TRIBHUVAN UNIVERSITY

INSTITUTE OF ENGINEERING

PULCHOWK CAMPUS

DEPARTMENT OF CIVIL ENGINEERING

M .Sc. Program in Structural Engineering

Thesis no: SS-00112

EVALUATION OF DAMAGED PRESTRESSED CONCRETE BRIDGE GIRDER AND IT'S RETROFIT

ANIL KUMAR SHRESTHA

(Final Thesis Draft)

IOE, Pulchowk, February 2010

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A thesis submitted by
ANIL KUMAR SHRESTHA

In partial fulfillment of the requirement for the degree of

MASTER OF SCIENCE
IN STRUCTURAL ENGINEERING

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CERTIFICATE

This is to certify that the work contained in this thesis entitled "**EVALUATION OF DAMAGED PRESTRESSED CONCRETE BRIDGE GIRDER AND IT'S RETROFIT**", in partial fulfillment of the requirement for the degree of Master of Science in Structural Engineering, as a record of research work, has been carried out by **Mr. Anil Kumar Shrestha** (062 / MSS / r / 101) under my supervision and guidance in the Institute of Engineering, Pulchowk Campus, Lalitpur, Nepal. The work embodied in this thesis has not been submitted elsewhere for a degree.

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ABSTRACT

Supervisor:- Mr Jagat kumar Shrestha Student:- Anil Kumar Shrestha
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The Highway is most important mode of transportation infrastructure facility used in our country. Nepal, being mountainous terrain to plain terrain with plenty of rivers and streams, construction of cheapest means of transportation like highway becomes difficult. So, number of bridges required to cross the rivers and streams for highway. Prestressed concrete is a method for overcoming the concrete's natural weakness in tension. As a result of both accidental and intentional events in connection with important structures all over the world, explosive loads have received considerable attention in recent years. In my thesis I had selected Surai-Naka prestressed concrete bridge as a case study, which has been blast by terrorist few year back and some part of it has been damaged. This Surai Naka prestressed concrete bridge is very important, as it is only one bridge for East West highway of Nepal. So using computer application, we can evaluate the strength or say capacity of this bridge so that developing country like Nepal can save this important bridge depending on its capacity.

During this research work , Field survey was carried out for the damage condition of bridge. I observed that anchorage block of left main girder was damaged and two number of tendon was seen totally damaged and will not be workable. One tendon was slip about 80 mm inside. Using SAP (version 10), 3-D model was prepared having solid element. Shell element and frame element for lane load. Class A and 10 wheeled truck load is taken for loading which is in real practice. After analyzing the model with normal

and damaged conditions, different stress values are obtained. From this value this double lane bridge was seen to allow for single traffic only if not retrofitted. While after retrofitting with CFRP, this Bridge regains its original strength so that this bridge got new life and can play important role in national east west highway.

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1. INTRODUCTION

1.1General

Now a day, new technology has been developed as a demand of time. FRP (Fiber Reinforced polymer) technique for the retrofit of concrete has been started since decade. Specially existing and old structure which need strengthening as per new requirement, which have got no old design data, becomes possible with this FRP technique and can be make serviceable for further more years. Also for the damaged structure it becomes costly and almost impossible, especially in our country, to replace or totally dismantle existing structure by new one, so in this situation, FRP is very much acceptable for us. These days Fiber reinforced polymer (FRP), which is light in weight, having higher tensile strength has in increased in retrofitting of various steel and concrete structure all over the world. Hence, this thesis is focused on the damaged bridge. Its residual capacity is calculated and possible retrofit for gaining original capacity of bridge is done.

Nepal, being mountainous terrain to plain terrain with plenty of rivers and streams, construction of cheapest means of transportation like highway becomes difficult. So, number of bridges required to cross the rivers and streams for highway. Depending upon span and cost bridges may be of Steel, RCC or Prestressed concrete.

Prestressed concrete is a method for overcoming the concrete's natural weakness in tension. It can be used to produce beams, floors or bridges with a longer span than is practical with ordinary reinforced concrete. Prestressing tendons (generally of high tensile steel cable or rods) are used to provide a clamping load which produces a compressive stress that offsets the tensile stress that the concrete compression member would otherwise experience due to a bending load. Traditional reinforced concrete is based on the use of steel reinforcement bars inside poured concrete.

Prestressed concrete is accomplished when two stressed materials are joined in such a manner that force acting in one is balanced by an opposite force in the other. Thus in prestressed concrete, a tensile force applied to steel tendons embedded in the concrete generates as compressive force in the concrete. The aim for prestressing concrete is to enable it to withstand tensile stresses, as is weak in tension which is approximately one tenth of its compressive strength.

As a result of both accidental and intentional events in connection with important structures all over the world, explosive loads have received considerable attention in recent years. The activity related to terrorist attacks has increased, and unfortunately, the present tendency suggests that it will be even larger in the future. Bridge engineers, however, have not typically considered security in the design process, and most of the current state of knowledge of the design of structures subjected to blast effects is based on the performance of buildings rather than bridges.

In my thesis I select the Surai Naka prestressed Concrete Bridge, located 80 km west to Butwol, for evaluation on its strength as it has been damaged by blast. Due to developing country, we are not able to replace the existing damaged bridge by new one so we need the study for its residual capacity and possibility of retrofit.

1.2 Problem Statement:-

Due to the terrorist activities in Nepal and rest of other countries, infrastructures like buildings, bridges dams etc are unsafe from that violence. In my thesis I had selected Surai-Naka prestressed concrete bridge as a case study, which has been blast by terrorist few year back and some part of it has been damaged. Almost in five locations, bomb has been blasted. Bearing of the girder has been damaged. Blast at support has damage three tendons and anchorage block, two of three was assumed unworkable and one was assumed 80 to 100 mm slip. This Surai Naka prestressed concrete bridge is very important, as it is only one bridge for East West highway of

Nepal. So using computer application, we can evaluate the strength or say capacity of this bridge so that developing country like Nepal can save this important bridge depending on its capacity.

Design detail of this bridge is not available as this was constructed by Indian government. This Bridge contains two main girders and seven cross girder. Eleven numbers of prestressing tendons are used. Due to this damage, Load carrying capacity suddenly has been reduced so, investigation is required and need possible retrofit.

Fig. 1.1 Damage due to Bomb Blast

Fig. 1.2 Damage at support

1.3 Objective :-

Surai-Naka Prestressed concrete bridge is taken as an example for the study, damage evaluation and necessary retrofit. This bridge has got no any design data, so its evaluation has been done and its residual strength is calculated.

Overall Objective:

Bridge section is selected considering a case of Surai Naka Bridge. Variation in strength of bridge, due to increase in slip value after damage is carried out. After damage of some tendons, residual strength of bridge, to carry the vehicle load is analyzed. Since, we have no design data of this bridge, the original strength is carried out with all normal cases. Use of CFRP for strengthening the bridge is only one solution.

Specific Objective:

1. Variation in strength is determined after damage condition.
2. To predict the residual capacity of bridge after slip of some tendon.
3. Strengthening damaged bridge to restore its original capacity.

1.4 Methodology:-

Design data was not available so, field visit was done to know the design parameters of the bridge. Sections of main girder, cross girder, deck slab was taken. Tendons were observed and we found tendons at anchorage block were damage (Fig2). Non Destructive test (Schmidt hammer test) was performed to know the grade of concrete.

Analytical software SAP was used. Discretization of bridge model was done as follows. Main Girder was modeled as 3-D solid element. Cross girder and Deck Slab was modeled as thin slab of 200mm thickness. Shell element for FRP. Frame element was used as track for wheel to apply the vehicle wheel load so that it distributed the wheel load to the solid nodes. So that stress at nodes can be calculate.

Tendon element as load is used which is embedded inside the main girder. Horizontal and vertical profile of tendon is assumed and applied using "**Standard Plans for Highway bridges" Volume-II**. SAP (version 10) has got tendon element which automatically discritize tendon to bar element.

1.5 Scope and limitations

This Surai Naka Prestressed concrete bridge is very much important for our country. With out this bridge, East-west highway will be close in rainy season that will break the link between various parts of our country. Since Nepal government has no much budget to dismantle the bridge and replace new one so why the strength evaluation of this bridge is required. In this thesis, the residual strength of the damaged bridge is analyzed so that the allowable vehicle load can be predicted. To get the original strength we can apply FRP as retrofit. Full load vehicle for classA and 10 wheeled truck load, which is now- a -days common in national highway is applied as vehicle load.

2. LITERATURE REVIEW

Literatures, about effects that occurred by Blast, tendon profiles in prestressed concrete according to Indian standard was reviewed. For the remedy of the damaged Prestressed Concrete girder, to obtain the required capacity of the structure, epoxy resins and Carbon FRP was selected so some literatures about these materials were also studied. Schmidt hammer test was done to predict the strength of concrete on site. FRP lamination of the Bridge was done to gain the full required strength so by using SAP, I had verified the Calculation of FRP laminate that has been provided to retrofit. SuraiNaka Bridge was studied in site, its dimensions, Number of main girders, cross girders are checked. Damaged condition of anchorage block, slabs tendons are checked.

2.1. Analysis and Design of Critical Bridges Subjected to Blast Loads

J. Structural Engineering Volume 131, Issue 8, pp. 1243-1255 (August 2005)

David G. Winget, Kirk A. Marchand, Eric B.Williamson,

Blast-resistant design has traditionally been considered only for essential government buildings, military structures, and petrochemical facilities. Recent terrorist threats to bridges have demonstrated the need to evaluate the vulnerability of our transportation infrastructure. Bridge engineers, however, have not typically considered security in the design process, and most of the current state of knowledge of the design of structures subjected to blast effects is based on the performance of buildings rather than bridges. This paper summarizes the results of ongoing research to develop performance-based blast design standards tailored specifically for bridges. Based on best practices obtained from an international literature review, the paper briefly discusses the incorporation of physical security and site layout principles into the design process. It then discusses the potential effects of blast loads on bridges and provides structural design and retrofit solutions to counter these effects.

Results from the current study have shown that bridge geometry can significantly affect the blast loads that develop below the deck. For bridges with deep girders, confinement effects can greatly enhance the blast loads acting on the girders and tops of the piers and in some cases may result in more damage than an explosion occurring on top of the deck. The clearance can also have a large impact on the results, as increasing the distance from the explosion to the deck can result in more damage to the girders due to the formation of a Mach front. However, higher clearances result in lower average loads on the piers due to the larger volume of space less confinement under the bridge and the increased average stand off distance to a given point on the pier. Explosions occurring near sloped abutments could possibly result in more damage than an explosion at mid span due to the confinement effects at the abutments. Finally, round columns will experience lower loads due to the increased angle of incidence from the curved surface. Design and retrofit options for girders include the use of Fiber-reinforced polymers FRPs, additional steel reinforcement on both the top for uplift and bottom faces, using blow out panels on the decks to help vent loads, and establishment of minimum dimensions, minimum material strengths, and reinforcement configurations. In addition, to prevent a span collapse, the girders and deck can be restrained at the supports with steel cables, or hinge restrainers can be used to hold the deck to the columns, which is currently being done in seismically active regions CALTRANS 2002. In addition, abutment seat sizes can be increased or hinge seat extensions can be used under expansion joints. Smaller piers are usually controlled by breaching failures and will require either standoff barriers or steel/arm or jackets. Although standoff seems to provide the best solution, it can be difficult to achieve for most bridges. Additionally, standoffs greater than 6m 20ft provide only a modest increase in level of protection from most truck bombs and for most bridge geometries. Larger piers or those that have been retrofitted with steel jackets tend to experience diagonal shear failures at the supports. Therefore, additional shear reinforcement may need to be provided to ensure the formation of a plastic hinge mechanism. Other design and retrofit options for the piers include lateral bracing and establishment of minimum pier diameters and reinforcement. Additional research is needed to further develop the proposed blast-resistant design guidelines for critical bridges. In addition, research is needed to investigate technology solutions to enhance physical security, improve structural response, or mitigate the consequences of an attack. Through the use of simple models similar to those presented in this paper, we can gain a better understanding of blast effects on bridge components and prioritize our efforts for experimental studies of the most promising retrofit solutions. The goal of the research is to investigate economical, unobtrusive, and effective methods to mitigate the risk of terrorist attacks against critical bridges. Based on the findings, the AASHTO Bridge LRFD could be modified to provide operational and structural guidelines for improving bridge security.

2.2 STANDARD PLANS FOR HIGHWAY BRIDGES VOL II

Prestressed concrete Beams & R.C.C Slab type Superstructure

The Indian Roads Congress

This standard had provided the drawings In this standard plans various drawings for various number of tendons, are provided which is of great use for adoption in field. Since I was not able to get the design of this Surai Naka prestressed concrete bridge, this standard plans was very much useful to know the possible arrangement of tendons.

