PROBABILISTIC LIFE-CYCLE COST ASSESSMENT OF STAINLESS STEEL REINFORCED CONCRETE BRIDGE GIRDER UNDER MULTIPLE HAZARDS

By

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CERTIFICATION

The project/thesis titled **"Probabilistic Life-Cycle Cost Assessment of Stainless Steel Reinforced Concrete Bridge Girder Under Multiple Hazards"** submitted by Tauhidul Islam, Roll No: 18MDEM009P, Session-2018-2019 has been accepted as satisfactory in partial fulfilment of the requirements for the degree of **Master of Engineering in Disaster Engineering and Management** on 12th December 2024.

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Dedication

To My Parents & Youngest Sister's Musaiva Fartune Intiva

Approval/Declaration by the Supervisor

This is to certify that **Tauhidul Islam** has carried out this research work under my supervision, and he has fulfilled the relevant Academic Ordinance of the Chittagong University of Engineering & Technology, so he is qualified to submit the following project/thesis in the application for the degree of MASTER OF ENGINEERING IN DISASTER ENGINEERING AND MANAGEMENT. Furthermore, the Project complies with the PLAGIARISM and ACADEMIC INTEGRITY regulations of CUET.

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Copyright Declaration

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Abstract

Reinforced concrete (RC) bridges are considered as the most important node for the transportation networks which play very crucial role in sustainable economic development of any nation. However, performance of reinforced concrete bridge with ordinary rebar under multiple hazards is found to be deteriorated severely. In order to maintain the performance of the bridge over a threshold value, regular inspection and maintenance actions are required to ensure their continuous service leading to higher lifetime cost. In addition to the increase in lifetime cost, the inspection and maintenance activities may also tremendously raise the indirect costs to the users due to traffic delays and loss of productivity. Furthermore, the inspection and maintenance activities may also hinder the continuous traffic movement over the bridge and the consequences in negative impact on environment and social life. Although, use of stainless steel rebar in constructing RC girder will increase construction cost, however, it will reduce the necessity of inspection and repair activities leading to lower lifetime cost. The higher construction cost of SS-RC bridge will be completely outweighed by incentives obtained from the lower repair and maintenance costs after a certain time referred as pay-off time ($t_{pay\text{-}off}$) for stainless steel reinforcement economic viability which primarily depends on severity of airborne chloride hazard, price of SS rebar, and design service life of the bridge. The key purpose of this study is to determine $t_{pay\text{-}off}$ for SS rebar economic viability by determining the time required to obtain identical life-cycle cost (LCC) value of SS- and CS-RC bridges. To fulfil this objective, two holistic flowcharts were developed to determine $t_{pay\text{-}off}$ for SS rebar cost-efficiency. Furthermore, the proposed approach was illustrated using an RC bridge girder considering different coastal marine environments in 20 cities in

Japan and the variation in SS price. The findings will assist transportation officials in deciding on selecting rebar type and cost of SS rebar to construct sustainable bridge with a nominal lifetime cost.

বিমূর্ত

রিইনফোর্সড কংক্রিট (আরসি) সেতুগুলি পরিবহন নেটওয়ার্কগুলির জন্য সবচেয়ে গুরুত্বপূর্ণ নোড হিসাবে বিবেচিত হয় যা যেকোনো জাতির টেকসই অর্থনৈতিক উন্নয়নে অত্যন্ত গুরুত্বপূর্ণ ভূমিকা পালন করে। যাইহোক, একাধিক (ট্রাফিক এবং এয়ারবর্ন ক্লোরাইড) হ্যাজারড বা বিপত্তির অধীনে সাধারণ রিবার দিয়ে তৈরি RC ব্রিজের গার্ডারগুলির কার্যকারিতা গুরতর ভাবে খারাপ হতে দেখা যায়। সেতুর কর্মক্ষমতা থ্রেশহোল্ড মানের উপর বজায় রাখার জন্য, নিয়মিত পরিদর্শন এবং রক্ষণাবেক্ষণের পদক্ষেপগুলি নিয়মিত ভাবে গ্রহণ করতে হবে যাতে করে সেতুটি দীর্ঘদিন ব্যবহার করা যায়। নিয়মিত ইন্সপেকশন এবং মেইনট্যানেনসের কারণে ট্রাফিকে অনেক বিলম্ব হয় এবং উৎপাদনশীলতা ব্যাহত হয়। এই কারণে ইনডিরেক্ট খরচ ও বেড়ে যায়। এছাড়া ও নিয়মিত পরিদর্শন এবং রক্ষণাবেক্ষণ কার্যক্রম সেতুর উপর দিয়ে ক্রমাগত যান চলাচলে বাধা সৃষ্টি করতে পারে এবং এর ফলে পরিবেশ ও সামাজিক জীবনে নেতিবাচক প্রভাব ও পড়তে পারে। এই ধরনের সমস্যা দূর করতে, ক্ষয়জনিত ব্যর্থতার সম্ভাবনা কমাতে এবং পরিদর্শন ও রক্ষণাবেক্ষণের ক্রিয়াকলাপের সাথে যুক্ত খরচ কমাতে কার্বন ইস্পাত (CS) এর পরিবর্তে এখানে একটি প্রতিশ্রুতিশীল ক্ষয় প্রতিরোধী স্টেইনলেস স্টিল (SS) রিবার প্রস্তাব করা হয়েছে। যদিও, আরসি ব্রিজ নির্মাণে SS এসএস রিবার প্রয়োগের ফলে নির্মাণ ব্যয় বাড়বে। তবে, এটি পরিদর্শন ও মেরামত কার্যক্রমের প্রয়োজনীয়তা হ্রাস করবে যার ফলে জীবনকাল কম খরচ হবে। নির্দিষ্ট একটা সময় পরে এসএস-আরসি সেতুর উচ্চতর নির্মাণ ব্যয় সম্পূর্ণরূপে কম মেরামত এবং রক্ষণাবেক্ষণ খরচ থেকে প্রাপ্ত প্রণোদনা থেকে বেশি হয়ে যাবে যাকে এখানে SS rebar খরচ-কার্যকারিতা এর পে-অফ টাইম (tpay-off) হিসাবে উল্লেখ করা হয়েছে। যেটি প্রাথমিকভাবে বায়ুবাহিত ক্লোরাইডের তীব্রতার, এসএস রিবারের দাম এবং সেতুর ডিজাইন সার্ভিস লাইফ এর উপর নির্ভর করে। এই অধ্যয়নের মূল উদ্দেশ্য হল , SS rebar খরচ-কার্যকারিতার জন্য tpay-off নির্ধারণ করা। এটা করার জন্য ,CS- এবং SS-RC সেতুগুলির অভিন্ন জীবন-চক্র খরচ (LCC) মান বের করার জন্য প্রয়োজনীয় সময় বের করতে হবে। এই উদ্দেশ্য পূরণের জন্য, SS rebar খরচ দক্ষতার জন্য tpay-off নির্ধারণ করতে একটি সামগ্রিক ফ্লোচার্ট তৈরি করা হয়েছিল। এছাড়াও, প্রস্তাবিত পদ্ধতিটি জাপানের 20টি শহরের বিভিন্ন উপকূলীয় সামুদ্রিক পরিবেশ এবং এসএস মূল্যের তারতম্য বিবেচনা করে একটি RC ব্রিজ গার্ডার ব্যবহার করে চিত্রিত করা হয়েছিল।

ফলাফলগুলি পরিবহন অফিশিয়ালদের কে নামমাত্র জীবনকাল খরচ সহ টেকসই সেতু নির্মাণের জন্য রিবারের প্রকার এবং এসএস রিবারের খরচ নির্বাচন করার সিদ্ধান্ত নিতে সহায়তা করবে।

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GLOSSARY OF TERMS

RC	Reinforced Concrete
SS	Stainless Steel
SUS	Steel Use Stainless
LCC	Life-Cycle Cost
JIS	Japan Industrial Standard
CS	Carbon Steel
W/C	Water-to-cement ratio
pH	Potential of Hydrogen
DLS	Durability Limit State
ULS	Ultimate Limit State
r	Wind to land Ratio
Y_1	Gaussian Distributed random variable
Y_2	lognormal based uncertainty related attenuation rule
$F_{Y1}(y_1)$	Likelihood Density functions (PDFs) of Y1
$F_{Y2}(y_2)$	likelihood density functions (PDFs) of Y2
MCS	Monte Carlo simulation
PDF	Probability Density Function
M	Chloride level in rebar
m	Designated concrete cover
Y_3	lognormal random variable describing model uncertainty with the estimate of C0
E_c	Concrete modulus of elasticity
γ	Volume expansion rate of the corrosion product
K_{air}	Amount of Airborne Chloride
K_T	Critical Chloride Concentration

Dc	Chloride Diffusivity Coefficient
Y_8	lognormal random variable
Y_9	lognormal random variable associated to the corrosion rate V1
T_{co}	number of years until steel corrosion happens
V_1	corrosion rate of the steel bar prior to corrosion cracking
W(t)	steel weight loss
C_0	Surface chloride concentration
F_{ca}	Flexural Bending Capacity
Y_5	model uncertainty related to the estimation of K
Y_6	Construction error of m
Y_7	model uncertainty related to the estimation of Dc
Q_{cr}	Critical threshold amount
Q_b	Steel corrosion product
t_1	corrosion beginning period
t_2	corrosion crack formation time

Chapter 1. INTRODUCTION

1.1 GENERAL

The introduction chapter highlights the effect of airborne chloride and traffic hazards on the behavior of RC bridge. The airborne chloride hazard can significantly deteriorate the capacity of existing reinforced concrete structure constructed near the marine area. To keep these deteriorated RC bridges in service, regular repair actions are necessary which leads the higher lifetime cost of the structure. Conventional repair techniques have several limitations including they cannot eliminate the possibility of corrosion damage of the repaired bridges. An application of novel stainless-steel reinforcement in damaged RC bridge can resolve the limitations of the ordinary repair techniques. In this chapter, the advantages of the application of stainless steel reinforcement in reinforced concrete bridges are introduced and necessity of this project is explained.

1.2 BACKGROUND AND MOTIVATION OF RESEARCH

Highway bridges have a major role in public life through providing transportation passage across rivers, mountain, valleys, or road crossings. The growth as well as disaster resilience of a society and the economy is mostly dependent on these bridges. Therefore, ensuring the safety of highway bridges is crucial to maintain efficient functioning of the transportation system of a nation. Nevertheless, the coastal marine environment poses a threat to the performance of RC bridges (Hasan, Akiyama, Kojima, & Izumi, 2022; Lim, Hasan, Akiyama, & Frangopol, 2020). The major mechanism in this backdrop is likely due to the chloride ions from the sea travels towards inland structure through wind. After a certain period of time, these chloride ions penetrate through the concrete surface and reaches to the reinforcing bars. When the chloride contents in the reinforcement becomes equal to the critical chloride content, the rusting initiation starts leading to the reduction in the cross-sectional area of the reinforcing bars. As a result, the capacity of the girders substantially decreases, and the bridges becomes unsafe for its service. Moreover, the traffic load is continuously increasing with time. These simultaneous issues are essentially important to consider for existing highway bridges particularly those located near the coastal marine environment.

Numerous existing highway bridges are found to be deteriorated near the coastal marine environment owing to airborne chloride hazard. For example, many existing RC bridges in the coastal area of Japan are found to be deteriorated due to chloride issue (Akiyama, Frangopol, & Yoshida, 2010; Akiyama, Frangopol, & Matsuzaki, 2011; Akiyama, Frangopol, & Suzuki, 2012). In order to keep these deteriorated bridges in service regular inspection, repair and maintenance activities must be performed to conserve their capacity. However, additional money required for inspection, repair and maintenance to a remarkable rise in life cycle cost. Furthermore, the traffics are required to put on halt causing subsequent effects on human life during inspection, repair and maintenance. Moreover, this could also cause major loss in production work hours due to traffic jam. According to Hashem (2011), the deterioration of concrete structures results in the loss or reduction of 0.3 percent of the global GDP. It should be noted that corrosion issues may also result in an approximate 10% rise in indirect expenses. Therefore, it is highly important to find a sustainable solution to overcome aforementioned problems.

Several researchers proposed or developed various methods or materials to mitigate this issue. A number of methods have been used in the past to increase the corrosion resistance of RC bridges. Some of the widely used methods are: (1) If high density concrete is utilized, low permeability can be achieved. (2) By adding more concrete cover, corrosive chemicals take longer to access the steel reinforcement and corrosion will take longer to start. (3) The reinforcement's resistance to corrosion is increased when an epoxy coating is applied, adding another layer of defence. (4) Fiber-reinforced polymer (FRP) reinforcement offers long-term corrosion resistance and increased structural lifetime by ensuring non-corrosive rebar. These methods and materials have a number of drawbacks that reduce their long-term usefulness in the construction of RC bridges that are resistant to corrosion (Hasan et al. 2022; Hasan, Akiyama, Kashiwagi, Kojima, & Peng, 2020; Hasan, 2020; Hasan, Yan, Lim, Akiyama, & Frangopol, 2019). Therefore, more research is needed to develop corrosion resistant sustainable constructions.

In this research, stainless steel rebars is proposed as promising alternative to mitigate corrosion issue in concrete bridges. This issue of research is undertaken in the context ongoing and future planned developments of transportation network with bridges adjacent to the coastal areas of Bangladesh. The Matarbari deep seaport is going to be the one of the mega projects in South Asia. The chromium oxide layer or coating is one of the elements that affect stainless steel's resistance to corrosion. Furthermore, chromium is a substance that exists independently of the steel's surface (Hasan, 2020). Numerous benefits can be achieved by using SS rebars in concrete bridges. For example, use of SS reinforcement instead of CS reinforcement will reduce necessity of inspection and repair frequency and magnitude and leading to lower corrosion induced inspection and repair cost (Hasan et al., 2019). Furthermore, Hasan et al. 2020 & 2022 examined the structural behaviour of SS reinforced concrete beam and reported that structural behaviour of SS- & CS-RC member is approximately similar and hence design guideline for regular RC construction could be used for SS-RC structure. Notably, price of SS rebar is significantly high compared to CS rebar and leading to high construction cost (Hasan, Akiyama, & Kashiwagi, 2020; Hasan, Yan, Qi, & Akiyama, 2018; Hasan, Lim, Akiyama, & Frangopol, 2020; Hasan, Yan, & Kashiwagi, 2019). However, this high initial cost of SS rebar will be balanced by the lower inspection, repair and demolition cost depending after a certain period of time beyond construction. This certain period of time which is named as pay-off period for stainless steel economic viability is highly important evaluated.

In current literature, there is no research to determine the pay-off time period for stainless steel economic viability applied in RC bridge under multiple hazards. In this research, a procedure to compute pay-off time period for stainless steel economic viability is presented. In an illustrative RC bridge example, the hazard curves, failure probabilities, life-cycle cost and pay-off time period for SS rebar is computed considering the variable bridges from the shore in 20 Japanese Cities.

1.3 OBJECTVES OF RESEARCH

The key objective of this research is to identify how much time is required for stainless steel rebars to be economical compared with carbon steel rebar applied in RC bridges depending on the location from the coastline, geographical position of the city, and prices of SS rebar. To achieve the main goal, following aims ae targeted:

(a) To develop a novel framework for computing pay-off time for stainless steel (SS) reinforcement to be economical compared to carbon steel (CS) reinforcement applied in reinforced concrete (RC) bridge that are subjected to traffic load in a maritime coastal setting.
(b) To demonstrate the computation procedure of pay-off time for stainless steel economic viability in an illustrative bridge example under multiple hazards.
(c) To examine the effect of coastal distance, geographical location, and price of stainless steel reinforcement on the pay-off time of stainless steel economic viability.

Chapter 2. LITERATURE REVIEW

2.1 GENERAL

This chapter focuses on a comprehensive review on the corrosion mechanism in reinforced concrete structure subjected to risk of airborne chlorine. Firstly, basic concepts for deterioration of RC bridges are discussed herein. After that, features responsible for triggering the damage of rebar in reinforced concrete are provided in depth. This discussion will help to understand the most severe causes responsible for corrosion of rebars in harsh environment. Then, longevity characteristics of RC bridge constructed in coastal area is presented since durability performance of the bridges is very important to define inspection and repair time. Furthermore, life-cycle performance of the RC bridge susceptible to vehicle load hazard constructed in the coastal marine area is briefly introduced here and the current repair methods are explained along with their limitations. Finally, experimental and analytical studies of stainless steel rebars are provided and their economic viability is presented.

2.2 BASIC CONCEPTS FOR DETERIORATION OF RC BRIDGES

The development of a thin defensive oxide layer on the external part of the reinforcement, known as a passive film, can be helped by the incidence of high absorptions of alkaline solution in the tiny pores of concrete (Bertolini, 2008). In concrete constructions, partial loss or damage to the passive coating of the rebar may result from carbonation and chloride penetration, leaving it open to corrosion. The most significant factor contributing to the degradation of reinforced concrete structures is thought to be corrosion brought on by chlorides in coastal marine environment. There are two main ways that chloride can show up in concrete. The first step involves combining and adding chlorides to the concrete using a mixture of seawater and admixtures rich in chlorides. Second, the air's chloride environment and the infiltration of the dicing salts cause chlorides to infiltrate the concrete. For the structural material of reinforced concrete (RC) bridges, airborne chloride is considered as one of the major problems. This presents a significant issue for engineers in ensuring the bridge's long-term durability and safety during its life cycle. Chloride ion dissolved in the sea water travels with sea wind and reach to the surface of the RC structure. After reaching the chloride content to the critical surface chloride content, the ingress inside of the concrete and finally reach to the surface of reinforcement steel. After a certain period, time, corrosion initiation in rebar inside the concrete begins and ultimate cross-sectional degradation of rebar occurred. The bridge's structure is frequently attacked by chloride of sea. Bridge’s structural degradation is mostly caused by damage to steel. Concrete cracking is a significant contributor to the corrosion of RC bridges. These fractures may result from temperature variations, stress from heavy traffic, or shrinkage that occurs as the concrete dries. Water and other dangerous substances can enter via these fissures, hastening the degradation process. These cracks allow for the entry of water and other hazardous materials, which speeds up the deterioration process. Concrete deteriorates over time due to cycles of freezing and thawing and carbonation. Concrete degrades with time as a result of carbonation and freeze-thaw cycles.

2.3 CORROSION MECHANISM OF REINFORCEMENT IN CONCRETE

A material's chemical response to its surroundings is known as corrosion. Steel corrodes due to a chemical reaction. In the presence of oxygen and water, iron undergoes a transformation process that results in oxides and hydroxides.

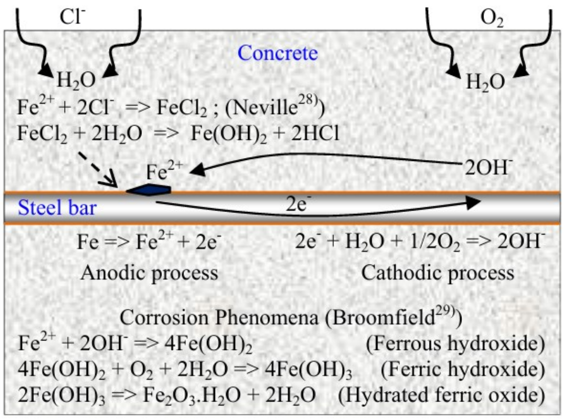


Fig. 2.1 Corrosion mechanism of reinforcement in concrete (Piyamachant, Ros, & Shima, 2013).

Fig. 2.1 demonstrates the basic mechanism of corrosion in reinforcement. The chemical reactions (Eqs. 2.1 to 2.5) occurred during the corrosion mechanism in RC structure owing to airborne chloride hazards are given below reported by Broomfield (2003):

$$Fe \rightarrow Fe^{2+} + 2e^- \quad (2.1)$$

$$Fe^{2+} + 2Cl^- \rightarrow FeCl_2 \quad (2.2)$$

$$FeCl_2 + 2H_2O \rightarrow Fe\,(OH)_2 + 2HCl \quad (2.3)$$

$$4Fe\,(OH)_2 + O_2 + 2H_2O \rightarrow 4Fe\,(OH)_3 \quad (2.4)$$

$$4Fe\,(OH)_3 \rightarrow Fe_2O_3 \cdot H_2O + 2H_2O \quad (2.5)$$

2.4 FACTORS AFFECTING CORROSION OF REBAR IN CONCRETE

Numerous fundamentals influence rebar corrosion in RC structure. These fundamentals fall into two universal classifications: internal and external reasons. Temperature, comparative humidity, moisture, oxygen, and exposure to ions that are aggressive, such as chlorides, are examples of external variables. The water-to-cement ratio (W/C), the depth of the concrete protection, the general building approaches, are the examples of internal variables. Additionally, environmental factors that accelerate corrosion include carbonation, and exposure to acidic pollutants. The factors influence the corrosion acceleration rate are summarized below:

i. Problems with concrete corrosion are prevalent in seaside places. Steel corrodes due to the penetration of chloride ions from marine air into the concrete (Berkely & Pathmanaban, 1990). Moisture and salt spray additional accelerate the concrete's acidic method. For this reason, buildings created in seaside locations need to take additional safety events.

ii. Steel is protected from corrosion by the alkalinity (pH) of the concrete. Steel corrosion is an option, nevertheless, in places wherever the concrete's pH decreases for example, as a result of carbonation or acid rain (ACI 440, 2008). Since the concrete's pH is dropped from 12 to around 9 during the carbonation process.

iii. Concrete can have variety of fundamentals to prevent corrosion. However, certain additives, such chloride raises possibility vulnerability to corrosion.

iv. Water, oxygen, and chloride ions can enter into steel due to gaps in the concrete caused by inadequate compaction during the casting process, which can lead to corrosion process.

v. Concrete's density and richness have an impact on corrosion. Concrete's obstruction to corrosion is reduced if the appropriate components are not employed or if the mix composition is not correctly stable. Steel may be shielded from the elements for a long time by high-quality concrete.

vi. Concrete constructions are more vulnerable to corrosion when suitable maintenance is ignored after building. The rate of corrosion raises quickly if quick maintenances and preventative processes are not affected. The lifetime of the construction is increased by proper maintenance.

vii. There is a great probability of corrosion if construction-related computable quality issues arise or if helpful actions are not taken.

