Bridge Failures Case Studies in India
(2016-2019)

By

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Prof. Pier Giorgio MALERBA, Supervisor

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I would like to thank all the professors for their continual support of my academic advancement throughout the graduate program at Politecnico di Milano.

I would like to thank my parents whose love and guidance are with me in whatever I pursue. They are the ultimate role models. I would like to sincerely thank my brother and sisters, for their encouragement through my formal educational endeavors. I feel happy to be the First graduate in my heritage.

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ABSTRACT

When discussing about the infrastructure in India, and more specifically, the problems facing India's infrastructure, bridge failures have been one of the leading problems facing India's infrastructure.

Bridge failures often are costly in the commerce foregone, lives lost, and replacement funds required to rebuild the failed bridge.

Infrastructure is the growth driver of the economy. India will invest as much as Rs 5.97 trillion (USD 84.84 Billion) in creating and upgrading infrastructure in the next financial year, finance minister Arun Jaitley said in his budget. The budgetary allocation for the Ministry of Road Transport and Highways has been increased to Rs 71,000 crore for 2018-19 from Rs 64,900 crore in the ongoing fiscal.

Experience can be an expensive teacher, and it is usually the best teacher. In reviewing the past bridge failures, it is natural, or manmade that with greatest loss of life and directly affects the individuals as well as country economy. From this point it is necessary to study the past failure to strengthen the future construction.

There are many failures which occur and are never heard about by the general public. They may be listed and described. Some may be settled out of court with respect to responsibility, or the person responsibly chooses to keep the failure secret to protect his/company reputation. Many failures are learned through technical papers and magazines, particularly failures of a minor nature. Thus, only technical people become aware of them, although the technical person must realize that all types of past failures must be given equal considerations.

Ongoing studies of following case study about bridge failures has been an important undertaking, which can greatly enhance the ability for engineers to predict and avoid the great costs associated with a bridge failure.

A large part of the technical knowledge connected with the bridge engineering today is based on the past failures of bridges. The purpose of the study is to make clear idea about past bridge failures and its summary to future awareness. The intent is to eliminate error in design, Maintenance and proper communication. At the same time 100% accuracy is not possible but we can minimize the failure which we can.

The findings of this Thesis will help better understanding which precautions should be taken while designing, constructing, and maintaining a bridge, as well as the factors that can help contribute to bridge failure, both distress and collapse.

The scope of the study is to investigate the causes of failure of bridges and factors that greatly increase the probability of these bridges failing by summarizing a bridge failure database of bridges within the Indian States. This thesis will also discuss these bridge failures and the lessons that can be learned as a result of them.

The goal of this thesis work is to learn from failures. This study will be conducted to help analyze why bridges fail and minimize the failures.

This Thesis is written in the hope that by contributing to the better understanding of cause of failures and a knowledge of bridge failures and types of bridge failures in the past, the future failures of this type may be reduced greatly.

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CHAPTER INTRODUCTION

1.1 General Introduction

India is emerging to become a developed country equivalent to other developed countries. The main thrust to achieve is the infrastructure of the country. The road network including the bridges connecting the shores across the many rivers across the nation, are the veins to the economic development which in turn contributes to the infrastructure. India follows the United States of America in close heels with the second largest road network in the world.

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There are many failures which occur and are never heard about by the general public. They may be listed and described. Some may be settled out of court with respect to responsibility, or the person responsibly chooses to keep the failure secret to protect his/company reputation. Many failures are learned through technical papers and magazines, particularly failures of a minor nature. Thus, only technical people become aware of them, although the technical person must realize that all types of past failures must be given equal considerations.

The bridge failures summarized in this report are grouped under headings listing the type of failure. It is unfortunate that there is one cause of failure which exists probably more often than any other carelessness during construction which is an error which may always be present. Ignorance, however, may be a factor and there are times when the economics of the construction sacrifices many lives. The most critical period in the, life of a structure is often during the construction period. There is a critical stage during construction, and after this stage the engineer can partially relax and be satisfied that his design is stable. Of course, there are other tests which the structure must also face during its early performance. But after construction is satisfactorily completed, a very Large part of the battle is won. Failures resulting not from insufficiencies of the structural design of the completed work but from unexpected movements and loadings during construction are, in the pub lie mind, not distinguished from structural design failures. Such incidents occur quite often near the completion of a job when progress is at the maximum scheduled rate and manpower is not enough to provide all the necessary precautions against failure.

1.2 Intent of Study

A large part of the technical knowledge connected with the bridge engineering today is based on the past failures of bridges. The purpose of the study is to make clear idea about past bridge failures and its summary to future awareness. The intent is to eliminate error in design, Maintenance and proper communication. At the same time 100% accuracy is not possible but we can minimize the failure which we can.

1.3 Objectives

Lessons have been learned from many failures and will yet continue to be learned. By giving an outline from the past failures of bridges and the cause of the failures, hoped that the same type of failures will not occur. It is, However, most of the failures are caused by same effect. All the Indian states are considered for past bridge failures and west of Indian states are needing to be considered when compared with the south Indian states.

The goal of this thesis work is to learn from failures. This study will be conducted to help analyze why bridges fail and minimize the failures.

This Thesis is written in the hope that by contributing to the better understanding of cause of failures and a knowledge of bridge failures and types of bridge failures in the past, the future failures of this type may be reduced greatly.

1.4 Scope

The scope of the study is to investigate the causes of failure of bridges and factors that greatly increase the probability of these bridges failing by summarizing a bridge failure database of bridges within the Indian States. This thesis will also discuss these bridge failures and the lessons that can be learned as a result of them.

1.5 Research Benefits

The damaged bridges are studied keenly, and their cause of the failure is studied. During my search there is no such topic like bridge failures case study in India. But a case study about bridge or Bridge failures Indian States or Particular Cause of failure is available. From the search, it is noted that insufficient in the field of case study for both major and minor bridges failures. It is also one of the reasons for inadequate knowledge in this filed. The findings of this Thesis will help better understanding which precautions should be taken while designing, constructing, and maintaining a bridge, as well as the factors that can help contribute to bridge failure, both distress and collapse.

1.6 Legal Responsibilities

There is a well-known phrase -- "A medica1 doctor buries his mistakes, an architect covers his mistakes with ivy, and the engineer must write a long report on his mistake."

Every time a failure occurs a single authority should take full responsibility for the lives and damages due to failures instead of fingering someone. It is not the intension of this summary to figure anyone, but only to intense the reader with the failures in some details along with the causes and lessons learned. It is the crucial requirement for the government to take notice importance of this situation and take necessary steps towards rectifying the issues. In present the administration of the national highways, state highways, and minor roads in the country are vested with different authorities.

If the administration is vested with one central authority then not only will it help in expediting the construction of roads and bridges and refurbishment of the old ones, but the quality will also improve as there would be no blame game with one administration accountable and responsible for the output.

1.7 Conclusion

The above-mentioned principle cause of failures is the primary cause of a Bridge failures, but there are the results of the two subcategories namely enabling and triggering causes. concurrently. Enabling and triggering causes are the means by which a bridge can fail, where enabling causes are generally internal to the bridge structure and triggering causes are external to the bridge; for an example enabling cause and a triggering cause could be inspection errors and tornado damage respectively. These causes can create a situation, where a total collapse or a partial collapse is probable; a total collapse is the situation where the traffic is no longer serviceable. Bridge Scour is one of the major failures of about 60 percent of total failure. Most of the bridge scour failure due to inadequate maintenances and complied database is listed in table. Second most failures occur during construction process due to improper management and in construction sequence and poor communication. In this work we discussed about this kind of failures in the section buckling failures.

In this thesis work the summary of the most common types of failures are presented. Each bridge failures due to different types of causes and some are major, and some are minor. From the case study it is clearly known that the failures are due to structural deficiency, improper management, Design error, Construction Sequence issues and careless. It is highly recommended to create a group of members responsible for the bridges, one authority and one responsible.

From the search, it is noted that insufficient in the field of case study for both major and minor bridges failures. It is also one of the reasons for inadequate knowledge in this filed. Engineers are not only responsible for design, construction…etc. but also for creating and shaping our society and improve the way we work and live. They should aware the responsibilities connected to the society and people’s life. The minimum qualification for doing practicing should be master’s degree is my own suggestion from this case study. There is a lag in this field of study, and I would like to invite future scholar and students to explore this filed and it greatly reduce the failures in India.

CHAPTER 2: BACKGROUND INFORMATION

2.1 Introduction

Engineers have studied many bridge failures, seeking to learn from them and the reasons for the structure's demise, hoping to gain insight and avoid costly mistakes in the future.

When a bridge experiences failure, a cause of failure is usually reported as to why the bridge failed, which is a principal cause; of this principal cause, there are two subcategories of failure causes which are enabling and triggering causes.

From these causes, a bridge may experience either partial collapse or total collapse, which are both forms of bridge failure.

A principal cause can be broken into two distinctly different causes of failure; enabling causes and triggering causes. These are both subcategories of failure.

The listed bridge failures occur after year 2000

Bridge	Type	Year, Cause	Casualties	Sources
Bhagalpur Pedestrian bridge (Bihar)	Arch bridge	collapsed onto a railway train as it was passing underneath Due to unstable arch	At least 30 killed	The Hindu News
Kota Chambal Bridge (Rajasthan)	Cable stayed Bridge	During Construction, the construction sequence was not followed	Claimed 48 lives	Reported by Road transport and highways ministry
Kadalundi River rail bridge (Kerala)	Girder Bridge	Scouring, Pier Unstable (Repair needed)	At least 57 killed	Reported by government investigators, Outlook India, Mapofindia.
Majerhat Bridge collapse (West Bengal)	RCC Girder Bridge	Mid-span failure of RCC Girder, (Repair required)	AT least 3 Killed	Reported by RVNL chief project manager
Vivekananda Flyover Bridge (West Bengal)	Steel girder, flyover bridge	Buckling Failure, Design error on Cantilever Beam	AT least 27 Killed	Collapse Of Kolkata Flyover-Practitioner's Perspective by N. Prabhakar & Dr. N.Subramanian
Sanvordem River Bridge (Goa)	footbridge made of steel	The bridge was closed for use but Overload a	2 Killed	India today News

		portion of Girder Collapsed		
Flyover bridge (Andhra Pradesh)	RCC Girder Bridge	Erosion of the soil, saturation and settlement But Poor Construction, Design Failure	15–30 killed	Committee by Government, commentonanything
Rafiganj rail bridge (Bihar)	Steel Girder Bridge	Terrorists sabotaged rail bridge, high-speed causing crash	130+ killed	Times of India News
Veligonda Railway Bridge (Andhra Pradesh)	Railway Bridge	Scouring Failure, flood washed rail bridge away	114 killed	Times of India News
Savitri River Bridge (Maharashtra)	Stone Arched Bridge	Dilapidated Condition. (Repair required), About 100 years old.	28 Killed	IIT Report
Bridge Collapse at Siliguri (West Bengal)	Girder Bridge	The bridge was in a dilapidated condition, (Repair required)	1 hurt	Times of India News
Charus Bridge	Deck type Truss Bridge	Buckling Failure	Unknown	Failure of chauras bridge, Harshad
Andheri Bridge	RCC Girder Bridge	Corrosion Failure	5 Injured	Economictimes. indiatimes
Jahu Bridge	RCC Girder Bridge	Scouring Failure	4 Killed	Jahu Bridge Pankaj
Langi Durg Road Bridge	RCC Girder Bridge	Scouring Failure	Unknown	Foundation failure of bridges, Narayan,2018
CAHDOORA BRIDGE	RCC Girder Bridge	Scouring Failure	Unknown	Bridge failures in extreme flood events, Azmat
Mumbai-Goa bridge	RCC Girder Bridge	Scouring Failure	2 Killed, 20 Missing	Indiatoday