2.3 STRENGTHENING OF IMPACT-DAMAGED BRIDGE GIRDER USING FRP LAMINATES

A. Nanni, P. C. Huang, and J. G. Tumialan

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The exterior prestressed concrete (PC) girder of Bridge A10062, located at the interchange of Interstates 44 and 270 in St. Louis County, Missouri, USA, was impact-damaged by an over height truck. Removal of the loose concrete showed that two prestressing tendons were fractured due to the impact. This resulted in approximately 10% reduction in flexural moment capacity. There has been limited research on the repair of PC bridge girders damaged by vehicular impact. Due to the repetitive nature of highway loading, repair methods such as internal strand splices and external post-tensioning were found to be questionable because they could not restore the ultimate strength to the damaged member. In this case study, it was decided to use carbon FRP (CFRP) laminates to restore the original structural capacity of the girder. It was demonstrated CFRP bonded reinforcement could be an effective repair technique in terms of installation as well as design. If the present trend in growing availability of FRP materials and design information were to continue, a sharp increase in FRP application could be forecast.

2.4 RC Beams and Slabs externally reinforced with Fiber Reinforced Plastic (FRP) panels

C. A. Ross, L. C. Muszynski, D. M. Jerome, J. W. Tedesco, R. L. Sierakowski

Considerable experimental and analytical studies concerning the external reinforcement of RC beams and slabs using fiber reinforced plastic (FRP) strips and panels have been accomplished in the past ten to twelve years. This paper will review the pertinent work in this area and in particular will review and present both experimental and analytical studies conducted by the United States Air Force at Wright Laboratory's Pavements and Facilities Section (WL/FIVCO), Tyndall Air Force Base, Florida.

Both experimental and analytical studies at WL/FIVCO have shown that RC beams and slabs retrofitted with carbon fiber reinforced plastic (CFRP) and Aramid fiber reinforced plastic (AFRP) show considerable flexural strength increases when compared to control beams without the FRP panels. These studies have been conducted on structural response to static loads, drop weight dynamic loads and blast loadings from conventional explosives. In all cases considerable strength enhancement has been observed. For beams retrofitted with bottom tensile CFRP strips the largest strength enhancement occurred for RC beams with steel reinforcing ratios of 1.5 percent or less. This corresponds to a ratio of steel area (tension only) to CFRP area of approximately 4.0 or less.

In cooperation with WL/FIVCO, the University of Florida developed a vacuum bond technique to apply prefabricated FRP panels to concrete surfaces using a commercial high performance epoxy adhesive. In addition to beam and slab structural response, WL/FIVCO has conducted tests of freeze-thaw cycling, ultraviolet exposure, heating, cooling, wetting and drying on concrete samples with and without various FRP strips. No detrimental effects were observed for these tests. Fatigue strength of externally reinforced beams (no steel) was also studied at WL/FIVCO using a non-reversed fatigue loading of 80 and 10 percent of static maximum load applied at a rate of 20 Hz for 2,000,000 cycles.

Based on results obtained in the WL/FIVCO studies, one span of a four girder multispan RC highway bridge was retrofitted by Auburn University with side glass fiber reinforced plastic (GFRP) and bottom CFRP. Traffic tests before and after retrofit of the bridge span showed an approximate 12 percent reduction of both midspan displacement and rebar strain of those of the unrehabilitated bridge span.

External bonding of very thin high-modulus, high-strength fiber reinforced plastic panels to concrete structures has been shown to give increased stiffness and larger load carrying capacity. Experimental and analytical results have verified that these advanced composite materials are very useful and practical in increased hardening and rehabilitation of existing concrete structures. Both nondestructive and destructive test methods show that laboratory CFRP/concrete beams show small detrimental effects as a result of environmental exposure to freeze-thaw cycling and ultraviolet light. The energy absorbing capacity of CFRP/concrete beams was increased by a factor of 50 over that of control beams, when tested statically.

2.5 Flexural behavior of aged prestressed concrete girder Strengthened with various FRP systems

Owen Rosen boom, Tare K. Hassan, Sami Rizkalla
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Many prestressed concrete bridges are in need of upgrading in order to increase their posted capacities. Departments of transportation across the country have been faced with larger financial burdens on the maintenance budget, negative psychological effects on highway users, long traffic delays during maintenance, potential safety hazards, and reduced service life as a result of the deficiencies. In response to considerable consultation with the North Carolina Department of Transportation (NCDOT), are search project with practical goals was initiated to evaluate the cost-effectiveness and value engineering of Carbon Fiber Reinforced Polymer (CFRP) repair and strengthening systems for prestressed concrete bridge girders.

This paper presents the first phase of the research program, involving the testing under static loading conditions of eight prestressed Concrete bridge girders, six strengthened with various CFRP systems. Results show that the ultimate capacity of prestressed concrete bridge girders can be increased by as much as 73% using CFRP without sacrificing the ductility of the original member. Transverse CFRPU-wrap reinforcements are recommended along the length of the girder to control debonding type failures. The second phase of the research will examine the fatigue behavior of the strengthened girders, and provide analysis under service loading conditions.

Eight 9.14m (30ft) prestressed concrete C-Channel girders were tested under static loading conditions, six strengthened with various CFRP systems and two tested as control girders. Based on the experimental program and analysis of the test results, the following conclusions can be made:

1. Proper design and installation of a CFRP strengthening system can lead to the failure mode of crushing of concrete in the compression zone, preserving the ductile structural response of an unstrengthened girder.
2. Externally bonded CFRP sheets are the most cost-effective strengthening technique and are the most applicable technique for these types of girders.
3. The most structurally efficient strengthening technique used is the near surface mounted (NSM) CFRP bars or strips system.
4. The ultimate flexural capacity of prestressed concrete can be increased substantially using CFRP materials. The flexural capacity of the C-Channel girders tested in this research program could be increased by as much as 73% with the use of externally bonded CFRP sheets.
5. The use of transverse CFRPU-wraps most likely delayed debonding failures in externally bonded CFRP systems.
6. For externally bonded CFRP wet lay-up systems, the experimental tensile strain in the CFRP outperformed the manufacturers provided value. Therefore, the use of a bond reduction coefficient, Km in ACI440F2002, m would be conservative.
7. The crack spacing and crack widths at ultimate can be substantially reduced using CFRP strengthening. Crack widths observed during the testing of the C-Channels were reduced by as much as 400% using CFRP materials in comparison to the unstrengthened girder.
8. The initial and secondary stiffness of C-Channel girders can be increased by the use of high modulus CFRP materials. Using strictly a serviceability criterion, high modulus CFRP materials outperformed the normal modulus CFRP materials.

2.6 COMPREHENSIVE DESIGN EXAMPLE FOR PRESTRESSED CONCRETE (PSC) GIRDER SUPERSTRUCTURE BRIDGE WITH COMMENTARY

Modjeska and Masters, Inc. November 2003

(THE FEDERAL HIGHWAY ADMINISTRATION)

This example is part of a series of design examples sponsored by the Federal Highway Administration. The design specifications used in these examples is the AASHTO LRFD Bridge design Specifications. The intent of these examples is to assist bridge designers in interpreting the specifications, limit differences in interpretation between designers, and to guide the designers through the specifications to allow easier navigation through different provisions. For this example, the Second Edition of the AASHTO-LRFD Specifications with Interims up to and including the 2002 Interim is used.

This design example is intended to provide guidance on the application of the AASHTO-LRFD Bridge Design Specifications when applied to prestressed concrete superstructure bridges supported on intermediate multicolumn bents and integral end abutments. The example and commentary are intended for use by designers who have knowledge of the requirements of AASHTO Standard Specifications for Highway Bridges or the AASHTO-LRFD Bridge Design Specifications and have designed at least one prestressed concrete girder bridge, including the bridge substructure. Designers who have not designed prestressed concrete bridges, but have used either AASHTO Specification to design other types of bridges may be able to follow the design example; however, they will first need to familiarize themselves with the basic concepts of prestressed concrete design.

This design example was not intended to follow the design and detailing practices of any particular agency. Rather, it is intended to follow common practices widely used and to adhere to the requirements of the specifications. It is expected that some users may find differences between the procedures used in the design compared to the procedures followed in the jurisdiction they practice in due to Agency-specific requirements that may deviate from the requirements of the specifications. This difference should not create the assumption that one procedure is superior to the other.

2.7 DESIGN RECOMMENDATIONS FOR CONCRETE STRUCTURES PRESTRESSED WITH FRP TENDONS. FHWA CONTRACT August1, 2001

Charles W. Dolan, H.R. Hamilton III (University of Wyoming), Charles E. Bakis (Pennsylvania State University) & Antonio Nanni (University of Missouri - Rolla)

This report presents the state of development of fiber-based (non-metallic) reinforcement for prestressed concrete structures. It summarizes work in progress, work executed in this project and presents design recommendations for the use of FRP prestressing materials. The term fiber reinforced polymer (FRP) is used to identify this type of primary reinforcement used for prestressed concrete members. The material presented in this report includes a basic understanding of flexure and axial loaded prestressed members, bond of FRP tendons and a preliminary understanding of FRP shear reinforcement for prestressing applications. Specifications for testing FRP tendons are presented. Recommendations for AASHTO SPECIFICATIONS changes to incorporate FRP prestressing are presented in the results of this research.

2.8 Bonded Repair and Retrofit of Concrete Structures Using FRP Composites

AMIR MIRMIRAN et.al, Department of Civil Engineering North Carolina State University

Raleigh, NC (NCHRP REPORT 514) Recommended Construction Specifications and Process Control Manual

Since its first applications in Europe and Japan in the 1980s, use of bonded repair and retrofit of concrete structures with fiber reinforced polymer (FRP) systems has progressively increased to the extent that today it counts for at least 25 Innovative Bridge Research and Construction (IBRC) projects in the United States, in addition to numerous projects independently undertaken by state departments of transportation (DOTs) and counties. Because of their light weight, ease of installation, minimal labor costs and site constraints, high strength-to-weight and stiffness-to-weight ratios, and durability, FRP repair systems can provide an economically viable alternative to traditional repair systems and materials. It is generally accepted that long-term performance of FRP systems is affected not only by the constituent materials, but also by the processes used during construction. However, the relationships between the long-term performance of FRP systems and the construction processes are not easy to quantify. Hence, there is a lack of generally accepted construction specifications and process control procedures for FRP repair systems, and state DOTs are heavily dependent on FRP manufacturers to provide construction process control. As the FRP technology matures and moves into widespread use, the need has become more urgent than ever to equip state DOTs with the means to specify and control the constituent materials and the adequacy of the construction process. This study was undertaken to develop recommended construction specifications and a construction process control manual for bonded FRP repair and retrofit of concrete structures that will ensure performance as designed. The three most common types of FRP repair systems were considered: wet lay-up, pre-cured, and near surface mounted. The study was based on then-current scientific and engineering knowledge, research findings, construction practice, performance data, and other information related to FRP constituent materials and FRP systems. The information was gathered from a literature search, existing databases, a questionnaire survey, telephone interviews, and a clearing house website.

A number of issues and parameters relevant to FRP repair were identified based on the collected data and were used in developing the recommended construction specifications and the process control manual.

The proposed specifications include eight main sections: General; Submittals; Storage, Handling, and Disposal; Substrate Repair and Surface Preparation; Installation of FRP System; Inspection and Quality Assurance; Repair of Defective Work; and Measurement and Payment. The proposed process control manual covers quality control (QC) and quality assurance (QA) prior to, during, and after completion of the repair project. It consists of planning, record keeping, inspection and QC tests. The manual includes the following main sections: QA Policy and Program Overview; QA Guidelines for Construction Activities; and Implementing and Monitoring of the QA Program. The manual also consists of a number of QA checklists for the FRP repair projects. Critical review of the FRP research indicates a general consensus on the most relevant issues and parameters for construction specifications and a process control manual. However, the primary concern throughout this study has been, and remains, to justify the rational basis for the specified tolerances, criteria, and procedures. The novelty of the FRP technology and its subtle differences from the traditional repair systems are reflected in the proposed specifications. Some of the proposed provisions may appear more restrictive than the current practice for traditional materials. Although the industry may find such restrictions counterproductive for further development of new FRP technology, the main objective has been to help protect state DOTs from low-quality applications with major defects. The decision on relaxing or replacing any of the restrictions ultimately lies with the American Association of State Highway and Transportation Officials (AASHTO) and its member states. The states can use the proposed specifications and process control as “model documents’ that need to be tailored to their specific needs as well as to the size and intent of each project. At the same time, it should be understood that as the FRP technology matures, and as new research data become available, some of those restrictions may be removed or relaxed. In fact, the report identifies provisions in the two documents that may need further refinement, and recommendations are made for future research to accomplish these refinements.