In order to slow down the corrosion procedure in protected concrete structures, effective plan, material selection, and maintenance are essential.

2.5 RC BRIDGE’S DURABILITY PERFORMANCE IN A COASTAL ENVIRONMENT

Chlorides moves around the bridge as it moves toward the girder rods and rods of bridge girder attacked by chlorides ions. When cracks develop in the concrete surface due to corrosion, chloride, water and oxygen are allowed to penetrate more and the cracks become larger or bigger. It is still of great interest to determine how long-lasting reinforced concrete constructions will be, especially in coastal marine situations. Chloride content is indirectly liable for the initiation rebar deterioration, according to recent study findings based on observations from real bridge performance and particularly designed long-term studies for a wide

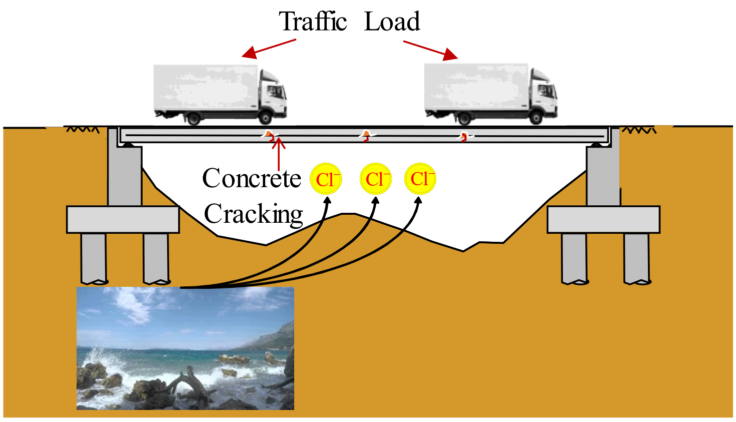


Fig. 2.2 Example of an reinforced concrete under vehicle in coastal area (Hasan, 2020).

variety of concretes. The loss of concrete alkalinity, or the concrete's ability to neutralize acids, which is primarily provided by its cement content, is a crucial component of high-quality concretes' long-term durability. The rate at which calcium hydroxide dissolves lowers alkalinity, usually very slowly. On the other hand, chlorides raise it. More interior surface area that is accessible for disintegration, as in low-quality, also contributes to its rise.

An example of a reinforced concrete (RC) bridge with combined effects from vehicle weight and airborne chloride is shown in Fig. 2.2. Bridge girders deteriorate gradually due to airborne salt (chloride) and vehicle weight. When a bridge is built, it is robust, but with time, the weight of the cars and the impact of the salt cause the bridge to deteriorate and eventually collapse. Because they impact bridge stability, the combined impacts of vehicle weight and chloride must be considered when assessing the long-term performance and longevity of bridges.

2.6 LIFE-CYCLE PERFORMANCE OF RC BRIDGE UNDER MULTIPLE HAZARDS

Reinforced concrete structures are often experienced by different hazards those are primarily categorized in two groups: (a) natural hazard that includes earthquakes, tsunami, landslides, snow storms, wildfires, flooding, tropical storms, and coastal inundation and (b) manmade hazard that includes mechanical, terrorist attacks, and industrial accidents (Li, Ahuja, & Padgett, 2012). Many factors hamper the durability of concrete bridges at the coastal areas, which in turn negatively affect their performance. Initially, corrosion induced cracks

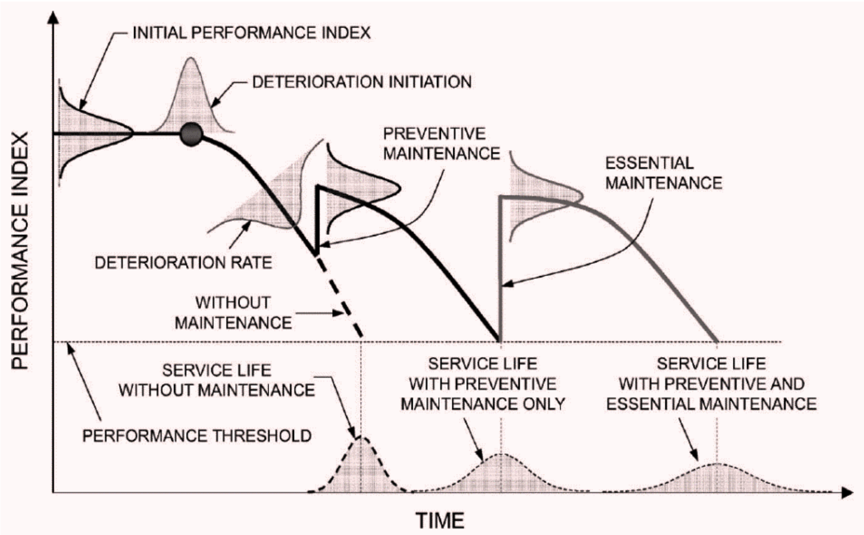


Fig. 2.3 Life-cycle performance curve of structure with maintenance interventions (Frangopol, 2011).

form in the concrete frame following swelling of steel bars by salty water. Moreover, temperature changes are responsible for the shrinkage or expansion of concrete hence it could generate minor splits leading to adverse effects over time. Similarly, the moisture content in air near these waters allows quicker corrosion of concrete, as well as encouraging algae growth that eventually weakens the physical strength and beauty present in the structure. Fig. 2.3 demonstrates the life-cycle performance curve of structure with maintenance intervention (Frangopol, 2011).

It is believed that the RC bridges under analysis were subjected to a variety of risks, such as those related to airborne chloride and traffic activity. However, due of particular factors, some bridge components are more prone to fail. The percentage of a day when winds blow from the sea towards land, the distance from the shore, the wind speed at a specific place, and the ratio of the sea wind are all taken into consideration when estimating the risk posed by airborne chlorides. The steel weight loss at time t after construction is approximated using nine random factors based on the airborne chloride dangers (Akiyama et al. 2010).

2.7 CURRENT CORROSION PREVENTIVE APPROACHES FOR RC STRUCTURE

For a nation's sustainable economic development, it is inevitable to have a sustainable transportation system. The performance of deteriorated reinforced concrete (RC) bridges can only be maintained by constant inspection and repair works. The conventional approach for repairing such bridges includes attaching steel plates outside them or placing reinforcement close to the surface area. But these traditional repair techniques have several shortcomings like recurrent corrosion. Because of these limitations in conventional repair procedures' (Swamy, Jones, & Charif, 1988; Zirba et al., 1994; and Hussain, Sharif, Baluch, Basenbul, & Al-Sulaimani, 1995), Sidy and Jabri (2011) recommended another novel method of fixing patches that tackle removing a damaged concrete layer, adding a new one, and sanding off rust scorned bars. If carbon steel is used to replace the rusted rebar in the overlying repair process, the beam that has been fixed will still corrode in future due to the continued access of chloride ions together with the air and water through the non-complete sealing of the concrete (Kobayashi & Rokugo, 2013). Even though they are simple and easy to use, these kinds of repairs are usually done all the time when used bridges get corroded after the reparation at first that means this method is very costly in long run depending upon number of times it is repaired in patches, so it costs more in future. These reveal that current corrosion preventive methods are not effective in avoiding corrosion damages in bridge component. Therefore, searching for a sustainable material rebar option is demanded to eliminate the necessity of frequent inspection and maintenance works.

2.8 STAINLESS STEEL REINFORCEMENT

2.8.1 Experimental Study

Selecting rebar for a construction project requires consideration of numerous factors including the particular needs of the project, the environment as well as financial constraints. In ordinary construction projects for RC bridge, ordinary carbon steel rebar is likely going to be the cheapest option although bridges in corrosive environments could use stainless steel or epoxy coated rebar so as to enhance durability. The advantages of corrosion resistance of stainless steel over that of carbon steel are mostly due to stainless steel being manufactured through a chromium containing protection approach for structures which involve exposure to aggressive conditions, more particularly chlorides. Stainless steel reinforcement bars and carbon steel rebar have quite diverse chemical profiles. There are three elements: Nickel, chromium and molybdenum, present during stainless steel production (Maaddawy & Soudki, 2007).

2.8.2 Economical Viability Study Using Life-Cycle-Cost

Even though stainless steel rebar provides superior corrosion resistance capacity than ordinary rebar, its usage in reinforcement concrete structures is still relatively low due to its more initial cost. Rebar constructed of stainless steel is particularly expensive to produce because of the grade and surface finishes of the steel that are chosen to match the conditions in which the material will be used. There are some other costs that push up the cost of making stainless steel rebar include labour, power, gas, etc. The introduction of stainless steel rebar in reinforced concrete structure may lessen corrosion-induced deterioration, resulting in cheaper renovation costs than ordinary rebar, even though the SS-RC structure initially costs more than the CS-RC structure.

Therefore, to determine whether it is more economically viable to use stainless steel rebar with better corrosion resistance than conventional rebar, performing life-cycle cost analysis is must. Furthermore, since there are numerous uncertainties associated with airborne chloride, probabilistic life-cycle cost is required to obtain reliable decision (Hasan et al. 2018).

Chapter 3. : FAILURE PROBABILITY ASSESSMENT OF RC BRIDGE

3.1 GENERAL

In this chapter, a novel flowchart for probabilistic failure assessment of reinforced concrete bridge subjected to the mechanical hazard constructed in coastal marine environment is introduced. Initially, environmental and mechanical hazards assessment procedure is explained. Based on the hazard assessment, time-variant deteriorated structural performance of the bridge infrastructure is quantified. Based on the structural performance assessment, the possibility of durability and final limit states failure for the considered bridges is reported and their computation procedure is described in detail.

3.2 PROCEDURE FOR FAILURE PROBABILITY ASSESSMENT

Fig. 3.1 shows the flowchart for calculating failure probability assessment for RC bridge under traffic load in coast area. The proposed flowchart consists of four steps as described below:

(i) Step 1: Environmental hazard assessment is performed to calculate its impact on the RC bridge girder's reinforcements' steel weight loss. In conducting hazard assessment, the procedure developed by Akiyama et al. (2010) is followed here and section 3.2.1 will provide a brief review of it.

(ii) Step 2: In this step, mechanical hazard (traffic load) is assessed in order to determine the structural demand of the considered bridge girder. Generally, load comprises two main types, namely live and dead load. The dead load is defined as the total weight of the bridge itself including its materials, road surface and any permanent equipment the bridge may have on it. This weight does not change over a long time making it constant/static. However, live load reflects the dynamic forces applied by people driving cars. When a load is put on a bridge, it disturbs the structure causing repeated stresses particularly when it's a live one which can lead to eventual wearing out over time thus weakening the girder. On the other hand, several factors such as density of traffic flow within an area as well as weight of vehicles determine additional stresses from a moving vehicle (i.e. car, truck or bus). Here, a numerical simulation utilizing vehicle statistics suggested by Matsuzaki, Akiyama, Ohki, Nakajima, and Suzuki (2010) is taken into consideration for traffic load analysis.

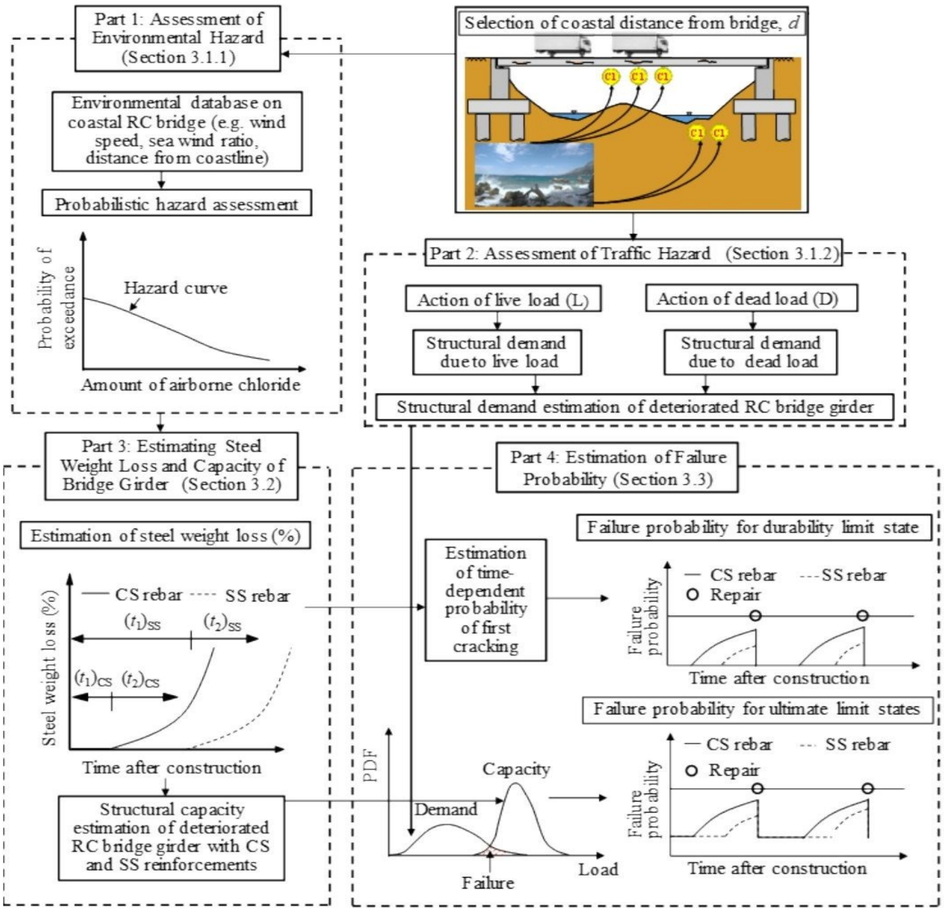


Fig. 3.1 Process flow diagram for determining failure probability.

(iii) Step 3: In this step, according to the evaluation of windborne chloride, estimation of steel weight loss due to corrosion of reinforcement is done and the details will be discussed in Section 3.3.1. The loss in strength over time that occurs when bridges are exposed to airborne chlorides leading to corrosion is investigated here. Chloride ions from the environment penetrate the concrete and get to the steel reinforcement causing corrosion. Due to the corrosion process, the cross-sectional area of the reinforcing bars decreases and, consequently, there is a continued loss of steel mass. The reduced steel area in the bridge girder lowers its structural capacity hence inability to withstand design loads in the end.

(iv) Step 4: In this step, time-variant failure possibility of CS and stainless steel (SS) reinforced bridge girders are estimated. Based on amount of steel weight loss in part 3, the failure probability for durability limit state is computed based on the time of first crack occurrence. On the other hand, time-variant failure probability for ultimate limit state of the bridge girder is estimated by comparing structural demand and capacity.

3.3 HAZARD ASSESSMENT

3.3.1 Environmental Hazard

The coastal environment has a remarkable impact on the performance of RC structure. For instance, air currents can carry marine chloride ions away, damaging RC structures. Several variables change with time and distance in relation to the concentration of chloride ions present. Akiyama et al. (2010) developed an airborne chloride hazard model which has been used in this research. This concept is used here to clarify the distinction between the likelihood of a failure and the expense of maintenance associated with a chloride ion assault. Considering numerous uncertainties, the correlation among the amount of windborne chlorine (K_{air}), location from the shore (d), and ratio of sea to land air (r) is noted as below (Akiyama et al. 2010):

$$K_{air}=Y_2 \cdot 1.29 \cdot r \cdot (Y_1 \cdot u)^{0.386} \cdot d^{-0.952} \tag{3.1}$$

Where Y_1 is a Gaussian distributed random variable, and u is the mean wind speed in meters per second and Y_2 refers to a lognormal based uncertainty related attenuation rule.

At a given position, the likelihood qs (k_{air}) of K_{air} greater than an allocated value k_{air} is given by Akiyama et al. (2010):

$$q_s(k_{air}) = \int_0^{\infty}\int_0^{\infty} P\left(K_{air} > K_{air} \middle| Y_1 = y_1, Y_2 = y_2\right) f_{y_1}(y_1)\, f_{y_2}(y_2)\, dy_1\, dy_2 \qquad (3.2)$$

Where $f_{Y1}(y_1)$ = likelihood density functions (PDFs) of Y_1 and $f_{Y2}(y_2)$ =likelihood density functions (PDFs) of Y_2.

3.3.2 Mechanical Hazard

The primary cause of bending failure in superstructures of bridges is often traffic load-related mechanical risks (Akguil and Frangopol, 2005a). The traffic load simulation is carried out assuming the location of cars in the middle of the girder acting on the RC bridge girder at the mid-span, taking into consideration the mixing proportion of traffic kinds, axle weight depending on vehicle load, axle locations, & distance in between two cars. Probabilistic model is implemented to create probability density function of vehicle load impacts considering the mixing rate of real vehicle types in live loads. Probability density functions (PDFs) of traffic load actions are used to measure stresses and deformations in the reinforced concrete bridge girders in order to more qualitatively assess the risk of bending damage. This entails considering material properties including steel reinforcement's and concrete's tensile and compressive strengths as well as their elastic modulus.

3.4 TIME-DEPENDENT STRUCTURAL PERFORMANCE ASSESSMENT

3.4.1 Estimating Statistical Metal Weight Loss of RC

The following performance function was given to determine the likelihood of steel corrosion occurring for RC bridge girders affected by airborne chloride attack (Akiyama et al. 2010):

$$g_1 = Y_4 K_T - K\,(m, D_c, C_0, t, W/C\,, K_{air}, Y_3, Y_5, Y_6, Y_7) \tag{3.3}$$

Where K_T = critical chloride concentration; K = chloride level in rebar; m = designated concrete cover; D_c = chloride diffusivity coefficient; C_o = surface chloride concentration; t = after construction time; W/C = water to cement ratio; Y_3 = model uncertainty using a lognormal random variable and an estimate of C_o. The model uncertainty related to the estimate of K is represented by the lognormal variable Y_5, the construction error of m is represented by the normal variable Y_6, and the model uncertainty related to the estimation of Dc is represented by the lognormal variable Y_7. A Gaussian variable called Y_4 is connected to the K_T assessment. There are several forms of stainless steel rebar based on chemical makeup. In the current research, the life-cycle cost (LCC) of the SS-RC girder and the CS-RC bridge girder are compared using SUS-410. This analysis takes into account the fact that the K_T values for the beginning of corrosion in CS and SS rebar's differ. Corrosion cracks happen while reinforcement corrosion outcome (Q_p) exceeds critical amount (Q_t). Akiyama et al. (2010, 2011) reported mathematical equation to calculate likelihood of cracking:

$$g_2 = Y_8 Q_t(m, E_c, f_c, \gamma Y_6) - Q_p(T_{co}, t, v_1, Y_9) \tag{3.4}$$

where E_c is the modulus of elasticity (MPa) of the concrete, fc' is its compressive strength in (MPa), and γ is the volume expansion rate of the corrosion product; The model uncertainty related with Q_{cr} calculation, Y_8 (the lognormal random variable), and Y_9 (the lognormal random variable associated to the corrosion rate

V_1) are related to the variables, T_{co} (the number of years until steel corrosion happens) and V_1 (the corrosion level of the steel bar prior to corrosion cracking).

After that, the rebar weight reduction $R_W(t)$ (%) is presented as (Hasan, 2020):

$$R_W(t)=\begin{cases} 0.00,\ t_1 \geq t \\ \frac{\pi\phi}{\left(\pi\phi^2\right)/4}(t\text{-}t_1)V_1Y_9,\ t_2 \geq t > t_1 \\ \frac{\pi\phi}{\left(\pi\phi^2\right)/4}\{(t_2\text{-}t_1)V_1Y_9+(t\text{-}t_2)V_2Y_9\},\ t > t_2 \end{cases} \tag{3.5}$$

Where, t_1 = the corrosion beginning period, t_2 = corrosion crack formation time, and V_2 is the corrosion level at which the corrosion crack forms in the rebar.

3.4.2 Computation of Time to First Crack Occurrence

Another factor related with longevity RC constructions is time until first crack occurs. The primary aim of this study is to predict when the tensile stresses in concrete will pass and exceed its tensile or breaking strength. The time frame depends on the materials used, exposure conditions (the environment), live loads including wind or earthquake and any corrosive chemicals such as CO_2 or chlorides that are present which accelerate the breakdown of embedded steel.

3.5 FAILURE POSSIBILITY ASSESSMENT

3.5.1 Failure Possibility Assessment for DLS

Concrete cracking and spalling due to reinforcing corrosion are considered as the main causes of the durability limit state (Q_{dls}) failure probability for RC bridges. In order to compute Q_{dls} of the bridges under study, possibility of first cracking occurrence on the concrete surface. As soon as the first crack is discovered during inspection, RC bridge repair operations will begin. When the initial fracture appears within a given time interval, the Q_{dls} of RC bridge girders may be calculated as follows (Hasan, 2020):

$$Q_{dls}(t) = P(t_2 \leq t) \tag{3.6}$$

3.5.2 Failure Possibility Assessment for Ultimate Limit State

Corrosion has a greater impact on the bending capacity of RC bridge girders than shear (Lin 1995). As a result, the final boundary state standard is defined as ultimate limit state which was supposed to occur when the corroded longitudinal rebar yields. Probability of failure for ULS (Q_{uls}) of an girder exposed to vehicle load during time t in a seaside oceanic environment is calculated as follows:

$$Q_{uls}(t)=\int_{0}^{100}\int_{0}^{\infty} P\left[F_{de} \geq F_{ca} | L_t=l_t, M_w(t) = m_w(t)\right] \cdot g(l_t) f\left(m_w(t)\right) dl_t dm_w \qquad (3.7)$$

$$F_{de}(L_t=l_t)=L_{max,t}(L_t=l_t)+D \qquad (3.8)$$

Where M_w (t) indicates rebar weight reduction at t; F_{ca} (M_W = m_w) indicates flexural ability of the decaying girder for M_W = m_w. F_{de} (L_t = l_t) means flexural demand for traffic load provided L_t = l_t. g(l_t) and f(m_w(t)) are the traffic load's PDFs and decrease of weight in steel, respectively. Bending moment resulting from the specified live load is L_{max}, t (L_t = l_t). T = t and D = bending moment as a result of dead weight. A decreased in diameter of rebar is taken into consideration when estimating flexural capacity (F_{ca}) of girders using cross-sectional analysis.

