IMPROVED BRIDGE RATING PROCEDURES INTEGRATING LOAD PATH REDUNDANCY: A COST-EFFECTIVENESS SIMULATION

by

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ABSTRACT

The current economic climate brings increasingly constrained funding for our aging national bridge inventory. Therefore, it is ever more important to those in charge of maintaining this infrastructure to ensure cost-effective asset management.

However, current AASHTO bridge rating practice contains arguably an unnecessary degree of inherent conservatism. This is due to the lack of acknowledgement of load path redundancy in current rating practice.

This thesis performs a literature review into current rating methodology and discusses where load path redundancy is not and could be accounted for as part of the process. A simulation of adjusted NBI 2010 sufficiency ratings for a set of composite steel girder-concrete deck bridges in the state of Delaware is described. Concurrently, the NBI estimates for the budget required for renewal of those bridges is adjusted to reflect the adjusted sufficiency ratings. These adjustments are done using a number of scenarios which considered parameters such as: the condition of the deck and super-structure and the average time required for those structures to deteriorate by one rating category. The three budgetary scenarios simulated are: No-Change, Deferral of Spending, and Decreased Spending.

The results indicate that the proposed changes to the current rating process are cost-effective. The proposed changes are an acknowledgement of load path redundancy by adjusting the way load carrying capacity is rated to be based more on system capacity rather than individual girder capacity and by supplementation of visual ratings with more informed field data.

The conclusion is that revising current rating practice to acknowledge load path redundancy affords better prioritization of federal funding and cost-effective asset management as well as the potential for multi-million dollar savings in the state of Delaware. Finally it is acknowledged that while this thesis focuses on the state of Delaware specifically, the hypothesis has the scope to be applied to bridges nationwide.

Chapter 1

INTRODUCTION

1.1 Problem Statement

Nationally we are faced with an aging bridge inventory. In Delaware alone, the average age of our bridge infrastructure is 41 years (Federal Highway Administration, Department of Transportation, 2010), compared to a design life of 50 years (U.S. D.O.T., FHWA, 2011). Couple this with a climate of increasing public awareness and concern for bridge safety standards following the I-35W Minneapolis bridge collapse (2007) and we are prompted, in addition to optimized management, to look at improving our understanding of bridge behavior structurally.

Further to that end, a 2009 white paper, by an ad hoc group reports the recommendations of industry professionals on bridge inspections and ratings. One such recommendation states the National Bridge Inspection Standards "can be improved" to be more reliable (ASCE/SEI-AASHTO Ad-Hoc Group on Bridge Inspection, 2009).

However, in terms of inspection standards and bridge rating, not all bridge system behavior is currently acknowledged in determining the load carrying capacity of bridges. Albeit on the side of conservancy in terms of safety, current bridge rating practice does not acknowledge *load path redundancy* in the structural behavior of composite steel girder-concrete deck bridges.

1.2 Motivation

The exclusion of the action of load path redundancy in the current bridge rating process and consequently the bridge management process was a key motivating factor behind this thesis. Load path redundancy in terms of bridges can be summarized as the presence of one or more routes through which a bridge can redistribute its load should a member fail, thereby enabling the bridge to function on the system level, to maintain its load carrying capacity. This thesis explores how accounting for this load path redundancy influences costs over the lifecycle for composite steel girder-concrete deck bridges.

In ever increasingly difficult economic times, the amount of federal funds available to states for the repair and rehabilitation of our nation's bridges under the *Highway Bridge Program* is highly unlikely to be expanded as our economy attempts to recover (Roberts & Shepard, 2007). This prompts an even greater need to optimize the management of both resources and inventory to affect overall cost-effectiveness. Meeting this need requires facing down a number of obstacles.

Providing an approach with which the rating and management process might be improved so as to afford bridge managers more cost-effective allocation of budgetary resources and a more prioritized approach to asset management is at the core of the motivation behind this thesis.

Promotion of a better understanding of bridge behavior structurally to inform more reliable standards, (the need for which has been expressed in the white paper as stated previously (ASCE/SEI-AASHTO Ad-Hoc Group on Bridge Inspection, 2009)), leading to wiser decisions and evaluations which can be reflected in an economic process further motivates this research.

These motivational factors informed the formulation of the following objectives to be met as part of this research.

1.3 Objectives

The primary objectives to be met in the completion of this thesis so that that the problems outlined in Section 1.1 might be addressed and progressed towards a resolution are as follows.

1. The first objective is to incorporate load path redundancy in composite steel girder-concrete deck bridges into the bridge rating process by proposing changes to current process. The objective is to do this for a sample group of such bridges listed on the National Bridge Inventory database for the state of Delaware in a quantified and simulated manner.
2. The second objective is to simulate changes in the National Bridge Inventory's estimated budget requirements for remedial works to the sample group of bridges. Through this simulation of changes to budget requirements, to provide an "economic reflection" (as described by Professor Jennifer McConnell) of the proposed changes to the rating process. This objective aims to provide an illustration of the economic benefits (if any) and resource allocation optimization benefits afforded to bridge managers if load path redundancy is acknowledged in the bridge rating process.
3. The third objective is to perform a preliminary investigation of the influence of aging on load path redundancy and to achieve this by investigating age-related deterioration of reinforced concrete bridge decks subjected to salt ingress and the effects this may have on the deck performance in terms of load path redundancy action in steel girder-concrete deck bridges.

The methodology implemented to meet these objectives is now outlined.

1.4 Overview of the Methodology

The methodology implemented to meet the objectives of this thesis includes a literature review of current bridge rating practice. Additionally, a cost-effectiveness analysis of the acknowledgement of load path redundancy in the bridge rating of selected composite steel girder-concrete deck bridges in the state of Delaware is made. Finally, a preliminary study into the effects of aging on load path redundancy is utilized.

1.4.1 Literature Review

The literature review is discussed beginning at Section 2.1. The literature reviewed is of the current bridge rating process. Of focus is the methodological approach to the rating process and the way in which funds from the Federal Highway Administration's *Highway Bridge Program* are allocated.

Also included is a review of data published on the deterioration of Illinois bridges which had been rated using the current process and whose rating data is part of the National Bridge Inventory database. The review of this literature informs time related parameters utilized later in the cost-effectiveness analysis and is discussed in Section 3.2.2.

1.4.2 Selection of Candidate Bridges for Analysis

The National Bridge Inventory (NBI) data for the State of Delaware is used for the assessment, with data for the year 2010 being the most recently available at the time of analysis. The number of bridges for cost-effectiveness analysis was determined by narrowing down the number of all bridges for which data is available to a candidate list of bridges which meet particular structural and rating characteristics under which the action of load path redundancy is applicable. This aspect of the methodology is discussed in detail in Section 3.1.

1.4.3 Cost-Effectiveness Analysis

The cost-effectiveness analysis involves simulation of three different scenarios under which the amounts ear-marked in the 2010 NBI for candidate bridges may change in some way. Those scenarios involved a no change, a deferral of spending and a reduced spending situation. This aspect of the methodology is discussed in detail in Sections 3.2-3.5.

1.4.4 Preliminary Study into the Influence of Aging on the Action of Load Path Redundancy

It is recognized that realistically not all bridges are in a structural state of health that is optimal and preferred so that load path redundancy action is optimally achieved to maximize the system capacity of the bridge. As a move to aid and supplement current investigation and research, a preliminary literature review into the quantification of the effects of aging on load path redundancy via the age-related deterioration of the mechanical properties of a reinforced concrete deck was undertaken. This aspect of the methodology is discussed in detail in Chapter 4.

1.5 Outline of Thesis

The thesis begins by providing a brief background on the current bridge rating process as it informs the allocation of federal funds under the *Highway Bridge Program*. The thesis analyzes the impact on bridge management budgetary resources were load path redundancy in composite steel girder-concrete deck bridges to be accounted for in the rating process and resource allocation. A cost-effectiveness analysis is performed on the data available for a selected number of such bridges in the state of Delaware. The material of the thesis is presented to the reader as follows.

Chapter 2 discusses the literature review of the current National Bridge Inventory bridge rating process and allocation of Federal funding under the *Highway Bridge Program* within a scope that suffices for the focus of this thesis. Also discussed are load path redundancy and the hypothesis that forms the basis of this thesis.

Chapter 3 describes the cost-effectiveness analysis performed on selected Delaware bridges as a function of their National Bridge Inventory data being adjusted to acknowledge the action of load path redundancy and the associated results of that analysis.

Chapter 4 outlines what is involved in the preliminary investigation into the effect of aging on load path redundancy. It then discusses the results of that investigation.

Chapter 5 draws conclusions on the research and results of this thesis and puts forward recommendations for areas of further investigation.

Chapter 2

LITERATURE REVIEW

2.1 Federal Funding and the Highway Bridge Program

The *Highway Bridge Program* (HBP) is the source of federal funding for repairs, rehabilitation and replacements of national infrastructure. It is through this program that each state receives funding for the management of its bridges (U.S. Department of Transportation, 2012).

When bridges within a state are rated, the data is entered in the National Bridge Inventory database (NBI). The ratings as they appear in the NBI allow the federal government to prioritize allocation of funding through the HBP to each state. Under current practice, to be eligible for funding, a bridge must have a '*Sufficiency Rating*' of 80% or less (Xanthakos, 1996). This sufficiency rating essentially serves as a quantitative indication of the degree of deficiency of a bridge, with a lower rating indicating a higher priority for remedial works. While the limitations of the sufficiency rating have been recognized, the sufficiency rating is the quantitative measure of bridge health used by the federal government (Roberts & Shepard, 2007).

Building on this description of the performance measure and decision variable for the allocation of funding (i.e. the *sufficiency rating*), the rating process behind the determination of that measure is briefly outlined. It can then be better understood where the shortcomings of this process lie and how they might affect management decisions.

2.1.1 The Rating Process behind the National Bridge Inventory

Determination of a bridge's sufficiency rating begins with a visual inspection. The data gathered from the inspection is entered into a '*Structure Inventory and Appraisal*' itemized form. (A sample of this form can be found in Appendix A). The guidelines for data entry of the inspection process are found in the *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridge's*, published by the U.S. Department of Transportation (Federal Highway Administration, 1995). This process provides information for input into the sufficiency rating equation.

2.1.2 The Sufficiency Rating Equation

The sufficiency rating equation is built on four factors: *Structural Safety; Serviceability and Functional Obsolescence; Essentiality for Public Use;* and an allowance for *Special Reductions* to the overall rating if items such as detour length or traffic safety features, like guard rails for example, are applicable; however it is the former three factors which are at the heart of the overall bridge sufficiency rating. The sufficiency rating equation is set out below (Equation 2-1), where S_1, S_2, S_3 and S_4 represent the four factors respectively.

Equation 2-1 $$S.R. = S_1 + S_2 + S_3 + S_4$$

Each of the four factors is calculated separately upon completion of the structural appraisal form before being combined to produce a single value for the overall bridge sufficiency rating. The maximum obtainable sufficiency rating is 100%, which would represent a bridge which is entirely sufficient but this is not commonplace.

The weighted influence carried by each of the four factors is not however equal. The fourth factor, *Special Reductions*, may or may not be included in the equation at all depending on the circumstances. This inequity between the four factors is now explained in more detail along with the importance it places in turn on load carrying capacity and decision making in the bridge rating process.

2.1.3 The Importance of Load Carrying Capacity in the Rating Process

The maximum percentage attributable to each of the three primary factors in the sufficiency equation is depicted in Figure 2-1 below.

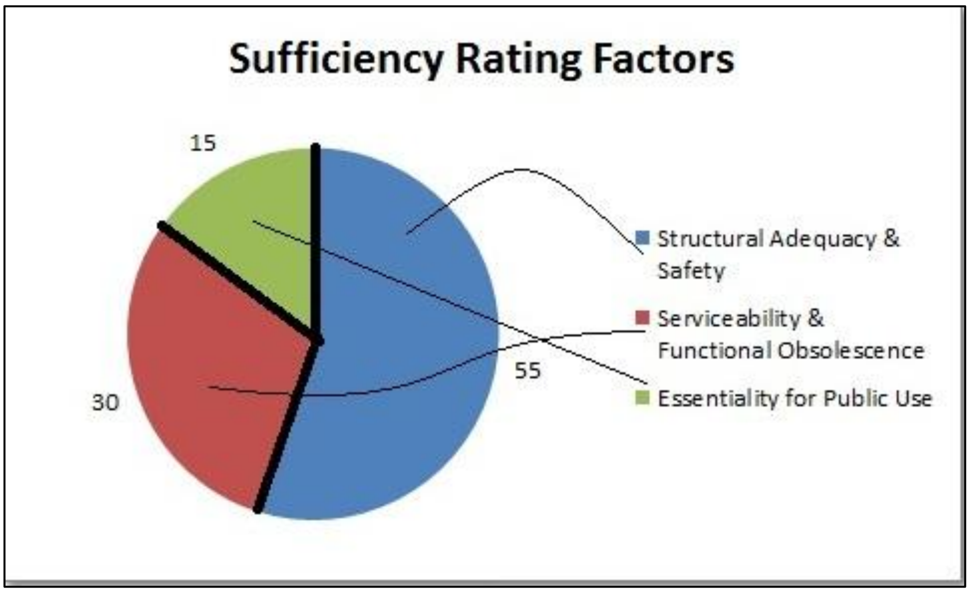

Figure 2-1 Sufficiency Rating Factors

It is apparent from Figure 2-1 that S_1, or *Structural Adequacy and Safety,* is attributed a 55% weighting in the sufficiency rating equation. Therefore, *Structural Adequacy and Safety* has the greatest influence on determining the overall sufficiency rating.

2.1.3.1 Structural Adequacy and Safety (S_1)

The sample structural appraisal form (Appendix A) and the worked example of the calculation of the sufficiency rating for one bridge provided by the FHWA (Appendix B) document the process for computing this component. The *Structural Adequacy and Safety* of the bridge is calculated in two parts.

The first, "A" is the combined value of the condition ratings for the super structure, sub-structure and culverts, which are items 59, 60 and 62, respectively. These items are defined in *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* and are seen on the sample structure inventory & appraisal form in Appendix A.

The second, part "B" is a value based on the load carrying capacity and the "Inventory Rating" of the bridge. The load carrying capacity of the bridge, can be determined by "analysis and, in some cases, load testing", which might involve "the Load Factor Method, Working Stress Design Method and Load and Resistance Factor Method". The reader is referred to Chapter 4 of the *DelDOT Bridge Design Manual* for a more detailed description (Delaware Department of Transportation, 2009). The 'Inventory Rating' is item 66 as defined in *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges* and is seen on the sample structure inventory & appraisal form in Appendix A.

The values of both "A" and "B" carry equal weight and when they are combined and subtracted from the upper bound of 55%, the S_1 factor is obtained. This factor represents the *Structural Adequacy and Safety* of the bridge. The reader is referred to Appendix B for an illustration of the process just described.

2.1.3.2 "Inventory Rating"

In the example shown in Appendix B, it can be seen under part "1. B" that the "Load Carrying Capacity" is related to the "Inventory Rating" on the structural appraisal form (Appendix A). The "Inventory Rating" is referred to in the *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridge's* under items 65 and 66. Item 65 being an indication of the type of load rating method used in assessing the bridge and item 66 indicating the load rating itself in metric tons. The load rating is in fact the "capacity rating, referred to as the inventory rating" (Federal Highway Administration, 1995). In other words the guide says that the "inventory rating" is synonymous with the load carrying capacity of the bridge.

2.1.3.3 Observations Related to Sufficiency Rating

The inventory rating influences the determination of the first factor, S_1, the *Structural Adequacy and Safety* of the bridge. This first factor, in turn is weighted the most influential in the calculation of the bridge's overall sufficiency rating, having a weighting of 55% in the sufficiency rating equation, Equation 2-1. Comparatively, the second and third factors, S_2 and S_3, respectively, are weighted less, at 30% and 15% each, respectively, in the sufficiency rating equation.

Therefore, it could be said that the load carrying capacity of the bridge holds the "highest weight in sufficiency rating formula" (Akgul & Frangopol, 2004). It is also "a crucial measure for bridge management and decision making", as is the argument promoted by Akgul and Frangopol (2004). How the load carrying capacity of the bridge relates to load path redundancy is now outlined.

2.2 Load Path Redundancy and Load Carrying Capacity

The action of load path redundancy can be of great benefit to the load carrying capacity of the bridge system. Before discussing how the current rating process neglects this action, a basic description of load path redundancy is given in the section immediately following, followed by a required description of the relationship between load path redundancy and load carrying capacity in Section 2.2.2.

2.2.1 What is Load Path Redundancy?

The girder is the primary load path in a composite girder and deck bridge system. With that understanding, load path redundancy then can be explained by the following definition given by the Missouri Department of Transportation:

> With respect to bridge structures redundancy means that should a member or element fail, the load previously carried by the failed member will be redistributed to other members or elements which have capacity to temporarily carry additional load and collapse of the structure may be avoided (Missouri Department of Transportation, 2000).

The term ‘load path’ then refers to the component/element/member of the bridge structure through which the load redistribution or ‘shedding’ can take place. For the ‘load path’ to be redundant, that configuration, or “the number of supporting elements” (Missouri Department of Transportation, 2000) must be greater than one. Or as Xanthakos (1996) puts it “at least one alternative load path exists and prevents collapse”. In other words, it is multi-girder bridges specifically which are referred to when discussing load path redundancy. The action of load path redundancy is illustrated in its most basic sense in Figure 2-2 below.

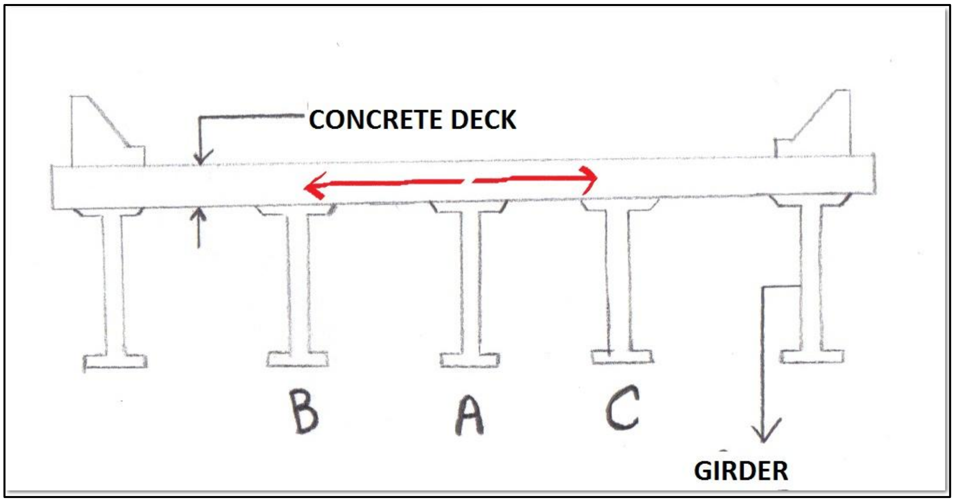

Figure 2-2 Load Path Redundancy

The principle is that if girder "A" were to experience some type of deterioration, over-loading or failure causing it to be unable to carry its load, that load would be redistributed to girders "B" and "C" adjacent to it. That redistribution of load can occur by travelling through the concrete deck as indicated by the arrows.

