and shown in Figure 2.29, were marked with white paint on the bridge top surface on the eight feet wide shoulder and the twelve feet northbound traffic lane (Figure 2.90).

The marked positions of the sensors on the bridge deck on the northbound lane matched the six load cases shown in Figure 2.87. Figures 2.88 through 2.96 show the positions of SU4 truck(s) during the static load-test.

Two full capacity (70 kips) SU4 trucks were selected for the load test. Six truck positions for static load were selected. The center of the rear three axles of the truck was positioned over the sensors. Sensor locations were marked on the bridge deck topside (Figure 2.90) in accordance with the layout planning shown in sub-section 2.10.3 Figure 2.29. The trucks were driven with crawl speed to the exact positions. The readings were taken after a 10 to 15 second delay, reading process was stopped and then the trucks were repositioned for the next load case.
The marked positions of the sensors on the bridge deck on northbound lane matched the six load cases shown in Figure 2.87. Figures 2.89 through 2.94 show the positions of SU4 trucks during the static load-test.
An on site laptop computer was used to collect the strain readings from DMI data conditioner via RS-232 communication cable (Figure 2.95).
2.16 Conclusions

Collected data from all six-load cases were stored in the computer. The strain values of all 16 sensors for each one of the six load cases are tabulated in Table 3.5 and graphical representation of this data is given in Chapter 3, section 3.0.

The contour of strain values at the locations of all 16 sensors for load case 1, 2, 3, 4, 5, and 6 and finite element model for load case 1, 2, and 3 and beam model analysis for case 1, 2, and 3 are also presented in Chapter 3.
CHAPTER 3. DESIGN AND ANALYSIS

3.1 Introduction

This chapter describes the application of LRFD and LFD design and bridge rating by LFD method. The formulation of LRFD and LFD methods are compared to initiate discussion and conclusions about relevance and benefits of using each method in design.

The design steps and formulas for East Bay Road Bridge using LRFD and LFD design methods are presented and the results are compared with the results of field static load test. The means of comparison between LRFD and LFD designs and actual bridge load test are based on the strain values obtained from DMI (signal conditioner) readings through use of fiber optic sensors. The use of SAP computer software for modeling and verification of results is also presented. In addition, frame analysis using the program MASTAN was also performed.

3.1.1 LRFD Code vs. AASHTO Standard Specifications

An extensive laboratory-testing program was conducted to investigate the shear strength of the prestressed concrete girders and published in an article titled “Shear Behavior of Full Scale Prestressed Concrete Girders: Comparison Between AASHTO Specification and LRFD Code”, Shahawy and Barrington, (1996). The test shear strengths are compared with predictions based on the
1989 AASHTO Standard Specifications for Highway Bridges and the application of the 1994 AASHTO LRFD Specifications. The results show that the application of the 1989 AASHTO Specifications gives a much better prediction of shear strength than the LRFD provisions. The average value of $V_{\text{test}} / V_{\text{LRFD}}$ was 1.37 vs. $V_{\text{test}} / V_{\text{AASHTO}}$ at 1.2.

The shear strength prediction ($V_n$) of 1989 AASHTO Specifications (LFD) and LRFD Code were compared with the result of tests. The shear strength prediction ($V_n$) of 1989 AASHTO Specifications is more in agreement with the result of tests than LRFD Code.

Kulicki et al. (1996), conducted LRFD method calculations for shear capacity of a test beam and AASHTO Method Calculations for shear capacity of an identical test beam. The LRFD values calculated by Kulicki were different than the values shown by Shahawy. However, Kulicki’s calculated values for LFD (AASHTO Specifications) were nearly similar values to those of Shahawy’s calculations.

At the conclusion of a discussion paper, Kulicki et al, implies that the LRFD Values are more variable than those of 1986 AASHTO Code. Further more, Shahawy and Kulicki demonstrated that the ratio of value of the LRFD Code to the value of test is at 52 percentile prediction, while the ratio of value of the LFD code to the value of test is at 21 percentile. Due to the variability of values of LRFD Code calculations to verify the test results, one could conclude that there might be some degrees of inconsistency in interpretation of LRFD Code provisions by different individuals. This perception of inconsistency would
adversely affect the level of reliability in LRFD code calculations. LRFD method is said (Shahawy and Kulicki, 1996) to be more conservative and would take more computation time than LFD method. The LRFD specifications were approved by AASHTO for use as alternative specifications to the AASHTO Standard Specifications for highway Bridges, LFD. Technically, LRFD was meant to be a parallel design method with LFD but inadvertently has taken a sharp turn away from LFD. The variability in design parameter and formulation between LFD and LRFD codes are illustrated in the next section.

3.1.2 LFD Design Method

The design live load for LFD method is either truck load or lane load. The live load design is either HS20-44 truck (44 denotes the publication of the 1944 edition of AASHTO Specification), Figure 3.1, or alternate military loading of two axles four feet apart with each axle weighing 24,000 pounds, Figure 3.2.

Lane load on continuous span as in the case of the East Bay Road Bridge is the combination of 0.64 kips per foot uniformly distributed load over the span and 18 kips concentrated load at the center of span. For maximum positive moment, only one concentrated load is used per lane as shown in Figure 3.3. In case of maximum negative moment, the second concentrated load is placed in series in the adjacent lane as shown in figure 3.4. The maximum live load moment calculated for the Loading condition is shown below:

\[
Design.Load = 1.3DL + 2.17LL(1 + Impact)
\]
Where

The impact factor is given by \( I = \frac{50}{L + 125} \)

for the East Bay Road Bridge, the span length is either 27’ or 33’ for the positive moment and for the negative moment, is the average of two adjacent spans (e.g. \( \frac{27 + 33}{2} \)). The distribution width, \( E_D \) is given by:

\[ E_D = (4 \text{ ft} + 0.06L) \times 2 \]

In the case of the East Bay Road Bridge, \( E_D = (4 \text{ ft} + 0.06 \times 30) \times 2 = 11.6 \text{ ft} \). The number of Design Lanes is determined by taking the integer part of ratio of \( \frac{W}{12} \), where, \( W \) is the clear roadway width in feet between the curbs or traffic barriers. Therefore the number of Design Live Lane for the East Bay Road Bridge would be \( \frac{40}{12} = 3.3 \) or 3.
Figure 3.1 Notional HS20-44 Truck, Axles, Wheel Spacing and Weights of Each Axle

24 kips              24 kips
8,000 LBS.  32,000 LBS.  32,000 LBS.
6,000 LBS.  24,000 LBS.

Figure 3.2 Alternate Military Loading

18 kips for moment   26 kips for shear
0.64 kips/ft uniform load

Figure 3.3 Lane Load on Continuous Span for Positive Moment
3.2 LRFD Design Method

The design live load for LRFD method is HL-93 loading and that is the combination of design truck or design tandem with design uniform lane load. The Design truck load is HS20-44 truck as shown in Figure 3.1, section 3.1.1 and Design Tandem is as shown in Figure 3.5 with each axle weighing 25,000 pounds. The design live load configuration for LRFD method is illustrated in Figure 3.6. Design truck load is increased by 1.33, which is the dynamic impact load allowance.

In form of a formula, Figure 3.5 appears as:

**Figure 3.5 LRFD Design Load Combinations (HL-93), Positive Moment**

In form of a formula, Figure 3.5 appears as:
The interpretation of LRFD Code provision for Design Load to produce maximum loading condition for negative moment is illustrated by Figure 3.5. The code allows for a 10% reduction for this case. The formulated form of Figure 3.5 is presented as:

\[
\text{Factored Design Load} = 1.25DL + 1.75(\text{Design Lane} + 1.33\text{Design truck}) \times 0.9
\]

![Figure 3.6 LRFD Code, Design Load to Produce Maximum Negative Moment](image)

Figure 3.6 LRFD Code, Design Load to Produce Maximum Negative Moment

Figure 3.7 represents cross section of East Bay Road Bridge superstructure.

![Figure 3.7 Distributions width \(E_D\), this Figure Shows Actual Cross Section of East Bay Road Bridge Superstructure](image)

The distribution width of slab \(E_D\) under the design live load based on LRFD and LFD Codes are defined as follows

\[
E_{D-LRFD} = 84'' + \frac{1.44\sqrt{LW}}{12}, \text{ and}
\]
\[ E_{D-LFD} = (4 \text{ feet} + 0.06L) \times 2, \]

Where

L = Actual span length in feet
W = Physical edge-to-edge of bridge in feet

Considering the East Bay Road Bridge with W = 55 feet and L = 30 feet, the distribution width in LRFD method can be calculated as:

\[ E_{D-LRFD} = 84'' + \frac{1.44 \sqrt{30 \times 55}}{12} = 11.874'' \]

And similarly, distribution width in LFD method can be calculated as:

\[ E_{D-LFD} = (4 \text{ feet} + 0.06 \times 30) \times 2 = 11.6' \]

The comparison between \( E_{D-LRFD} \) and \( E_{D-LFD} \) indicate that LRFD method is more conservative than LFD method, although by a small margin.

### 3.3 Bridge Load Rating Using Load Factor Method

The Load Factor Design (LFD) method has been predominantly used in analysis of Bridge Load Rating. LFD method was presented in the first edition, first printing of AASHTO Manual for Maintenance Inspection of Bridges in July 1970. Since then, several editions have been printed. Second edition, first printing was out in June 1974 and the fist printing of third edition was in January 1979. There has not been any significant modification in formulation and application of the LFD method during this period.
The AASHTO manual for maintenance inspection of bridges requires highway bridges to be rated at two load levels, either by load factor or by working stress methods.

**3.3.1 Operating Rating**

At the first or upper level, rating is referred to as Operating Rating. The operating rating will result in the absolute maximum permissible load level to which the structure may be subjected. Based on the 1979 AASHTO manual for maintenance inspection of bridges, the following expressions has been used to determine the operating rating of structures.

Operating Strength Analysis General expression:

\[
\phi S_u = 1.3 \left[ S_D + \left( RF \right) \left( S_L + I \right) \right]
\]

**3.3.2 Inventory Rating**

At the second or lower level, rating is referred to as inventory rating. The inventory rating will result in a load level, which can safely utilize an existing structure of an indefinite period of time. Based on the 1979 AASHTO Manual for maintenance inspection of bridges, the following expressions has been used to determine the inventory rating of structures.

Inventory Strength Analysis General expression:

\[
\phi S_u = 1.3 \left[ S_D + \left( \frac{5}{3}RF \right) \left( S_L + I \right) \right]
\]

where

\[\phi = \text{capacity reduction factor as per standard specification for highway bridges}\]
\[ S_u = \text{ultimate theoretical strength} \]
\[ S_D = \text{effect of dead load} \]
\[ S_L + I = \text{effect of live load plus impact from the rating vehicle} \]
\[ RF = \text{rating factor} \]

For concrete members, the code specifies for strength and serviceability, the area of tension steel at yield to be used in computing the ultimate moment capacity not to exceed 75 percent of the required steel for balanced condition.

The Code, further specified the yield strength of steel shown in Table 3.1.

**Table 3.1 Yield Strength of Different Grades of Steel**

<table>
<thead>
<tr>
<th>Reinforcing Steel</th>
<th>Yield Point ( F_y (\text{psi}) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unknown steel prior to 1954</td>
<td>33,000</td>
</tr>
<tr>
<td>Structural Grade</td>
<td>36,000</td>
</tr>
<tr>
<td>Intermediate Grade and unknown after 1954</td>
<td>40,000</td>
</tr>
<tr>
<td>Hard Grade (Grade 50)</td>
<td>50,000</td>
</tr>
<tr>
<td>Grade</td>
<td>60,000</td>
</tr>
</tbody>
</table>

The AASHTO manual for maintenance inspection of bridges (17th edition), specify the following procedure for Bridge Rating using LFD. The moment live load, \( M_{LL} \) is due to application of the load combination,

\[ 1.3DL + 2.17LL(1 + \text{Im pact}) \]

To determine the Rating Factor, \( RF \) for the Operating Level, all six Florida legal trucks, SU2, SU3, SU4, C3, C4, C5 and two design vehicles, described in
table 3.2, HS20-44, HL-93 and military loading must be investigated and the rating factor, $RF$ for all nine cases must satisfy,

$$RF = \frac{M_u - 1.3M_{DL}}{1.3M_{LL} (1 + \text{Lmpact})} > 1$$

In determination of rating factor for the inventory level, only design truck, HS20 must be investigated and $RF$ for this case must satisfy,

$$RF = \frac{M_u - 1.3M_{DL}}{2.17M_{LL} (1 + \text{Lmpact})} > 1$$

In a simplified form, $RF$ for Inventory level can be obtained in one step, multiply $RF$ of operating level by $\frac{1}{1.67}$.

**Table 3.2 Florida Legal Load and Design Live Load Trucks**

<table>
<thead>
<tr>
<th>Truck</th>
<th>Description of trucks</th>
<th>Gross Vehicle Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>SU2</td>
<td>Single Unit 2 axles, gross vehicle weight</td>
<td>GVW = 34.0 kips</td>
</tr>
<tr>
<td>SU3</td>
<td>Single Unit 3 axles, gross vehicle weight</td>
<td>GVW = 66.0 kips</td>
</tr>
<tr>
<td>SU4</td>
<td>Single Unit 4 axles, gross vehicle weight</td>
<td>GVW = 70.0 kips</td>
</tr>
<tr>
<td>C3</td>
<td>Combination, tractor and trailer, 3 axles</td>
<td>GVW = 56 kips</td>
</tr>
<tr>
<td>C4</td>
<td>Combination, tractor and trailer, 4 axles</td>
<td>GVW = 73.3 kips</td>
</tr>
<tr>
<td>C5</td>
<td>Combination, tractor and trailer, 3 axles</td>
<td>GVW = 73.21 kips</td>
</tr>
<tr>
<td>HS20</td>
<td>A notional Design Truck</td>
<td>GVW = 72.0 kips</td>
</tr>
<tr>
<td>ST5</td>
<td>Tractor pulling Tandem Trailers</td>
<td>GVW = 80 kips</td>
</tr>
<tr>
<td>HL-93</td>
<td>A notional Design Truck</td>
<td>HS20/Tandem + lane</td>
</tr>
<tr>
<td>military</td>
<td>Two axles four feet apart</td>
<td>24 kips each axle</td>
</tr>
</tbody>
</table>
3.4 Steps in Designing of East Bay Road Bridge

The Florida Department of Transportation dictates the body of codes and specifications to be used in design of a bridge structure. The required codes and specifications are described in the following sub-sections. Two vital information are necessary to guide the structural engineer in determining the specifics of the design of a bridge. First, bridge hydraulic recommendation is the information required to establish the bridge vertical alignment. Second, report of core boring is required to establish the foundation type. (e.g. Piling, pier, Spread Footing etc.)

3.4.1 General Specifications

The Florida Department of Transportation standard specification for road and bridge construction, 2000 edition and supplemental thereafter, was used for design.

3.4.2 Design Specifications

3.4.3 Design Method

Load and Resistance Factor Design Method (LRFD)

3.4.4 Design Loading

(a) Dead Load, Unit weight of reinforced concrete = 0.150 kcf
(b) Future wearing surface = 0.015 kcf
(c) Traffic railing barrier = 0.418 klf each
(d) Live load HL-93 Loading

3.4.5 Material Property

Cast-in-place Deck, 4500 PSI minimum compressive strength at 28-days. All Reinforcing Steel are ASTM A615 Grade 60.

3.4.6 Code Distribution Width, $E_D$

Equivalent design distribution width per lane = 11.874 ft

3.5 Analysis

FDOT live load generator was used to obtain positive and negative moments for service live load. Dead load moment was obtained from beam analysis. The deflection and moment envelopes for the different truck configuration are shown in Figures 3.8 thru 3.16. The results of positive and negative live load moments and dead load moments for Florida legal load
configurations, SU2, SU3, SU4, C3, C4, C5, and notional design live load configurations, HS-20 and HL-93 are tabulated in Table 3.3.

### 3.5.1 Service Moments

The following live load positive and negative moments were obtained from Live Load Generator. Dead load moment was obtained from beam analysis.

\[
M_{pos} = \begin{bmatrix} 250.6 \\ 170.8 \\ 135 \end{bmatrix} \quad \text{Positive live load moment} \\
M_{neg} = \begin{bmatrix} 6.250 \\ -135 \end{bmatrix} \quad \text{Negative live load moment} \\
M_{dead} = \begin{bmatrix} 135 \end{bmatrix} \quad \text{Dead load moment} \\
\text{kip} - \text{ft}
\]

### 3.5.2 Cracked Section Analysis

The major assumption made in designing the East Bay Road Bridge was that the deck would crack under service load. Later in this chapter, the results of truck load test would indicate that the cracked section assumption is very conservative.

Given design parameters

\[
E_D = 11.874, \text{ft} \\
\text{Tributary width for a single truck}
\]

\[
\text{spacing}_{pos} = 6, \text{in} \\
\text{Spacing of rebars for top and bottom mat, center to center of the bars}
\]

\[
d_{cover} = 2, \text{in} \\
\text{Clear Cover for reinforcing steel}
\]

\[
d_{cpos} = 18\text{in} - d_c - \frac{9}{8}\text{in} \cdot \frac{1}{2} \\
d_{cpos} = 1.286, \text{ft} \\
\text{Depth to C.L. of steel}
\]

\[
n_{bar} = \frac{b}{\text{spacing}_{pos}} \\
\text{Number of bars per design width of slab}
\]

\[
n_{bar} = 23.475
\]
\[ A_{s} \text{pos} = n_{\text{bar}} \times 1 \text{in}^2, \quad A_{s} \text{pos} = 24 \times 1 \text{in}^2 = 24 \text{in}^2 \] Total steel area in tributary width.

Calculate effective tension area of concrete around the flexural reinforcement

\[ A = \frac{(b) \times (2d_c)}{n_{\text{bar}}} \]

Substitute for \( b \), \( d_c \) and \( n_{\text{bar}} \), the tension area of concrete is:

\[ A = 24 \text{in}^2. \]

Knowing the values of \( z \), \( d_c \) and \( n_{\text{bar}} \), the service limit state stress for reinforcing steel is given by

\[ f_{sa} = \min \left[ \frac{z}{(d_c \times A)^{1/3}} \times 0.6x f_y \right] \]

thus:

\[ f_{sa} = 36.0 \text{ ksi} \]

Calculate the neutral axis of the section to determine the actual stress in reinforcing steel. There is an iterative process, therefore assume an initial value of \( X_{Na} = 4.8 \text{ in} \)

Given:

\[ \frac{1}{2} bX^2 = \frac{E_s}{E_{c\text{slab}}} A_{s} \text{pos} \left( d_{s} \text{pos} - X \right) \]

and, the result is

\[ X_{Na \text{pos}} = 5.3 \text{ in} \]

Calculate the tensile force in reinforcing steel due to the service limit state moment.
Given:

\[ T_s = \frac{M_{pos}}{d_{s\ pos} - \frac{X_{NA\ pos}}{3}} \]

and the tensile force,

\[ T_s = \begin{pmatrix} 220 \\ 150 \\ 119.4 \end{pmatrix} \text{ksi} \]

Calculate actual stress in reinforcing steel due to the service limit state moment.

