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Decision Support Tools for Concrete Infrastructure Rehabilitation (using FRP composites)

Report 2002-005-C-05

Industry focussed Summary

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Research Program C:

Delivery and Management of Built Assets

Project 2002-005-C

Decision Support Tools for Concrete Infrastructure rehabilitation

Content Summary

List of tables

List of figures

EXEXUTIVE SUMMARY

This report presents a summary of the research conducted by the research team of the CRC project 2002-005-C, “Decision support tools for concrete infrastructure rehabilitation”.

The project scope, objectives, significance and innovation and the research methodology is outlined in the introduction, which is followed by five chapters covering different aspects of the research completed.

Major findings of a review of literature conducted covering both use of fibre reinforced polymer composites in rehabilitation of concrete bridge structures and decision support frameworks in civil infrastructure asset management is presented in chapter two. Case study of development of a strengthening scheme for the “Tenthill Creek bridge” is covered in the third chapter, which summarises the capacity assessment, traditional strengthening solution and the innovative solution using FRP composites. The fourth chapter presents the methodology for development of a user guide covering selection of materials, design and application of FRP in strengthening of concrete structures, which were demonstrated using design examples.

Fifth chapter presents the methodology developed for evaluating whole of life cycle costing of treatment options for concrete bridge structures. The decision support software tool developed to compare different treatment options based on reliability based whole of life cycle costing will be briefly described in this chapter as well.

The report concludes with a summary of findings and recommendations for future research.

1 INTRODUCTION

In Australia, over 60% of bridges for local roads are over 50 years old and approximately 55% of highway bridges are over 20 years old (Stewart 2001). Strengthening or rehabilitation of existing bridge structures require careful analysis of capacity of the existing structure, identification of the deficiencies and then selection of the most efficient solution for treatment of the bridge. In selecting a treatment option, the decision maker has a number of accepted solutions as well as emerging solutions available to them. Innovative and emerging solutions usually have a low perceived reliability compared to proven solutions, which has to be considered in decision-making.

The project has addressed two major issues in supporting decision making in rehabilitation of concrete infrastructure. A design guideline has been developed for use of Fibre Reinforced Polymer Composites in rehabilitation of concrete bridge structures, combining outcomes of published research work and provisions and expectations of the Austroads Bridge Design Code as well as the Australian concrete structures code. A decision making tool for comparing whole of life cycle costing of different treatment options considering different elements of costs to both the road authority and the user as well as risk of probability of failure and the corresponding cost has been developed.

1.1 Objectives

The objective of the project was to develop a decision support tool to enable asset managers of concrete infrastructure to select the most suitable technique for rehabilitation of aging concrete structures. Specifically the project focussed on using FRP composites in strengthening of concrete bridges although the decision support methodology is sufficiently generic and easily adaptable for selection of other options. The objectives were achieved in the form of two major deliverables:

- User friendly guideline for rehabilitation of concrete bridge structures using FRP composites (Hardcopy)
- Decision support software tool for comparing different treatment options using whole of life cycle costing which incorporated cost of failure as a measure of reliability of the selected option.

1.2 Significance and Innovation

The user friendly guide, was specifically developed for concrete bridge structures. However, it can be easily adopted for other concrete structures as well. The decision support software tool has the flexibility of being adaptable to any infrastructure asset management situation. Therefore, the project has made a significant contribution towards effective management of civil infrastructure assets.

Research team has reviewed published literature on use of FRP composites in rehabilitation or strengthening of bridge structures as well as decision support frameworks that can be adopted for bridge rehabilitation. Potential solutions have been identified and were summarized. Through a detailed analysis of a case-study, the team has then developed a strengthening solution for a concrete bridge headstock. In developing the solution, the research team has considered the level of risk, which the authority is prepared to accept, and methods of translating research outcomes into design guidelines complying with Austroads requirements.

The decision support software tool developed for whole of life cycle costing has been extremely innovative in the methodology as well as the application. The whole of life costing of a given treatment option is determined using probabilistic evaluation of the Net present Value of a given scenario which is then combined with the cost of failure to account for the risk of a given option.

1.3 Research methodology

The comprehensive research methodology covered a number of tasks. Major elements can be summarized as:

- A critical analysis of national and international practices on use of FRP composites in rehabilitation of concrete structures.
- Consultation with suppliers of FRP materials to identify practical issues
- Structural capacity assessment methods prior to application of FRP
- Design methodology development encapsulating outcomes of published research and limit state design philosophy
- Case study of development of a strengthening solution
- Development of a methodology for whole of life costing incorporating reliability
- Development of a software tool and calibration

These elements are discussed in detail in the following chapters.

2 REVIEW OF PUBLISHED RESEARCH WORK

2.1 Use of FRP composites in structural strengthening

The report 2002-005-C-01 has covered a detailed analysis of published research work. In conclusion, Table 2.1 has been developed summarizing the findings of the review. In all the applications identified here, materials, design methodology and application issues have been identified and reported in 2002-005-01.

Table 2-1 Summary of strengthening techniques

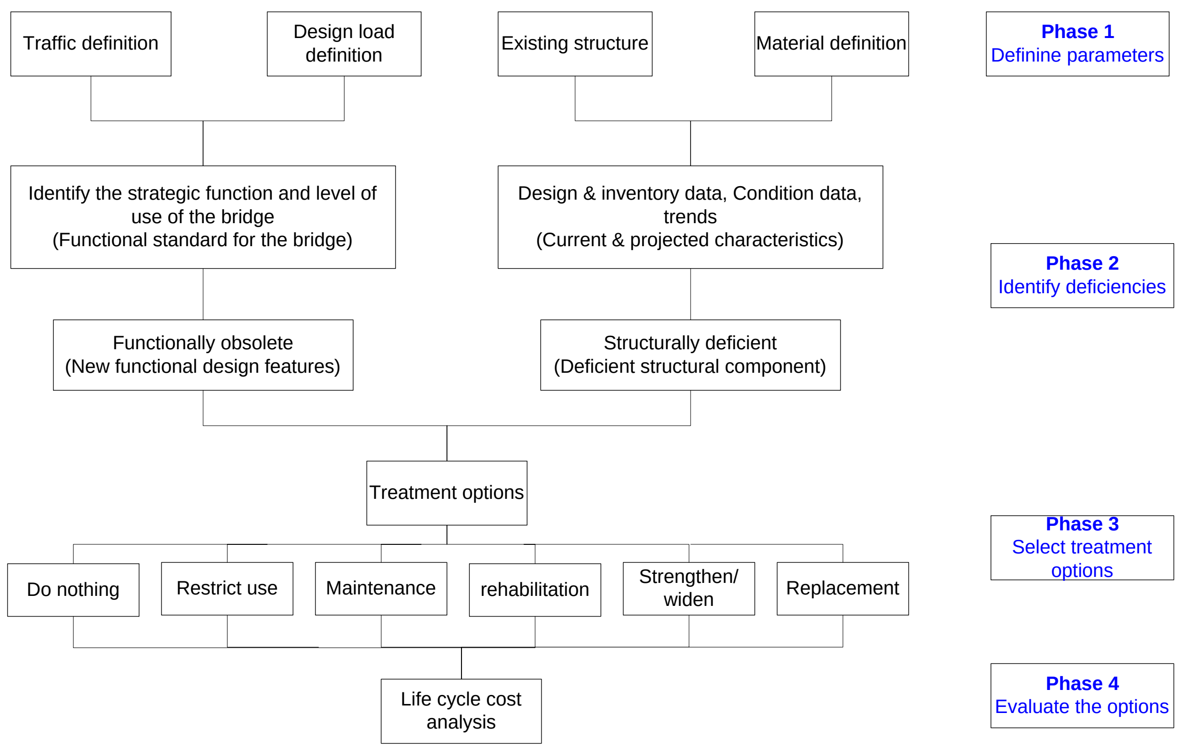
Strengthening Method	Design Action	Type of FRP	Special Considerations
Wet lay up of FRP sheets to the tension zone of the soffit of a beam or slab	Flexural strengthening	Sheets or strips	De-bonding
Attaching prefabricated FRP sheets to the tension zone of the soffit of a beam or slab	Flexural strengthening	Sheets or strips	De-bonding
The different types of wrapping schemes to increase the shear strength of a beam or column	Shear strengthening	Sheets	Direction of fibre
Automated winding of wet fibre under a slight angle around columns or other structures,	Shear and axial compression strengthening	Sheets	Equipment availability
Attaching prestressed FRP strips to the tension zone of the soffit of a beam or slab	Flexural strengthening	Strips	Anchorage
Fusion-bonded pin-loaded straps	Flexural and shear strengthening	Pin-loaded Straps	Equipments availability
In-situ fast curing using heating device	Flexural strengthening	Strips	-
Prefabricated U or L shape strips for shear strengthening	Shear strengthening	Strips	Direction of fibre
Bonding FRP strips inside concrete slits	Flexural strengthening	Strips	-
FRP impregnation by vacuum to the tension zone of the soffit of a beam or slab	Flexural Strengthening	Strips	Equipments availability
Prefabricated FRP shells or jackets for the confinement of circular or rectangular columns	Axial compression strengthening and ductility enhancement	Sheets	-
FRP wrapping for axial compression strengthening and ductility enhancement	Axial compression strengthening and ductility enhancement	Sheets	-