2.2.2 How Does this Affect the Load Carrying Capacity?

The action of load path redundancy allows the bridge to maintain its load carrying capacity. Rather than losing its capacity due to the reduced performance, over-loading or failure of one (or more) girders, the bridge maintains that capacity due to the uptake of the shed load by the other bridge elements. This load sharing is a system effect, which is now explained.

2.3 Load Carrying Capacity is a *System* Effect

Current bridge design practice can be described as conservative. That is, girders are designed on an individual component level for a worst case scenario. This design approach means that it is assumed each girder carries a constant proportion of the load throughout its lifetime (McConnell J. , 2011).

It should be clear from the above description of the action of load path redundancy that what happens in reality is significantly different from what is conservatively designed for. That is, the redistribution of load between girders means that the proportion of load carried by an individual girder may not be constant. In fact, a girder has the reserve capacity to take an additional proportion of its neighbor's load. This tells us that the bridge elements (girders) act as a system.

From Figure 2-2 it is clear that there is an additional element belonging to the success of this system performance, which is the deck. Figure 2-2 indicates by way of the arrows that the deck is a primary means through which the load of a girder unable to carry the weight it is subjected to alone is redistributed laterally to the other bridge elements. The deck therefore, is a key component in the realization of load path redundancy and therefore in achieving the full potential of the bridge's load carrying capacity, i.e., through a system process.

It should be noted that lateral bracing members are a second additional means for redistributing load to adjacent load paths, but this work will focus solely on load redistribution via the deck. Load redistribution through cross-frames of steel girders is the subject of other work (Ambrose, 2012).

2.4 Scope for Improvement in the Bridge Rating and Funding Process

It should now be clear how due to load path redundancy, the load carrying capacity of a bridge can be described a system behavior rather than being attributable solely to individual girder capacity. Furthermore, system capacity can afford greater load carrying capacity than that determined on an individual girder basis. It is demonstrable therefore, how this source of reserve capacity in bridges (that) is not currently accounted for in bridge design and rating (McConnell, Mc Carthy, & Wurst, 2012).

Referring again to the NBI sample structural appraisal form and sufficiency rating example calculation (Appendix A and Appendix B respectively), item 66 on the form is 'Inventory Rating'. Inventory Rating is used in the appraisal synonymously with load capacity rating or load carrying capacity. It is here under item 66, assigning the inventory rating, that an acknowledgement of load path redundancy and the system rather than component load carrying capacity of the bridge could be made. In so doing, the end product (sufficiency rating) of the bridge could reflect the action of load path redundancy.

As the rating process is currently completed, assessment is still reflective of a component level capacity only. This evaluation approach "yields a conservative measure of actual load carrying capacity" (Wang, O'Malley, Ellingwood, & Zureick, 2011). Based upon this rationale, if the NBI rating process acknowledged load path redundancy as part of the sufficiency rating, bridges currently rating "Poor" in sufficiency might have improved ratings. This would reflect the additional system capacity that is achievable.

Of course this yields the need for improved appraisal procedures that can capture the system behavior and condition of the bridge. It is sensible to suggest that the most appropriate place where this may be achievable is via evaluation of items 58, 59 and 66 when completing the structure inventory and appraisal. These items visually rate the deck and super-structure and give the inventory rating (load carrying capacity). It remains to be resolved how a connection can be made between the visual rating and what we as scholars know to be the physical behavior on a non-visual level, for example we know load path redundancy to exist in action but it is not something that is visually apparent at a field inspection.

As set out under the objectives of this thesis, an economic reflection of improvements in the NBI rating process and allocation of funding under the Highway Bridge Program that could potentially be achieved with the acknowledgement of load path redundancy within the current sufficiency ratings process is then simulated in Chapter 3 under Section 3.3. A number of bridges in the state of Delaware are chosen to form the basis of the cost-effectiveness simulation. The rationale behind the selection of those particular bridges and the details of the economic analysis follow in Chapter 3.

Chapter 3

COST-EFFECTIVENESS ANALYSIS

It is implicit in times of constrained funding that bridge managers want to serve the needs of their bridge inventory by optimizing the allocation of their repair and rehabilitation budget to deliver the highest level of service across the network. Chapter 2 ends with a description of the shortcomings in the current bridge rating process and federal funding program, which can be summarized as: a necessary conservative rating of the true load carrying capacity of the bridge system by disregarding load path redundancy due to current knowledge gaps.

The cost-effectiveness analysis completed in this chapter provides an economic reflection of simulated changes in the NBI rating data for a number of bridges in the state of Delaware. These changes acknowledge the action of load path redundancy in those bridges. That analysis is now described in detail.

3.1 Selecting Bridges for Analysis

The scope of the analysis is restricted to the state of Delaware, the home state of the University of Delaware. The most recently available NBI data for the state at the time of analysis was from the year 2010. That inventory contains data on a total of 1159 bridges. The analysis was further constrained to composite steel girder bridges with concrete decks both for the purpose of a timely completion but also to reflect characteristics both locally and nationally, where this bridge configuration is common. To contextualize this, the bridges in the state which are known to be steel girder-concrete deck bridges make up approximately 28% of the bridge inventory with available data in the State of Delaware (Federal Highway Administration, Department of Transportation, 2010). What is meant here is that there are a large number of bridges in the 2010 NBI for Delaware which are given no description of their structure kind or type. These bridges were therefore omitted from consideration. Additionally, the remaining inventory is made up of concrete girder-concrete deck configurations, or a small number of other configurations that are outside the scope of this thesis, such as steel suspension or timber bridges.

From here, bridges which are currently ‘Closed’ or ‘Posted’ were removed from the list for analysis due to having a ‘Serious’ rating of some nature where more detailed structural information would be required to determine the full extent of that health implication and its effects on the accommodation of load path redundancy action, which are steps outside the scope of this thesis. A ‘Serious’ rating is considered here as a bridge which is assigned a numerical condition rating of 3, 2, 1 or 0. These numerical ratings are described as ‘Serious’, ‘Critical’, ‘Imminent Failure’ or ‘Failed’ in the *Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation’s Bridge’s* and all have major or advanced deterioration and/or section loss.

Additionally, those bridges which are “Functionally Obsolete” were not included as part of the analysis as those bridges are not deficient in the sense of structural safety or reliability but rather they have geometric deficiencies which mean they do not meet their ‘Level Of Service’ requirements. Level of Service can be defined as a rating scale for a bridge from ‘A’ to ‘F’, which describes how well or hoe poorly the bridge is providing the service for which is was designed, where ‘A’ is the best achievable rating and ‘F’ is the worst.

As stated previously, the focus of this analysis is on cost-effectiveness. For this reason, the inventory of bridges was narrowed further to include only those bridges which have a sufficiency rating less than 80% making them eligible for funding under the *Highway Bridge Program.*

At this stage it was necessary to look at the structural ratings of the remaining bridges to determine their suitability for analysis. What is meant by this statement is: a suitable bridge is one which can enable the full system load carrying capacity of the bridge to be realized, where the condition of the concrete deck is good, thus permitting redistribution of load to girders, i.e. the deck can successfully function to create an alternate load path. The 2010 NBI data for the bridges which were analyzed is included in an edited format to the original source made available from the FHWA (Appendix C). The original form was edited due to superfluous content to that which was required for the scope of the analysis in this thesis. As evidenced in that database under items 58 and 59 is the structural rating of the deck and super-structure respectively. (The girders are included under the definition of 'super-structure'). To reflect a scenario where load path redundancy benefits girder load redistribution, it was decided to narrow the scope to bridge's whose decks were rated no lower than 'Good' i.e. having a numerical rating of 7 and above, and whose girders were rated less than 'Good' i.e. having a numerical rating of 6 and below. These are the bridges that will be most benefitted by the inclusion of load path redundancy. This leaves a total of 14 bridges to be analyzed.

It was required to make an assumption following this step. It can be seen in the database that item 075A indicates the type of remedial measures recommended for funding, be it rehabilitation, repair or replacement. Where the recommendation was replacement "because of substandard load carrying capacity or substandard bridge roadway geometry", it was assumed that the recommendation was due to substandard load carrying capacity to maintain conservatism. What is meant here is that, a bridge which is recommended for replacement due to substandard roadway geometry is not indicated to have any structural fault, rather, the volume of traffic or the type of traffic utilizing it, is not being accommodated sufficiently for geometric reasons. For example, when the bridge was built 2 lanes may have been sufficient to carry the traffic volume at that time but perhaps more lanes would better serve the traffic slow now. On the other hand, a bridge which is substandard in load carrying capacity is structurally deficient in that it cannot carry the load it is required to. For this reason, there is a public safety issue and bridges in this category would receive priority in replacement. In other words, budget allocations would definitely be spent on these bridges, so for the purposes of maintaining a conservative or 'worst-case' estimate of spending which is required, the aforementioned assumption on item 075A was made.

One final assumption was made before the scope for economic scenarios was defined. Item 94 in the database indicates the estimated required funds in thousands of dollars for the proposed remedial measures. However, many of the bridges identified for analysis had no indication of the associated costs of improvements. The process then was to return to the original list of all 1159 Delaware bridges and to determine the average estimated cost of improvements per bridge. This amount was determined to be approximately \$2 million dollars (2005 dollars). The assumption made was that those bridges on the final list of bridges identified for the cost-effective analysis could be assigned the average associated cost of improvements, lacking any more accurate data.

When the scope was narrowed according to the above criteria, the final number of bridges to be analyzed was reduced from 327 i.e. the number of all composite steel girder-concrete deck bridges, to 14, as stated previously. These 14 bridges carry a combined associated cost of improvements of \$23.4M. The economic investment scenarios which can be implemented in the management of these bridges and which comprise the analysis are now described.

3.2 Scope of Investment Scenarios

It was decided to limit the scope of the cost-effectiveness simulation to three scenarios. This scope allows an exploration and assessment of the economic value of acknowledging load path redundancy in bridge rating practices and allocation of funds while the impact of aging on the effectiveness of load path redundancy is still under investigation and yet to be quantified structurally.

Those 14 Delaware bridges in the NBI for 2010 in which it can be said successful load path redundancy is expected to be feasible are herein referred to as “the simulation bridges” (Appendix C) . By considering the federal funds which are currently estimated for the simulation bridges, the chosen scenarios assess three ways in which the investment of that funding might be managed, they are outlined in sections 3.2.1-3.2.3 following.

3.2.1 Scenario 0: *No Change*

This scenario simulates maintaining the current practice according to the NBI for rating the load carrying capacity and structural adequacy and safety of bridges, whereby load path redundancy is disregarded. According to Scenario 0, therefore, the repair, rehabilitation or replacement measures as currently have been determined to be required on the simulation bridges is assumed to remain valid. This is therefore a traditional “Do-Nothing” scenario, whereby it is assumed the federal funds which are currently allocated to be spent on each of those bridges be spent as estimated. The specifics of this scenario are discussed in Section 3.3.1 and the results are available in Table 3-1.

3.2.2 Scenario 1: *Deferral of Investment*

Scenario 1 is one of two scenarios which simulate the acknowledgement of load path redundancy within the NBI rating practice. Under this scenario, it is assumed that load path redundancy is valid for the simulation bridges and therefore improves the load rating for each of those bridges.

As discussed in Chapter 2, load rating carries significant weight in the determination of the bridge's 'Sufficiency Rating' such that the result of this improved load rating under this simulation would mean the simultaneous improvement of sufficiency ratings.

Under this scenario it is argued that the true state of deficiency of these bridges is not as bad as is estimated according to current methods, indeed the bridges are more *sufficient* than estimated. This is reflected economically under Scenario 1 by assuming that remedial works on these bridges are not only not required to the extent currently anticipated but that they are deferred for a number of years, as such so are the associated costs in investment.

This scenario simulates a spending deferral for A) 5 years and B) 10 years. The rationale behind the choice of time frames is based in the results of research published by Bolukbasi et al. (2004). Bolukbasi et al.'s research looks at condition rating data for over 2,600 bridges in the State of Illinois, included in which are steel girder bridges with concrete decks. Bolukbasi et al. used this data to create deterioration models to predict the future condition of bridge elements. *Table 4* of this research provides us with information on the duration both the deck and super-structure can be expected, on average, to remain in one condition state before deteriorating to the next. (Bolukbasi, Mohammadi, & Arditi, 2004).

The average time has been estimated as 5 years between falling from one condition state to the next. A period of 10 years then represents two changes in condition state. This information forms the basis for choosing to simulate deferring spending on the Delaware bridges for a period of 5 and 10 years. As the bridges chosen for analysis have been rated as currently having very good/good decks and less than good girders, coupled with the acknowledgement of the action of load path redundancy, it has been deemed reasonable to assume that these bridges could be allowed to go without having remedial measures implemented for either 5 or 10 years, therefore falling by one or two condition ratings. The first implicit assumption here is that load path redundancy increases the bridge's strength and the second is that it increases it enough to be adequate so that remedial works are not required. The details of this scenario are discussed at more length in Section 3.3.2 and results can be found in Table 3-2.

3.2.3 Scenario 2: *Reduced Investment*

Scenario 2 is the second scenario under which the acknowledgment of load path redundancy within the NBI bridge rating practice is simulated. The same assumptions that were made under Scenario 1 are made under Scenario 2: *system* load carrying capacity is in effect, thus load rating and sufficiency rating are improved.

The argument for this is the same as per Scenario 1: the true state of deficiency of the simulation bridges is not as pessimistic as is estimated according to current methods. This implication is reflected economically under Scenario 2 by assuming that remedial works on these bridges are not required to the extent currently anticipated and are not performed to that extent. Scenario 2 therefore simulates performing a reduced degree of remedial work at the present time which causes a reduced amount of spending. This scenario suggests therefore that some funding is given now to the simulation bridges but not to the full extent as is currently estimated under the NBI ratings. The degree to which that spending is reduced was determined based on the results obtained of a recent investigation into the reserve system capacity of a decommissioned bridge.

The thesis put forward by Michaud (Michaud, 2011) compares the load carrying capacity of a steel I-girder bridge as determined by current practice according to AASHTO and the load carrying capacity of the bridge determined by acknowledging path redundancy using destructive load testing and finite element modeling (FEA).

As current practice determines capacity based on the strength of individual members, and not on the system capacity as load path redundancy does, the capacity of the bridge as determined by current practice using the individual member strength was converted to a representative system capacity by summing the strength of the four bridge girders (Michaud, 2011).

Michaud's results (Michaud, 2011) indicate the system load carrying capacity of the bridge as determined using current practice to be 20% less than the system capacity enabled by load path redundancy as determined in the FEA and supported by field testing. If load factors are included in determining the system capacity following current practice, this difference could be as much as 300%!

It is assumed therefore that the amount by which funding is reduced in the simulation for Scenario 2 is 20% of currently estimated requirements, therefore making a direct correlation with additional load carrying capacity. This thesis acknowledges the assumptions involved in making this correlation, however lacking any more sufficient structural data at this time, a decision to make the direct correlation was made. While this 20% reduction in spending will likely not be true for any specific bridge, it may provide a good average estimate for a population of bridges considered here.

3.3 Analysis Details

Before the simulation was performed, a number of mathematical and economic parameters had to be determined, beginning with adopting a constant year for analysis.

The first task involved a transformation of dollars represented in all other years to that constant, or base, year.

The federal funds which are currently estimated for the simulation bridges under the NBI 2010 database are indicative of 2005 dollars. It was decided to adopt 2011 as the constant year (or base year) for the cost-effectiveness simulation of all scenarios; therefore it was required to transform those 2005 dollars to 2011 dollars. Equation 3-1 below indicates the method used to transform those 2005 dollars to 2011 dollars, accounting for the rate of inflation.

Equation 3-1 $\$_{\text{constant year}} = \$_{\text{original year}} * (HCCI_{\text{constant yr}}/HCCI_{\text{original yr}})$

'Constant year' represents 2011 while 'original year' represents 2005 above. 'HCCI' or *Highway Construction Cost Index* is representative of the inflation rates for the associated years as set out by the FHWA (U.S. Department of Transportation, Federal Highway Administration, 2011). For the purposes of the simulation the average of the first two quarters, (the only quarters with available data for 2011,) and the average of all four quarters for 2005 were used as the inflation index, as represented by the 'HCCI' input in Equation 3-1. Those average values are 1.18 and 1.06, for 2005 and 2011 respectively.

The second task was to decide the discount rate, 'r', which is used in obtaining the Net Present Value (NPV) of deferred funds under Scenario 1. The White House's *Office of Management and Budget* circular is the source which dictates the rate to be used for cost-effectiveness analysis on investments involving public funds (i.e. projects funded by tax payer dollars). The real discount rate chosen from those outlined in the Office's circular (The White House, 2011), was that for a 20-year horizon, (1.7%). The rationale behind the choice is that we expect that remedial measures we perform on these bridges will last a long time. The equation used to determine Net Present Value (NPV) is shown below, where 'r' is the discount rate, 1.7% and 't' is the time of deferral in years:

Equation 3-2 $$\text{Amount} * 1/ (1+r)^{t}$$

With the mathematical and economic details set out as described, the simulation is performed and is detailed in the sections following.

3.3.1 Scenario 0: *Simulation & Results*

Details of the simulation and data utilized for Scenario 0-*No Change* are shown in Table 3-1. The simulation shows the federal funds currently estimated for remedial measures to the 14 bridges in Delaware chosen for the simulation as outlined in the NBI 2010 for Delaware. The funds required for remedial works are shown firstly as they appear in the NBI, which is in 2005 dollars ('original year' in the simulation). The equivalent costs in 2011 dollars ('constant year' in the simulation) calculated as per Equation 3-1, are also shown. The sum total of costs under Scenario 0 in constant year dollars are shown to be $23.4 million. All costs are approximate and are represented in thousands of dollars in Table 3-1. This is representative of the total cost assuming all remedial measures as determined using current bridge rating and management practices are invested in and performed today.