The long-term benefits of this research include lower maintenance costs and longer service life for repaired and retrofitted structures. These benefits will reduce the annual backlog for bridge replacement, resulting in lower costs to maintain or improve the transportation system. It is expected that bridge construction inspectors, general contractors, FRP subcontractors, and FRP and adhesive material suppliers will use the results of this research. Therefore, a four-element implementation plan is suggested for use by highway agencies. The plan includes training and technology transfer, a shakedown period, trial field applications, and an updating process.

2.9 Strengthening of an impact- damaged PC Girder

Antonio Nanni, PhD, PE

Repair of impacted prestressed and reinforced concrete structures using traditional and emerging technologies has been the subject of several studies. Fiber reinforced polymer (FRP) systems composed of fibers embedded in polymeric matrix exibit properties that make them suitable for their use as structural reinforcing elements. FRP composites corrosion resistant and are expected to perform better than other construction materials in terms of weathering behavior. Several types of fibers have been developed for use in FRP composites. For this project, carbon fibers, which are recognized to be the stiffest and most durable, were used.

The objective of this project was to restore the original flexural capacity of an accidentally impact-damaged PC girder. Two prestressing tendons in the central girder were fractured due to the impact.

2.10 Finite Element analysis of historic Bridge strengthened with FRP laminates

Damian I. kachlakev, Ph.D., P.E. California Polytechnic State university

A three-dimensional finite element model is developed to examine the structural behavior of the Horsetail Creek Bridge in Oregon both before and after applying FRP laminates. Nonlinear finite element analysis is performed using the ANSYS program. SOLID65, LINK8, and SOLID46 elements represent concrete, discrete reinforcing steel bars, and FRP laminates, respectively. Based on each component's actual characteristics, nonlinear material properties are defined for the first two types of elements. Truck loadings are applied to the FE bridge model at different locations, as in the actual bridge test. The comparisons between ANSYS predictions and field data are made in terms of concrete strains. The analysis shows that the FE bridge model does not crack under the applied service truckloads. The FE bridge model very well predicts the trends in the strains versus the various truckload locations. In addition, effects of FRP strengthening on structural performance of the bridge are observed in the linear range.

The comparisons between ANSYS predictions and the experimental data show that the proposed FE models are good representations for both the HCB in terms of the number of elements, structural details, and, especially, reasonably accurate results in general. The HCB does not crack under the service applied truckload. Consequently, the uncracked bridge structure still behaves linearly. The trends in the strain results for the various locations of the truck obtained from both ANSYS and SAP2000 models are similar to those from the field test data. However, the ANSYS strain results differ from the field strain data by approximately 60% up to 130%. This may be because of inaccurate material properties for the concrete or an incorrect strain calibration in the field. The FEM analysis shows that when the single axle of the empty truck is positioned at about 11.05 m from the end of the bridge deck, a maximum strain values is developed in the transverse beam. This is because the loads from the tandem axles strongly influence the beams when positioned at this location. For the influence of FRP strengthening, the differences in structural responses before and after the retrofit are not significant in the linear behavior range.

2.11 IITK-GSDMA GUIDELINES on MEASURES TO MITIGATE EFFECTS OF TERRORIST ATTACKS ON BUILDINGS

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Explosions almost instantaneously damage the built environment. If more devices than one are used in a chain, then the duration of the threat is enhanced and the extent of damage is greater. The extent of damage is determined by the type, quality and quantity of explosive used, and the stand-off distance from the structure. Damage can vary over a spectrum of possibilities – from non-structural element loss, structural element damage, structural element collapse, to progressive failure of part/whole building. When an explosion takes place, an exothermic chemical reaction occurs in a period of few milliseconds. The explosive material (in either solid or liquid form) is converted to very hot, dense, high-pressure gas. This highly compressed air, traveling radially outward from the source at supersonic velocities is called the shock wave front. It expands at very high speeds and eventually reaches equilibrium with the surrounding air. Usually, only about one-third of the chemical energy available in explosives is released in the detonation process; the remaining two-thirds energy is released relatively slowly as the detonation products mix with air and burn. While this process of burning has little effect on the initial blast wave because of its delayed occurrence than the original detonation, it can influence the later stages of the blast wave, particularly in explosions in confined spaces. As the shock wave expands, pressure decreases rapidly with distance D (as $1/D^3$) because of spherical divergence and dissipation of energy in heating the air. Also, pressure decays rapidly over time (as exponential function), typically in milliseconds. Thus, a blast causes an almost instantaneous rise in air pressure from atmospheric pressure to a large overpressure. As the shock front expands, the pressure drops but becomes negative. Usually, this negative pressure is sustained for duration longer than the positive pressure and is less important in design of structures than the positive phase. When this high-pressure shock front strikes a surface (is it ground or structure) at an angle, it is reflected producing an increase in the pressure of the air. The pressure of air in the reflected front is greater than that in the incident front, even at the same distance from the explosion. The reflected pressure varies with the angle of incidence of the shock wave – a maximum when it impinges normal to the surface and a minimum when it passes parallel to it. In addition, the reflected pressure is dependent on the incident pressure, which in turn is a function of the net explosive weight and distance from the detonation. Further, for all explosions, the reflected pressure coefficients are significantly greater closer to the explosion. The magnitude and distribution of the blast loading effectively acting on a structure vary greatly with (a) properties of explosive (type of material, quantity of explosive and energy output), (b) Location of detonation relative to the structure, and (c) reflections of shock front on the ground and structure. The damage in a building depends on the energy imparted to it through the reflected shock front of explosion, which is contributed by both the positive and negative phases of the pressure-time history.The pressure and hence forces on the building vary in time and space over the exposed surface of the building, depending on the location of the detonation in relation to the building. Therefore, when studying the response of a structure under a specific blast, the location of detonation which produces the most severe effects on the structure should be identified.

Blast effects are distinctly different from other hazards, like earthquakes, winds, or wave. The following are some of the distinguishing features:

1. The blast pressures generated on a targeted building can be several orders of magnitude greater than those generated by wind or wave. For example, the peak incident pressure on a building in an urban setting can be larger than 8 MPa, due to a vehicle weapon parked along its curb. At such pressures, major damages and failure are expected in the building.

2. Explosive pressures decay extremely rapidly with distance from the source.

Therefore, damages on the side of the building facing the explosion may be significantly more severe than those on the other sides. Hence, air blasts tend to cause localized damage. When the building is surrounded by other buildings as in an urban setting, reflections off surrounding buildings can cause increased damages even on the back side of the building.

3. The duration of the blast shock front is of the order of milliseconds. This is in contrast to the duration of loading of seconds during earthquakes and of hours during wind or flood. As a result of this, the mass of the structure has a strong mitigating effect on the building response. This is in contrast to the situations in earthquakes, wherein larger mass can induce increased inertia forces, which can worsen the damage.

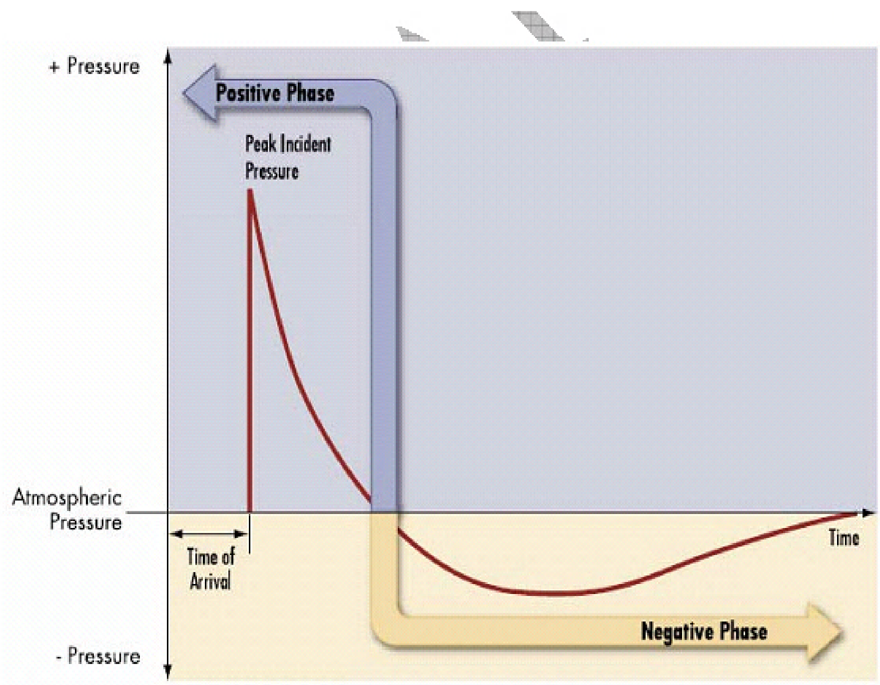


Figure 2.1: *Shock Front Characteristics*: Overpressure-time history indicating sharp initial drop and extended negative phase (FEMA 426, 2003)

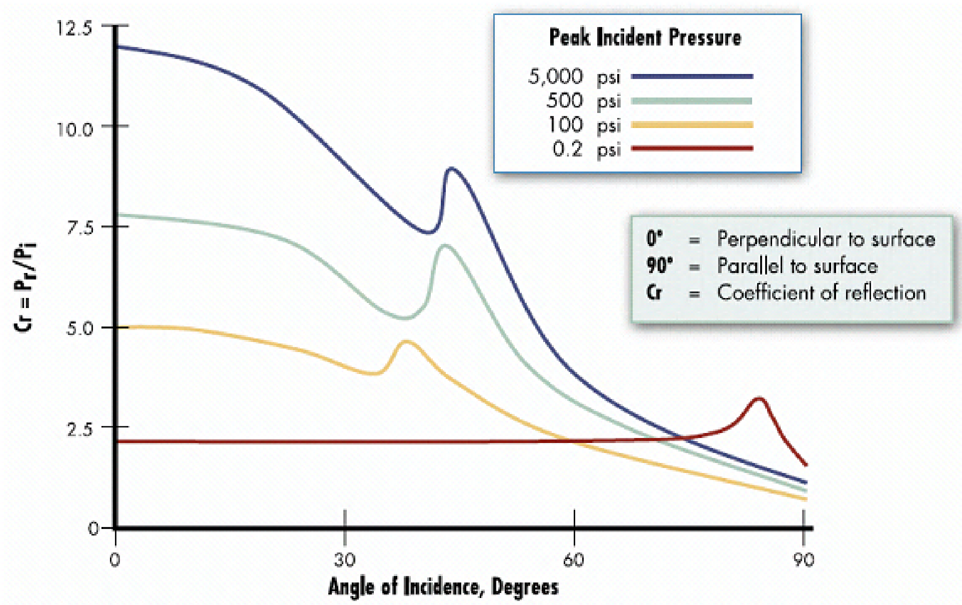


Figure 2.2: *Reflected Shock Front Characteristics*: Influence of angle of incidence of shock front and pressure of incident shock (FEMA 426, 2003)

2.12 Performance of AASHTO Girder Bridges under Blast Loading

A.K.M. Anwarul Islam (2005)

The purpose of this research is to assess the performance of an American Association of State Highway and Transportation Officials (AASHTO) girder bridge under blast loading. The AASHTO has specified probability based design methodology and load factors for designing bridge piers against ship impact and vehicular collision. Currently, no specific AASHTO design guideline exists for bridges against blast loading. Structural engineering methods to protect infrastructure systems from terrorist attacks are required.

Bridges are unique in terms of materials used, length, width, skew ness, span, loading, traffic conditions, and overall geometry. This research investigated the most common types of concrete bridges on the interstate highways. A 2-span 2-lane bridge with Type III AASHTO girders was used for modeling. Girder spacing was chosen based on the span length and loading conditions. AASHTO Load and Resistance Factor

Design (LRFD) methods were used for bridge design. Performance of AASHTO girders, piers and columns under typical blast loading were analyzed and documented for future use in blast resistant design of concrete bridges.

The model bridge failed under typical blast loads applied over and underneath the bridge. The research findings concluded that the AASHTO girders, pier cap and columns could not resist typical blast loads. The amount of blast loads, which the individual members can resist before failure, was determined. The model bridge columns were capable of resisting typical blast loads if the explosion occurs at a minimum standoff distance. From these research findings, necessary measures to protect existing bridges from explosion impact are specified and recommended for bridge modification and design guidelines.