Chapter 4. PROBABILISTIC LIFE-CYCLE COST ASSESSMENT

4.1 GENERAL

This chapter presents a novel flowchart for determining pay-off time for stainless steel rebars through probabilistic life-cycle cost analysis. In determining the pay-off time for stainless steel economical viability, details of statistical life-cycle cost procedure is explained. In addition, the mathematical formulation along with the various costing scheme is provided in detailed. The variation in initial cost for CS- and SS-RC bridge is also considered.

4.2 PROCEDURE FOR PROBABILISTIC LIFE CYCLE COST ASSESSMENT

To compute the pay-off time for stainless steel (SS) rebar cost-effectiveness over carbon steel (CS) reinforcement, probabilistic life-cycle cost (LCC) must be performed. Fig. 4.1 illustrates the procedure for conducting probabilistic LCC of RC bridges under multiple hazards. Following steps must be performed:

(a) For RC bridges (CS- and SS-RC bridge) located at any location, failure probabilities assessment considering environmental and traffic hazards must be performed as discussed in chapter 3.

(b) For computing LCC of RC bridges construction, observation, renovation and damage cost are determined. Since application of SS rebar will increase the construction cost of SS-RC bridge by 20%, hence this effect is also incorporated by assuming the construction of SS-RC bridge 5 to 20% higher than the CS-RC bridge (Hasan et al. 2019). Inspection cost of bridges depends on the inspection period. Although, SS-RC bridge will require relatively less inspection number than CS-RC bridge, in this research inspection period is assumed same for both bridge cases. The repair of bridges will be performed after identifying the occurrence of first crack during inspection. Lastly, the failure cost is computed, which includes the cost of replacing the bridge, the cost of fatalities and injuries, the interruption of function, and the effects on the environment and society. Since, the failure probability of durability and ultimate limit states are different, these affect the repair and failure cost of CS-

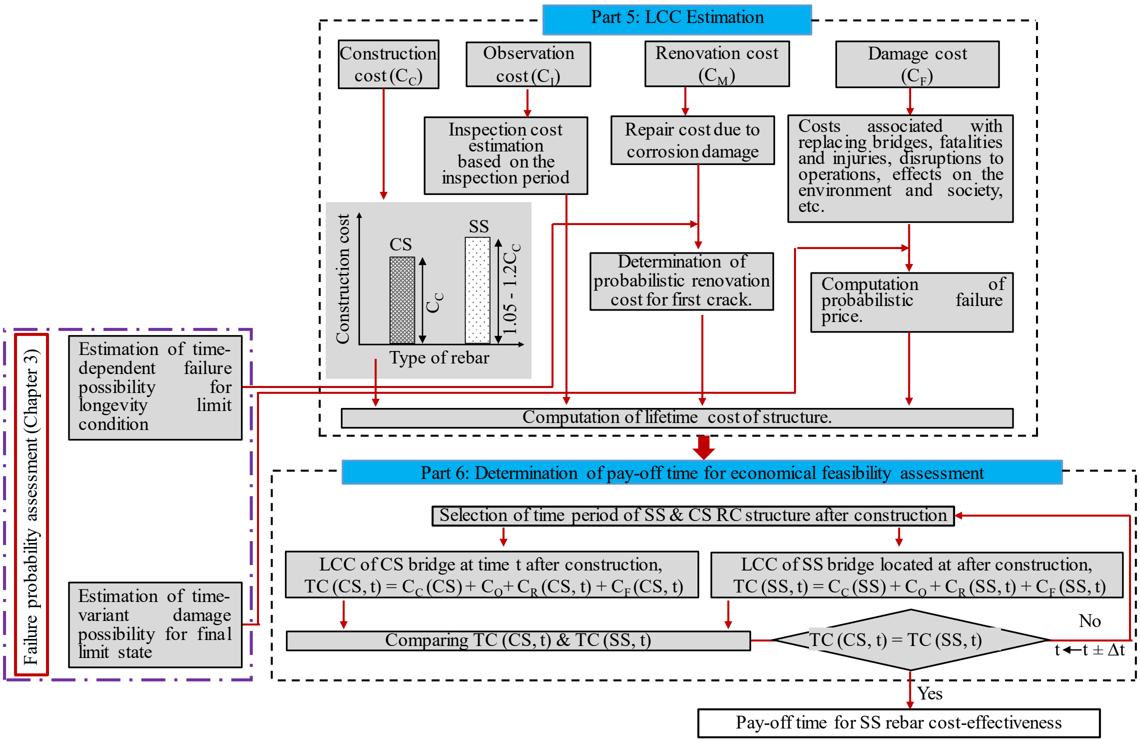


Fig. 4.1 Flowchart for probabilistic life-cycle cost assessment to determine pay-off time for stainless steel rebar cost-effectiveness.

and SS-RC bridge. By accumulating all these costs, total LCC od CS- and SS-RC bridge are obtained.

(c) In this part, LCC of CS-RC bridges is compared with SS-RC bridge to determine the pay-off time for SS rebar cost-effectiveness and following sub steps are executed:

i) At first, any random time (t) is assumed after the construction of both bridge cases.

ii) After that, LCC of SS-RC & CS-RC girders are calculated and differentiated.

iii) By comparing LCCs, if they are identical, the corresponding time (t) is called pay-off time for SS rebar cost-effectiveness. If not, another t will be selected and the procedure, sub step (i) to (iii) will be repeated until they are equal.

The LCC analysis requires the computation of the starting, observation, renovation, and final failure costs. The expenditure of an inspection may change depending on how long it takes. The cost of replacing the bridge, fatalities and injuries, and operational disruption will be covered by the failure cost. Repair cost due to corrosion damage will be under the maintenance cost. LCC is a technique for calculating total cost throughout the course of a project. Through this method many decisions can be made in project management. Through this method we have to calculate many expenses or costs. Which covers the costs associated with building, inspecting, maintaining, and failing. According to LCC estimates, a structure's extended service duration has a 2% money discount rate. (Alipour, Shafei, and Shinozuka 2013); and Weitzman (2001). According to FHWA (2002), a 100-year lifespan for the bridge is assumed.

Four primary cost components are taken into account when assessing life cycle costs (LCC) (Hasan et al. 2019). The first one is the cost of construction, which includes the price of purchasing and installing materials, with SS reinforcements costing 1.05–1.20 more than CS (Hasan et al. 2020). Periodic inspection service cost is the price paid for inspection services, the more inspections, the greater the cost. The cost of fixing malfunctions, particularly those brought on by corrosion, is covered by the maintenance budget. In conclusion, the total cost of failure includes additional expenses like replacing the damaged bridge, societal costs like traffic disruptions, and the costs to the environment.

4.3 BASIC EQUATION FOR ESTIMATING LCC

LCC is a calculation or analysis of total cost that includes construction cost, inspection cost, repair cost, and failure cost. In LCC analysis all effects must be converted in economic perspective. Therefore, the LCC equation can be expressed as follow:

$$\mathrm{LCC}(t) = C_C + C_I + C_M(t) + C_D(t) \quad (4.1)$$

Where C_C means construction cost, C_I means inspection cost, C_M means maintenance cost and C_D means failure cost. Construction and final failure costs can occur only once, but maintenance and inspection may occur many times over the service life (Hasan et al. 2019).

4.3.1 Construction Cost

Some construction costs are fixed for RC bridge construction. These include designing, material cost, fabrication cost, procurement cost, and transportation cost. The original RC bridge cost is typically used as the beginning cost baseline when CS is employed as reinforcement. The substitution of ordinary rebar with stainless steel, total primary cost will increase approximately 20 percent (McDonald, Sherman, Pfeifer, and Virman, 1995). SS Rebar prices may vary worldwide based on production costs.

4.3.2 Inspection Cost

Regular observation of bridges is necessary as defects and damage must be checked regularly to guarantee the longevity and safety of the structure (Rashidi & Gibson, 2011; and Liu & Frangpol, 2019). In order to investigate the inspection time period of the effectiveness of SS and CS, two types of inspection techniques are determined based on the life-cycle reliability of the bridges.

4.3.3 Maintenance Cost

Regular maintenance is vital to prevent bridge aging and to ensure structural safety. Maintenance costs include many types of costs, some of the notable costs are repair costs, retrofit costs, and replacement costs. Repair during the first crack in RC bridge girder for DLS is investigated in this study. The total cost of maintenance at different inspection intervals may be calculated using the formula below.

$$C_M(t) = \sum_{n=1}^{N}\sum_{t=1}^{T} P_{dls}(t) \times C_m(n,t) \times (1+s)^{-t} \tag{4.2}$$

$$C_m(n,t) = 2\ C_c + C_{QA} \tag{4.3}$$

Where N = overall repairs made, s = discount amount. C_m (n,t) = value of the n-th repair in year t, C_{QA} = a measure of the cost of quality assurance. In Eq. 4.2, the function of failure probability for DLS is expressed here as (P_{dls} (t)). Due to their different failure probabilities for DLS, reinforced girders constructed using CS and SS have different maintenance costs. In comparison to CS reinforced bridges, SS reinforced bridges may require more time to maintain. because to SS's significantly greater resistance to corrosion than CS. For the sake of comparison, the two bridge scenarios are considered to have the same repair duration. The cost of a structure failure for DLS is double the primary cost of quality assurance, as per Equation 4.3. (Val & Stewart, 2003). This cost is divided into two parts, as shown in Eq. 4.3: the first phase's cost of removal (CC) and the second phase's cost of replacement. This research assumes that repairing the bridge girder would not enhance its capacity or longevity (C_{QA} = 0).

4.3.4 Failure Cost

The estimated failure cost for the ultimate limit state (ULS) of the reinforced concrete (RC) bridge must account for all material and insubstantial losses that arise from the bridge collapse. Ang (2011) and Sunurwar, Itoh, and Nagata (1998) state that it is possible to determine damage cost for the girder's ULS at t.

$$M_D(t) = \sum_{n=1}^{N} \sum_{t=1}^{T} P_{uls}(t) * M_d(n, t) * (1+s)^{-t} \tag{4.4}$$

$$M_d(n, t) = M_{FR} + M_{FL} + M_{FH} + M_{FD} + M_{FEN} \tag{4.5}$$

The damage possibility for ultimate limit state at t is represented by $P_{uls}(t)$. The damage loss for ULS at n-th observation at t is denoted by M_d (n, t). There are several costs associated with managing bridges. M_{FD} stands for the cost of functional disruptions like traffic jams and lost productivity; M_{FEN} stands for environmental and social costs; M_{FR} stands for the cost of replacing the bridge; M_{FL} stands for the cost of lives lost and injuries sustained; and M_{FH} for the cost of cultural and historical effects. In below Table 3.1 lists the startup, observation, & final damage economic losses components in LCC assessment of the RC bridge.

Table 4.1. Overall lifetime cost components.

Cost components	Categorization of expenses	Amount	References
Primary cost (C_C)	Design cost Expenses for construction Cost of load testing	7% of Cc 90% of C_C 3% of C_C	Ang (2011)
Observation cost (C_I)	Cost of the inspection	0.5% of C_C	Alipour, Shafei, & Shinozuka (2010)
Final damage loss (C_D)	Cost of replacing a bridge losses in terms of human life and bodily harm Historical and cultural costs Cost of functional disruption Social and environmental costs	$10C_C$	Ang (2011); Stewart & Val (2003)

Chapter 5. ILLUSTRATIVE EXAMPLE

5.1 GENERAL

In this chapter, an illustrative reinforced concrete bridge example is considered which is assumed to subjected to under multiple hazards. For considered twenty cities in Japan at six assumed locations from the coastline, airborne chloride hazard is computed. At the considered location in the twenty cities, the failure probabilities for the girders reinforced by SS and ordinary reinforcement are presented. Based on the computed failure probability for DLS and ULS, the probabilistic LCC of SS- and CS-RC girders is illustrated. These results reveal the cost-effectiveness of stainless steel rebars in RC girders at different locations in different cities in Japan. Finally, the pay-off time in which relative economic feasibility of stainless steel than ordinary rebar is calculated for 0.1 km, 0.50 km, 1.0 km, 2.0 km, 3.0 km, and 4.0 km from the coastline in the considered twenty cities in Japan. Finally, in this chapter, it is investigated the effect of SS price on its cost-effectiveness in the coastline area in different cities in Japan.

5.2 DESCRIPTION OF RC BRIDGE GIRDER

To examine the durability and ultimate limit states failure probabilities of RC bridges made using CS and SS rebar, an example bridge is examined that was built in 20 Japanese cities at various points away from the shore. The RC bridge girder geometry is examined using probabilistic LCC to ascertain the pay-off time period of SS rebar cost-effectiveness, as shown in Fig. 5.1. This bridge girder was developed in accordance with the Japan Road Association's Highway Bridge (DSHB) design specification (JRA, 2000A). The bridge under consideration featured six 15-meter-long girders. 2.18% is the mid-span tensile steel ratio. When the outside girder, which makes up a significant portion of the bridge, fails, it is presumed that the bridge has failed as well because the two are intertwined. The girder's dimensions are 1500 mm in depth, 1500 mm for the flange, and 500 mm for the web.

An external girder's failure, which accounts for a significant portion of structure, is regarded the complete bridge's failure since it is subject to a greater bending requirement than internal girders (Yanweerasak, Pansuk, Akiyama, & Frangopol, 2018). Courbon’s method was applied to assess the impact of transverse bending moment distribution on the target girder, using vehicle load distribution data obtained from traffic simulations (Yanweerasak et al., 2018). It’s important to note that repairs are typically necessary for corroded RC structures, which were originally designed under outdated codes. These older design codes did not account for the durability performance of the structures.

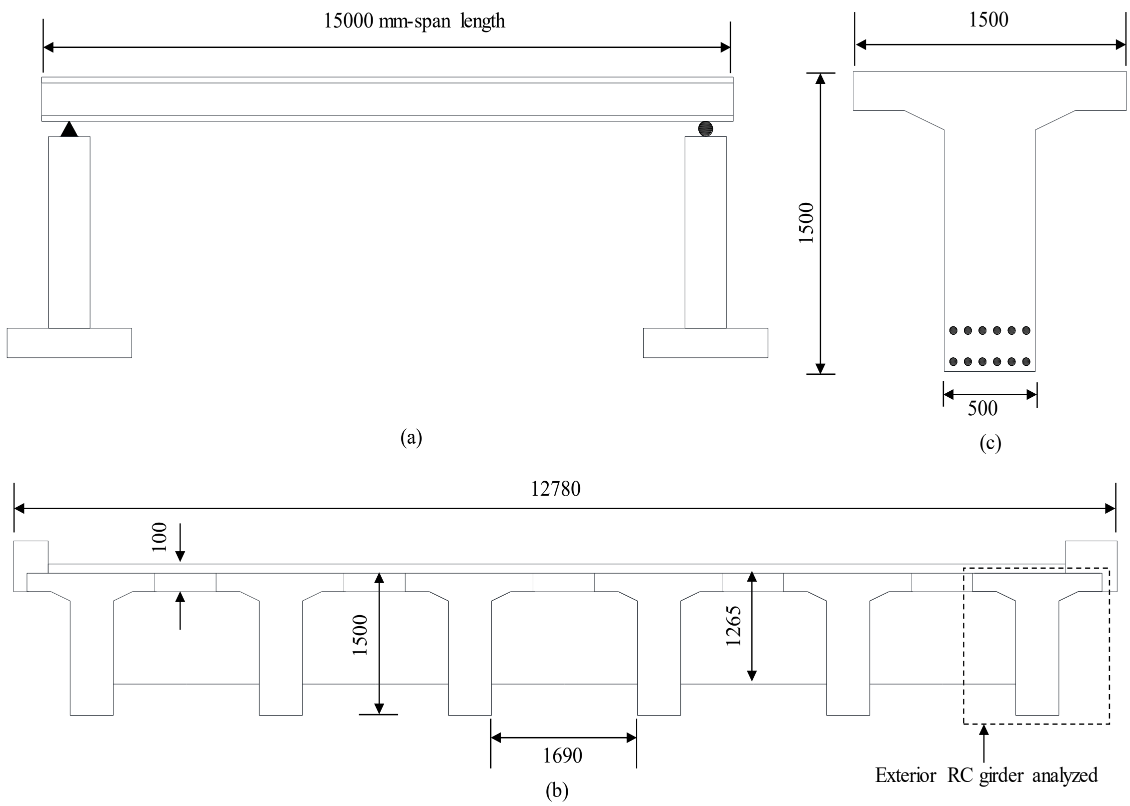


Fig. 5.1 A details of RC bridge girder: (a) Elevation of RC bridge (b) girder system geometry, and (c) girder cross-section.

5.3 MULTIPLE HAZARDS

5.3.1 Airborne Chloride Risk

The windborne chloride hazard assessment was performed at six locations (d = 0.1 km, 0.5 km, 1.0 km, 2.0 km, 3.0 km, and 4.0 km) in twenty considered cities in Japan. Figs. 5.2.1 to 5.2.20 represent the airborne hazard curves at d = 0.1 km, 0.5 km, 1.0 km, 2.0 km, 3.0 km, and 4.0 km from the coastline in twenty cities (see Table 5.1). Notably, for demonstrating effects of coastal distance (location of structure from coastline) and cities on airborne chloride hazard, six locations and twenty cities are considered. The cities are chosen based on their geographical position near the coastline.

Table 5.1. Twenty Japanese Cities.

SI.	Region	SI.	Region	SI.	Region
1	Wakkanai	**8**	Maizuru	**15**	Shionomisaki
2	Tomakomai	**9**	Sakai	**16**	Yonagunijima
3	Nemuro	**10**	Hagi	**17**	Miyakojima City
4	Fukaura	**11**	Nagasaki	**18**	Iriomotejima
5	Onahama	**12**	Amakusa	**19**	Minamidaitojima
6	Niigata	**13**	Aburatsu	**20**	Naha
7	Wajima	**14**	Muroto		

As shown in Figs. 5.2.1 to 5.2.20, the airborne chloride hazard is extremely high when d is small. For example, the probability of exceedance of 2 mdd at d = 100 m in zone 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, and 20 is 84.80%, 77.36%, 83.57%, 43.67%, 64.32%, 78.80%, 70.80%, 82.30%, 62.85%, 52.01%, 67.33%, 72.38%, 46.56%, 51.85%, 42.10%, 71.00%, 67.79%, 65.10%, 64.47%, and 59.25%, respectively. With the increasing distance from coastline, the probability of exceedance of particular airborne chloride content is remarkably reduced. For all cities, effects of coastal distance are found to be identical. It is also clear from the figures that airborne chloride hazard is different among the considered cities. To explain more precisely, two cities have been considered: (i) Niigata City (city no. 6) example of severe hazard prone city, and (ii) Maizuru City (city no. 8), example mild hazard prone city.

Fig. 5.2.6 depicts the airborne chloride hazard curve at several locations in Niigata City. It is observed that the probability of exceedance of particular airborne chloride amount is remarkably high. The probability of exceedance of 1 mdd at d = 0.1 km, 0.5 km, 1.0 km, 2.0 km, 3.0 km, and 4.0 km are 94.44%, 50.50%, 25.11%, 8.86%, 4.03%, and 2.10%, respectively. This result reveals that for increasing coastal distance from 0.1 km to 0.50 km, the reduction in probability of exceedance of 1.0 mdd airborne chloride decreased by 46.52%. On the other hand, in Maizuru City (see Fig. 5.20.8), probability of exceedance of 1.0 mdd of airborne chloride at d = 0.1 km, 0.5 km, 1.0 km, 2.0 km, 3.0 km, and 4.0 km are

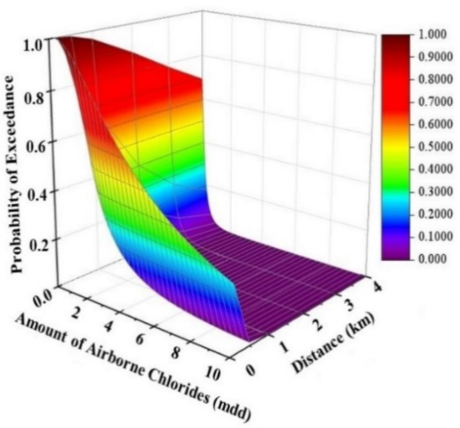


Fig. 5.2.1 Airborne chloride hazard cruves in Wakkanai City.

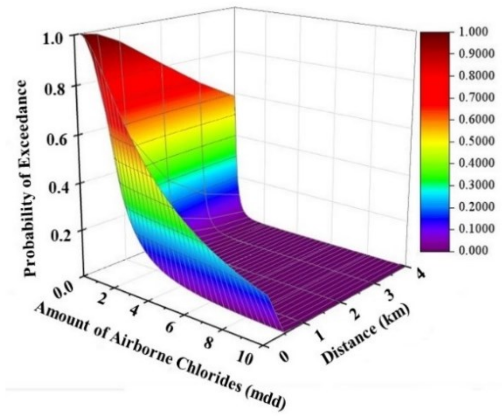


Fig. 5.2.2 Airborne chloride hazard cruves in Tomakomai City.

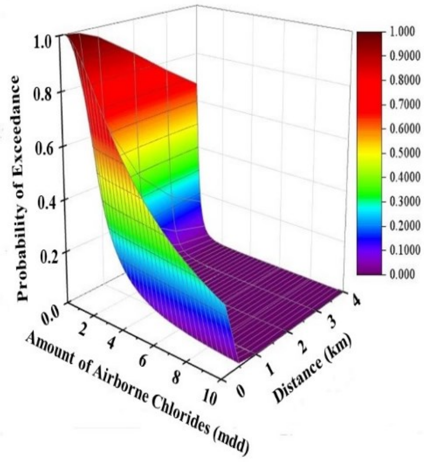


Fig. 5.2.3 Airborne chloride hazard cruves in Nemuro City.

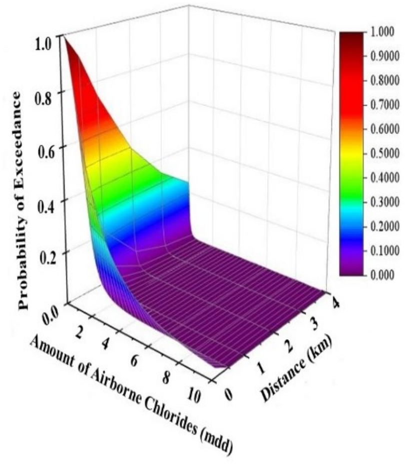


Fig. **5.2.4** Airborne chloride hazard cruves in Fukaura City.