FRP wrapping for torsional strengthening	Torsional strengthening	Sheets	Direction of fibers

2.2 Decision Support using Life Cycle Costing

After a comprehensive review of literature covering decision-making in bridge asset management, the framework shown in the Figure 2-1 has been developed with a significant contribution from project participants. The research methodology and the development of the framework was published in Nezamian et al (2004 a).

Figure 2-1 Flow-chart for the rehabilitation of bridge structures



3 CASE STUDY

3.1 Description of the Case Study

The case study was selected after a number of discussions with the Queensland Department of Main Roads and identifying that the headstock of reinforced and pre-stressed concrete bridge structures is currently the weak link, which requires strengthening to satisfy the current requirements of traffic and other loading.

The bridge studied in this report used to carry traffic between Ipswich and Toowoomba over Tenthill Creek in Gatton (Gatton Helidon Rd), Queensland, Australia. This simply supported reinforced concrete, pre-stressed-beam structure was built in 1970’s. The bridge is 82.15 m long and about 8.6 m wide and is supported by a total of 12 pre-stressed 27.38 m long beams over three spans of 27.38 m. Side and cross views of the Tenthill Bridge are shown in Figure 3.1. The beams are supported by two abutments and two headstocks.

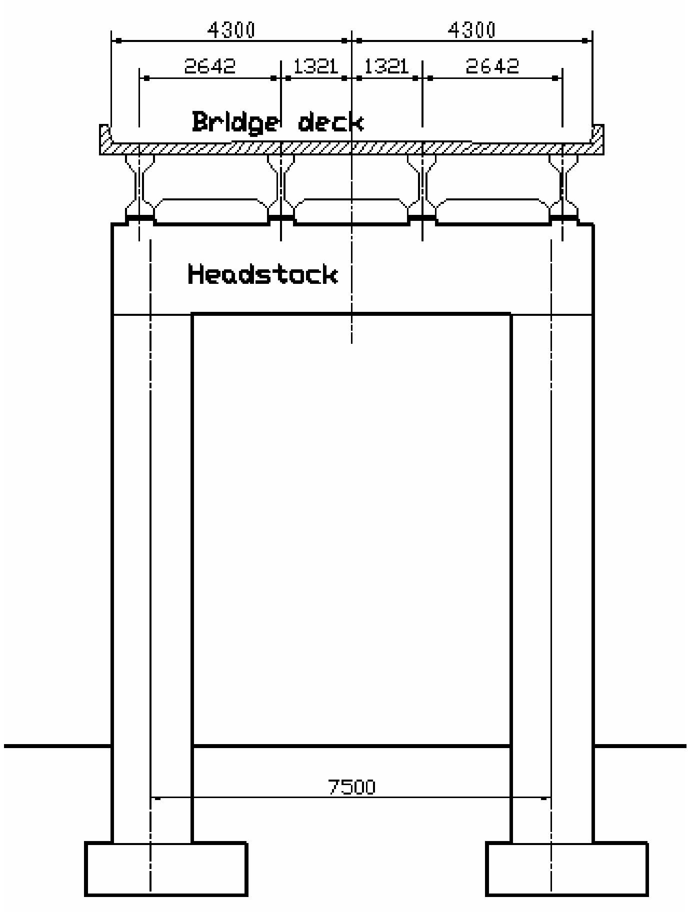
Figure 3-1 Side and cross views of the Tenthill Bridge

3.2 Preliminary structural assessment

The first stage of the rehabilitation of the bridge headstock is identifying the headstock deficiencies. Queensland Department of Main Roads (QDMR) has a comprehensive asset management system of inspections, condition data, analysis and prioritization tools, maintenance manuals, and heavy load routing systems (Fenwick & Rotolone 2003).

The asset management system aims to maintain the bridges in a condition that allows heavy vehicles free access to all parts of the network. In other words, avoid placing load restrictions on any bridge in the primary (state-controlled roads) network. The Tenthill Bridge has been observed to require immediate strengthening to avoid such restriction.

Figure 3-2 Schematic view of the headstock



The suitable rehabilitation system for the headstock is decided based on condition assessment of the existing structure including establishing its existing load-carrying capacities, and determining the condition of the concrete substrate. The overall evaluation included a thorough field inspection, review of existing design or as-built documents, and a structural capacity analysis in accordance with AS 3600 and Austroads Bridge Design Code (1992). Existing construction and operational documents for the bridges were reviewed, including the design drawings, project specifications, as-built information and past repair documentation. Austroads bridge design code (1992) was used for assessment of the bridge to ascertain the capacity of the bridge. The Gatton Helidon road over Tenthill Creek is selected as functional Class 3 from the Table 2.3.4 of Austroads code (1992).

3.3 Structural analysis

The headstock has been analysed as a portal frame considering all necessary design situations and load combinations according to Austroad Bridge Code (1992) for ultimate limit state and serviceability limit state. The grillage analysis (lane analysis) was used to calculate traffic load on the headstock.

The traffic loading models of T44 and Heavy Load Platform HLP 320 in one and two lanes were used in grillage analysis. The computer program SAP2000 has been used for structural analysis. Pre-stressed beams were analysed as simply supported beams to determine the applied dead load from the secondary beams on the headstock.