2.13 IS:4991-1968, Criteria for blast Resistant Design of Structures for Explosions above Ground

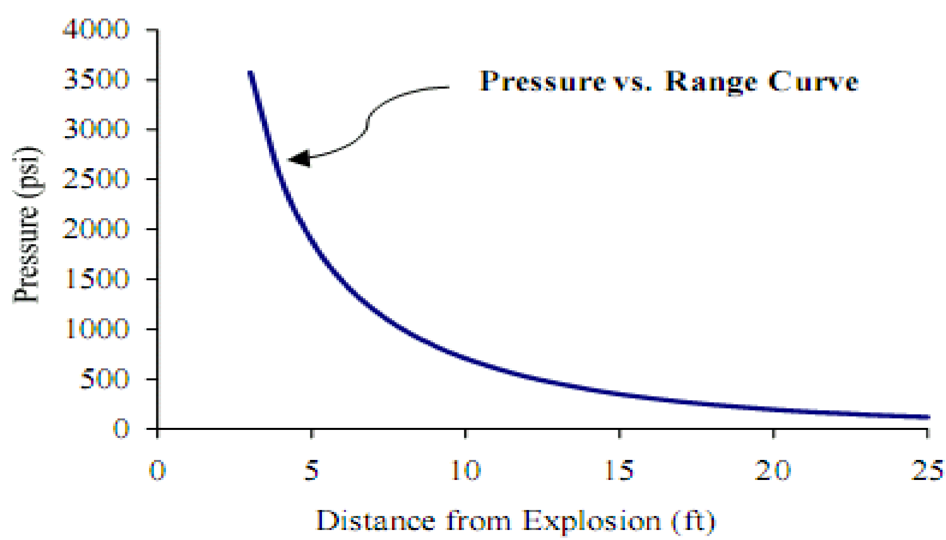
Blast load for design consideration is different then other loads. During Blast, the peak intensity last for a very small duration only. To design a structure capable of resisting these intense but short duration loads, members and joints are permitted to deflect and strain much greater than is allowed for usual static loads. This permitted deflection is , ordinarily, well into the plastic range of the material. Large amount of energy are absorbed during this action, thus reducing the required design strength considerably below that required by conventional design within elastic range. Moreover, under higher rates of loading the strength developed by the material, increases with the rate of loading, and may often be adequately described as the function of time within a certain range. If the location of the ground zero and the bomb are known, the corresponding blast loading for an existing structure may be found by the methods explained in this standard. But it is difficult and not certain to have exact data for specifying the ground zero and bomb size. Therefore, three different standard blast loading are recommended in this standard. This standard also contains necessary informations for evaluating various parameters of blast wave generated by any other size of explosion at a given distance and the design of structure for the same.

The method of determining equivalent blast load due to an explosion is a complex phenomenon. The blast pressure diminishes with distance from the point of explosion. In the TM 5-1300 Manual, Structures to Resist the Effects of Accidental Explosions, developed by the US Department of Defense in December 1990 (DOD 1990), an empirical formula, as shown in Eq. 3.1, was used to find the scaled distance. The amount of blast pressure generated due to an explosion is inversely proportional to the scaled distance, which is presented in a chart in the TM 5-1300 Manual. The empirical formula to find the scaled distance, Z (ft), is:

$Z = R/W^{(1/3)}$

Where, R = Distance of target from point of explosion (ft),

W = Equivalent TNT weight of charge (lb).



2.3 Variation of Pressure with Distance from Explosion

3. BRIDGE MODELLING:

3.1 Discretization of bridge model

Length of Bridge is 29 meter, double lane. Main girder consists of 11 tendon having 12 strands of 7mm diameter in each tendon. Main girder was discretising as solid element of various sizes with length 1m and 0.5m. SAP (version 10) has facility of using tendon element with which tendon was embedded to solid element. There is 7 number of cross girder and modeled as shell element. Deck Slab was also modeled as thin shell. Shell element was modeled for CFRP. Frame element on deck was also used for applying vehicle wheel load on it, so that load from wheel is transfer to the deck slab through frame element.

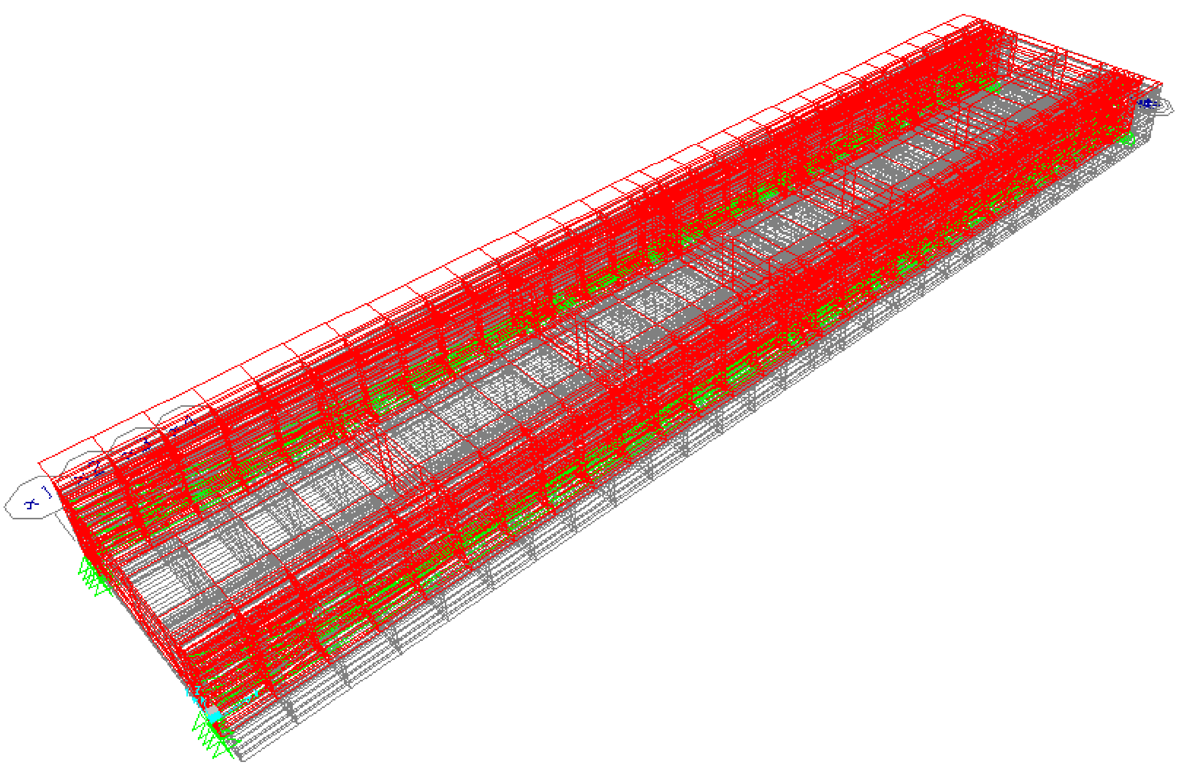


Fig. 3 1 3D model of real Bridge in SAP

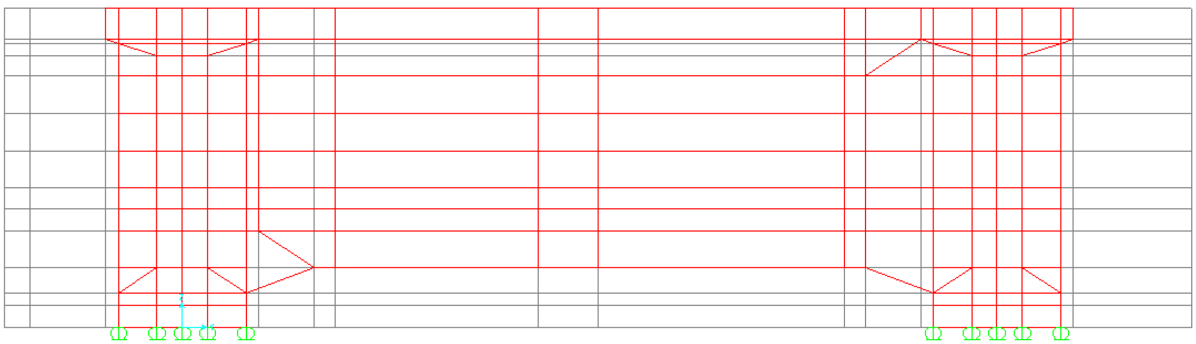


Fig.3.2 Cross Section of the bridge model in SAP

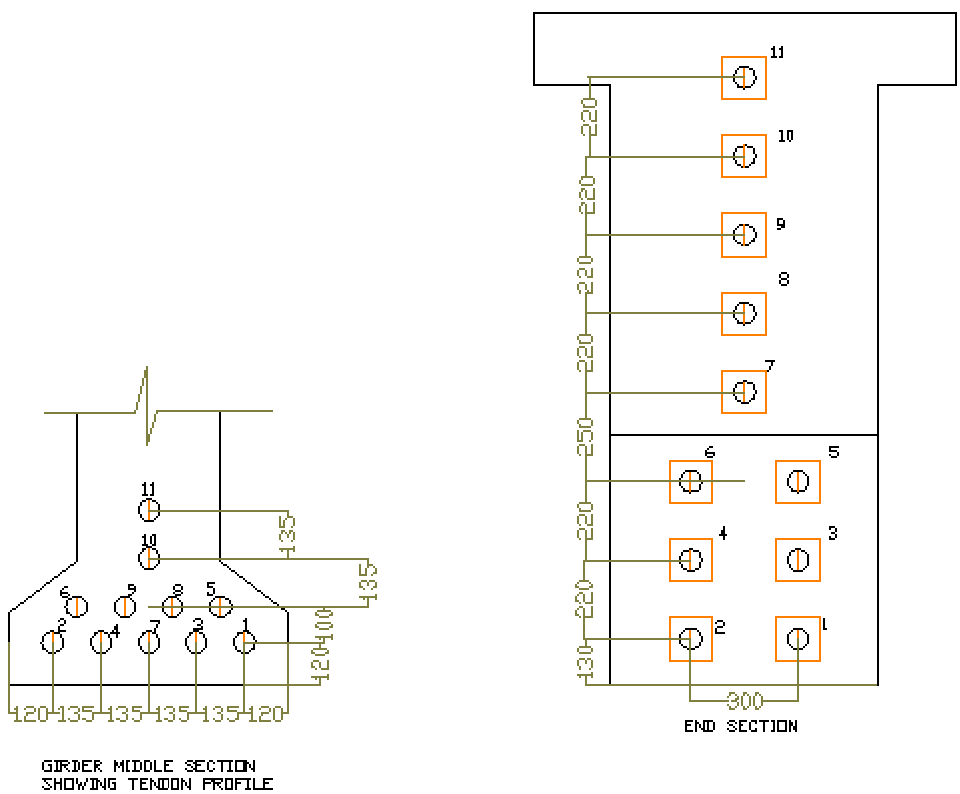


Fig. 3.3 Position of the tendon in support and mid section of the Girder

3.2 Tendon Profile:-

Table 3.1. Vertical Profile

	Z1	Z2	Z3	Z4	Z5	Z6	Z7	Z8	Z9	Z10	Z11
0	0.130	0.130	0.350	0.350	0.570	0.570	0.820	1.040	1.260	1.480	1.700
1	0.104	0.104	0.185	0.185	0.390	0.390	0.490	0.683	0.807	1.030	1.235
2	0.100	0.100	0.133	0.133	0.328	0.328	0.367	0.548	0.636	0.858	1.055
3	0.100	0.100	0.108	0.108	0.289	0.289	0.283	0.454	0.517	0.736	0.927
4	0.100	0.100	0.100	0.100	0.264	0.264	0.221	0.384	0.428	0.641	0.827
5	0.100	0.100	0.100	0.100	0.250	0.250	0.175	0.330	0.359	0.566	0.746
6	0.100	0.100	0.100	0.100	0.245	0.245	0.141	0.288	0.307	0.506	0.680
7	0.100	0.100	0.100	0.100	0.245	0.245	0.118	0.258	0.268	0.457	0.627
8	0.100	0.100	0.100	0.100	0.245	0.245	0.104	0.237	0.241	0.419	0.583
9	0.100	0.100	0.100	0.100	0.245	0.245	0.100	0.224	0.225	0.391	0.549
10	0.100	0.100	0.100	0.100	0.245	0.245	0.100	0.220	0.220	0.371	0.523
11	0.100	0.100	0.100	0.100	0.245	0.245	0.100	0.220	0.220	0.359	0.504
12	0.100	0.100	0.100	0.100	0.245	0.245	0.100	0.220	0.220	0.355	0.494
13	0.100	0.100	0.100	0.100	0.245	0.245	0.100	0.220	0.220	0.355	0.490
14	0.100	0.100	0.100	0.100	0.245	0.245	0.100	0.220	0.220	0.355	0.490
14.5	0.100	0.100	0.100	0.100	0.245	0.245	0.100	0.220	0.220	0.355	0.490