Table 1 Listed bridge failures occur after year 2000

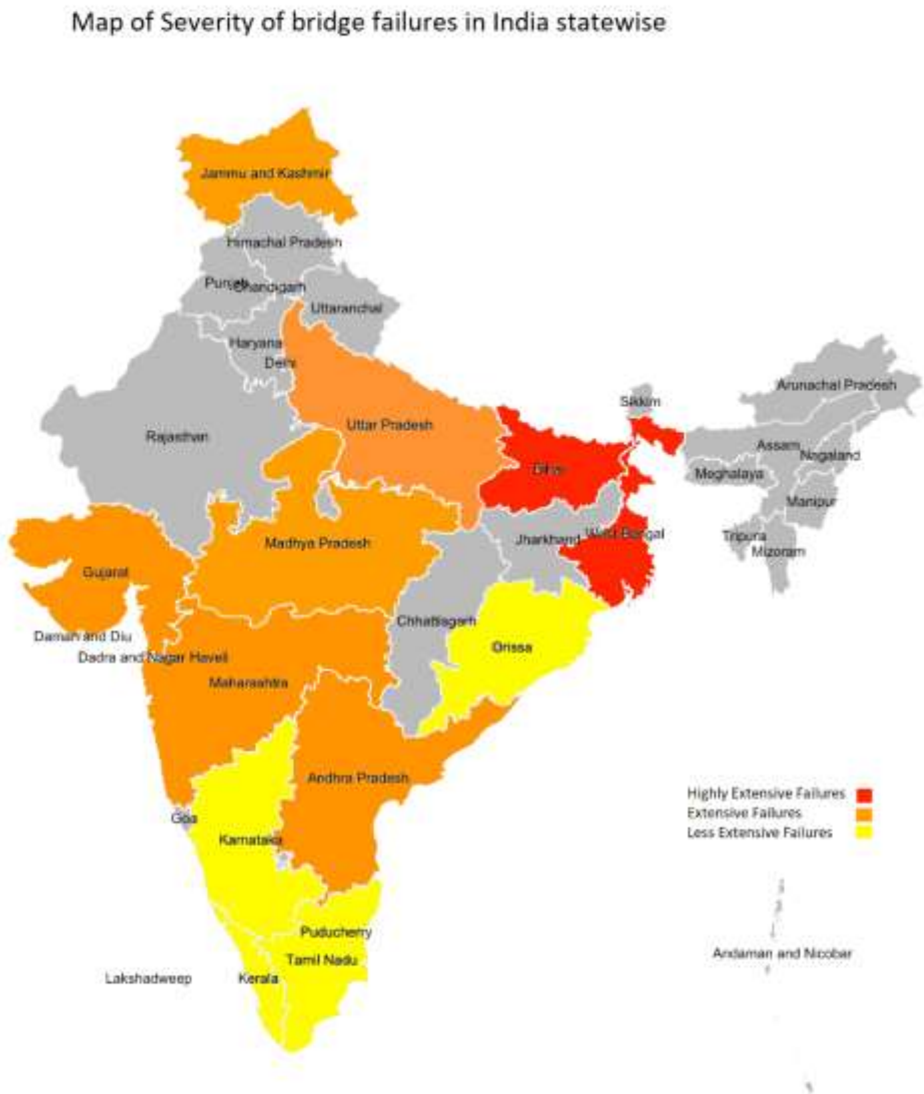

Figure 1 Map of Severity of bridge failures statewise

2.2 Principal Causes of Failure

Principal cause of failure for this study are defined as, detailing, or construction, errors in design, floods, the use of improper materials, unanticipated effects of stress concentrations, lack of proper maintenances, unexpected events, Accidents, Design flaws and manufacturing errors, Fires, Earthquakes. Unexpected events are generally referring to either severe partial collapse of the bridge, which would require partial or total replacement of the bridge components. Cause of failure can be any or combination of the above all, which could bring the bridge down catastrophically or induce distress into the structure.

Of all principal causes of failure, Wardhana and Hadipriono (2003) found that unintended external events were found to contribute the highest amount of bridge failures in past analysis of bridge failures, overall, followed secondly by maintenance issues. The study found external causes to lead to the most collapses, followed by maintenance and during construction.

2.3 Enabling Causes of Failure

An enabling cause as any issue with the bridge that can be identified as an internal weakness or deficiency that leaves the structure vulnerable to failure in their most recent study on bridge failures (Wardhana and Hadipriono 2003).

This can be due to many reasons such as material deficiencies, construction faults, design error or anything internal to the structure which can lead to failures. It can be prevented by many methods, but it may be hard to catch before the problem has been made known through observed defects construction or in progress. It can be avoided by proper communication between designer and supervisor, following standard quality materials, increased care and diligence in both the design and construction phases.

Symbol	Enabling Causes
E1	Construction Issues
E2	Design Issues
E3	Maintenance Issues
E4	Material Issues

Table 2 Enabling Causes (Wardhana and Hadipriono 2003)

2.4 Triggering Causes of Failure

Triggering causes are those which are external to the bridge. These are usually hard to predict and are much more wide-ranging than enabling causes, and can include: wind, hurricanes, flooding, terrorism, and any other external cause (Wardhana and Hadipriono 2003).

This type of causes is more difficult to predict and much more likely to result in full or partial collapse. Because since that are externally controlled or the impact of external action but must be accounted for during the design phase as accurately as possible, using factors of safety while avoiding overdesign of the bridge.

Symbol	Triggering Cause
T1	Hydraulic – scour/Flood
T2	Deterioration
T3	Detailing
T4	Collision
T5	Overload

Table 3 Triggering Causes (Wardhana and Hadipriono 2003)

2.5 Collapse

In a bridge's service to the traveling public, collapse is often an engineer's greatest fear, but needs to be discussed when studying failure. Two types of collapse will be investigated in the study; total collapse and partial collapse. Partial collapse refers to an incident where the bridge undergoes some deformation or section loss but still can remain serviceable, whereas total collapse refers to an incident where the bridge is unable to service traffic flows. An example for each would be locked bearings and pier collapse, respectfully.

2.6 Conclusion

The above-mentioned principle cause of failures is the primary cause of a Bridge failures, but there are the results of the two subcategories namely enabling and triggering causes. concurrently. Enabling and triggering causes are the means by which a bridge can fail, where enabling causes are generally internal to the bridge structure and triggering causes are external to the bridge; for an example enabling cause and a triggering cause could be inspection errors and tornado damage respectively. These causes can create a situation, a total collapse or a partial collapse is probable; a total collapse where traffic is no longer serviceable. Bridge Scour is a major failure of about 60 percent of total failure.

In this thesis work the summary of the most common types of failures are presented. Each bridge failures due to different types of causes and some are major, and some are minor. From the case study it is clearly known that the failures are due to structural deficiency, improper management, Design error, Construction Sequence issues and careless. It is highly recommended to create a group of members responsible for the bridges, one authority and one responsible. From the search, it is noted that insufficient in the field of case study for both major and minor bridges failures. It is also one of the reasons for inadequate knowledge in this filed. Engineers are not only responsible for design, construction…etc. but also for creating and shaping our society and improve the way we work and live. They should aware the responsibilities connected to the society and people's life. The minimum qualification for doing practicing should be master's degree.

CHAPTER 3. BRIDGE SCOUR

3.1 Introduction

The flow of water in rivers and streams excavates and moves material from bed and banks of streams and from around the bridge piers and abutments. Correspondingly, foundations of the structures are undermined by this erosive action of the flowing water, which is named as scour. Past observations show that scouring is a noteworthy problem in bridge hydraulics topic. When hydraulic and structural interaction is not evaluated accurately, during the high floods scour can give rise to destruction of structure, loss of life and property.

The addition of these components presents the total scour. The types of scour that can occur at a bridge are illustrated in Figure. It is assumed that each of them occurs independently.

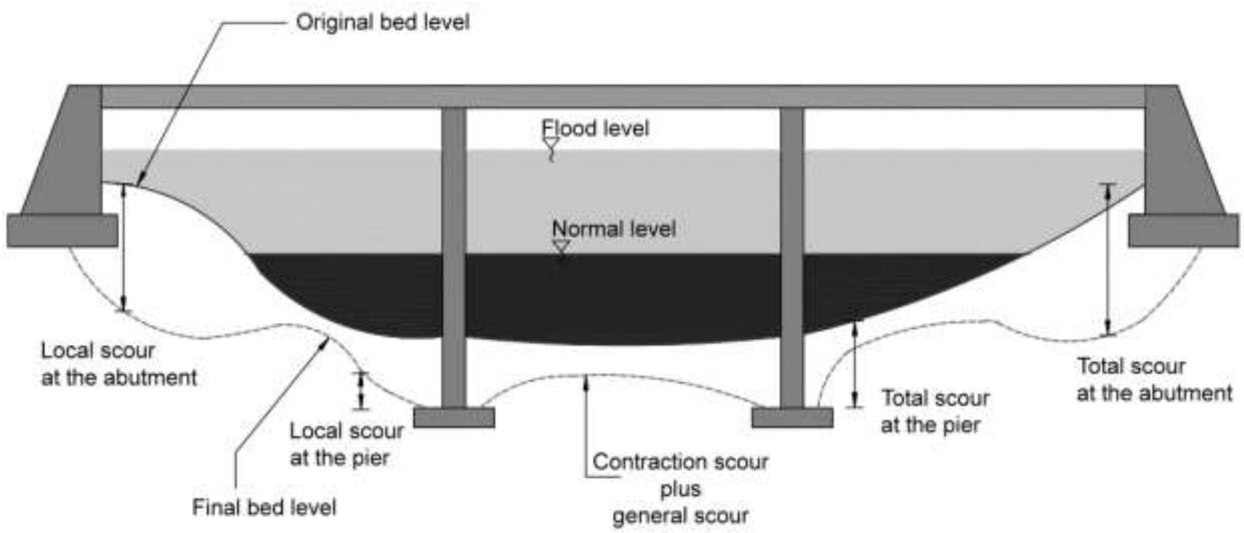

Figure 2 The types of scour that can occur at a bridge (Melville and Coleman, 2000)

3.1.1 General Scour

General scour forms as a result of natural processes whether there is a bridge or not at the cross section. It can be referred as bed aggradation/degradation and categorized as short-term scour and long-term scour according to the time it takes to reach the scour. Short-term general scour occurs during single or sequential floods (daily, weekly, monthly or seasonally). Scour at channel confluences, scour at bends, scour arising from a shift in the channel thalweg and bed-form migration are included in short-term general scour (Coleman and Melville, 2001). Because formation of contraction scour and local scour dominate over that of short-term scour, it is very hard to anticipate it at structure, hence short-term scour is not included in the design computations. Long-term general scour forms naturally or develops with some modifications at watershed and stream and undoubtedly occurs over the years and has a relatively longer time scale. Human causes of long-term general scour are channel alterations, streambed mining and dam/reservoir construction. In addition; channel straightening, tectonic activities, fire and climate change develop long-term scour naturally. The engineer has to specify the present condition of stream and watershed and estimate the future streambed changes (Melville and Coleman, 2000).

3.1.2 Contraction Scour

Contraction scour is the result of reduction in the flow area of a stream either by natural contraction i.e. this can occur naturally between rock outcrops preventing the stream from migrating or by a bridge or when overbank flow is confined by roadway embankments. From the continuity principle, flow area inversely proportional to the average velocity and bed shear stress. From this, increase in erosive forces leads to more bed material removed from the contracted section than transported to the section. Bed elevation is lowered, average velocity and bed stress decrease at the reach until the equilibrium is reached. The amount of the bed material removed from the reach becomes equal to that of bed material transported to the reach.

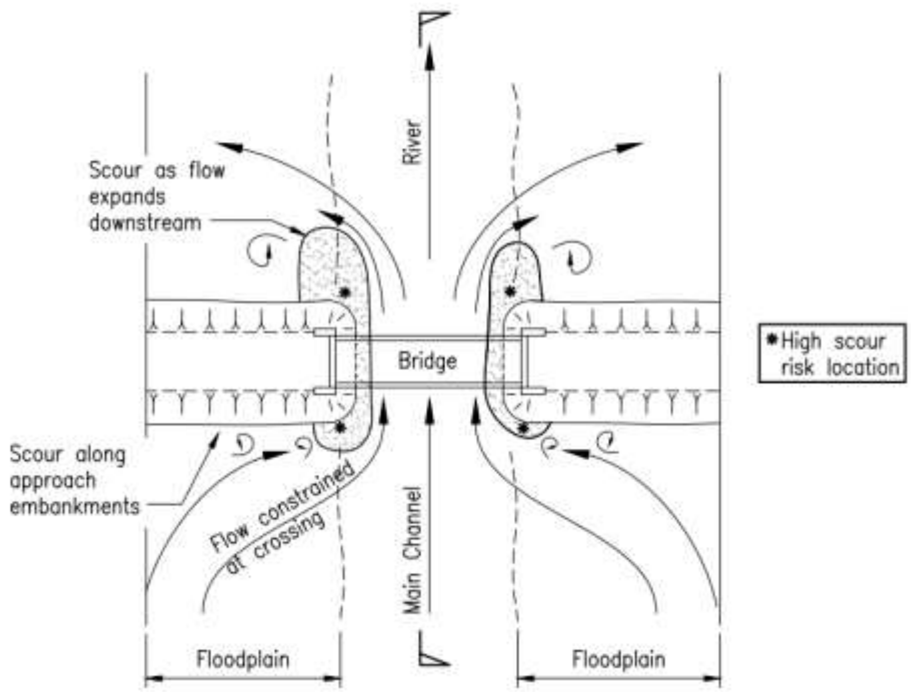

Figure 3 Contraction scour and high-risk locations (State of Queensland -Bridge scour manual 2013)