Table 3-1 Scenario 0 Simulation

Structure No.	Base YR of Costs ('Original Yr.')	Bridge Improvement Cost ($K 2005)	Bridge Improvement Costs (Constant Yr. $K 2011)
1119 261	2005	297	266.80
1223 6278	2005	2000	1796.61
1229B011	2005	2992	2687.73
1281 366	2005	2000	1796.61
1394S022	2005	2000	1796.61
1440 014	2005	2000	1796.61
1698A000	2005	2000	1796.61
1711 348	2005	1949	1750.80
1814 009	2005	2000	1796.61
1815 009	2005	2000	1796.61
2050A050	2005	2000	1796.61
3131 600	2005	2000	1796.61
3254N003	2005	800	718.64
3818 038	2005	2000	1796.61
Total			**$23.4M**

3.3.2 Scenario 1: *Simulation & Results*

Details of the simulation and data utilized for Scenario 1- *Deferral of Investments* are shown in Table 3-2. Following the methodology set out in Scenario 0, this simulation begins by also showing the federal funds currently estimated for the 14 bridges in Delaware in 2005 dollars and the equivalent costs in 2011 dollars, again calculated as per Equation 3-1. All costs are approximate and are presented in thousands of dollars.

Using the premise of successful load path redundancy enabling greater system load carrying capacity, the simulation next reflects the possible economic effects. With 2011 as the constant year for the simulation, the cost-effectiveness of deferring the suggested remedial measures for those 14 Delaware bridges for 5 and 10 years respectively is simulated using Equation 3-2 and is represented by the NPV of those deferrals, as seen in Table 3-2. The table shows the NPV of deferring investments for remedial works by 5 and 10 years to be $21.5 million and $19.8 million, respectively. All costs are approximate.

Table 3-2 Scenario 1 Simulation

Structure No.	Base YR of Costs ('Original Yr.')	Bridge Improvement Cost ($K 2005)	Bridge Improvement Costs (Constant Yr. $K 2011)	NPV 5 Yr. Deferral (Constant YR $K 2011)	NPV 10 Yr. Deferral (Constant YR $K 2011)
1119 261	2005	297	266.80	245.23	225.41
1223 6278	2005	2000	1796.61	1651.39	1517.90
1229B011	2005	2992	2687.73	2470.48	2270.78
1281 366	2005	2000	1796.61	1651.39	1517.90
1394S022	2005	2000	1796.61	1651.39	1517.90
1440 014	2005	2000	1796.61	1651.39	1517.90
1698A000	2005	2000	1796.61	1651.39	1517.90
1711 348	2005	1949	1750.80	1609.28	1479.20
1814 009	2005	2000	1796.61	1651.39	1517.90
1815 009	2005	2000	1796.61	1651.39	1517.90
2050A050	2005	2000	1796.61	1651.39	1517.90
3131 600	2005	2000	1796.61	1651.39	1517.90
3254N003	2005	800	718.64	660.56	607.16
3818 038	2005	2000	1796.61	1651.39	1517.90
Total			**23390.07**	**21499.42**	**19761.59**
Savings				**$23.4 - $21.5M = $1.9M**	**$23.4 - $19.8M = $3.6M**

3.3.3 Scenario 2: *Simulation & Results*

Details of the simulation and data utilized for Scenario 2- *Reduced Investment* are shown in Table 3-3. The same methodology outlined in Scenario 0 and Scenario 1 is implemented to show to the 2011 dollar cost of suggested remedial measures for the bridges. All costs are approximate and are represented in thousands of dollars.

Using the results of Michaud (2011) as the basis, the premise for Scenario 2 is: successful load path redundancy enables system load carrying capacity which is 20% more effective than current practice dictates. The simulation reflects the economic effects of reducing federal funds currently estimated to be required for those 14 Delaware bridges in the 2010 NBI by 20%, with 2011 as the constant year for comparison. While the assumption of a direct correspondence between load carrying capacity and cost is somewhat arbitrary, the intent is to provide a sense of the potential impact of cost savings.

As seen in Table 3-3 the cost-effectiveness of Scenario 2 is a difference in investment (or funding) requirements between $23.4 million required under current methodology and $18.7 million were spending reduced, as demonstrated with the sum total of investment costs. Again, all costs are approximate.

Table 3-3 Scenario 2 Simulation

Structure No.	Base Yr. of Costs ('Original Yr.')	Bridge Improvement Cost ($K 2005)	Bridge Improvement Costs (Constant Yr. $K 2011)	Bridge Improvement Costs: Investment Reduced 20%
1119 261	2005	297	266.80	213.44
1223 6278	2005	2000	1796.61	1437.29
1229B011	2005	2992	2687.73	2150.18
1281 366	2005	2000	1796.61	1437.29
1394S022	2005	2000	1796.61	1437.29
1440 014	2005	2000	1796.61	1437.29
1698A000	2005	2000	1796.61	1437.29
1711 348	2005	1949	1750.80	1400.64
1814 009	2005	2000	1796.61	1437.29
1815 009	2005	2000	1796.61	1437.29
2050A050	2005	2000	1796.61	1437.29
3131 600	2005	2000	1796.61	1437.29
3254N003	2005	800	718.64	574.92
3818 038	2005	2000	1796.61	1437.29
Total			**23390.07**	**18712.05**
Savings				**$23.4M - $ 18.7M = $4.7M**

3.4 Discussion of Results

The results of the simulations for Scenario 0, Scenario 1 and Scenario 2 lead to a primary observation on the economic reflection of acknowledging load path redundancy in bridge rating practices for steel girder-concrete deck bridges whose super-structure has begun to deteriorate and whose decks are in good condition; the proposed changes are cost-effective in their current scope. That is, they are cost-effective under assuming that the hypothesis of load path redundancy affording additional reserve system load carrying capacity is applicable to the bridges selected.

The Scenario 0 results show that current practices suggests \$23.4 million dollars is estimated to be needed in federal funds for remedial works to these bridges. Comparatively, the Scenario 1 simulation demonstrates that deferral of investments and remedial works by 5 and 10 years will save the state of Delaware \$1.9 million and \$3.6 million, respectively. While the saving may at first appear comparatively small next to the total federal funds estimated to be needed for the state of Delaware for remedial works for 2010, approximately \$591 million,[1] the savings are significant. All cost estimates are again approximate measures. When considering the average cost of remedial works to a bridge, which is calculated at \$2 million, this demonstrates that these savings could be spent elsewhere and are not an insignificant figure in a climate where bridge managers are seeking to save every possible penny.

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[1] The total State allocation was determined using the 2010 NBI for Delaware (Appendix C) and assigning the average funding of \$2 million to those bridges which qualified as deficient, (having a sufficiency rating of 80 or less), and whose specific funding information was absent from the inventory (Federal Highway Administration, Department of Transportation, 2010).

Similarly, a comparison of Scenario 2 results with Scenario 0 indicates the 20% saving in investments to be \$4.7 million, for which a similar cost-effectiveness argument to that for Scenario 1 can be made.

Another way in which the savings seen in Scenario 1 and Scenario 2 could be contextualized is: the amount saved covers the average cost of repair/rehabilitation for 1 or 2 bridges.

Outside of the cost-effectiveness of the approaches and changes simulated in Scenario 1 and Scenario 2, there are additional valuable benefits which can be afforded.

Firstly, deferral of the estimated required funds allows an immediate saving in the bridge improvement budget, though that saving may be comparatively small next to the overall allocation for the state, in theory the amount saved could be invested to gain interest.

Second and perhaps being of more tangible influence, the savings in funding can be targeted instead towards those bridges in the NBI whose sufficiency ratings are more critical and show a greater need for remedial works. The latter benefit is of particular benefit to management in that it can allow an alleviation of a back log of bridges requiring attention and can return a number of “Posted” or otherwise critically deficient bridges to the sufficient state, removing them from the inventory of bridges requiring remedial work.

The results of Scenario’s 0, 1 and 2 however, do not represent the full cost-effectiveness potential of acknowledging load path redundancy in bridge rating. To better understand this potential it is necessary to consider the impact of aging on load path redundancy.

3.5 Considering the Impact of Aging on Potential Cost-Effectiveness

The above scenarios represent the cost-effectiveness of acknowledging load path redundancy in only those composite steel girder-concrete deck bridges whose super-structure has started to deteriorate and whose decks are in good condition, therefore the scope is limited.

The potential benefits to the budgetary funds required for bridge improvements which the additional system load carrying capacity load path redundancy affords can be better demonstrated if composite steel girder-concrete deck bridges whose deck conditions are rated less than "Good" (i.e. a numerical rating of 6), are considered for simulation. It was found that, an additional 25 bridges would qualify as meeting this description (Appendix D).

To this end, a further simulation was performed using those additional 25 bridges whose decks currently achieve a rating below "Good". Scenario 3 and Scenario 4, mirror the methodology utilized for Scenario 1 and Scenario 2, respectively, but apply that methodology to those steel girder-concrete deck bridges in the Delaware 2010 NBI whose super-structure and decks are deteriorating. Note that for these 25 bridges, not all improvement costs were given with 2005 as the base year. Costs were given in 2004, 2006, 2007 and 2009 dollars also. Where this was the case, the average HCCI values for all four quarters were used for those years and were 1.07, 1.35, 1.3 and 1.1, respectively. Similarly to Scenario 1 and Scenario 2, 2005 was assumed as the base year, with associated average remedial costs of $2 million dollars per bridge, where information was absent.

As witnessed in Table 3-4 and Table 3-5, were those 25 bridges included with those 14 which can be rated differently due to acknowledgment of successful load path redundancy, another $185.6 million dollars of funds which are currently estimated to be required for bridge improvements becomes available for simulation of changes in investment and cost-effectiveness analysis.

Results of Scenario 3, represented in Table 3-4 under NPV, show that the ability to defer remedial works to those bridges for 5 and 10 years can potentially save $15 million and $28.8 million dollars respectively. Furthermore, as the results of Scenario 4 in Table 3-5 show, the ability to reduce spending on those bridges can potentially save $37.1 million in required funds ($185.6M - $148.5M). All cost savings are approximate measures.

However the impediment is that the degree to which aging effects the performance of the concrete deck in terms of the role the deck plays as part of load path redundancy is not currently entirely understood. This causes hesitation in making the assumption that the cost-effectiveness simulation of these additional 25 bridges could theoretically be done, or at least making that assumption with the same degree of confidence as was done for the other 14 bridges. It is clear the potential savings and the cost-effectiveness of acknowledging load path redundancy in bridge rating takes on a larger scope if this impediment is removed.

To begin to alleviate this impediment a literature review forming an initial investigation quantifying the influence of aging on the performance of the role of the concrete deck within load path redundancy is completed. That review and its findings are now discussed in Chapter 4.

Table 3-4 Scenario 3 Simulation

Structure No.	Base YR of Costs ('Original Yr.')	Bridge Improvement Cost ($K Original Yr. dollars)	Bridge Improvement Costs (Constant Yr. $K 2011)	NPV 5 Yr. Deferral (Constant Yr. $K 2011)	NPV 10 Yr. Deferral (Constant Yr. $K 2011)
1161 027A	2005 assumed	2000	1796.61	1651.39	1517.90
1130 261	2009	255	245.73	225.86	207.61
1257 018A	2005 assumed	2000	1796.61	1651.39	1517.90
1501 006	2009	8000	7709.09	7085.96	6513.19
1501A6262	2009	1000	963.64	885.74	814.15
1501B6263	2009	1000	963.64	885.74	814.15
1655 031	2005	800	718.64	660.56	607.16
1676 006	2007	1000	815.38	749.48	688.89
1677 006	2005 assumed	2000	1796.61	1651.39	1517.90
1703 387	2005	800	718.64	660.56	607.16
1703A387	2005	800	718.64	660.56	607.16
1714 347	2005	800	718.64	660.56	607.16
1748 059	2005	32649	29328.76	26958.08	24779.02
1759 059	2005	800	718.64	660.56	607.16
1809 369	2005 assumed	2000	1796.61	1651.39	1517.90
2012B012	2009	1600	1541.82	1417.19	1302.64
3154A023	2009	1400	1349.09	1240.04	1139.81
3156 050	2007	150000	122307.69	112421.40	103334.24
3163 493	2005	800	718.64	660.56	607.16
3239 046	2005	126	113.19	104.04	95.63
3254S003	2005	800	718.64	660.56	607.16
3365N002	2005 assumed	2000	1796.61	1651.39	1517.90
3365S002	2005 assumed	2000	1796.61	1651.39	1517.90
1494016	2006	5265	4134.00	3799.84	3492.70
1496002	2004	334	330.88	304.13	279.55
Total			**185613.07**	**170609.73**	**156819.13**
Savings				**$15M**	**$28.8M**

Table 3-5 Scenario 4 Simulation

Structure No.	Base Yr. of Costs ('Original Yr.')	Bridge Improvement Cost (Original Yr. $K)	Bridge Improvement Costs (Constant Yr. $K 2011)	Bridge Improvement Costs Investment Reduced by 20%
1161 027A	2005 assumed	2000	1796.61	1437.29
1130 261	2009	255	245.73	196.58
1257 018A	2005 assumed	2000	1796.61	1437.29
1501 006	2009	8000	7709.09	6167.27
1501A6262	2009	1000	963.64	770.91
1501B6263	2009	1000	963.64	770.91
1655 031	2005	800	718.64	574.92
1676 006	2007	1000	815.38	652.31
1677 006	2005 assumed	2000	1796.61	1437.29
1703 387	2005	800	718.64	574.92
1703A387	2005	800	718.64	574.92
1714 347	2005	800	718.64	574.92
1748 059	2005	32649	29328.76	23463.01
1759 059	2005	800	718.64	574.92
1809 369	2005 assumed	2000	1796.61	1437.29
2012B012	2009	1600	1541.82	1233.45
3154A023	2009	1400	1349.09	1079.27
3156 050	2007	150000	122307.69	97846.15
3163 493	2005	800	718.64	574.92
3239 046	2005	126	113.19	90.55
3254S003	2005	800	718.64	574.92
3365N002	2005 assumed	2000	1796.61	1437.29
3365S002	2005 assumed	2000	1796.61	1437.29
1494016	2006	5265	4134.00	3307.20
1496002	2004	334	330.88	264.70
Total			**185613.07**	**148490.46**
Savings				**$31.7M**

Chapter 4

PRELIMINARY INVESTIGATION OF THE EFFECTS OF AGING ON LOAD PATH REDUNDANCY

As explained in Section 2.3, the success of load path redundancy, which causes enhanced bridge system load carrying capacity, may be heavily dependent on the ability of the deck to transfer load between girders. This successful transfer by the deck is in turn dependent upon its health. This preliminary investigation into the effects of aging on load path redundancy therefore focuses on understanding the deterioration of the concrete deck with aging and quantifying that process in terms of its mechanical properties.

4.1 Deterioration of Reinforced Concrete

Deterioration of concrete can be defined as a reduction in the materials durability, performance and/or life span. Durability is defined herein as the ability to resist deterioration. Performance is defined here as functional operation in service, for example load carrying capacity, bending deflection and other related characteristics. While life span is defined in this thesis as the functional life time relative to the design life.

Concrete can suffer from an array of both physical and chemical processes which cause it to deteriorate. Concrete deterioration can occur due to factors originating within the concrete itself in the form of flawed design and deleterious composition, for example, a water/cement ratio that is too high causing a weak and permeable concrete, or from external factors in the form of poor construction and environmental assailants, for example, poor compaction forming porous joints and chloride ingress.

It is widely believed that chloride induced corrosion of reinforcing steel is one of the most detrimental problems associated with the deterioration of concrete today, significantly impacting the material's durability and consequently reducing its service life (Huang, Bao, & Yao, 2005), (Vorechovska & Podrouzek, 2009).

Zimmermann et al. (2000) go one step further and call this deterioration mechanism not one of but "*the* main cause of damage and early failure of reinforced concrete structures". The chloride induced corrosion process and its effects are now discussed.

4.1.1 Chloride Ion Ingress and the Corrosion of Reinforcing Steel

In terms of the reinforcing steel, hydroxide ions (OH^+), which are constituents of the cement paste, provide alkalinity to pore water, allowing the formation of a layer of ferric oxide (Fe_2O_3) to act as a protective cushion to the reinforcing steel (herein referred to as rebar). This cushion offers protection against chloride ions whose presence can induce corrosion of the rebar. In the case of concrete bridge decks, de-icing salt agents, (herein referred to as deicers), can be a typical sources of chloride ions which ingress into the concrete to cause deterioration.

Chloride ions (Cal^-) which ingress into the concrete and are in the combined presence of moisture, oxygen and OH^+ ions cause a chemical reaction dissolving the rebar by using up the alkaline OH^+ ions and reducing pore water alkalinity, thereby breaking the protective cushion around the steel rebar. When the rebar is fully protected by the hydroxide cushion it is referred to as being in a "passive" state, but when that cushion is broken, the rebar is said to be de-passivated (Rendell, Jauberthie, & Grantham, 2002).

4.1.2 Effects of Chloride Induced Corrosion

Rebar corrosion can present itself as surface spalling or transverse cracking however it can also be present without giving any outward signs (Weyers & Cady, 1987). Spalling and cracking of the concrete occurs due to expansion of the rebar. The latter causes compressive pore stress development within the concrete micro-structure due to volumetric expansions from the formation of rust which is of greater volume than its parent material, steel. This stress development results in fissuring, cracking, spalling and weakening of the concrete (Hobbs, 2001).

Meanwhile inside the pore structure there is a loss of rebar cross-section and/or a reduced bond between the rebar and concrete (Vorechovska & Podrouzek, 2009). Furthermore, due to rust formation, the most insidious effect of this deterioration is the loss of rebar cross section and hence ultimate loss of strength of the concrete section.

The effects of spalling, cracking and mass loss of rebar due to chloride induced corrosion cause more far-reaching and serious effects on the mechanical properties of the concrete deck. The effects on the mechanical properties and their level of severity are now discussed.

4.1.3 Deterioration Effects on Concrete Mechanical Properties

The primary mechanical properties of concrete affected by chloride ion ingress and associated rebar corrosion are mass loss of rebar, strength and stiffness. Both stiffness and strength are influenced by the degree of porosity/permeability of the concrete. Concrete afflicted by the deleterious effects of Cl^- ion ingress and rebar corrosion will have an increased porosity and therefore reduced strength and stiffness (Basheer, Kropp, & Cleland, 2001).