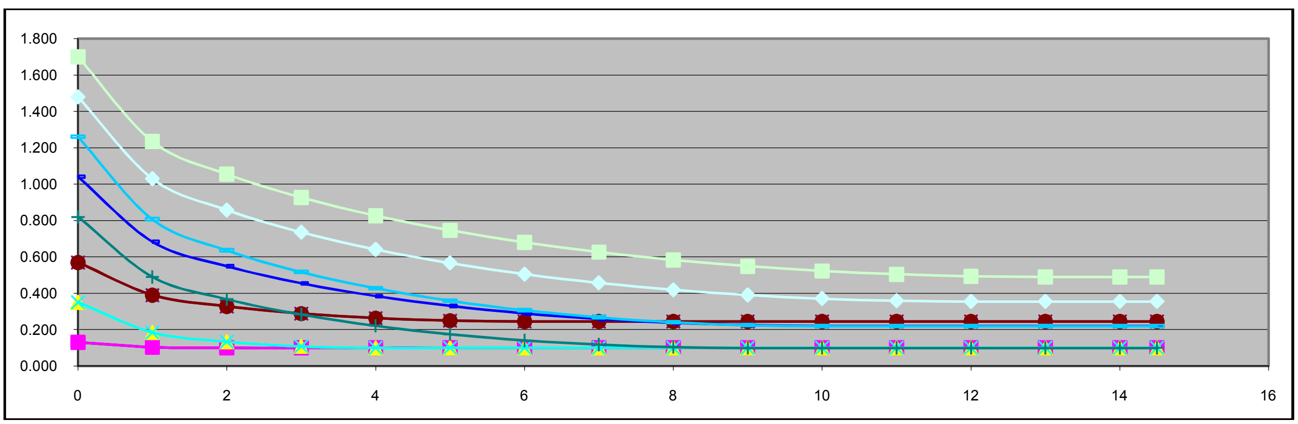


Fig.3.4 Vertical profile of 11 tendons

Table 3.2. Horizontal Profile

	Y1,2	Y3,4	Y5,6	Y7	Y8	Y9	Y10	Y11
0	0.150	0.150	0.150	0.000	0.000	0.000	0.000	0.000
1	0.150	0.150	0.150	0.000	0.000	0.000	0.000	0.000
2	0.210	0.150	0.150	0.000	0.000	0.000	0.000	0.000
3	0.270	0.143	0.176	0.000	0.000	0.000	0.000	0.000
4	0.270	0.135	0.203	0.000	0.000	0.000	0.000	0.000
5	0.270	0.135	0.203	0.000	0.000	0.000	0.000	0.000
6	0.270	0.135	0.203	0.000	0.034	-0.034	0.000	0.000
7	0.270	0.135	0.203	0.000	0.068	-0.068	0.000	0.000
8	0.270	0.135	0.203	0.000	0.068	-0.068	0.000	0.000
9	0.270	0.135	0.203	0.000	0.068	-0.068	0.000	0.000
10	0.270	0.135	0.203	0.000	0.068	-0.068	0.000	0.000
11	0.270	0.135	0.203	0.000	0.068	-0.068	0.000	0.000
12	0.270	0.135	0.203	0.000	0.068	-0.068	0.000	0.000
13	0.270	0.135	0.203	0.000	0.068	-0.068	0.000	0.000
14	0.270	0.135	0.203	0.000	0.068	-0.068	0.000	0.000
14.5	0.270	0.135	0.203	0.000	0.068	-0.068	0.000	0.000

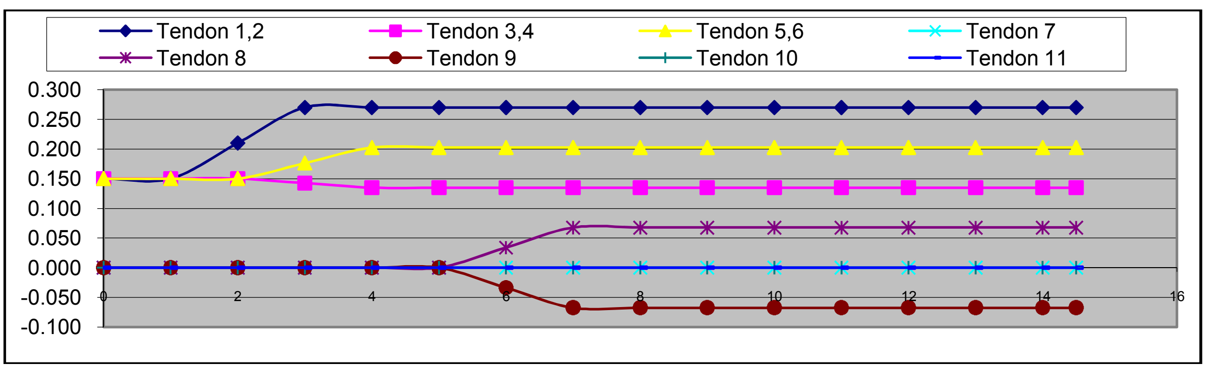


Fig3.5 Horizontal profile of 11 tendons (Symmetrical at distance Y=0.00)

3.3 Material properties

3.3.1 Prestressing Cable

i. Standard Freyssinet Prestressing Cables.

a.	12 Number of 7mm dia strands were used in each cable.	
b.	Nominal U.T.S of Cable	=1500N/mm2
c.	Nominal steel area of cable	=462mm2
d.	Nominal ultimate Breaking force of cable	=691 KN.
e.	Maximum allowable Prestressing Force	= 553 KN
f.	Maximum allowable Initial Stress	= 1200N/mm2
g.	App. Weight per unit length of cable	= 3.6 kg/m

ii. Friction and Anchorage Losses

1. Curvature Coefficient (unitless)	=.15
2. Wobble Coefficient (l/m)	$=8.3x10^{-5}$
3. Anchorage slip	=vary as per damage condition

iii. Other loss parameters

1. Elastic Shortening Stress(KN/m^2)	=20684
2. Creeep Stress (KN/m^2)	=34473
3. Shrinkage stress (KN/m^2)	=48263
4. Steel relaxation stress(KN/m^2)	=34473

3.3.2. Concrete properties

Grade of concrete was determined by non destructive test.

To predict the original grade of concrete, Schmidt hammer test has been performed at site so that it can be used for analysis. More than 50 rebound values are collected, And tabulated in table in Appendix A.

The strength of the concrete at different confidence level is shown in table Appendix A.. The test determines the hardness of the concrete surface, and although there is no unique relation between hardness and strength, empirical relationship can be determined for similar concretes cured in such a manner that both the surfaces tested by the hammer and the central regions, in whose strength we are interested, have the same strength. The correlation of compressive strength and rebound numbers shown in this report are very approximate because very few data are used here for correlation. And also various factors affect to find the compressive strength accurately such as surface condition of tested surface, presence of large aggregates at test points, other manual unseen errors etc. Thus For my thesis model Grade of concrete was taken 40MPA.

3.3.3. CFRP properties:-

Mbrace Laminates CFK (200/2000) is applied for strengthening.

Fiber areal weight (g/m2):-	=200
Width /thickness	=50/1.4mm/mm
Modulus of elasticity	=200GPa
Tensile strength at Break	=2500N/mm^2
Tensile force at elongation of 0.6/0.85	=84/112*10^{3}N

MBrace CF 240(430 GSM) - Unidirectional Carbon Fiber Reinforcement System High Tensile CF

Modulus of elasticity	=240 kN/mm^2
Tensile strength	= 3800 N/mm^2
Weight of C fiber (main direction)	= 400 g/m^2
Total weight of sheet	= 430 g/m^2
Density	= 1.7 g/cm^3
Ultimate %	=1.55
Thickness for static design weight / density	= 0.234 mm
Safety factor for static design	= 1.2 (recommended)
(Manual lamination / UD-product)	

Laminated Area

1. Layer (area 1) = 50*1.4*(6+2+1) =630 mm^2

Length of laminate =3.4m

Equivalent thickness =Area/ width of Girder bottom Flange

= 630/750 =0.84mm

2. Layer (area 2) = 50*1.4*(2+1) =210 mm^2

Length of laminate =10m

Equivalent thickness =Area/ width of Girder bottom Flange

= 210/750 =0.7 mm

3. Layer (area 3) = 50*1.4*(1) =70 mm^2

Length of laminate =20m

Equivalent thickness =Area/ laminated width at Girder bottom

= 70/300 =0.233 mm

4. NUMERICAL ANALYSIS

1. **Dead Load**: - DL Self weight of the Bridge and tendons which SAP calculates itself.

2. **Prestressing load** :-

Total 11 strands are used in each girder. Each tendon strand contains 12 number of 7 dia wires, Upper 5 tendon strands are not damaged (see Fig.5). Lower P1 tendon was slipping about 80mm (Bottom right side tendon) and Lower P3 and P5 tendon has been damaged (Right side top and middle tendons) as shown in figure below.

Fig.4.1 Showing damaged tendon P3, P5

In SAP (version 10), the Prestressing cables are modeled as Tendon element in which we can assign different property of tendons as described in clause 3.3.1. Here in analysis, for normal condition of cables (NPL), all the tendon transfer the prestressing load of 540KN and having slipping value of only 10mm is assigned. For various trials for slips of tendons that described above, load cases SPL0, SPL50, SPL 100 is assigned. Where load case of SPL0 means all other tendons are normal except, in left girder, tendon P1 has slip of 100mm, and tendons P3, P5 was assumed damages so no load can transfer by these two tendons. So in SAP for load case SPL0, above values are assigned and analysis was done and different load combinations are checked for comparing the stress values.

Similarly, Load case SPL50 means, all other tendons are normal except, in Left girder, tendons P1,P3 and P5 having slip value 50 mm is assigned in SAP model for Analysis. And also for load case SPL100, for the same tendons slip value of 100mm is assigned to check the stress variations.

3. Live Load

Live load of Class A train of vehicle loading from IRC 6-2000 is taken. These days 10 wheeled Trucks are commonly used which is also taken on study. In the case of traffic jams, vehicles are so queued that the loading data becomes different from the code so why I used the 10 wheeled truck load as it is used practically. Below shows the positioning of the vehicle load for maximum case which is assigned for analysis. Load cases for maximum Shear force. Maximum bending moment for both the cases is done.

Here 10 wheeled truck was assumed that it transfer total of 23 tones load, that is taken From TATA truck catalogue.

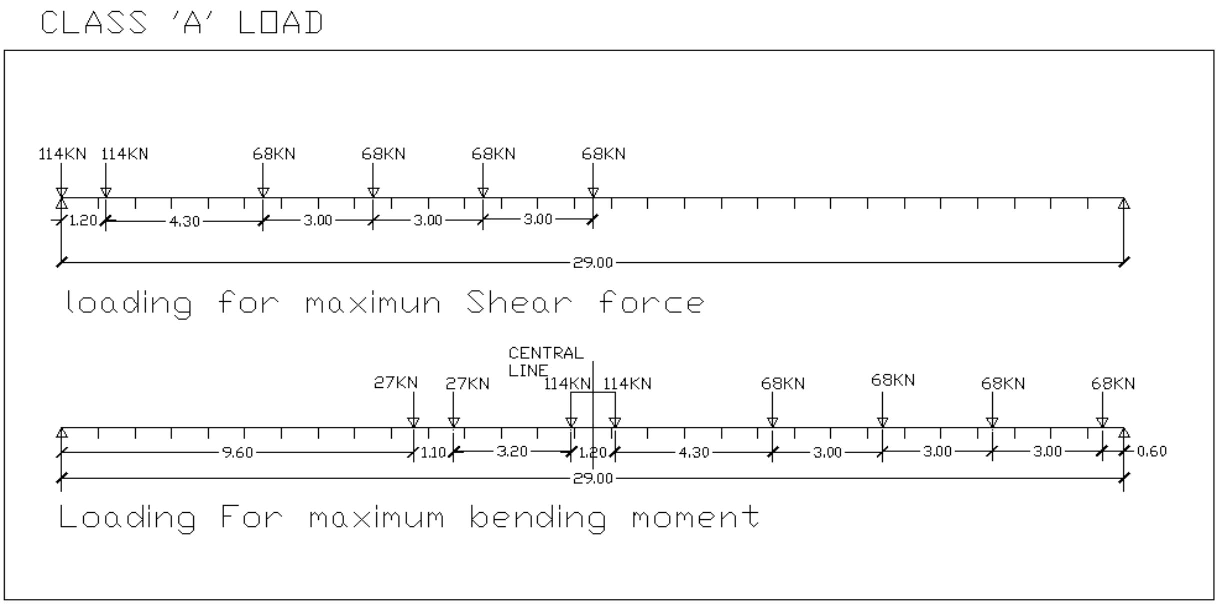


Fig.4.2 Class A loading for maximum Shear force and bending moment.