3.1.3 Scour in Different Conditions of Transport

In bridge hydraulics,

Bed materials transportation is considering to be the greater effect because the foundation design of the river structure is affected by it. According to the conditions of transport, Clear-water scour, and live-bed scour are divided, which will be explained in following sections. Variation of scour depth under clear water scour and live bed scour conditions as a function of time is shown below,

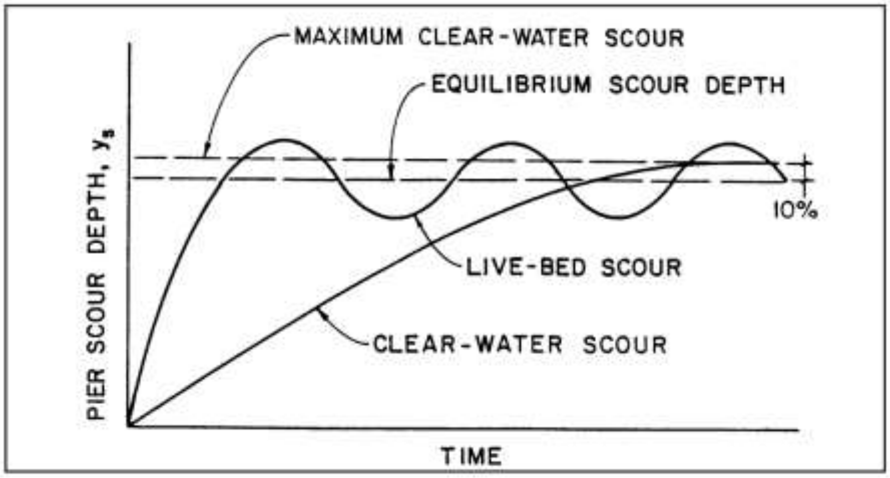

Figure 4 Variation of scour depth under clear-water scour and live bed scour conditions as a function of time (State of Queensland - Bridge scour manual 2013)

3.1.4 Clear-Water Scour

Clear-water scour occurs when the bed material at the upstream is not transported to the downstream reach or the bed material at the upstream of the bridge structure has no motion. In this condition, the velocity of the river is less than the critical velocity of the bed material in the river (i.e. V/v_c <1). The shear stress created by flow of water at the bed should be smaller than the critical shear stress. The maximum local scour depth is reached when the flow can no longer remove bed material from the scour area.

The flow properties in scour hole are affected by pier geometry and inertia of flow. Under clear-water conditions, the area of the contracted section increases until, in the limit, the velocity of the flow (V) or the shear stress (τ_0) on the bed is equal to the critical velocity (v_c) or the critical shear stress (τ_c) of a certain particle size (D) in the bed material.

Past researchers have studied the hydraulic conditions necessary for the start of clear-water scour. Shen et al. (1969) recommends that clear-water scour starts when the mean approach velocity (v_0) reaches half of the critical shear velocity value (v_{*c}) . Raudkivi (1986) proposes a different relationship, which tells that the clear-water scour begins when mean flow velocity (V) approaches 50% of mean critical velocity (v_c). Additionally, Chiew (1995) states that shear velocity (v_*) must be at least 30% of the critical shear velocity (v_{*c}) to initiate the clear-water scour.

Critical Velocity $\qquad v_c = 6.19 y_{ms}^{1/6} D_{50}^{1/3}$

The recommended clear-water contraction scour equation is:

$$(y_{ms})_c = \left[\frac{0.025Q^2}{D_m^{2/3} W^2}\right]^{3/7}$$

Clear Water Equation HEC-18

V/v_c <1 $$y_s = (y_{ms})_c - y_0$$

$$D_m = 1.25 D_{50}.$$

3.1.5 Live Bed Scour

Bed material sedimentation transport occurs from the upstream reach into the bridge cross section. When flow intensity increases until bed shear stresses at the upstream exceed the critical shear stress, bed load transport in the flow direction is developed. The stream velocity is higher than the critical velocity of the bed materials (i.e. $\mathrm{V}/v_c > 1$).

Live-bed contraction scour depths restrained by large sediment particles in the bed material. Under this condition we can calculate depth of live bed scour using the smaller depth between live bed scour and clear water scour equation. Development of scour hole depends on the flow in the scour hole and flow conditions in the upstream together. Scour hole development rate rapidly increases at first, then decreases in time.

Scour with the bed material sediment transport (live bed scour) takes place when flow initiates the general sediment transport by the river. When flow intensity increases until bed shear stresses at the upstream exceed the critical shear stress, bed load transport in the flow direction is developed. The equilibrium condition is reached when the rate of sediment entering the scour hole becomes equal to the rate being taken out.

Different from the clear-water conditions, under live bed conditions, an average equilibrium scour depth, an average maximum scour depth and an average minimum equilibrium scour depth are defined (Raudkivi 1991). Variation of scour depths with mean approach velocity for clear-water and live bed conditions.

A modified version of Laursen's 1960 equation for live-bed scour at a long contraction recommended to predict the depth of scour in a contracted section (Laursen 1960). The e assumes that bed material is being transported from the upstream section.

Live-Bed Conditions	$0.59 < k_1 > 0.69$
$\mathrm{V}/v_c \geq 1$	mostly contact-be transport to mostly
Richardso 1995 modified	suspended-bed material transport
from Laursen 1960	

$$\frac{(y_{ms})_c}{y_{ms}} = \left[\frac{Q_2}{Q_{1m}}\right]^{6/7} \left[\frac{W_1}{W_2}\right]^{k_1}$$

$$d_s = y_0 - (y_{ms})_c$$

Where,

$(y_{ms})_c =$ Average equilibrium depth in the contracted section after contraction scour, (m)

y_{ms} = Average depth in the upstream main channel, (m)

Q_2 = Total flow rate through the contracted section, m^3/s, $Q_2 = Q_{1m}$ X % area open through bridge

Q_{1m} = flow rate in the approach main channel transporting sediment m^3/s

W_1 = Bottom width of the approach channel, m

W_2 = Bottom width of the contracted channel, m

W = Bottom width of the contracted channel less pier width (s), m

$$k_1 = \begin{cases} Exponent = 0.59 \; if \; V*/w < 0.5 \; (mostly\ contact - bed\ transport\ to\ mostly\ suspended - bed\ material\ transport \\ Exponent = 0.64 \; if \; V*/w = 0.5 - 2.0 \; (some\ suspended - bed\ material\ transport) \\ Exponent = 0.69 \; if \; V*/w > 2.0 \; (mostly\ suspended - bed\ material\ transport) \end{cases}$$

d_s = Depth of scour, m

y_0 = Average existing depth in contracted section, (m)

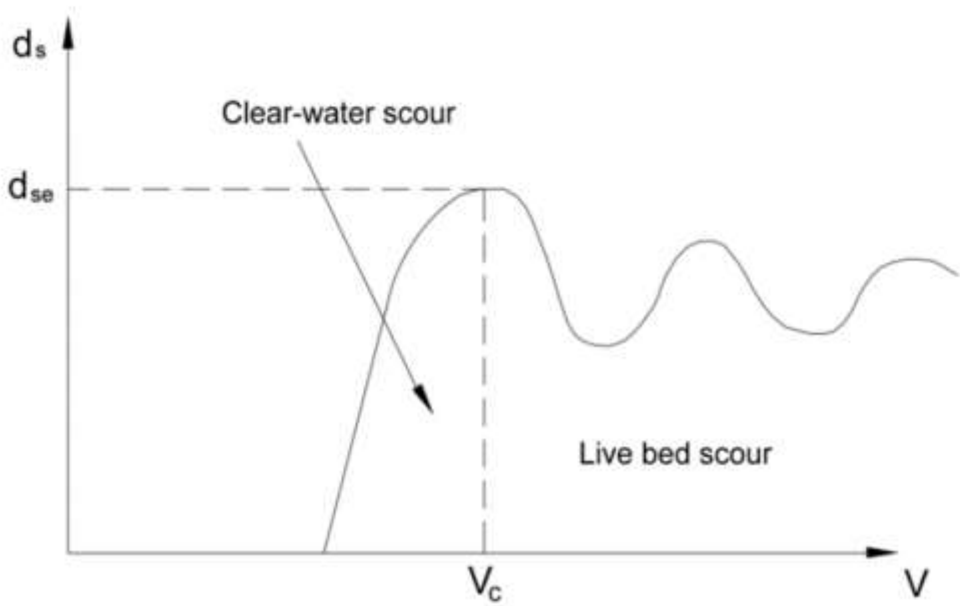

Figure 5 Variation of scour depth with approach flow velocity (Yanmaz, 2002)

3.2 Local Scour

3.2.1 Types of Scour

There are 3 main components of total scour:

1. General scour of the riverbed

2. Contraction scour at the bridge cross section

3. Local scour around bridge piers and abutments

Local scour is removal of sediments from around bridge structures like piers, abutments, spurs, and embarkments. The formation of vortices is the basic mechanism causing local scour at piers or abutments. Disturbances to fluid flows will alter the velocity and pressure distributions around and downstream of the obstruction.

The transport rate into the region is smaller than the transport rate of sediment away from the base region, and, consequently, a scour hole develops. As increases in depth of scour, the strength of the horseshoe vortex is reduced, thereby reducing the transport rate from the base region. Eventually, for live-bed local scour, equilibrium is reestablished and scouring ceases. For clear-water scour, scouring ceases when the shear stress caused by the horseshoe vortex equals the critical shear stress of the sediment particles at the bottom of the scour hole.

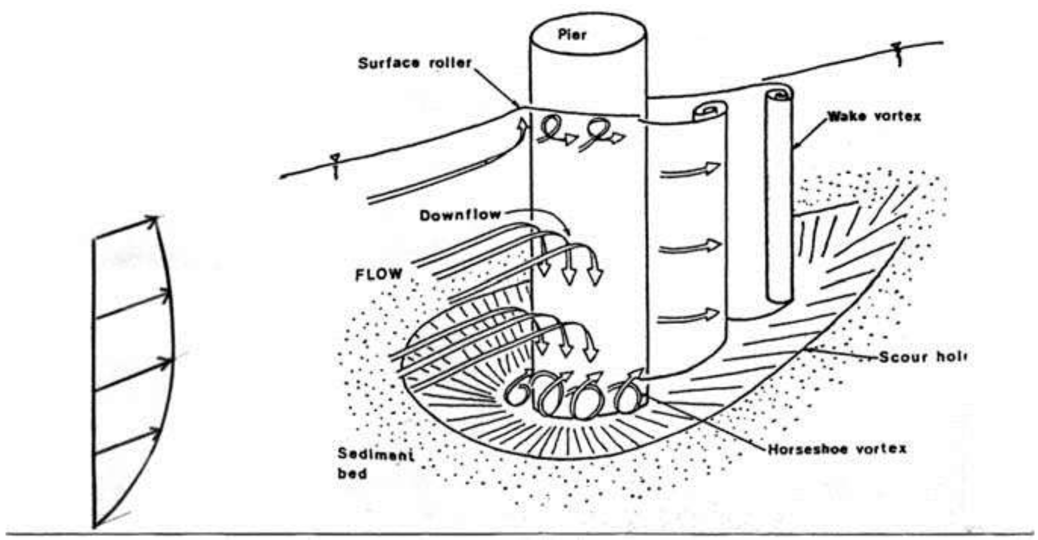

Figure 6 Scour at a bridge pier (HEC-18, 2012)

Show the processes behind localized scour at piers and abutments. Vortices form upstream and downstream of pier and abutment.

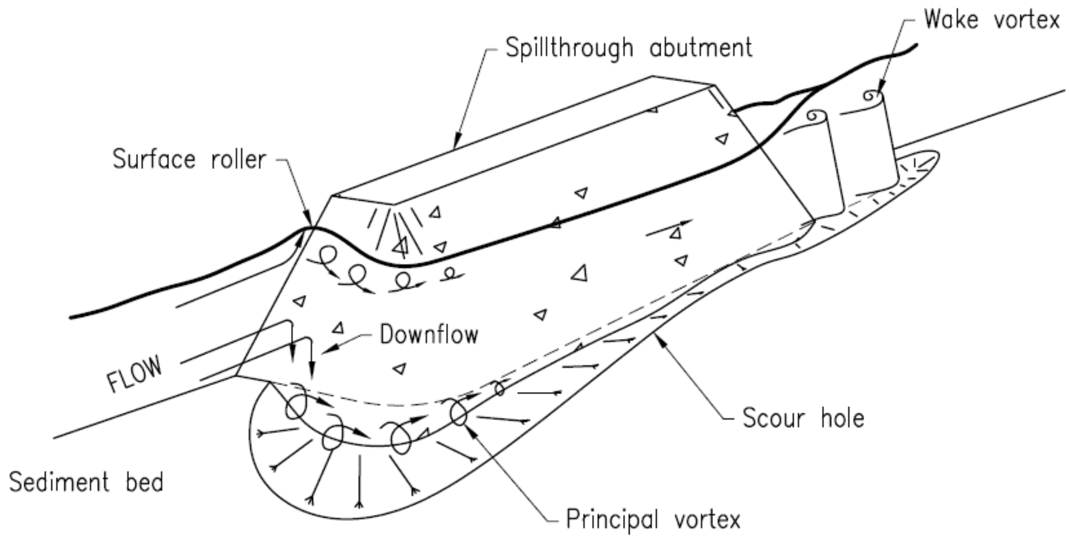

Figure 7 Scour at a bridge abutment (HEC-18, 2012)

Factors which affect the magnitude of local scour at piers and abutments are (1) width of the pier, (2) discharge intercepted by the abutment and returned to the main channel at the abutment (in laboratory flumes this discharge is a function of projected length of an abutment into the flow), (3) length of the pier if skewed to flow, (4) depth of flow, (5) velocity of the approach flow, (6) size and gradation of bed material, (7) angle of attack of the approach flow to a pier or abutment, (8) shape of a pier or abutment, (9) bed configuration, (10) ice formation or jams, and (11) debris

3.3 Local scour - bridge piers

The design and configuration of a bridge substructure will impact on scour development at the bridge piers and abutments. Local scour at piers can lead to severe damage to footings as shown in Figure. The shape of the piers and the footing type alter the flow pattern around the pier. While pier design is dependent on-site specific factors such as the superstructure, soil conditions and construction procedures, the pier's influence on the flow should also be considered.