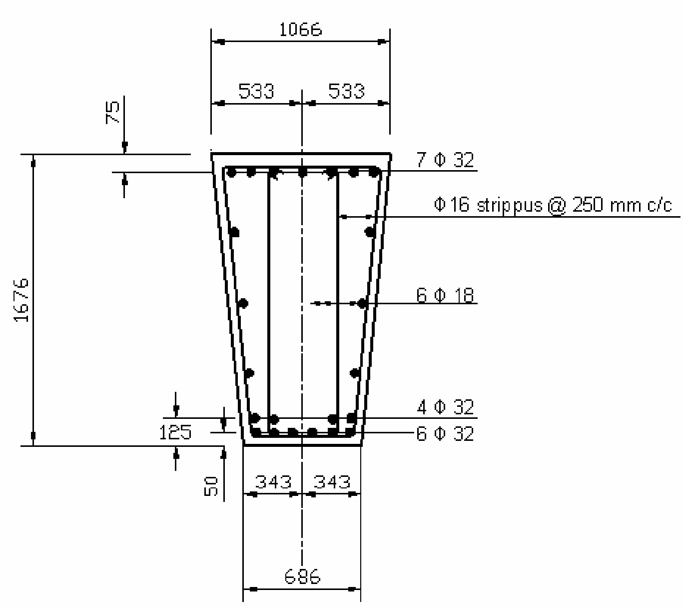
The strengthening target for ultimate bending moment and shear force resulted from combination of ultimate traffic loads of HLP 320 and permanent effect (dead load). The ultimate bending moment of 5520 kN-m and ultimate shear force of 2525 kN and serviceability bending moment of 2526 kN-m and serviceability shear force of 1797 kN were then calculated for the load combination.

3.4 Existing Capacity of the headstock

In accordance with the Australian codes of practice for structural design, the capacity analysis methods contained in this section are based on ultimate limit-state philosophy. This ensures that a member will not become unfit for its intended use. The capacity analysis results would be compared with structural analysis results to identify the deficiencies. This approach sets acceptable levels of safety against the occurrence of all possible overload situations. The nominal strength of a member is assessed based on the possible failure modes and subsequent strains and stresses in each material.

A typical beam section of the headstock is shown in Figure 3-3. The positive and negative flexural and shear capacities of the section were calculated in accordance with Australian standards AS 3600. The nominal steel–rebar areas, nominal steel yield strength of 400 MPa for longitudinal reinforcement and 240 MPa for shear reinforcement and nominal concrete compressive strength of 20 MPa were used in the section capacity analysis. The degradation due to corrosion of the steel and creep and shrinkage of the concrete were ignored.

Figure 3-3 Beam cross section



The following assumptions form the basis for the calculation of the ultimate strength of the concrete element strengthened in flexure.

- Design calculations are based on the actual dimensions, internal reinforcing steel arrangement, and material properties of the existing member.
- The strain in reinforcement and concrete are directly proportional to the distance from the neutral axis, that is, a plane section before loading remains plane after loading.

3.5 Strengthening Using External Post tensioning

External post-tensioning of concrete members has been effectively used to increase the flexural and shear capacities of both reinforced and pre-stressed concrete members. With this type of upgrading, the external forces are applied to the structural member using a post-tensioned cable to resist part of the internal forces caused by applied loads. This method is effective and economical for long span beams due to the negligible additional weight of the repair system and use of existing material. QDMR has successfully employed post-tensioning to enhance flexural and shear capacities, control and correct excessive deflections and, cracking of reinforced concrete and pre-stressed concrete members of bridge structures. QDMR has decided to use post tensioning strengthening techniques to increase shear and flexural capacity of the headstock of Tenthill Bridge.

The post-tensioned reinforcement is exposed; hence the issues such as corrosion, fire and aesthetics need to be considered. These issues can be resolved by encasement of the strengthening system in concrete, grouted ducts or other protection methods.

3.5.1 Design of strengthening system

For post-tensioning strengthening of the bridge headstock, wire strands or bars can be used as the pre-stressing tendons. In this particular case study, based on the preliminary calculations it was decided to use VSL McAlloy steel bars of diameter 38 mm. It is assumed that the strengthening is in accordance with the current calculations of post-tensioning method. From Table 6.3.1 of Australian Standards (AS 3600), the minimum breaking load for this particular steel bar should be 1230 kN. With 20% reduction in the breaking force, the pre-stressing force for one bar is calculated as 980 kN. Therefore the total pre-stressing force is 4 x 980 kN.

3.5.2 Flexural capacity of the strengthened member

In pre-stressed concrete beams, pre-stressing steel is tensioned during the construction. This results in a pre-compression force in concrete. In the post cracking behaviour, the pre-stressing steel in the tensile region causes a significant contribution to the moment capacity of the section. The design bending moment capacity of 6270 kN-m was then calculated based on the AS 3600 for the strengthened member using the post tensioning strengthening system.

3.5.3 Shear capacity of the strengthened member

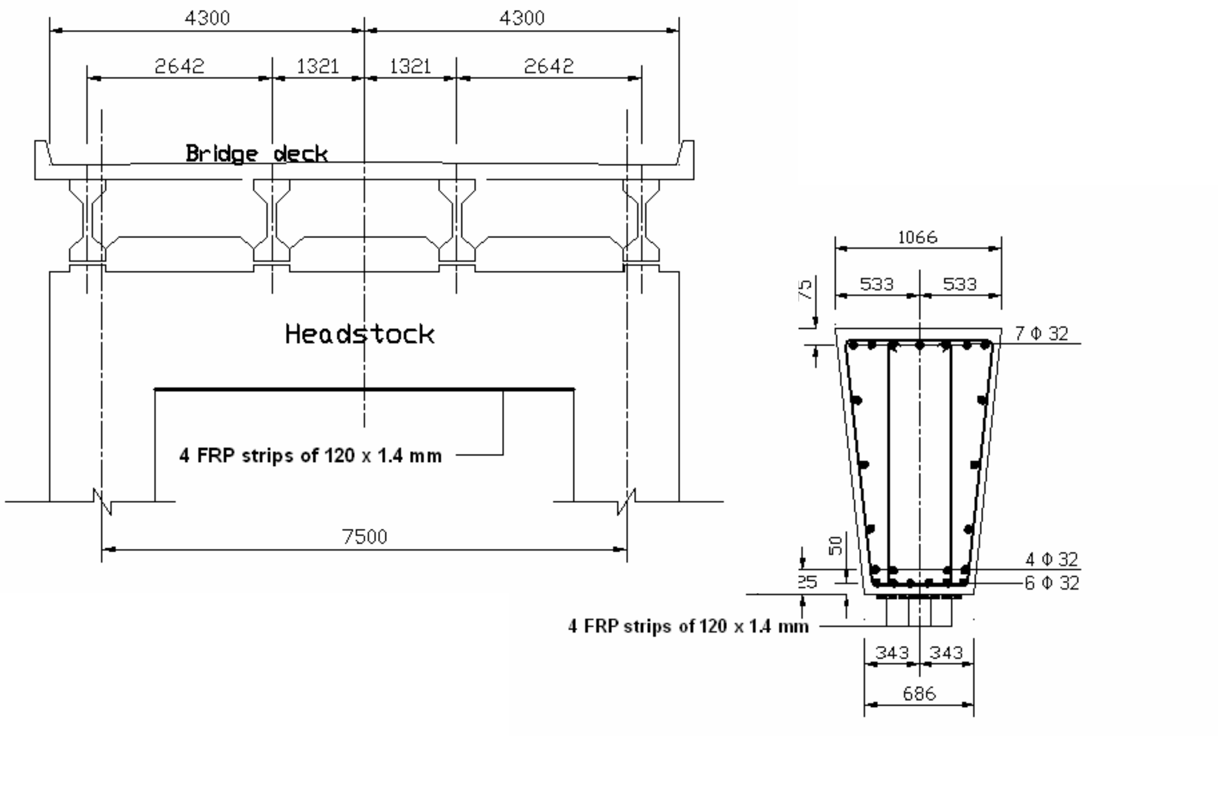
Pre-stressing has a significant influence in the load capacity in shear and shear in diagonal cracking of a pre-stressed member. The flexural and inclined crack formation can be delayed with the horizontal component of pre-stressing force. The vertical component of the pre-stressing force affects the shear force acting on concrete. Shear capacity after post-tensioning strengthening needs to be checked for the loads at both flexural-shear cracking and web shear cracking. A question is raised about sliding failure of the beam along the shear crack at high pre-stress forces. Therefore post-tensioning solution perhaps requires an upper limit imposed on the applied pre-stress force to prevent this. A search of literature did not yield any published work covering this area. A simple friction calculation can be done to establish an approximate upper limit.