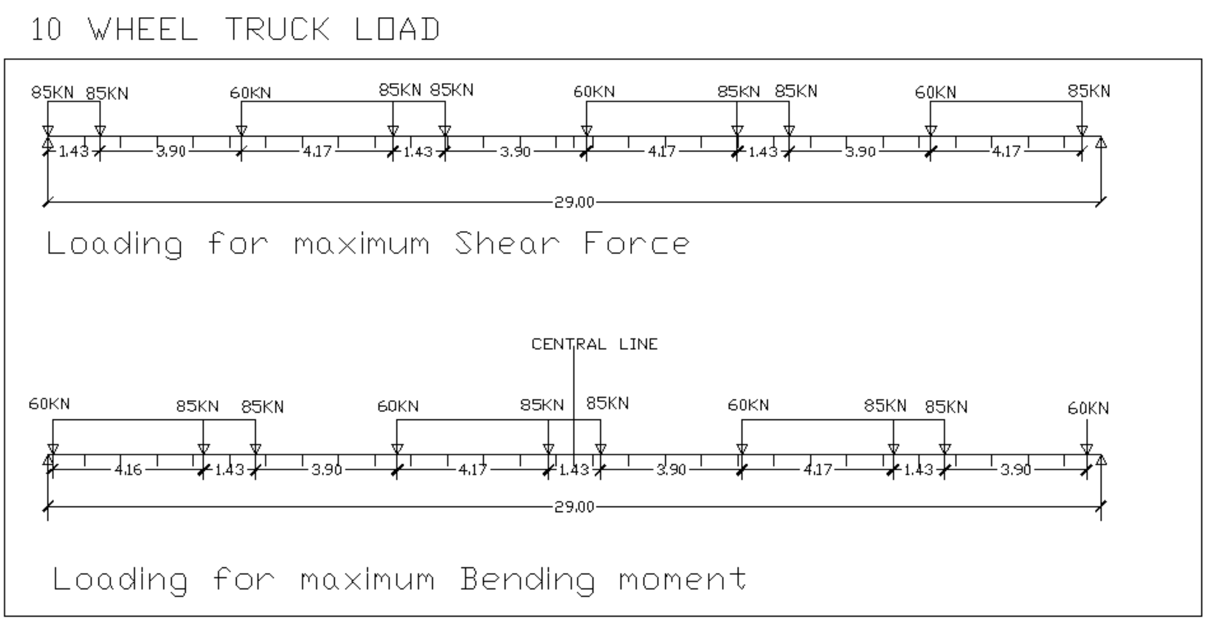


Fig. 4.3 Loads arrangement for TATA 10 wheeled truck

4. Load Combinations

NP= Sum of all normal case of tendon loading

Npl= P1.1+P2.1…………+ P6.1+P7…………+P11

SPL = Combination of prestressing loading all normal except P1 having 100mm slip, P3 and P5 are not workable.

SPL = P1.100+P2.1+P4.1+P6.1+P7…………+P11

Combination No:-

4. DL+NPL+SF.CA
5. DL+NPL+SF.10WL
6. DL+NPL+BM.CA
7. DL+NPL+BM 10WL
8. DL+NPL+SF.CA1
9. DL+NPL+SF.10WL1
10. DL+NPL+BM.CA1
11. DL+NPL+BM 10WL1
12. DL+SPL0+SF.CA
13. DL+SPL0+SF.10WL
14. DL+SPL0+BM.CA
15. DL+SPL0+BM 10WL
16. DL+Npl
17. DL+Spl
18. DL+SPL0+SF.CA1
19. DL+SPL0+SF.10WL1
20. DL+SPL0+BM.CA1
21. DL+SPL0+BM 10WL1
22. DL+SPL100+SF.CA
23. DL+SPL100+SF.10WL
24. DL+SPL100+BM.CA
25. DL+SPL100+BM 10WL

26. DL+SPL100+SF.CA1
27. DL+SPL100+SF.10WL1
28. DL+SPL100+BM.CA1
29. DL+SPL100+BM 10WL1

5. FRP is used to Strengthening the girder.

Internationally, there are lots of work that has been done on retrofitting by using FRP. Being light in weight, having higher tensile strength, it becomes only one options for the purpose of retrofitting work. It has got the ability to give the structures new life by increasing the tensile (flexural) strength.

Here, MBrace CFK laminates of (50/1.4mm) was used. 6 nos. of CFk laminate of length 3.4m, 2 Nos. of CFK laminate of length 10m and 1 Nos. of Length 20m has been used in the girder for strengthening.

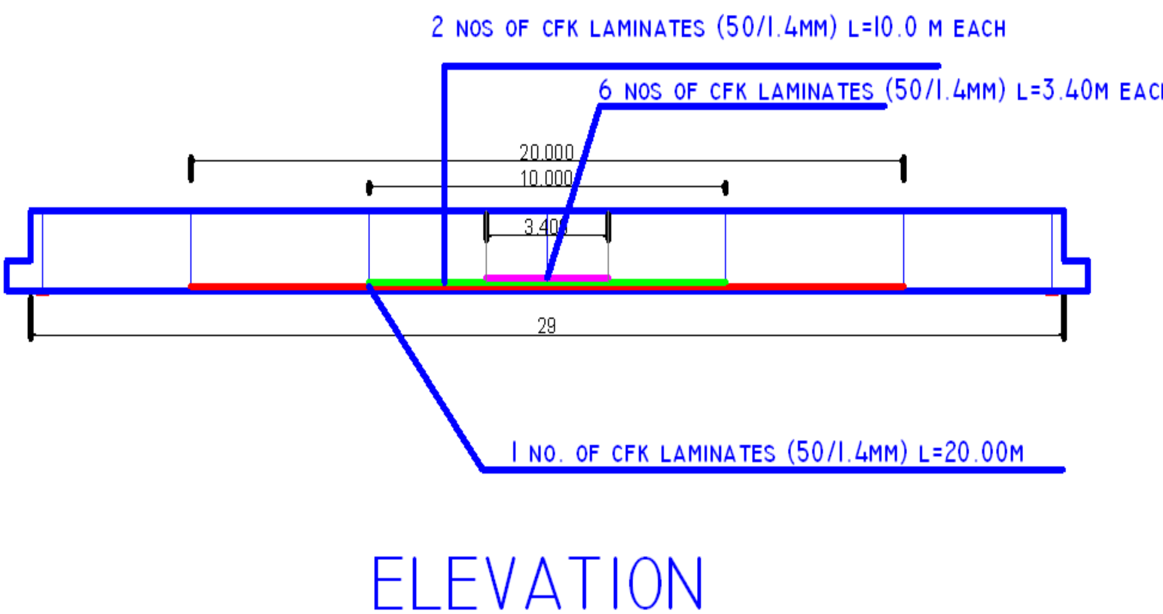


Fig.4.4 Showing the CFK laminates arrangement as per site.

5. RESULTS AND DISCUSSIONS

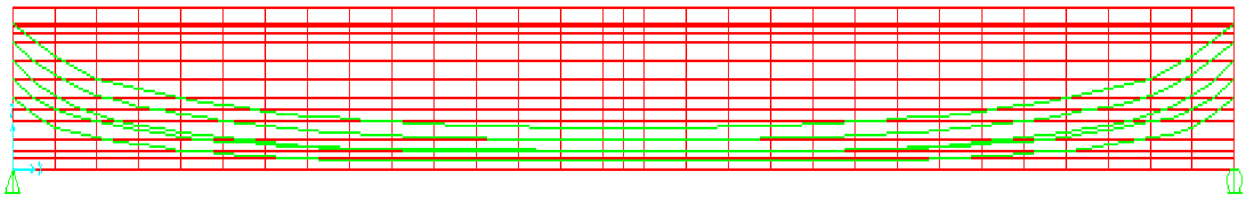


Fig.5.1 showing the longitudinal section of girder, stress at bottom of this girder is tabulated at described X distance

Table 5.1 Normal Stress S11 variation at bottom flange of left girder

DIST X	DL+NPL	DL+SPL.0	DL+SPL50	DL+SPL100
3	-7.68	-5.17	-6.92	-5.79
8	-5.84	-3.39	-5.12	-4
13	-4.75	-2.36	-4.07	-3.04
14.5	-4.754	-2.35	-4.08	-3.06
16	-5.02	-2.57	-4.34	-3.31
22	-7.23	-4.55	-6.53	-5.46
25	-9.01	-6	-8.24	-7.03

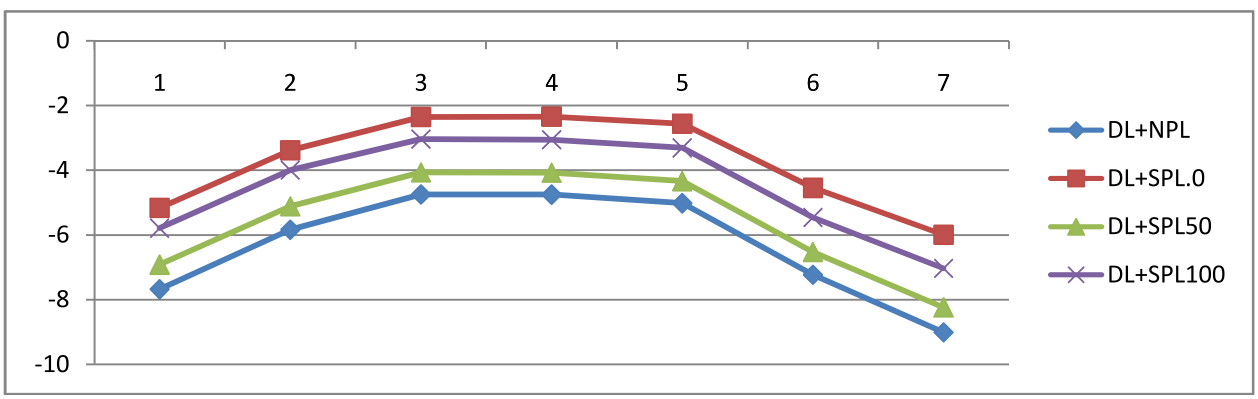


Figure 5.2 Graph comparing normal stress variation in different slip condition

SPL0 = All prestressing cable normal, but P1 with 100mmslip and P3 & P5 not workable

SPL50 = All prestressing cable normal, but bottom P1,2,3,4,5,6 are slip by 50 mm

SPL100 = All prestressing cable normal, but bottom P1,2,3,4,5,6 are slip by 100 mm

From Above table and graph, variation of the normal stress at different condition of prestressing cable is defined. This shows that the slip condition of SPL0 gives lowest strength compare with other condition and normal condition. So further analysis was done, considering slip condition SPL0.

Now, the Analysis was again done for the bridge with Slip condition SPL0, and different load combination is done as shown in clause 4.4. The selected results in terms of Normal stress are tabulated below.

From the analysis result data tabulated, the load combination DL+SPL+BM 10WL (Com S15) gives the maximum normal stress 3.97Mpa in the bottom flange at middle section of the main girder at two lane traffic. Which seen that double lane load can't be allow as the stress in the bottom flange exceed relative stress capacity of the girder in Normal condition which is 1.57Mpa obtain from load Combination 7 in (Table 5.2). From the load combination of DL+SPL+BM 10WL1 that is single lane load only, it results that single lane load can be allow which gives the stress of 1.29Mpa that is below the normal value 1.57 MPa. In Graph stress value below 1.57 only can be allow and safe for the bridge without retrofitting so the stress values for double lane load, in slip conditions exceed the normal stress of value 1.57Mpa.