Hydrodynamically shaped piers help reduce the generation of turbulent flow. Flow alignment will contribute to increased erosion. A river will respond to alterations to flow conditions through erosion until an equilibrium state is reached.

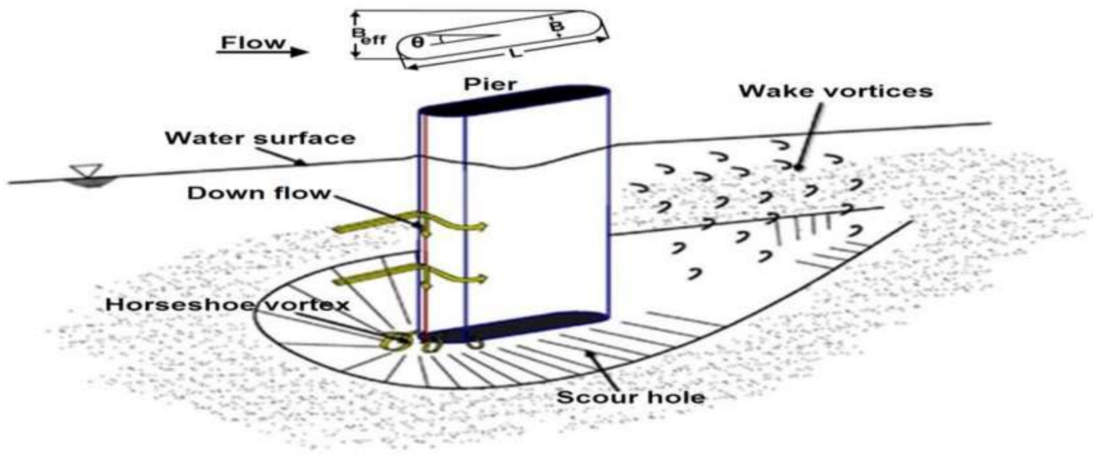

Figure 8 Schematic vortex structures around circular pier (Mojtaba Karimaei Tabarestani 2017)

3.4 Local scour - bridge abutments

Scour occurs at abutments when the abutment and roadway embankment obstruct flow. Several causes of abutment failures during post-flood field inspections of bridge sites have been documented.

These failures were due to:

overtopping of abutments or approach embankments;

lateral channel migration or stream widening processes;

contraction scour; and/or,

local scour at one or both abutments

Flow through a bridge waterway narrowed by a bridge abutment is essentially flow around a short streamwise contraction. The characteristic flow features and the link between the contraction and the formation of a complex flow field around the abutments.

Abutment scour depends on the interaction of the flow obstructed by the approach and the flow in the main channel. The discharge returned to the main channel at the abutment is not simply a function of the abutment and roadway length. Abutment scour depth depends on abutment shape, flow in the main channel, flow intercepted by the abutment and directed to the main channel, sediment characteristics, cross-sectional shape of the main channel (especially the depth of flow in the main channel and depth of the overbank flow at the abutment), and alignment. In addition, field conditions may have tree-lined or vegetated banks, low velocities, and shallow depths upstream of the abutment. Most of the early laboratory research failed to replicate these field conditions.

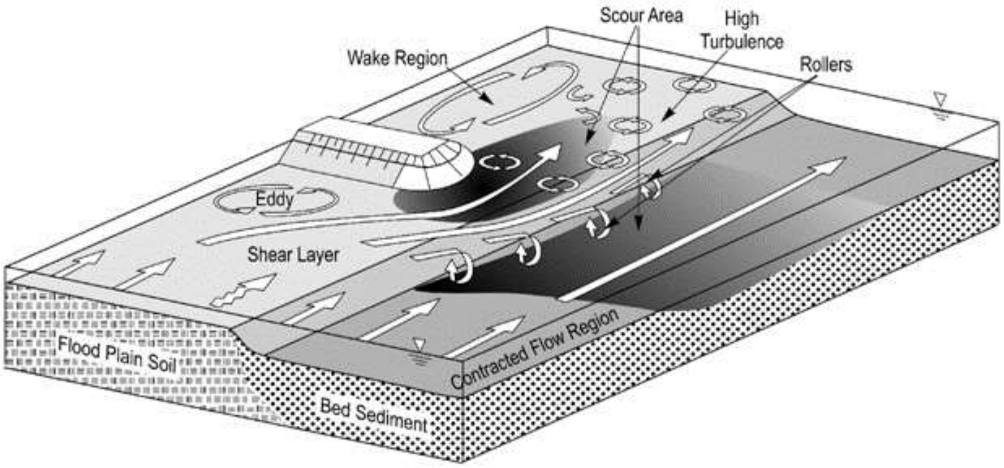

Figure 9 Flow structure generated by floodplain/main channel flow interaction (NCHRP 2011b)

3.3 LOCAL SCOUR MECHANISM AROUND BRIDGE PIERS

3.3.1 General Information

Local scour is the erosive action of accelerated flow due to the presence of an obstacle (pier) in rivers and streams. As the flow passes the pier, mean flow velocity increases and vortices are formed at piers face. The formation of this flow pattern adjacent to a cylinder causes the scour.

3.3.2 LOCAL PIER SCOUR EQUATION

To determine pier scour, an equation based on the CSU equation is recommended for both live-bed and clear-water pier scour. The equation is:

$$\frac{y_s}{y_1} = 2.0\, k_1 k_2 k_3 k_4 \left[\frac{a}{y_1}\right]^{0.65} Fr_1^{0.43}$$

As a Rule of Thumb, the maximum scour depth for round nose piers aligned with the flow is:

$y_s \leq 2.4$ times the pier width (a) for Fr ≤ 0.8

$y_s \leq 3.0$ times the pier width (a) for Fr > 0.8

In terms of y_s/a, Equation 6.1 is:

$$\frac{y_s}{a} = 2.0\, k_1 k_2 k_3 k_4 \left[\frac{a}{y_1}\right]^{0.65} Fr_1^{0.43}$$

y_s = Scour depth, m (ft)

y_1 = Flow depth directly upstream of the pier, m (ft)

k_1 = Correction factor for pier nose shape

k_2 = Correction factor for angle of attack of flow

k_3 = Correction factor for bed condition

k_4 = Correction factor for armoring by bed material size

a = Pier width, m (ft)

L = Length of pier, m (ft)

Fr_1 = Froude Number directly upstream of the pier = $V_1/\ (g* y_1)^{1/2}$

V_1 = Mean velocity of flow directly upstream of the pier, m/s (ft/s)

g = Acceleration of gravity (9.81 m/S^2) (32.2 ft/S^2)

The correction factor, K2, for angle of attack of the flow, θ, is calculated using the following equation:

K= $(\text{Cos } \theta + \text{L/a Sin } \theta)^{0.65}$.

If L/a is larger than 12, use L/a = 12 as a maximum in Equation

Notes:

1.) The correction factor K1 for pier nose shape should be determined using for angles of attack up to 5 degrees. For greater angles, K2 dominates and K1 should be considered as 1.0. If L/a is larger than 12, use the values for L/a = 12 as a maximum.

2.) The values of the correction factor K2 should be applied only when the field conditions are such that the entire length of the pier is subjected to the angle of attack of the flow. Use of this factor will result in a significant over-prediction of scour if (1) a portion of the pier is shielded from the direct impingement of the flow by an abutment or another pier; or (2) an abutment or another pier redirects the flow in a direction parallel to the pier. For such cases, judgment must be exercised to reduce the value of the K2 factor by selecting the effective length of the pier subjected to the angle of attack of the flow. Equation should be used for evaluation and design

3.) The correction factor K3 results from the fact that for plane-bed conditions, which is typical of most bridge sites for the flood frequencies employed in scour design, the maximum scour may be 10 percent greater than computed. In the unusual situation where a dune bed configuration with large dunes exists at a site during flood flow, the maximum pier scour may be 30 percent greater than the predicted equation value. This may occur on very large rivers, such as the Mississippi. For smaller streams that have a dune bed configuration at flood flow, the dunes will be smaller, and the maximum scour may be only 10 to 20 percent larger than.

4.) Piers set close to abutments (for example at the toe of a spill through abutment) must be carefully evaluated for the angle of attack and velocity of the flow coming around the abutment.

The correction factor K4 decreases scour depths for armoring of the scour hole for bed materials that have a D50 equal to or larger than 2.0 mm and D95 equal to or larger than 20 mm. The correction factor results from recent research by Molinas and Mueller. Molinass research for FHWA showed that when the approach velocity (V1) is less than the critical velocity (Vc90) of the D90 size of the bed material and there is a gradation in sizes in the bed material, the D90 will limit the scour depth.(30, 52) Mueller and Jones developed a K4 correction coefficient from a study of 384 field measurements of scour at bridges. The equation developed by Jones given in HEC-18 Third Edition should be replaced with the following:

- If $D_{50} < 2$ mm or $D_{95} < 20$ mm, then $k_4 = 1$
- If $D_{50} \geq 2$ mm and $D_{95} \geq 20$ mm

then:

$$k_4 = 0.4(V_R)^{0.15}$$

Where,

$$V_R = \frac{V_1 - V_{ic\,D_{50}}}{V_{c\,D_{50}} - V_{ic\,D_{95}}} > 0$$

and:

$V_{ic\,D_{95}}$ = the approach velocity (m/s or ft/sec) required to initiate scour at the pier for the grain size D_x (m or ft)

$$V_{ic\,D_{95}} = 0.645 \left[\frac{D_x}{a}\right]^{0.053} V_{c\,D_x}$$

$V_{c\,Dx}$ =the critical velocity (m/s or ft/s) for incipient motion for the grain size D_x (m or ft)

$V_{c\,Dx} = K_u\, y_1^{1/6} D_x^{1/3}$

$y_{1.}$ = Depth of flow just upstream of the pier, excluding local scour, m (ft)

V_1 = Velocity of the approach flow just upstream of the pier, m/s (ft/s)

D_x = Grain size for which x percent of the bed material is finer, m (ft)

K_u = 6.19 SI Units

K_u. = 11.17 English Units

While k_4 provides a good fit with the field data the velocity ratio terms are so formed that if D_{50} is held constant and D_{95} increases, the value of k_4 increases rather than decreases.(53) For field data an increase in D_{95} was always accompanied with an increase in D_{50}. The minimum value of k_4 is 0.4 and it should only be used when $V_1 < V_{ic\,D_{95}}$.

3.4 CASE 1

3.4.1 Characteristic of CAHDOORA BRIDGE

The Chadoora Bridge is located at tehsil chadoora in budgam district Kashmir, India. This bridge connects the two important tehsils. The trade, communication and public transport fully depend on this bridge. It was constructed by R and B division Kashmir in the 2001 – 2002, the bridge is a balanced cantilever bridge with Total length of the bridge is 108.3mtr. It consists of 2 Dummy abutments and Solid wall type piers RCC with circular ends having open foundation. Two Balanced Cantilever bridges each supported on two piers formed by four longitudinal precast prestressed girders with 7.5mtr carriage way.

Figure 10 Chadoora Bridge taken after floods (Azmat Hussain 2016)

The description of the substructure and superstructure of this bridge is listed below

The total length of the bridge deck system is 108.3mtrs and was supported by four piers in river section as Two Dummy abutment structures placed behind major deep drains located apart both riverbanks. The piers were solid wall type Reinforced concrete construction with circular ends having open foundation. The Dummy abutment structures are located on the bank of the river.