Given:

\[ f_{s\ actual} = \frac{T_s}{A_{s\ pos}} \]

and the stresses are,

\[ f_{s\ actual} = \begin{pmatrix} 9.3 \\ 6.3 \\ 5.0 \end{pmatrix} \text{ksi} \]

Given:

\[ E_s = 29000\text{ksi} \quad \text{Modulus of Elasticity of reinforcing steel} \]

\[ E_{c\ slab} = 3.475 \times 10^3\text{ksi} \quad \text{Modulus of Elasticity of concrete} \]

Substitute the values of stress and the modulus of elasticity in strain formulas:

\[ \varepsilon = \frac{f_{\text{steel}}}{E_{\text{steel}}} \]

\[ \varepsilon = \begin{pmatrix} 319.471 \\ 217.74 \\ 173.376 \end{pmatrix} \quad \begin{align*} &\text{Positive live load moment} \\ &\text{Negative live load moment} \\ &\text{Dead load moment} \end{align*} \]
3.5.3 Uncracked Section Analysis

It is necessary to obtain the steel strain of uncracked section to compare with the actual steel strain under service load. The first step is to determine the center of gravity, \( y_{CG} \)

\[
y_{CG} = \frac{\left( \frac{b \times (18 \text{ in})^3}{12} + b \times 18 \text{ in} \left( \frac{18 \text{ in}}{2} - y \right)^2 + A_{s \text{ pos}} \frac{E_s}{E_{c \text{ slab}}} \left[ y - \left( \frac{d_c + \frac{9}{8} \times \text{ in}}{2} \right) \right]^2 \right)}{2}
\]

\[
y_{CG} = 8.358 \text{ in} \quad \text{C.G. of Uncracked Section}
\]

Calculate moment of inertia of uncracked section as

\[
I = b \times \frac{(18 \text{ in})^3}{12} + b \times 18 \text{ in} \left( \frac{18 \text{ in}}{2} - y \right)^2 + A_{s \text{ pos}} \frac{E_s}{E_{c \text{ slab}}} \left[ y - \left( \frac{d_c + \frac{9}{8} \times \text{ in}}{2} \right) \right]^2
\]

Calculate steel stresses for uncracked section as

\[
\sigma = \frac{M_{pos}}{I} \frac{E_s}{E_{c \text{ slab}}} \quad \sigma = \begin{cases} 1.591 \\ 1.33 \\ 1.059 \end{cases} \text{ksi}
\]

Given steel stresses, \( \sigma \) and steel modulus of elasticity, \( E_s \), calculate steel strains, \( \varepsilon \)

\[
\varepsilon = \frac{\sigma \times 10^6}{E_s} \quad \varepsilon = \begin{cases} 67.275 \text{ Positive live load moment} \\ 45.852 \text{ Negative live load moment} \\ 36.51 \text{ Dead load moment} \end{cases} \text{ kip - ft}
\]

3.6 Application of Florida Legal Loads

Florida legal load consists of six known truck configurations with the maximum allowable gross vehicle weight, defined in Table 3.2.
The positive live load moment \((LL_{\text{pos}})\), Negative Live Load Moments \((LL_{\text{neg}})\), and Dead load Moments are tabulated in Table 3.3. The detail calculations of values shown in Table 3.3 are shown in Appendix A. The values in Table 3.3 thru 3.8 have been extracted from Figure 3.8 to 3.16.

**Table 3.3 Moments due to Design Truck Loading, kip-ft**

<table>
<thead>
<tr>
<th>Moments kip-in</th>
<th>SU2</th>
<th>SU3</th>
<th>SU4</th>
<th>C3</th>
<th>C4</th>
<th>C5</th>
<th>HS20</th>
<th>HL-93</th>
</tr>
</thead>
<tbody>
<tr>
<td>(LL_{\text{pos}})</td>
<td>1563.8</td>
<td>2774.9</td>
<td>3019.4</td>
<td>1685.7</td>
<td>2467</td>
<td>1900.8</td>
<td>2830.4</td>
<td>3007.2</td>
</tr>
<tr>
<td>(LL_{\text{neg}})</td>
<td>989.7</td>
<td>1893.3</td>
<td>2050.1</td>
<td>1752.8</td>
<td>2350</td>
<td>2268.8</td>
<td>2262.5</td>
<td>2049.6</td>
</tr>
<tr>
<td>DL</td>
<td>1632</td>
<td>1632</td>
<td>1632</td>
<td>1632</td>
<td>1632</td>
<td>1632</td>
<td>1632</td>
<td>1632</td>
</tr>
</tbody>
</table>

The Tensile forces, \(T_s\) in the reinforcing steel due to service limit state moment are shown in Table 3.4.

**Table 3.4 Tensile Forces \(T_s\), due to Service Limit State Moment, kips**

<table>
<thead>
<tr>
<th>(T_s) kips</th>
<th>SU2</th>
<th>SU3</th>
<th>SU4</th>
<th>C3</th>
<th>C4</th>
<th>C5</th>
<th>HS20</th>
<th>HL-93</th>
</tr>
</thead>
<tbody>
<tr>
<td>(LL_{\text{pos}})</td>
<td>114.7</td>
<td>203</td>
<td>220.9</td>
<td>123.3</td>
<td>180.5</td>
<td>139.1</td>
<td>207.1</td>
<td>220</td>
</tr>
<tr>
<td>(LL_{\text{neg}})</td>
<td>72.4</td>
<td>138.5</td>
<td>150</td>
<td>128.2</td>
<td>171.9</td>
<td>166</td>
<td>165.5</td>
<td>150</td>
</tr>
<tr>
<td>DL</td>
<td>119.4</td>
<td>119.4</td>
<td>119.4</td>
<td>119.4</td>
<td>119.4</td>
<td>119.4</td>
<td>119.4</td>
<td>119.4</td>
</tr>
</tbody>
</table>

Cracked section analysis has resulted in the values of actual steel stress and strain shown in following Tables 3.5 and 3.6.
Table 3.5 Actual Stress, $f_s$ in Reinforcing Steel, ksi

<table>
<thead>
<tr>
<th>$f_s, \text{ksi}$</th>
<th>SU2</th>
<th>SU3</th>
<th>SU4</th>
<th>C3</th>
<th>C4</th>
<th>C5</th>
<th>HS20</th>
<th>HL-93</th>
</tr>
</thead>
<tbody>
<tr>
<td>$LL_{pos}$</td>
<td>4.8</td>
<td>8.5</td>
<td>9.3</td>
<td>5.2</td>
<td>7.6</td>
<td>5.9</td>
<td>8.7</td>
<td>9.3</td>
</tr>
<tr>
<td>$LL_{neg}$</td>
<td>3.0</td>
<td>5.8</td>
<td>6.3</td>
<td>5.4</td>
<td>7.2</td>
<td>7.0</td>
<td>7.0</td>
<td>6.3</td>
</tr>
<tr>
<td>DL</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
<td>5.0</td>
</tr>
</tbody>
</table>

Steel strain of cracked section due to service limit state moment is shown in Table 3.6.

Table 3.6 Steel Strain, $\varepsilon$ of Cracked Section, $\mu \varepsilon$

<table>
<thead>
<tr>
<th>$\varepsilon$</th>
<th>SU2</th>
<th>SU3</th>
<th>SU4</th>
<th>C3</th>
<th>C4</th>
<th>C5</th>
<th>HS20</th>
<th>HL-93</th>
</tr>
</thead>
<tbody>
<tr>
<td>$LL_{pos}$</td>
<td>166.60</td>
<td>249.79</td>
<td>320</td>
<td>179.08</td>
<td>262.08</td>
<td>201.93</td>
<td>300.68</td>
<td>319.47</td>
</tr>
<tr>
<td>$LL_{neg}$</td>
<td>105.14</td>
<td>201.13</td>
<td>217.79</td>
<td>186.20</td>
<td>249.65</td>
<td>241.02</td>
<td>240.35</td>
<td>217.7</td>
</tr>
</tbody>
</table>

The stress and strain due to service limit state moments of uncracked section are tabulated in Tables 3.7 and 3.8 for all eight configurations of design trucks.

Table 3.7 Stress, $\sigma$ of Uncracked Section, ksi

<table>
<thead>
<tr>
<th>$\sigma$</th>
<th>SU2</th>
<th>SU3</th>
<th>SU4</th>
<th>C3</th>
<th>C4</th>
<th>C5</th>
<th>HS20</th>
<th>HL-93</th>
</tr>
</thead>
<tbody>
<tr>
<td>$LL_{pos}$</td>
<td>1.017</td>
<td>1.8</td>
<td>1.959</td>
<td>1.094</td>
<td>1.601</td>
<td>1.233</td>
<td>1.536</td>
<td>1.951</td>
</tr>
<tr>
<td>$LL_{neg}$</td>
<td>0.642</td>
<td>1.228</td>
<td>1.33</td>
<td>1.037</td>
<td>1.525</td>
<td>1.472</td>
<td>1.468</td>
<td>1.33</td>
</tr>
<tr>
<td>DL</td>
<td>1.059</td>
<td>1.059</td>
<td>1.059</td>
<td>1.059</td>
<td>1.059</td>
<td>1.059</td>
<td>1.059</td>
<td>1.059</td>
</tr>
</tbody>
</table>
Table 3.8 Strain, $\varepsilon$ of Uncracked Section, $\mu \varepsilon$

<table>
<thead>
<tr>
<th></th>
<th>SU2</th>
<th>SU3</th>
<th>SU4</th>
<th>C3</th>
<th>C4</th>
<th>C5</th>
<th>HS20</th>
<th>HL-93</th>
</tr>
</thead>
<tbody>
<tr>
<td>LL$_{pos}$</td>
<td>35.085</td>
<td>62.078</td>
<td>67.548</td>
<td>37.711</td>
<td>55.19</td>
<td>42.523</td>
<td>63.32</td>
<td>67.275</td>
</tr>
<tr>
<td>LL$_{neg}$</td>
<td>22.141</td>
<td>42.356</td>
<td>45.863</td>
<td>39.212</td>
<td>52.573</td>
<td>50.756</td>
<td>50.615</td>
<td>45.852</td>
</tr>
<tr>
<td>DL</td>
<td>36.51</td>
<td>36.51</td>
<td>36.51</td>
<td>36.51</td>
<td>36.51</td>
<td>36.51</td>
<td>36.51</td>
<td>36.51</td>
</tr>
</tbody>
</table>

The values of positive and negative moments shown in captions of the following Figures 3.9 through 3.16 are taken from the preceding Table 3.1. These values are the results of static load analysis by Florida Department of transportation, FDOT MathCAD software program (Mathsoft 2002).

Figure 3.8 Graph of (+) 0.009 and (-) 0.023 inches of Deflections due to HS-20 Design Truck Loading
Figure 3.9 Graph of (+) 631.8 and (-) 882.3 Moments due to 0.64-kip/in Uniform Lane Load (Moment Values are Shown in Table 3.3)

The Code values will be evaluated through static and dynamic testing of the bridge and finite element modeling. The details are given next.

Figure 3.10 Graph of (+) 1568.3 and (-) 989.7 Moments due to SU2 Truck Loading (Moment Values are Shown in Table 3.3)
Figure 3.11 Graph of (+) 2774.9 and (-) 1893.3 Moments due to SU3 Truck Loading (Moment Values are Shown in Table 3.3)

Figure 3.12 Graph of (+) 3019.4 and (-) 2050.1 Moments due to SU4 Truck Loading (Moment Values are Shown in Table 3.3)
Figure 3.13 Graph of (+) 1685.7 and (-) 1752.8 Moments due to C3 Truck Loading (Moment Values are Shown in Table 3.3)

Figure 3.14 Graph of (+) 2467 and (-) 2350 Moments due to C4 Truck Loading (Moment Values are Shown in Table 3.3)
Figure 3.15 Graph of (+) 1900.8 and (-) 2268.8 Moments due to C5 Truck Loading (Moment Values are Shown in Table 3.3)

Figure 3.16 Graph of (+) 2830.4 and (-) 2262.5 Moments due to HS-20 Truck Loading (Moment Values are Shown in Table 3.3)
3.7 Static and Dynamic Load Testing of Bridge

To evaluate the previous design calculations, static and dynamic load tests were performed on the East Bay Road Bridge using single unit four axles, SU4 trucks as shown in Figure 3.17. SU4 trucks are the most effective due to their short configuration and heavy weight (70 kips). Static tests were performed for six different loading conditions. In all six load cases, the third axle of a 70kip SU4 truck was positioned in the middle of the span. Strains were measured using the installed FP sensors for the six different load cases, respectfully. The strain contour lines of the 16 FP sensors are shown in Figures 3.18 through 3.23 respectively. The results of these tests will be compared in the next section with a detailed finite element model. Dynamic tests of the bridge under moving trucks with different speeds were also performed to confirm the sensors accuracy under dynamic loading.
Figure 3.17 Bridge Load Test with SU4 Trucks

In figures 3.18 through 3.24, the scale at the bottom edge of contour graph is the distance along the bridge length in feet and the scale at the right side edge of contour graph is the distance along the bridge width in feet.
Figure 3.18 Experimental Strain Contour Lines, Load Case 1, Truck Positioned at Mid Span 1. Units in με
Figure 3.19 Experimental Strain Contour Lines, Load Case 2, Truck Positioned at Mid Span 2. Units in $\mu E$
Figure 3.20 Experimental Strain Contour Lines, Load Case 3, Trucks Positioned at Mid Span 1 and Mid Span 2. Units in \( \mu \varepsilon \)
Figure 3.21 Experimental Strain Contour Lines, Load Case 4, Trucks Positioned at Mid Span 2, Both Trucks are in North Direction. Units in $\mu$ε
Figure 3.22 Experimental Strain Contour Lines, Load Case 5, Trucks Positioned at Mid Span 2, Northbound and Southbound. Units in $\mu$ε.
Figure 3.23 Experimental Strain Contour Lines, Load Case 6, Trucks Positioned at Mid Span 1, Northbound and Southbound. Units in $\mu$ε.

Figure 3.24 illustrates the dynamic response of the bridge subject to two SU4 truck in tandem, each weighing 70 kips GVW. The truck traversed over the bridge at the speed of 10 mph. The first truck goes on the bridge and the DSM sensor take the reading at 17 $\mu$ε. At two seconds later, the sensor reads 15 $\mu$ε,
due to the effect of second truck on the bridge. More detailed description of the
dynamic behavior of trucks is discussed in Chapter 4.

![Figure 3.24 Dynamic Strain](image)

Tables 3.9 through 3.14 contain the strain values of different sets of
sensors subject to SU4 truck load at different constant speed. The evolution of
strain values indicate that the change in strain reading of one sensor subject to a
single SU4 truck is not significantly smaller than the reading of the sensor subject
to SU4 truck load in tandem at the same speed. In Table 3.9, CSM sensor is
surface mounted to the bottom surface of slab at the center of span 1. These
tables were generated during data collection process in chapter 4 and shown
here for analytical discussions.
Table 3.9 Dynamic Response of One Surface Mount Sensor

<table>
<thead>
<tr>
<th>Speed MPH</th>
<th>Load SU4</th>
<th>One channel was on, Readings are in $\mu e$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>CSM</td>
</tr>
<tr>
<td>10</td>
<td>Single</td>
<td>21</td>
</tr>
<tr>
<td>10</td>
<td>Tandem</td>
<td>16.5, 18.5</td>
</tr>
<tr>
<td>20</td>
<td>Single</td>
<td>13.5</td>
</tr>
<tr>
<td>20</td>
<td>Tandem</td>
<td>12, 13.5</td>
</tr>
<tr>
<td>30</td>
<td>Single</td>
<td>13</td>
</tr>
<tr>
<td>30</td>
<td>Tandem</td>
<td>13.5, 15.5</td>
</tr>
<tr>
<td>35</td>
<td>Single</td>
<td>13</td>
</tr>
<tr>
<td>35</td>
<td>Tandem</td>
<td>12, 13.5</td>
</tr>
<tr>
<td>40</td>
<td>Single</td>
<td>13</td>
</tr>
</tbody>
</table>

In Table 3.10, sensor F is bonded to bottom mat reinforcing steel.

Table 3.10 Dynamic Response of One Embedded Sensor

<table>
<thead>
<tr>
<th>Speed MPH</th>
<th>Load SU4</th>
<th>One channel on, Readings are in $\mu e$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>F</td>
</tr>
<tr>
<td>10</td>
<td>Single</td>
<td>19</td>
</tr>
<tr>
<td>10</td>
<td>Tandem</td>
<td>15, 16</td>
</tr>
<tr>
<td>20</td>
<td>Single</td>
<td>17</td>
</tr>
<tr>
<td>20</td>
<td>Tandem</td>
<td>12.5, 14</td>
</tr>
<tr>
<td>30</td>
<td>Single</td>
<td>14</td>
</tr>
<tr>
<td>30</td>
<td>Tandem</td>
<td>17, 18.5</td>
</tr>
<tr>
<td>35</td>
<td>Single</td>
<td>16</td>
</tr>
<tr>
<td>35</td>
<td>Tandem</td>
<td>14, 15</td>
</tr>
<tr>
<td>40</td>
<td>Single</td>
<td>12</td>
</tr>
</tbody>
</table>
Table 3.11 Dynamic Response of Two Sensor Channels were on, Readings are in $\mu$e

<table>
<thead>
<tr>
<th>Speed MPH</th>
<th>Load SU4</th>
<th>CSM F</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 Single</td>
<td>17.5</td>
<td>11</td>
<td></td>
</tr>
<tr>
<td>10 Tandem</td>
<td>4.5, 14</td>
<td>13.5, 15</td>
<td>Anomalies, CSM reading (4.5)</td>
</tr>
<tr>
<td>20 Single</td>
<td>12</td>
<td>11.5</td>
<td></td>
</tr>
<tr>
<td>20 Tandem</td>
<td>11.5, 12</td>
<td>9, 13</td>
<td></td>
</tr>
<tr>
<td>30 Single</td>
<td>13</td>
<td>15</td>
<td></td>
</tr>
<tr>
<td>30 Tandem</td>
<td>7, 11</td>
<td>8, 14</td>
<td></td>
</tr>
<tr>
<td>35 Single</td>
<td>11</td>
<td>12</td>
<td></td>
</tr>
<tr>
<td>35 Tandem</td>
<td>6.75, 9</td>
<td>7, 10</td>
<td></td>
</tr>
<tr>
<td>40 Single</td>
<td>8.5</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>40 Tandem</td>
<td>--------</td>
<td>--------</td>
<td>unsafe to maintain distance</td>
</tr>
<tr>
<td>45 Single</td>
<td>7.5</td>
<td>6.5</td>
<td></td>
</tr>
<tr>
<td>45 Tandem</td>
<td>--------</td>
<td>--------</td>
<td>unsafe to maintain distance</td>
</tr>
</tbody>
</table>

Table 3.12 Dynamic Response of Four Sensors

<table>
<thead>
<tr>
<th>Speed MPH</th>
<th>Load SU4</th>
<th>4 Channels were on, Readings are in $\mu$e</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>H CSM P2 F</td>
</tr>
<tr>
<td>10 Single</td>
<td>19</td>
<td>13</td>
</tr>
<tr>
<td>10 Tandem</td>
<td>18.5, 14.5</td>
<td>11.5, 6.5</td>
</tr>
<tr>
<td>20 Single</td>
<td>17</td>
<td>10</td>
</tr>
<tr>
<td>20 Tandem</td>
<td>15, 14.5</td>
<td>10/805</td>
</tr>
<tr>
<td>30 Single</td>
<td>7</td>
<td>9</td>
</tr>
<tr>
<td>30 Tandem</td>
<td>11.5, 11.5</td>
<td>4.5, 5.5</td>
</tr>
<tr>
<td>35 Single</td>
<td>10</td>
<td>8</td>
</tr>
<tr>
<td>35 Tandem</td>
<td>7, 10.5</td>
<td>4.5, 5</td>
</tr>
</tbody>
</table>