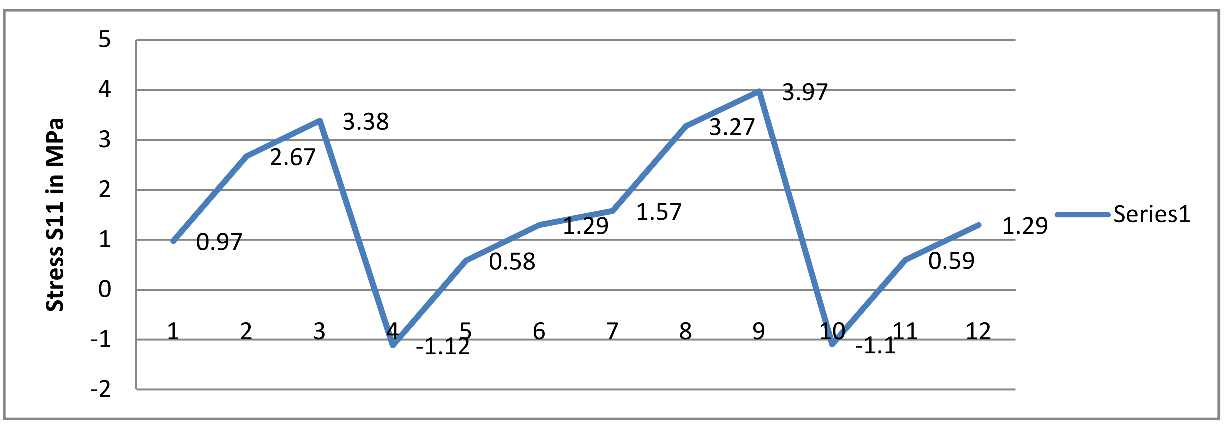


Fig.5.3. Showing Stress S11 at middle of the bottom flange of the main girder

Table 5.2. Normal Stress S11, Obtain from different load Combination for maximum bending

DIST. X(m)	D.N.CA	D.S100.CA	D.S0.CA	D.N.CA1	D.S100.CA1	D.S0.CA1
	Com.6	Com.24	Com.14	Com.10	Com.28	Com.20
3	-6.57	-4.65	-4.07	-7.06	-5.15	-4.56
5	-4.95	-3.07	-2.48	-5.79	-3.91	-3.32
8	-2.51	-0.68	-0.06	-3.8	-1.97	-1.35
10	-0.98	0.78	1.42	-2.58	-0.81	-0.017
12	0.06	1.79	2.45	-1.75	-0.023	0.64
13	0.52	2.23	2.91	-1.4	0.31	0.99
14	0.8	2.52	3.22	-1.23	0.49	1.2
14.5	**0.97**	**2.67**	**3.38**	**-1.12**	**0.58**	**1.29**
16	0.45	2.16	2.9	-1.53	0.18	0.92
18	-0.48	1.24	2.03	-1.95	-0.62	0.164
20	-1.71	0.045	0.89	-2.88	-1.64	-0.79
22	-3.32	-1.54	-0.64	-4.17	-2.98	-2.07
25	-6.44	-4.45	-3.43	-6.27	-5.39	-4.37

dist X	**D.N.10W**	D.S100.10w	**D.S0.10w**	D.N.10w1	D.S100.10w1	D.S0.10w1
	Com.7	Com.25	Com.15	Com.11	Com.29	Com.21
3	**-5.58**	-3.66	-3.07	-6.51	-4.59	-4.01
5	**-3.34**	-1.46	-0.86	-4.91	-3.03	-2.44
8	**-1.07**	0.76	1.38	-3.11	-1.28	-0.66
10	**0.19**	1.96	2.6	-2.1	-0.33	0.3
12	**0.93**	2.67	3.33	-1.51	0.22	0.88
13	**1.26**	2.98	3.66	-1.257	0.46	1.14
14	**1.45**	3.17	3.87	-1.175	0.55	1.24
14.5	**1.57**	**3.27**	**3.97**	**-1.1**	**0.59**	**1.29**
16	**1.06**	2.77	3.5	-1.51	0.2	0.94
18	**0.81**	1.8	2.59	-2.3	-0.58	0.2
20	**-1.21**	0.54	1.39	-3.36	-1.6	-0.76
22	**-2.81**	-1.04	-0.013	-4.67	-2.89	-1.98
25	**-5.91**	-3.93	-2.9	-7.213	-5.23	-4.2

The existing bridge have decreased its capacity, in which only one way traffic can be allowed at a time, So after using the FRP laminates its strength can be increased by which bridge can permits two way traffic at a time. Table below shows the increase in the strength of the bridge after Strengthening with FRP and the stress developed due to double lane traffic is within the relative stress to the normal condition tabulated in table 5.3.

Table 5.3. Normal Stress S11, at bottom Flange after 3 layer FRP laminates

dist X	D.S100.BM 10WL	D.S100.CA	D.S0 BM 10WL	D.S0 CA
	Combn.25	Combn.24	Combn.15	Combn.14
3	-2.61	-3.54	-2.02	-2.95
5	0.33	-1.112	0.92	-0.53
8	1.442	0.83	1.74	1.123
10	1.545	1.21	**1.83**	1.45
12	1.207	1.031	1.36	1.18
13	0.92	0.81	1.03	0.92
14	0.467	0.4	0.57	0.47
14.5	0.342	0.3	0.39	0.35
16	0.882	0.8	1.02	0.92
18	1.13	0.85	1.117	1.01
20	1.34	1.3	**1.95**	1.71
22	0.99	0.31	0.96	0.73
25	-0.783	-3.29	-1.47	-2.09

Table 5.4 Normal .Stress S11, at bottom Flange after 2 layer FRP laminates

dist X	D.S100.BM 10WL	D.S100.CA	D.S0 BM 10WL	D.S0 CA
	Combn.25	Combn.24	Combn.15	Combn.14
3	-2.883	-3.88	-2.28	-3.275
5	-0.12	-1.75	0.51	-1.12
8	**2.49**	1.11	**3.115**	1.73
10	**3.8**	**2.78**	**4.37**	**3.37**
12	1.08	0.91	1.22	1.051
13	0.82	0.71	0.92	0.81
14	0.4	0.37	0.475	0.42
14.5	0.3	0.27	0.35	0.312
16	0.783	0.7	0.89	0.81
18	0.7	0.6	0.8	0.743
20	**3.47**	**2.87**	**4.48**	**3.881**
22	0.87	0.328	1.83	1.28
25	-2.92	-3.45	-1.88	-2.41

Table 5.5 Normal Stress S11, at bottom Flange after 1 layer FRP laminates

dist X	D.S100.BM 10WL	D.S100.CA	D.S0 BM 10WL	D.S0 CA
	Combn.25	Combn.24	Combn.15	Combn.14
3	-3.25	-4.23	-2.657	-3.65
5	-0.75	-2.37	-0.15	-1.77
8	1.93	0.49	2.57	1.12
10	**3.48**	**2.29**	**4.14**	**2.96**
12	5	4.04	5.17	4.8
13	3.76	3.18	4.29	3.71
14	0.212	0.18	0.25	0.21
14.5	0.27	0.22	0.32	0.27
16	**3.583**	**3.12**	**4.2**	**3.7**
18	**3.1**	**2.57**	**3.87**	**3.33**
20	1.771	1.27	2.62	2.12
22	-0.06	-0.06	0.85	0.34
25	-3.351	-3.88	2.32	-2.85

From table 5.3 and Figures B4,B5,B6, it shows that after applying the three layers of laminates as shown in Fig 11. It becomes safe for double lane traffic. The maximum stress from table 5.3 are 1.95 MPa which is also in limit of $1/10^{th}$ of concrete strength 0f 40Mpa.

Form Table 5.4 & 5.5 it shows vehicle in two lanes cannot be allowed.

6. CONCLUSIONS

Bridge is modeled in 3 D in SAP. Main girder is modeled as 3D solid, cross girder; deck slab is modeled as shell element. Also frame element is used as lane for track load transferring as Live load.

Class A and 10 wheeled truck load is used as live load. And so arranged for Maximum shear force and bending moment. From this analysis (Figures in Appendix B), following points is seen:-

1. This Bridge, if not retrofitted with FRP, only one way traffic is allowed, as its two prestressing tendons are badly damaged and totally slipped at anchorage block.
2. MBrace CFK laminates has been provided in the bottom flange of the girder, as it is light in weight and has greater tensile strength. From table 5.5, Only 3.4m length of FRP laminate was provided, but the stress value show that the portion within the laminated area is very much safe in flexural strength, but the stress is concentrated just at the edges of the laminate, and those stress value is very much higher which shows that the miss use of the laminate can bring disaster.
3. In table 5.4, the stress values are obtain after the laminated area is extended to 10 m in middle portion, which shows laminated area is again insufficient and stresses are concentrated just at end of the lamination.(see Figures in Appendix B)

7. RECOMMENDATION

During my thesis work, I did little work on various profile of tendons arrangement. I got some changes in stresses in concrete. But detail work on profiles can bring better benefit in concrete strength. Also due to time restriction I was not able to apply blast loading so that we can understand the damage procedure that had happen in this bridge. This can give idea to understand the critical position of bridge which can be design strong enough to resist blast loading.

REFERENCES

1. David G. Winget, Kirk A. Marchand, Eric B.Williamson, "Analysis and Design of Critical Bridges Subjected to Blast Loads"
J. Structural Engineering Volume 131, Issue 8, pp. 1243-1255 (August 2005)
2. A. Nanni, P. C. Huang, and J. G. Tumialan ,
"STRENGTHENING OF IMPACT-DAMAGED BRIDGE GIRDER USING FRP LAMINATES"
Center for Infrastructure Engineering Studies ,University of Missouri – Rolla
Rolla, MO 65409, USA
3. C. A. Ross, L. C. Muszynski, D. M. Jerome, J. W. Tedesco, R. L. Sierakowski
"RC Beams and Slabs externally reinforced with Fiber Reinforced Plastic (FRP) panels"
4. "Standard Plans for Highway bridges" Volume-II one end stressing
Prestressed concrete Beams and R.C.C Slab type Superstructure
The Indian Road Congress
5. A.K.M. Anwarul Islam (2005). "Performance of AASHTO Girder Bridges under Blast Loading"
6. Eray Baran, Arturo E. Schultz, Catherine E. French "Evaluation and Modifications of the AASHTO Procedures for Flexural Strength of Prestressed Concrete Flanged Sections"
7. N.Krishna Raju, "Prestressed Concrete". Fourth Edition
Mc Graw-Hill Publishing Company limited, 2007 New Delhi
8. IS: 1343-1980, Indian Standard Code of Practice for Prestressed Concrete (First Revision), Indian Standards Institute, New Delhi, 1981
9. Thomas Telford "Repair and Strengthening of Concrete Structures"
10. E.Pennells "Concrete Bridge Designer's manual"
11 V.KRaina "Concrete Bridge Practice" Construction. Maintenance and rehabilitation.

12. Owen Rosen boom, Tare K. Hassan, Sami Rizkalla
"Flexural behavior of aged prestressed concrete girder Strengthened with various FRP systems"
Department of Civil Engineering, North Carolina State University, Centennial Campus, Raleigh, NC27965-7533, United States
13. Modjeska and Masters, Inc. November 2003 "COMPREHENSIVE DESIGN EXAMPLE FOR PRESTRESSED CONCRETE (PSC) GIRDER SUPERSTRUCTURE BRIDGE WITH COMMENTARY"
(THE FEDERAL HIGHWAY ADMINISTRATION)
14. Charles W. Dolan, H.R. Hamilton III (University of Wyoming), Charles E. Bakis (Pennsylvania State University) & Antonio Nanni (University of Missouri - Rolla) "DESIGN RECOMMENDATIONS FOR CONCRETE STRUCTURES PRESTRESSED WITH FRP TENDONS." FHWA CONTRACT August 1, 2001
15. AMIR MIRMIRAN et.al, "Bonded Repair and Retrofit of Concrete Structures Using FRP Composites" Department of Civil Engineering North Carolina State University
16. Antonio Nanni, PhD, PE "Strengthening of an impact- damaged PC Girder"
17. Damian I. kachlakev, Ph.D., P.E. "Finite Element analysis of historic Bridge strengthened with FRP laminates" California Polytechnic State university
18. IITK-GSDMA GUIDELINES on MEASURES TO MITIGATE EFFECTS OF TERRORIST ATTACKS ON BUILDINGS
C. V. R. Murty, Indian Institute of Technology Kanpur
19. IS: 4991-1968, Criteria for blast Resistant Design of Structures for Explosions above Ground
20. IS: 1343-1980, Indian Standard Code of Practice for Prestressed Concrete (First Revision), Indian Standards Institute, New Delhi, 1981
21, IRC 6-2000, "Standard Specifications and Code of Practice for road Bridges" section II, Loads and Stresses (Fourth revision) The Indian Roads Congress.

APPENDICES

APPENDIX A

NON DESTRUCTIVE TEST RESULTS

Grade of concrete was determined by non destructive test.

To predict the original grade of concrete, Schmidt hammer test has been performed at site so that it can be used for analysis.

Observation and calculation:

Table A.1 Rebound values:

Test points	Rebound number					Average
	1	2	3	4	5	
1	41	40	38	42	42	**40.6**
2	48	50	42	36	41	**43.4**
3	41	32	46	41	44	**40.8**
4	40	41	41	40	45	**41.4**
5	43	42	48	41	42	**43.2**
6	42	37	38	40	38	**39**
7	44	48	40	40	42	**42.8**
8	44	40	42	43	38	**41.4**
9	48	49	46	27	30	**40**
10	32	40	34	46	33	**37**

Development of correlation chart

[redacted]

S.N	Rebound number	Compersive strength
	(Average)	(Mpa)
1	**38.4**	60.00
2	**41.1**	41.00
3	**38.7**	56.40
4	**40.3**	55.50
5	**32.8**	26.00

The calibration chart is shown in figure below. A best-fit line has been shown in the chart.