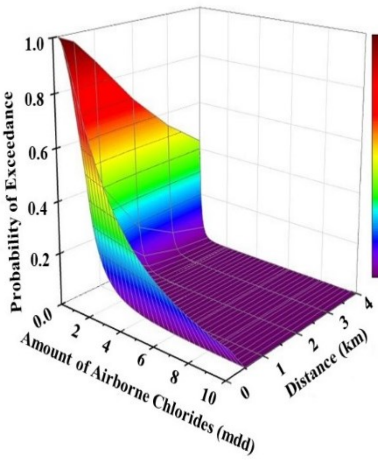


Fig. 5.2.5 Airborne chloride hazard cruves in Onahama City.

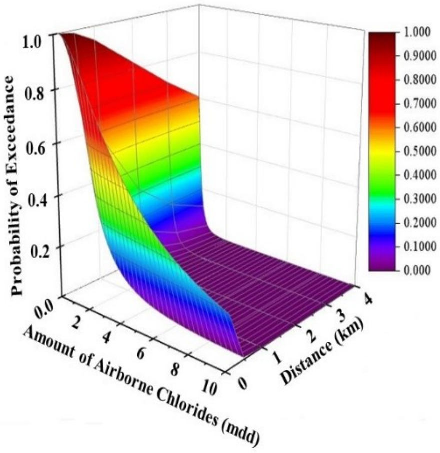


Fig. 5.2.6 Airborne chloride hazard cruves in Niigata City.

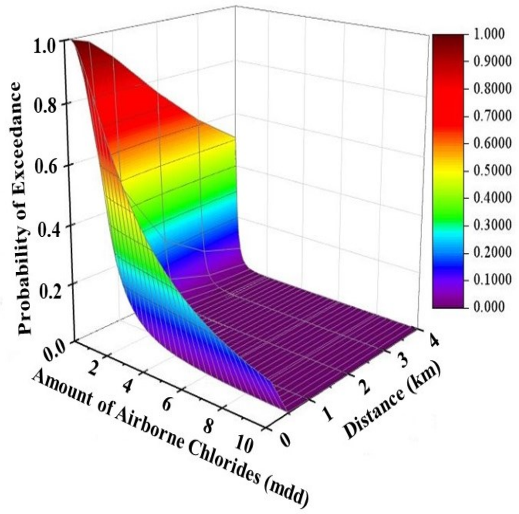


Fig. 5.2.7 Airborne chloride hazard cruves in Wajima City.

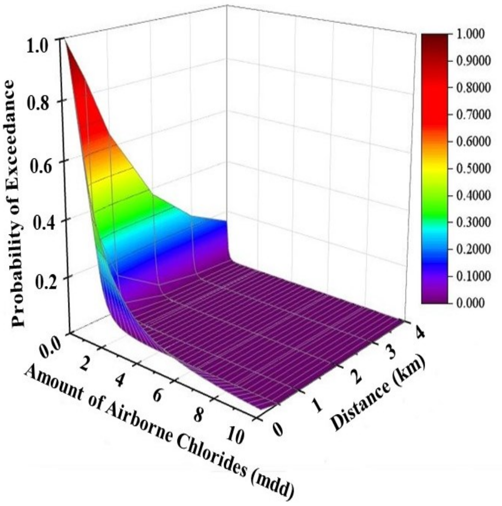


Fig. 5.2.8 Airborne chloride hazard cruves in Maizuru City.

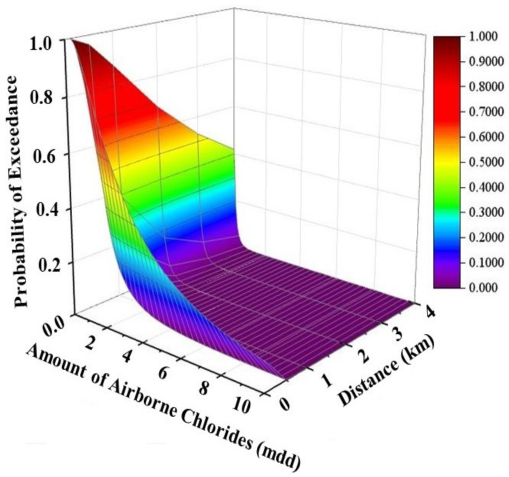


Fig. 5.2.9 Airborne chloride hazard cruves in Sakai City.

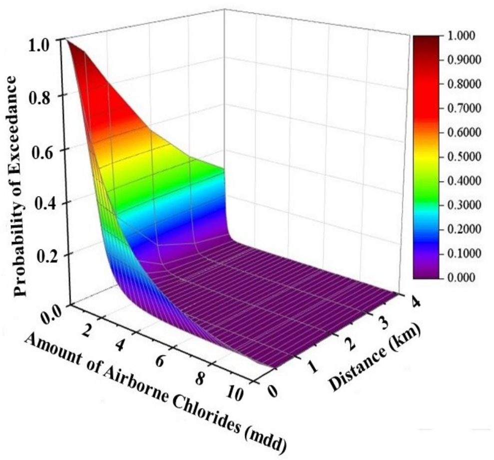


Fig. 5.2.10 Airborne chloride hazard cruves in Hagi City.

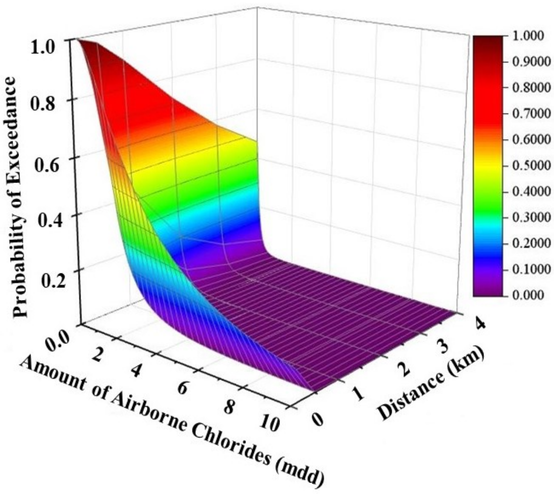


Fig. 5.2.11 Airborne chloride hazard cruves in Nagasaki City.

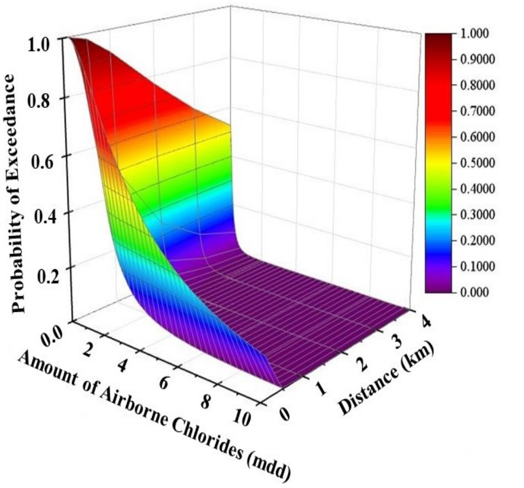


Fig. 5.2.12 Airborne chloride hazard cruves in Amakusa City.

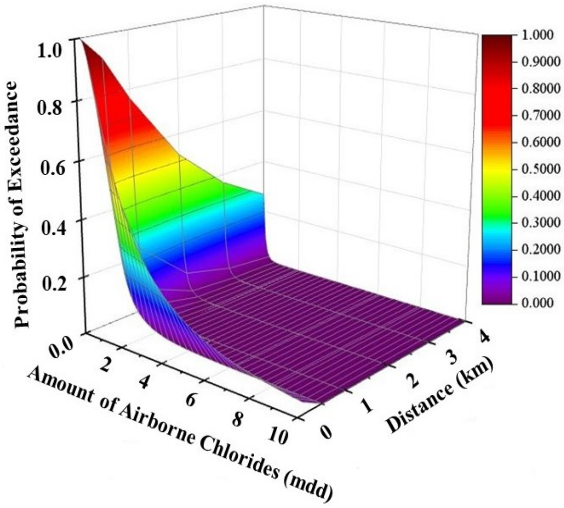


Fig. 5.2.13 Airborne chloride hazard cruves in Aburatsu City.

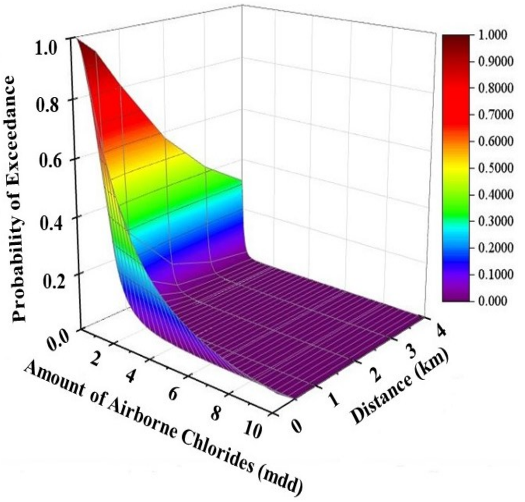


Fig. 5.2.14 Airborne chloride hazard cruves in Muroto City.

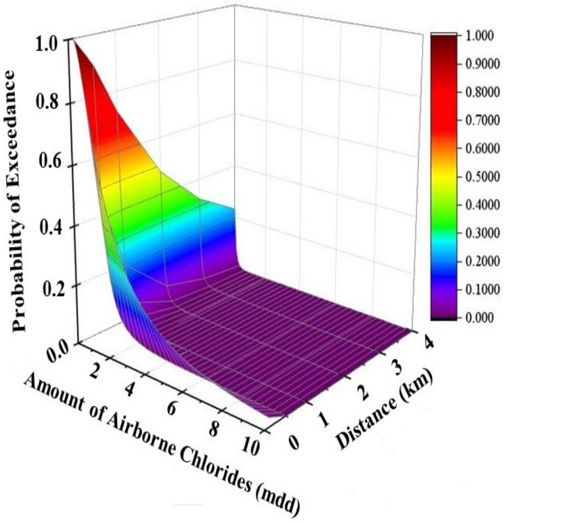


Fig. 5.2.15 Airborne chloride hazard cruves in Shionomisaki City.

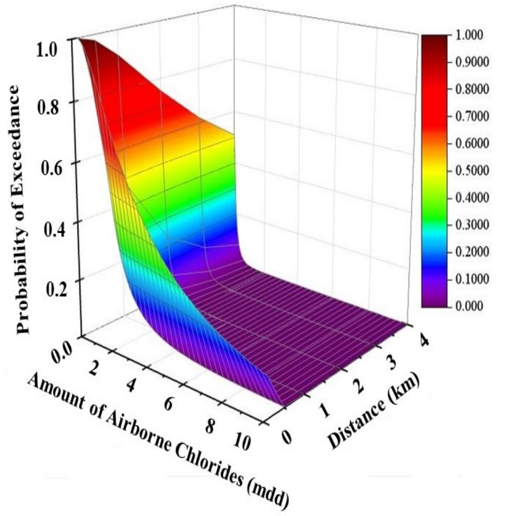


Fig. 5.2.16 Airborne chloride hazard cruves in Yonagunijima City.

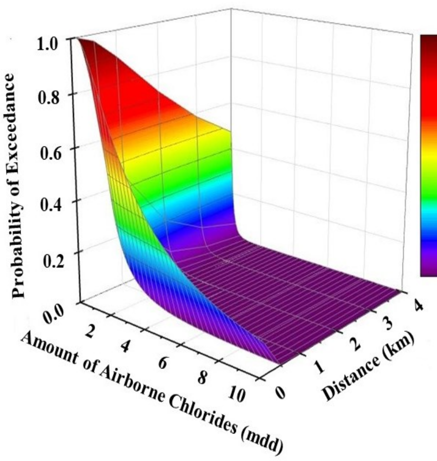


Fig. 5.2.17 Airborne chloride hazard cruves in Miyakojima City.

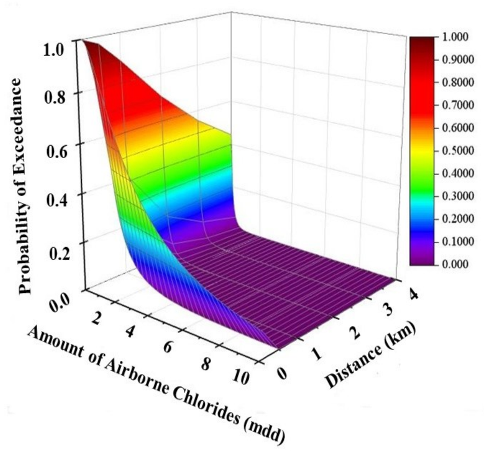


Fig. 5.2.18 Airborne chloride hazard cruves in Iriomotejima City.

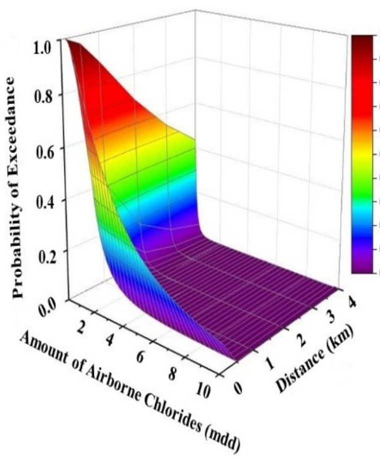


Fig. 5.2.19 Airborne chloride hazard cruves in Minamidaitojima City.

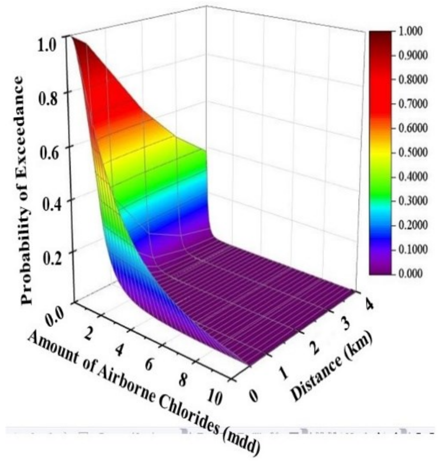


Fig. 5.2.20 Airborne chloride hazard cruves in Naha City.

70.97%, 15.23%, 4.4%, 0.8%, 0.2%, and 0.1%, respectively. Although the distance from the coastline in both cities are same, however, there is significant difference in airborne chloride hazard. This could be due to variation in wind speed between the Niigata and Maizuru Cities.

5.3.2 Risk of Vehicle Load

For bridge superstructures, vehicle weight is thought to be severe prevalent kind of risk. It can be said that each road has a load which is about the average number of vehicles on that road. Thus, no matter which city road one considers, its traffic load distribution pattern is more or less the same. This uniformity in load caused by vehicles follows that a traffic load simulation graph produced for the city streets will most likely have this same outlook. For concrete bridge superstructures subjected to traffic loads, flexural failure is commonly identified as the principal dominant failure (Akgül and Frangopol 2005a, 2005b). Over life-cycle reliability assessment to be accurate, live load impacts must be accurately predicted over the duration of a bridge. The figure functions as an important aspect of structural reliability analysis because it shows the building up of bending moment diagrams in order to maintain the functional protection of the system & it shows the Cumulative Distribution Function or CDF corresponding to the Maximum Annual Bending Moment for a structural element. This graphical representation shows a dramatic increase inside a given range,

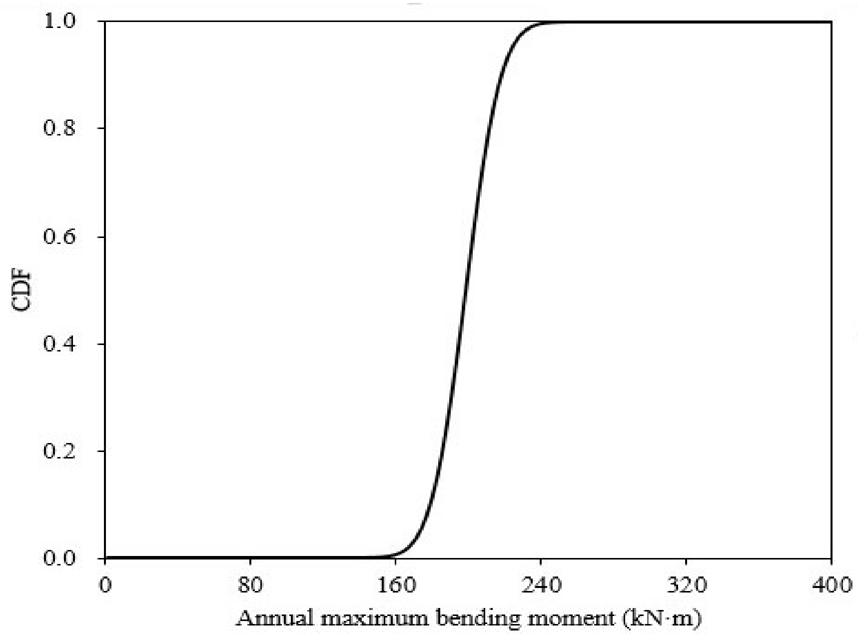


Fig. 5.3 CDF of an annual maximum bending moment derived from a simulation of traffic loading.

suggesting that the majority of bending moments are contained within this range with very low probabilities of lower and higher values. Such a pattern is a representation of the regular occurrences of bending forces on the structure in a particular year. Fig. 5.3 presents the CDF of yearly largest moment of flexural for considered reinforced concrete girder because of vehicle loading. Tamakoshi, Nakasu, and Ishio (2006) generated a bridge weigh-in-motion system for data on traffic, which was used in this project. It should be noted that identical vehicle pattern was assumed for all considered cities.

5.4 FAILURE PROBABILITY FOR RC BRIDGE UNDER MULTIPLE HAZARDS

5.4.1 Failure Probability of RC Bridge for Durability Limit State

The dynamic damage probability of SS- & CS-RC girder constructed at several locations in considered twenty cities are presented in Figs. 5.4.1 to 5.4.40. It can be observed from these figures that failure probability for DLS in all locations is approximately zero initially and rises remarkably with t after the

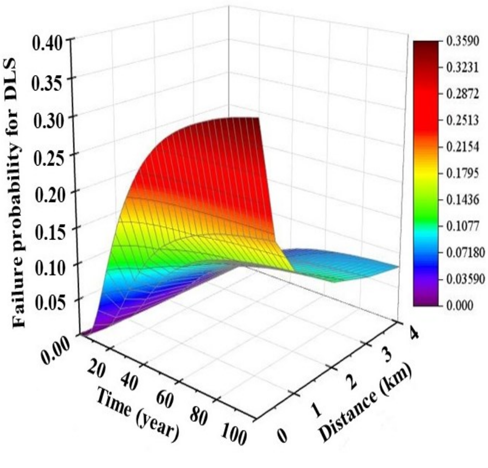


Fig. 5.4.1 Time-variant P_{dls} of CS-RC girder in Wakkanai City.

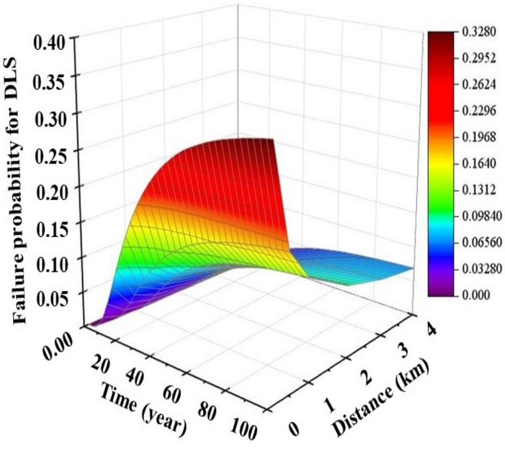


Fig. 5.4.2 Time-variant P_{dls} of CS-RC girder in Tomakomai City.

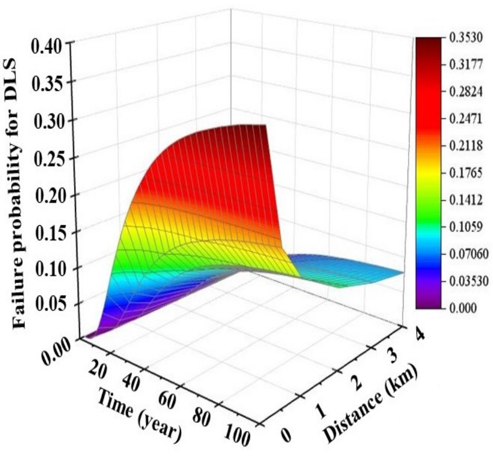


Fig. 5.4.3 Time-variant P_{dls} of CS-RC girder in Nemuro City.

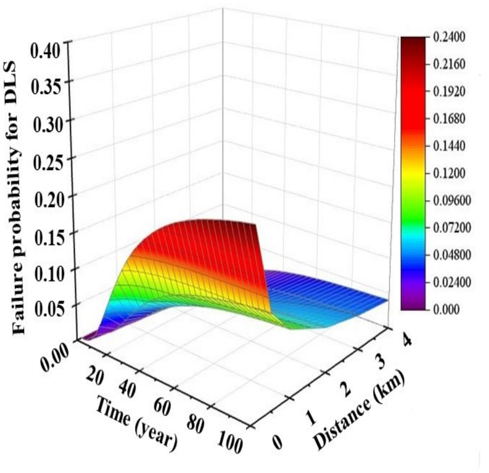


Fig. 5.4.4 Time-variant P_{dls} of CS-RC girder in Fukaura City.

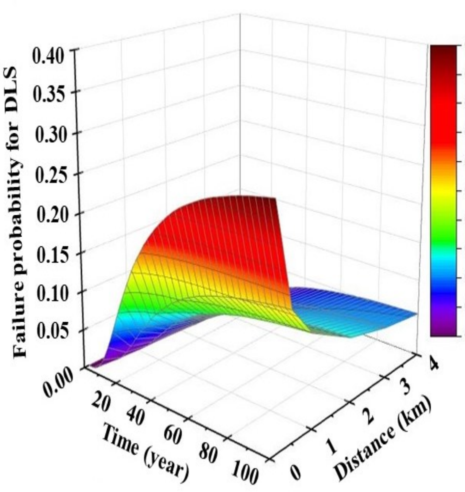


Fig. 5.4.5 Time-variant P_{dls} of CS-RC girder in Onahama City.