In Table 3.12, sensor “H” is bonded to the top mat reinforcing steel.
Sensor “P2” is surface mount to the deck topside, ¾ “ below the concrete surface.
Table 3.13 Dynamic Response of Eight Sensors, Two SU4 Trucks in Southbound Direction

<table>
<thead>
<tr>
<th>Speed MPH</th>
<th>Load SU4</th>
<th>8 Channel were on, Readings are in $\mu$E (Southbound direction)</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>H</td>
<td>I</td>
<td>DSM</td>
<td>CSM</td>
<td>P1</td>
<td>P2</td>
</tr>
<tr>
<td>10</td>
<td>Single</td>
<td>14</td>
<td>11</td>
<td>10</td>
<td>12</td>
<td>6.5</td>
<td>7</td>
</tr>
<tr>
<td>10</td>
<td>Tandem</td>
<td>12, 6.5</td>
<td>4.5, 2.5</td>
<td>5.5, 7.5</td>
<td>8.5, 10.5</td>
<td>4.5, 6</td>
<td>2.5, 4.5</td>
</tr>
<tr>
<td>20</td>
<td>Single</td>
<td>10</td>
<td>5.5</td>
<td>4</td>
<td>7</td>
<td>4</td>
<td>4.5</td>
</tr>
<tr>
<td>20</td>
<td>Tandem</td>
<td>6.5, 7.5</td>
<td>2.5, 2</td>
<td>3, 0.5</td>
<td>3.5, 6.5</td>
<td>2, 3.5</td>
<td>2.5, 3</td>
</tr>
<tr>
<td>30</td>
<td>Single</td>
<td>8.5</td>
<td>3</td>
<td>2.5</td>
<td>4</td>
<td>3</td>
<td>2.5</td>
</tr>
<tr>
<td>30</td>
<td>Tandem</td>
<td>5, 6.5</td>
<td>1.5, 0.5</td>
<td>2, 1</td>
<td>3.5, 5</td>
<td>3, 2.5</td>
<td>2, 2.5</td>
</tr>
<tr>
<td>35</td>
<td>Single</td>
<td>6</td>
<td>1.5</td>
<td>0.5</td>
<td>1</td>
<td>4</td>
<td>2</td>
</tr>
<tr>
<td>35</td>
<td>Tandem</td>
<td>8, 4.5</td>
<td>0.5, 0.5</td>
<td>0.5, 2.5</td>
<td>-0.5, 8.5</td>
<td>4.5, 2.5</td>
<td>1.5</td>
</tr>
</tbody>
</table>
Table 3.14 Dynamic Response of Eight Sensors, Two SU4 Trucks in Both Directions

<table>
<thead>
<tr>
<th>Speed MPH</th>
<th>Load SU4</th>
<th>8 Channel were on, Readings are in $\mu$S</th>
<th>Traffic was in both directions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>H</td>
<td>I</td>
</tr>
<tr>
<td>10</td>
<td>Single</td>
<td>8</td>
<td>6</td>
</tr>
<tr>
<td>10</td>
<td>Tandem</td>
<td>14, 10</td>
<td>14, 6.5</td>
</tr>
<tr>
<td>20</td>
<td>Single</td>
<td>11.5</td>
<td>8.5</td>
</tr>
<tr>
<td>20</td>
<td>Tandem</td>
<td>3.5, 11</td>
<td>5.5, 10.5</td>
</tr>
<tr>
<td>30</td>
<td>Single</td>
<td>10</td>
<td>9</td>
</tr>
<tr>
<td>30</td>
<td>Tandem</td>
<td>4, 9</td>
<td>9.5, 11</td>
</tr>
<tr>
<td>35</td>
<td>Single</td>
<td>11</td>
<td>8.5</td>
</tr>
<tr>
<td>35</td>
<td>Tandem</td>
<td>3.5, 7</td>
<td>6.5, 11.5</td>
</tr>
<tr>
<td>40</td>
<td>Single</td>
<td>6</td>
<td>8</td>
</tr>
</tbody>
</table>
3.8 Finite Element Modeling

A finite element model for the bridge was developed using the commercial software SAP2000 (Computers and Structures, Inc., 2004). The bridge deck was modeled using 4-node shell elements. The nodes along the bent lines were assumed fixed in the vertical direction only. The nodes along the central bent #3 were assumed fixed for both displacements and rotations due to the presence of hook anchorages extending from the bent. Only half of the deck was modeled, as the presence of the fixed supports along the central bent prevents any forces to be transferred from one side of the deck to the other. The model was used to study the behavior of the bridge under the same loading condition of the static test described in the previous section. In this case eight point loads were used to represent the wheel loads. The analytical strain contour lines for load case two are shown in Figure 3.25 is compared to the experimental contours of Figure 3.19. The maximum strain value obtained under the wheels is $35 \mu \varepsilon$. The corresponding recorded experimental value was $32 \mu \varepsilon$. From these results and Figures 3.18 and 3.23, it can be concluded that the FP sensors are capable of providing a high degree of accuracy in sensing the response of the bridge under the truck loads.

Figure 3.19 below, is repeated as Figure 3.26 for a closer viewing and for comparison with Figure 3.25.
FIGURE 3.25 Analytical Strain Contour Lines for SU4 Truck on Span 2 Units in $\mu\varepsilon$

Figure 3.26 Experimental Strain Contour Lines, Load Case 2, $\mu\varepsilon$
In Tables 3.4 and 3.6, the values of strain for cracked and uncracked section due to SU4 are listed as 320 and 67.548 micro strains, \( \mu \varepsilon \), respectively. It would be difficult not to notice that the value of strain, 320 \( \mu \varepsilon \), for cracked section, in case of SU4 truck loading, is excessively high and conservative. This comparison simply invalidates the assumption of “cracked section” in designing the bridge. On the other and, the assumption of “Uncracked section” which yields in strain value of 67.548 \( \mu \varepsilon \), is somewhat more in agreement with the results of field truck load test. These strain values, as shown in Figure 3.18 thru 3.23 are in range of 21 to 24.5 \( \mu \varepsilon \).

Even though these values are for statistically determinant structures, the restraining effects of indeterminate structure will only slightly reduce it.

The following section describes the beam model representing the bridge deck subject to SU4 truck static load for three different load cases as shown in section 3.9, Figures 3.9, 3.10 and 3.11.

3.9 Beam Model Analysis Subject to Static Load

The finite element modeling is used to evaluate single beam models. The program MASTAN was used to conduct the beam analysis. Six different load positions (load cases) were considered in the analysis. Section 2.14, Figure 2.87 depicts the layout plan of action to perform these six different load cases. The process of performing this task was evaluated in detail prior to commencement and is presented in Section 2.15, Figures 2.88 through 2.95.
Figures 3.26, thru 3.28 show the moment diagrams of the beam model for three load cases.

In load case 1, a single SU4 truck was placed on center of span 1. The corresponding maximum positive moment at Span 1 was +2783 kip in and the corresponding maximum negative moment at Bent 2 was -1500 kip in, (Figure 3.26). The corresponding strain values for case 1 loading is 61 με in span 1 in tension (maximum positive moment was 2783 kip in) and 35 με over Bent 2 in compression (maximum negative moment was 1500 kip in).

![Figure 3.27 Moment Diagram for Beam Model for Case 1 Static Load Test](image)

In load case 2, a single SU4 truck was placed on center of span 2. The corresponding maximum positive moment at Span 2 was +2338 kip-in and the corresponding maximum negative moment at Bent 3 was -3796 kip-in (Figure 3.27),
corresponding strain values for case 2 loading is $56 \, \mu e$ in span 2 in tension (maximum positive moment was 2338 kip-in) and $90 \, \mu e$ over Bent 3 in compression (maximum negative moment was 3796 kip-in).

Figure 3.28 Moment Diagram for Beam Model for Case 2 Static Load Test

In load case 3, two SU4 trucks were placed in tandem on center of span 1 and span 2. The corresponding maximum positive moment at Span 1 was $+2136$ kip-in and maximum negative moment at Bent 2 was $-2973$ kip-in. The corresponding maximum positive moment at Span 2 was $+1918$ kip-in and maximum negative moment at Bent 3 was $-2956$ kip-in (Figure 3.28).

The corresponding strain values for case 3 loading is $51 \, \mu e$ in span 1 in tension (maximum positive moment was 2136 kip in) and $71 \, \mu e$ over Bent 2 in compression (maximum negative moment was 2973 kip-in).
Figure 3.29 Moment Diagram for Beam Model for Case 3 Static Load Test

Strains at the cracking condition due to the positive and negative moments (+3019.4, -2050.1 kip-in) are +320 and -217.79 με respectively. Strains at the uncracked section are +67.54 and -45.8 με respectively. In comparison, the strain value of finite element model is 35 με and strain value due to the static load test (SU4 truck) was 24.5 με. This strain value (24.5 με) is less than half strain value of 61 με calculated for the beam model for case 1 loading on span 1 over sensors CSM and DSM.

The program that was used to calculate the moments for beam model does not calculate the strain. The following steps are used to determine the strain values of beam model for three different load cases.

Given:

Area of concrete section is, $A_c = h \times d$
Where:

\[ h = \text{concrete deck thickness} \]
\[ d = \text{distribution width, 11.87 feet} \]

Therefore

\[ A = 18'' \times (11.87 \times 12) = 2564^2 \]

The moment of inertia can be calculated as:

\[ I = \frac{1}{12} bh^3 \]
\[ I = \frac{1}{12} \times (11.87 \times 12)(18)^3 = 69,225.84 \text{in}^4 \]

Stress is:

\[ \sigma = \frac{Mc}{I} \quad \text{the moment for load case 1 is 2783 kip-in} \]

Substitute for \( M, c \) and \( I \) to get the strain

\[ \sigma = \frac{2783 \times 7}{69,225.84} = 0.259 \text{ksi} \]

And strain:

\[ \varepsilon = \frac{\sigma}{E_c} \]

where

\[ E_c \text{ is modulus of elasticity of concrete} \]

\[ E_c = 57000 \sqrt{5500 \text{ psi}} = 4,227,233.13 \text{ psi} \approx 4227 \text{ksi} \]

\[ \varepsilon = \frac{0.259}{4227} = 0.000061 \text{ or } 61 \mu \varepsilon \]

This value of strain is relatively high indicating that the distribution width, \( E_D \) is possibly too small. This assumption can be investigated and verified by recalculating the strain values with a larger distribution width, \( E_D \).
CHAPTER 4. DATA COLLECTION

This chapter describes the methodology for collecting data from the bridge under service load. This data was used to check the East Bay Road Bridge designed by LRFD code, and compare the values of strains obtained by using The Florida Department of Transportation software program with the experimental values of strains obtained from static load test of bridge with SU4 truck. Details of this evaluation are described in Chapter 3.

The static load was performed under a controlled weight and speed condition. The bridge was closed to traffic while the locations of sensors were marked on the deck top surface (Figures 4.1 and 4.2). With no truck on the bridge, the sensors readings were recorded. These readings are the baseline or the zero readings of the sensors (Table 4.1). A SU4 truck with gross weight of 67,360 pounds was placed on the marks on the bridge. The tires of the middle rear axle were placed directly over the sensors, (Figure 4.3).

Four sensors designated as C, D, E, and F were bonded to the primary reinforcing steel of the bottom mat in span 2 for monitoring strain due to the positive moment. Two sensors designated as P1 and P2 were placed on the deck $\frac{3}{4}$" below the surface over bent 1. Four sensors designated as G, H, I and J were bonded to the primary reinforcing steel top mat over bent 3 (Refer to Chapter 2, experimental Section 2.10.3, Figures 2.31, 2.32 and 2.33).
FISO Commander standard software, version 2 developed by FISO Technologies Inc was used for data collection. A laptop computer was directly connected to DMI-16, data logger through RS-232 communication cable on site (Figure 4.4). Table 4.1 represents the strain values of static load test. Sensors G, J, C and D are located at the exterior lane (8 feet shoulder, the emergency lane). Sensors G and J are bonded to the top of the reinforcing mat over bent 3 and sensors C and D are bonded to the bottom reinforcing mat at the mid-span 2. The alphabetically out of order position of sensor “J” was due to the shorter length of the fiber optic cable. However, the ascending numbers of channels were assigned to the sensors in alphabetical order (e.g., G = Channel 1, H = 2 I = 3 and J = 4). The strain readings of sensors G, J, C and D are due to the negative and positive moments. The higher values of strain readings for sensors H, I, E and F are relevant to the positions of the sensors located in the travel lane (Chapter 2, Section 2.10.3, Figure 2.31 through 2.33). Sensors P1 and P2 are embedded in the concrete ¾” below the surface over bent 2 to capture strain due to the negative moment.

Tables 4.2 and 4.3 represent remotely collected data from the static load. The connection between the desktop computer and DMI took place through the modem. The bridge was closed to traffic and the office was notified to record the zero readings. The rear middle axle of the truck was positioned on the bridge deck over the sensors designated as E and F, for the flexural condition. All channels were turned on and signals were transmitted through the modem to the computer.
Table 4.1 On Site Data Collection with a Laptop Computer from Static Load Test with SU4 Truck, GVW= 67,360 lbs

<table>
<thead>
<tr>
<th>Channel</th>
<th>Sensor</th>
<th>Zero Readings (με) No traffic on bridge</th>
<th>Truck on the bridge (με)</th>
<th>Resultant readings (με)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>G</td>
<td>-0.6</td>
<td>3.8</td>
<td>4.4</td>
</tr>
<tr>
<td>2</td>
<td>H</td>
<td>0.4</td>
<td>9.4</td>
<td>9.0</td>
</tr>
<tr>
<td>3</td>
<td>I</td>
<td>1.2</td>
<td>9.4</td>
<td>8.2</td>
</tr>
<tr>
<td>4</td>
<td>J</td>
<td>0.6</td>
<td>8.8</td>
<td>8.2</td>
</tr>
<tr>
<td>9</td>
<td>P1</td>
<td>2.2</td>
<td>8.4</td>
<td>6.2</td>
</tr>
<tr>
<td>10</td>
<td>P2</td>
<td>2.4</td>
<td>6.6</td>
<td>4.2</td>
</tr>
<tr>
<td>11</td>
<td>T1</td>
<td>34.60°C</td>
<td>34.60°C</td>
<td>34.60°C</td>
</tr>
<tr>
<td>12</td>
<td>T2</td>
<td>24.78°C</td>
<td>24.78°C</td>
<td>24.78°C</td>
</tr>
<tr>
<td>13</td>
<td>C</td>
<td>2.0</td>
<td>8.2</td>
<td>6.2</td>
</tr>
<tr>
<td>14</td>
<td>D</td>
<td>2.6</td>
<td>13.2</td>
<td>10.6</td>
</tr>
<tr>
<td>15</td>
<td>E</td>
<td>3.0</td>
<td>28.4</td>
<td>25.4</td>
</tr>
<tr>
<td>16</td>
<td>F</td>
<td>3.0</td>
<td>28.2</td>
<td>25.2</td>
</tr>
</tbody>
</table>
Table 4.2  Remotely Collected Data with a Desktop Computer from Static Load Test with SU4 Truck, GVW= 67,360 lbs

<table>
<thead>
<tr>
<th>Channel</th>
<th>Sensor</th>
<th>Zero Readings (με) No traffic on bridge</th>
<th>Truck on the bridge (με)</th>
<th>Resultant readings (με)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>G</td>
<td>-0.2</td>
<td>3.6</td>
<td>3.8</td>
</tr>
<tr>
<td>2</td>
<td>H</td>
<td>-0.2</td>
<td>9.4</td>
<td>9.6</td>
</tr>
<tr>
<td>3</td>
<td>I</td>
<td>0.0</td>
<td>8.4</td>
<td>8.4</td>
</tr>
<tr>
<td>4</td>
<td>J</td>
<td>0.2</td>
<td>7.8</td>
<td>7.6</td>
</tr>
<tr>
<td>9</td>
<td>P1</td>
<td>-2.2</td>
<td>9.0</td>
<td>11.2</td>
</tr>
<tr>
<td>10</td>
<td>P2</td>
<td>-1.8</td>
<td>10.0</td>
<td>11.8</td>
</tr>
<tr>
<td>11</td>
<td>T1</td>
<td>34.60°C</td>
<td>34.60°C</td>
<td>34.60°C</td>
</tr>
<tr>
<td>12</td>
<td>T2</td>
<td>24.78°C</td>
<td>24.78°C</td>
<td>24.78°C</td>
</tr>
<tr>
<td>13</td>
<td>C</td>
<td>1.2</td>
<td>8.0</td>
<td>7.8</td>
</tr>
<tr>
<td>14</td>
<td>D</td>
<td>1.8</td>
<td>13.0</td>
<td>11.2</td>
</tr>
<tr>
<td>15</td>
<td>E</td>
<td>-2.4</td>
<td>23.6</td>
<td>25.6</td>
</tr>
<tr>
<td>16</td>
<td>F</td>
<td>-2.4</td>
<td>25.4</td>
<td>27.8</td>
</tr>
</tbody>
</table>
Table 4.3 Verification of Remotely Collected Data with a Desktop Computer from Static Load Test with SU4 Truck, GVW= 67,360 lbs

<table>
<thead>
<tr>
<th>Channel</th>
<th>Sensor</th>
<th>Zero Readings ((\mu\varepsilon)) No traffic on bridge</th>
<th>Truck on the bridge ((\mu\varepsilon))</th>
<th>Resultant readings ((\mu\varepsilon))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>G</td>
<td>0.0</td>
<td>3.6</td>
<td>3.6</td>
</tr>
<tr>
<td>2</td>
<td>H</td>
<td>-0.5</td>
<td>9.4</td>
<td>9.9</td>
</tr>
<tr>
<td>3</td>
<td>I</td>
<td>1.0</td>
<td>8.4</td>
<td>7.4</td>
</tr>
<tr>
<td>4</td>
<td>J</td>
<td>1.0</td>
<td>7.8</td>
<td>6.8</td>
</tr>
<tr>
<td>9</td>
<td>P1</td>
<td>-0.5</td>
<td>9.2</td>
<td>9.7</td>
</tr>
<tr>
<td>10</td>
<td>P2</td>
<td>0.5</td>
<td>10.0</td>
<td>9.5</td>
</tr>
<tr>
<td>11</td>
<td>T1</td>
<td>34.60°C</td>
<td>34.60°C</td>
<td>34.60°C</td>
</tr>
<tr>
<td>12</td>
<td>T2</td>
<td>24.78°C</td>
<td>24.78°C</td>
<td>24.78°C</td>
</tr>
<tr>
<td>13</td>
<td>C</td>
<td>-0.5</td>
<td>7.8</td>
<td>8.3</td>
</tr>
<tr>
<td>14</td>
<td>D</td>
<td>0.5</td>
<td>13.2</td>
<td>12.7</td>
</tr>
<tr>
<td>15</td>
<td>E</td>
<td>0.0</td>
<td>23.6</td>
<td>23.6</td>
</tr>
<tr>
<td>16</td>
<td>F</td>
<td>0.0</td>
<td>25.4</td>
<td>25.4</td>
</tr>
</tbody>
</table>
Figure 4.1 Locating the Sensors on Deck Topside

Figure 4.2 Marking the Locations of the Sensors on Deck Topside

Figure 4.3 SU4 Truck was Placed with the Tires of the Middle Rear Axle Placed on the Mark Over the Sensors on Span 2

4.1 Remote Monitoring of the East Bay Road Bridge

The data logger (DMI) located 35 miles from the office was remotely connected to a desktop computer via modem. DMI was set to the “direct data acquisition” mode (Figure 4.10). This mode utilizes the graph of strain as a raw
form during the scanning. This data was saved on the computer’s hard drive for later analysis. Screen shots of the dynamic strain profiles were taken in the office are shown (Figures 4.5 and 4.6). The desired channels were turned on. Speeds of the trucks or cars going over the bridge were unknown. The large spikes indicate the passage of large trucks traveling over the bridge. Small spikes in the graph indicate the passage of cars over the bridge. The traffic data are currently being continuously collected and analyzed for the purpose of investigating the long term bridge behavior under traffic loading. These data were used to compare maximum recorded stresses to LRFD design values, and detecting possible future deficiencies through a long term remote monitoring of the bridge. Discussion on this subject will be presented in Chapter 5. Figure 4.4 shows field truck load test and on site live data collection.
Figure 4.5 Graph of Heavy Trucks on the Bridge

FIGURE 4.6 Graph of Cars and Light Trucks on the Bridge
4.2 Running the Software, FISO Commander

4.2.1 Detect Comport for Laptop Communication Port

Figures 4.7 through 4.12 show the necessary steps in using the software to connect to the DMI for data collection. The first step for using FISO commander Standard Edition v1.9.8 is to determine the comport of the laptop computer that will be used for on site direct connection. Click on start menu, click on settings, open control panel and select system’s icon from control panel. Next, click on Hardware then Device Manager. At this point, click on the “+” sign next to “Port (Com & LPT)". The communication port will be indicated in the parenthesis, e. g., (Com1).