From chart: **f = 2.8618*R-61.712** Correlation equation

Where, f = Compressive strength in N/mm^2 & R= Rebound number

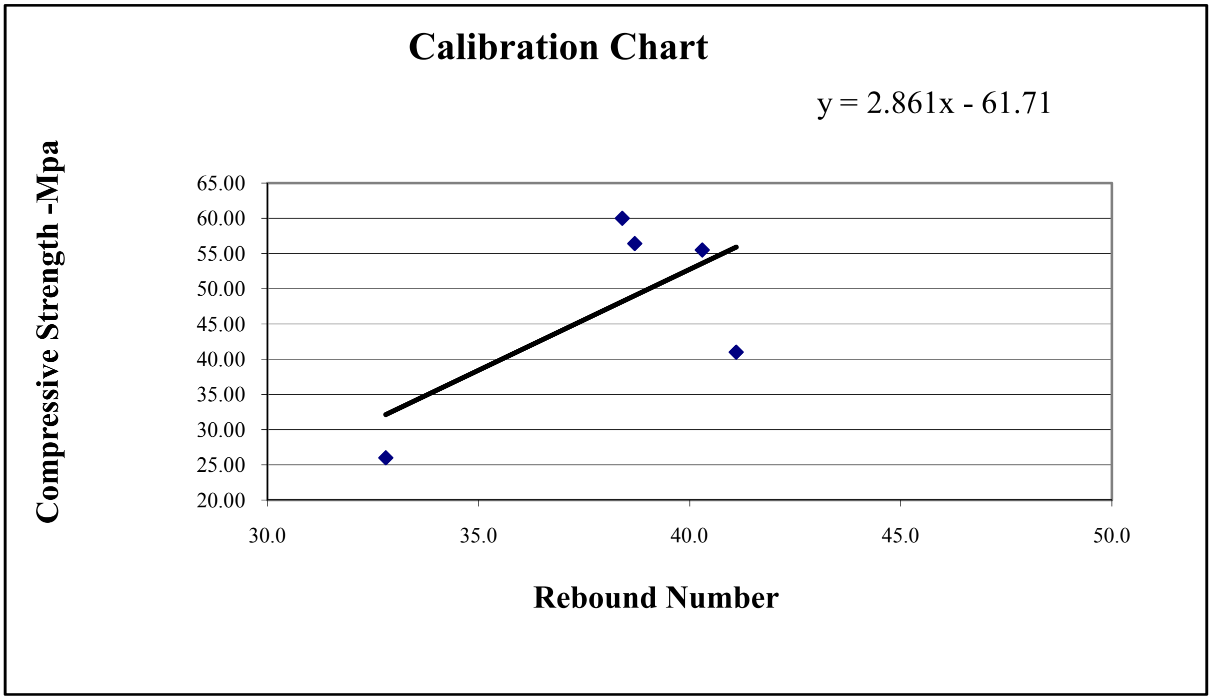


Figure A.1 Calibration chart for compressive strength

Cube strength from average rebound values:

Test points	Average rebound values	Cube strength (N/mm²)
1	40.6	54.48
2	43.4	62.49
3	40.8	55.05
4	41.4	56.77
5	43.2	61.92
6	39	49.90
7	42.8	60.77
8	41.4	56.77
9	40	52.76
10	37	44.17
	Mean	**55.51**
	S.D	**5.38**

Cube strength of rebound value at different confidence level

Confidence level	Relation	**Compressive strength (Mpa)**
95 %	55.51-1.64*5.38	**46.69**
90 %	55.51-1.28*5.38	**48.62**

APPENDIX B
ANALYSIS RESULT AND FIGURES

Longitudinal section at the middle of the girder showing the stress variation due to 3 layer FRP laminate at the bottom flange of girder. (Fig 13, 14, 15& 16)

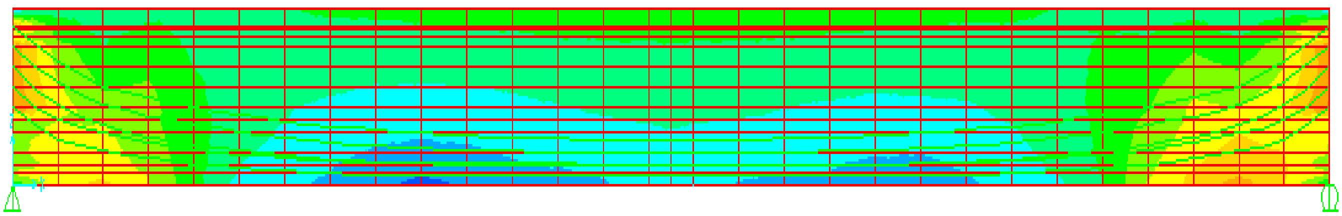


Fig.B.1. Load combination of DL+SPL100+BM 10 wheel load double lane.

Fig.B.2. Load combination of DL+SPL100+Class A double lane.

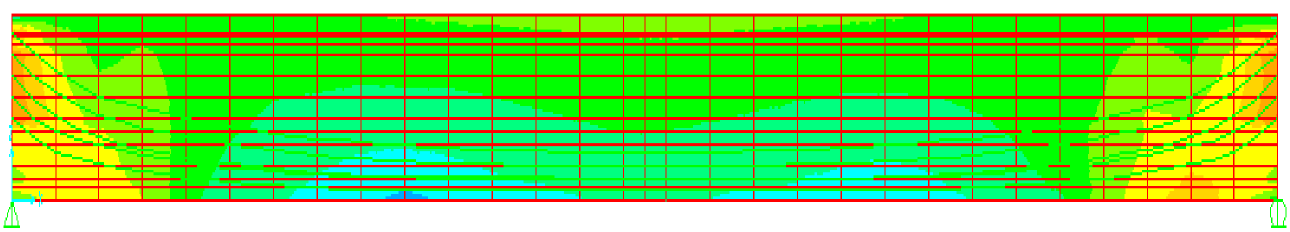


Fig.B.3. Load combination of DL+SPL0+BM 10 wheel load double lane.

Fig.B.4. Load combination of DL+SPL0+Class A double lane.

Longitudinal section at the middle of the girder showing the stress variation due to 2 layer FRP laminate at the bottom flange of girder. (Fig B.4)

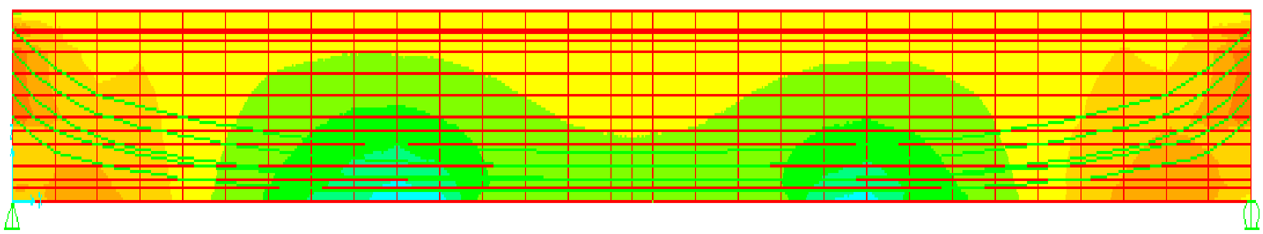


Fig.B.5. Load combination of DL+SPL0+BM 10 wheel load double lane.

Longitudinal section at the middle of the girder showing the stress variation due to 1layer FRP laminate at the bottom flange of girder. (Fig B.5)

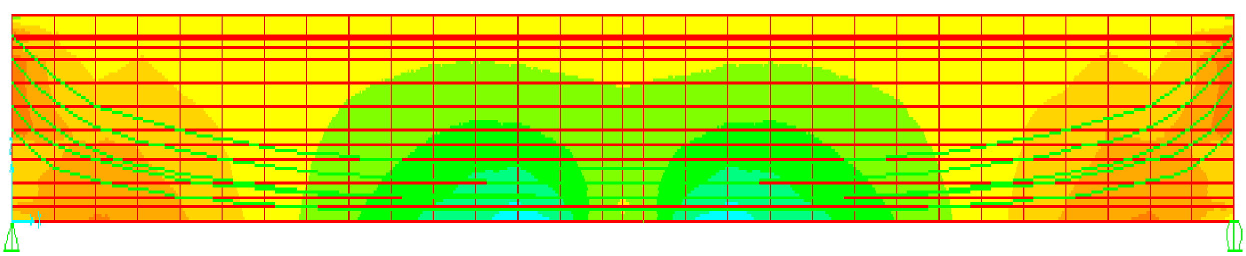
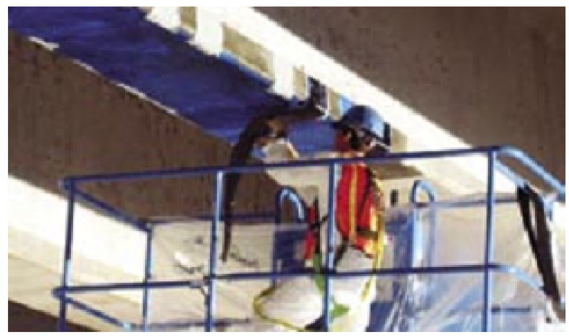


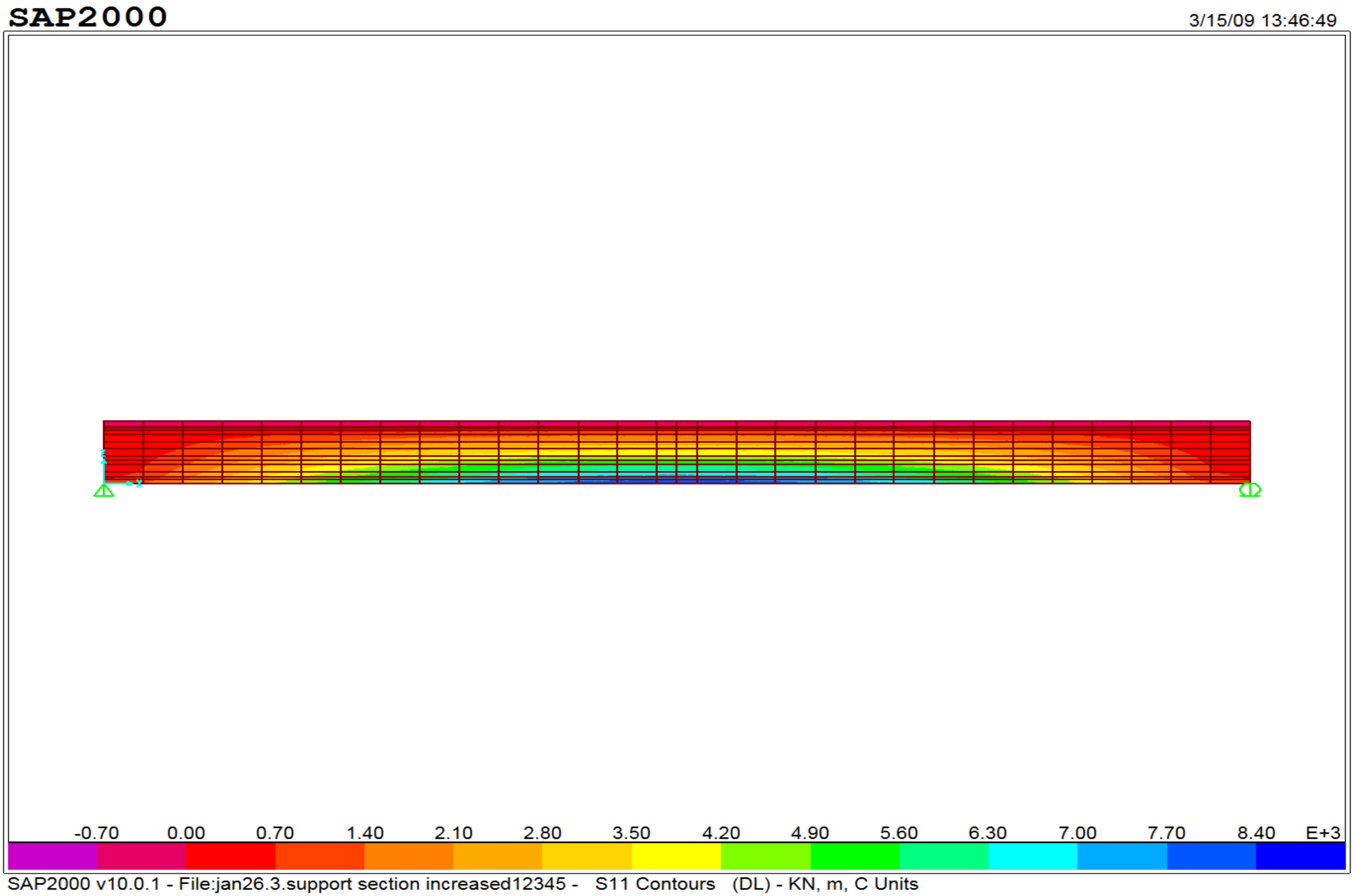
Fig.B.6 Load combination of DL+SPL0+Class A double lane.

Application of FRP laminates in Bridge Girder (not in Nepal)

Surface preparation



Application of U-wraps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