3.6 FRP Strengthening of the Headstock

The design of FRP strengthening system for the Tenthill bridge headstock can be summarized as follows:

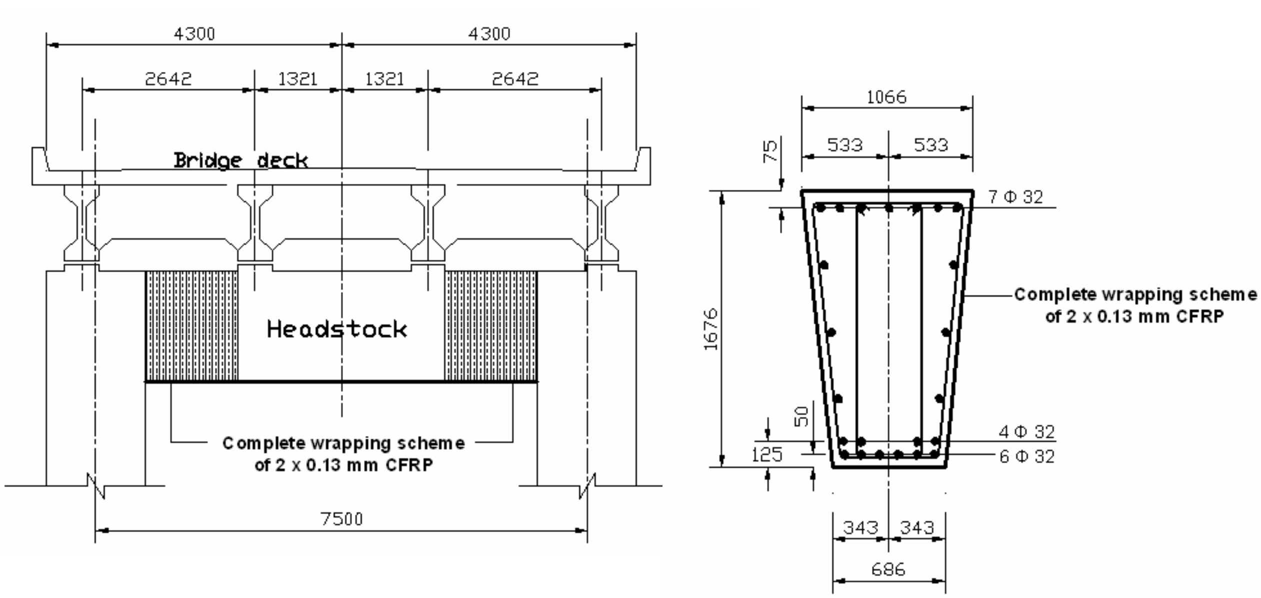
- The flexural strength of the headstock can be increased from 3800 kN-m to 5854 kN-m by bonding four FRP unidirectional strips of 120 x 1.4 mm to the tension face of the beam section (bottom fibre) of the headstock with fibres oriented along the length of the member (Figure 3-4).
- The shear strength of the headstock can be increased from 2065 kN to 2711 kN by complete wrapping of the beam with two layers of 0.13 mm thick carbon fibres oriented along the transverse axis of the beam section (Figure 3-5).

Figure 3-4 Flexural strengthening



Details of the analysis of the section and the rationale for design decisions are given in CRC report 2002-005-C-02. It has been shown that the recommendations of ACI 440 (2003) can be easily aligned with the provisions of the AustRoads Bridge design code as well as AS3600 concrete structures code. In design against anchorage failure, the provisions of the FIB (2003) were considered more appropriate and were adopted for applications with provisions of the local codes.

Figure 3-5 Shear strengthening



4 A USER FRIENDLY GUIDE FOR REHABILITATION OF CONCRETE BRIDGE STRUCTURES USING FRP COMPOSITES

CRC report 2002-005-C-04 is a “User Friendly Guide for Rehabilitation or Strengthening of Bridge Structures using Fibre Reinforced Polymer Composites”. A summary of the content of the guide is given here

4.1 Material Properties

The characteristics of FRP composites depend on many factors such as type of fibre, its orientation and volume, type of resin used and quality control used during the manufacturing process. It is possible to obtain the characteristics of commercially available FRP composites from the manufacturer. However, some generic material characteristics were described in the report.

FRP composite materials for strengthening of civil engineering structures are available today mainly in the form of:

- thin unidirectional strips (with thickness in the order of 1 mm) made by pultrusion
- flexible sheets or fabrics, made of fibres in one or at least two different directions, respectively (and sometimes pre-impregnated with resin)

FRP systems come in a variety of forms, including wet lay-up systems and precured systems. The FRP system and its form should be selected based on acceptable transfer of structural loads and ease and simplicity of application. The manufacture of FRP materials is outlined and some general guidance on selection of FRP system and materials for particular strengthening applications are also provided. Indicative physical and mechanical material properties of some FRP prefabricated strips and fibres are included in the guide. The other related aspects of FRP materials such as durability, fire and electricity resistance, safety and environmental impact on material properties for different types of fibres are also discussed.

4.2 Construction Requirements

FRP system shipping, storage and installation procedure is normally developed by the system manufacturer. It differs between systems and even within a system depending on the condition of the structure. The user guide gives general guidelines for FRP system installation based on international guidelines (FIB Bulletin 14, 2001 and ACI Committee 440, 2002) as well as procedures developed by Australian manufacturers

4.3 General Design Considerations

The successful structural repair and upgrading involves four basic elements: concepts used in system design, compatibility and composite behaviour of existing members with upgraded system, field application methods, and most importantly, design details.

The status of the structure to be strengthened should be investigated and repairs should be performed as appropriate. It is of great importance to select the best FRP system for a particular rehabilitation need. Proper designing, detailing and applying it in a particular structure should guarantee the overall structural behaviour and safety of the strengthened member.