As soon as the comport is determined, execute the software by clicking on the application icon in program files. A screen as shown in Figure 4.7 will come up and show the information on the system. To initialize communication with DMI, scroll down and select communication port and wait. After about 3 to 4 seconds, a dialogue box will come up and prompts for telephone number input. Input the telephone number and click OK (Figure 4.8)
Figure 4.7 Initializing DMI

Figure 4.8 Modem Communication Initialization Dialogue Box

Figure 4.9 shows the software malfunction. During an attempt to connect to DMI data conditioner, the software could not detect DMI but found a portable single channel data logger, FTI-10. This error frequently reoccurred. In this
case, quit the program and restart the software until the correct data conditioner is detected.

### 4.2.2 Detect Comport for Desktop Remote Connection

To determine the comport for remote connection at the office located anywhere, the following steps should take place. Click on the start menu, click on settings, click on the control panel, double click on the phone and modem options, click on the modems button and the modem comport will be indicated.

![Init Communication](image)

Figure 4.9 Conditioner Initialization Error (FTI-10)

After communication has been established, a dialogue box as shown in Figure 4.10 will come up. This dialog box has several buttons for various applications. “Configure conditioner” button will allow to change the type of scan
from automatic to manual and set the scan rate. “Configure transducer” Button will allow sensors to be turned on or off. After desired transducers are selected, the “save” button must be clicked before proceeding to the next function (Figure 4.11). Finally, the “Direct acquisition (graph)” allows starting, stopping and saving the data acquisition (Figure 4.12). The “Delay acquisition” function (Figure 4.13), was not used in this experiment. However, this function allows for setting the time and duration of the acquisition for the conditioner similar to a common timer.

![Figure 4.10 Configure Conditioner](image)

**Figure 4.10 Configure Conditioner**
Figure 4.11 Configure Transducer/Sensors Assignment

Figure 4.12 Direct Acquisition with Graph
4.3 Using Different Versions of FISO Commander Software

The data collection from the field experiment began with FISO commander software “v1.9.8”. During the early days of the experiment, data (strains) appeared to be within the expected range. However, remote connection to the laptop was not possible. Soon, the integrity of collected data began to deteriorate. A new version, “v1.9.9” was developed for this experiment and used for a short period of time. The remote connection with laptop was not achieved and poor integrity in collected data was noticeable. A newer version, “v1.9.9 (Build E2) !!! for Ebrahim only !!!” was developed and put to use. The same symptoms as with v1.9.9 were immediately apparent when using this version (Figures 4.14 through 4.17)
FISO Technologies took an approach in developing software compatible with DMI and the laptop computer, as well as the desktop computer. Most of the features of this latest software is comparable with the last version, “v1.9.9 (Build E2)!!! For Ebrahim only!!!”. This version, v2 has resolved the laptop remote connection, and the processed raw data was opened in MS Excel, (stain values ~26 $\mu$e) was close to the strain value of ~28 $\mu$e obtained with direct connection to DMI via RS-232 communication cable.
4.4 Running FISO Commander v2 Software

Program initialization of FISO commander v2 is similar to v1.9.9 and opens from the program files. After the comport is determined using the steps described for v1.9.9, execute the software by clicking on the application icon in the program files. Figure 4.18 will appear and show the information on the system. To initialize communication with DMI, place a check mark in the modem box, then scroll down on the connection box, click on the desired comport and wait. A few seconds later, a dialog box will appear and the prompts for the telephone number input. Input the telephone number and click on the dial button (Figure 4.19). The telephone number is stored in the memory for the subsequent connections.

Every attempt to connect to DMI with this version of FISO commander has been successful. As soon as the connection takes place, a dialog box shown in Figure 4.19 will come up with all buttons being active.

All of the buttons on the dialogue box in Figure 4.20 were working properly in every connection attempt. A factory default is preset in the system configuration display. The preset information did not change during this experiment. (Figure 4.21)
Figure 4.18 Connection Initialization

Figure 4.19 Modem Setup Dialogue Box

Figure 4.20 Application Selection Dialogue Box
Figure 4.21 System Configuration Information

The gage/channel configuration button in Figure 4.20 is the most frequently used option in this experiment. When this button is clicked, Figure 4.22 will come up. In this box, the gage information must be accurately input in the gage list column. Gage name, gage factor and channel number must correspond to each other, otherwise erroneous data will be gathered during the experiment. When the gage list is completed, it will remain unchanged during the entire experiment and likely through the life of the bridge. The purpose of the gage setting is to turn on or off as many gages as are desired for a particular load test condition.

For the first time, all gages (sensors) are set to the “Off” position. To turn on a gage, click on the gage number and scroll up or down to select the corresponding gage name. The gage factor will automatically appear next to the gage name and in the next column the gage zero will show up. The gage zero is
a number that is internally calculated for each gage based on the gage length and gage factor. This number is the baseline for the gage readings when subject to a load condition.

Figure 4.22 Gauge List and Channel Setting

When the desired gages in Figure 4.22 are selected, click on the graphic acquisition button in Figure 4.20 and wait. The dialogue box in Figure 4.23 will come up. In this box, insert the desired duration of data acquisition or check the "infinite" for up to 50,000 bytes of scanning, then either click the on start button to begin scanning or click on the advance configuration button to go to next dialog box. When the on start button is clicked, scanning will begin with pre-determined color of graphs by default. This option may cause color contrast problems by transparent colors that are not clearly visible.
If the color contrast is not acceptable, click on the stop button and then the advanced configuration button to change the selection. A graphic configuration box will appear. At this point, click on the “+” sign next to the “Graphic” option in the “Graphic visible channel” section of the dialogue box seen on Figure 4.24. Select the channel to check the pre-determined color of channel and click OK if the color meets an acceptable contrast. To select another color, click on the color (green in this case). This is shown in the “channel properties” section of the box in Figure 4.24. This action will bring up the basic color chart (Figure 4.25). Select the desired color and click OK. The graphic acquisition dialog box of Figure 4.23 will appear. Click on the start button to begin scanning. A warning dialogue box will appear, with prompts of “Yes” or “No” options, Figure 4.26.
Click on “Yes” button the initiate the scan. This step will take return to the dialogue box in Figure 4.23. Data will be saved in sub-directory “DATA” of the program for analysis use or deletion.

![Figure 4.24 Graphic Configuration Dialogue Box](image)

Click on the “Start” button to initialize the data acquisition application program. If “No” button is clicked, the program will be aborted. It is
recommended to avoid aborting the program since restarting the program is time consuming and there is a possibility of not restarting.

“Memory Acquisition” application of the v2 of the software (Figure 4.27) acquires data similar to v1.9.9. However, version v2 has a more appealing graphic presentation than version v1.9.9 (Figure 4.23). In this application, scanning can be programmed for long term monitoring at various time intervals similar to v1.9.9 (All version preceding v2 are now considered obsolete).

Figure 4.27 Memory/Delay Acquisition

“File Acquisition” application of v2 is similar to the “Direct Acquisition (File)” application of v1.9.9. This application is in the process for further development (Figure 4.28).
A desktop computer at the office was remotely connected to the DMI system. In case of static load test, all channels can be turned on to see the true load distribution sensed by the gauges placed in critical locations (Tables 4.1 through 4.3). However, in dynamic test, when the truck goes over a sensor and reaches the next sensor, DMI is still scanning the first channel and the reading of the other channels are different than those recorded from static load with the same truck load. These differences also vary for different vehicle speeds of travel. The manufacturer does not recommend DMI-16 data conditioner more than eight channels for dynamic load test.

The following tables and graph indicate that when four channels are turned on, a significant drop in the two last channels are observed. Questions may arise concerning the other data conditioners available capability of dynamic scanning of more than two channels for fast moving vehicles. The answer is yes
such a system is available (BUS Commander by FISO) however; the price was beyond the research budget for this study.

The following figures are the depiction of real-time truck activities over the bridge. The data acquisition is completely uncontrolled, that is, the position, number of trucks and their speeds are unknown. All the data was remotely acquired via telephone line through the modem. The “on” and “off” modes of the sensors were controlled remotely by a desktop and a laptop computer. The caption for each figure indicates which sensor was on while the off sensors were not of any concern. Remotely acquired data by DMI was stored in a directory on the local hard-drive for later analysis. Figure 4.29 shows the truck load and temperature response by four sensors, H, I, T1, and T2. The sensors “H” and “I” are those bonded to the top reinforcement for negative moment. Figure 4.30 also shows the truck load and temperature response by four other sensors, E, I, T1, and T2. The sensor “E” is for positive movement and “I” is for negative moment, respectively.

Unlike version v1.9.9, the v2 version of this software does not display the strains values as they are detected by the sensors. In version v2 of the software, the raw data has to be decoded (processed) in Microsoft Excel through comma-delimited feature of the program. When processing data in MS Excel, all channels have to be set at zero otherwise the graph will show different points of origins for the channels. On the other hand, v1.9.9 had an advantage for directly displaying the strain values on the graph as shown in previous Figures 4.14 through 4.17.
Figure 4.29 Channels H (Green), I (Blue), T1 (Red) & T2 (Yellow); Strain and Temperature Graphs. (Speed is Unknown)

Figure 4.30 Graph of Strain Values for Channels E (Yellow) and I (Blue). Where T1 (Red) & T2 (Pink) are Graphs of Temperature for Top and Bottom of Slab
Figure 4.31 Graph of Response from P1 and P2 Sensors

Figure 4.32 Response of E and P2 Sensors
The following tables illustrate the collected data under a controlled condition for the load and the speed of vehicles. Two SU4 trucks were employed for this on site dynamic load test. The purpose of this test was to determine the maximum strain values at the critical location of the bridge and investigate the effect of moving vehicle on other parts of the bridge (load distribution). Also, the effect of the same load with different speeds was investigated. For location of sensors, refer to Chapter 2, section 2.10, and Figures 2.29 through 2.33.
### Table 4.4 Strain Values of Sensor “CSM” for Different Speeds

<table>
<thead>
<tr>
<th>Speed MPH</th>
<th>Load SU4</th>
<th>One channel was on, Readings are in με</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>CSM</td>
</tr>
<tr>
<td>10</td>
<td>Single</td>
<td>21</td>
</tr>
<tr>
<td>10</td>
<td>Tandem</td>
<td>16.5, 18.5</td>
</tr>
<tr>
<td>20</td>
<td>Single</td>
<td>13.5</td>
</tr>
<tr>
<td>20</td>
<td>Tandem</td>
<td>12, 13.5</td>
</tr>
<tr>
<td>30</td>
<td>Single</td>
<td>13</td>
</tr>
<tr>
<td>30</td>
<td>Tandem</td>
<td>13.5, 15.5</td>
</tr>
<tr>
<td>35</td>
<td>Single</td>
<td>11</td>
</tr>
<tr>
<td>35</td>
<td>Tandem</td>
<td>12, 13.5</td>
</tr>
<tr>
<td>40</td>
<td>Single</td>
<td>13</td>
</tr>
</tbody>
</table>

Remarks:
- Close to Tandem at 20 mph
- One second delay between the trucks

### Table 4.5 Strain Values of Sensor “F” for Different Speeds

<table>
<thead>
<tr>
<th>Speed MPH</th>
<th>Load SU4</th>
<th>One channel turned on, Readings are in με</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>F</td>
</tr>
<tr>
<td>10</td>
<td>Single</td>
<td>19</td>
</tr>
<tr>
<td>10</td>
<td>Tandem</td>
<td>15, 16</td>
</tr>
<tr>
<td>20</td>
<td>Single</td>
<td>17</td>
</tr>
<tr>
<td>20</td>
<td>Tandem</td>
<td>12.5, 14</td>
</tr>
<tr>
<td>30</td>
<td>Single</td>
<td>14</td>
</tr>
<tr>
<td>30</td>
<td>Tandem</td>
<td>17, 18.5</td>
</tr>
<tr>
<td>35</td>
<td>Single</td>
<td>16</td>
</tr>
<tr>
<td>35</td>
<td>Tandem</td>
<td>14, 15</td>
</tr>
<tr>
<td>40</td>
<td>Single</td>
<td>12</td>
</tr>
</tbody>
</table>

Remarks:
- Two seconds between the trucks

### Table 4.6 Strain Values of Sensors “CSM and F” for Different Speeds

<table>
<thead>
<tr>
<th>Speed MPH</th>
<th>Load SU4</th>
<th>Two Channels were on, Readings are in με</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>CSM</td>
</tr>
<tr>
<td>10</td>
<td>Single</td>
<td>17.5</td>
</tr>
<tr>
<td>10</td>
<td>Tandem</td>
<td>4.5, 14</td>
</tr>
<tr>
<td>20</td>
<td>Single</td>
<td>11.5, 12</td>
</tr>
<tr>
<td>20</td>
<td>Tandem</td>
<td>7, 11</td>
</tr>
<tr>
<td>30</td>
<td>Single</td>
<td>9.5</td>
</tr>
<tr>
<td>30</td>
<td>Tandem</td>
<td>6.75, 9</td>
</tr>
<tr>
<td>35</td>
<td>Single</td>
<td>8.5</td>
</tr>
<tr>
<td>35</td>
<td>Tandem</td>
<td>-----------</td>
</tr>
<tr>
<td>40</td>
<td>Single</td>
<td>7.5</td>
</tr>
<tr>
<td>40</td>
<td>Tandem</td>
<td>-----------</td>
</tr>
</tbody>
</table>

Remarks:
- Anomalies, CSM Readings, (4.5)
- Truck were close to each other
- Unsafes for this condition
In Table 4.6, there are no strain values for Tandem trucks at speeds of 40 and 45 MPH. Driving two fully loaded SU4 trucks in Tandem position at 40 and 45 MPH were not performed to prevent rear end collisions.

Table 4.7 Strain Values of Sensors “H, CSM, P2 and F” for Different Speeds

<table>
<thead>
<tr>
<th>Speed MPH</th>
<th>Load SU4</th>
<th>4 Channels were on, Readings are in με</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>H</td>
<td>CSM</td>
</tr>
<tr>
<td>10</td>
<td>Single</td>
<td>19</td>
</tr>
<tr>
<td>10</td>
<td>Tandem</td>
<td>18.5, 14.5</td>
</tr>
<tr>
<td>20</td>
<td>Single</td>
<td>13</td>
</tr>
<tr>
<td>20</td>
<td>Tandem</td>
<td>9.5, 8</td>
</tr>
<tr>
<td>30</td>
<td>Single</td>
<td>7</td>
</tr>
<tr>
<td>30</td>
<td>Tandem</td>
<td>11.5, 11.5</td>
</tr>
<tr>
<td>35</td>
<td>Single</td>
<td>10</td>
</tr>
<tr>
<td>35</td>
<td>Tandem</td>
<td>7, 10.5</td>
</tr>
</tbody>
</table>

At 30 mph and higher speeds, the strain values may not be considered at a true tandem. At these speeds, it was very difficult to drive the trucks close to each other. The difference in time between the trucks to the point of load varied from 2 to 4 seconds.

The following graphs (Figures 4.43 thru 4.40) indicate strain values of sensors at different locations for single and tandem truck load test at 10 mph speed. A close observation of values in the tables reveals that as more channels are turned on, drop in strain readings are noticed. The drop in strain readings are also observed as the speed is increased.
Table 4.8 Strain Values of Eight Sensors, "H, I, DSM, CSM, P1, P2, E and F" for Different Speeds, Both Directions

<table>
<thead>
<tr>
<th>Speed MPH</th>
<th>Load SU4</th>
<th>8 Channels were turned on, Readings are in μεs</th>
<th>Traffic was in both directions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>H</td>
<td>I</td>
</tr>
<tr>
<td>10</td>
<td>Single</td>
<td>8</td>
<td>6</td>
</tr>
<tr>
<td>10</td>
<td>Tandem</td>
<td>14 &amp; 10</td>
<td>14.6.5</td>
</tr>
<tr>
<td>20</td>
<td>Single</td>
<td>11.5</td>
<td>8.5</td>
</tr>
<tr>
<td>20</td>
<td>Tandem</td>
<td>3.5, 11</td>
<td>5.5, 10</td>
</tr>
<tr>
<td>30</td>
<td>Single</td>
<td>10</td>
<td>9</td>
</tr>
<tr>
<td>30</td>
<td>Tandem</td>
<td>4.9</td>
<td>9.5, 11</td>
</tr>
<tr>
<td>35</td>
<td>Single</td>
<td>11</td>
<td>8.5</td>
</tr>
<tr>
<td>35</td>
<td>Tandem</td>
<td>3.5, 7</td>
<td>6.5, 11</td>
</tr>
<tr>
<td>40</td>
<td>Single</td>
<td>6</td>
<td>8</td>
</tr>
</tbody>
</table>
Table 4.9 Strain Values of Eight Sensors, "H, I, DSM, CSM, P1, P2, E and F" for Different Speeds, Southbound

<table>
<thead>
<tr>
<th>Speed MPH</th>
<th>Load SU4</th>
<th>8 Channel were on, Readings are in $\mu e$ (Southbound direction)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$H$</td>
</tr>
<tr>
<td>10</td>
<td>Single</td>
<td>14</td>
</tr>
<tr>
<td>10</td>
<td>Tandem</td>
<td>12, 6.5</td>
</tr>
<tr>
<td>20</td>
<td>Single</td>
<td>10</td>
</tr>
<tr>
<td>20</td>
<td>Tandem</td>
<td>6.5, 7.5</td>
</tr>
<tr>
<td>30</td>
<td>Single</td>
<td>8.5</td>
</tr>
<tr>
<td>30</td>
<td>Tandem</td>
<td>5, 6.5</td>
</tr>
<tr>
<td>35</td>
<td>Single</td>
<td>6</td>
</tr>
<tr>
<td>35</td>
<td>Tandem</td>
<td>8, 4.5</td>
</tr>
</tbody>
</table>
Dynamic load test, two SU4 trucks in tandem at 10 mph
CSM=16.5 and 18.5 $\mu$ε

**Figure 4.34** Dynamic Load Test, Two SU4 Trucks in Tandem at 10 mph Over a Single Sensor, CSM=16.5 and 18.5 $\mu$ε

Dynamic Load test, two SU4 trucks in tandem at 10 mph
F=15 & 16 $\mu$ε

**Figure 4.35** Dynamic Load Test, Two SU4 Trucks in Tandem at 10 mph, F=15 & 16 $\mu$ε
Figure 4.36  Dynamic Load Test, Single SU4 Truck at 10 mph, CSM=17.5 and F=11 με

Figure 4.37  Dynamic Load Test, Two SU4 Trucks in Tandem at 10mph, CSM=14&9.4 and F=15&13.5 με
Figure 4.38 Dynamic Load Test, Two SU4 Trucks in Tandem at 10 mph
H=18.5 & 14.5, CSM=11.5 & 6.5, P2=13 & 15, F=12 & 14 με

Figure 4.39 Dynamic Load Test, Single SU4 Truck at 10 mph, H=14 & 10, l=14 & 6.5, DSM=18 & 9, CSM=19 & 5, P1=8.5, P2=10.5, E=14.5 & 6.5, F=15 & 8.5 με
The following graphs (Figures 4.41 thru 4.50) indicate strain values of sensors at different locations for single and tandem truck load test at 20 mph speed.