How these mechanical properties are affected are now analyzed further and quantified in the sections following by considering the results of three studies into the deterioration of reinforced concrete beams due to rebar corrosion induced by chloride ion ingress.

4.2 Review of Studies on Concrete Deterioration due to Chloride ion Ingress

There is a minimal amount of literature on the effects of chloride induced corrosion on the mechanical properties of *reinforced* concrete (in contrast to plain concrete which is not of concern within the scope of this thesis). In particular, a comprehensive and uniform quantification of those effects is lacking.

This thesis focuses on three studies which investigate reinforced concrete deterioration due to chloride induced re-bar corrosion. These studies appear to be the extent of good data which is available on reinforced concrete deterioration due to chloride-induced rebar corrosion, and which reflect a natural corrosion methodology. These studies provide data from which conclusions can be made on the effects of this deterioration on the mechanical properties of reinforced concrete beams, which may be applied to bridge decks as is the scope of this thesis.

These studies were chosen because the techniques used in their experiments closely represent a natural environment in which salt (chloride ions) might enter concrete and cause deterioration, for example from percolating snow melt which was melted using deicers. As an aside, another common method used in experimentation to induce chloride corrosion is submersion of reinforced concrete specimens in a chloride-laden water bath with simultaneous application of an electric current to the re-bar. This method however has been cited as an unrealistic representation of the natural corrosion process because the electric current causes uniform corrosion along the re-bar (Li & Zheng, 2005). Additionally, some studies are known to have used corrosion inducing chemicals which were not chloride based and therefore neither representative of the natural corrosion process. Furthermore, the time to achieve an advanced degree of corrosion using the electric current method is significantly less than that required using more natural methods and for this reason it is difficult to translate the time it may take naturally for the same degree of corrosion which would be reflective of the effects on mechanical properties seen in the laboratory using this method. Some examples of these studies are Huang et al. (2005) and Wang et al. (2006).

Furthermore, the three studies which are of focus here, namely, Gu et al. (2010), Li and Zheng (2005) and Oyado et al. (2011), all use specimen sizes which are structurally significant and therefore the results which are obtained from destructive load testing of specimens, which allows conclusions to be made on the results presented in the studies on which can be applied to what can be expected of reinforced concrete in the field. The details of each study are summarized in the following sections.

4.2.1 Study 1 – "*Propagation of Reinforcement Corrosion in Concrete and its Effects on Structural Deterioration*" (Li & Zheng, 2005)

In this study reinforced concrete beams of structurally significant size were corroded in a laboratory setting within an environmental chamber. To induce corrosion, a method reflective of natural exposure was utilized: beams were subjected to salt water spraying with alternate wetting and drying cycles.

The corrosion process was accelerated by two factors. Firstly, the drying periods of the cycles was intensified so as to speed up the absorption of fluid from the beam surface. Intensification of the drying periods was achieved by carrying out the experiment in a special environmental chamber where the climate could be adjusted as required, for example temperature and relative humidity. Secondly, while the entirety of the beams was under general salt spray, an additional direct spraying of the sodium chloride (NaCL) or "salt solution" onto surface cracks was utilized to accelerate the corrosion process.

These cracks were produced by lead weights representing service load conditions and were kept constant on the cantilever ends of beams until such time as the beams were removed from the environmental chamber for destructive load testing.

Residual flexural strength was determined by destructive load testing under 4-point-bending and was measured as the ultimate failure loads of corroded beams in comparison to un-corroded replicate beams. Residual flexural stiffness was measured by the deflection at the cantilever end of corroded beams in comparison to that of un-corroded beams. Three sets of results are presented in the paper, representing the three time periods over which beams were corroded, 3, 5 and 7 months respectively. It was at the end of each of these periods that destructive testing was performed. Mass loss was determined by a different method which is discussed under the following section.

An explanation of how real time corresponds to accelerated laboratory time in this study is now outlined below before results of the study are discussed.

4.2.1.1 Concept of Time Transformation

As these laboratory tests involved *accelerated* corrosion of rebar in beams, a time transformation concept is outlined by Li and Zheng (2005) to allow results to be interpreted for 'natural' exposure to salt (chloride ion ingress). The time transformation is achieved by use of an *'acceleration factor'*. This factor was determined by the authors using long term calibration tests on identical beams under natural exposure or un-accelerated conditions. Details of the test can be found in Li (2000).

The source explicitly and simply states the issue concerning utilization of accelerated time for testing to determine results in natural/real time:

> "The essential problem is to determine the equivalent time period of one cycle of wetting and drying under the accelerated conditions to the real time period of a wetting and drying cycle under natural conditions."

The parameter chosen as the basis on which the acceleration factor could be determined was moisture content of the concrete, measured by weighing of specimens. It was found that:

> "on the average, one natural cycle of wetting and drying takes approximately 47 days as measured by weight of specimen."

Under accelerated testing meanwhile a cycle of wetting and drying lasted 3 days. From this, the following observations and conclusion can be made:

- 1 natural cycle of wetting and drying = 47 days
- 1 accelerated cycle of wetting and drying = 3 days
- 47 'natural' days of wetting and drying represents 1 laboratory cycle

If it is assumed 30 days to a month, then finally the natural time that is represented by one month of accelerated laboratory corrosion can be determined:

- 1 month in laboratory = 30 days/3-day-cycle = 10 accelerated cycles per 1 month in lab.
- 10*47 = 470

Therefore there are 470 natural days per 1 month of laboratory testing.

Therefore in utilizing data from this study the approach was to multiply each 1 month of laboratory testing by 470 to obtain the time in natural days. The number of natural days was then converted to natural months and from that to natural years. For example, for the 3 month Series the following three steps can be used to convert this accelerated laboratory time period to natural time:

1. 3*470 = 1410 natural days
2. 1410/30 = 47 natural months
3. 47/12= 3.79 natural years

The acceleration factor of 47 was used to convert both strength and stiffness data to natural time from accelerated testing time. The situation varied slightly for “mass loss” of rebar data however.

Data for mass loss data is considered in this thesis to be represented by the ‘corrosion current density’ (i_{corr}) readings. The rationale behind this is discussed in detail in Section 4.2.1.2 below. These readings are presented in Fig. 3 of the study by Li and Zheng (2005) and shown below as Figure 4-1. The study uses these corrosion current density (i_{corr}) readings to obtain strength loss measurements by a non-destructive method. When the authors measured strength loss by destructive load testing and compared the results with the corrosion current density method, they found the latter to *underestimate* the strength loss, as is observable between the data shown in Figure 4-2 below. To account for this underestimation, an acceleration factor of 51.7 was used in the study to transform accelerated time to natural time for corrosion current density data (or “mass loss” as it is considered here), as opposed to 47 used for strength and stiffness. As destructive testing shows the true strength (mass) losses at time ‘X’ and the corrosion current density method underestimates those losses at the same time ‘X’, this implies that it requires a longer time if measuring losses by corrosion current density, to obtain the same losses as seen with measuring by destructive load testing. Consequently, the acceleration factor for corrosion current data (mass loss data) is greater than that for strength and stiffness data for transforming accelerated time to natural time (51.7 as opposed to 47, respectively). Mass loss of rebar is now discussed in more detail.

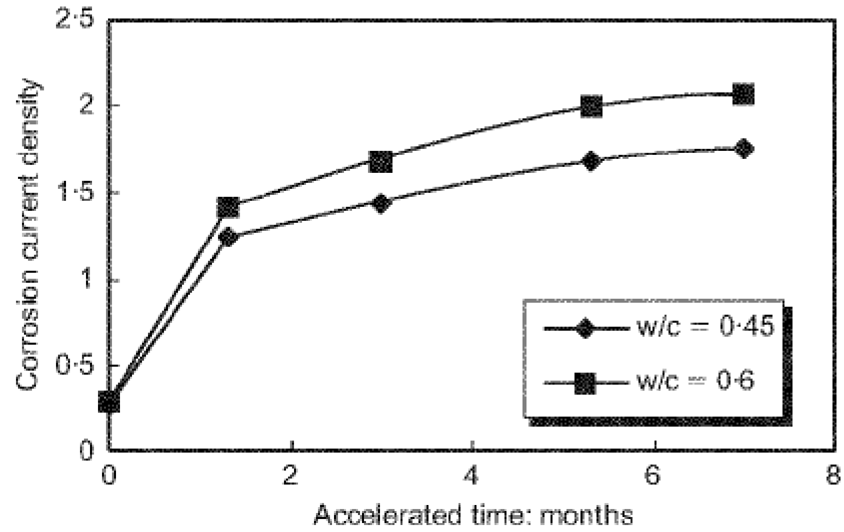

Fig. 3. Corrosion propagation as measured in corrosion current density i_{corr} ($\mu A/cm^2$)

Figure 4-1 Basis for Time Transformation for Li and Zheng Mass Loss Data (Li & Zheng, 2005)

4.2.1.2 Rebar Mass Loss

Rebar mass loss was not measured by gravimetric means. However, the study states that from corrosion current density, (i_{corr}), readings "the metal loss of rebar can be determined using Faraday's Law" and that "this metal loss is then translated to the reductions of cross-sectional area of rebar" (Li & Zheng, 2005). The study indicates that corrosion current density readings have units of "$\mu A/cm^2$", and states that is "uses 1 $\mu A/cm^2$ of corrosion current density to equal to 11.6 µm/year of metal loss of rebar in the radial direction" (Li & Zheng, 2005). Fig. 15 of the study, referred to herein as Figure 4-2, showing strength deterioration represented by corrosion current density readings, was interpreted in for this thesis the change/reduction in corrosion current density readings and therefore as being representative of mass loss. The mass loss results discussed in the following sections of this thesis were extrapolated from Figure 4-2.

In this figure, corrosion current density readings for a period of years in real/natural time are plotted against a deterioration factor. So for example, in the study a deterioration factor of 0.98 at 4 years represents a mass loss of rebar of 2% at 4 years. The maximum mass loss observed in the study is approximately 5.5% after 8.8 years (natural time). Rebar mass loss is a primary cause of strength loss of the concrete beams, which is now deliberated in the following section.

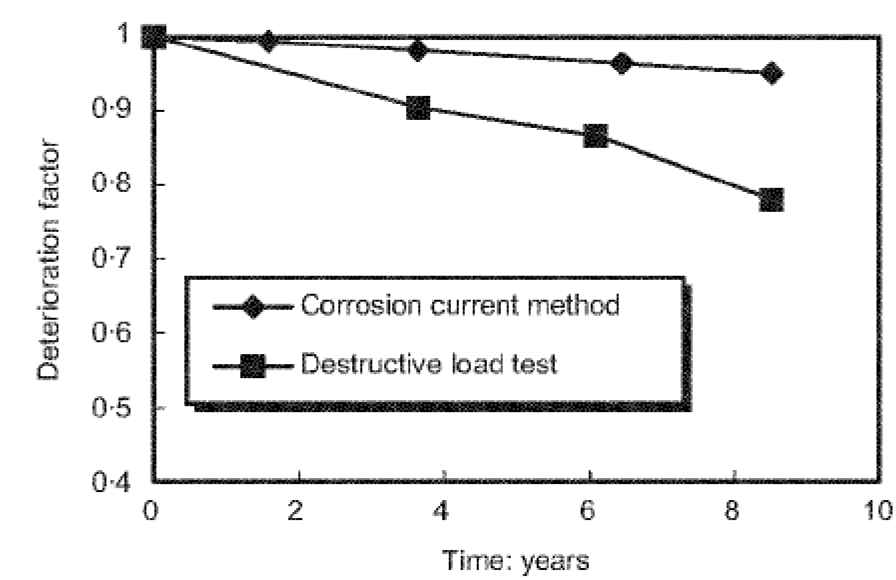

Fig. 15. Comparison of strength deterioration determined by different methods

Figure 4-2 Corrosion Current Density Reading Data used to Derive Strength & Mass Loss Values (Li & Zheng, 2005)

4.2.1.3 Strength Loss

As outlined in Section 4.2.1, strength loss was determined in the study by way of destructive load testing under 4-point-bending of beams which were corroded for 3, 5 and 7 months. Beams were loaded to ultimate failure load and losses were determined by comparison of results obtained with the ultimate failure load of healthy, un-corroded replicate specimens and represented by a deterioration factor.

Similar to the approach for mass loss results discussed in the previous section, the strength loss results discussed in the following sections of this thesis were extrapolated from Figure 4-2 where strength loss readings over real/natural time in years is plotted against the deterioration factor. So for example, in the study a deterioration factor of 0.9 at 4 years represents a strength loss of 10% after 4 years. The maximum strength loss observed in the study is 21.25% after 8.8 years (natural time), where all estimates are approximate. Finally, the method of determining stiffness loss in the study by Li and Zheng is explained in the next section.

4.2.1.4 Stiffness Loss

Beams were loaded under 4-point-bending and deflection at the cantilever end of the beams was measured to represent stiffness whereby the amount of deflection is correlated to the amount of stiffness lost. Load versus deflection data for un-corroded beams and beams corroded for 3 and 7 months (note no data is supplied for 5 month readings by the authors) is given in Fig. 6 of the study, herein referred to as Figure 4-3.

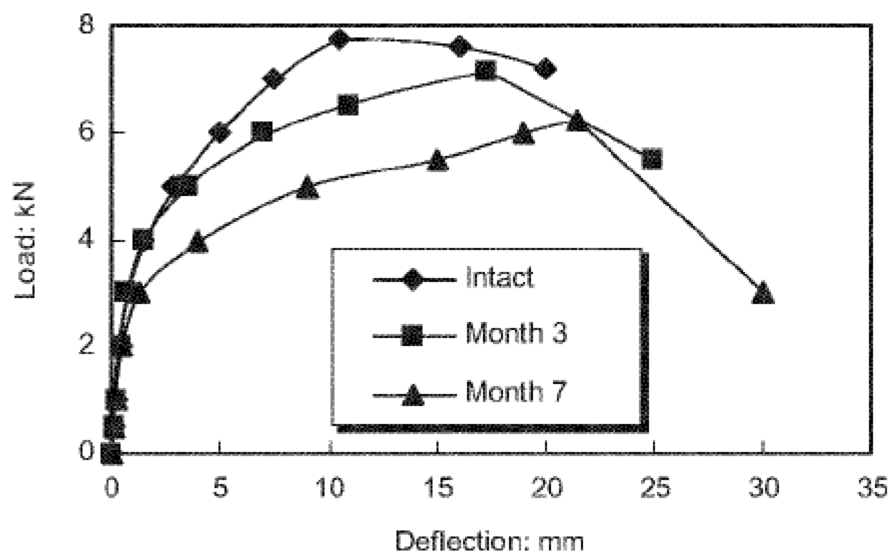

Fig. 6. Load–deflection relationship for different deteriorated specimens

Figure 4-3 Stiffness Loss Basis (Li & Zheng, 2005)

Stiffness losses are expressed by the authors as changes in deflection between healthy beams and corroded beams which are loaded incrementally to maximum failure load. The changes are represented by a deterioration factor. Stiffness losses are explicitly given in Fig. 9 of the study, referred to herein as Figure 4-4, and are represented by the changes in the deterioration factor plotted over accelerated time in months.

For this thesis, the losses in stiffness due to corrosion recorded by the study were interpreted from the data presented in Figure 4-4 by converting accelerated time to real/natural time using the acceleration factor of 47. So in the study for example, a deterioration factor of 0.587 at 4 months of accelerated time indicates a stiffness loss of 41.3% after 5 years (natural time), all estimates being approximate. The maximum stiffness loss observed in the study is about 50% after 8.8 years (natural time). The second study which is reviewed for the preliminary investigation into the effect of aging on load path redundancy is now described in detail. As an aside, the strength data seen in Figure 4-4 below is the same as the strength data shown for destructive load testing in Figure 4-2 above.

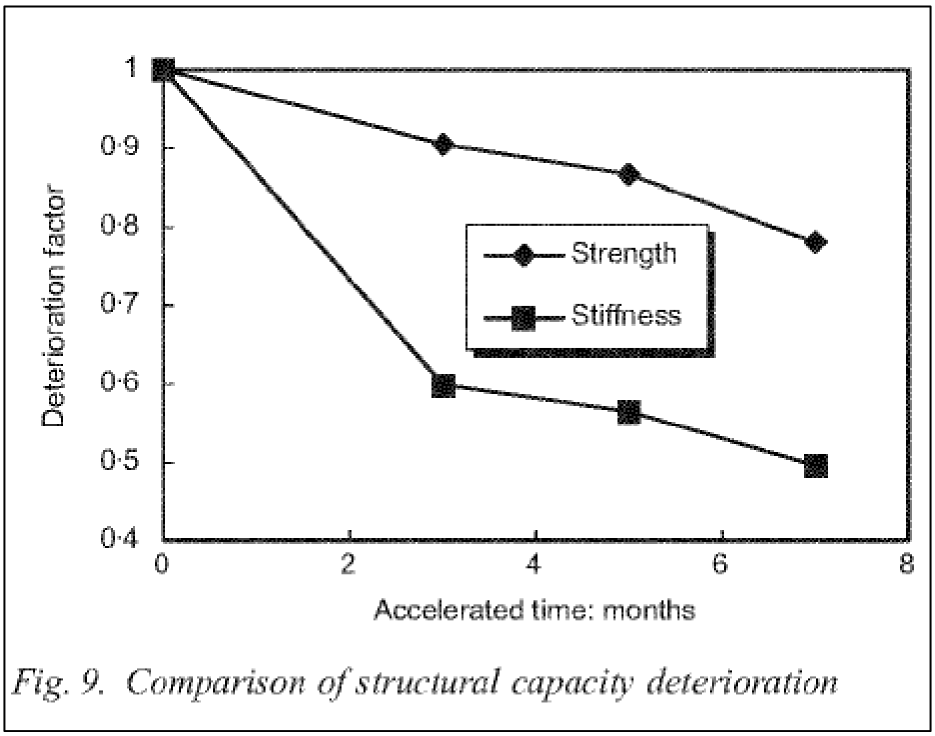

Figure 4-4 Data used to Derive Stiffness Loss Values from Li and Zheng (Li & Zheng, 2005)

4.2.2 Study 2 – "*Bending Performance of Reinforced Concrete Member Deteriorated by Corrosion*" (Oyado, Kanakubo, Sato, & Yamamoto, 2011)

Reinforced concrete beams of structurally significant size were corroded using the 'natural' salt water spraying technique and the greatly accelerated 'electrical' technique described above in section 4.2. As outlined in that section, the latter method is treated with skepticism as to the accuracy and reliability of its results because the corrosion achieved does not realistically represent what occurs in the field i.e. uniform rebar corrosion does not happen in reality. For this reason, this thesis neglects the information from this study on those beams which were corroded using the ‘electrical’ method, namely ‘Series C’ as they are referred to in the study. Thus, the information analyzed in this thesis belongs to those beams of ‘Series S’ and ‘Series M’ which were corroded using the ‘natural’ method which is referred to as ‘EX’ in the study.