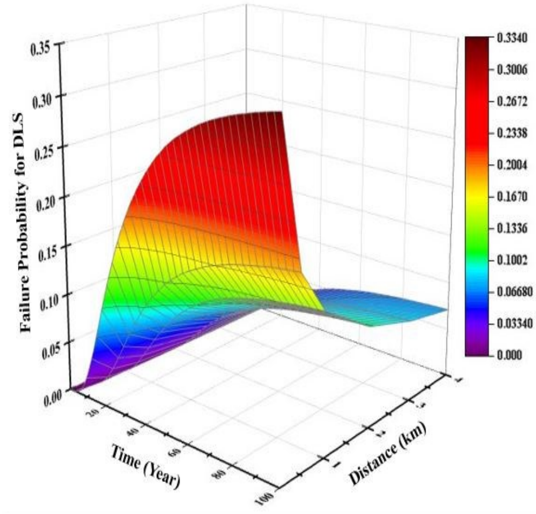


Fig. 5.4.6 Time-variant P_{dls} of CS-RC girder in Niigata City.

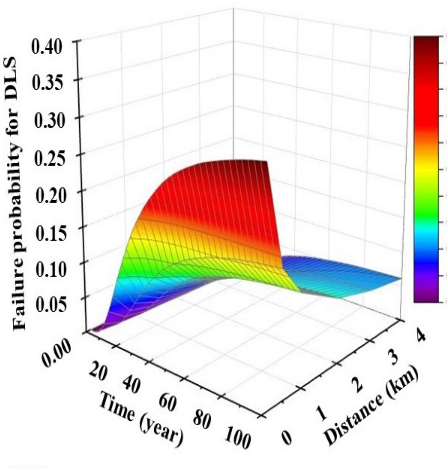


Fig. 5.4.7 Time-variant failure P_{dls} of CS-RC girder in Wajima City.

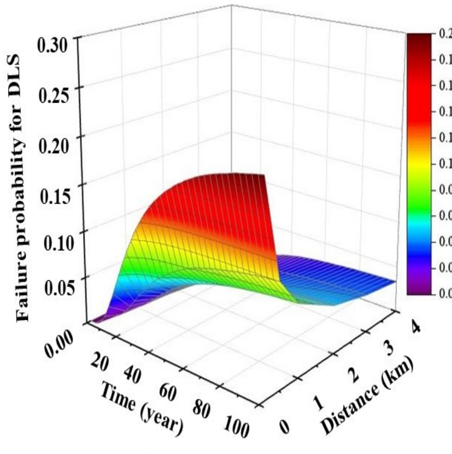


Fig. 5.4.8 Time-variant P_{dls} of CS-RC girder in Maizuru City.

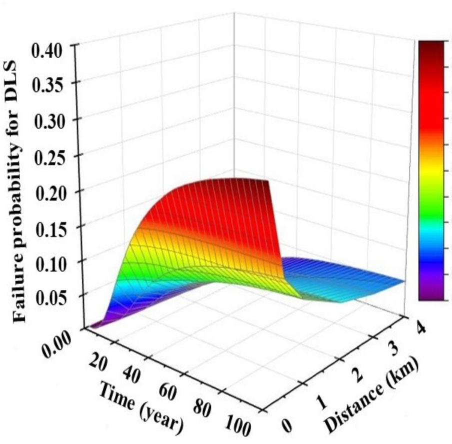


Fig. 5.4.9 Time-variant P_{dls} of CS-RC girder in Sakai City.

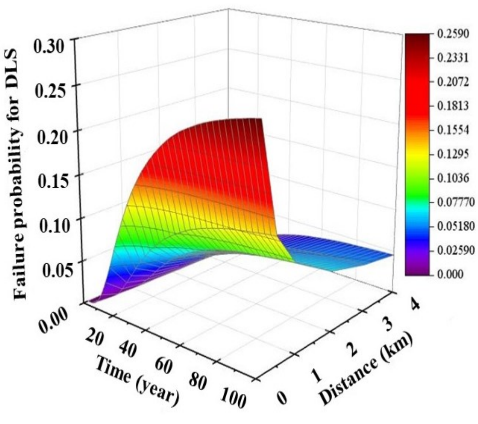


Fig. 5.4.10 Time-variant P_{dls} of CS-RC girder in Hagi City.

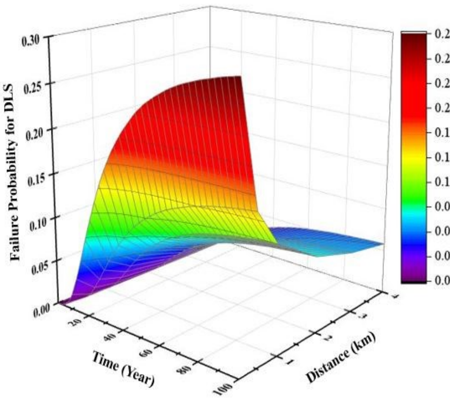


Fig. 5.4.11 Time-variant P_{dls} of CS-RC girder in Nagasaki City.

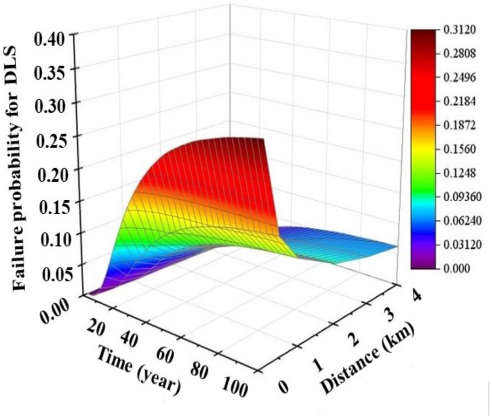


Fig. 5.4.12 Time-variant P_{dls} of CS-RC girder in Amakusa City.

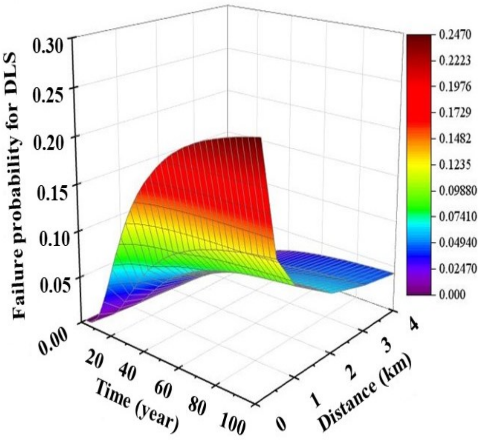


Fig. 5.4.13 Time-variant P_{dls} of CS-RC girder in Aburatsu City.

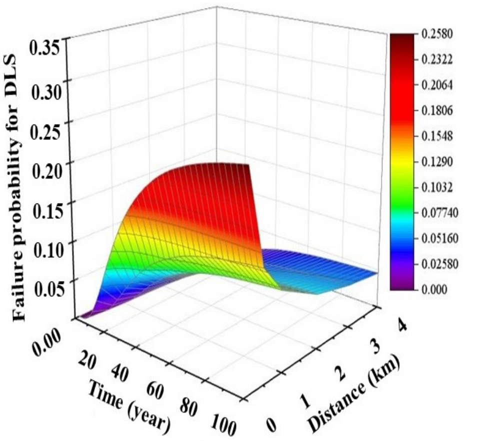


Fig. 5.4.14 Time-variant P_{dls} of CS-RC girder in Muroto City.

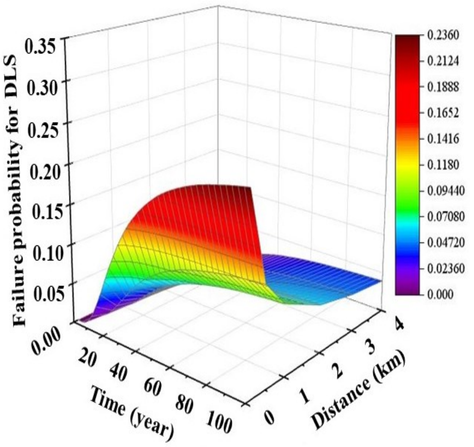


Fig. 5.4.15 Time-variant P_{dls} of CS-RC girder in Shionomisaki City.

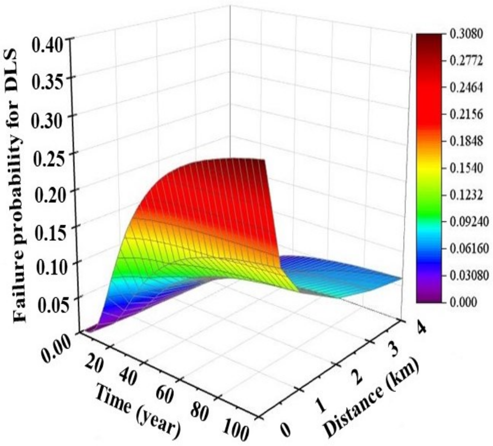


Fig. 5.4.16 Time-variant P_{dls} of CS-RC girder in Yonagunijima City.

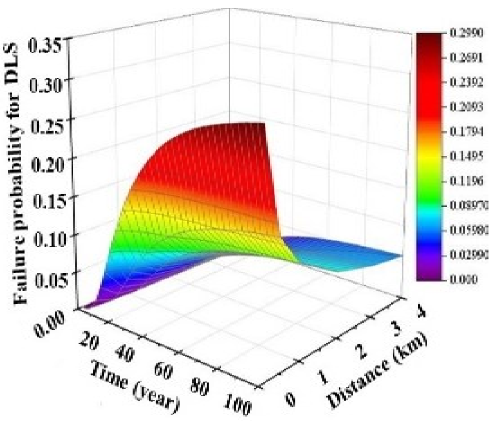


Fig. 5.4.17 Time-variant P_{dls} of CS-RC girder in Miyakojima City.

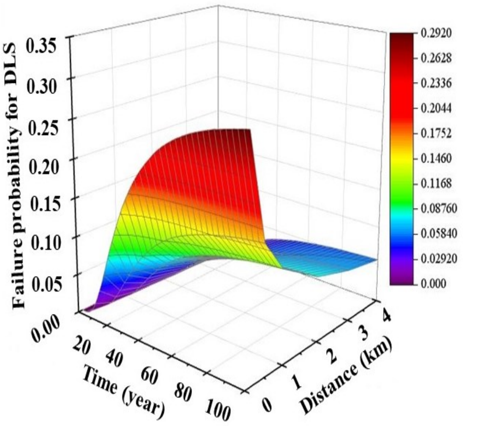


Fig. 5.4.18 Time-variant P_{dls} of CS-RC girder in Iriomotejima City.

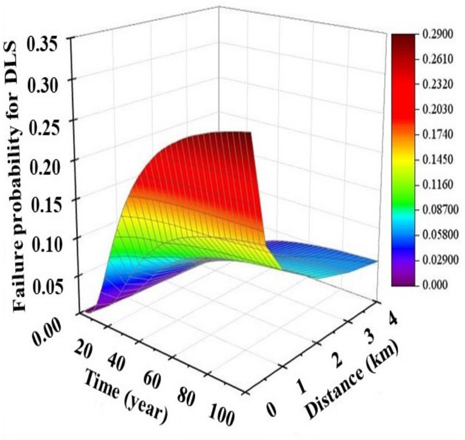


Fig. 5.4.19 Time-variant P_{dls} of CS-RC girder in Minamidaitojima City.

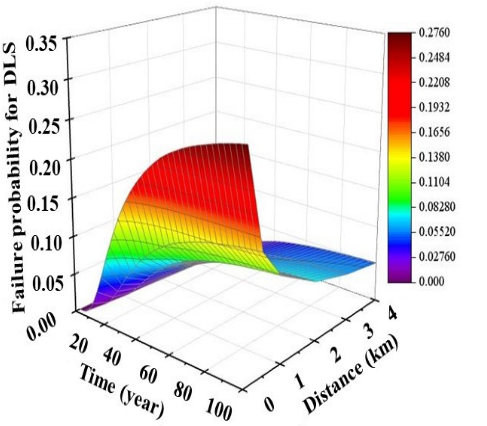


Fig. 5.4.20 Time-variant P_{dls} of CS-RC girder in Naha City.

corrosion initiation begins. When the considered bridges are near the coastline, the failure probability for DLS is substantially high and with as the distance from shore rises the magnitude of failure probability for DLS reduces. For d = 100 m in region no. 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, and 20 the P_{dls} for CS-RC bridge is 35.82%, 32.79%, 35.24%, 23.95%, 28.91%, 33.31%, 30.73%, 21.32%, 28.52%, 25.84%, 29.74%, 31.17%, 24.64%, 25.80%, 23.57%, 30.77%, 29.86%, 29.15%, 28.96%, and 27.60%, respectively. Furthermore, the failure probability of RC bridges for DLS among the twenty cities at identical locations are different owing to different wind characteristics. To explain more clearly the effect of coastal distance, rebar type, and city on the failure probability for DLS, two cities are discussed here briefly. As shown in Fig. 5.4.6, the failure probability of CS-

RC bridge at d = 0.1 km is 33.30%, while this value decreased by 43.6% when coastal distance increased to d = 0.5 km. Similarly, from Fig 5.4.8, for CS-RC bridge in Maizuru City (City 8), the failure probability for DLS at d = 100 m and d = 500 m, are 21.32% and 10.7%, respectively.

However, once ordinary rebar is replaced by stainless steel, the drastic reduction in failure probability for DLS is noted. For example, for d = 100 m, as ordinary reinforcement is replaced by corrosion resistant RC girder, the P_{dls} in City no. 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, and 20 the P_{dls} for CS-RC bridge is 8.18%, 7.07%, 7.95%, 4.36%, 5.80%, 7.25%, 6.37%, 3.67%, 5.70%, 4.93%, 6.04%, 6.51%, 4.55%, 4.90%, 4.28%, 6.38%, 6.07%, 5.88%, 5.82%, and 5.43%, respectively. This is owing to the high corrosion resistance performance of stainless steel rebars over ordinary rebar. Notably, there is chromium layer surface over the SS rebar which provides corrosion resistance performance. From Fig. 5.4.26, the failure probability of SS- RC bridge girder at d = 0.1 km is 7.25% and this value reduced to 2.99% for d = 0.5 km. This result confirms that use of stainless steel reinforcement instead of conventional reinforcement reduced failure probability for DLS by 77.6%. Similarly,

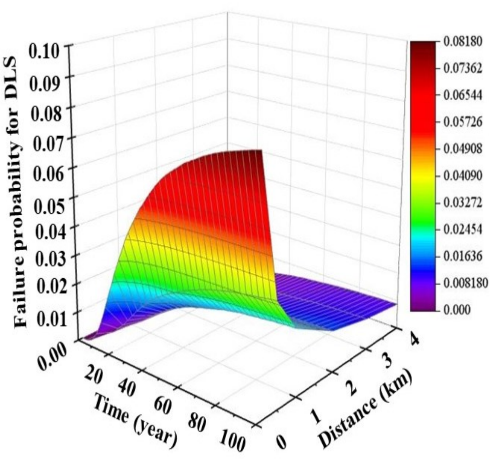


Fig. 5.4.21 Time-variant P_{dls} of SS-RC girder in Wakkanai City.

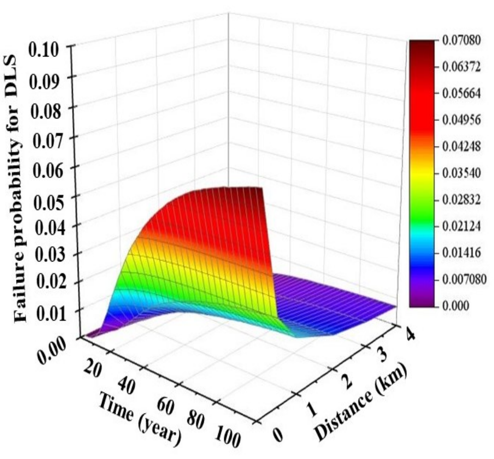


Fig. 5.4.22 Time-variant P_{dls} of SS-RC girder in Tomakomai City.

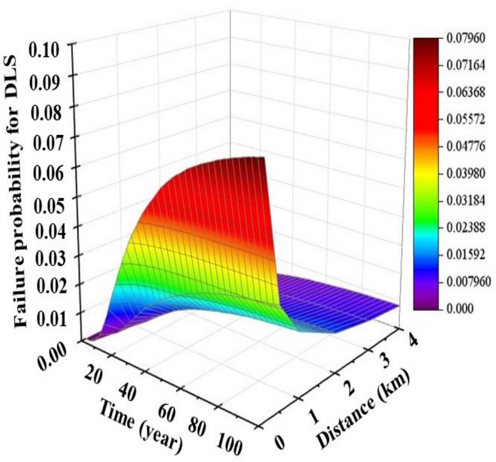


Fig. 5.4.23 Time-variant P_{dls} of SS-RC girder in Nemuro City.

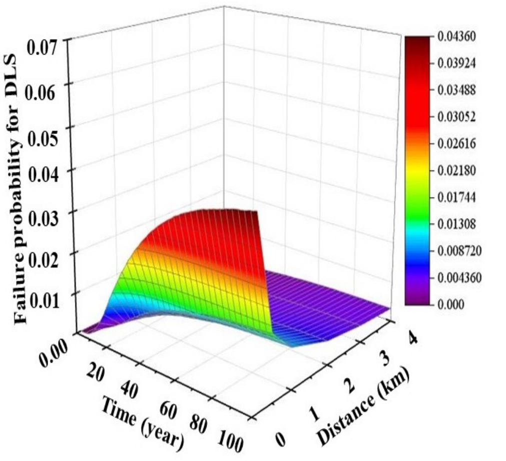


Fig. 5.4.24 Time-variant P_{dls} of SS-RC girder in Fukaura City.

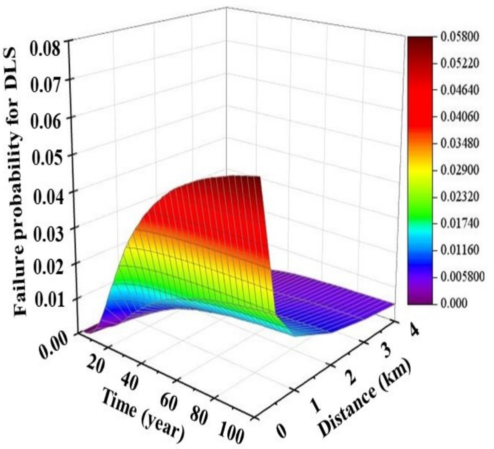


Fig. 5.4.25 Time-variant P_{dls} of SS-RC girder in Onahama City.

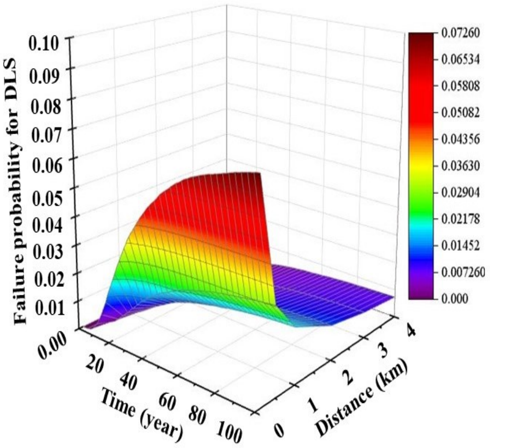


Fig. 5.4.26 Time-variant P_{dls} of SS-RC girder in Niigata City.

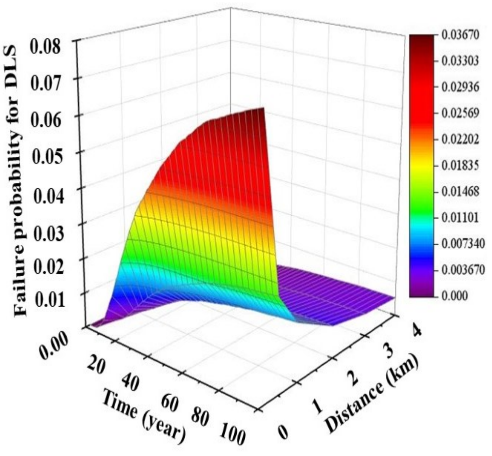


Fig. 5.4.27 Time-variant P_{dls} of SS-RC girder in Wajima City.

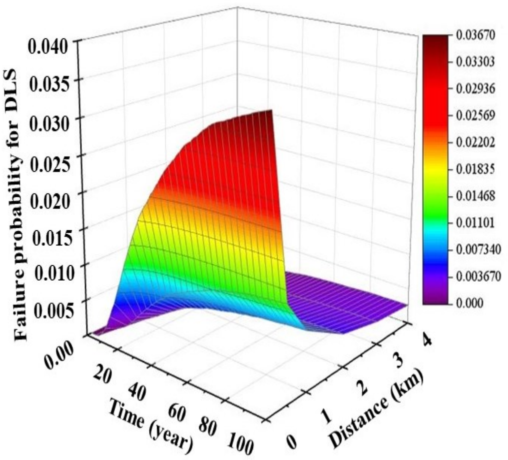


Fig. 5.4.28 Time-variant P_{dls} of SS-RC girder in Maizuru City.

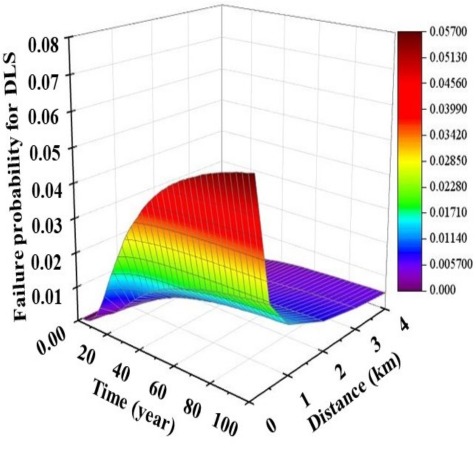


Fig. 5.4.29 Time-variant Pdls of SS-RC girder in Sakai City.

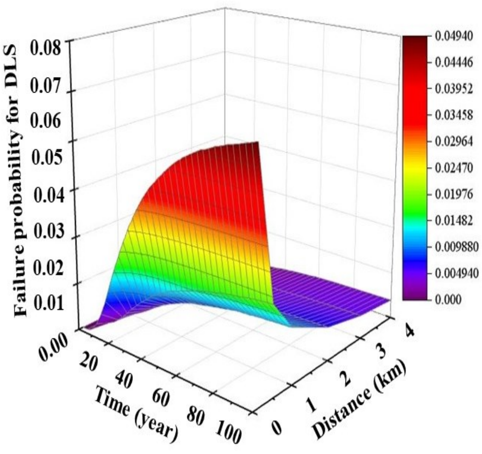


Fig. 5.4.30 Time-variant Pdls of SS-RC girder in Hagi City.