Chapter 5 of the user guide covered in detail the general design considerations covering design philosophy, safety and strengthening limits and ductility issues of rehabilitated systems.

4.4 Strengthening in Flexure, Shear, Torsion and in axial compression

In flexural strengthening of concrete beams using FRP composites, the major issues to be covered were identified as analysis of initial situation, calculation of design strength considering all possible failure modes, special consideration of anchorage failure and creep-rupture and stress limits. In shear strengthening, wrapping schemes, effective strain in laminates and limits on reinforcement were identified as critical and were detailed in the report. These were covered in detail in the user guide. In axial compression, effectiveness of confinement provided by FRP, and ductility were major issues and were addressed in the report 4.

4.5 Design Examples

A major feature of the user guide is worked design examples covering typical strengthening scenarios mentioned in 4.4.

5 Reliability Based Whole of Life Cycle Costing

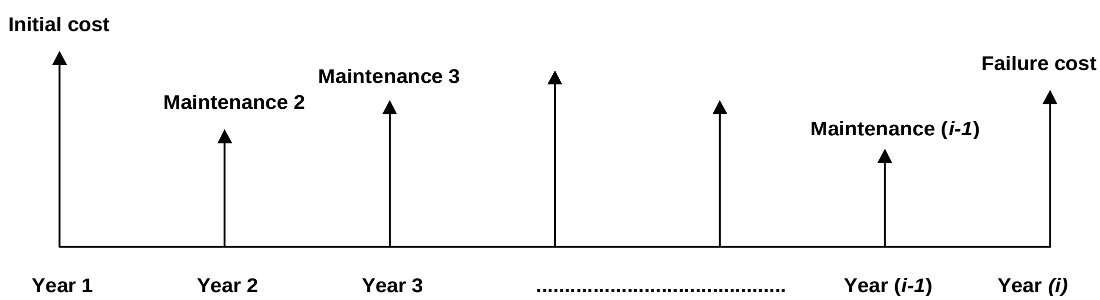
Whole of Life cycle cost analysis (WLCCA) is an evaluation method, which uses an economic analysis technique that allows comparison of investment alternatives having different cost streams. WLCCA evaluates each alternative by estimating the costs and timing of the cost over a selected analysis period and converting these costs to economically comparable values considering time-value of money over predicted whole of life cycle. The analysis results can be presented in several different ways, but the most commonly used indicator in road asset management is net present value of the investment option. The net present value of an investment alternative is equal to the sum of all costs and benefits associated with the alternatives discounted to today's values (Darter and Smith 2003).

A decision to treat a bridge structure may be based on minimising the whole of life cycle cost, which incorporates the risk of failure. Such a decision analysis is referred as a whole of life cycle costing, cost-benefit or cost-benefit-risk analysis. Life cycle costs will assess the cost effectiveness of design decisions, quality of construction or inspection, maintenance and repair strategies (Stewart 2001). The costs associated in a rehabilitation project may initially include:

- Initial cost
- Maintenance, monitoring and repair cost
- Costs associated with traffic delays or reduced travel time (Extra user cost)
- Estimated cost of failure

As shown by Austroads (1996), all of these costs are valued in resource cost terms (i.e. Market prices + subsidies - taxes). If monitoring, repair, extra user cost are considered as the maintenance cost then the cash flow for any rehabilitation method can be shown as in Figure 5-1.

Figure 5-1 Cash flow for the rehabilitation of bridge



In order to be able to add and compare cash flows, these costs should be made time equivalent. It can be presented in several different ways, but the most commonly used indicator in road asset management is the Net Present Value (NPV). The present value analysis has to be considered together with Internal Rate of Return (IRR).

5.1 STUDY PERIOD

The study period begins with the base date, that is the date to which all cash flows are discounted. Because the cost of each alternative rehabilitation strategy can be compared reasonably, only if the benefits gained are the same, the alternatives should be compared over the same operational time period which is known as study period. As a rule of thumb, the analysis period should be long enough to incorporate all or significant component of each alternative's life cycle including one rehabilitation in each alternative. Generally study period or the evaluation period is based on the economic life of major asset in the project. For bridges, the study period is normally longer than the pavements (more than 40 years). Assets with economic life longer than the evaluation period should be given a residual value (resale value).

5.2 DISCOUNT RATE AND INFLATION

The costs are incurred in a project in different times. The interest rate used to discount is a rate that reflects an investor's opportunity cost of money over time. Discount rate is defined as "the rate of interest reflecting the investor's time value of money (Mearig et al. 1999). It is the interest rate that would make an investor feel the same way if he receives a payment now or a large payment at sometime in the future. The LCCA can be performed in constant dollars or current dollars. Constant dollar analyses exclude the rate of general inflation. Current dollar analyses include the rate of general inflation in all costs, discount rates and price escalation rates. Both methods give an identical present value.

It is obvious that the discount rates are normally influenced by the economic, social and political factors. Discount rates used by various countries are different. For example Australia 4%, US 2-3%, UK Department of Transport 8%, Sweden 4% and Finland 6% (Val and Stewart 2003). The discount rates normally are updated and published. Therefore, a standard discount rate can be obtained from such published data. For AUSTROADS or national work, the recommended discount rate is 7% (Austroads, 1996).

5.3 FORMULATION OF WHOLE OF LIFE CYCLE COST

Objective function for the optimal bridge rehabilitation can be formulated as the maximization of,

$$W = B_{lifecycle} - C_{lifecycle} \qquad \text{Eq. 1}$$

Where $B_{lifecycle}$ is the benefit, which can be gained from the existence of the bridge after rehabilitation and $C_{lifecycle}$ is the cost associated with the bridge during its whole life. Since the benefit from the bridge will be the same irrespective of the rehabilitation method considered, it is possible to consider only the cost component. Therefore the new objective function will be the minimization of the total cost during its whole life cycle subjected to reliability and other constraints. The whole of life cost can be estimated as,

$$C_{lifecycle} = C_{initial} + C_{repair} + C_{user} + C_{failure} \qquad \text{Eq. 2}$$

When all these input costs are defined it is straightforward to calculate the present value of them. However all the input costs have a high degree of uncertainty. In order to deal with such uncertainties it is necessary to include the probabilistic behaviour of the input costs.

5.3.1 Modelling of the initial cost

Initial rehabilitation cost will include preliminary design cost, start up, material and labour costs (supervisors, skilled and unskilled). All these costs will incur in the base time of the project.

5.3.2 Modelling of the maintenance (repair) cost

Modelling of the future maintenance cost is complicated. Thoft-Christensen (2000) divided this cost into three categories namely, functional repair cost $C_1(t_{r,i})$, fixed repair cost $C_2(t_{r,i})$, and unit dependent repair cost $C_3(t_{r,i})$, if a repair is to be taken place at the time $t_{r,i}$. r is the discount rate and i is the number of occurrence of repair. Therefore the corresponding maintenance cost may be defined as (Thoft-Christensen 2000),

$$C_{maintenance}(t_{r,i}) = C_1(t_{r,i}) + C_2(t_{r,i}) + C_3(t_{r,i}) \qquad \text{Eq. 3}$$

The expected repair cost discounted to the time $t=0$ is the summation of the single repair cost.

$$C_{repair} = \sum_{i=1}^{n} \left(1 - P_f(t_{r,i})\right) C_{maintenance}(t_{r,i}) \frac{1}{(1+r)^{t_{r,i}}} \qquad \text{Eq. 4}$$

where n is the number of failures during the life cycle of the bridge and P_f is the updated failure probability at each repair time.