In the study, Series S and Series M beams were exposed for 3 months outdoors in an “urban environment away from the coast” (Oyado, Kanakubo, Sato, & Yamamoto, 2011). After which time NaCl solution was sprayed onto cracks in the beams 3 times daily for 17 months to expedite corrosion initiation. Series S beams were removed from the outdoor location at this point, having been exposed for 20 months in total, while Series M continued to be exposed for a total of 12 years. The mass loss of re-bar observed in the study is discussed next.

4.2.2.1 Rebar Mass Loss

Rebar mass loss was measured by gravimetric means whereby “the mass loss of every specimen was measured by weighing rust removed bars” (Oyado, Kanakubo, Sato, & Yamamoto, 2011). What this means is that corroded, rusted bars were removed of their rust until just the parent material, steel, was once again visible and weighed. The corroded weight of the bars was compared against the weight of a replicate un-corroded bar.

The mass loss results analyzed here were extrapolated from data found in Table 4 of the study, referred to herein as Figure 4-5, which shows time plotted against the ‘C’ ratio for each beam. The ‘C’ ratio represents the ratio of mass lost to the original mass of the re-bar. The ratios were simply multiplied by a factor of 100 to obtain the percentage (%) mass loss of re-bar for each specimen. The ‘C’ ratios used for analysis were the “Average” values given in the study.

So for example, from Figure 4-5, for specimen M1, which was corroded for 12 years and has a ‘C’ ratio of 0.14, a mass loss of rebar of 14% is indicated and is also the maximum mass loss observed in this study. The strength losses observed in the study are discussed in the following section.

Table 4. Test result P_u and C.

Corrosion	Name	P_u^*	P_{uc}/P_{un}	C of each test piece
EL	C1	50.6	1.00	–
	C2	43.0	0.85	0.20, 0.22
	C3	42.7	0.84	0.10, 0.11
	C4	36.2	0.72	0.22, 0.26
	C5	8.7	0.17	0.37, 0.50
EX	M1	36.4	0.72	0.13, 0.14, 0.16***
	M2	40.5	0.80	0.13, 0.13, 0.12, 0.15
	M3	42.2	0.83	0.10, 0.11, 0.12, 0.12
	S-0N	52.5	1.00	–
	SD-1N	47.1	0.90	0.00, 0.03
	SD-2N	45.6	0.87	0.03, 0.04
	SD-3N	48.1	0.92	0.00, 0.03
	SD-4N	49.5	0.94	0.02, 0.02

Figure 4-5 Data used to Derive Mass & Strength Loss Values from Oyado et al. (Oyado, Kanakubo, Sato, & Yamamoto, 2011)

4.2.2.2 Strength Loss

Strength loss was determined by way of destructive load testing under 4-point-bending after corrosion for 20 months and 12 years for the Series S and Series M beams, respectively. Beams were loaded to ultimate failure and losses were determined by comparison with the ultimate failure load of an un-corroded replicate specimen (S-0N).

The strength loss results discussed here were based on the data in Figure 4-5 which shows a ratio P_{uc}/P_{un} for each beam type. This is the ratio of the ultimate strength of a corroded beam specimen to the ultimate strength of the un-corroded beam specimen. It should be noted that the beam 'Name' allows indication of the length of corrosion (deterioration) time. For example, beam 'Name', "M1", indicates a beam from the series that has been corroded for 12 years, while beam 'Name', "SD-1N", indicates a beam from the series that has been corroded for 20 months. To obtain the percentage strength loss, the ratio was subtracted from 1.00 and the result multiplied by and factor of 100. So for example, from Figure 4-5, a P_{uc}/P_{un} ratio of 0.9 for beam specimen SD-1N indicates a strength loss of 10% after 20 months of corrosion. The maximum strength loss observed in this paper was 28% after 12 years. A similar discussion of stiffness losses is now made.

4.2.2.3 Stiffness Loss

Beams were loaded under 4-point-bending and deflection at the mid-point of the beams was measured to represent stiffness. Load versus deflection data for Series M was scaled from Figure 6b of the study, reproduced herein as Figure 4-6. No data is given in the study for Series S beams, however as Series M were those beams corroded for the longest period of time the disadvantage of this omission is not so great.

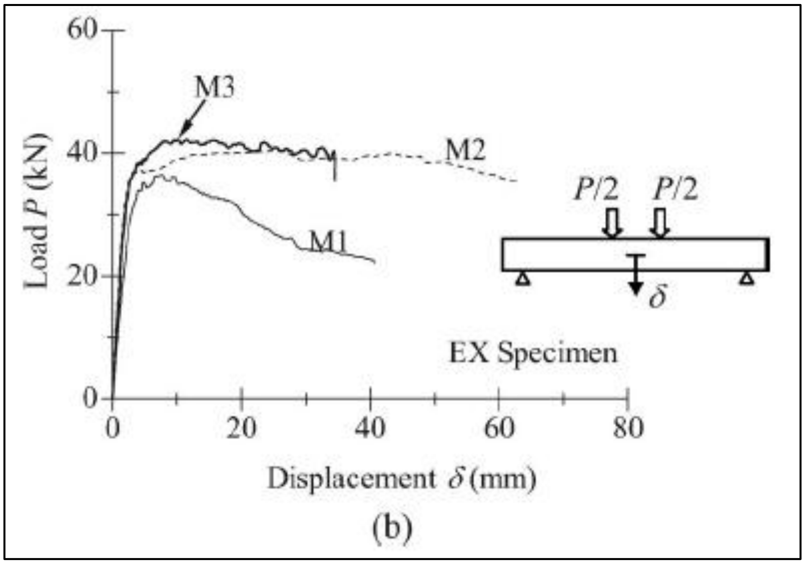

Figure 4-6 Data used to Derive Stiffness Loss Data from Oyado et al. (Oyado, Kanakubo, Sato, & Yamamoto, 2011)

Stiffness losses are expressed as changes in deflection under load between the actual deflections recorded for corroded beams and the expected deflection of a replicate un-corroded beam. The latter was calculated based on assumptions related to material characteristics, information supplied on specimen geometry and the loading case. This information was input to the following formula which, it should be noted, gives elastic deflection, the consequence of which being that deflections at the ultimate load will be underestimated:

Equation 4-1 $Fa/24EI\ (3L^2 - 4a^2)$

The variables in this equation are based both on the loading case seen in Figure 4-6 and the geometry of beam, where: the breadth of the beam, "b", is 100 mm; the depth of beam, "d", is 200 mm; the moment of inertia, "I", is found by using the equation

"$bd^3/12$" and is 66670000 mm^4; based on typical reinforced concrete characteristics, Young's Modulus, "E", is estimated at 26,000 MPa or 26 kN/mm^2 for the beam[2]; the length of the beam, "L", is 1800mm; and finally, the point loads, "P/2", are represented in Equation 4-1 as "F", and the distance between them is represented in the Equation 4-1 by "a", and is 700mm.

Based on the data obtained using Equation 4-1 the recorded ultimate load and ultimate deflection for corroded beams was compared with the calculated deflection of an un-corroded beam under the same load. In this way the difference between both results can represent the percentage loss in stiffness of the beams due to corrosion. The load and deflection data which was scaled from Figure 4-6 to calculate the expected ultimate deflection of M Series beam specimens M1, M2 and M3, is shown below in Table 4-7. The estimated ultimate deflection of un-corroded Series M beams which was calculated using Equation 4-1 is shown along with the observed ultimate deflection in Table 4-8. The percentage of stiffness loss is also indicated there. Maximum losses are estimated to be 73.9% at 12 years, which is for beam M2. However when compared to the losses seen in beams M1 and M3, this value is treated as an outlier as will be discussed further in Section 4.3.3.

The details of the third and final study reviewed are now discussed in the following section.

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[2] This is an estimate obtained from an online source (MATBASE, 2009).

Table 4-1 Data used to Calculate Expected Ultimate Deflection of Series M beams

M1 Load-F (kN)	**M1 Defl.(mm)**	**M2 Load-F (kN)**	**M2 Defl.(mm)**	**M3 Load-F (kN)**	**M3 Defl. (mm)**
10	1.31	10	1.31	10	1.31
20	2.61	20	2.61	20	2.61
30	3.92	30	3.92	30	3.92
36.5	4.77	37	4.83	37	4.83
36.25	4.73	40	5.22	42	5.48

Table 4-2 Estimated Stiffness Losses for Oyado et al. (2011) Data

	M1	**M2**	**M3**
Expected Ult. Defl. (mm) un-corroded beams	4.77	5.22	5.48
Observed Ult. Defl. (mm) corroded beams	8	20	10
Ratio of Expected/Observed Deflection	0.596	0.261	0.548
% Stiffness Loss	40.38	73.90	45.20

4.2.3 Study 3 – *"Flexural Behavior of Corroded Reinforced Concrete Beams"* (Gu, Zhang, Shang, & Wang, 2010)

This study focuses on the use of the 'electrical' method for accelerated corrosion of 3 groups of beams as described in Section 4.2. As discussed, results from studies of this nature are disregarded in this thesis because of the degree of skepticism in the field over their accuracy. The authors of this study recognize those discrepancies and state, "It was found that the accelerated corrosion process, as used in most of the investigations, has quite different effects from the natural corrosion process" (Gu, Zhang, Shang, & Wang, 2010) .

However, this study refers to 'Group D' beam samples taken from "an existing building which has gone through decades of natural corrosion" which would be representative of the 'natural' corrosion process. Unfortunately additional details on the historical particulars of the corrosion process of these beams and the time frame of exposure are not supplied in this study. Certain data is available from this study on rebar mass loss, stiffness losses and strength measurements for those beams. However, it was decided to treat this data as available for informational purposes only rather than as a basis for any conclusions. Perhaps, the most unsatisfying aspect related to this set of beam data is the inability to quantify time of exposure any better than with a description of "decades". Nonetheless, the best available data on rebar mass loss available in the paper is now discussed followed by strength and stiffness loss data.

4.2.3.1 Rebar Mass Loss

Mass loss of rebar was determined by the gravimetric method whereby bars were cleaned and weighed. Given that the beams from Group D are from buildings which had suffered decades of corrosion, it is assumed in this thesis that the rebar diameters for those beams, seen in Figure 4-7 below, were estimated by Gu et al. to be 12mm or that some healthier, more in-tact rebar was present in the beams to allow confirmation of the original rebar size. Similarly, it is assumed here that Gu et al. utilized the average weight of an un-corroded 12mm rebar to obtain the mass lost by corrosion, by comparing that un-corroded weight to the measured weight of the corroded rebars. The average mass loss ratios provided in Table 1 of the study, herein referred to as Figure 4-7, were multiplied by a factor of 100 to obtain the percentage mass loss. The maximum mass loss calculated for Group D beams exposed to "decades of natural corrosion" is estimated at 9.2%. The strength data provided by the study is now considered. As an aside, it is also interesting to note that beam specimens D3 and D4 (Figure 4-7), with the greatest mass loss, 9.2% and 7.5%, respectively, are also noted as having spalling, but the remaining specimen, D2, with a mass loss of 3.4%, does not exhibit this characteristic.

Table 1. Corrosion Progress and Results. (crack width measured in mm)

Group	Beam no.	Rebar diameter	Corrosion duration	Mass loss ratio			Crack width
				Prescribed	Average	Maximum	
A	A1	12	—	—	—	—	—
	A2	12	225	0.10	0.065	0.132	0.7
	A3	12	451	0.20	0.141	0.189	1.49
	A4	12	677	0.30	0.325	0.477	2.2
B	B1	16	—	—	—	—	—
	B2	16	301	0.10	0.084	0.098	0.59
	B3	16	602	0.20	0.103	0.114	0.7
	B4	16	903	0.30	0.210	0.253	1.45
C	C1	14	—	—	—	—	—
	C2	14	1307	0.10	0.062	0.069	1.1
	C3	14	2614	0.20	0.148	0.168	2.5
	C4	14	3920	0.30	0.262	0.421	5.0
D	D2	12	—	—	0.034	0.059	—
	D3	12	—	—	0.092	0.140	Spalling
	D4	12	—	—	0.075	0.193	Spalling

Figure 4-7 Data used to Derive Mass Loss Values from Gu et al. (Gu, Zhang, Shang, & Wang, 2010)

4.2.3.2 Strength

Beams were loaded under 3-point-bending to ultimate failure load. Results are provided in Table 2 of Gu et al.'s study, herein referred to as Figure 4-8. The original ultimate strength of the "decades old" beams remains unknown. It is assumed that beam

D1 is an un-corroded replicate beam and that from the load versus deflection data for beam D1 the original ultimate strength could be extrapolated. That data is shown in Figure 3d of Gu et al., herein referred to as Figure 4-9. This approach would return a strength gain rather than loss for beams corroded over decades which may at first appear illogical, but it should be remembered that these beams were from a building, not a bridge, so they were not subjected to deicing salts, so that larger strength losses would not be expected. However, the fact that these beams are from a building and not a bridge, provides yet another reason why the data in this study is treated as available for informational purposes only. Finally, available data for stiffness losses from this study are discussed in the section immediately following.

Table 2. Comparison of *P* Calculated from Different Methods (unite: kN).

Data source	Beam No.	Test results	Numeric results		Results of Eq. (2)
			Perfect bond	No bond	
This study	A2	38.4	40.0	38.4	35.7
	A3	37.3	36.1	35.3	31.8
	A4	30.6	30.7	29.1	23.9
	B2	57.0	55.9	55.9	59.6
	B3	58.1	60.7	60.7	57.8
	B4	49.8	50.4	49.9	50.6
	C2	64.6	65.2	64.2	57.1
	C3	50.1	51.4	51.0	49.6
	C4	46.2	42.0	37.8	42.0
	D2	42.9	37.2	36.6	37.6
	D3	42.1	37.2	34.0	38.3
	D4	45.5	36.2	33.0	38.2

Figure 4-8 Data used to Derive Strength Loss Values from Gu et al. (Gu, Zhang, Shang, & Wang, 2010)

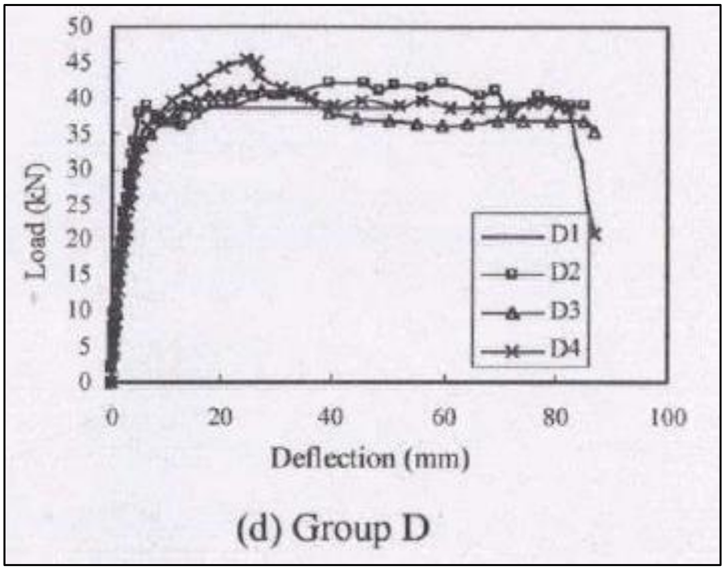

Figure 4-9 Data used to Derive Stiffness Loss Values from Gu et al. (Gu, Zhang, Shang, & Wang, 2010)

4.2.3.3 Stiffness Loss

Beams were loaded under 3-point-bending and deflection at the mid-point of the beams was measured. Figure 4-9 shows the load versus deflection data for Group D beams and appears to indicate, by the presence of beam D1 that a replicate un-corroded beam was tested to obtain stiffness losses, but there is also the possibility that the curve for D1 is a theoretical approximation, due to the bilinear nature of this curve. Those losses are regarded here as the change in deflection under load between the un-corroded and corroded beams at the proportional limit. The "remaining stiffness of corroded RC beams" is given explicitly in Figure 6 of the study, referred to herein as Figure 4-10. From this figure the stiffness loss for Group D beams was scaled off. For example Figure 4-10 shows beam D1 to have a remaining stiffness of 100%, indicating a stiffness loss of

0%. The maximum stiffness losses seen in this study are approximately 15% after "decades".

Having discussed the methods and procedures of the three studies and some of the significant results observed for rebar mass loss, strength and stiffness losses, a number of general observations and conclusions on the data are outlined following.

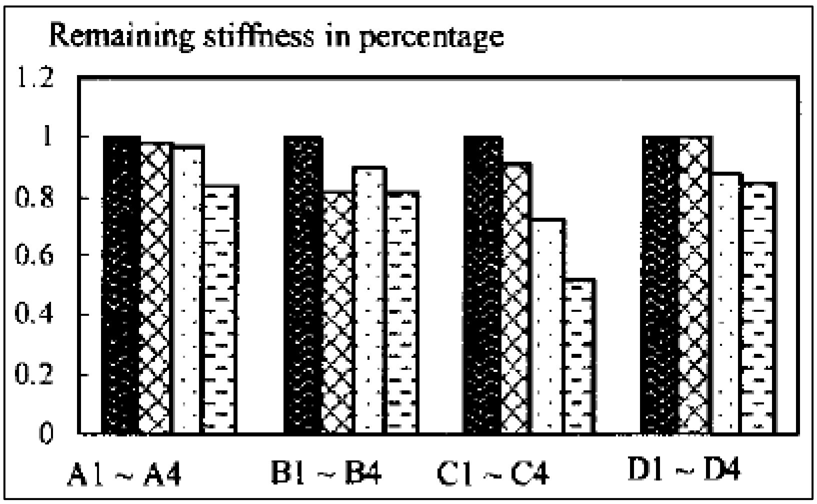

Figure 4-10 Data used to Derive Stiffness Loss Values from Gu et al (Gu, Zhang, Shang, & Wang, 2010)

4.3 Observations and Preliminary Conclusions on the Effects of Aging on Load Path Redundancy

As outlined at the beginning of this chapter, the focus of this preliminary investigation into the effects of aging on load path redundancy is to quantify the deterioration of the mechanical properties of the concrete deck with aging. This quantification should better inform assumptions made on how this deterioration might affect the system load carrying capacity of the bridge and the successful action of load path redundancy.