Figure 4.40 Dynamic Load Test, Two SU4 Trucks in Tandem at 10 mph
G=2, H=13, I=9, J=9, P1=5, P2=7.5, ASM=2.5, BSM=5, CSM=6, DSM=6, C=6, D=4, E=4.5, F=0.5 με

Figure 4.41 Dynamic Load Test, Single SU4 Truck at 20 mph, CSM=13.5 με
Figure 4.42 Dynamic Load Test, Two SU4 Trucks in Tandem at 20 mph, CSM=12 & 13.5 με

Figure 4.41 shows sensor CSM has recorded 13.5 με under a single SU4 at 20 mph. Figure 4.42 shows sensor CSM has recorded 13.5 με under a second truck in tandem and Figure 4.43 shows sensor F has recorded 13.5 με under second truck in tandem. This comparison shows that the strain value of second truck in tandem is close to the strain value for a single truck. This concludes that stress for a tandem truck under the same condition is close to the stress for a single truck. Also, from the same figures, it can be seen that the value of strain for surface mount sensor is slightly larger than the sensor bonded to rebar, for the similar loading condition.
Figure 4.43 Dynamic Load Test, Two SU4 Truck in Tandem at 20 mph, F= 12.5 and 14 με

Figure 4.44 Dynamic Load Test, Single SU4 Truck at 20 mph, CSM=12, F=11.5 με
Figure 4.45 Dynamic Load Test, Two SU4 Trucks in Tandem at 20 mph, CSM=11.5&12, F=9&13 \( \mu e \)

Figure 4.46 Dynamic Load Test, Single SU4 Truck at 20 mph H=11.5, I=8.5, DSM=13, CSM=6.5, PI=5.5, P2=5.5, E=5.5, F=3 \( \mu e \)
Figure 4.47 Dynamic Load Test, Single SU4 Truck at 20 mph, $H=10$, $CSM=6.5$, $P2=6.5$, $F=5 \, \mu \varepsilon$

![Graph showing dynamic load test results for a single SU4 truck.](image)

Figure 4.48 Dynamic Load Test, Two SU4 Trucks in Tandem at 20 mph, $H=3.5 \, & \, 11$, $I=5.5 \, & \, 10.5$, $DSM=13.5 \, & \, 1$, $CSM=12.5 \, & \, 3 \, \mu \varepsilon$
Figure 4.49 Dynamic Load Test, Single SU4 Truck at 20 mph, H=11, I=10.5, DSM=13.5, CSM=12.5, P1=8, P2=8.5, E=17.5, F=12 με

Figure 4.50 Dynamic Load Test, Single SU4 Truck at 20 mph, DSM=19.5 and H=10 με
In Figure 4.50, two readings are depicted, other reading were very low values of strain. The horizontal lines indicate the temperature readings.

The following graphs (Figures 4.51 thru 4.57) indicate strain values of the sensors at different locations for single and tandem truck load test with a speed of 30 mph.

Figure 4.51 Dynamic Load Test, Two SU4 trucks in tandem at 30 mph, CSM=13.5 & 15.5 με, CSM is a Surface Mount Sensor

Figure 4.51 shows the readings of strain values for a true tandem truck condition. Normally, this is not a reoccurring situation except for the field experiment. The graphs shows increase in strain value due to the second truck. Figure 4.52 also shows a true tandem condition, however, the strain due to the second truck has decreased. It is stipulated that this decrease in the strain value can be due to the location of the sensor bonded to the reinforcing steel.
Figure 4.52 Dynamic Load Test, Two SU4 Trucks in Tandem at 30 mph, F= 17 and 15 με

Figure 4.53 Dynamic Load Test, Two SU4 Truck in Tandem at 30 mph, CSM=7 & 11, F=8 & 14 με

Figure 4.51 indicates that two trucks of equal weights were approaching the bridge at the speed of 30 mph. However, the second truck fell too far behind the first truck. The graph shows that the second truck did have an apparent
effect on the sensors and increased the readings. It is not clear why the second truck resulted in the sensors readings to increase since there is almost a 40 second time lapse between the two trucks. It is possible that the second truck had reduced its speed to increase the readings by 1.5 to 3 $\mu\varepsilon$.

![Dynamic Load Test, Two SU4 Trucks at 30 mph, CSM=6&7.5, F=7 & 10 $\mu\varepsilon$](image1)

**Figure 4.54 Dynamic Load Test, Two SU4 Trucks at 30 mph, CSM=6&7.5, F=7 & 10 $\mu\varepsilon$**

![Dynamic Load test, single SU4 truck at 30 mph, CSM=13 $\mu\varepsilon$](image2)

**Figure 4.55 Dynamic Load Test, Single SU4 Trucks at 30 mph, CSM=13 $\mu\varepsilon$**
Figures 4.56 Dynamic Load Test, Two SU4 Trucks in Tandem at 30 mph, H= 11.5 and 11.5, CSM = 4.5 and 5.5, F= 5 and 10.5 $\mu e$

Figure 4.57 Dynamic Load Test, Single SU4 Truck at 30 mph, H=7, CSM=9, P2=6.5, F=3 $\mu e$
Dynamic Load test, single SU4 truck at 30 mph, G=3, H=6.5, I=9, J=8.5, DSM=15, CSM=1.5, ASM=1.5, BSM=-1.5, P1=4.5, P2=4, C=2.5, D=4, E=10.5, F=16, T1=12.55, T2=18.15 °C

**Figure 4.58** Dynamic Load Test, Single SU4 Truck at 30 mph, G=3, H=6.5, I=9, J=8.5, DSM=15, CSM=1.5, ASM=1.5, BSM=-1.5, P1=4.5, P2=4, C=2.5, D=4, E=10.5, F=16 με, T1=12.55, T2=18.15 °C

The next graphs (Figures 4.59 thru 4.68) show strain values of the sensors at different locations for single and tandem truck load test at 35 mph speed. It was determined not to run the test with tandem trucks since there was a risk of an accident for the trucks due to the proximity of the trucks. In one case, eight sensors were turned on. For the sake of clarity, the eight-sensor graph was divided into two four-sensor graphs. Figure 4.59 is the graph of the first four sensors of an eight sensor group from Figure 4.61. Figure 4.60 is the graph of second four sensors of an eight sensor group from Figure 4.61. These graphs are illustrated in Figures 4.59, 4.60 and 4.61.
Figure 4.59 Dynamic Load Test, Single SU4 Truck 35 mph, H=11, I=8.5, DSM=4, CSM=9.5 με

Figure 4.60 Dynamic Load Test, Single SU4 Truck, at 35 mph, P1=7, P2=6, E=6, F=12 με
All 16-channels were turned on for the fully loaded SU4 truck to drive over the bridge with a speed of 40 mph. Figure 4.62 shows that sensors G and P2 did not respond to the truck while the response of others were 1 or 2 $\mu$e with a maximum J=5 $\mu$e.
4.6 Dynamic Response of the Bridge Subject to Live Traffic Load

The next group of graphs represents vehicles traversing the bridge. The type, size and speeds of these vehicles were not known. The highest peak indicates the heaviest vehicle in this group. This data was collected remotely approximately 35 miles from the bridge. In some cases, the load was observed to be predominantly SU4 trucks hauling dirt to a construction site near the bridge. The difference between the values for each period can generate predicted increase in strain until it will reach the safe operating value at which time, the management can make an intelligent decision with regard to the bridge repair or replacement. Refer to Chapter 3 for design strain values at which the bridge can safely and indefinitely operate. Figures 4.63 through 4.68 show the strain values for four consecutive intervals. The initial strain values and three subsequent readings recorded after 6 months, 11 months and 13 months from the initial readings are presented in these graphs. The increase in strain values after each period of time indicate the effect of time and traffic loading on the bridge.

![Dynamic Strain Readings in Real-Time](image)

**Figure 4.63 Dynamic Strain Readings in Real-Time Recorded on February 4, 2005 at 4:30 pm, H=5, I=7, E=7, F=5.5 με**
Figure 4.64 indicates an increase in strain values for “I and F” sensors by as much as 6 and 4 με respectively (Figure 4.64) after about six months later from the initial readings.

![Dynamic Strain Readings in Real-Time, Recorded in June 2005 at 11:39 am, I=13, F=9.5 με](image)

Figure 4.65 depicts the increase in strain values for “H” and “F” sensors by as much as 8 and 7.5 με respectively approximately six months after from initial readings.
The following Figure 4.66 shows the increase in strain values for “E” and “F” sensors by as much as 12 and 22.5 $\mu \varepsilon$, respectively. This increase took place approximately one year after the initial readings. Sensor “D” is not in the traffic lane, therefore, the strain value of 11 $\mu \varepsilon$ is due to the load distribution.
The strain values of sensors E and P2 (the pair sensors to F and P1 sensors) shown in Figure 4.67 are lower than those shown in Figure 4.68 by magnitude of 6.5 and $2 \mu \varepsilon$ . This indicates a reasonable change in the magnitude of the strain values in that period and thus the strains are indeed time dependent variables.
Dynamic Strain readings in real-time, recorded in November 30, 05 at 741 am, E=21.5, P2=11 με

Figure 4.67 Dynamic Strain Readings in Real-Time, Recorded on November 30, 2005 at 7:41 am, E = 21.5 and P2= 11 με

Dynamic Strain readings in real-time recorded in January 2, 2006 at 7:45 am, P1=13, F=28 με

Figure 4.68 Dynamic Strain Readings in Real-Time, Recorded on January 2, 2006 at 7:45 am, F=28, P1=13 με
4.7 Conclusions

Although, there was a serious problem with the software for communication and data collection, the end result was a successful experiment. The communication problem was eventually resolved with the development of version v2 software. Credit for accurate and sensible data collection goes to the meticulous and skillful installation of the sensors, with both surface mount and those bonded to the reinforcing steel. Truck load test proved to be an effective and accurate for both load tests, static and dynamic conditions. The strain values of P1, P2, E and F sensors from Tables 4.1, 4.2 and 4.3 are a close match to the strain values of the sensor from the graphs of Figures 4.60 to 4.65. The strain values of E and F sensors show an increasing trend with time from January 2005 to January 2006. The difference for this period was 22.5 με. The reading of sensor “F” remained the same (28 με) from January 2 to January 15, 2006. Synchronized cameras and weighing scales combined with the selected sensors will provide a complete invaluable data for analysis of bridge behavior in various loading situations.
CHAPTER 5. RESULTS AND COMPARISONS

5.1 Introduction

In summary, this chapter will discuss the issues presented in the contexts of preceding chapters. First, a brief discussion on evolution of the topic of this research is presented. Then, the other items of discussions are presented as follows:

(a) Why the fiber optic sensing technology was preferred over the non-fiber optic measuring system. (b) Why a particular sensing system from a group of systems within the same technology was selected and how these systems will compare. (c) Site specific implementation of measuring system in comparison to other common procedures. (d) The significance of objectives of this study and how requirements of these objectives were met. (e) How the analytically predicted strain values and moments compared to the experimental results (Evaluation of collected data).

5.2 Evolution of the Topic of this Dissertation

The author has been involved with issues related to design, construction and inspection of existing and new bridges for more than 20 years. During early years of observations, there were questions about weight restrictions imposed on newly constructed bridges that were designed using LFD-based format
presented in AASHTO Standard Specifications for highway Bridges (1944). This issue was soon resolved (RSH 1990’s) (Waters Avenue bridge in Tampa, Florida, designed by “Reynold Smith and Hills”, a structural engineering consultant, a Jacksonville, Florida based office). SU4 truck was used as an alternative design live load truck in addition to HS20-44. The purpose of this trial was to investigate the effect of SU4 in the design of the bridge and compare the results of analytical stress/strain prediction with those resulted from HS20-44. The incorporation of SU4, a short base four-axle 70,000 pounds truck as live load design proved to be effective and the newly constructed bridges were no longer subject to weight restrictions due to SU4 trucks. Later, the design was checked for all Florida legal load trucks.

The other controversial and contradictory issue was about structural integrity of more than eighty existing bridges. These bridges were typically reinforced concrete structures. The substructure was consisted of prestress concrete piles and cast-in-place concrete pile caps. The superstructure was composed of prestress concrete channel beams and cast-in-place concrete deck slab (figure 2.28). these bridges were designed and constructed in 1970’s. The design and construction of such a large number of bridges was possible due to the simplicity of the design (AASHTO Standard Specifications for highway Bridges, LFD method) which had helped to construction such a large number bridges efficient and quick in a short period.

Although, these bridges were posted for about 1/3 of their allowable load carrying capacity, they were in route for and subject to variety of different trucks
in a daily basis. They were routinely observed and inspected for signs of stress
due to heavy truck traffic and overload. A long term monitoring resulted in a
conclusion that these bridges were capable of carrying load at their full capacity
without jeopardizing their structural integrity. The good conditions of bridges
were verified through a three phases of non-destructive test and structural
investigation. The result of this investigation concluded that none of those
bridges required weight restrictions. The process for removal of weigh
restrictions was tedious and time consuming (more than four years). A nominal
effort by the author and minimal budget of about $150,000.00 resolved a
monumental issue of budgeting and spending several millions of dollars to
replace more than eighty bridges. The resolution was approved by Hillsborough
County (Board of County Commissioners). All weight restriction signs were
removed and the bridges were open to all types of trucks including Florida legal
loads. These bridges were eventually classified as functionally obsolete and
were placed in a long term program for widening or replacement.

The point of this discussion is, if the present technology was present at
that time, the bridges could have been fitted with smart sensors, they could have
be continuously monitored for any sign of stress at the critical locations and
managed confidently and effectively. Sensors transmit stresses in quantitative
values as micro strain (\( \mu \varepsilon \)) and these values can be compare with pattern of
strain history that has been kept in record. Any abnormal condition can be
detected instantly and then a closer observation and investigation can follow.
This is how the concept for topic of this study was conceived. New bridges can
be instrumented with a fraction of total replacement budget. The bridge engineers and management can sit back, relax and just watch the monitor for any sign of over stress. Figures 4.61 thru 4.65 clearly demonstrate bridge behavior under real live load quantitative measure of strain values.

5.3 Selection of Sensing System for Bridge Instrumentation

Smart sensing system is fairly a new technology and there is a limited literature available practically with regard to the topic of this research. The objectives of the research were to investigate addressed through a literature review process. Literature review provided sufficient information resulting in selection of sensing system suitable for East Bay Road Bridge instrumentation. Literature review compared the non-fiber optic measuring systems fiber optic sensing technologies. These sensing systems are insensitive to electromagnetic interference, they are very small and light, they are ideally suited to be embedded in composite material, they do not affect the mechanical properties of host material, they are insensitive to corrosive environment thus will not corrode and they are capable of withstanding high temperatures. A laboratory experiment was set up to examine Fabry-Perot strain gauge and data acquisition system (Chapter 2 Experimental). Two other commonly used sensing systems were researched and evaluated for the final application on the bridge.
5.4 Comparison Between the Most Commonly Used Sensors

Three different types of fiber optic sensors have been commonly used in civil engineering infrastructure. Each type has specific application, advantage and disadvantage with respect to others. The following sections 5.4.1 and 5.4.2 present a brief description of each type.

5.4.1 Fiber Bragg grating and Long Gauge Fiber Optic Sensors

Fiber Bragg grating and long gage fiber Optic Sensors are considered Distributed Optical Sensing system that can possibly measure (sense) at multiple points with a single optical fiber. Several sensors can be attached to the same cable for up to 14 miles long. This process is called multiplexing.

However, these systems have multiple disadvantages for being implemented in this study. These disadvantages are listed as: (1) complex techniques that often have to be used for signal processing, (2) require highly stable and expensive laser light source, (3) precision depends on wavelength stability and Bragg grating isolation capacities, (4) affected by vibration and temperature effects, (5) unique fiber optic cable is brittle and must be handled with caution. Use of these systems are associated with potential risk of loosing all the installed sensors if the cables are damaged or broken, (6) difficult to monitor very small wavelength changes (FBG), Considerable time averaging is often required to assess and map the spatial changes in loss or scattering coefficients along a fiber (Back scattering) and finally detection units (Data
loggers) are often incompatible with FBG and unreliable because they are built by different manufacturer.

### 5.4.2 Fabry-Perot Fiber Optic Sensor

Except two minor disadvantage (1) can run only two to three miles and (2) can not measure multiple points with one optical fiber (multiplexing), this “White Light Interferometry FISO Point Measurement” system has many advantages which made it completely suitable for this study.

The advantages of this system include but not limited to: (1) High sensitivity for multiple quantities such as temperature, strain, pressure, displacement and reflective index with the same signal conditioner, (2) a simple White Light Interferometry technique for treatment of the signal, and that is an optical instrument that allows two beams of light derived from a single source (and thus of the same frequency and in phase at identical distances from the source) to traverse paths whose difference in length determines the nature of the interference pattern obtained when the beams are allowed to interfere. The wavelength of light can be measured if the path length difference is known, and vice versa, (3) Inexpensive and calibration free signal conditioner (data logger), (4) Thirty two channel signal conditioner has the capability of adding and/or replacing one sensor at the time for installation and maintenance, (5) it is tolerant to light loss, (6) data can be collected from each individual channel with a portable hand held signal conditioner FTI-10, (7) unlike Distributed Optical Sensing systems, the sensors and fiber optic cables can be repaired on the site.
of the structure, (8) it has a full diagnostic function available at all time and capable of up to 200,000 Hz sampling rate, (9) it has a high precision suitable for medical applications. Because of all the above reasons, Fabry-Perot fiber optic sensors were used in this study.

5.3 Site Specific Instrumentation

Sensors are normally surface mount or embedded. The surface mount sensors are bonded to the host structure with adhesive. On the other hand, the embedded sensors are either welded to structural steel or bonded to the smooth and properly prepared surface of reinforcing steel. A specific case required installation of sensors on topside of concrete slab. This unusual condition not been practiced before and no instructions were available. The step-by-step process is described in Chapter 2 and illustrated by numerous photographs.