5.3.3 Modelling of user cost

User cost may be of two folds, during initial rehabilitation and during the next periodic rehabilitation. User cost may be calculated in terms of costs associated with traffic delay, and in case of using alternate routes wear and tear of user vehicle. The expected user cost may be formulated as,

$$C_{user} = \sum_{i=1}^{n} C_{user}(t_{r,i}) \frac{1}{(1+r)^{t_{r,i}}} \qquad \text{Eq. 5}$$

5.3.4 Modelling of the failure costs

Expected cost of failure (E_c) may be defined as (Stewart et al. 2000),

$$E_c = \sum_{j=1}^{M} p_{f,j} C_{f,j} \qquad \text{Eq. 6}$$

where p_{fj} is the probability of failure for limit state i and C_{fj} is the cost associated with occurrence of limit state i. Failure cost will include the damages, injury and loss of life costs.

5.3.5 Life cycle cost

The formulation of the life cycle cost can be preformed in a spreadsheet. All the possible cost components need to be then added to this spreadsheet for each and every rehabilitation option considered. Cash flows can be given as input variables for the respective year and finally the calculation of present value is performed using the built in financial function for NPV.

The probabilistic behaviour (mean and standard deviations) of any of the input cost should be entered to the respective cells of the spreadsheet in terms of the considered distribution function.

In a similar way each cost component can be given, as input parameters and include the probabilistic behaviour. Eventually the decision analysis should be subjected to a sensitivity analysis to make sure that the decision is not unreasonably affected due to the uncertainties of the costs associated.

5.4 SENSITIVITY ANALYSIS

LCCA estimations should be checked for the sensitivity to the uncertain parameters of the analysis such as analysis period, discount rate, traffic growth rates, traffic speeds, capital costs and accident predictions. Austroads (1996) has suggested the variables and ranges for a road project as shown in Table 5-1.

Table 5-1**:** Sensitivity tests – variables and ranges (Austroads, 1996**)**

Variable	Suggested minimum value	Suggested maximum value
Capital cost (final costing)	-10% of estimate	+10 to 20% of estimate
Operating and maintenance cost	-10% of estimate	+10% of estimate
Total traffic volume	-10 to 20% of estimate	+10 to 20% of estimate
Proportion cars in work time	-5 percentage points	+5 percentage points
Average car occupancy	-0.3 from estimate	+0.3 from estimate
Normal traffic growth rate	-2% pa (absolute) of the forecast rate	+2% pa (absolute) of the forecast rate
Traffic generated or diverted by project	-50% of estimate	50% of estimate
Traffic speed changes	-25% of estimated change in speed	+25% of estimated change in speed
Accident changes	-50% of estimated change	+50% of estimated change

@RISK 4.5 for Excel can be used to evaluate the impact of uncertain model parameters on the final result. For assumed values for input costs, the sensitivity of the net present value can be identified using @RISK software. It calculates the NPV for a base input parameter and then for a range of values (base value ± 10%). Similarly for all the input cost values the sensitivity of the NPV can be determined and compared.

Table 5-2: LCCA input variables

LCCA component	Input variable	Source
Initial and future costs	Preliminary engineering	Estimate
	Construction	Estimate
	Maintenance	Assumption
Timing of costs	Bridge performance	Projection
User costs	Current traffic	Estimate
	Future traffic	Projection
	Hourly demand	Estimate
	Vehicle distributions	Estimate
	Dollar value of delay time	Assumption
	Work zone configuration	Assumption
	Work zone hours of operation	Assumption
	Work zone duration	Assumption
	Work zone activity years	Projection
	Crash rates	Estimate
	Crash cost rates	Assumptions
NPV	Discount rate	Assumption

Risk ranking can be used to compare the relative risks of various alternatives. This can be done using the deterioration rates, relative frequency of over load, costs of failure, cost and efficiency of repair strategies etc (Stewart et al. 2000). The traffic delays or the reduced travel time depends on the traffic volume. Therefore, expected cost of failure is a more meaningful measure for the risk ranking. Thoft-Christensen (2000) defined the risk for a failure mode as the product of the failure cost and the probability associated with that. Damage cost and costs associated with loss of life and injury can be considered as the cost of failure. Cost of failure must be discounted to a present value. The probability associated with the failure is related to structural reliability. In this approach, the reliabilities for each option of rehabilitation can be ranked from higher risk to lower risk and a decision of selecting the optimal rehabilitation method can be based on both life cycle cost analysis and risk ranking.

It has been proposed by Thoft-Christensen (2000) that for a bridge rehabilitation program, a risk based structural optimization is more suitable than reliability based optimization.

5.5 Structural reliability

The reliabilities of bridge structures cannot be directly decided based on the observations of failures or other experimental studies. In such situations, reliability calculations are based on predictive models and probabilistic models. As shown by Stewart (2001), when the load effect (S) exceeds the resistance (R), the failure of structural element occurs. Therefore, reliability can be expressed as the probability of failure (pf) as follows,

$$p_f = \Pr(R \le S) = \Pr(R - S \le 0) = \Pr(G(R,S) \le 0) = \int_0^{\infty} F_R(r)\, f_s(r)\, dr$$

where G(R,S) is the “limit state function” and FR(r) is the cumulative probability density function of the resistance. Limit states normally selected for reliability analysis are:

- Ultimate limit state – flexural failure, shear failure, collapse
- Serviceability limit state – cracking, durability, deflection, vibration

5.5.1 Reliability distributions

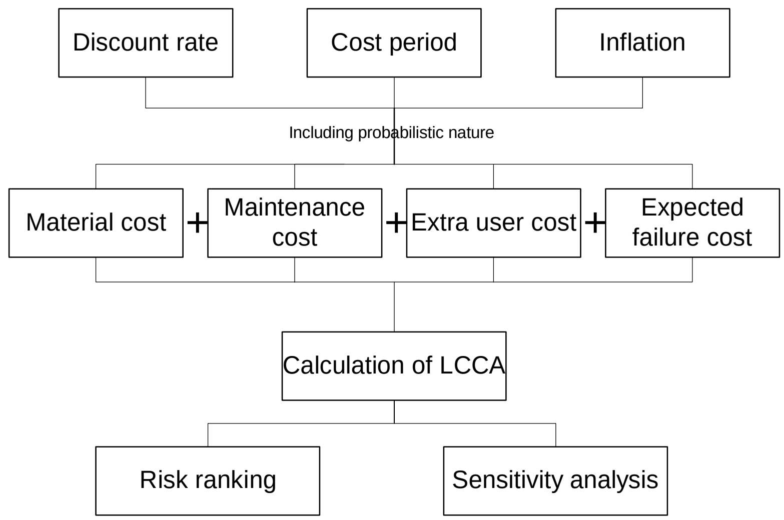
A group of bridges were considered from the Department of Main Roads in order to find a number of distributions for reliability based optimal design for bridge rehabilitation. Thoft-Christensen (2000) reported a lognormal distribution for the initial reliability, weibull distribution for the corrosion initiation time and a uniform distribution for the deterioration rate. Similar kind of distributions may need to be established for this particular project using the existing data. Finally optimal rehabilitation strategy can be selected based on such distributions.