The effected mechanical properties are strength and stiffness, and mass loss of rebar is one mechanism causing the changes in these mechanical properties which is relatively easily identified. Observations and conclusions in relation to both properties made from the three studies reviewed are discussed below but firstly a number of noteworthy points on the approach to data treatment are made.

As discussed previously the data presented by Gu et al. (2000) has been disregarded for the basis of conclusions relevant to this thesis. Therefore, observations and conclusions herein are based upon synthesized data from the results in the papers by Li and Zheng (2010) and Oyado et al. (2011). The latter studies both use what is considered to be the more natural and realistic method (salt-spraying) to induce corrosion. Furthermore, the paper by Li and Zheng (2010) provides the information and means necessary to transform accelerated corrosion time in the laboratory to 'natural' time. Finally, on the point of time transformation, the details of the procedure laid out within the study by Oyado et al. (2011) indicate that the corrosion process was more "natural" than accelerated. This is interpreted to mean that results in that study do not require a time conversion. While each of these studies has some uncertainty regarding the representative "natural time", results from these two studies correlate well comparatively as will be discussed in sections following.

The observation times for the experiments carried out by Li and Zheng (2010) and Oyado et al. (2011) are estimated at 8.8 years and 12 years, respectively. It is acknowledged that this is a short time frame for analysis from which to draw conclusions on what can be expected in the relatively longer scope (approximately 30-50 years) associated with bridge deck life. None the less, it is assumed that both studies likely have sound scientific reasoning for their choice in the observed testing time.

With the above preface made, observations and conclusions on the results of the studies which can be applied to concrete deck deterioration with aging due to salt exposure and its effects on load path redundancy are now discussed, beginning firstly with strength.

4.3.1 Effects on Strength

Strength losses are regarded here as the reduction in the ultimate flexural load carrying capacity of beams. Results from Li and Zheng (2010) and Oyado et al. (2011) indicate losses in the range of 21-28% after a period of 8-12 years approximately. A summary of results can be observed below in Figure 4-11. Based on that data an "upper bound" of strength loss for the time period studied may be approximated as 30%. This thesis is interested in predicting losses over a longer period of time reflective of typical bridge life. The trends in the data are analyzed in order to make that prediction.

As McConnell et al. (2012) note, the data from the study by Li and Zheng (2010) show a largely linear trend whose associated equation with an R^2 value of 0.987 is:

Equation 4-2 $$\%\ \text{strength loss} = 2.2989 * \text{number of years}$$

In contrast, McConnell et al. (2012) show that the linear trend average results at each time period, (including zero loss at time zero as a data point), from the study by Oyado et al. (2011) returns an R^2 value of 0.5053. This indicates a poor correlation in the Oyado et al. (2011) data when using a linear trend fit. McConnell et al. (2012) conclude this to be due to differences in testing conditions between 20 months and 12 years. That is, after 3 months exposure, beams were sprayed with salt solution three times daily for 17 months at which point Series S beams were removed from exposure for testing and Series M beams remained under natural exposure for 12 years. This difference in exposure conditions between Series S and Series M beams would explain the increased rate of strength loss seen in earlier on in years 0 and 1.67.

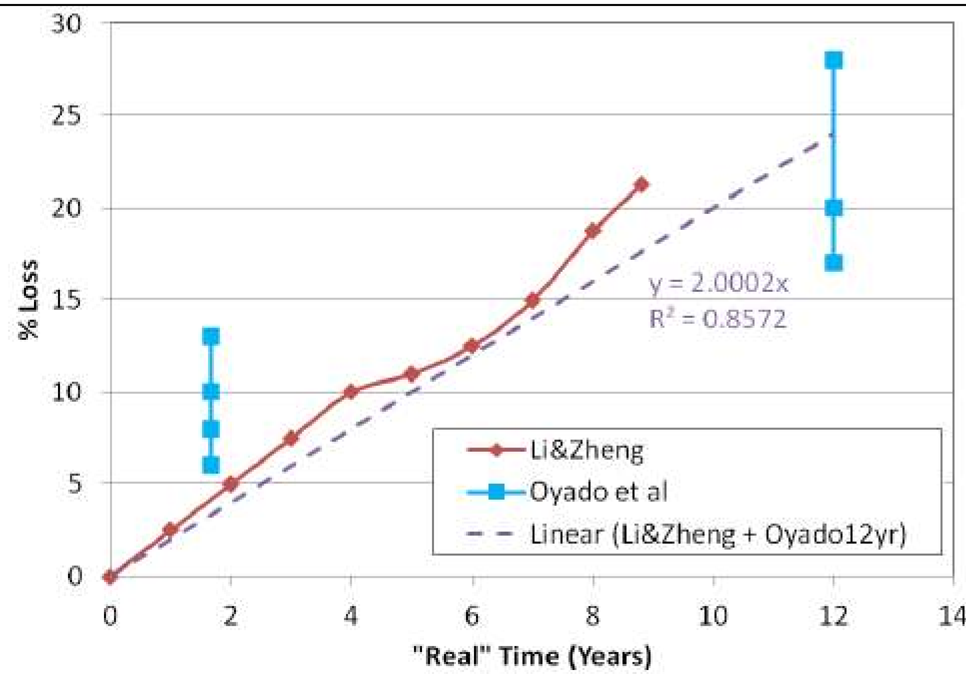

Figure 1. Strength losses reported in literature.

Figure 4-11 Strength Loss Conclusions (McConnell, Mc Carthy, & Wurst, 2012)

McConnell et al. (2012) point to the short period of thrice daily salt spraying to explain the difference between maximum strength losses seen in both studies at the 1.67 year mark. The paper then indicates that if the Li and Zheng (2010) data is extrapolated to the 12 year mark using Equation 4-2 above, there is agreement with the upper range of losses observed by Oyado et al. (2011). McConnell et al. (2012) conclude that both studies are in general agreement with one another as regards strength loss results.

McConnell et al. (2012) predict expected strength losses over a range of time that agrees with that which this thesis is concerned with. They do so by making the following conclusion: strength losses can be represented linearly if uniform test conditions had existed across both studies. The paper uses the linear curve fit for the Li and Zheng (2010) and Oyado et al. (2011) data to make a prediction for a 25 year horizon.

However, the paper states that the Oyado et al. (2011) data for 1.67 years represents specimens sprayed with salt three thrice daily for 17 months. For this reason McConnell et al. (2012) exclude the Oyado et al. results at 1.67 years from the final data set used to make long-term strength predictions.

The final data set which McConnell et al. (2012) use to make strength loss predictions is shown as Figure 4-12. The associated linear equation which predicts a strength loss of 50% after 25 years is:

Equation 4-3 $$\% \text{ strength loss} = 2.000 * \text{number of years}$$

McConnell et al. (2012) acknowledge this to be “an appropriately conservative estimate based on the inherent conservatism of the data”.

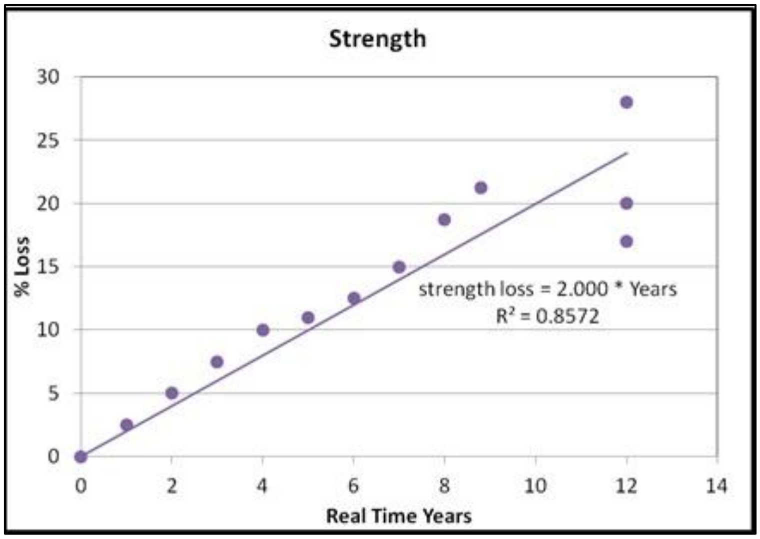

Figure 4-12 Final Strength Loss Predictions over 25 years (McConnell, Mc Carthy, & Wurst, 2012)

4.3.2 Rebar Mass Loss

Rebar mass losses are regarded here as the percentage original rebar mass lost to corrosion. Losses can be estimated in the range of approximately 9-14% after 8-12 years of chloride induced corrosion from the results seen by Li and Zheng (2010) and Oyado et al. (2011). This is treated as an average value however as rebar mass loss is not uniform along rebars, with areas of pitting being commonplace.

Rebar mass loss has been frequently mentioned along with strength in the literature reviewed and it has been concluded that its effects are linked more strongly with a cause of strength losses rather than stiffness losses which are discussed now.

4.3.3 Stiffness Loss

Stiffness losses are regarded here as the change in deflection under load between corroded and un-corroded replicate beams. It is noteworthy that, conversely, when stiffness is expressed as a change in elastic stiffness there are no significant changes observed in either of the studies.

In making a loss prediction equation for the longer 25-year range, McConnell et al. (2012) consider only data supplied by Li and Zheng (2010). The Oyado et al (2011) data is omitted in making this prediction equation because, while mass loss and strength loss data from that study are plentiful, data for stiffness losses is only supplied for Series M beams (12 year exposure). This means a trend in results was not readily retrievable.

Data for Li and Zheng (2010) (Figure 4-13), shows a bi-linear relationship between ultimate deflection and time (McConnell, Mc Carthy, & Wurst, 2012). McConnell et al. (2012) extrapolate the second linear portion of the trend results seen in Figure 4-13 over a long term, 25-year, horizon to predict expected stiffness losses. Figure 4-13 shows that the trend results overlap with the Oyado et al. (2011) data. The associated equation for stiffness loss over time is:

Equation 4-4 (% loss) = 2.0347*number of years + 31.559

Results from Li and Zheng (2010) and Oyado et al. (2011) show losses can be expected in the range of 40-50% after an estimated 8-12 years of exposure to chloride ion ingress. This result ignores the outlier present in the Oyado et al. (2011) results (Figure 4-13) where over a 70% increase in deflection is recorded at 12 years. The extrapolated results by McConnell et al. (2012) shown in Figure 4-13, indicate an 82% increase in ultimate deflection after a 25-year period. In the absence of better data, in particular that related to elastic stiffness, this result is treated as the assumed degree of stiffness loss by which decks in poor condition can be represented in subsequent future analysis and research.

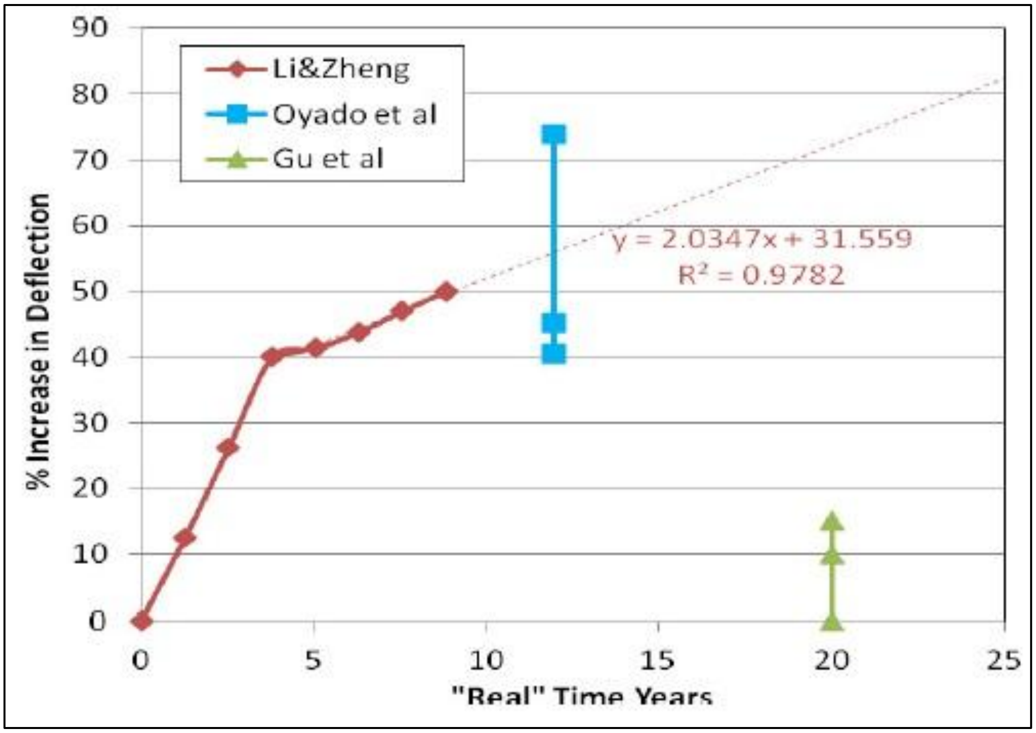

Figure 4-13 Stiffness Loss Conclusions (McConnell, Mc Carthy, & Wurst, 2012)

Chapter 5

CONCLUSIONS

5.1 Summary

This thesis addresses the issue of a current climate of constrained funds for our national bridge inventory by proposing adjustments to current bridge rating practice which would prioritize and optimize budgetary spending to provide more cost-effective asset management.

The approach is a multi-scenario analysis of an economic reflection of the proposed adjustments. Hypothetical changes to the sufficiency ratings of a number of bridges in the state of Delaware are simulated. These changes reflect an acknowledgment of load path redundancy and the additional system load carrying capacity it affords.

The motivation behind this research is the arguably inherent over-conservatism in the current AASHTO bridge load rating practice. This practice does not acknowledge the aforementioned additional system load carrying capacity permitted by load path redundancy and instead determines capacity based on the strength of individual members.

Another motivating factor is that results herein provide savings of taxpayer dollars by way of minimizing estimated federal funds required for allocation to bridge repairs or rehabilitation and/or by way of better allocation of those funds. The latter could lead to a reduction in the list of bridges in critical condition requiring remedial measures and mean that time, resources and money are better spent.

A review of the objectives which this thesis set out to achieve is made now before the significant results and findings of this research are summarized.

5.2 Objectives Reviewed

In Chapter 1 the three primary objectives of this thesis were set out. Those objectives were:

1. To achieve an incorporation of load path redundancy in composite steel girder-concrete deck bridges into the bridge rating process by proposing changes to current process for a sample group of such bridges in a quantified and simulated manner,
2. To simulate changes in the National Bridge Inventory's estimated budget requirements for remedial works to those bridges with simultaneous simulation of changes to budget requirements providing an illustration of the economic benefits (if any) and resource allocation optimization benefits to acknowledging load path redundancy is acknowledged in the bridge rating process, and
3. To perform a preliminary assessment of the influence of aging on load path redundancy by quantifying age-related deterioration of reinforced concrete bridge decks subjected to salt ingress and the effects this may have on the deck performance in terms of load path redundancy in steel girder-concrete deck bridges.

The above, is a summarized version of what appears in Section 1.3. In making the final conclusions it is observed that the primary objectives were met.

Acknowledgement of load path redundancy in the rating process for those bridges described within the scope of this thesis was simulated in a quantified economic manner. This was achieved by using the results of a literature review of contemporary research on the subject of bridge deterioration by Bolukbasi et al. (2004) to decide on a level of deterioration at which the concrete deck can be assumed to retain the ability for providing load path redundancy. Concurrently reviewing current ratings for the deck and super-structure for a set of Delaware bridges and adjusting them based on the findings of the literature review completed the achievement of the first objective and set up the means with which to achieve the second.

The second objective was met in two steps, firstly by simulating a deferral of 2010 repair and rehabilitation measures to a set of 14 bridges in Delaware. The deferral time was based upon the rate of deterioration of the concrete deck as observed in literature reviewed and was chosen to be 5 or 10 years. The deferral represents an economic reflection of implementing proposed changes to rating practice: proposals result in improved sufficiency ratings and a decreased need for remedial measures.

The second step in meeting the second objective was indicating the benefits the proposed changes can bring. These benefits have been stated explicitly as part of the results of the cost-effectiveness analysis in Chapter 3. In summary the illustrated benefits are: an immediate and long term saving of tax payer dollars and an opportunity to better target resources towards bridges in more critical need of remediation measures producing a reduction in the inventory of structurally critical bridges and achieving saving in resources and time spent on remedial works.

The third and final objective of this thesis was met through a successful preliminary assessment of the influence of aging on load path redundancy by quantifying age-related deterioration of reinforced concrete bridge decks subjected to salt ingress and the effects this may have on the deck performance in terms of load path redundancy action in steel girder-concrete deck bridges. This was achieved by way of an extensive literature review returning valuable quantifiable input data for further research using finite element modeling.

The significant results as discussed previously in this thesis are summarized once again in the following section.

5.3 Results Outlined

2010 federal funding for a set of 14 steel girder-concrete deck bridges in Delaware is estimated from data provided in the NBI for that year, at $23.4 million. That set represents bridges whose structural condition appears to easily permit load path redundancy and the additional load carrying capacity it affords.

Cost-effectiveness scenarios assuming improvements in bridge rating to that set of bridges show various degrees of budgetary savings and optimized asset management. Scenario 1 shows that deferral of the annual sum of estimated funds required for that set of bridges by 5 and 10 years permits savings of between $2 million and $3.6 million, respectively, with all values being approximate.

Scenario 2 uses a somewhat loose assumption on the percentage by which spending can be reduced in so far as that percentage is directly correlated to the percentage increase in load carrying capacity when referring to the system load capacity for a prototype bridge field tested in prior work (Michaud, 2011). Nonetheless, this scenario indicates a potential for savings of $4.7 million.

Current limitations to the proposed changes are the number of bridges to which the hypothesis can be applied. A quantification of the effects of aging on load path redundancy is required and the implications this may have on the additional load carrying capacity afforded by LPR under conditions of good health. This quantification would permit an additional 25 bridges whose deck as well as super-structure shows deterioration to be rated confidently using suggested changes to practice.

Incorporation of these additional 25 bridges as simulated in Scenario 3 indicates deferral of spending can save an additional \$15-\$29 million. Scenario 4 indicates the ability to reduce spending on these bridges can potentially save an additional \$39 million. All of these cost values are approximate.

Notably, these savings are in addition to those seen under Scenarios 1 and 2. Therefore, it is concluded that the proposed changes to current bridge rating practices, whereby load path redundancy is acknowledged, are cost effective within the scope of the assumptions made for the first 14 bridges assessed. Furthermore, those proposed changes have the potential to be cost-effective outside of this scope, pending the results of other ongoing research by McConnell and Wurst, which is elaborated upon below in Section 5.4.