The bridge superstructure was consisting of 2 Balanced Cantilever type deck units each supported on two piers, formed by four longitudinal precast prestressed girders in tandem with cast in situ RCC diaphragms and deck slab forming the composite girder system. The cantilever arms of these units with articulation provision support shore span on one end and central suspended span on the other end. POT bearings are used at seating of girders of central span. At ends of the bridge deck, Strip seal movement joints are provided and at one end of the central suspended span only and all other three joints were hinged/rotation joints only for improved riding comfort.

3.4.2 Description of the collapse

The bridge is located at chadoora area on doodganga nallah, Jammu & Kashmir. The source of the doodganga Nallah is pir Panchal catchment. Bed grade of the Nallah is 1 in 39. Longitudinal slope of the Nallah is 1 in 1659. Catchment area is 300 to 630sqkm. Most common cause of failure of the bridges on river is scour. Scour is the result of the erosive action of the running water, excavating and carrying away material from stream beds and banks. Scour or soil erosion at a bridge is caused by the dynamic effects of the water in motion. The length of this bridge is 108.3metre long before flooding.

The main cause of the failure of this bridge is scour type failure. According to the reports, the discharge which was recorded earlier on 1973 was 5000 cusecs. But during the floods on 9 September 2014, it was 9504 cusecs. This high discharge is hardly a criterion for the failure of the Chadoora Bridge. The length of the bridge is 108.3mtrs, but during floods the linear waterway was recorded only 36.6mtrs which is negligible for such a major bridge. This waterway of the bridge was not enough to counteract the flood water. The flow of the water has been restricted to 36.6 meter with concentration of flow in only 27.8mtrs width while the bridge is about 108.3mtr in length with four piers in between and two side abutments. The waterway was grossly inadequate.

Debris changes both the geotechnical and hydraulic characteristics of a bridge. Debris accumulation in the bridge may result in substantial block in bridge opening, the waterway opening area is considerably reduced. With reduced waterway, velocity would increase, and greater scour depths would be involved, requiring deeper foundations. But after constructing the bridge, these things became negligible and lead to the failure of the bridges. Scouring around the foundation was taken place removed the fines from sand, gravel, boulder matrix. The scouring was predominant on upstream side and slightly less on the downstream side. That is why the pier first tilted towards upstream side and settled by about 30 – 40 cm. The scouring of the bridge has also taken place due to massive slides on river side of left abutment. The obvious cause of the scouring has been encroachment of almost 75% of total waterway, both on upstream and downstream side by the way of the creation of the eidgah and sumo stand respectively. The width of the eidgah and sumo stand is 71.7 mtrs. So, the total waterway was 36.6 meter only. This cause the scour of the pier.

Figure 11 The detailed site plan of the bridge (Azmat Hussain & Sumaira Jan 2016)

3.4.3 Lesson

The discharge of 9504 cusecs which should be pass through at least 91.5mtr width, only passed through 36.6m, thereby causing the scour failure of the bridge.

The construction of the shed, taxi stand etc reduced the waterway of the bridge and thereby not giving free and full passage of the flood to pass which lead to scour failure of the bridge.

3.4.4 Summary

By analyzing this failure in the bridges, the future planning can be better. Therefore, the aim should be to design the bridges for all times and for all occasions. Foundation of new bridges, bridges to be widened, replaced shall be designed to resist the scour for 100-year flood criteria, which may create the deepest scour at foundations.

3.5 CASE 2

3.5.1 Characteristic of LANGI DURG ROAD BRIDGE

This bridge IS situated m Km. 26/6 on Lang Uurg Road near Balaghat district of state of Madhya Pradesh. India. Total length of the bridge is 30.50 mtr. It consists of 2 spans of 15.25 m. c/c. R. C. C. solid slab. C.R. Masonry in cement mortar 1:4 Open foundation in cement concrete 1:2:4. Two Balanced Cantilever bridges each supported on two piers formed by four longitudinal precast prestressed girders with 7.5mtr carriage way with 0.45 wide kerb.

The bridge is situated in Km. 26/6 on Langi Durg Road. It was constructed in the year I 980-81. This bridge has two solid slab spans of 15.25 meters each. The substructure is of solid coarse rubble stone masonry. The base dimensions of pier foundation are 3780 mm. in traffic direction and 10080 mm. in current direction at R.L. 94.00m. The pier foundation is resting on boulder strata.

Figure 12 Langi Durg Road Bridge (Pier Failure of Bridge and Geotechnical J. K. Jain 1998)

3.5.2 Description of the collapse

The foundation was resting on boulders strata about 5m below the lowest bed level. Position of pier is at the center of the stream. It is hilly track, the river flows with high velocity} and thus causing heavy scouring near the pier due to obstruction. The central portion of the stream got disturbed due to construction of pier at the center with the passage of time the binding material of boulders is taken away by the floods during rainy season. This process continued and the boulders so loosened are taken away by the floods and boulders strata below the foundation disturbed during the floods causing uneven settlement. Due to this the masonry pier collapsed. No damage or disturbance is seen in the masonry abutment and wing walls. During flood of 1995, the pier foundation unevenly settled causing collapse of masonry pier up to bed level. Due to this both the deck slab came at the bed level with no damage in them.

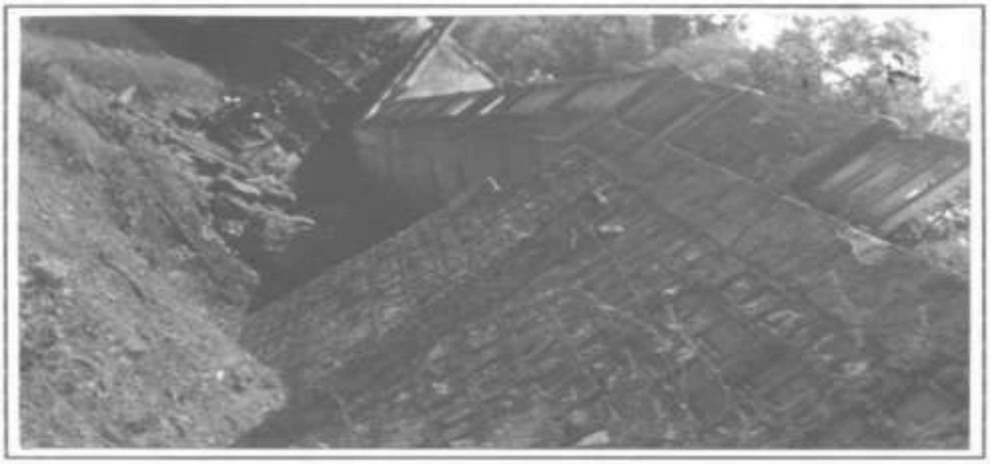

Figure 13 Showing no damage signs in abutment and wings (Pier Failure of Bridge and Geotechnical Investigation – J.K. Jain, 1998)

ii) Design Investigations

The foundation of the pier is checked for base pressure with maximum and minimum loads and calculations shows that the foundation sizes provided are adequate. The maximum base pressure at R.L. 94.00 m is 30.68 T/m2 and minimum base pressure is 8.04 T/m2. These base pressures are within maximum permissible limits and no tension on foundation base.

The pier section at R.L. 98.80 m is also checked and maximum stress about this level is 37.75 T/m2 and minimum stress is 11.44 T 1m2 in compression. No tension is observed at this level hence sections provided are adequate.

Hence the failure of pier foundation and pier is not due to the insufficient design requirements

The investigation shows that heavy flood with high velocity caused removal of binding soil between the boulders. The process accelerated due to obstruction at the mid-stream by construction of pier at the center. The floating material and tree logs damaged the masonry leading to failure of pier and consequently the collapse of the complete bridge.

3.5.3 Lesson

In boulders strata it is very difficult to decide the foundation level. As the streams in the hilly track flows with high velocity disturbs the binding soil of the boulders and the strata becomes hollow the foundations get loosened and with passage of time due to uneven settlement such failure takes placed.

3.5.4 Summary

Hence the failure of pier foundation and pier is not due to the insufficient design requirements

The investigation shows that heavy flood with high velocity caused removal of binding soil between the boulders. The process accelerated due to obstruction at the mid-stream by construction of pier at the center. The floating material and tree logs damaged the masonry leading to failure of pier and consequently the collapse of the complete bridge.

3.6 CASE 3

3.6.1 Characteristic of JAHU BRIDGE

The bridge was located at distance of 460 km from New Delhi and 30 km from District Hamirpur. This is important link road which serves and connect three districts Mandi, Bilaspur and Hamirpur. This Point is considered as the center of Himachal Pradesh. On August 11, 2007 & August 12, 2007 due to heavy rain the concrete bridge got collapsed. Then with in two-year government constructed a new bridge over the site. But on August 14th, 2014 the steel bridge also got collapsed due to soil erosion from foundation. This study is important because bridge collapsed two times due to same reason i.e. scouring of foundation. Local scour around the bridge abutment is one of the most critical causes of bridge failure.

Figure 14 Collapse of Jahu Bridge – (Jahu Bridge, Pankaj, 2018)

The site of bridge is at very critical point where two rivers i.e. Seerkhadd and Jabothi khadd meets each other. During summer these rivers have very low water level. The main source of water is only rain, which fed these rivers during Monsoon. The mean annual precipitation of Jahu (Himachal Pradesh, India) is 1411mm and the mean annual temperature is $21.9°C$. The site includes clay and loose rocky strata with boulder deposits.

Bridge type Concrete bridge (arch bridge) Span 56.30m Damaged Portion of bridge 30.72m Total damaged portion with surrounding 92m Construction era 1961-1966 Foundation Shallow foundation.

The construction of Jahu bridge was during third five-year plan (1961-1966). The span of bridge was 56.30m and damaged portion of bridge due to vented causeway was 30.72 m. The bridge was resting over two abutment and pier was located at the center. Load is transferring between pier and abutment is by mean of arch.

3.6.2 Description of the collapse

Scouring of foundation is considered primary causes of failure as it occupies around 60% of total bridge failure together with other hydraulic causes. This bridge collapsed mainly due to scouring of soil below the foundation. Scouring of foundation can occur all over the year, it reaches peak when flood comes in water body over which a bridge is spanning. History of bridge failure indicates scouring of streambed around abutments and piers of bridge, led to maximum bridge failures.

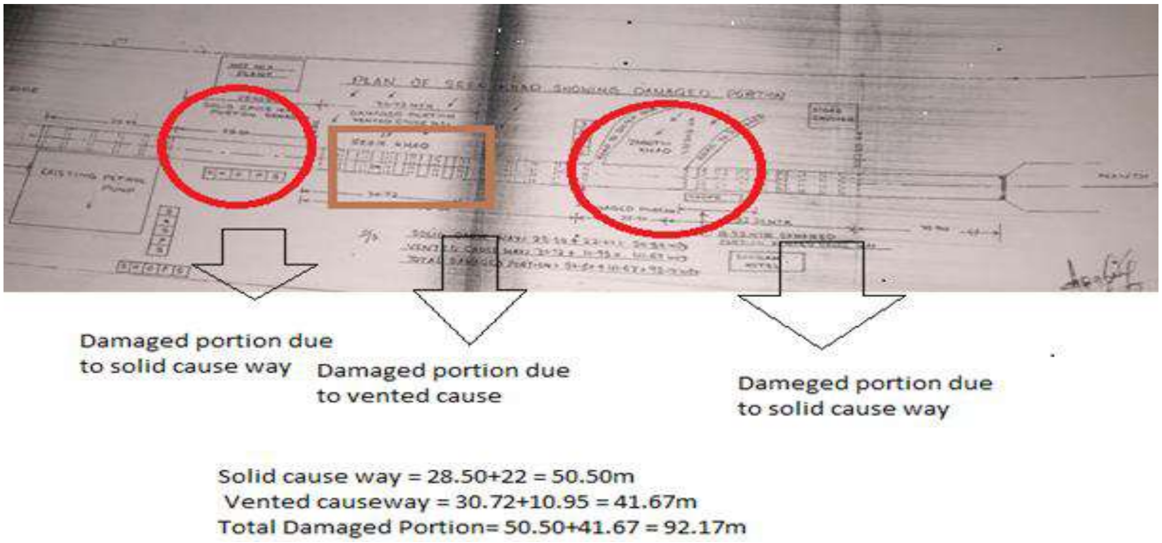

Figure 15 Plan of Seer Khadd Showing Damaged Portion – (Jahu Bridge, Pankaj, 2018)

Scour at bridges is a very complex process. Scour and channel instability processes, including local scour at the piers and abutments, contraction scour, channel bed degradation, channel widening, and lateral migration, can occur simultaneously.