5.4 The Significance of Objectives of this Study

A case study for the application of Fiber Optic Sensors (FOS) for remote health monitoring of bridge structures is presented. A total of sixteen Fabry-Perot FOS sensors were installed on the East Bay Road Bridge, in Hillsborough County, Florida. The bridge is a 4-span continuous reinforced concrete deck-type structure. The bridge is considered the first smart structure in the state of Florida. The Fabry-Perot sensors were both bonded to the longitudinal reinforcing bars and surface-mounted to the concrete deck. Detailed step-by-step description of the installation process is presented in Chapter 2. Static and dynamic tests of
the bridge under SU4 trucks were conducted. A finite element model was
developed, and its output was compared to the experimental data obtained from
the truck load tests. The results confirmed the accuracy of Fabry-Perot sensors
in evaluating the bridge behavior under traffic loads. A remote communication
system was established through phone lines in order to connect the acquisition
system to the Internet. This technique enables live traffic monitoring from a
central station located in the county’s maintenance office. Live traffic data are
currently being collected and stored on Compact Disk to generate a long term
strain history for the bridge. This data will be used to facilitate the bridge
maintenance process, receive early warnings regarding possible structural
deficiencies, and assist in decision-making processes regarding functionality of
the bridges. The proposed remote health monitoring technique with FOS sensors
proved to be practical, cost-effective, and efficient providing its installation is
performed in a very careful, accurate and skillful manner. Data analysis and
evaluation confirmed current LRFD specifications for deck-type bridges are
highly conservative.
5.5 Discussions

Current AASHTO LRFD design width used in analysis produced strain higher than those of field measured corresponds to a conservative design approach. To have more accurate design, the distribution width can be increased. Analysis with distribution width is almost twice more than the one used from the code (11.87'). Strain resulted from incorporating twice the code distribution width (20’) seems to have decreased and match better with the collected data. This assumption is based on observed uncracked condition.

Further continuous monitoring might indicate an increase in collected strain possibly up to the crack. At this point, after several years of monitoring, the refinement of distribution width might be possible.

Continuous monitoring of the bridge subject to traffic is essential to collect data for condition evaluation and damage assessment. This data can also be used to predict the useful life of the bridge. Theoretical life time expectancy of East Bay Road Bridge is 75 years. Author has reviewed design and construction documents for East Bay Road Bridge and has not come across any technical information or references verifying the 75 years predicted life expectancy for this structure.

The results of SU4 truck tests along with the output of the finite element model, as well as the data collected from remote monitoring suggest that the bridge deck did not experience cracking under traffic loads, or experienced only secondary widely spaced cracks not visible to naked eye. Also, close physical inspection and Visual observations of bridge deck underside confirmed this
finding. To evaluate the performance of the bridge under service loads, the moment-curvature relationship of 1ft strip of the bridge section was developed using inelastic fiber beam models. The fiber constitutive models used for confined and unconfined concrete followed the modified Kent and Park (1971) model, and the reinforcing steel stress-strain behavior was assumed to be elasto-plastic. From the moment-curvature plot in Figure 3.23, it can be concluded that:

(a) The bridge is over-reinforced, as the concrete crushing point (ultimate strength level) occurs before steel yielding.

(b) The cracking point is higher than the traffic level point. The bending moment corresponding to traffic level was evaluated from finite element analysis of the bridge under SU4 trucks. These values also match with the data recorded through remote monitoring.

(c) The ultimate strength of the bridge highly exceeds the ultimate moment demand assumed in the LRFD design process.

The preceding observations, along with the data collected through remote monitoring (Tables 4.1 thru 4.3 and Figures 4.60 thru 4.65) suggest that the current design specifications for deck-type bridges are highly conservative under service loading. Further studies and data collection are needed to confirm this conclusion. In addition, research and data analysis need to be performed at the ultimate stage.
5.6 Evaluation of Collected Data

In this section, an evaluation of the design specifications for the East Bay Road Bridge is performed using the collected FOS data. It should be emphasized that the current data is changing on a daily basis due to the gradual deterioration of the bridge condition, as confirmed by the observed behavior described in Chapter 4. An accurate evaluation of the design specifications should be based on the maximum recorded strain values over the entire life time of the bridge. Since these strain values can not be predicted presently, the current study was based on the values recorded so far. These values are assumed to represent the service condition of the bridge. The increase in recorded data, which suggests gradual damage of the bridge, will be discussed in the next section.

The maximum positive strain recorded since the opening of the bridge was observed on 1/30/06 for sensor F, and is equal to 28 $\mu$ε. The maximum negative strain recorded equals 18.5 $\mu$ε, and was recorded on 1/30/06 for sensor H. These values suggest that the bridge deck did not experience cracking under traffic loads, or experienced only secondary widely spaced cracks, as the cracking strain is 320 $\mu$ε. Visual observations also confirmed this fact. Since the original design was based on cracked section analysis, this design is assumed to be conservative.

To evaluate the performance of the bridge under service loads, the moment-curvature relationship of an 11.87ft strip of the bridge section was developed using inelastic fiber beam models (Ayoub and Filippou 2000; Ayoub 2003). The fiber constitutive models used for confined and unconfined concrete
followed the modified Kent and Park model (Kent and Park 1971), and the reinforcing steel stress-strain behavior was assumed to be elasto-plastic. The width of the strip was assumed according to AASHTO specifications for tributary widths for design trucks (AASHTO 2004) as discussed previously in Chapter 3. From the moment-curvature plot in Figure (5.1), it can be concluded that:

(a) The bridge is over-reinforced, as the concrete crushing point (ultimate strength level) occurs before steel yielding.

(b) The cracking point is higher than the actual service level point. The bending moment corresponding to actual service level was evaluated from finite element analysis of the bridge under SU4 trucks, which also matches with the data recorded through remote monitoring. These moments are also slightly lower than the value used in the service design process based on dead load plus lane and truck loads, which equals 4600 kip-in. However, since the design process assumed the concrete section to crack under service loads, the corresponding computed maximum concrete strain was 319.4 $\mu\varepsilon$, which highly exceeds the recorded values.

(c) The ultimate strength of the bridge highly exceeds the ultimate moment assumed in the design process, which equals 7200 kip-in.

The preceding observations, along with the data collected through remote monitoring suggest that the current design specifications for deck-type bridges are highly conservative under service loading. Additional data are currently being collected in order to confirm this conclusion.
To further elaborate on the issue of recorded strains before and after cracking, the following discussion is presented.

### 5.7 Flexural Cracking in Bridge Concrete Deck

Consider a concrete element reinforced with steel bars. In this case tensile stresses are transmitted across the crack through the bonded reinforcing steel.

The following example serves as illustration of this behavior:

A reinforced concrete member is subjected to axial tension in Figure (5.2a). If the axial stress does not exceed the tensile strength of concrete, the member is ideally free of cracks. This state is referred to as state 1. The steel and concrete strains, $\varepsilon_{s1}$ and $\varepsilon_{c1}$, respectively, are compatible along the member. The average strain is:
\[ \varepsilon_{s1} = \varepsilon_{c1} = \frac{N}{E_c (A_c + n A_s)} = \frac{N}{E_c A_1} \]  

(5.1)

where \( N \) is the value of the axial force, \( A_c \) and \( A_s \) are the cross sectional areas of concrete and steel and \( n = E_s / E_c \) with \( E_s \) and \( E_c \) being the moduli of elasticity of steel and concrete. When the concrete stress exceeds the tensile strength, cracks appear. At a crack the stress is completely carried by the reinforcement and the concrete stress is zero. This condition will be referred to as state 2. The steel stress and strain are given by:

\[ \sigma_{s2} = \frac{N}{A_s} \]  

(5.2)

\[ \varepsilon_{s2} = \frac{N}{E_s A_s} \]  

(5.3)

In the portion between two cracks part of the tensile stress carried by the steel at the crack is transferred to the concrete through bond. The stress and strain are in an intermediate state between states 1 and 2, as depicted in Figure (5.2). Midway between consecutive cracks, the section is in state 1 and the steel stress is less than \( \sigma_{s2} \). At a crack the section is in state 2 with the steel stress at its maximum value \( \sigma_{s2} \) and with the concrete stress equal to zero. The difference in steel stress is transmitted to the concrete through bond, so that the member elongates less than the bare steel. Denoting the average strain of the cracked member in Figure (5.2a) as \( \varepsilon_m \), then

\[ \varepsilon_m = \frac{\Delta L}{L} \]  

(5.4)

where \( L \) is the original length of the member and \( \Delta L \) is the member elongation.
Before cracking, compatibility of strains is maintained so that Eq. 5.1 holds
\( (\varepsilon_m = \varepsilon_{s1} = \varepsilon_{c1}) \). After cracking, the value of \( \varepsilon_m \) lies for a given stress level between the steel strain in the perfectly bonded case \( \varepsilon_{s1} \) and the steel strain at the crack \( \varepsilon_{s2} \).

![Diagram of stresses at cracking](image)

**Figure 5.2 Stress at Cracking**
Denote the reduction in steel strain due to the participation of concrete between cracks by $\Delta \varepsilon$, then

$$\varepsilon_m = \varepsilon_{s2} - \Delta \varepsilon$$  \hspace{1cm} (5.5)

Based on experimental evidence, it is assumed that $\Delta \varepsilon$ varies inversely with the applied axial load $N$ (CEB 1985):

$$\Delta \varepsilon = \Delta \varepsilon_{\text{max}} \frac{N_r}{N}$$  \hspace{1cm} (5.6)

where $N_r$ is the cracking load and $\Delta \varepsilon_{\text{max}}$ is the steel strain difference between states 1 and 2 at the first crack.

From the graph in Figure (5.3):

$$\Delta \varepsilon_{\text{max}} = (\varepsilon_{s2} - \varepsilon_{s1}) \frac{N_r}{N}$$  \hspace{1cm} (5.7)

Substitution of Eq. 5.6 in Eq. 5.7 gives the average strain value of the member:

$$\varepsilon_m = (1 - \zeta)\varepsilon_{s1} + \zeta\varepsilon_{s2}$$  \hspace{1cm} (5.8)

where $\zeta$ is a dimensionless parameter that represents the amount of cracking and is given by:

$$\zeta = 1 - \left( \frac{N_r}{N} \right)^2$$  \hspace{1cm} (5.9)

$\zeta = 0$ for an uncracked member. The difference between the solid line and the line representing the bare steel in Figure 5.3 is referred to as tension stiffening. It represents the increase in stiffness due to the concrete contribution between cracks. Tension stiffening can be significant up to the yielding of the
reinforcement, but drops considerably near the yield point. After yielding of the reinforcement at the most critical section, the member elongates without significant increase in load and the tension carried by the concrete becomes negligible.

The installed FOS sensors are either embedded and bonded to the rebars or surface-mounted to the concrete. The surface-mounted sensors would record the strain $\varepsilon_{c1} = \varepsilon_{s1}$ before cracking. After cracking, if the sensor is exactly located at the crack position, its reading will drop to zero. It is more likely, however, that the sensor exists between two cracks. In this case the reading of the sensor will drop but to a non-zero value. Gradual decrease of the sensor readings indicate
the formation of additional cracks until the deck becomes severely cracked. In this case, the readings will approach zero values. The role of the surface-mounted sensors therefore is to detect the formation of the initial cracks and to monitor the crack propagation with time. After the deck becomes severely cracked, these sensors will not be able to record service strain values.

The role of the embedded sensors on the other hand, is to monitor the service strain values in addition to recording the maximum steel stress values at cracked locations. Before cracking the sensors would record the strain \( \varepsilon_{s1} = \varepsilon_{c1} \). When the strain \( \varepsilon_{c1} \) reaches the concrete cracking strains, this will indicate the formation of the first crack. The steel strain at the crack location will increase and reach the value of \( \varepsilon_{s2} \), but the steel strain between cracks will be less than \( \varepsilon_{s2} \). If the sensor is exactly located at the crack position, it will record the value of \( \varepsilon_{s2} \). This is however unlikely to happen, and it is assumed that the sensor is recording an average value that equals \( \varepsilon_{m} \) as defined in Eq. (5.5). In order to extrapolate the value of the steel strain at the crack location, Eq. (5.8) is used to estimate the value of the axial force \( N \) resisted by the reinforced concrete section, which is again used with the help of Eq. (5.3) to evaluate the steel strain at the crack location \( \varepsilon_{s2} \). A further increase in the value of either \( \varepsilon_{m} \) or \( \varepsilon_{s2} \) under the same loading conditions indicate the formation of additional cracks or a decrease in the value of the crack spacing \( S \) identified in Figure (5.2). Currently, the recorded FOS strain values indicate no cracking or the existence of minor and widely space cracked. As the bridge deteriorates with time, the sensors readings should increase, and the process described above for both surface-mounted and
embedded sensors will be implemented to detect the formation and propagation of cracks, as well as the maximum steel stresses at the crack locations.

5.8 Evaluation of Design Specifications

As stated earlier, the maximum recorded positive strain value was \(28 \mu \varepsilon\), while the maximum negative strain was \(13 \mu \varepsilon\). The corresponding design values are \(319 \mu \varepsilon\) and \(218 \mu \varepsilon\) for positive and negative cases respectively for cracked conditions, and \(67 \mu \varepsilon\) and \(48 \mu \varepsilon\) for uncracked conditions. These values indicate that the design process was highly conservative for the assumed cracked conditions. Even if the section is assumed to be uncracked, the design values exceed the maximum recorded values. The discrepancy between the design and recorded strain values could be attributed to the following parameters: (a) The assumed distribution width in the design calculations, (b) the inclusion of the barrier wall in the analysis, and (c) the overestimation of the actual truck loads acting on the bridge. Each of these items is described in more details herein.

(a) Distribution width: The distribution width assumed in the analysis equals 11.87 ft. To evaluate the accuracy of this distribution width, finite element analysis of the bridge deck using shell elements and under the static load of an SU4 truck is performed and compared to analysis with frame elements and also to experimental results. These analyses were described in details in Chapter 3 (Figures 3.19, 3.25 and 3.26). The shell finite element results indicate that the strains within the 11.87 ft strip around the wheel load are within the range of 70-
100% of the peak strain. The width of the strip around the wheel load with non-zero stresses actually equals 18 ft. This conclusion is also valid from the experimental plots of Figure (3.19). The frame analysis of the bridge deck with an equivalent width of 11.87 ft produced maximum strains of 61 με as described at the end of Chapter 3. This value is clearly overestimated as the maximum recorded value was 28 με. If the frame analysis was repeated with a section width of 11.87' x 61/28 = 25.85 ft, the maximum resulting strain would equal 40 με, which matches with the recorded data. In conclusion, it appears that the distribution width of 11.87 ft provided by the code is highly conservative assuming the current uncracked condition of the bridge. A value of 18 ft seems to better match with recorded data. This conclusion, however, is expected to change as the bridge starts cracking and deteriorates with time. The author will continue to monitor the behavior of the bridge and re-evaluate the distribution width that matches with cracked conditions. It is the author's belief, however, that (b) **Barrier Wall**: Traffic barrier walls in solid slab bridges act as upward vertical beams which enhance the moment capacity of the bridge significantly (Shahawy et al., 1999). The effect of traffic barrier walls and bridge sidewalk parapets were observed in East Bay Road Bridge. This effect was sensed by the gauges located near the walls. Sensors C, D, G, J, ASM and BSM had strain values of 1.5 to 2.5 με while at the same time under the same loading condition, sensors E, F, H, I, CSM and DSM located under the wheel load had strain values ranging from 19 to 28 με. Neither AASHTO standard specifications (LFD) nor AASHTO LRFD code have considered the effect of barrier walls in design of bridge slab.
Overall low strain readings of sensors are due to slab stiffness attributed by the barrier walls.

To further investigate this conclusion, the finite element analysis of the bridge deck was repeated with the inclusion of the barrier walls. The barrier walls were modeled as additional shell elements acting at the edges of the deck. The strain contour plots for this case are shown in Figure 5.4 and are compared to the ones described earlier in Chapter 3 and shown again in Figure 5.5, where the barrier walls were not simulated. From the figures two conclusions are drawn: (a) The strains near the edge beams were minimal for the case of the model with the barrier walls confirming the observed recorded behavior, and (b) the maximum strains under the wheel loads dropped from a value of $15.63 \mu\varepsilon$ to $13.2 \mu\varepsilon$, which accounts for a $16\%$ decrease.

From the discussion above and from the recorded strain values, it is the author's belief that there exists a major need to include the effect of barrier walls in the design and analysis of bridge structures.
Figure 5.4 Strain Contours with Inclusion of Barrier Walls

Figure 5.5 Strain Contours without Inclusion of Barrier Walls
(c) **Actual Load:** East Bay Road Bridge was designed based on AASHTO LRFD Code with the governing design live load LH-93. LH-93 is a notional non-existing truck that has been configured to produce maximum critical live load condition. Without application of specialized equipment such as scale and cameras, there is a little information to verify the actual trucks weight and type traversing over the bridge. However, abundant of SU4, C4 and C5 trucks moving over the bridge is evident by frequent field observation. The strain values recorded through remote monitoring is also in conformance with the strains obtained from bridge load test subject to fully loaded SU4 trucks. The small strain values (28 and 19 με for positive and negative moments respectively) sensed by FOS will only confirm the conservative state of LRFD design, conservative live load distribution width and effect of traffic barrier walls rather than absence of actual load in motion over the bridge.

(d) **Bridge Rating:** The bridge analysis under Florida legal trucks was performed and presented in Section 3.6. As stated earlier, the current practice for this analysis is based on LFD procedures, while the original design is performed in accordance with LRFD procedures. This incompatibility between the design and rating procedures has caused confusions between design engineers and has led the Florida Department of Transportation lately to suggest that the rating be performed in accordance with LRFD procedures. For instance, in several cases bridges designed in accordance with LRFD procedures did not pass the rating test before the opening of the bridge, and the bridge therefore needed to be posted. Current design and analysis tools, however, are still tailored to match
with LFD procedures and there exists a need to modify these tools and so they match with the LRFD approach.

For the East Bay bridge, the maximum positive strains for the Florida legal trucks were presented in Table 3.4 for cracked conditions. The maximum positive strain was that of the SU4 truck and is equal to $320\ \mu e$, while the maximum negative strain was that of the C4 truck and is equal to $249\ \mu e$. While the maximum positive strain is close to the maximum positive design strain of $319.4\ \mu e$ under the HL-93 truck, the maximum negative strain of $249\ \mu e$ exceeds the maximum negative design value of $217\ \mu e$. This conclusion implies that the C4 truck was more critical for the East Bay bridge than the design HL-93 truck. Considering the fact that the C4 truck is a real truck that is likely to be moving over the bridge, the strain obtained from this truck confirms the conclusion that the assumption of cracked behavior used in the design is conservative.

The maximum positive and negative strains for uncracked conditions are presented in Table 3.6. The maximum positive values are $67\ \mu e$ for both the design and the critical SU4 truck. The maximum negative values are $45\ \mu e$, and $52\ \mu e$ for the design HL-93 and the C4 trucks respectively. These values confirm the earlier conclusion that the C4 truck is more critical than the design truck even for uncracked conditions. Comparing the strains due to the legal trucks to the maximum recorded values implies that the current design guidelines are still conservative even assuming uncracked conditions. The reasons were discussed earlier and are related to the distribution width, the presence of the edge beam, and the estimation of the real load acting on the bridge.
5.9 Damage Identification of the East Bay Road Bridge

The installed health monitoring system will be also used as a tool to detect long term damage of the East Bay Road Bridge. The process is described as follow:

(a) The readings of all FOS sensors will be collected and stored. The maximum positive and negative strains among all sensors will be identified.