The anticipated effects of aging on the mechanical properties of reinforced concrete were found to be: a 50% loss of strength after 25 years; an 84% increase in deflection after 25 years with no loss of elastic stiffness observed; both strength and stiffness losses are tied to mass loss of re-bar with the former being more acutely so.

Further conclusions are: immediate savings are permitted by deferral of spending requirements; the dollar amounts saved could potentially be invested to earn interest over time; where monetary savings or deferrals are not made, prioritization of remedial works can be made by targeting spending which would have been made on the simulation bridges towards bridges in more critical need of rehabilitation allowing possibly even the removal of a ‘posted’ status from a number of bridges and saving time and resources which would have otherwise been spent elsewhere.

To end, a number of suggestions for future research and investigation are outlined following.

5.4 Proposals for Future Research

Given that there are some limitations to the scope of this research, a number of proposals are made here for further research. Suggestions made here could be used to further develop the findings of this thesis.

Firstly, of particular merit for further research is the quantification of the degree to which age-related deterioration of the concrete deck affects the additional load carrying capacity afforded by load path redundancy in steel girder-concrete deck bridges (found to be 20% by Michaud (2011)). This is an area currently under investigation by Professor Jennifer McConnell and Diane Wurst of the University of Delaware. Their research involves the implementation of finite element modeling in order to make the aforementioned quantification. It is anticipated those results will be published in the near future.

Following that publication, it is proposed here that the cost-effectiveness scenario which considers reduced spending be re-addressed. This scenario could be revised to acknowledge the results of the work by McConnell and Wurst. This would allow consideration of any changes to the anticipated potential monetary savings.

Additionally, the results of the ongoing research by McConnell and Wurst will aid in permitting a better estimate on the degree to which spending might be reduced under the cost-effectiveness analysis (reference to the 20% estimate utilized in this thesis). In other words, the amount by which repair and rehabilitation budgetary allocations can be reduced could be more accurately estimated when based on better informed structural analysis. This is in comparison to the methodology utilized here i.e. a direct correlation with the percentage increase in load carrying capacity.

It is suggested that the scope of the methodology and analysis outlined in this thesis be widened to include bridges outside of Delaware. This is feasible by addressing NBI data for all of the 50 states. The approach presented here could therefore be utilized easily to investigate the scale of the benefits on a national level by adjusting current bridge rating practice and bridge management to acknowledge load path redundancy.

Finally, this thesis recognizes the lack of good data in the available literature on bridge aging as regards the effects on mechanical properties of the concrete deck. In particular is a lack of good data on the effects on chloride (salt) ingress on reinforced concrete. Plain concrete in contrast has an abundant amount of literature available which addresses this topic, which any typical bibliographic search will return. Therefore, a suggested topic of further research is development of good data which quantifies the effects of long term aging and age-related deterioration, in particular chloride (salt) ingress, on the mechanical properties of reinforced concrete.

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APPENDICES

Appendix A

STRUCTURE INVENTORY & APPRAISAL FORM

EXAMPLE DATA

OMB No. 2125-0501
NATIONAL BRIDGE INVENTORY - - - - - - - STRUCTURE INVENTORY AND APPRAISAL 10/15/94

********** IDENTIFICATION **********************
(1) STATE NAME - YOUR STATE NAME CODE 999
(8) STRUCTURE NUMBER
(5) INVENTORY ROUTE (ON/UNDER) - ON = 131000440
(2) HIGHWAY AGENCY DISTRICT 03
(3) COUNTY CODE 075 (4) PLACE CODE 59767
(6) FEATURES INTERSECTED - SR 772, ROARING LION R. *
(7) FACILITY CARRIED - STATE ROUTE 44
(9) LOCATION - 9.7 KM SW. OF RICHMOND
(11) MILEPOINT/KILOMETERPOINT 0036.008
(12) BASE HIGHWAY NETWORK - PART OF NET CODE 1
(13) LRS INVENTORY ROUTE & SUBROUTE #000000277503
(16) LATITUDE 35 DEG 27 MIN 18.55 SEC
(17) LONGITUDE 081 DEG 05 MIN 50.65 SEC
(98) BORDER BRIDGE STATE CODE 888 % SHARE 40 %
(99) BORDER BRIDGE STRUCTURE NO. #ABC003790243009

********** STRUCTURE TYPE AND MATERIAL *********
(43) STRUCTURE TYPE MAIN: MATERIAL - STEEL
TYPE - DECK TRUSS CODE 309
(44) STRUCTURE TYPE APPR: MATERIAL - STEEL
TYPE - GIRDER & FLOORBEAM SYSTEM CODE 303
(45) NUMBER OF SPANS IN MAIN UNIT 002
(46) NUMBER OF APPROACH SPANS 0004
(107) DECK STRUCTURE TYPE - CONCRETE C-I-P CODE 1
(108) WEARING SURFACE / PROTECTIVE SYSTEM:
A) TYPE OF WEARING SURFACE - CONCRETE CODE 1
B) TYPE OF MEMBRANE - NONE CODE 0
C) TYPE OF DECK PROTECTION - UNKNOWN CODE 8

********** AGE AND SERVICE *********************
(27) YEAR BUILT 1948
(106) YEAR RECONSTRUCTED 0000
(42) TYPE OF SERVICE: ON - HIGHWAY-PEDESTRIAN
UNDER - HIGHWAY-WATERWAY CODE 56
(28) LANES: ON STRUCTURE 02 UNDER STRUCTURE 02
(29) AVERAGE DAILY TRAFFIC 019500
(30) YEAR OF ADT 1993 (109) TRUCK ADT 05 %
(19) BYPASS, DETOUR LENGTH 013 KM

********** GEOMETRIC DATA **********************
(48) LENGTH OF MAXIMUM SPAN 0097.5 M
(49) STRUCTURE LENGTH 00312.0 M
(50) CURB OR SIDEWALK: LEFT 00.0 M RIGHT 02.5 M
(51) BRIDGE ROADWAY WIDTH CURB TO CURB 007.9 M
(52) DECK WIDTH OUT TO OUT 011.8 M
(32) APPROACH ROADWAY WIDTH (W/SHOULDERS) 12.2 M
(33) BRIDGE MEDIAN - NO MEDIAN CODE 0
(34) SKEW 00 DEG (35) STRUCTURE FLARED NO
(10) INVENTORY ROUTE MIN VERT CLEAR 99.99 M
(47) INVENTORY ROUTE TOTAL HORIZ CLEAR 07.9 M
(53) MIN VERT CLEAR OVER BRIDGE RDWY 99.99 M
(54) MIN VERT UNDERCLEAR REF - HIGHWAY 10.46 M
(55) MIN LAT UNDERCLEAR RT REF - HIGHWAY 06.2 M
(56) MIN LAT UNDERCLEAR LT 00.0 M

********** NAVIGATION DATA *********************
(38) NAVIGATION CONTROL - BR PERMIT REQ CODE 1
(111) PIER PROTECTION - FUNCTIONING CODE 2
(39) NAVIGATION VERTICAL CLEARANCE 18.3 M
(116) VERT-LIFT BRIDGE NAV MIN VERT CLEAR __.__ M
(40) NAVIGATION HORIZONTAL CLEARANCE 047.2 M

****

SUFFICIENCY RATING = 37.4
STATUS = STRUCTURALLY DEFICIENT

********** CLASSIFICATION ******************CODE
(112) NBIS BRIDGE LENGTH - YES
(104) HIGHWAY SYSTEM - ROUTE ON NHS 1
(26) FUNCTIONAL CLASS - OTHER PRIN ART URBAN 14
(100) DEFENSE HIGHWAY - NOT DEFENSE 0
(101) PARALLEL STRUCTURE - NONE EXISTS N
(102) DIRECTION OF TRAFFIC - 2 WAY 2
(103) TEMPORARY STRUCTURE - NOT TEMPORARY _
(105) FEDERAL LANDS HIGHWAYS - NOT APPLICABLE 0
(110) DESIGNATED NATIONAL NETWORK - PART OF NET 1
(20) TOLL - ON FREE ROAD 3
(21) MAINTAIN - STATE HIGHWAY AGENCY 01
(22) OWNER - STATE HIGHWAY AGENCY 01
(37) HISTORICAL SIGNIFICANCE - NOT ELIGIBLE 5

********** CONDITION ********************** CODE
(58) DECK 4
(59) SUPERSTRUCTURE 5
(60) SUBSTRUCTURE 6
(61) CHANNEL & CHANNEL PROTECTION 8
(62) CULVERTS N

********** LOAD RATING AND POSTING ******** CODE
(31) DESIGN LOAD - H-15 OR M-13.5 2
(63) OPERATING RATING METHOD - LOAD FACTOR 1
(64) OPERATING RATING - MS-14 25.2
(65) INVENTORY RATING METHOD - LOAD FACTOR 1
(66) INVENTORY RATING - MS-11 19.8
(70) BRIDGE POSTING - POSTING REQUIRED 2
(41) STRUCTURE OPEN, POSTED OR CLOSED - P
DESCRIPTION - POSTED FOR LOAD

********** APPRAISAL ********************** CODE
(67) STRUCTURAL EVALUATION 5
(68) DECK GEOMETRY 3
(69) UNDERCLEARANCES, VERTICAL & HORIZONTAL 6
(71) WATERWAY ADEQUACY 8
(72) APPROACH ROADWAY ALIGNMENT 8
(36) TRAFFIC SAFETY FEATURES 1100
(113) SCOUR CRITICAL BRIDGES 8

********** PROPOSED IMPROVEMENTS ***************
(75) TYPE OF WORK - REPLACE FOR DEFICIENCY CODE 311
(76) LENGTH OF STRUCTURE IMPROVEMENT 00317.0 M
(94) BRIDGE IMPROVEMENT COST $ 4,200,000
(95) ROADWAY IMPROVEMENT COST $ 300,000
(96) TOTAL PROJECT COST $ 5,000,000
(97) YEAR OF IMPROVEMENT COST ESTIMATE 1995
(114) FUTURE ADT 025600
(115) YEAR OF FUTURE ADT 2014

********** INSPECTIONS *************************
(90) INSPECTION DATE 03/94 (91) FREQUENCY 12 MO
(92) CRITICAL FEATURE INSPECTION: (93) CFI DATE
A) FRACTURE CRIT DETAIL - YES - 06 MO A) 09/94
B) UNDERWATER INSP - NO - __ MO B) __/__
C) OTHER SPECIAL INSP - NO - __ MO C) __/__

Figure A- 1 Structure Inventory & Appraisal Form

Appendix B

SAMPLE SUFFICIENCY RATING EXAMPLE

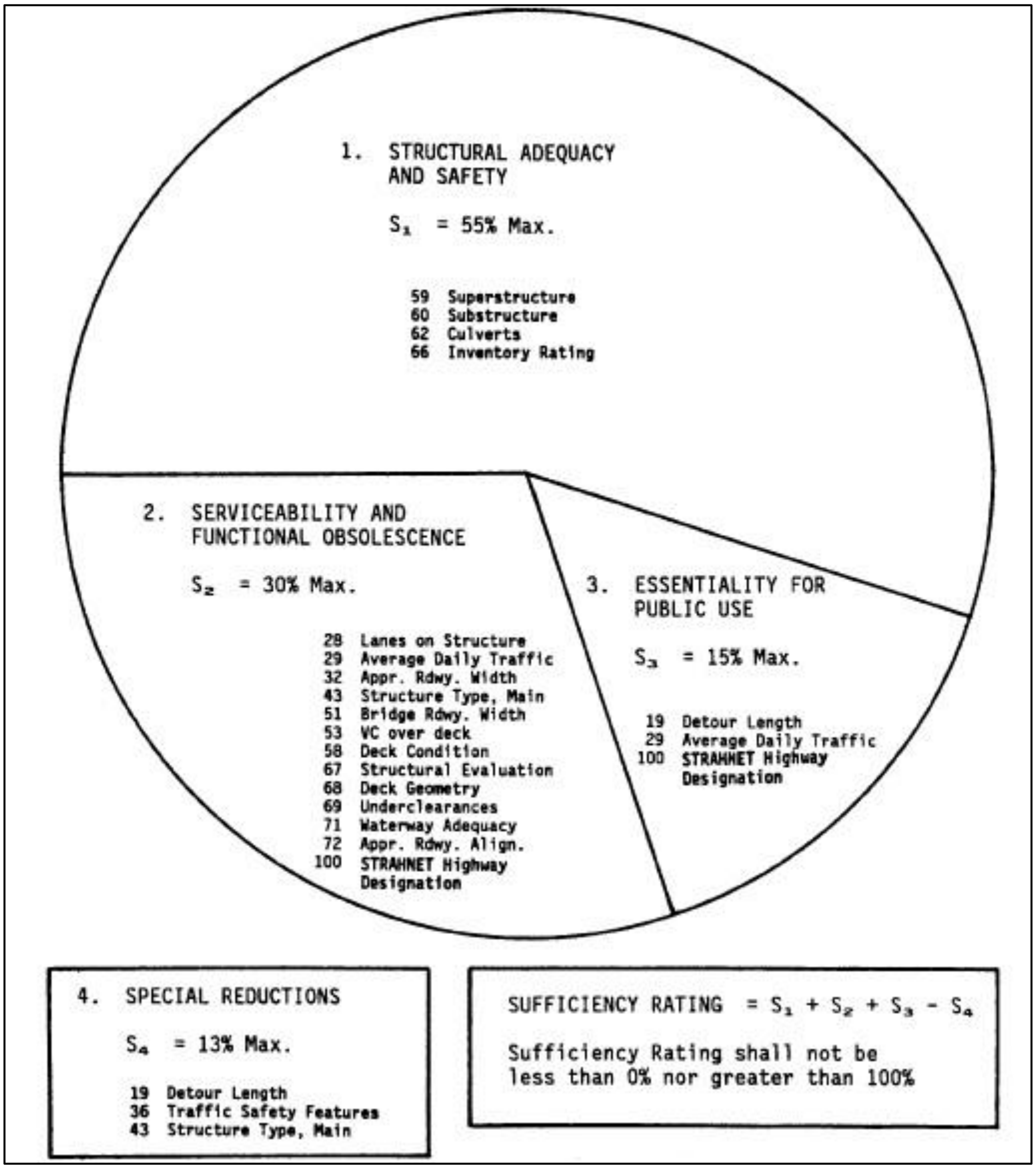

Figure B- 1 Sufficiency Rating Example (U.S. D.O.T., FHWA, 2011)

1. Structural Adequacy and Safety (55% maximum)

a. Only the lowest rating code of Item 59, 60, or 62 applies.

If Item 59 (Superstructure Rating) or Item 60 (Substructure Rating) is	≤ 2	then	A = 55%
	= 3		A = 40%
	= 4		A = 25%
	= 5		A = 10%

If Item 59 and Item 60 = N and Item 62 (Culvert Rating) is	≤ 2	then	A = 55%
	= 3		A = 40%
	= 4		A = 25%
	= 5		A = 10%

b. Reduction for Load Capacity:

Calculate using the following formulas where IR is the Inventory Rating (MS Loading) in tons or use Figure 2:

$$B = (32.4 - IR)^{1.5} \times 0.3254$$

or

If $(32.4 - IR) \leq 0$, then $B = 0$

"B" shall not be less than 0% nor greater than 55%.

$$\mathbf{S_1 = 55 - (A + B)}$$

S_1 shall not be less than 0% nor greater than 55%.

Figure B-1 continued

2. Serviceability and Functional Obsolescence (30% maximum)

a. Rating Reductions (13% maximum)

If #58 (Deck Condition) is	≤ 3	then	A = 5%
	= 4		A = 3%
	= 5		A = 1%
If #67 (Structural Evaluation) is	≤ 3	then	B = 4%
	= 4		B = 2%
	= 5		B = 1%
If #68 (Deck Geometry) is	≤ 3	then	C = 4%
	= 4		C = 2%
	= 5		C = 1%
If #69 (Underclearances) is	≤ 3	then	D = 4%
	= 4		D = 2%
	= 5		D = 1%
If #71 (Waterway Adequacy) is	≤ 3	then	E = 4%
	= 4		E = 2%
	= 5		E = 1%
If #72 (Approach Road Alignment) is	≤ 3	then	F = 4%
	= 4		F = 2%
	= 5		F = 1%

J = (A + B + C + D + E + F)

J shall not be less than 0% nor greater than 13%.

b. Width of Roadway Insufficiency (15% maximum)

Use the sections that apply:

(1) applies to all bridges;
(2) applies to 1-lane bridges only;
(3) applies to 2 or more lane bridges;
(4) applies to all <u>except</u> 1-lane bridges.

Also determine X and Y:

$$X \text{ (ADT/Lane)} = \frac{\text{Item 29 (ADT)}}{\text{first 2 digits of \#28 (Lanes)}}$$

$$Y \text{ (Width/Lane)}^* = \frac{\text{Item 51 (Bridge Rdwy. Width)}}{\text{first 2 digits of \#28 (Lanes)}}$$

Figure B-1 continued

(1) Use when the last 2 digits of #43 (Structure Type) are not equal to 19 (Culvert):

If(#51 + 0.6 meters) < #32 (Approach Roadway Width) G = 5%

(2) For 1-lane bridges only, use Figure 3 or the following:

If the first 2 digits of #28 (Lanes) are equal to 01 and

$Y < 4.3$	then	$H = 15\%$
$Y \geq 4.3 < 5.5$		$H = 15\left[\frac{5.5-Y}{1.2}\right]\%$
$Y \geq 5.5$		$H = 0\%$

(3) For 2 or more lane bridges. If these limits apply, do not continue on to (4) as no lane width reductions are allowed.