The sum and interaction of all these river processes create a very complex phenomenon that has, so far, eluded mathematical modeling. To further complicate a mathematical solution, mitigation measures, such as riprap, grout bags, and gabions, may be in place at the abutments and piers. Any mathematical model would have to account for these structures as well. The interactions of the processes of local scour, contraction scour, channel bed degradation, channel widening, and lateral migration are unknown. The total vertical erosion at the bridge is then simply the sum of the scour and bed degradation. Because no other formation is available, this assumption provides a conservative estimate. Lateral channel instabilities are typically considered separately from scour and bed degradation, and the estimate of their effect on bridge foundations is often based on judgment and experience. The interactions of scour and channel instabilities are very difficult to predict. Certainly, the processes may not be independent but rather related to each other and the resulting impact on the bridge.

3.6.3 Lesson

It became important to under water inspection or instrumentation as a bridge management tool i.e. visual monitoring followed by flood watch and follow-up monitoring of scour for critical bridges. Use of modern techniques: - The use of wireless and remote sensors enables the movements of bridges to be monitored around the clock. This is most desirable in flood situations. Modern sensors, when installed on scour critical bridges, minimize the possibility of collapse and serve as a warning for a bridge to be closed. Depth of foundation is not only preferred according to design some time according to location it may vary. The way of river should be properly cleaned such that to avoid meandering of river. A proactive approach must be made in order to limit any of the effects from concrete spalling. Investigation through non-destructive methods can provide information early on so a preemptive response can be conducted. Examples of non-destructive methods include ground penetrating radar (GPR) and infrared analysis. Ultimately, a regular inspection schedule is crucial for proper care of any bridge structure.

3.6.4 Summary

Poor communication between the various design professionals involved and supervision. Poor communication between fabrications and erectors. This type of bridge requires proper repair and maintenance, so if there is any crack then it must apply grouting operation. But there should not be such operation was made.

CHAPTER 4: FAILURES DUE TO GENERAL COLLAPSE

4.1 General Collapse

Following is a list of causes of failures which are very General:

1. Design

a. Incompetent men in charge of design, construction or inspection

b. Supervision and maintenance by men without necessary knowledge or experience

c. Lack of enough preliminary information

d. Revision of design by persons lacking knowledge of the original requirements

2. Negligence during construction

a. A contractor takes a chance while completely aware of the risks involved

b. An engineer, usually competent and careful, shows negligence in some part of a design

c. Lack of coordination in production of plans, construction procedures and inspection

3. Economy

a. Restrictions in initial cost

b. Lack of maintenance

4.1.1 Member Failure

Member failure causes dynamic behavior of a truss bridge, and displacement in the dynamic behavior can exceed that in the static behavior. Therefore, dynamic analysis would be relevant for evaluating the behavior of a truss bridge after member failure. Yet dynamic analysis requires much computational time, and the redundancy investigation usually requires a large number of analyses in each of which the failure of a different member is assumed, since the member to fail cannot be singled out in general.

In 2007 a truss bridge in Japan was found to have a severed member due to corrosion. Fortunately, the bridge escaped the complete collapse. In 2010, a concrete bridge in Japan lost one of its bridge piers due to scour, but still held on.

4.1.2 Lack of attraction to critical details

Failure of connections due to overstress from bolt tightening, failure of formwork, local buckling of scaffolding, crane collapse. The stability of girders during stage construction and the deck placement sequence need to be investigated and the temporary bracing provided. Expansion bearings need to be temporarily restrained during erection.

4.1.3 Corrosion

For steel bridges one of the most dominant forms of deterioration is corrosion. The major effect of corrosion is the loss of metal section resulting in a reduction of structural carrying capacity. Three possible changes to a steel girder bridge can be considered; an increase in stress, a change in geometric properties (e.g. decrease of section modulus), and a buildup of corrosion products.

There are two fact about reinforced concrete: one is concrete cracks, and another is steel rusts. There is no escaping from these facts; eventually both will happen. We cannot stop entire action due to corrosion, but we can control by suitable methods. Concrete cracks because it is comparatively weak in tensile strength, but it is comparatively strong in compressive strength. Steel has a high tensile strength. The combination of these two materials is synergistic, in that they provide a superior composite material, and both have the capability to destruct through their respective deterioration mechanisms.

When the steel rusts, the oxide formation creates an expansive force within the concrete that causes cracking, spalling and eventual section loss. And it will lead to direct exposure with the atmosphere to steel. When concrete cracks, the steel is exposed to a combination of factors that accelerate its corrosion leading to further cracking of the concrete, and if allowed to continue will lead to collapse of the structure.

4.2 FAILURES DUE TO CORROSION

4.2.1 Corrosion in Concrete

The mechanism of corrosion is of electrochemical nature. This means that the (Anodic reaction) oxidation of the metal is counterbalanced by the reduction of another substance in another region of the metallic surface. Therefore, zones (anodes and cathodes) with different electrochemical potential, develop.

The two main causes of electrochemical corrosion are carbonation and the presence of chlorides. Carbonation usually induces a generalized corrosion while chloride will lead into localized attack. The corrosion can be easily recognized by the rust presence on the rebar and by the appearance of cracks running parallel to the rebars. It is also identified another corrosion, the stress corrosion cracking (SCC), that develops in prestressed wires subjected to special aggressive conditions.

In the case of concrete, the electrolyte is constituted by the pore solution, which is very alkaline. This pore solution is formed by mainly a mixture of KOH and $NaOH$ presenting pH values ranging between 12,6-14. The solution is saturated in $Ca(OH)_2$. Steel embedded in concrete is naturally protected by this high alkalinity and by the barrier effect of the cover itself.

4.2.2 Corrosion Mechanism

The mechanism of corrosion is of electrochemical process that consisted of anodic and cathodic reactions. The important part of the mechanism is the separation of negatively charged areas of metal or 'anodes' where corrosion occurs and positively charged areas or 'cathodes' where a harmless charge balancing reaction occurs.

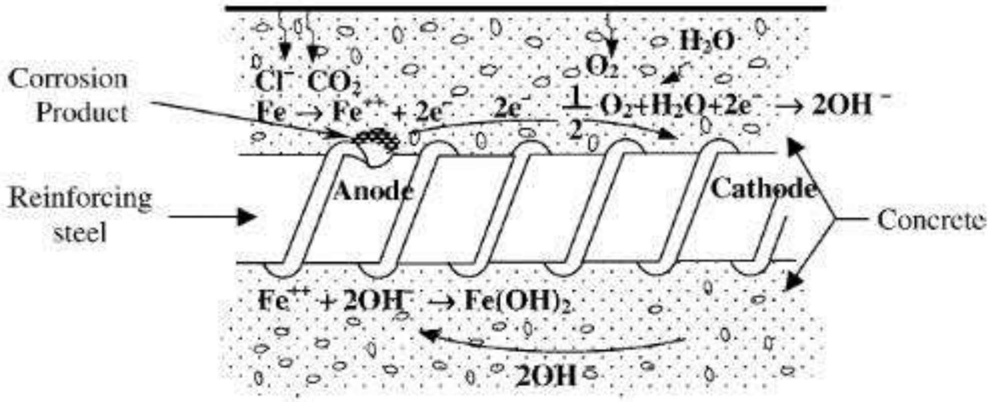

Figure 16 Schematic Illustration of Corrosion of Reinforcement Steel in Concrete as an Electrochemical Process (Ahmad 2003)

The possible anodic reactions in the embedded steel are:

3Fe +4H_2O → Fe_3O_4+8H^++8e^-

2Fe +3H_2O → Fe_2O_3+6H^++6e^-

Fe +2H_2O → HFeO_2^-+3H^++2e^-

Fe → Fe^{++}+2e^-

The possible cathodic reactions depend on the P^H of the vicinity of concrete and

availability of oxygen.

Fe +2H_2O → HFeO_2^-+3H^++2e^-

Fe +2H_2O → HFeO_2^-+3H^++2e^-

4.2.3 Damages to concrete

4.2.3.1 Corrosion Deterioration in RC Structures

It is the major deterioration problem in RC bridge structure. It reduces the life of the structures. It causes the strength and serviceability loss in the reinforced concrete elements. (Hansson 1984; Wryers et al. 1993; Liu and Weyers 1998; Chen and Mahadevan 2008; Zhang et al. 2010) Studies are conducted to define deterioration process in RC Structures contaminated with free chloride ion. This is some of the studies found that the corrosion process mainly depends on the concrete diffusion property, chloride threshold for reinforcement, concrete cover, diameter of reinforcement, environmental factors like moisture content, oxygen, carbon dioxide. Etc., and surface chloride content, we will see surface corrosion in the upcoming topics.

And researchers have defined the corrosion in terms of metal loss and corrosion current density based on Faraday's law (Liu and Weyers 1998; Vu et al. 2005; Chen and Mahadevan 2008). Corrosion current density of 1 A/m2 is equivalent to the corrosion penetration of 1.16mm/year (Hansson 1984). This result is based on the experiment in RC beam, Zhang et al. (2010) found to develop empirical relation for reinforcement corrosion loss in term of corrosion attack penetration. The corrosion deterioration also was explained in terms of the corrosion damage of the surface area due to cracking, spalling, and delamination (Wryers et al. 1993). The rate of damage was identified and used for the prediction of life in case of the bridge deck.

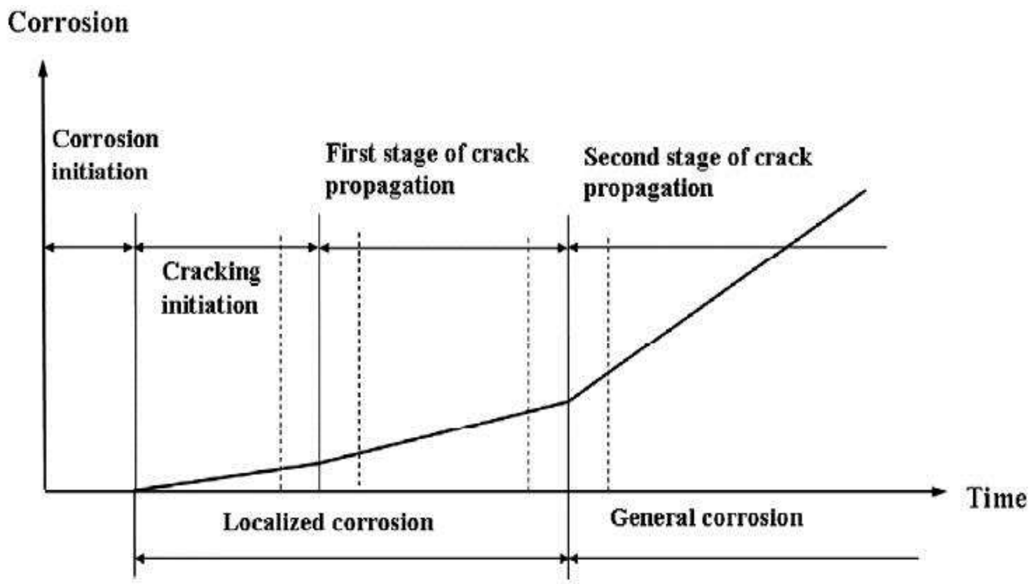

Figure 17 Corrosion Pattern under Natural Chloride-Induced Corrosion (Zhang et al. 2010)

Service life of RC structure depends on corrosion deterioration and acceptable damage level. Wryers et al. (1993) tells the chloride corrosion deterioration process for a concrete in three different stages and its 1st stage is corrosion initiation follows corrosion period or cracking and corrosion propagation. For deck structure, the authors used these deterioration processes to determine the rehabilitation time. Zhang et al. (2010) described for the case of natural chloride induced corrosion and it shown in the above fig**.** The experiment for RC beam conducted by these authors and observed the pattern in three phases and namely 1st phase corrosion initiation like before we discussed. In this phase the local pitting corrosion was observed and in localized corrosion was observed during the first stage of crack propagation followed by 2nd stage of crack propagation. And for the bridge pier, the effect of corrosion damage and reinforcement loss described by Tapan and Aboutaha (2008)

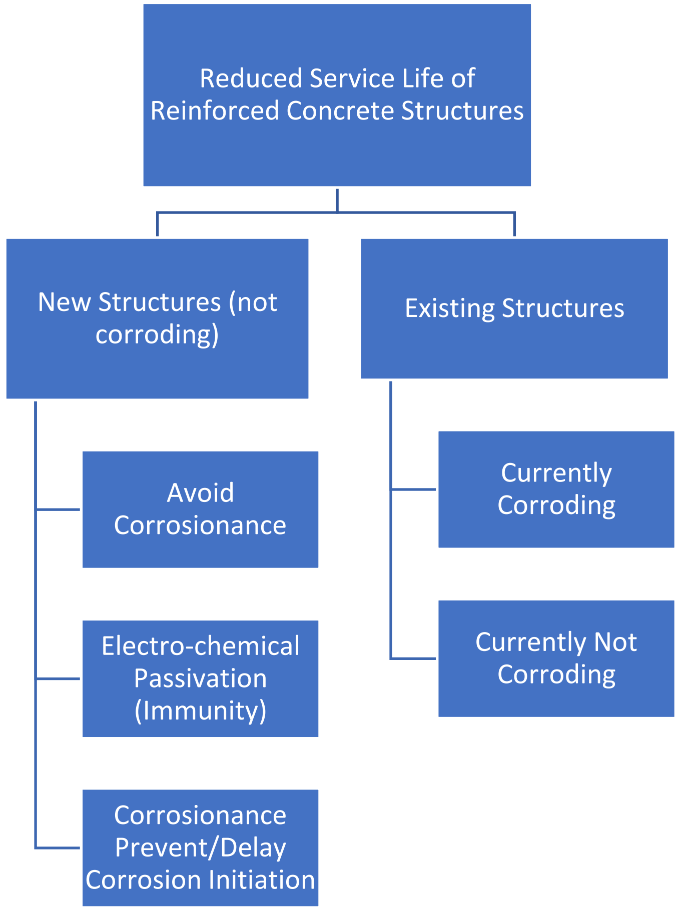

Figure 18 Reduced service life of reinforced concrete (Wryers et al. 1993).