(b) A damage index for service conditions \((DI)_{\text{service}}\) that represents the damage condition of the bridge will be evaluated. The damage index is defined as follow:

\[
(DI)_{\text{service}} = \frac{\varepsilon_{\text{max}}}{\varepsilon_{\text{capacity}}}
\]

Where \(\varepsilon_{\text{max}}\) is the maximum recorded strain at time \(t\), and \(\varepsilon_{\text{capacity}}\) represents the maximum strain that an element can resist. Theoretically, the maximum service capacity should equal the steel yield strain of 1897 \(\mu \varepsilon\), however the current maintenance practice requires using a value of 0.85 \(\varepsilon_y = 1518 \mu \varepsilon\). The value of \((DI)_{\text{service}}\) is assumed to equal zero at the initial stage of the bridge. A schematic diagram of the expected shape of the time vs \((DI)_{\text{service}}\) plot is shown in Figure (5.6). The values for \((DI)_{\text{service}}\) have been computed for the bridge condition so far and are shown in red in Figure (5.4). The dotted line in Figure (5.4) shows the expected behavior over the life time of the bridge. The maximum value obtained at the end of the first year, however, is less than 0.03 due to the uncracked condition of the bridge. The author will continue to monitor the behavior of the bridge in collaboration with Hillsborough County, and construct the damage
function over the entire life cycle of the bridge. The data corresponding to the behavior of the first year are shown in Table (5.1) below.

**Table 5.1 Strain Progression with Respect to Time, \( \mu e \)**

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(c) A frame finite element model of the bridge will be developed and subjected to the AASHTO design truck. The stiffness coefficient \((ES)\) of the model will be tuned in order for the model to match with the maximum recorded strains, \(E\) being Young’s modulus and \(S\) the section modulus. Two values for the stiffness term \((ES)\) will be evaluated, namely a value for matching with maximum positive strains \((ES)^+\) and a value for negative strains \((ES)^-\). The critical of the two will be used for the calculations to follow. The initial stiffness value \((ES)_o\) which corresponds to the initial cracking condition will be documented. The stiffness coefficient \((ES)_t\), at time \(t\) will be evaluated and compared to the initial \((ES)_o\) in order to monitor the strength deterioration rate of the bridge.
Figure 5.6 Damage Index of the East Bay Road Bridge
CHAPTER 6. SUMMARY, CONCLUSIONS, AND FUTURE WORK

6.1 Research Planning

6.1.1 Laboratory Test and Field Investigation

The laboratory test was performed with project economy in mind. The literature review resulted in the evaluation of three sensors. (1) Fabry-Perot strain gauge, (2) Fiber Bragg grating optic strain gages and (3) long-gauge strain gauge.

Limited application and lack of suitable data acquisition make Fiber Bragg grating optic strain gauges and long-gauge strain gauge sensing systems a poor choice for the bridge load test instrumentation at this time.

These systems use two fibers, one as a transducer and one as a reference fiber. These systems are capable of multiplexing (e.g., sensors are used in series and only one fiber optic cable leads to data logger). Using one fiber optic cable presents a serious and potential risk of loosing all installed sensors.

On the other hand, Fabry-Perot Interferometer was found to be a suitable sensing system for this research. This system offered ease and simplicity in installation and operation. Fabry-Perot uses interferometry technique, a unique way of utilizing the light emitted from a white light source.
6.2 Laboratory Application of Sensors

The laboratory application of Fabry-Perot sensors was economical and easy. Installation of sensors on test specimen was easy, quick and clean. Connection to readout unit was simple and successful. The results of load test were closer to analytical values than to digital strain gauge. The versatility, ease of application and data collection and accuracy of collected data with this new smart sensing technology has rendered older conventional instrumentation obsolete.

6.3 Field Application of Sensors

The field application of sensors for East bay Road bridge proved to be as simple as the laboratory application. The successful bonding of the sensors was verified by a potable readout unit and the field test results were verified by analytical models as presented in Chapter 3.

6.4 Conclusions and Recommendations

6.4.1 Fine Tuning of LRFD Code

The literature review indicates the absence or the lack of much needed research study and field verification of AASHTO LRFD design specifications for concrete bridges. Results of field data, beam modeling and FEM have indicated over conservative design for the East Bay Road bridge. The results of field experiments have indicated that the LRFD design method has been expanded and diverted much beyond its intended purpose and technicality.
6.4.2 Cracked Section vs. Uncracked Section

The sensors readings as well as the visual observations suggest that the current condition of the East Bay Road bridge corresponds to uncracked behavior. The design of the bridge, however, is based on cracked analysis which resulted in over conservative cross sections. This conclusion, however, is based on the current observed behavior and may change as the bridge deteriorates with time.

6.4.3 Load Distribution Width (Code Tributary Width)

The recorded data along with finite element modeling confirmed that current specifications for distribution widths are conservative. This conclusion, however, is based on the observed uncracked condition of the bridge, and might change as the bridge deck starts cracking. There exists a need to develop more accurate criteria for distribution widths that better matches with observed behavior.

6.4.4 Discuss the Effect of Parapets and Traffic Barriers on Bridge Deck Stiffness

Analytical investigations as well as sensors readings showed that the traffic barrier wall has a considerable effect on the stiffness and load distribution of the East Bay Road Bridge. The current bridge design has ignored this effect, which contributed to the conservative behavior of the bridge. There is a need to
revisit the current design guidelines to account for the increase in stiffness due to the presence of the barrier wall.

6.5 Future Studies

6.5.1 Continuous Monitoring of the East Bay Road Bridge

The author will continue to monitor the behavior of the East Bay Road bridge in collaboration with Hillsborough County officials. The conclusions drawn on current data will be revisited based on the new data as they become available. It is expected that the bridge will eventually start cracking, which will be reflected in an increase of the sensors readings.

6.5.2 Damage Identification of the East Bay Road Bridge

The author proposed earlier a methodology in Chapter 5 to evaluate the structural health and damage condition of the East Bay Road Bridge through a damage index function. The author will continue to collect data and construct the entire damage function of the bridge. This damage function will serve as basis to accurately evaluate the real life time expectancy of the bridge. This study will help better understand the performance of similar bridge structures, and improve their maintenance process accordingly.

6.5.3 Weight-In-Motion (WIM) Systems

Future studies will aim at accurately evaluating the weight of trucks moving on the bridge in addition to the resulting strain readings. This will be
made possible through the use of Weight-in-Motion (WIM) systems. Weight-in-motion systems are reliable tools used across the nation to obtain the following information: axle weight of trucks and cars, axle spacing, and speed. The collected truck information will help better evaluate the collected sensors readings, and therefore better understand the bridge behavior under traffic loads.

6.5.4 Wireless Sensors

The technology for health monitoring of bridge structures is moving with a fast pace. While the sensors used for this project performed adequately, wireless technology offers additional features. In this case, sensors communicate wirelessly, which will eliminate the need for on-site cabling. Installation of such sensors might be more complex though, as they still need to be attached to an electric card, which will require additional care and innovation during construction. Furthermore, most of these sensors are battery operated, which renders long-term use impractical. Current research is undergoing, however, to solve this issue, using several innovative techniques. Future research should focus on the use of such advanced sensors and their applicability for bridge monitoring.

6.5.5 Estimate of Bridge Life Expectancy

Continuous monitoring of the bridge subject to traffic is essential to collect data for the condition, evaluation and damage assessment. This data can also be used to predict the useful life of the bridge. Theoretical life time expectancy of
the East Bay Road Bridge is 75 years. The author has reviewed design and construction documents for East Bay Road bridge and has not come across any information or references verifying 75 years predicted life expectancy for this structure. Continuous data collection, if formulated properly, will provide invaluable tool for societal and economical management of civil engineering infrastructure and will predict its normal true life time expectancy. The suggested formulation consists of the following variables: (1) initial material properties and strength at the time of construction, (2) collected data from nondestructive material testing and strength in every five years period, (3) FOS strain readings at the same time line. The difference between the values for each period can generate predicted increase in strain until it will reach the safe operating value at which time, the management can make an intelligent decision about the bridge.

6.5.6 Development of New Bridge Management Systems Using Remote Health Monitoring Techniques

It is the author’s hope that the current study becomes a starting point into development and implementation of new bridge maintenance systems that follows the present technological era. In this case, the new maintenance structure will rely on a centralized bridge management office where data gathering and data evaluation is performed. The current system for bridge maintenance requires engineers to make periodical checks to assess bridge damages. With the implementation of the Fiber Optic sensors, the ultimate goal would be to decrease the frequency of inspecting for bridge damages. The main
objective of the new system is to determine who is in charge of gathering data, analyzing data and taking the proper actions recommended by the data analysis. It is critical that the new system works efficiently to ensure public safety. It is imperative that the channel of communication and the management structure be in line with the new system so that data does not get overlooked or lost. The author hopes that Hillsborough County be the first to employ such an advanced system, and to work closely with their bridge management team to evaluate current procedures, propose new procedures and resolve any issues that might arise due to the implementation of the new technology. Such new methodologies will improve the safety of these bridges, improve the emergency response following possible failures, and minimize the impact of traffic delay due to possible bridge closure, resulting in millions of dollars of savings to the County.
REFERENCES


APPENDICES
References

Reference: C:\fdot_str\designexamples\flatslab\Ex2_FlatSlab\201DesignLds.mcd(R)

Description

This section provides the design for the flat slab superstructure.

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<td>B8. Summary of Reinforcement Provided</td>
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A. Input Variables

**Maximum positive moment and corresponding fatigue values**

- **Service**: $M_{\text{pos}} = 64.4 \text{ft-kip}$
- **Strength**: $M_{r,\text{pos}} = 100.9 \text{ft-kip}$
- **Fatigue**: $M_{\text{fatigue,pos}} = 46.7 \text{ft-kip}$
  - $M_{\text{range,pos}} = 16.1 \text{ft-kip}$
  - $M_{\text{min,pos}} = -1.6 \text{ft-kip}$

**Maximum negative moment and corresponding fatigue values**

- **Service**: $M_{\text{neg}} = 61.9 \text{ft-kip}$
- **Strength**: $M_{r,\text{neg}} = -93.2 \text{ft-kip}$
- **Fatigue**: $M_{\text{fatigue,neg}} = -50.7 \text{ft-kip}$
  - $M_{\text{range,neg}} = 148 \text{ft-kip}$
  - $M_{\text{min,neg}} = -13 \text{ft-kip}$
B. Moment Design

The design procedure consists of calculating the reinforcement required to satisfy the design moment, then checking this reinforcement against criteria for crack control, minimum reinforcement, maximum reinforcement, shrinkage and temperature reinforcement, and distribution of reinforcement. The procedure is the same for both positive and negative moment regions.

B1. Positive Moment Region Design - Flexural Resistance [LRFD 5.7.3.2]

Factored resistance \( M_r = \phi \cdot M_n \)

Nominal flexural resistance

\[
M_n = A_{ps} f_{ps} \left( d_p - \frac{a}{2} \right) + A_s f_y \left( d_s - \frac{a}{2} \right) - A_{s'} f_{y'} \left( d_{s'} - \frac{a}{2} \right) + 0.85 f_c (b - b_w) \beta_1 h_f \left( \frac{a}{2} - \frac{h_f}{2} \right)
\]

Simplifying the nominal flexural resistance

\[
M_n = A_s f_y \left( d_s - \frac{a}{2} \right) \quad \text{where} \quad a = \frac{A_s f_y}{0.85 f_c b}
\]

Using variables defined in this example.....

\[
M_r = \phi \cdot A_{s, pos} f_y \left[ d_s - \frac{1}{2} \left( \frac{A_{s, pos} f_y}{0.85 f_{c, slab} b} \right) \right]
\]

where

\[
M_{r, pos} = 106.9 \text{ ft kip}
\]

\[
f_{c, slab} = 4.5 \text{ ksi}
\]

\[
f_y = 60 \text{ ksi}
\]

\[
\phi = 0.9
\]

\[
t_{slab} = 18 \text{ in}
\]

\[
A_s
\]

\[
l_{slab} = h
\]
Initial assumption for area of steel required

Size of bar:..........................  
bar = 9 in

Proposed bar spacing:..................  
spacing ps = 6 in
Bar area: $A_{\text{bar}} = 1.000 \text{ in}^2$

Bar diameter: $\text{dia} = 1.128 \text{ in}$

Area of steel provided per foot of slab:

$A_{s,\text{pos}} = 2.00 \text{ in}^2$

$A_{s,\text{pos}} := \frac{A_{\text{bar}} \times 1 \text{ ft}}{\text{spacing}_{\text{pos}}}$

Distance from extreme compressive fiber to centroid of reinforcing steel:

$d_{s,\text{pos}} := t_{\text{slab}} - \text{coverslab} - \frac{\text{dia}}{2}$

$d_{s,\text{pos}} = 15.4 \text{ in}$

Solve the quadratic equation for the area of steel required:

Given $M_{r,\text{pos}} = \phi \cdot A_{s,\text{pos}} \cdot f_y \left[ d_{s,\text{pos}} - \frac{1}{2} \left( \frac{A_{s,\text{pos}} \cdot f_y}{0.85 \cdot f_{c,\text{slab}} b} \right) \right]$

Reinforcing steel required:

$A_{s,\text{reqd}} := \text{Find}(A_{s,\text{pos}})$

$A_{s,\text{reqd}} = 1.55 \text{ in}^2$

The area of steel provided, $A_{s,\text{pos}} = 2.00 \text{ in}^2$, should be greater than the area of steel required, $A_{s,\text{reqd}} = 1.55 \text{ in}^2$. If not, decrease the spacing of the reinforcement. Once $A_{s,\text{pos}}$ is greater than $A_{s,\text{reqd}}$, the proposed reinforcing is adequate for the design moments.

Moment capacity provided:

$M_{r,\text{positive, prov}} := \phi \cdot A_{s,\text{pos}} \cdot f_y \left[ d_{s,\text{pos}} - \frac{1}{2} \left( \frac{A_{s,\text{pos}} \cdot f_y}{0.85 \cdot f_{c,\text{slab}} b} \right) \right]$

$M_{r,\text{positive, prov}} = 127.2 \text{ ft-kip}$
B2. Negative Moment Region Design - Flexural Resistance [LRFD 5.7.3.2]

Variables:

\[ |M_{r,neg}| = 93.2 \text{ ft-kip} \]
\[ f_{c,\text{slab}} = 4.5 \text{ ksi} \]
\[ f_y = 60 \text{ ksi} \]
\[ \phi = 0.9 \]
\[ t_{\text{slab}} = 18 \text{ in} \]
\[ b = 1 \text{ ft} \]

Initial assumption for area of steel required

Size of bar ........................................

Proposed bar spacing ............................

\[ b_{\text{neg}} := 9\text{ in} \]

\[ \text{spacing}_{\text{neg}} := 6\text{ in} \]
Bar area

\[ A_{\text{bar.neg}} = 1.000 \text{ in}^2 \]

Bar diameter

\[ d_{\text{neg}} = 1.128 \text{ in} \]

Area of steel provided per foot of slab

\[ A_{s.neg} = 2.00 \text{ in}^2 \]

Distance from extreme compressive fiber to centroid of reinforcing steel

\[ d_{s.neg} = \left( -t_{\text{slab}} + \text{cover}_{\text{slab}} + \frac{d_{\text{neg}}}{2} \right) \]

Solve the quadratic equation for the area of steel required

\[ M_{r.neg} = \phi \cdot A_{s.neg} \cdot f_y \cdot \left[ d_{s.neg} + \frac{1}{2} \left( \frac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c,\text{slab}} \cdot b} \right) \right] \]

Reinforcing steel required

\[ A_{s.reqd} = \text{Find}(A_{s.neg}) \]

The area of steel provided, \( A_{s.neg} = 2.00 \text{ in}^2 \), should be greater than the area of steel required, \( A_{s.reqd} = 1.43 \text{ in}^2 \). If not, decrease the spacing of the reinforcement. Once \( A_{s.neg} \) is greater than \( A_{s.reqd} \), the proposed reinforcing is adequate for the design moments.

Moment capacity provided

\[ M_{r.negative.prov} = \phi \cdot A_{s.neg} \cdot f_y \cdot \left[ d_{s.neg} + \frac{1}{2} \left( \frac{A_{s.neg} \cdot f_y}{0.85 \cdot f_{c,\text{slab}} \cdot b} \right) \right] \]

\[ M_{r.negative.prov} = -127.2 \text{ ft-kip} \]
B3. Crack Control by Distribution Reinforcement [LRFD 5.7.3.4]

Concrete is subjected to cracking. Limiting the width of expected cracks under service conditions increases the longevity of the structure. Potential cracks can be minimized through proper placement of the reinforcement. The check for crack control requires that the actual stress in the reinforcement should not exceed the service limit state stress (LRFD 5.7.3.4). The stress equations emphasize bar spacing rather than crack widths.

Stress in the mild steel reinforcement at the service limit state.......................... \[ f_{sa} = \frac{z}{1} \leq 0.6f_y \]

\[ (d_w A)^{3/2} \]

Crack width parameter.................................................. \[ z = \begin{cases} \text{"moderate exposure"} & 170 \text{ kip/in} \\ \text{"severe exposure"} & 130 \text{ kip/in} \\ \text{"buried structures"} & 100 \text{ kip/in} \end{cases} \]
The environmental classifications for Florida designs do not match the classifications to select the crack width parameter. For this example, a "Slightly" or "Moderately" aggressive environment corresponds to "moderate exposure" and an "Extremely" aggressive environment corresponds to "severe exposure".

\[
\text{Environment}_{\text{super}} = \text{"Slightly" \ aggressive environment}
\]

\[
z := 170 \frac{\text{kip}}{\text{in}}
\]

**Positive Moment**

Distance from extreme tension fiber to center of closest bar (concrete cover need not exceed 2 in.).

\[
d_c := \min \left( t_{\text{slab}} - d_{s,\text{pos},2} - \text{in} + \frac{\text{dia}}{2} \right)
\]

\[
d_c = 2.564 \text{ in}
\]
Service moments
Live Load positive and negative moments were obtained from Live Load generator
Dead load moment was obtained from beam analysis

\[
\begin{pmatrix}
250.6 \\
170.8 \\
136
\end{pmatrix}
\text{kip-ft}
\]

positive live load moment
negative live load moment
dead load moment

cracked section analysis

\[ b_t = 11.874 \text{ft} \]
Tributary width for a single truck

\[ \text{spacing} = 6 \text{in} \]
Spacing of rebars for top and bottom

\[ d_{cov} = 2 \text{in} \]
Clear cover

\[ d_{s, \text{pos}} = 18 \text{in} - d_c - \frac{9}{8} \text{in} - \frac{1}{2} \]  
Depth to C.L of steel

\[ d_{s, \text{pos}} = 1.286 \text{ft} \]

Number of bars per design width of slab...

\[ n_{\text{bar}} := \frac{b}{\text{spacing}_{\text{pos}}} \]
\[ n_{\text{bar}} = 23.748 \]

\[ A_{\text{pos, max}} := n_{\text{bar}} \cdot 1 \text{in}^2 \]
Total steel area in the tributary width

Effective tension area of concrete surrounding the flexural tension reinforcement...

\[ A := \frac{(b)(2d_c)}{n_{\text{bar}}} \]
\[ A = 24.0 \text{ in}^2 \]
Service limit state stress in reinforcement:

\[ f_{sa} := \min \left[ \frac{z}{0.6f_y}, \frac{1}{\left(d_c - A\right)^3} \right] \quad f_{sa} = 36.0 \text{ ksi} \]

The neutral axis of the section must be determined to determine the actual stress in the reinforcement. This process is iterative, so an initial assumption of the neutral axis must be made.

\[ \delta_c := 4.8 \text{ in} \]