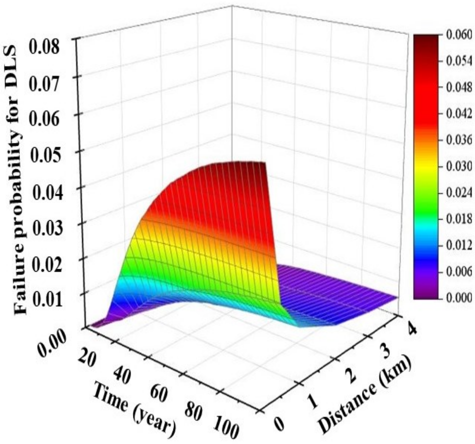


Fig. 5.4.31 Time-variant Pdls of SS-RC girder in Nagasaki City.

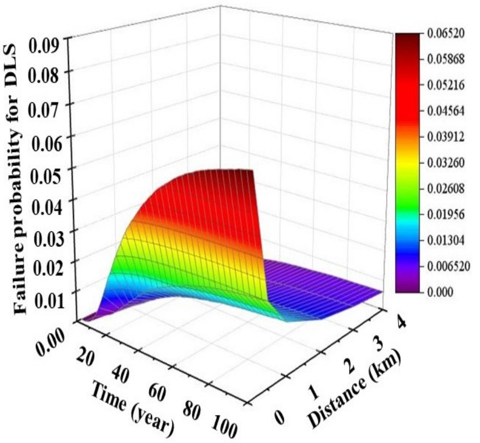


Fig. 5.4.32 Time-variant Pdls of SS-RC girder in Amakusa City.

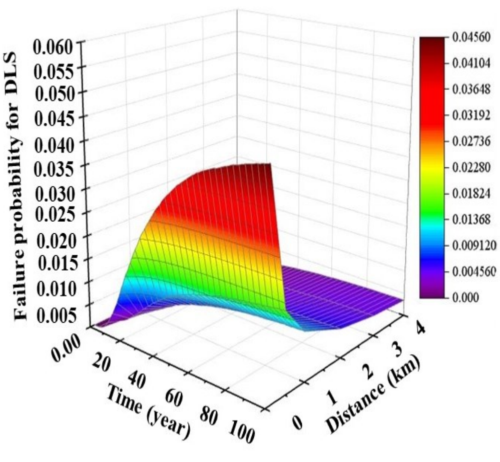


Fig. 5.4.33 Time-variant Pdls of SS-RC girder in Aburatsu City.

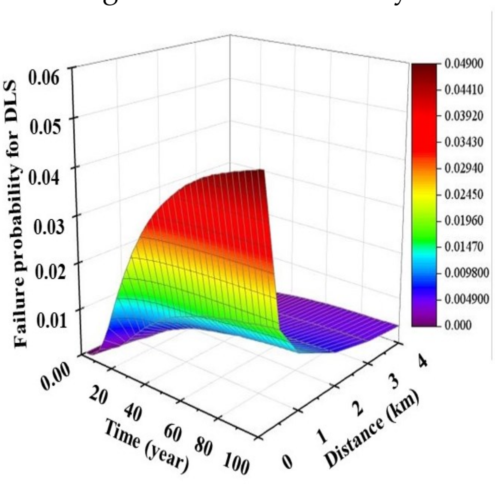


Fig. 5.4.34 Time-variant Pdls of SS-RC girder in Muroto City.

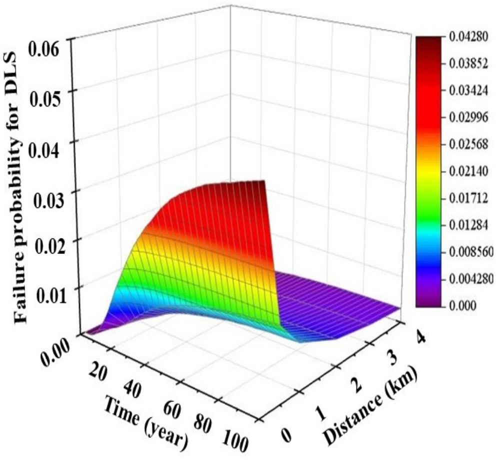


Fig. 5.4.35 Time-variant P_{dls} of SS-RC girder in Shionomisaki City.

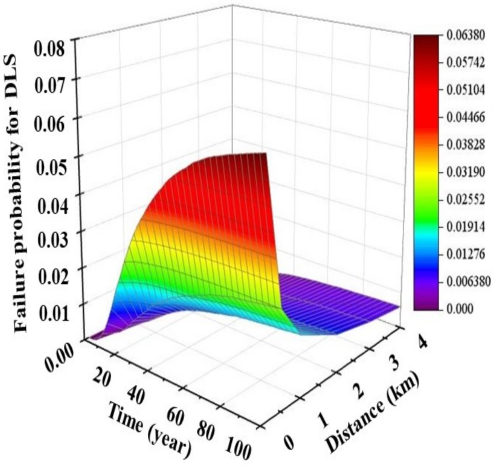


Fig. 5.4.36 Time-variant P_{dls} of SS-RC girder in Yonagunijima City.

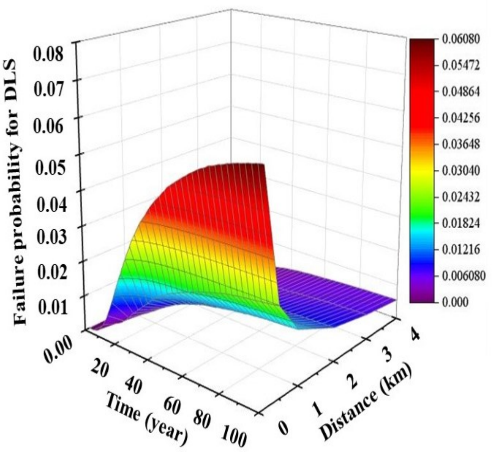


Fig. 5.4.37 Time-variant P_{dls} of SS-RC girder in Miyakojima City.

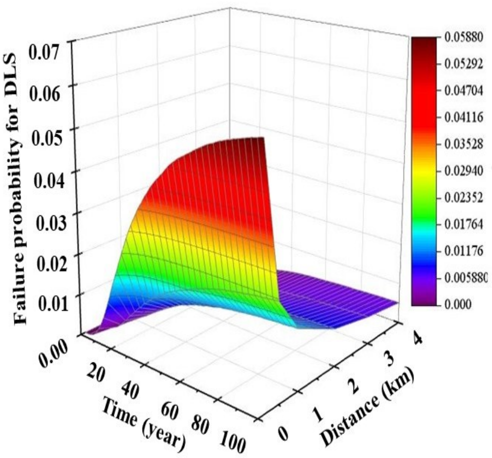


Fig. 5.4.38 Time-variant P_{dls} of SS-RC girder in Iriomotejima City.

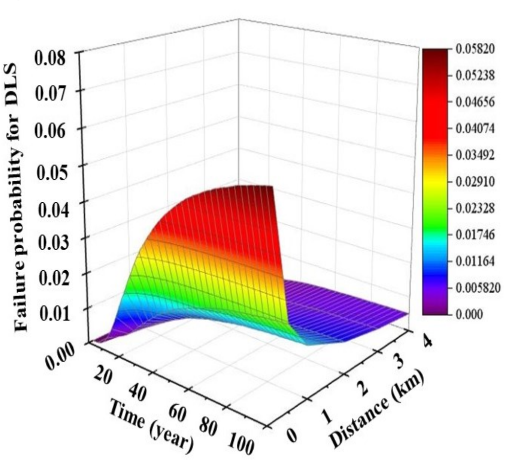


Fig. 5.4.39 Time-variant P_{dls} of SS-RC girder in Minamidaitojima City.

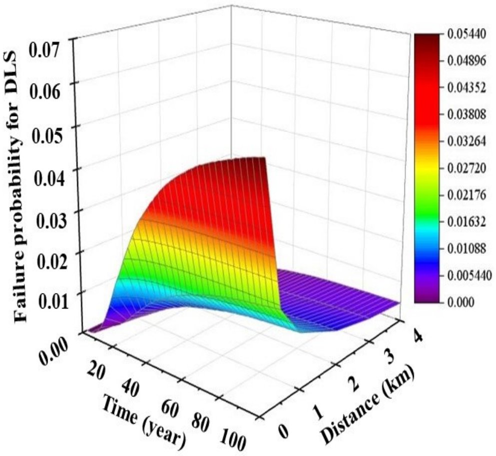


Fig. 5.4.40 Time-variant P_{dls} of SS-RC girder in Naha City.

in Maizuru city, failure probability of SS-RC bridge for DLS at d = 0.1 km and 0.5 km are found 3.67% and 1.29%, respectively.

5.4.2 Damage Probability of RC Bridge for Final Limit State

The time-variant damage probability of SS- & CS-RC bridges for ultimate limit state constructed at several locations in considered twenty cities are presented in Figs. 5.5.1 to 5.5.40. As shown in these figures, it clear that failure probability for CS- and SS-RC bridges for ultimate limit state, ULS. Since the P_{uls} of girder mostly relies on vehicle load, it is initially rather flat in all cities and develops extremely low rate beyond starting location. Beyond this first flat area, P_{uls} grows in two stages over time as a result of the two reinforcement layers' growing deterioration and steel weight loss. The lowest reinforcement corrodes earlier compared to the upper rebars because airborne chloride enters from the RC girder's bottom surface. P_{uls} rises in the first stage as a result of the first rebar layer corroding. P_{uls} continues to rise in the second stage, signifying the deterioration of the rebars' second layer. This suggests that tensile rebar arrangement determines nature of time-variant P_{uls} curves. When the considered bridges are near the coastline, the failure probability for ULS is substantially high and with the rising of d the magnitude of failure probability for ULS reduces. For

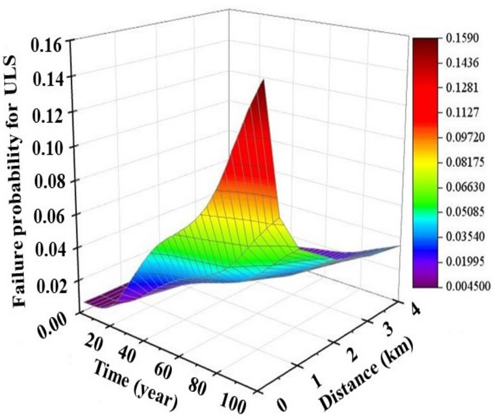


Fig. 5.5.1 Time-variant P_{uls} of CS-RC girder in Wakkanai City.

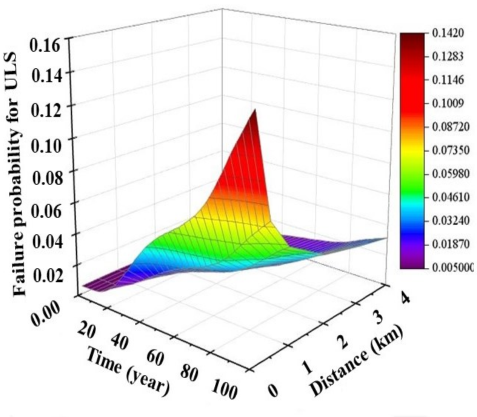


Fig. 5.5.2 Time-variant P_{uls} of CS-RC girder in Tomakomai City.

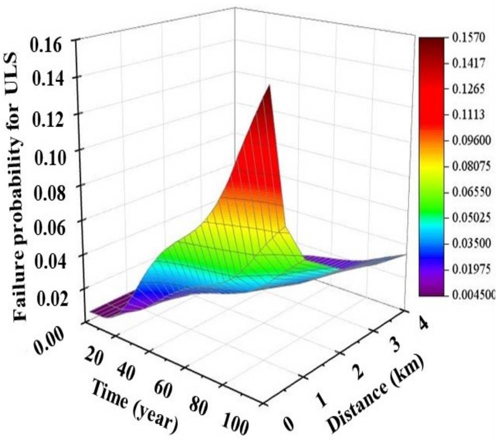


Fig. 5.5.3 Time-variant P_{uls} of CS-RC girder in Nemuro City.

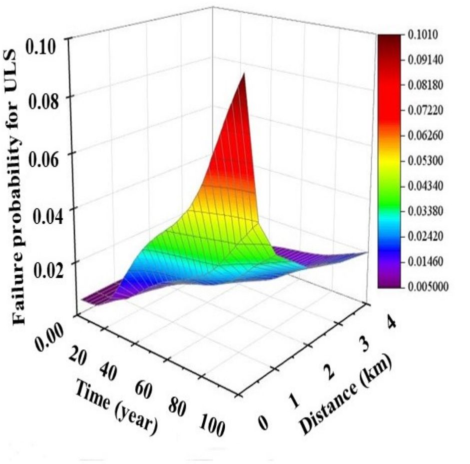


Fig. 5.5.4 Time-variant P_{uls} of CS-RC girder in Fukaura City.

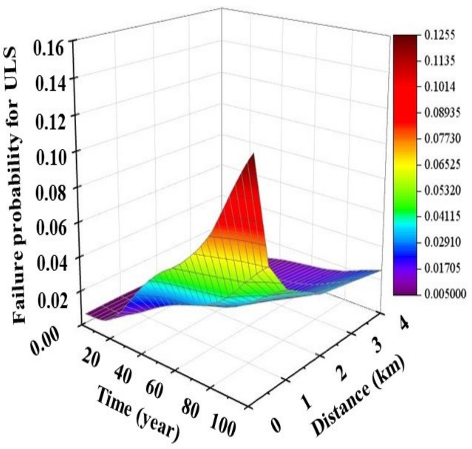


Fig. 5.5.5 Time-variant P_{uls} of CS-RC girder in Onahama City.

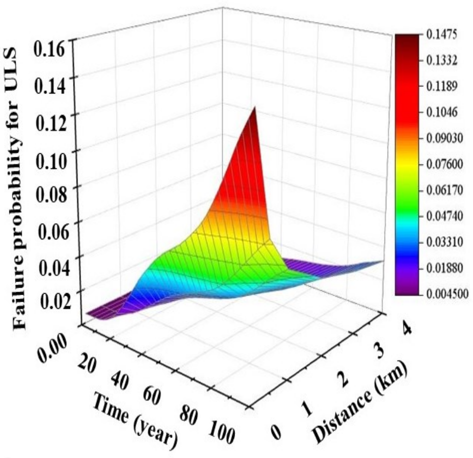


Fig. 5.5.6 Time-variant P_{uls} of CS-RC girder in Niigata City.

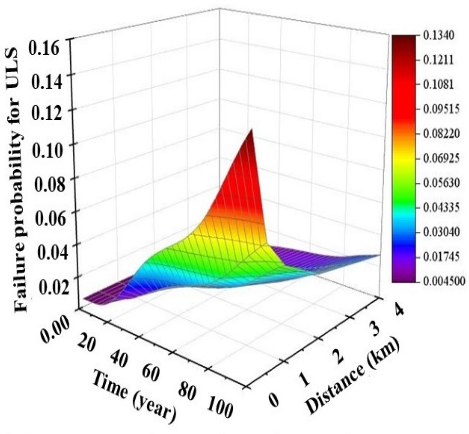


Fig. 5.5.7 Time-variant P_{uls} of CS-RC girder in Wajima City.

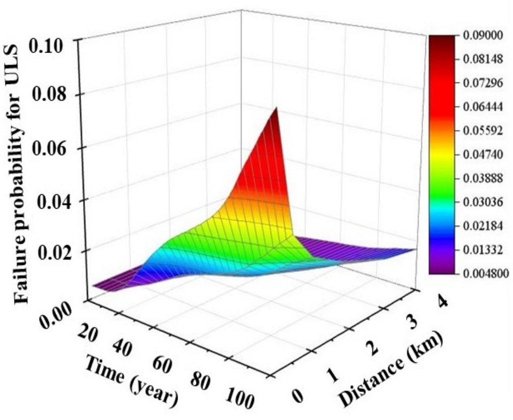


Fig. 5.5.8 Time-variant P_{uls} of CS-RC girder in Maizuru City.

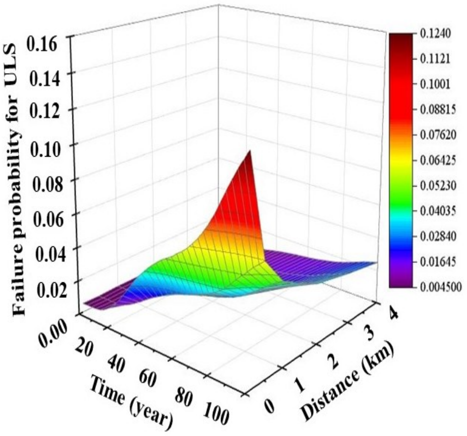


Fig. 5.5.9 Time-variant P_{uls} of CS-RC girder in Sakai City.

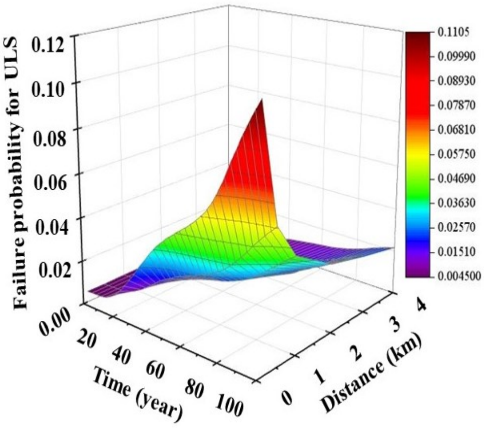


Fig. 5.5.10 Time-variant P_{uls} of CS-RC girder in Hagi City.

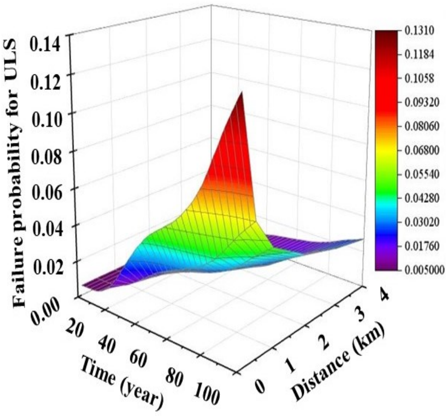


Fig. 5.5.11 Time-variant P_{uls} of CS-RC girder in Nagasaki City.

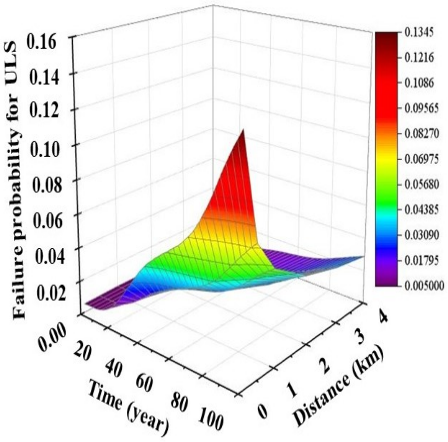


Fig. 5.5.12 Time-variant P_{uls} of CS-RC girder in Amakusa City.

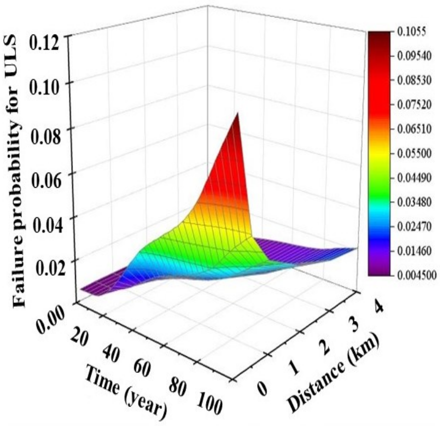


Fig. 5.5.13 Time-variant P_{uls} of CS-RC girder in Aburatsu City.

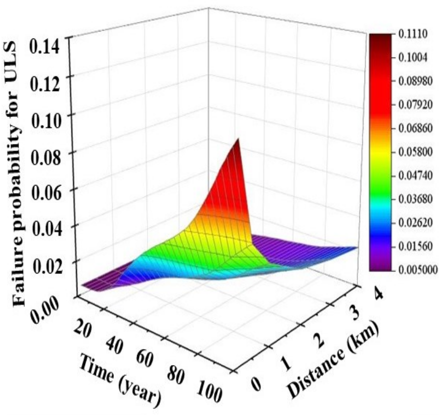


Fig. 5.5.14 Time-variant P_{uls} of CS-RC girder in Muroto City.

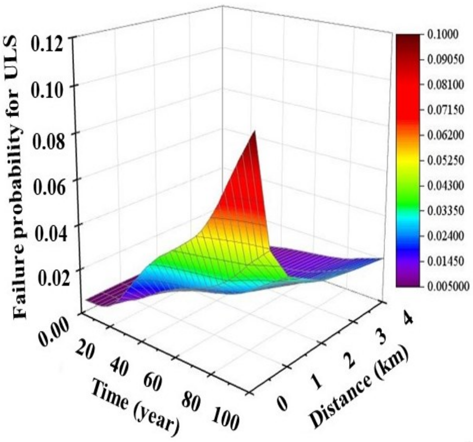


Fig. 5.5.15 Time-variant P_{uls} of CS-RC girder in Shionomisaki City.

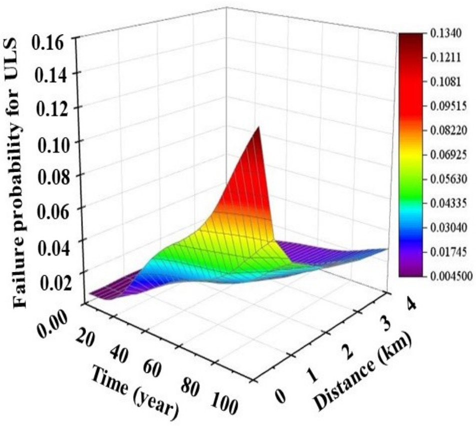


Fig. 5.5.16 Time-variant P_{uls} of CS-RC girder in Yonagunijima City.