4.2.3.2 Structural Effects

Loss of material may affect any one of three modes of resistance in a girder; bending, shear, and bearing. Loss of flange material will cause a reduction in the net area available to resist bending. The moment of inertia will be reduced, causing an increase in deflection. Also, the ultimate bending strength will be reduced, causing a reduction in maximum carrying capacity. In this study a section analysis program was used which calculates bending based on a composite strip method. In the method a section is treated as a collection of composite segments. Each segment has a defined stress-strain relationship. A strain level is set for the top layer and the correct depth to neutral axis is iteratively determined. From this, the bending moment and curvature relationship is developed, providing the initial bending stiffness and ultimate moment capacity of the section.

The loss of web material may influence the resistance modes of shear and bearing. Shear capacity can be calculated based on standard methods developed from plate theory. Bearing capacity, though, will depend on whether a stiffener is installed at the support. If a stiffener is present, column analogy can be applied to an effective width of the web. If no stiffener is present, plate theory can be used, assuming the ultimate capacity of the web in bearing is reached once the panel begins to buckle. The equations used to calculate bending, shear and bearing capacity can be combined with sampling methods to determine the statistics of resistance.

The modeling of resistance is very difficult due to the large variation in experimental observations. From the compilation of corrosion studies, it is apparent that the parameters A and B have a large and inconsistent variation. For the behavior of the web panel in buckling, there is an uncertainty in the boundary conditions of the plate. This boundary condition is reflected in the plate coefficient k. An important parameter is the amount of shear load distributed to each girder. In ordinary design, an assumption of simple beam deck behavior is used. This method of distributing loads has proved adequate for new bridges, however it remains uncertain and possibly critical for old bridges. To investigate the effect of variations in shear distribution, a shear factor, SF, is used to represent a linear increase or decrease in the shear load per girder. The four parameters, A, B, k, and SF are investigated separately in a sensitivity analysis.

4.2.4 Causes and Parameter of corrosion

4.2.4.1 Carbonation

Carbonation occurs when atmospheric carbon dioxide reacts with hydroxides such as calcium hydroxide to form carbonates. In the reaction with calcium hydroxide, calcium carbonate is formed is as follows,

$$\mathrm{Ca}(OH)_2 + CO_2 \rightarrow CaCO_3 + H_2O$$

This reaction reduces the P^H of the pore solution and level the passive film on the steel is not stable. Carbonation is a diffusion process and its depth given by an exponential attenuation along the time. The modelling of the carbonation depth is given by simplified expression,

$$x = k(CO_2)\sqrt{t}$$

Where,

x = Carbonation depth

t = Time and

$k(CO_2)$ = Carbonation factor of the concrete

It is not applicable for fully saturated or very dry conditions concrete. As the corrosion is generalized, cracks will appear running parallel to the rebars. Usually they won't appear before 20 years life for a cover of 20-25mm, it means that the corrosion rates are generally slow process. Spalling will be produced later stages.

4.2.4.2 Chloride attack

The chloride ions may be present in the concrete if they are added in the mix (admixtures, water or aggregates). However, this is fortunately not common. The most frequent is that chlorides penetrate from outside, either due to the structure is placed in marine environments or because deicing salts are used.

Chlorides induce local disruption of steel passive layer dealing into pits or localized attack. In submerged zones or in fully saturated concrete, chlorides penetrate by diffusion. However, in aerial zones or when submitted to cycles (deicing salts), capillary absorption may be a faster mechanism of penetration. In both cases, the penetration is as well dependent of the square root of time. Therefore, its modelling may be made similarly to the carbonation, by means of the simplified expression,

$$x = k(\mathrm{Cl})\sqrt{t}$$

The factors influencing the chloride threshold are:

- Type of cement: finess, amount of gypsum, blending materials etc.

- Water/cement ratio – w/c (porosity).
- Curing and compaction (porosity).
- Moisture content and variation.
- Type of steel and surface roughness and condition (pre-rusted or not).
- Oxygen availability (corrosion potential when arriving the chlorides).

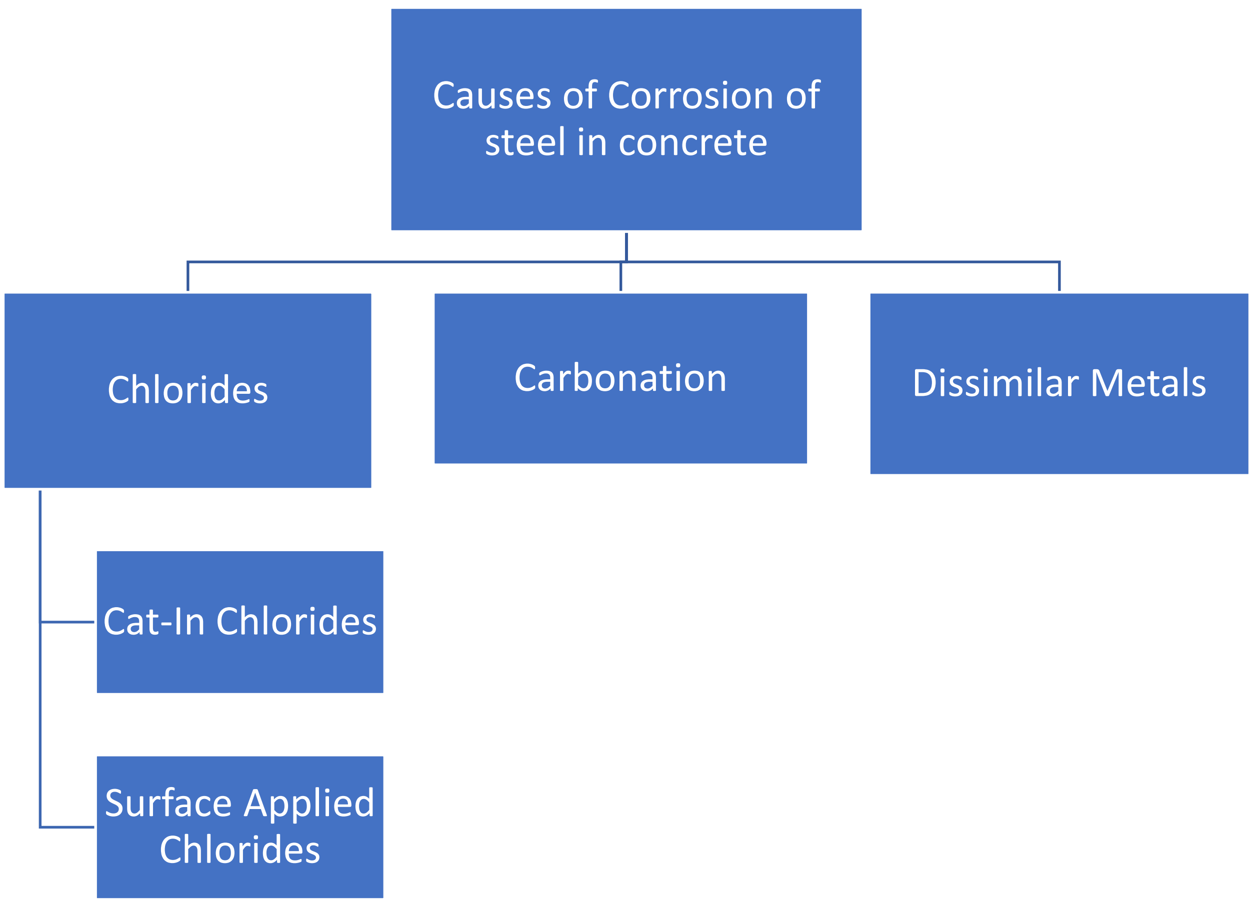

Figure 19 Causes of corrosion of steel in concrete(Wryers et al. 1993).

4.2.4.3 Surface Corrosion

Jack R. Kayser and Andrzej S. Nowak (1989) described the corrosion loss follows an exponential function. Using large amount of data has been collected on the account of rate of material loss of metal specimens in different environment is as follows from M.E. Komp, Atmospheric corrosion ratings of weathering steels--Calculations and significance, Mater. Perform., 26 (7) (July 1987) 42-44

$\mathrm{C} = At^{B}$

Where,

C = Average corrosion penetration in μm

t. = Number of years

A, B = Parameters determined from the analysis of experimental data and its random variables.

Therefore, actual corrosion loss C is also become a random variable. The values A and B depends on environmental factor where the bridge is located. Albrecht and Naeemi [P. Albrecht and A.H. Naeemi, Performance of weathering steel in bridges, National Cooperative Highway Research Program, Report 272, July 1984.] have summarized corrosion test results for various environments.

4.2.4.4 Stress Corrosion Cracking

The SCC is a specialized type of corrosion which is produced when mechanical stresses act simultaneously to some specific aggressive agents. This type of corrosion may then develop in prestressed or posttensioned wires. The mechanism of this type of corrosion is not yet well understood and several theories exist in the literature. The phenomenon may occur accompanied by an embrittlement of the steel due to the penetration in the steel of hydrogen gas produced by a corrosion reaction. The three conditions necessary to develop the phenomenon are:

1) A type of steel susceptible to suffer this type of corrosion,

2) The steel must be stressed beyond a minimum threshold below which the process is very slow, and

3)Aa specific aggressive media (producing or not hydrogen gas)

When the three conditions are found simultaneously the process develops in three steps:

1) One or several microcracks are generated at the surface of the steel,

2) These cracks grow until they reach a certain depth and then they propagate very quickly until

3) It aims into the brittle failure of the wire. This failure may be enhanced by hydrogen embrittlement.

Figure 20.A Start of stress corrosion cracking. (Kamnik, April 2008)

Figure 20.B Start of stress corrosion cracking. (Kamnik, April 2008)

4.6 Corrosion prevention

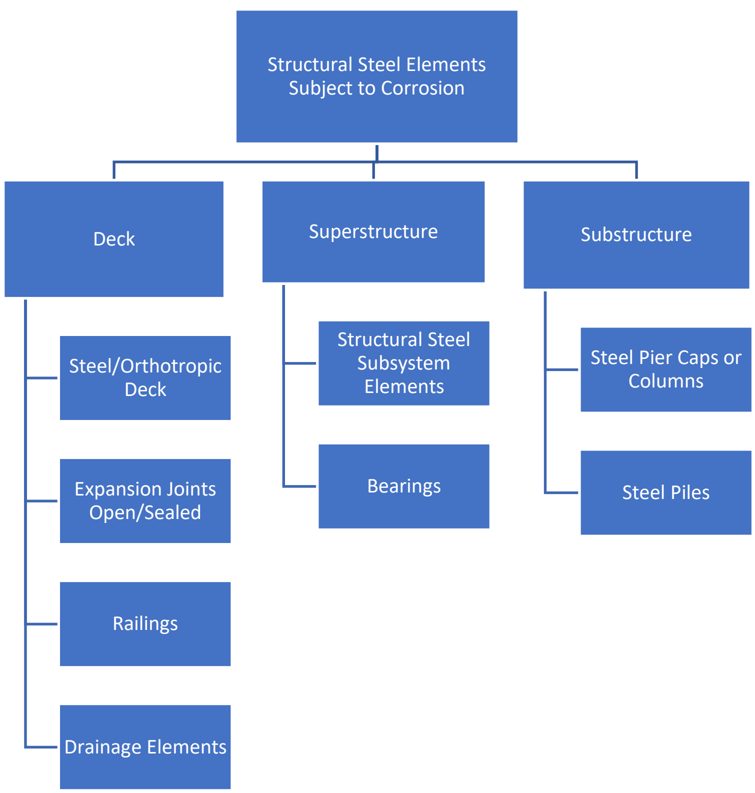

Figure 21 Structural steel elements subject to corrosion (Kamnik, April 2008)

4.3 CASE 1

4.3.1 Characteristic ANDHERI BRIDGE

It is a Railway crossing two-way bridge composed of, formed by four longitudinal precast prestressed girders each with cast in situ RCC diaphragms and deck slab forming the composite girder system.