Probabilistic life cycle costing together with the risk ranking offers prominent improvements in selecting the most suitable rehabilitation strategy. This approach is superior to the deterministic approach used in traditional bridge management systems.

6 Decision support software tool for selecting the optimal rehabilitation strategy

The basic steps involved in the LCCA estimation are shown in Figure 6-1.

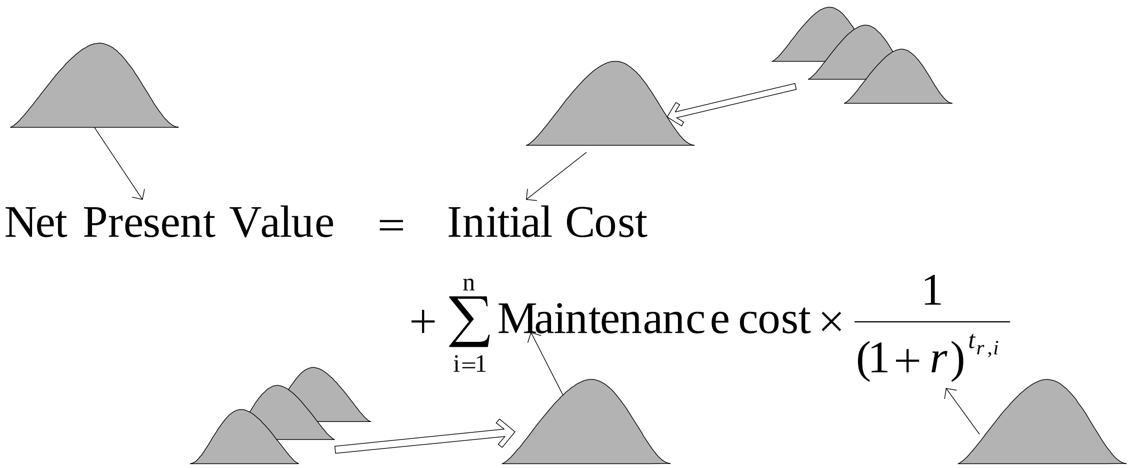
Figure 6-1: Flow-chart for the LCCA estimation



Initially, the research team attempted use of a commercially available software program @Risk 4.5 for the Life Cycle Costing Analysis through a spreadsheet interface. However, as the analysis became more and more complicated, need for a user-friendly interface for the analysis was evident. Subsequently, the research team developed a stand alone software tool, which runs with @Risk software. CRC report 2002-005-C-06 is the user manual for the software.

Figure 6-2 shows the NPV formula as an economic indicator in analysing the rehabilitations options for the selected bridge. Risk analysis approach uses random samples from the probability based uncertain input variables (initial cost, future cost, discount rate and year of rehabilitation) to generate probabilistic description of the output result, NPV. Using Monte Carlo Simulation it is possible to select thousands of samples from each input distribution and generate the output result (NPV) for a separate *what-if* scenario. The results calculated from each what-if scenario can be saved and further statistical analysis can be performed. As a result, risk analysis results can be illustrated in the form of probability distributions. It shows a range of possible outcomes and the weight of its occurrence as well. This is necessary in making a consensus decision.

Figure 6-2: Probability distributions in Net Present Value calculations



$$\text{Net Present Value} = \text{Initial Cost} + \sum_{i=1}^{n} \text{Maintenance cost} \times \frac{1}{(1+r)^{t_{r,i}}}$$

r = discount rate
tr,i = year of maintenance
n = number of maintenance

Figures 6.3 to 6.6 show the basic interfaces of the software tool and the graphical output.