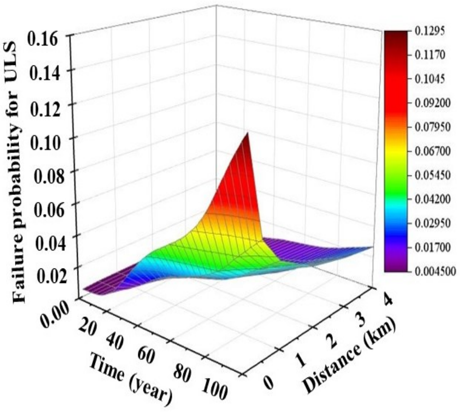


Fig. 5.5.17 Time-variant P_{uls} of CS-RC girder in Miyakojima City.

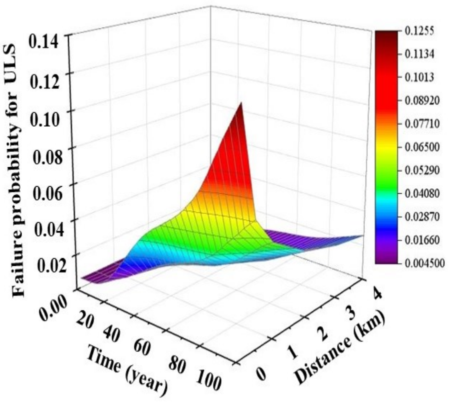


Fig. 5.5.18 Time-variant P_{uls} of CS-RC girder in Iriomotejima City.

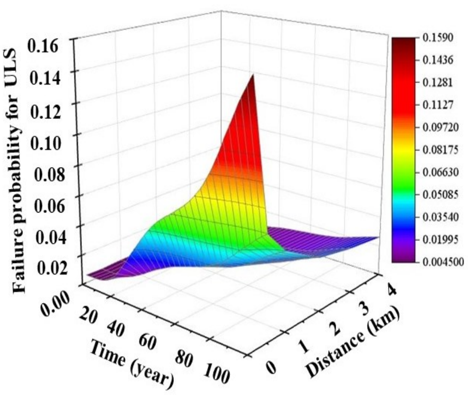


Fig. 5.5.19 Time-variant P_{uls} of CS-RC girder in Minamidaitojima City.

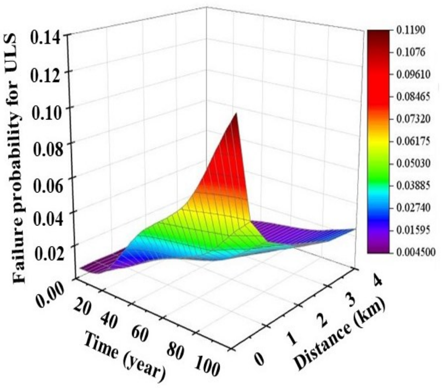


Fig. 5.5.20 Time-variant P_{uls} of CS-RC girder in Naha City.

example, at d = 100 m, the P_{uls} for ordinary girder constructed in region no. 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, and 20 the P_{uls} is 15.89%, 14.15%, 15.68%, 10.09%, 12.54%, 14.71%, 13.40%, 8.98%, 12.36%, 11.01%, 13.07%, 13.40%, 10.51%, 11.07%, 10.00%, 13.38%, 12.92%, 12.52%, 15.89%, and 11.86%, respectively. Furthermore, the failure probability of RC bridges for ULS among the twenty cities at identical locations are different owing to different wind characteristics. To explain more clearly the effect of coastal distance, rebar type, and city on the failure probability for ULS, two cities are discussed here briefly. As shown in Fig. 5.5.6, the failure probability of CS-RC bridge for ULS at d = 0.1 km in Niigata City is 14.71%, while this value decreased by 45.9% when coastal distance increased to d = 0.5 km. Similarly, from Fig 5.5.8, for CS-RC bridge in Maizuru City (City 8), the failure probability for ULS at d = 100 m and d = 500 m, are 8.98% and 4.54%, respectively.

However, once ordinary reinforcement is replaced by stainless steel, drastic reduction in failure probability for ULS is noted. For example, at d = 100 m, the P_{uls} for stainless steel reinforced concrete girder constructed in region no. 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, and 20 the P_{uls} is 3.68%, 3.18%, 3.47%, 2.21%, 2.70%, 3.30%, 2.99%, 1.89%, 2.72%, 2.34%, 2.87%, 3.00%, 2.23%, 2.40%, 2.12%, 3.02%, 2.87%, 2.77%, 2.76%, and 2.69%, respectively. This is owing to the high corrosion resistance performance of stainless steel rebars over ordinary rebar. Notably, there is chromium layer surface over the SS rebar which provides corrosion resistance performance. From Fig. 5.5.26, the failure probability of SS-RC bridge girder for ULS at d = 0.1 km in Niigata City is 3.30% and this value reduced to 1.76% for d = 0.5 km. This result confirms that replacement of ordinary reinforcement by stainless steel reduced the failure probability for ULS by 77.56%. Similarly, in Maizuru city, failure probability of SS-RC bridge for ULS at d = 0.1 km and 0.5 km are found 1.89% and 1.05%, respectively.

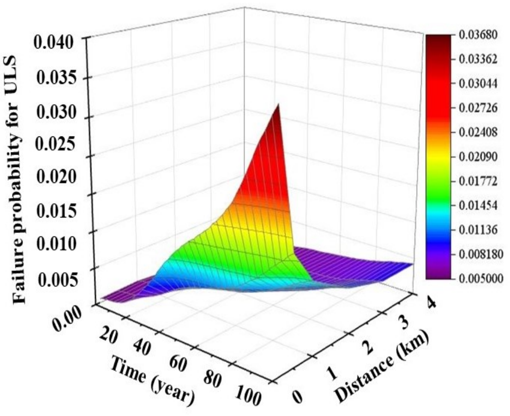


Fig. 5.5.21 Time-variant P_{uls} of SS-RC girder in Wakkanai City.

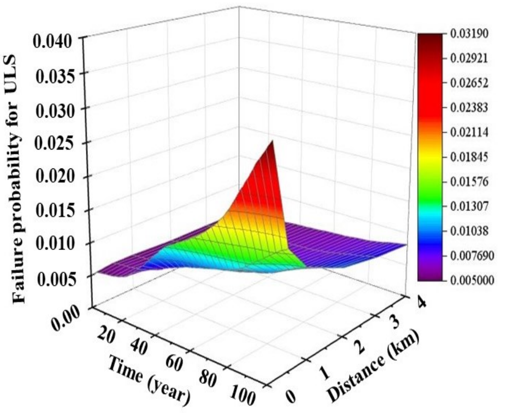


Fig. 5.5.22 Time-variant P_{uls} of SS-RC girder in Tomakomai City.

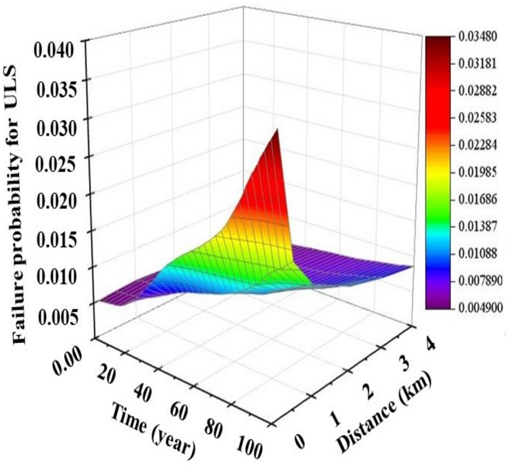


Fig. 5.5.23 Time-variant P_{uls} of SS-RC girder in Nemuro City.

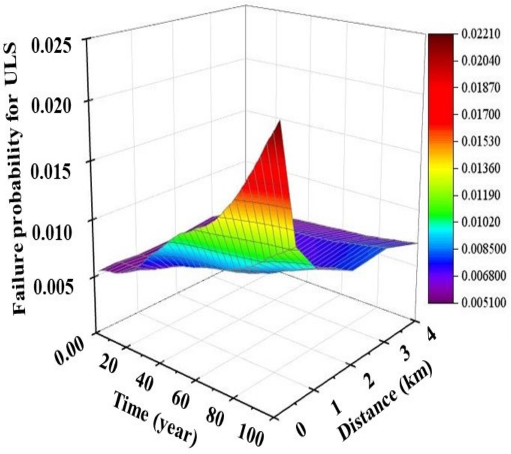


Fig. 5.5.24 Time-variant P_{uls} of SS-RC girder in Fukaura City.

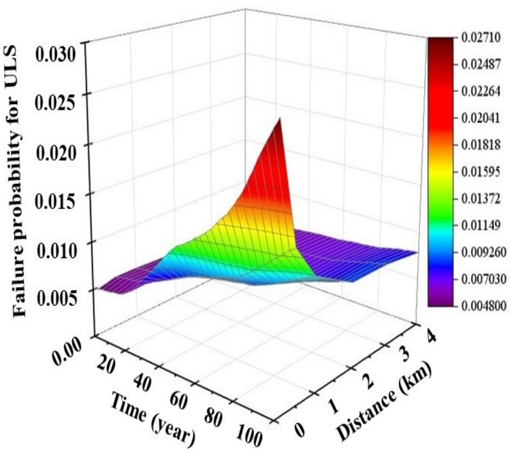


Fig. 5.5.25 Time-variant P_{uls} of SS-RC girder in Onahama City.

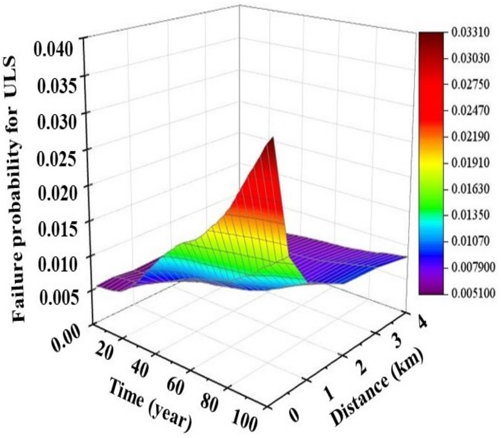


Fig. 5.5.26 Time-variant P_{uls} of SS-RC girder in Niigata City.

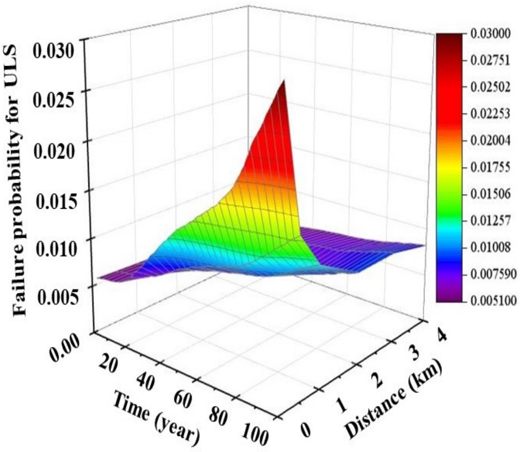


Fig. 5.5.27 Time-variant P_{uls} of SS-RC girder in Wajima City.

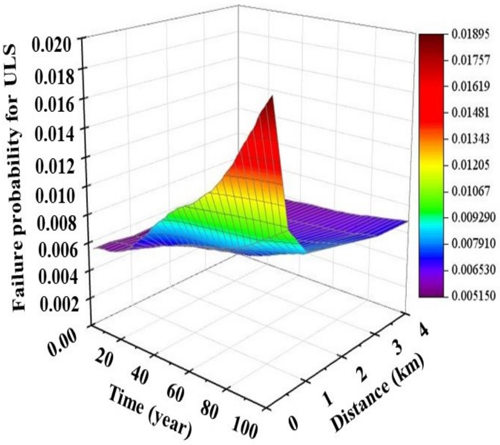


Fig. 5.5.28 Time-variant P_{uls} of SS-RC girder in Maizuru City.

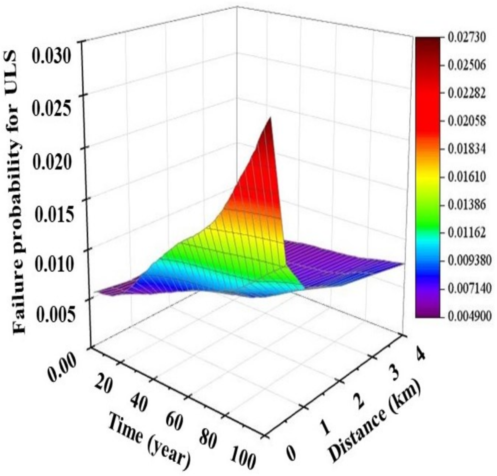


Fig. 5.5.29 Time-variant P_{uls} of SS-RC girder in Sakai City.

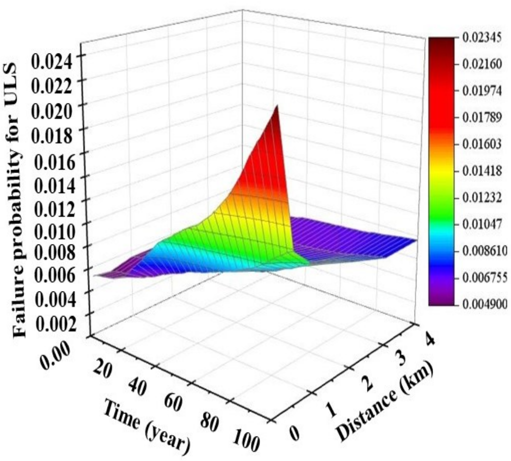


Fig. 5.5.30 Time-variant P_{uls} of SS-RC girder in Hagi City.

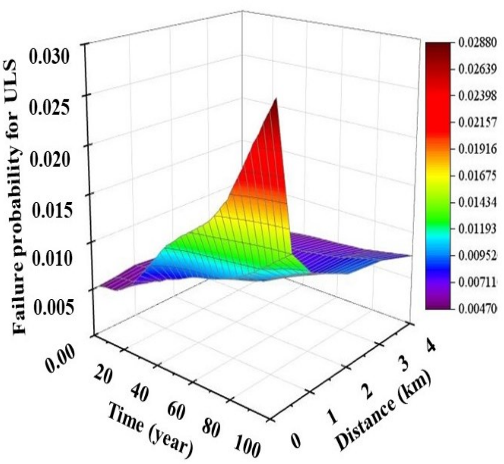


Fig. 5.5.31 Time-variant P_{uls} of SS-RC girder in Nagasaki City.

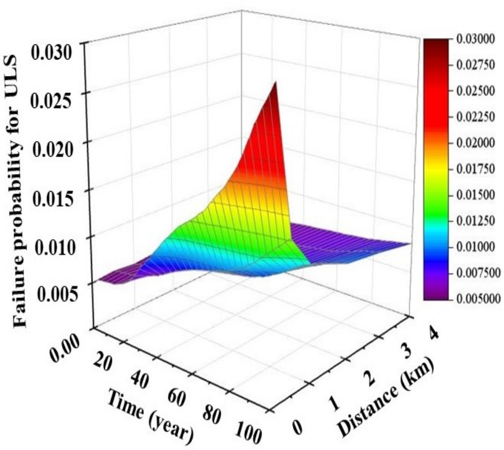


Fig. 5.5.32 Time-variant P_{uls} of SS-RC girder in Amakusa City.

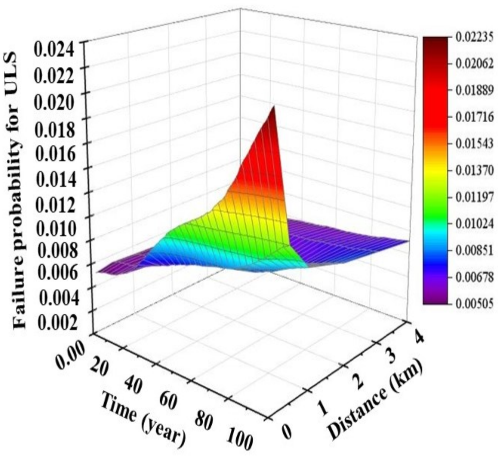


Fig. 5.5.33 Time-variant P_{uls} of SS-RC girder in Aburatsu City.

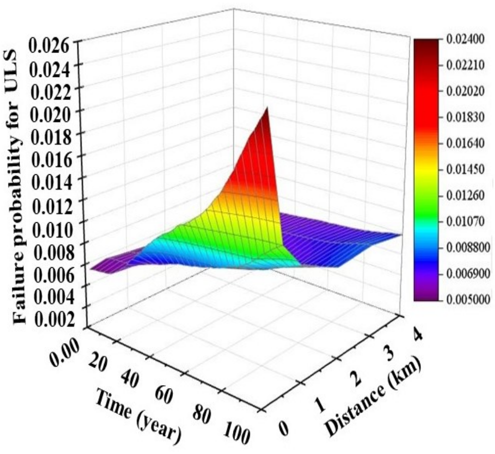


Fig. 5.5.34 Time-variant P_{uls} of SS-RC girder in Muroto City.

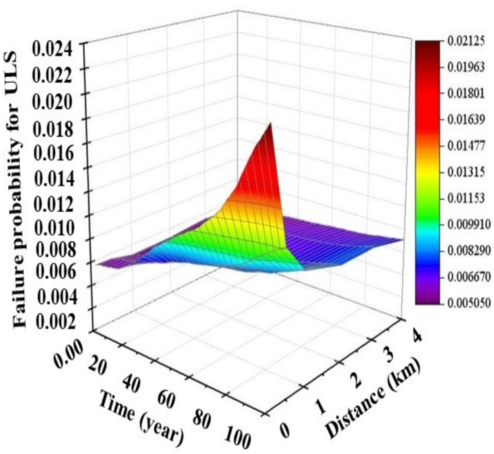


Fig. 5.5.35 Time-variant P_{uls} of SS-RC girder in Shionomisaki City.

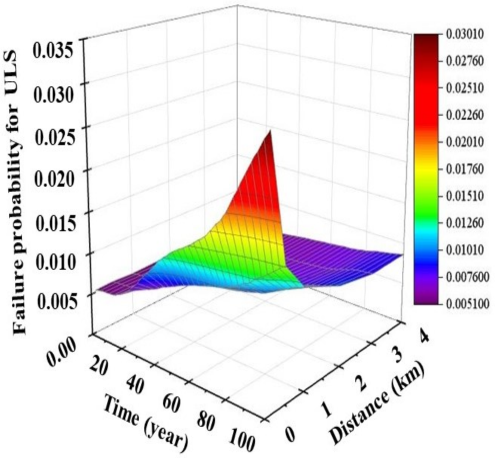


Fig. 5.5.36 Time-variant P_{uls} of SS-RC girder in Yonagunijima City.

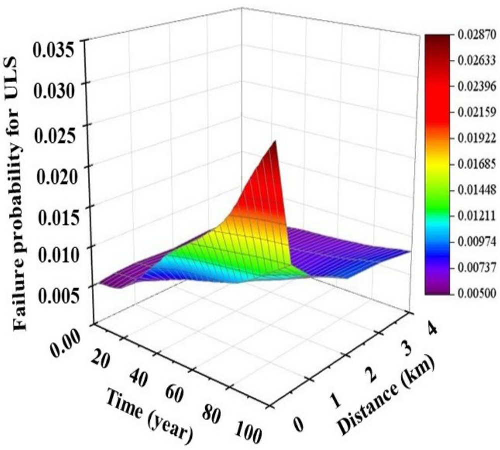


Fig. 5.5.37 Time-variant P_{uls} of SS-RC girder in Miyakojima City.

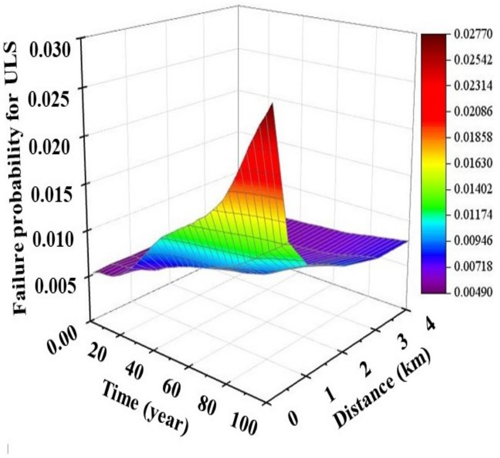


Fig. 5.5.38 Time-variant P_{uls} of SS-RC girder in Iriomotejima City.

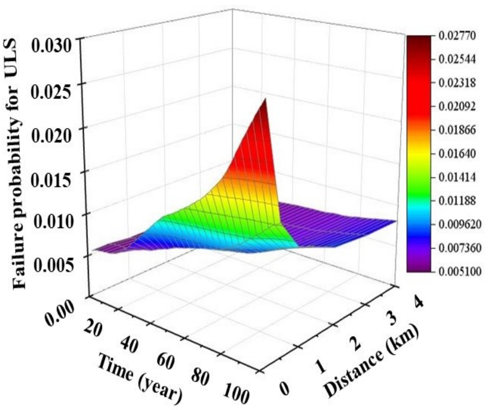


Fig. 5.5.39 Time-variant P_{uls} of SS-RC girder in Minamidaitojima City.

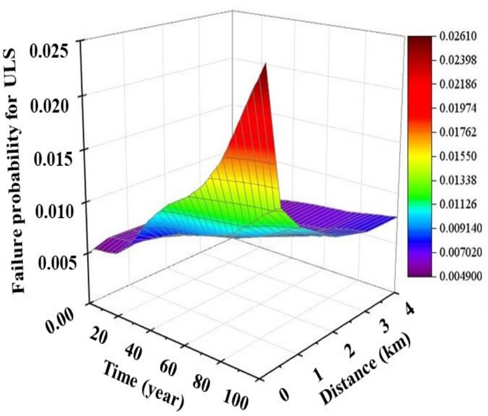


Fig. 5.5.40 Time-variant P_{uls} of SS-RC girder in Naha City.