Given

\[ \frac{1}{2} b \cdot x^2 = \frac{E_s}{E_c, \text{slab}} \cdot A_{s, \text{pos}} \left( d_{s, \text{pos}} - x \right) \]

\[ y_{na, \text{pos}} := \text{Find}(x) \]

\[ y_{na, \text{pos}} = 5.3 \text{ in} \]

Compare the calculated neutral axis \( y_{na, \text{pos}} \) with the initial assumption \( x \). If the values are not equal, adjust

\[ x = 4.8 \text{ in} \text{ to equal } y_{na, \text{pos}} = 5.3 \text{ in}. \]

Tensile force in the reinforcing steel due to service limit state moment:

\[ T_s := \frac{M_{\text{pos}}}{d - \frac{y_{na, \text{pos}}}{3}} \quad T_s = \begin{pmatrix} 220 \\ 150 \\ 119.4 \end{pmatrix} \text{ kip} \]
\[ E_{\text{steel}} = \frac{29000 \text{ kip}}{\text{in}^2} \]

Modulus of elasticity of steel

\[ E_{c,\text{slab}} = 3.475 \times 10^3 \text{ ksi} \]

Modulus of elasticity of concrete
Actual stress and strain in the reinforcing steel
due to service limit state moment.

\[ f_{s,\text{actual}} = \frac{T_s}{A_{s,\text{pos}}} \]
\[ f_{s,\text{actual}} = \begin{pmatrix} 9.3 \\ 6.3 \\ 5.0 \end{pmatrix} \text{ksi} \]

Positive live load moment

Negative live load moment

Dead load moment

Uncracked section analysis

\[ y := \frac{b \cdot (18\text{in})^2}{2} + \frac{E_s}{E_{c,\text{slab}}} \left( d_c + \frac{9}{8} \text{ in} \right) A_{s,\text{pos}} \]

\[ y = 8.538 \text{ in} \quad \text{C.G. of uncracked Section} \]

\[ I := b \cdot \frac{(18\text{in})^3}{12} + b \cdot 18\text{in} \left( \frac{18\text{in}}{2} - y \right)^2 + A_{s,\text{pos}} \frac{E_s}{E_{c,\text{slab}}} \left[ y - \left( d_c + \frac{9}{8} \text{ in} \right) \right]^2 \]

Moment of Inertia of Uncracked Section

\[ \sigma := \frac{M_{\text{pos}}}{I} \left( \frac{y - (d_c + \frac{9}{8} \text{ in})}{1} \right) \frac{E_s}{E_{c,\text{slab}}} \]

\[ \sigma = \begin{pmatrix} 1.951 \\ 1.33 \\ 1.059 \end{pmatrix} \text{ksi} \]

Steel Stress for Uncracked Section
Steel Strain of Uncracked Section in Microstrains

\[ e := \frac{\sigma \cdot 10^6}{E_s} \]

positive live load moment

\[ e = \begin{cases} 
67.275 \\
45.852 \\
36.51 
\end{cases} \]

negative live load moment
dead load moment

The service limit state stress in the reinforcement should be greater than the actual stress due to the service limit state moment.

\[ LRFD_{5.7.3.3.4a} := \begin{cases} 
"OK, crack control for +M is satisfied" & \text{if } f_{s,\text{actual}} \leq f_{s,a} \\
"NG, crack control for +M not satisfied, provide more reinforcement" & \text{otherwise} 
\end{cases} \]

Negative Moment

Distance from extreme tension fiber to center of closest bar (concrete cover need not exceed 2 in.)

\[ d_{al} = \min \left( t_{slab} + d_{s,\text{neg}}, 2\text{-in} + \frac{d_{\text{neg}}}{2} \right) \]
\[ d_c = 2.564 \text{ in} \]

Number of bars per design width of slab:

\[ n_{\text{bar}} = \frac{b}{\text{spacing}_{\text{neg}}} \]

Effective tension area of concrete surrounding the flexural tension reinforcement:

\[ A = \frac{(b) \cdot (2 \cdot d_c)}{n_{\text{bar}}} \]

Service limit state stress in reinforcement:

\[ f_{\text{sa}} = \min \left[ \frac{z}{1 - 0.6 \cdot t_2}, \frac{0.6 \cdot t_2}{(d_c \cdot A)^{\frac{3}{2}}} \right] \]

The neutral axis of the section must be determined to determine the actual stress in the reinforcement. This process is iterative, so an initial assumption of the neutral axis must be made.

\[ A = 4.8 \text{ in}^2 \]

Given

\[ \frac{1 - b \cdot x^2}{2} = \frac{E_s}{E_{c, \text{slab}}} \cdot A_{s, \text{neg}} \left( -d_{s, \text{neg}} - x \right) \]

\[ x_{n.a, \text{neg}} := \text{Find}(x) \]

\[ x_{n.a, \text{neg}} = 1.788 \text{ in} \]

Compare the calculated neutral axis \( x_{n.a} \) with the initial assumption \( x \). If the values are not equal, adjust

\[ x = 4.8 \text{ in} \] to equal \[ x_{n.a, \text{neg}} = 1.8 \text{ in} \]
Tensile force in the reinforcing steel due to service limit state moment.................

\[ T_s := \frac{M_{neg}}{d_{s.neg} + \frac{s_{na.neg}}{3}} \]

\[ T_s = 1 \text{ kip} \]

Actual stress in the reinforcing steel due to service limit state moment.................

\[ f_{s.actual} := \frac{T_s}{A_{s.neg}} \]

\[ f_{s.actual} = 1 \text{ ksi} \]

The service limit state stress in the reinforcement should be greater than the actual stress due to the service limit state moment.

\[
\text{LRFD}_{5.7.3.3.4b} := \begin{cases} 
\text{"OK, crack control for } -M \text{ is satisfied" if } f_{s.actual} \leq f_{sa} \\
\text{"NG, crack control for } -M \text{ not satisfied, provide more reinforcement" otherwise}
\end{cases}
\]

B4. Limits for Reinforcement [LRFD 5.7.3.3]

Maximum Reinforcement

The maximum reinforcement requirements ensure the section has sufficient ductility and is not overreinforced. The greater reinforcement from the positive and negative moment sections is checked.

Area of steel provided..............................

\[ A_{s.pos} = 23.75 \text{in}^2 \]

\[ A_{s.neg} = 2.00 \text{in}^2 \]
Stress block factor

$$\beta_1 := \max \left[ 0.35 - 0.05 \cdot \left( \frac{f_{c,\text{slab}} - 4000 \text{ psi}}{1000 \text{ psi}} \right) , 0.65 \right]$$

$$\beta_1 = 0.825$$

Distance from extreme compression fiber to the neutral axis of section

$$e_{\text{pos}} = \frac{A_{s,\text{pos}} f_y}{0.85 f_{c,\text{slab}} \beta_1 b}$$

$$e_{\text{neg}} = \frac{A_{s,\text{neg}} f_y}{0.85 f_{c,\text{slab}} \beta_1 b}$$

Effective depth from extreme compression fiber to centroid of the tensile reinforcement.

$$d_e = \frac{A_s f_{ps} d_p + A_{s,\text{pos}} f_y d_s}{A_{ps} f_{ps} + A_{s,\text{pos}} f_y}$$

Simplifying for this example

$$d_{e,\text{pos}} = d_{s,\text{pos}} \quad \text{and} \quad d_{e,\text{neg}} = -d_{s,\text{neg}}$$

$$d_{e,\text{pos}} = 15.4 \text{ in}$$

$$d_{e,\text{neg}} = 15.4 \text{ in}$$
The $\frac{c_{\text{pos}}}{d_{e,\text{pos}}} = 0.205$ ratio should be less than 0.42 to satisfy maximum reinforcement requirements.

\[ \text{LRFD}_{5.7.3.3.1} = \begin{cases} \text{"OK, maximum reinforcement in }+M\text{ region"} & \text{if } \frac{c_{\text{pos}}}{d_{e,\text{pos}}} \leq 0.42 \\ \text{"NG, section is over-reinforced in }+M\text{ region, see LRFD eq. C5.7.3.1-1"} & \text{otherwise} \end{cases} \]

The $\frac{c_{\text{neg}}}{d_{e,\text{neg}}} = 0.017$ ratio should be less than 0.42 to satisfy maximum reinforcement requirements.

\[ \text{LRFD}_{5.7.3.3.1} = \begin{cases} \text{"OK, maximum reinforcement in }-M\text{ region"} & \text{if } \frac{c_{\text{neg}}}{d_{e,\text{neg}}} \leq 0.42 \\ \text{"NG, section is over-reinforced in }-M\text{ region, see LRFD eq. C5.7.3.1-1"} & \text{otherwise} \end{cases} \]

Minimum Reinforcement

The minimum reinforcement requirements ensure the moment capacity provided is at least 1.2 times greater than the cracking moment.

Modulus of Rupture................................. $f_r := 0.24 \cdot \sqrt[6]{f_{c,\text{slab}}} \text{ksi}$

\[ f_r = 509.1 \text{ psi} \]

Section modulus................................. $S := \frac{b \cdot l_{\text{slab}}^2}{6}$

\[ S = 7.7 \times 10^3 \text{ in}^3 \]
Cracking moment

\[ M_{cr} := f'_r S \]

\[ M_{cr} = 326.4 \text{ kip-ft} \]

Required flexural resistance (+M)

\[ M_{r,reqd} := \min(1.2 \cdot M_{cr}, 133\% \cdot M_{r,pos}) \]

\[ M_{r,reqd} = 134.2 \text{ ft-kip} \]

Check that the capacity provided, \( M_{r,positive.prov} = 127.2 \text{ ft-kip} \), exceeds minimum requirements,

\[ M_{r,reqd} = 134.2 \text{ ft-kip} \]

\[
LRFD_{5.7.3.3.2} := \begin{cases} 
"OK, minimum reinforcement for positive moment is satisfied" & \text{if } M_{r,positive.prov} \geq M_{r,reqd} \\
"NG, reinforcement for positive moment is less than minimum" & \text{otherwise} 
\end{cases}
\]

\[
LRFD_{5.7.3.3.2} = "NG, reinforcement for positive moment is less than minimum"
\]
Required flexural resistance (-M)............

\[ M_{r,reqd} = 124.0 \text{ ft-kip} \]

Check that the capacity provided, \( M_{r,\text{negative,prov}} = -127.2 \text{ ft-kip} \), exceeds minimum requirements,

\[ M_{r,reqd} = 124 \text{ ft-kip} \]

\[
\text{LRFD}_{5.7.3.2.2} = \begin{cases} 
"OK, minimum reinforcement for negative moment is satisfied" & \text{if } M_{r,\text{positive,prov}} \geq M_{r,reqd} \\
"NG, reinforcement for negative moment is less than minimum" & \text{otherwise}
\end{cases}
\]

\[
\text{LRFD}_{5.7.3.2} = "OK, minimum reinforcement for negative moment is satisfied" 
\]

B5. Shrinkage and Temperature Reinforcement [LRFD 5.10.8.2]

Shrinkage and temperature reinforcement provided

Size of bar ("4" "5" "6")............

\[ \text{bar}_{st} := "5" \]

Bar spacing........................................... \[ \text{bar}_{spa,st} := 9 \text{ in} \]

Bar area............................................ \[ A_{\text{bar}} = 0.31 \text{ in}^2 \]

Bar diameter....................................... \[ \text{dia} = 0.625 \text{ in} \]

Gross area of section........................... \[ A_g = 2.6 \times 10^3 \text{ in}^2 \]

\[ A_g := b \cdot t_{\text{slab}} \]
Minimum area of shrinkage and temperature reinforcement

\[ A_{ST} := \frac{0.11 \cdot \text{ksi} \cdot A_g}{f_y} \]

\[ A_{ST} = 4.70 \text{ in}^2 \]

Maximum spacing for shrinkage and temperature reinforcement

\[ \text{spacing}_{ST} := \min \left( \frac{b}{\frac{A_{ST}}{A_{bar}}}, 3 \cdot t_{slab}, 18 \cdot \text{in} \right) \]

\[ \text{spacing}_{ST} = 9.4 \text{in} \]

The bar spacing should be less than the maximum spacing for shrinkage and temperature reinforcement

\[ \text{LRFD}_{5.7.10.8} := \begin{cases} 
\text{"OK, minimum shrinkage and temperature requirements"} & \text{if } \frac{\text{bar} \cdot \text{spa}_{,st}}{A_{ST}} \leq \text{spacing}_{ST} \\
\text{"NG, minimum shrinkage and temperature requirements"} & \text{otherwise} 
\end{cases} \]

\[ \text{LRFD}_{5.7.10.8} = \text{"OK, minimum shrinkage and temperature requirements"} \]

Transverse distribution reinforcement shall be placed in the bottom of the slab. The amount to place is based on a percentage of the longitudinal main steel.

Distribution reinforcement provided

Size of bar ("A", "5", "6") …………………… bar\_dist := "5"

Bar spacing…………………………… bar\_spa\_dist := 12\_in

Bar area……………………………… A_{bar} = 0.31\text{ in}^2

Bar diameter…………………………… dia = 0.625\text{ in}

The area for secondary reinforcement should not exceed 50% of the area for primary reinforcement……………………………

\%A_{Steel} := \min \left( \frac{100}{L_{span} / \text{ft}}, 50\% \right)

Required area for secondary reinforcement

A_{s, DistR} \geq A_{s, pos} \%A_{Steel}

Maximum spacing for secondary reinforcement……………………………

MaxSpacing_{DistR} := \frac{b}{\left( \frac{A_{s, DistR}}{A_{bar}} \right)}

MaxSpacing_{DistR} = 11.0\text{ in}
The bar spacing should not exceed the maximum spacing for secondary reinforcement.

\[
LRFD_{5.14.4} := \begin{cases} 
    "OK, distribution reinforcement requirements" & \text{if } \text{bar}_{\text{spa}.\text{dist}} \leq \text{MaxSpacing}_{\text{DistR}} \\
    "NG, distribution reinforcement requirements" & \text{otherwise}
\end{cases}
\]

\[
LRFD_{5.14.4} = "NG, distribution reinforcement requirements"
\]
B7. Fatigue Limit State [LRFD 5.5.3]

The section properties for fatigue shall be based on cracked sections where the sum of stresses due to unfactored permanent loads and 1.5 times the fatigue load is tensile and exceeds $0.095 \sqrt{f_c}$.

Allowable tensile stress for fatigue ..................

$$f_{\text{tensile}} = 0.202 \text{ ksi}$$

Positive Moment Region

Stress due to positive moment ......................

$$f_{\text{fatigue.pos}} = \frac{M_{\text{fatigue.pos}}}{S}$$

$$f_{\text{fatigue.pos}} = 0.073 \text{ ksi}$$

Fatigue section :=

- "Use Cracked section" if $f_{\text{fatigue.pos}} > f_{\text{tensile}}$
- "Use Uncracked section" otherwise

Fatigue section = "Use Uncracked section"

Minimum stress in reinforcement due to minimum live load ......................

$$f_{\text{min}} := \frac{M_{\text{min.pos}}}{A_{\text{s.pos}} \left( d_{\text{s.pos}} - \frac{x_{\text{na.pos}}}{3} \right)}$$

$$f_{\text{min}} = -0.059 \text{ ksi}$$

Ratio of $r/h$ is taken as $r/h = 0.3$, therefore the allowable stress range is given by......

$$f_{\text{allow}} := (21 \cdot \text{ksi} - 0.33 \cdot f_{\text{min}}) + 8 \cdot \text{ksi} \cdot (r/h)$$

$$f_{\text{allow}} = 23.42 \text{ ksi}$$
Actual stress range.

\[ f_t := \frac{M_{\text{range.pos}}}{A_{s,\text{pos}} \left( d_{s,\text{pos}} - \frac{x_{n_a,\text{pos}}}{3} \right)} \]

\( f_t = 0.595 \text{ ksi} \)

\[ \text{LRFD}_{5.5.3.2} := \begin{cases} 
"\text{OK, fatigue stress range requirement for +M region}" & \text{if } f_t \leq f_{t,\text{allow}} \\
"\text{NG, fatigue stress range requirements for +M region}" & \text{otherwise} 
\end{cases} \]

\[ \text{LRFD}_{5.5.3.2} = "\text{OK, fatigue stress range requirement for +M region}" \]
Negative Moment Region

Stress due to negative moment

\[ f_{\text{fatigue.neg}} = \frac{|M_{\text{fatigue.neg}}|}{S} \]

\[ f_{\text{fatigue.neg}} = 0.079 \text{ ksi} \]

Fatigue section:
- "Use Cracked section" if \( f_{\text{fatigue.neg}} > f_{\text{tensile}} \)
- "Use Uncracked section" otherwise

Fatigue section = "Use Uncracked section"

Minimum stress in reinforcement due to minimum live load

\[ f_{\text{min}} = \frac{M_{\text{min.neg}}}{A_{\text{s.neg}} \left( d_{\text{s.neg}} + \frac{x_{\text{na.neg}}}{3} \right)} \]

\[ f_{\text{min}} = 5.256 \text{ ksi} \]

Ratio of \( r/h \) is taken as 0.3, therefore the allowable stress range is given by

\[ f_{\text{allow}} = 21.665 \text{ ksi} \]

Actual stress range

\[ f = 59.839 \text{ ksi} \]

LRFD:\[5.5.2.2\] =
- "OK, fatigue stress range requirements for \(-M\) region" if \( f \leq f_{\text{allow}} \)
- "NG, fatigue stress range requirements for \(-M\) region" otherwise

LRFD:\[5.5.3.2\] = "NG, fatigue stress range requirements for \(-M\) region"
B8. Summary of Reinforcement Provided

Main reinforcing
  Top bar size (-M) \( \text{bar}_\text{neg} = "9" \)
  Top spacing \( \text{spacing}_\text{neg} = 6.0 \text{ in} \)
  Bottom bar size (+M) \( \text{bar} = "9" \)
  Bottom spacing \( \text{spacing}_\text{pos} = 6.0 \text{ in} \)

Shrinkage and temperature reinforcing
  Bar size \( \text{bar}_{\text{st}} = "5" \)
  Bottom spacing \( \text{bar}_{\text{spa.st}} = 9.0 \text{ in} \)
  \( \text{LRFD}_{5.7.10.8} = "OK, minimum shrinkage and temperature requirements" \)

Longitudinal Distribution reinforcing
  Bar size \( \text{bar}_{\text{dist}} = "5" \)
  Bottom spacing \( \text{bar}_{\text{spa.dist}} = 12.0 \text{ in} \)
  \( \text{LRFD}_{5.14.4} = "NG, distribution reinforcement requirements" \)
Ebrahim Mehrani was born in Abadan, Iran. He received his High school diploma in math major in 1962 from a public school in Abadan and attended school of Architect in Tehran. He was later admitted to Abadan College of petroleum engineering. He worked in the design and construction industry for the next few years. In Spring of 1974, he was admitted to the University of South Florida, where he received his B.S. and M.S. degrees in Structural Engineering. He worked in the private sectors before accepting the position in Hillsborough County as “The County Structural Engineer”. His duties within this capacity included the design and construction of the new county bridges, and the inspection of the county existing bridges. He is currently the owner and president of DCI Solutions, Inc. in Tampa, Florida, a structural design-build consulting firm that specializes in design and construction of residential and commercial buildings, and inspection of bridges. He has over 30 years of experience in the field of structural design and construction of buildings and bridges. He is a registered professional engineer and a special inspector in the state of Florida, and both a state certified general contractor and a state certified bridge inspector. He has published M.S. thesis, two conference papers, one to appear in ASCE 2006 and one in ACI Spring Convention 2006. He has submitted two journals to ASCE for publication.