If the first 2 digits of #28 = 02 and $Y \geq 4.9$, $H = 0\%$

If the first 2 digits of #28 = 03 and $Y \geq 4.6$, $H = 0\%$

If the first 2 digits of #28 = 04 and $Y \geq 4.3$, $H = 0\%$

If the first 2 digits of #28 $\geq$ 05 and $Y \geq 3.7$ $H = 0\%$

(4) For all <u>except</u> 1-lane bridges, use Figure 3 or the following:

If $Y < 2.7$ and $X > 50$	then	$H = 15\%$
$Y < 2.7$ and $X \leq 50$		$H = 7.5\%$
$Y \geq 2.7$ and $X \leq 50$		$H = 0\%$

If $X > 50$ but ≤ 125 and

$Y < 3.0$	then	$H = 15\%$
$Y \geq 3.0 < 4.0$		$H = 15(4-Y)\%$
$Y \geq 4.0$		$H = 0\%$

If $X > 125$ but ≤ 375 and

$Y < 3.4$	then	$H = 15\%$
$Y \geq 3.4 < 4.3$		$H = 15(4.3-Y)\%$
$Y \geq 4.3$		$H = 0\%$

Figure B-1 continued

If $X > 375$ but ≤ 1350 and

$Y < 3.7$	then	$H = 15\%$
$Y \geq 3.7 < 4.9$		$H = 15\left[\frac{4.9 - Y}{1.2}\right]\%$
$Y \geq 4.9$		$H = 0\%$

If $X > 1350$ and

$Y < 4.6$	then	$H = 15\%$
$Y \geq 4.6 < 4.9$		$H = 15\left[\frac{4.9 - Y}{1.2}\right]\%$
$Y \geq 4.9$		$H = 0\%$

G + H shall not be less than 0% nor greater than 15%.

c. Vertical Clearance Insufficiency - (2% maximum)

If #100 (STRAHNET Highway Designation) > 0 and

#53 (VC over Deck) $\geq$ 4.87	then	I = 0%
#53 < 4.87		I = 2%

If #100 = 0 and

#53 $\geq$ 4.26	then	I = 0%
#53 < 4.26		I = 2%

$$S_2 = 30 - [\ J + (G + H) + I\ ]$$

S_2 shall not be less than 0% nor greater than 30%.

3. Essentiality for Public Use (15% maximum)

a. Determine:

$$K = \frac{S_1 + S_2}{85}$$

Figure B-1 continued

b. Calculate:

$$A = 15\left[\frac{\#29(ADT)x\#19(DetourLength}{320{,}000xK}\right]$$

"A" shall not be less than 0% nor greater than 15%.

c. STRAHNET Highway Designation:

If #100 is > 0	then	B = 2%
If #100 = 0	then	B = 0%

$S_3 = 15 - (A + B)$

S_3 shall not be less than 0% nor greater than 15%.

4. Special Reductions (Use only when $S_1 + S_2 + S_3 \geq 50$)

a. Detour Length Reduction, use Figure 4 or the following:

$A = (\#19)^4 \ x \ (7.9 \ x \ 10^{-9})$

"A" shall not be less than 0% nor greater than 5%.

b. If the 2nd and 3rd digits of #43 (Structure Type, Main) are equal to 10, 12, 13, 14, 15, 16, or 17; then

B = 5%

c.

If 2 digits of #36 (Traffic Safety Features)	= 0	C = 1%
If 3 digits of #36	= 0	C = 2%
If 4 digits of #36	= 0	C = 3%

$S_4 = A + B + C$

S_4 shall not be less than 0% nor greater than 13%.

Sufficiency Rating = $S_1 + S_2 + S_3 - S_4$

The Rating shall not be less than 0% nor greater than 100%.

Figure B-1 continued

EXAMPLE

Calculation of Sufficiency Rating

1. Structural Adequacy and Safety

   A = 10%

   B = $[32.4 - (19.8 \text{ metric tons})]^{1.5} \times 0.3254 = 14.6$

   $S_1 = 55 - (10 + 14.6) = 30.4$

2. Serviceability and Functional Obsolescence

   A = 3%, B = 1%, C = 4%, D = NA, E = NA, F = NA

   J = (3 + 1 + 4) = 8%

   $X = \frac{18500}{2} = 9250 \quad Y = \frac{7.9 \text{ m}}{2} = 3.95$

   (1) If (7.9 + 0.6) < 12.2 then G = 5

   (2) Not Applicable

   (3) Not Applicable

   (4) If X = 9250 and Y = 3.95 then H = 15

   G + H = 5 + 15 = 20 (however, maximum allowable = 15)

   I = 0

   $S_2 = 30 - [8 + (15) + 0] = 7.0$

3. Essentiality For Public Use

   $K = \frac{30.4 + 7.0}{85} = 0.44$

   $A = 15\left[\frac{18{,}500 x 12.8 Km}{320{,}000 x 0.44}\right] = 25.2 (however, \max .allowable = 15)$

   B = 0

Figure B-1 continued

4. Special Reductions

$S_1 + S_2 + S_3 = (30.4 + 7.0 + 0.0) = 37.4 < 50$

$\mathbf{S_4} = \mathbf{NA}$

SUFFICIENCY RATING = 30.4 + 7.0 + 0.0 = 37.4

Figure B-1 continued

Appendix C

NBI 2010 DATA FOR DE BRIDGES ASSUMED TO CURRENTLY ENABLE LOAD PATH REDUNDANCY

Table C- 1 NBI 2010 Data for Delaware Bridges Assumed to Currently Enable Load Path Redundancy

STRUCTURE NUMBER (008)	FEATURES DESC (006A)	FACILITY CARRIED (007)	LOCATION (009)	YEAR BUILT (027)	AGE	OPEN/CLOSED/POSTED (041)		STRUCTURE KIND (043A)
1119 261	'RED CLAY CREEK '	'CREEK RD/SR82 '	'ASHLAND '	1939	73	A	Open, No Restriction	3
1223 6278	'KOREAN WAR VET MEM HGWY '	'MALL ENTRANCE '	'CHRISTIANA MALL '	1978	34	A	Open, No Restriction	4
1229B011	'WHITE CLAY CREEK '	'CAPITOL TRAIL/SR2 '	'WINDY HILLS N/E NEWARK '	1955	57	A	Open, No Restriction	3
1281 366	'CHRISTINA CREEK '	'CHRISTINA PARKWAY '	'NEWARK '	1980	32	A	Open, No Restriction	4
1394S022	'DRAWYERS CREEK '	'US13SB/DUPONT PKY '	'NORTH OF ODESSA '	1964	48	A	Open, No Restriction	4
1440 014	'CATTAIL BRANCH '	'SUMMIT BRIDGE RD '	'N. TOWNSEND LIMITS '	1936	76	A	Open, No Restriction	3
1698A000	'MILL RACE '	'SOUTH PARK DR '	'BRANDYWINE PARK '	1978	34	A	Open, No Restriction	3
1711 348	'I 95 / JFK MEM HGWY '	'SALEM CHURCH RD '	'EAST OF NEWARK '	1963	49	A	Open, No Restriction	3
1814 009	'NORFOLK SOUTHERN '	'12TH ST '	'WILMINGTON '	1977	35	A	Open, No Restriction	4
1815 009	'I-495 RD 60 '	'TWELTH ST '	'WILMINGTON '	1977	35	A	Open, No Restriction	4
2050A050	'BEAVERDAM DITCH '	'HALLTOWN RD/SR8 '	'EAST OF MARYDEL '	1922	90	A	Open, No Restriction	3
3131 600	'NANTICOKE RIVER '	'FAWN RD '	'NE OF BRIDGEVILLE '	1930	82	A	Open, No Restriction	3
3254N003	'NANTICOKE RIVER '	'US 13 N & SR 20 W '	'S/W SEAFORD '	1951	61	A	Open, No Restriction	3
3818 038	'SLAUGHTER CREEK '	'CODS ROAD '	'WEST OF FOWLER BEACH '	1931	81	A	Open, No Restriction	3

Table C-1 continued

STRUCTURE TYPE (043B)	Structure Kind & Type	DECK STRUCTURE TYPE (107)	Deck Type	DECK COND (058)	Deck Condition	SUPERSTRUCTURE COND (059)
3	Steel:Girder & Floor-Beam System	1	Concrete:Cast-in-Place	7	Good	5
2	Steel-Continuous:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	7	Good	6
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	7	Good	5
2	Steel-Continuous:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	7	Good	6
2	Steel-Continuous:Stringer/Multi-Beam/Girder	2	Concrete:Precast-Panels	8	Very Good	6
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	7	Good	5
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	8	Very Good	5
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	7	Good	6
2	Steel-Continuous:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	7	Good	5
2	Steel-Continuous:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	7	Good	6
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	7	Good	6
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	7	Good	5
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	7	Good	5
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	7	Good	5

Table C-1 continued

Super-Structure Condition	WORK PROPOSED(075A)	Proposed Work
Fair (minor section loss/cracking/spalling)	35	Bridge rehabilitation because of general structure deterioration or inadequate strength
Satisfactory (minor deterioration)		
Fair (minor section loss/cracking/spalling)	35	Bridge rehabilitation because of general structure deterioration or inadequate strength
Satisfactory (minor deterioration)		
Satisfactory (minor deterioration)		
Fair (minor section loss/cracking/spalling)		
Fair (minor section loss/cracking/spalling)		
Satisfactory (minor deterioration)	35	Bridge rehabilitation because of general structure deterioration or inadequate strength
Fair (minor section loss/cracking/spalling)		
Satisfactory (minor deterioration)		
Satisfactory (minor deterioration)		
Fair (minor section loss/cracking/spalling)		
Fair (minor section loss/cracking/spalling)	31	Replacement of bridge or other structure because of substandard load carrying capacity or substandard
Fair (minor section loss/cracking/spalling)		

Table C-1 continued

BRIDGE IMP COST (094)	YEAR OF IMP (097) BASE YR COSTS	YEAR RECONSTRUCTED (106) (last improved)	SUFFICIENCY RATING
297	2005	2003	72.9
		0	59
2992	2005	2002	55.4
		0	79.3
		1995	78.2
		1948	79.8
		0	74.1
1949	2005	0	77.6
		0	65.7
		0	75.7
		1952	78
		0	78.4
800	2005	1996	70.8
		0	67.5

Appendix D

NBI 2010 DATA FOR DE BRIDGES WHICH COULD POTENTIALLY ENABLE LOAD PATH REDUNDANCY

Table D-1 NBI 2010 Data for Delaware Bridges which could Potentially Enable Load Path Redundancy

STRUCTURE NUMBER (008)	FEATURES DESC (006A)	FACILITY CARRIED (007)	LOCATION (009)	YEAR BUILT (027)	AGE	OPEN/CLOSED/POSTED (041)		STRUCTURE KIND (043A)
1161 027A	'LITTLE MILL CREEK '	'OLD DUPONT RD. '	'ELSMERE '	1926	86	A	Open, No Restriction	3
1130 261	'RED CLAY CREEK '	'MT CUBA ROAD '	'S/E OF ASHLAND '	1969	43	A	Open, No Restriction	3
1257 018A	'CHRISTINA RIVER '	'MAIN ST/CHRISTIANA'	'CHRISTIANA '	1937	75	A	Open, No Restriction	3
1501 006	'CHRISTINA R. AMTRAK	'SR 141 '	'NEWPORT '	1978	34	A	Open, No Restriction	4
1501A6262	'AYRES ST '	'RAMP SR141N TO SR4'	'NEWPORT '	1978	34	A	Open, No Restriction	4
1501B6263	'AYRES ST '	'SR 4 TO SR 141 S '	'NEWPORT '	1978	34	A	Open, No Restriction	4
1655 031	'CSX RAILROAD '	'LIMESTONE RD / SR7'	'N/W OF STANTON	1960	52	A	Open, No Restriction	3
1676 006	'I 95 & I 295 SB & 6089 '	'BASIN RD/SR 141 NB'	'NEWPORT '	1963	49	A	Open, No Restriction	3
1677 006	'I 95 & I 295 SB & 6089 '	'BASIN RD/SR 141 SB'	'NEWPORT '	1963	49	A	Open, No Restriction	3
1703 387	'I 95 / JFK MEM HGWY	'S. COLLEGE AVE. NB'	'SOUTH OF NEWARK	1963	49	A	Open, No Restriction	3
1703A387	'I 95 DELAWARE TURNP	'S. COLLEGE AVE. SB'	'SOUTH OF NEWARK	1970	42	A	Open, No Restriction	3
1714 347	'I 95 DELAWARE TURNP	'CHAPMAN RD '	'E. OF NEWARK '	1963	49	A	Open, No Restriction	3
1748 059	'MARYLAND AVE. & AM	'I 95 '	'SOUTH OF WILMINGTON	1964	48	A	Open, No Restriction	3
1759 059	'BRANDYWINE CK RAILI	'I 95 RD 59 '	'WILMINGTON '	1964	48	A	Open, No Restriction	4
1809 369	'I 495 '	'ROGERS RD '	'EAST OF WILMINGTON	1975	37	A	Open, No Restriction	4
2012B012	'LEIPSIC RIVER '	'SMYRNA-LEIPSIC RD '	'LEIPSIC '	1952	60	A	Open, No Restriction	4
3154A023	'LEWES-REHOBOTH CAN	'FREEMAN HIGHWAY '	'LEWES '	1966	46	A	Open, No Restriction	4
3156 050	'INDIAN RIVER INLET	'SR1 / COASTAL HGWY'	'INDIAN RIVER INLET '	1965	47	A	Open, No Restriction	4
3163 493	'BROAD CREEK '	'BETHEL RD. '	'BETHEL '	1968	44	A	Open, No Restriction	4
3239 046	'DEEP CREEK '	'OLD FURNACE RD. '	'OLD FURNACE MILL	1933	79	A	Open, No Restriction	3
3254S003	'NANTICOKE RIVER	'US 13 S & SR 20 E '	'S/W SEAFORD '	1951	61	A	Open, No Restriction	3
3365N002	'RECORDS POND '	'SUSSEX HWY/US13 NB'	'EAST OF LAUREL '	1951	61	A	Open, No Restriction	3
3365S002	'RECORDS POND '	'SUSSEX HWY / US13 '	'EAST OF LAUREL '	1951	61	A	Open, No Restriction	3
1494016	'C&D CANAL '	'DEL 896 US71 301S'	'9 MI W OF REEDY POINT	1959	53	A	Open, No Restriction	3
1496002	'C&D CANAL '	'DELAWARE RT. 9 '	'1/2 MI S OF DEL. CITY '	1969	43	A	Open, No Restriction	3

Table D-1 continued

STRUCTURE TYPE (043B)	Structure Kind & Type	DECK STRUCTURE TYPE (107)	Deck Type	DECK COND (058)	Deck Condition
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	5	Fair (minor section loss/cracking/spalling)
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	5	Fair (minor section loss/cracking/spalling)
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
6	Steel-Continuous:Box-Beam/Girder(single or spread)	1	Concrete:Cast-in-Place	5	Fair (minor section loss/cracking/spalling)
6	Steel-Continuous:Box-Beam/Girder(single or spread)	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
6	Steel-Continuous:Box-Beam/Girder(single or spread)	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	5	Fair (minor section loss/cracking/spalling)
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
2	Steel-Continuous:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
2	Steel-Continuous:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	5	Fair (minor section loss/cracking/spalling)
2	Steel-Continuous:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
2	Steel-Continuous:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
2	Steel-Continuous:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
2	Steel-Continuous:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	5	Fair (minor section loss/cracking/spalling)
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
2	Steel:Stringer/Multi-Beam/Girder	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
10	Steel:Truss-Thru	1	Concrete:Cast-in-Place	6	Satisfactory (minor deterioration)
10	Steel:Truss-Thru	1	Concrete:Cast-in-Place	5	Fair (minor section loss/cracking/spalling)

Table D-1 continued

SUPERSTRUCTURE COND (059)	Super-Structure Condition	WORK PROPOSED (075A)	Proposed Work
6	Satisfactory (minor deterioration)		
4	Poor (advanced section loss/spalling/advanced deterioration)	35	Bridge rehabilitation because of general structure deterioration or inadequate strength
5	Fair (minor section loss/cracking/spalling)		
3	ierious (section loss/spalling/shear cracks/local failure possible	35	Bridge rehabilitation because of general structure deterioration or inadequate strength
4	Poor (advanced section loss/spalling/advanced deterioration)	35	Bridge rehabilitation because of general structure deterioration or inadequate strength
4	Poor (advanced section loss/spalling/advanced deterioration)	35	Bridge rehabilitation because of general structure deterioration or inadequate strength
6	Satisfactory (minor deterioration)	31	Replacement of bridge or other structure because of substandard load carrying capacity
6	Satisfactory (minor deterioration)	34	Widening of existing bridge with deck rehabilitation or replacement
6	Satisfactory (minor deterioration)		
6	Satisfactory (minor deterioration)	31	Replacement of bridge or other structure because of substandard load carrying capacity
6	Satisfactory (minor deterioration)	31	Replacement of bridge or other structure because of substandard load carrying capacity
6	Satisfactory (minor deterioration)	31	Replacement of bridge or other structure because of substandard load carrying capacity
6	Satisfactory (minor deterioration)	35	Bridge rehabilitation because of general structure deterioration or inadequate strength
5	Fair (minor section loss/cracking/spalling)	31	Replacement of bridge or other structure because of substandard load carrying capacity
5	Fair (minor section loss/cracking/spalling)		
5	Fair (minor section loss/cracking/spalling)	35	Bridge rehabilitation because of general structure deterioration or inadequate strength
5	Fair (minor section loss/cracking/spalling)	35	Bridge rehabilitation because of general structure deterioration or inadequate strength
5	Fair (minor section loss/cracking/spalling)	31	Replacement of bridge or other structure because of substandard load carrying capacity
6	SAtisfactory (minor deterioration)	31	Replacement of bridge or other structure because of substandard load carrying capacity
6	SAtisfactory (minor deterioration)	35	Bridge rehabilitation because of general structure deterioration or inadequate strength
6	Satisfactory (minor deterioration)	31	Replacement of bridge or other structure because of substandard load carrying capacity
5	Fair (minor section loss/cracking/spalling)		
5	Fair (minor section loss/cracking/spalling)		
5	Fair (minor section loss/cracking/spalling)	35	Bridge rehabilitation because of general structure deterioration or inadequate strength
6	SAtisfactory (minor deterioration)	38	Other structural work, including hydraulic replacements

Table D-1 continued

BRIDGE IMP COST (094) (in 1,000's $)	YEAR OF IMP (097) BASE YR COSTS	YEAR RECONSTRUCTED (106) (last improved)	SUFFICIENCY RATING
		2000	54.1
255	2009	0	63.9
		1997	70.8
8000	2009	0	36
1000	2009	0	69.9
1000	2009	0	64.4
800	2005	1995	71
1000	2007	2001	71.1
		2001	75.9
800	2005	1971	79.8
800	2005	0	76.4
800	2005	1999	77.8
32649	2005	1980	77
800	2005	1984	73.1
		0	77.1
1600	2009	1998	50.5
1400	2009	0	56
150000	2007	1976	52
800	2005	0	75.1
126	2005	0	73.2
800	2005	1996	70.8
		2003	78
		2003	79.2
5265	2006	1987	70.7
334	2004	1985	73.1

Appendix E

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