5.5 RELIABILITY-BASED INSPECTION PERIOD

The inspection time period dominates lifetime cost of bridge since it influences the observation, maintenance, and eventual failure costs. This case study used a reliability-based inspection period to identify the ideal location for the usage of SS rebar in RC bridges. Notably, life-cycle reliability analysis is used to establish the observation time for stainless steel structure reinforced bridges because current observation rules do not define this information. As per Yanweerasak et al. (2018), the critical dependability factor (βt) of 2.3 was applied to calculate girder's renovation time subjected to ULS. The reliability of girder in Niigata is computed at d = 100 m, that presents the most intensity of chloride threat. According to Fig. 5.6, the performance of the bridge girder reinforced with ordinary, and corrosion resistant steel will be equal to their critical level at 18 & 29 years, respectively. Although an ordinary reinforced concrete bridge will be constructed in challenging area (like Niigata), its performance will remain above the critical limit at least for 100 years if it is inspected every 18 years. The time it

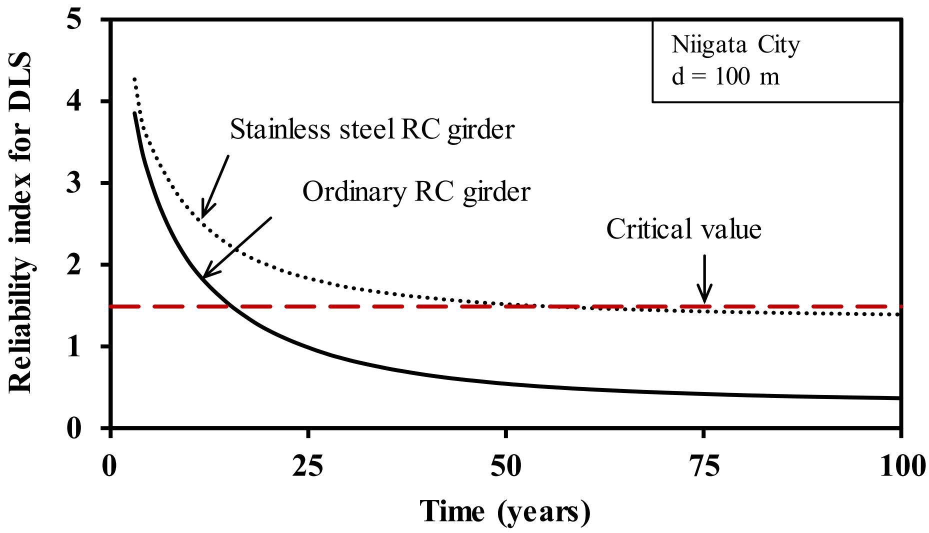


Fig. 5.6 Reliability index for the durability limit state (DLS) over time.

takes to reach the critical level is significantly longer for cities with less severe airborne chloride hazards. Fib (2013) suggests that β_t = 1.5 for longevity design. The lifetime reliability linked to corrosion cracking & the critical suggested by Fib (2013) are contrasted in Fig. 5.7. The reliability of an ordinary RC bridge at 15 years, as illustrated in Fig 5.7, is nearly identical to that utilized in Yanweerasak et al. (2018). Hence, the observation period of 15 yrs. is assumed to both SS & CS girders.

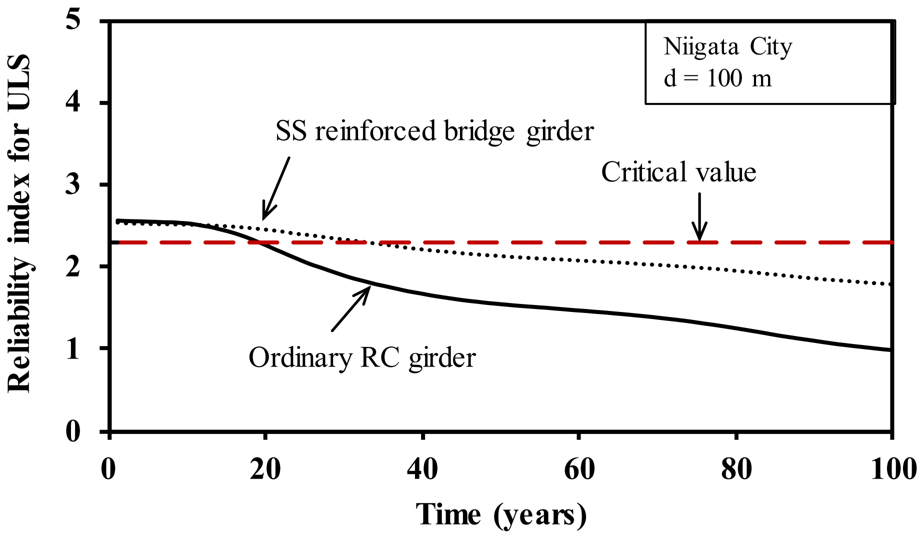
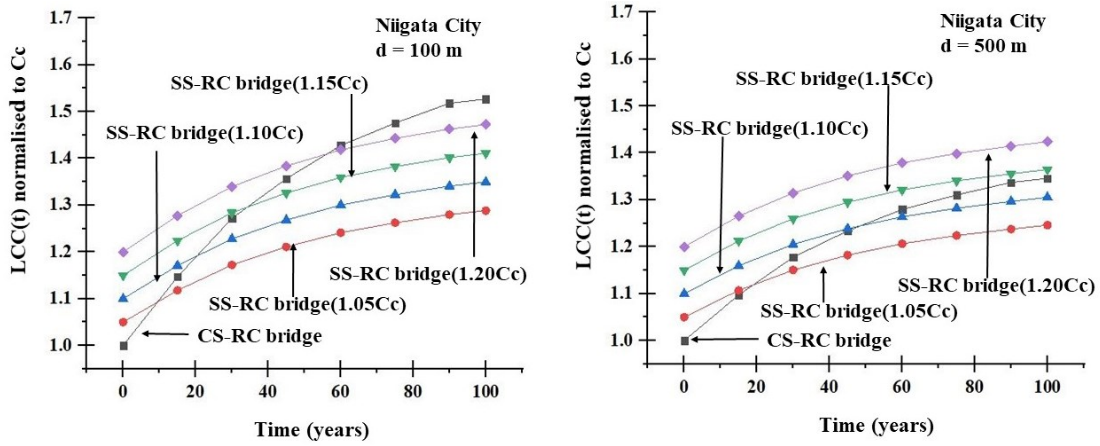


Fig. 5.7 Reliability index for the ultimate limit state (ULS) over time

5.6 LCC OF CS- AND SS-RC BRIDGE

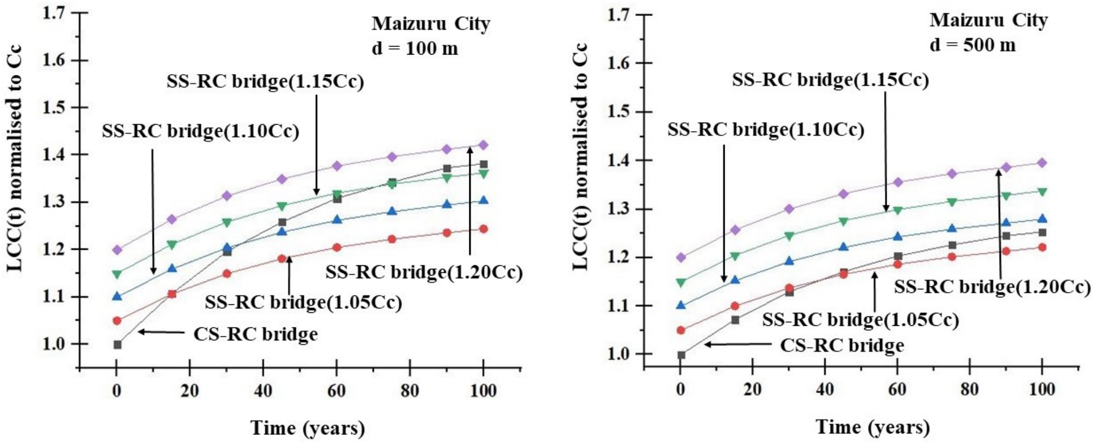
It's critical to investigate how the type of rebar, the distance from the coast, and urban areas affect the time-variant life-cycle cost (LCC) of RC bridges. Although, the time-variant LCC of SS- & CS-RC girders were investigated at all considered distances in all cities. However, to explain precisely, time-dependent LCC of SS- & CS-RC bridges in Niigata City at d = 100 m and d = 500 m are presented in Fig. 5.8. As shown in Fig. 5.8, the long-term behaviour of the stainless-steel rebar alternative surpasses compared to ordinary RC girder depending on the initial cost of SS rebar, even though the former is initially less expensive than the latter. However, the time required for SS rebar to be cost-effective over CS rebar is strongly influenced by location of structure from the coastline, price of SS rebar and in which city the bridge constructed. For example, it is cleared from Fig. 5.8a, at d = 100 m in Niigata town, at 9.54 year after construction, LCC of SS- & CS-RC bridge with $1.05C_C$ is equal. However, with increasing cost of SS rebar by 10%, 15%, and 20%, the equal LCC is found at the year of 19.84 year, 33.69 year, and 55.80 year, respectively. Nevertheless, with increasing coastal distance to d = 0.5 km, the equivalent LCC for CS- and SS-RC bridges with $1.05C_C$ and $1.10C_C$, are 18.55 year and 48.2 year, respectively. It should be noted that for SS-RC bridge at d = 0.5 km in Niigata City, equivalent LCC is not found indicating that in this location these grades of SS rebar will be cost-ineffective.

Fig. 5.9 illustrated the LCC variation of RC bridges at d = 0.1 km and d = 0.5 km in Maizuru City, an example of mild hazard prone city. The general trend of LCC curves are approximately identical. With the increasing time after construction, the LCC increases nonlinearly as the cost due to inspection, maintenance and failure increases. It should be noted that time required to obtain equivalent LCC at identical location is longer in Maizuru City compared to that in Niigata City. This could be because of lower windborne chlorine risk in



(a) (b)

Fig. 5.8 Time-variant LCC of SS- & CS-RC girder constructed in Niigata town at (a) d = 100 m, and (b) d = 500 m.



(a) (b)

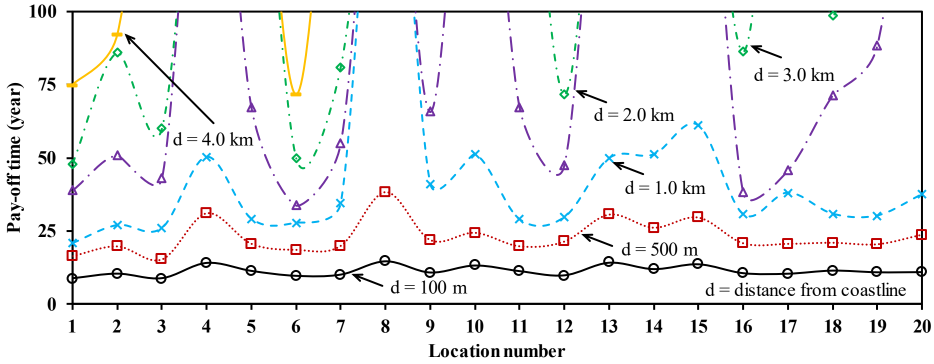
Fig. 5.9 Time-variant LCC of SS- & CS-RC bridge girder constructed in Maizuru City at (a) d = 100 m, and (b) d = 500 m.

Maizuru city that need longer time to balance initial construction cost by lowering the inspection, maintenance, and failure cost. Fig. 5.9a reports that at d = 100 m in Maizuru town, time required to obtain identical lifetime cost for SS- and CS-RC girder with 1.05C_C, 1.10C_C, and 1.15C_C is 14.7 year, 33.5 year, and 70.5 year, respectively. Nevertheless, at d = 0.5 km in Maizuru City, the equivalent LCC for SS RC bridge with 1.05CC is found 38.3 year. It should be noted that for SS-RC bridge with 1.10C_C, 1.15 C_C, and 1.20 C_C, the equivalent LCC is not found. Therefore, it is crucial to determine the time required to obtain equivalent LCC for SS- & CS-RC girder which is referred as pay-off time for SS rebar cost-effectiveness.

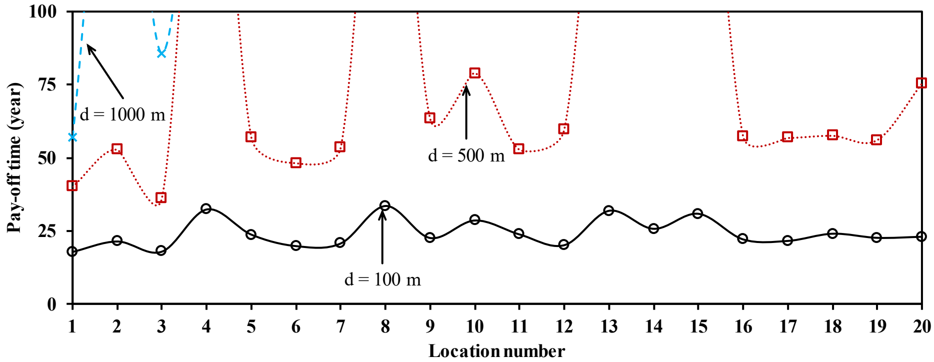
5.7 PAY-OFF TIME FOR STAINLESS STEEL REINFORCEMENT ECONOMICAL VIABILITY

Pay-off time is one of critical indicators for assessing the economic viability of a project. It is defined as the time required for SS rebar to be cost-effective over CS rebar. When the lifetime price of CS and SS-RC girders at identical location is same, the corresponding time is called pay-off time ($t_{pay\text{-}off}$). Fig. 5.10 represents the $t_{pay\text{-}off}$ for stainless steel reinforcement economic viability considering variation in prices over CS rebar used in RC bridges at several locations from the coastline (d = 0.1 km, 0.50 km, 1.00 km, 2.00 km, 3.00 km, and 4.00 km) in twenty cities in Japan. The effect of costal distances on the $t_{pay\text{-}off}$ is remarkable. When RC bridges are located closer to the coastline, the $t_{pay\text{-}off}$ is smaller indicating the after few years of construction, the SS rebar will be economically feasible over ordinary reinforcement. With increasing distance from coastline, the $t_{pay\text{-}off}$ is significantly high due to the lower airborne chloride hazard. At far away from the coastline, the reduction in inspection and maintenance cost by SS rebar cannot completely balanced the higher initial cost of SS rebar. For example, for city 6 (Niigata), the $t_{pay\text{-}off}$ for SS rebar with 5% increased initial cost at d = 0.1 km, 0.50 km, 1.00 km, 2.00 km, 3.00 km, and 4.00 km are 8, 15, 26, 35, 49, and 75 years, respectively.

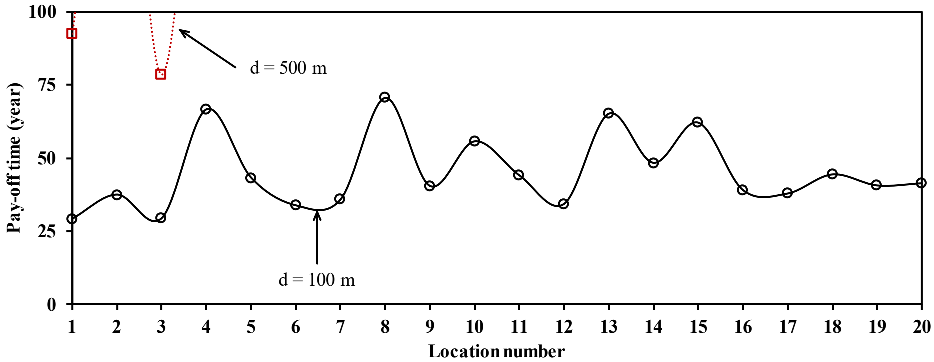
Moreover, the impact of coastal cities to the $t_{pay\text{-}off}$ for stainless steel reinforcement economical viability is significantly observed. When d = 0.1 km, the effect of coastal cities on $t_{pay\text{-}off}$ is marginal. As shown in Fig. 5.10, when d = 0.1 km, the $t_{pay\text{-}off}$ are almost identical. For d = 0.1 km, $t_{pay\text{-}off}$ for SS rebar with 5% increased initial cost in City no. 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, and 20 is 8.67, 10.31, 8.71, 14.14, 11.29, 9.53, 9.98, 14.71, 10.72, 13.20, 11.20, 9.71, 14.20, 11.93, 13.64, 10.48, 10.29, 11.39, 10.80, and 10.86 year, respectively. However, with the increasing distances from the coastline, the variation in $t_{pay\text{-}off}$ among the twenty cities are significant. For d = 0.5 km, $t_{pay\text{-}off}$ for SS rebar with 5% increased initial cost in City no. 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17,



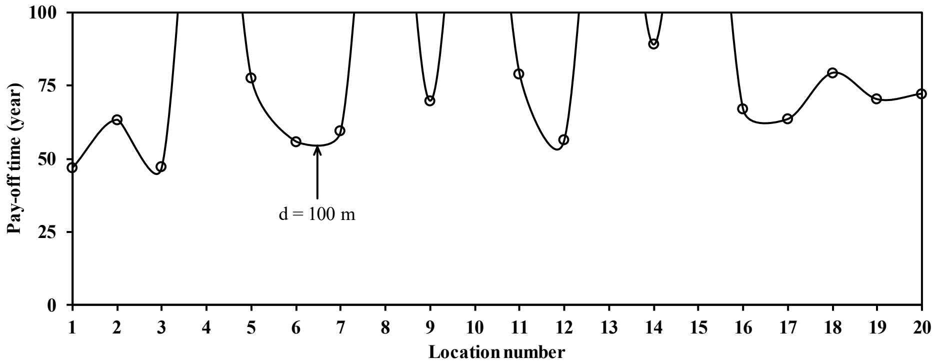
(a) SS-RC bridge with 5% increased construction cost.



(b) SS-RC bridge with 10% increased construction cost.



(c) SS-RC bridge with 15% increased construction cost.



(d) SS-RC bridge with 20% increased construction cost.

Fig. 5.10 Variation of pay-off time with location number at various locations from the shoreline for SS-RC bridge that increase the initial cost: (a) 5%, (b) 10%, (c) 15%, and (d) 20%.

18, 19, and 20 is 16.41, 19.73, 15.28, 31.06, 20.63, 18.55, 19.94, 38.30, 21.92, 24.39, 19.77, 21.38, 30.60, 26.12, 29.58, 20.81, 20.60, 20.98, 20.46, and 23.71 year, respectively. For d = 0.1 km, 0.50 km, in all cities, $t_{pay\text{-}off}$ is observed. However, for d = 1.00 km, the $t_{pay\text{-}off}$ for SS rebar in city is not found. This could be due the lower airborne chloride hazard that require more time than the design life of the bridge to provide identical life-cycle cost for CS- and SS-RC bridge. For d = 4.00 km, only in three cities (i.e., 1, 2, 6), SS rebar with $1.05C_C$ will be cost-effective over CS rebar. In remaining cities, SS rebar will not be cost effective.

Furthermore, the effect of SS rebar price on the pay-off time period is illustrated in Figs. 10(a-d). As shown these figures, the pay-off time for stainless steel economic viability significantly increases with rise in cost of SS rebar. For example, for city 6 (Niigata town), at d = 100m, the pay-off time for SS-RC bridge with 5%, 10%, 15% and 20% is 9.53 year, 19.84 year, 33.69 year, and 55.82 year, respectively.

Chapter 6. CONCLUSIONS & FUTURE WORKS

6.1 GENERAL

The chapter explores the potential of stainless steel as a sustainable alternative for carbon steel rebar subjected to traffic load hazard constructed in the coastal reinforced concrete bridges. The study proposed novel framework that can help in computing the pay-off time for stainless steel reinforcement which is crucial for selecting the design span of the bridge. Based on this study, a computation guideline in an illustrative bridge example is presented herein. Furthermore, sensitivity of price of SS rebar, coastal distance, and geographical location on pay-off time is also investigated.

6.2 CONCLUSIONS

The aim of this research was to compute pay-off period for stainless steel rebars cost-effectiveness over ordinary reinforcement applied in reinforced concrete bridges under traffic hazard in coastal marine environment. In this research study, a modelling exercise is undertaken to analyse effects of considering RC girder constructed at various location from the seashore in twenty coastal Japanese cities. Both type of rebars are considered to be used in the RC bridges. After that failure possibility of durability and final limit states were investigated. The probabilistic life cycle cost analysis of the both bridge cases were performed to evaluate the pay-off time period for stainless steel cost-effectiveness. The following conclusions are reached.

i) The pay-off time period ($t_{pay\text{-}off}$) for stainless steel reinforcements strongly rely on RC bridge's location. Near the coastline, the pay-off time period for SS rebar is small which indicates that degree of cost-effectiveness of SS rebar is remarkable. While, with the increasing distance from the coastline, $t_{pay\text{-}off}$ also increases.

Therefore, the economical viability of stainless steel reinforcement decreases with increase of shore distance.

ii) Two holistic flowcharts were proposed to assess the failure probability and probabilistic life-cycle cost of RC bridges constructed using stainless steel (SS) and carbon steel (CS) rebars.

iii) The $t_{pay\text{-}off}$ for SS rebar economic viability strongly depends on the coastal cities. In a coastal city with high wind speed, the $t_{pay\text{-}off}$ is smaller compared to city with relatively low wind speed at identical distances.

iv) The price of SS rebar has remarkable effect on $t_{pay\text{-}off}$. When the cost of stainless steel reinforcement is marginally larger compared to ordinary rebar, $t_{pay\text{-}off}$ is lower indicating that after few years of construction, the SS rebar becomes economically feasible over CS rebar.

Based on this research, it is confirmed that the pay-off time period for stainless steel reinforcement's economic feasibility depends on several factors. Therefore, before selecting SS rebar application, a reliable assessment should be performed to evaluate the cost-effectiveness of SS rebar.

6.3 FUTURE WORKS

In order to ascertain the payoff period for stainless steel cost-effectiveness at varying locations from the shoreline in twenty Japanese town, this study offered probabilistic LCC of reinforced concrete under numerous threats. Nevertheless, identical mechanical hazard is considered in all locations. Therefore, more studies on the LCC analysis of RC bridges should be conducted, and the following recommendations are made for future research.

a) It is possible to calculate the pay-off time period for SS rebar cost-effectiveness by taking into account actual traffic data because the pay-off time period is also influenced by the traffic danger.

b) The airborne chloride hazard data were collected at different cities in Japan. Since, the reliable airborne chloride hazard data for the coastal cities in Bangladesh is not available, hence it was not possible to perform this kind of research for the reinforced concrete bridges constructed in Bangladesh. It should be emphasized to collect airborne chloride hazard data in the coastal cities in Bangladesh to investigate their effects on the performance of RC bridges.

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