Figure 6.3: Window, which offers selection of alternatives

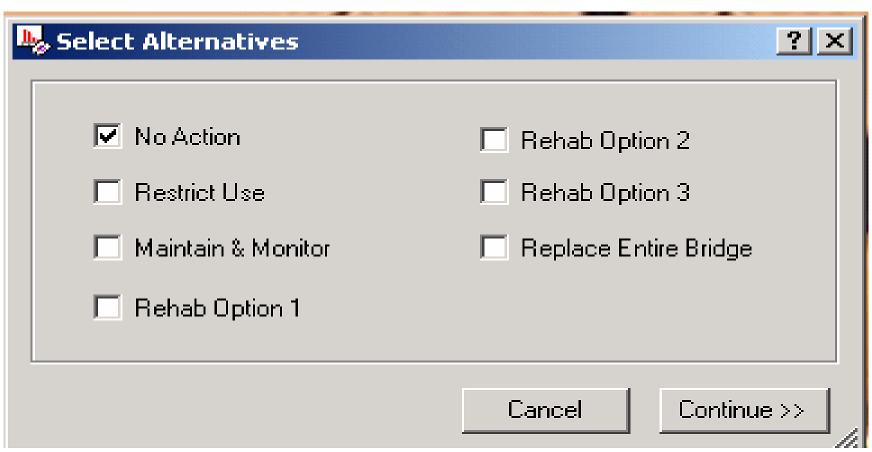


Figure 6.4: User interface for entering elements of cost

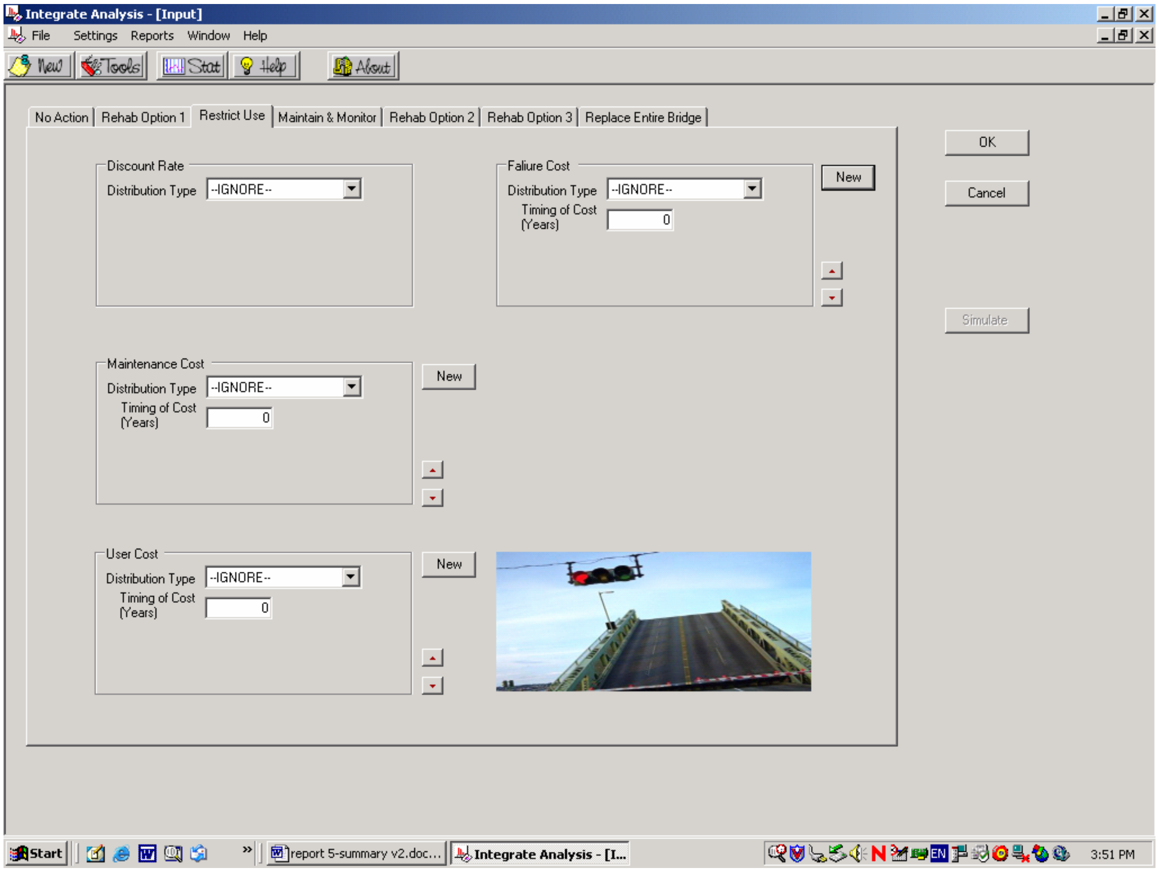


Figure 6.5: User interface for simulation

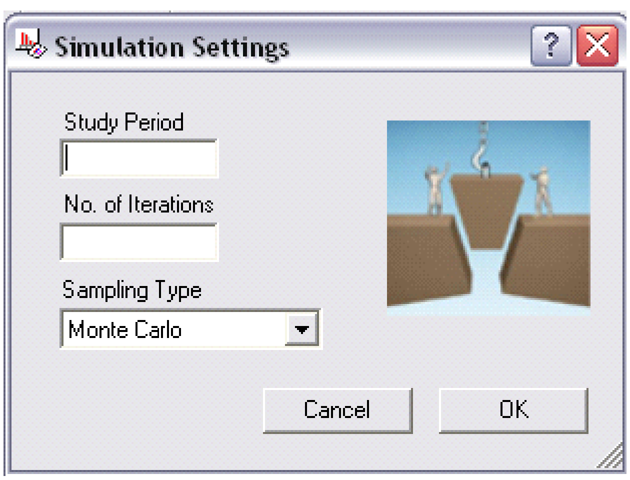
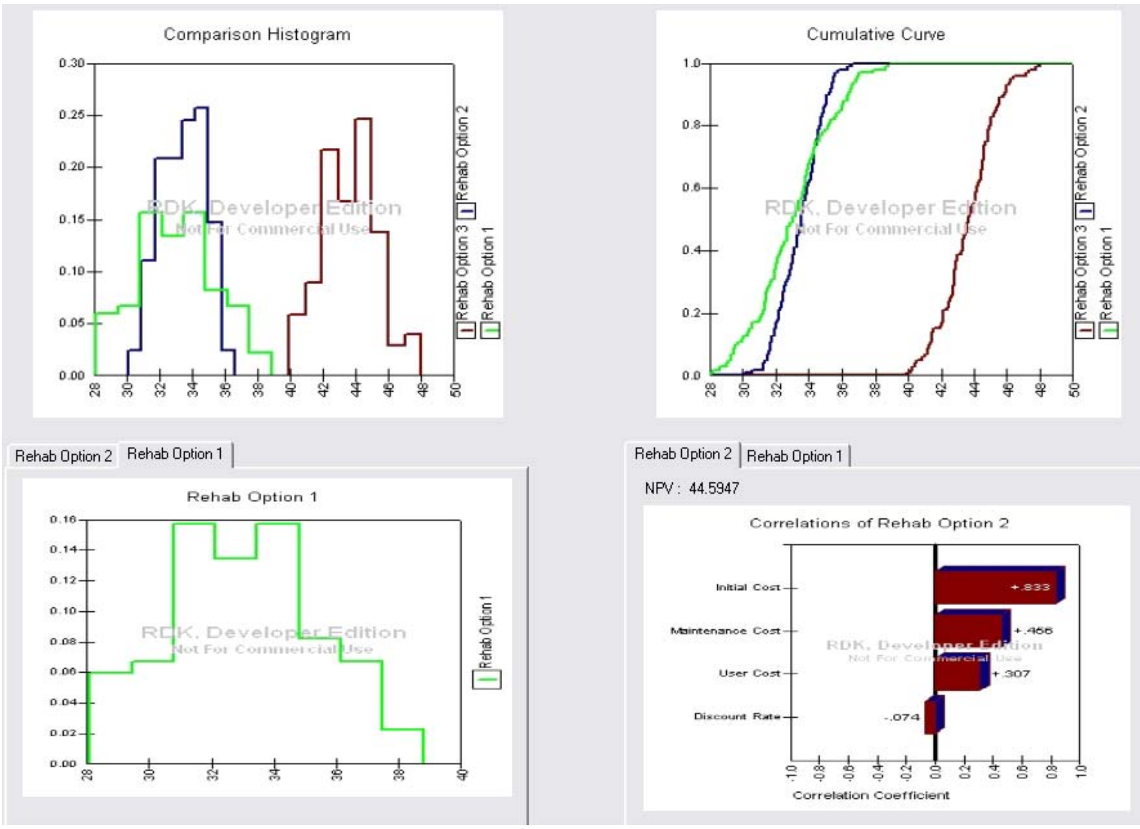


Figure 6.6: Output



7 Conclusions and Recommendations for Future Research

7.1 Conclusions

Summary of conclusions drawn from the research conducted by the team of project 2002-005-C are given below.

- Outcomes of published research work can be adopted for design and construction of FRP strengthening schemes for concrete bridge structures with significantly economical solutions. However, both design and practical implementation require careful analysis of the existing structure, surface preparation, FRP materials and design concepts. Use of one international guideline is not appropriate in designing a strengthening scheme satisfying provisions of the Austroads code. A complete user guide has been developed as report 2002-005-C-04.

- FRP strengthening schemes will have a lesser reliability compared to proven traditional solutions. In comparing two strengthening solutions the risk needs to be incorporated into the cost of innovative solutions.

- Whole of life cycle costing based on a reliability analysis is extremely powerful in comparing treatment options for strengthening of bridge structures. The method can be used to develop a common platform for comparing different strengthening scenarios.

- Incorporation of cost of failure into life cycle costing can allow for the risk of innovative solutions when comparing against proven solutions.

- The cost of failure can be calculated as probability of failure times the cost of failure. The methodology for calculating the probability of failure is complex and requires consideration of all the deficiencies of a given bridge structure. A simple methodology can be developed assuming if the capacity exceeds the design actions, a 5% probability of failure is assumed and depending on the gap between the capacity and the design actions, the probability of failure is scaled up.

7.2 Recommendations for Future Research

- The overall framework developed for decision support software tool will need more fundamental research to populate different input parameters.

- The tool can be accompanied by a database, which can collect data on input parameters over the years, which can be used to generate the input parameters.

- Developing the methodology for evaluating probability of failure is being addressed in detail by the new CRC project 2004-018-C, “Sustainable Infrastructure for Aggressive Environments”

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