The Andheri road over bridge that collapsed onto the railway tracks on Tuesday, resulting in five people suffering injuries and a day-long disruption of train services, was a disaster waiting to happen, according to the Western Railway's

It was this extra weight of nearly 124 tones — comprising dry sand, paver blocks and cable wires — that led to the collapse.

Figure 22 Andheri Bridge – (economictimes,2018)

4.3.2 Description of the collapse

The pedestrian pathway at Andheri train station, the failure of the ROB pathway was contributed by the additional load of various cables, sand, paver blocks etc. The additional loads were not considered at the time of the original design. There was heavy corrosion and pitting of the cantilever steel brackets supporting the pathway resulting in thinning down of section and failure of the pathway ROB, reported that during the post-collapse inspection of Gokhale Bridge, railway authorities had stated that additional load created by cables and paver blocks are among the reasons behind the collapse.

The accident occurred due to heavy/deep corrosion and pitting of cantilever steel brackets supporting the pathway resulting in thinning down of section. Also, the additional load of various cables, sand, paver blocks, etcetera, provided by the BMC without prior permission from the Western Railway, not contemplated when the bridge was designed, also contributed to its weakening.

Figure 23 Failure due to corrosion – (economictimes,2018)

4.3.3 Lesson

This accident is classified as the failure of railway staff and others. Because the responsibility of the maintenance and repair work of this bridge was of the Western Railways."

4.3.4 Summary

It was this extra weight of nearly 124 tones — comprising dry sand, paver blocks and cable wires — that led to the collapse. From the western railway reports it is due to maintenances and repair work issues. It comes under category maintenances and careless issues.

CHAPTER 5: FAILURES DUE TO BUCKLING

5.1 Types of Buckling in Structural Members

What is buckling!!! Buckling is a phenomenon occurs when a compressive force or stresses applied on a member or a part of a member displaces laterally or simply, we can say out of plane displacement. The displacements are related with flexural stresses whose magnitude depends on the slenderness of the member. The buckling load of the member is limited by these flexural effects.

The type of buckling is described below,

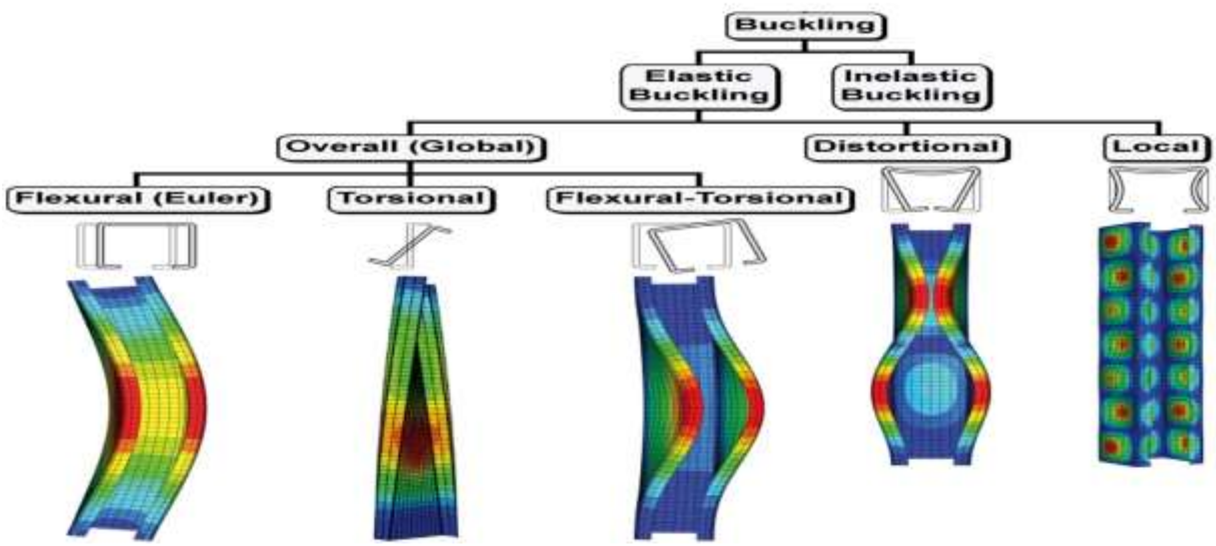

Figure 24 Types of Buckling in Structural Members – (Krisda Piyawat & Thomas H.-K. Kang 2012)

5.1.1 Flexural Buckling

It is the most easily recognized mode of buckling for members. Let's consider a pin-ended member which is nominally straight, but it has a small lateral displaces or imperfection (out-of-straight), out-of-straightness increases under axial compression. The buckling load is reached for a steel member, when the combined action of axial and flexural stresses reaches its yield stress at some point in the cross section or the bending moment reaches the plastic bending resistance as modified by the axial force. Let's consider an I-section member, unrestrained against displacement in any transverse direction, the displacement and flexural stresses due to bending about the minor axis of the cross section is greater than the major axis then the member is said to buckle about its weak axis and its shown in below fig,

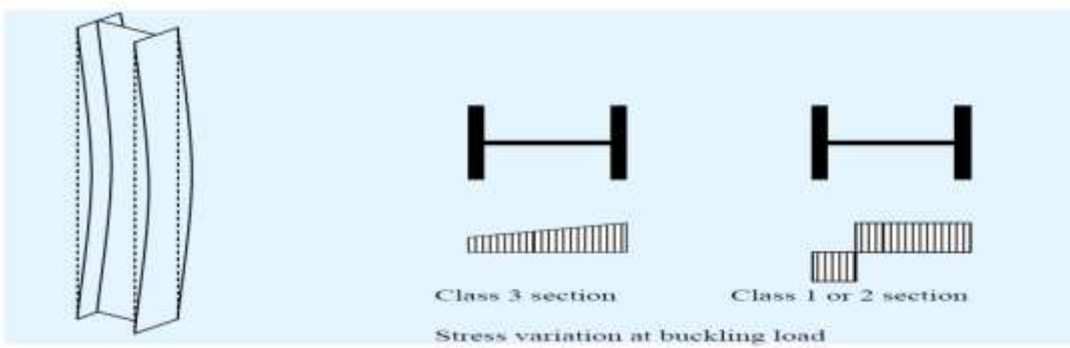

Figure 25 Flexural Failure – (Determining the buckling resistance of steel and composite bridge structures, D C ILES, 2012)

For Steel Bridges, Flexural buckling is usually related for trusses (where chord and diagonals are in compression) and for bracing members which are subjected to compression. Similarly, for composite Bridges, it is about for bracing members.

5.1.2 Torsional and Flexural-Torsional Buckling

In a torsional buckling mode, doubly symmetric sections can buckle by involving only twisting about their longitudinal axis. For sections such as a cruciform section the buckling load may be less than that of flexural buckling, when the member is short. Such cross sections are rarely used in bridges. For the case of monosymmetric sections and asymmetric sections, torsional and flexural buckling modes are linked, and it may occur at a lower load than flexural buckling about the minor axis. These modes of buckling are only relevant to angle and channel bracing members in bridges.

5.1.3 Lateral Torsional Buckling

It is the continual consideration for the design of I-section members without intermediate restraint. A member which is bent about its major axis, the compression flange will tend to buckle laterally, the flange is effectively a compression member that is free to buckle only in one direction. Because the flange is connected to the web and it can only displace by twisting the cross section by imposing a smaller lateral displacement of the tension flange.

5.1.4 Distortional Buckling

Let's consider Bridge deck formed by a RC slab supported by a I-section steel girder and acting compositely with them. Lateral buckling of the bottom flange can occur when they are subjected to compressive stress due to bending but in practical situation axial force alone is too small to lead to buckling. It is possible to consider this situation for the case of series Tee sections, side by side, that could buckle in a lateral torsional buckling mode, while it can be prevented by the continuity of the slab more significantly, provides a flexible torsional restraint at the top of the web. The mode of the buckling then becomes a lateral torsional mode.

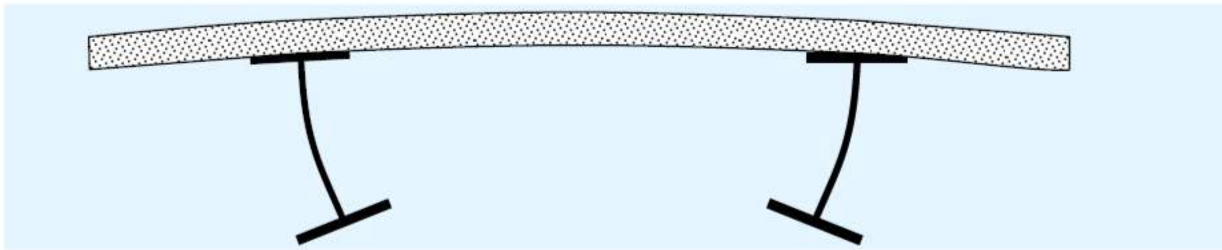

Figure 26 Distortional buckling (hogging region of continuous composite deck) – (Determining the buckling resistance of steel and composite bridge structures, D C ILES, 2012)

5.2 Design of Member Subjected to Buckling

5.2.1 Introduction

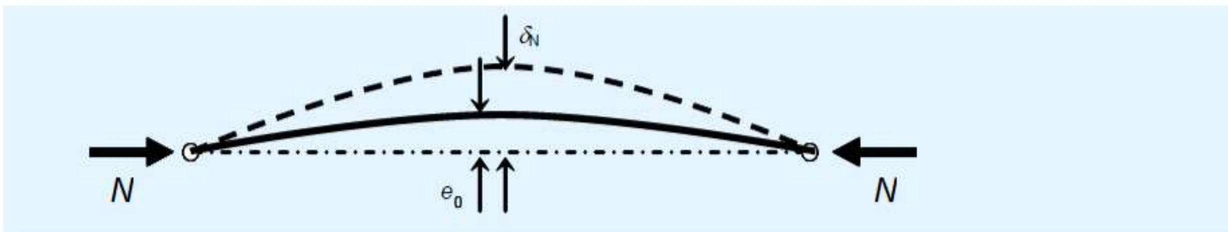

Figure 27 Flexural Buckling – (Determining the buckling resistance of steel and composite bridge structures, D C ILES, 2012)

Deflection under compression is given by,

$$\delta_N = \frac{e_0}{(1-{}^{N}/_{N_c})}$$

Where,

e_0 – Initial out of straight

δ_N – Out-of-straight under compression

N_c – Elastic Critical force for flexural Buckling (Euler Load)

The elastic limit is reached at an extreme fiber is given by the sum of axial and bending stresses reaches yield,

$$\frac{N}{A} + \frac{N\delta_N}{W} = f_y$$

The value of elastic critical force is essential to determine the buckling resistance of the member. Its value is depending on the member geometry and material stiffness, but the strength of the material does not affect the theoretical critical value.

5.2.2 Flexural Buckling

The buckling load is referred to in Eurocode 3 as the "elastic critical buckling force and for a pin ended struct, is the axial force

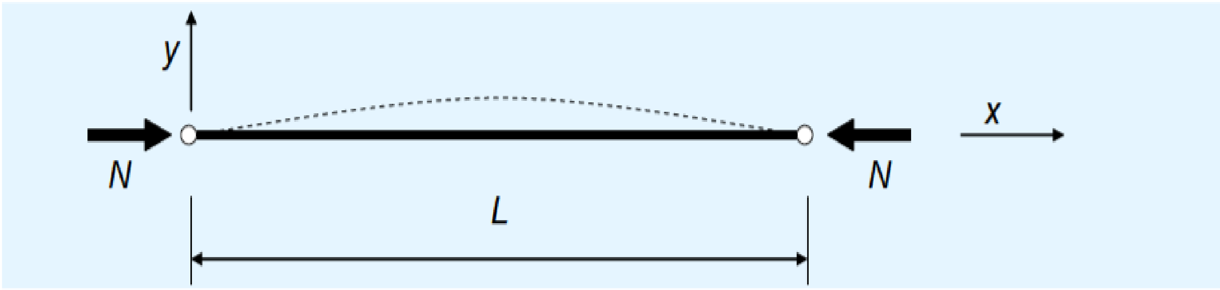

Figure 28 Flexural Buckling – (Determining the buckling resistance of steel and composite bridge structures, D C ILES, 2012)

$$\frac{d^2y}{dx^2} + \frac{Ny}{EI} = 0$$

The lowest solution to this expression, for a simple sinusoidal flexural displacement is,

$$N_{cr} = \frac{\pi^2 EI}{L^2}$$

Where,

E – Modulus of Elasticity

I – Second moment of area of the member about the minor axis

The elastic critical buckling force for flexural buckling N_{cr} is referred as the Euler Load.

$$N_{cr} = \frac{\pi^2 EI}{(kL)^2}$$

Where,

K = 1 for both ends pinned, k = 0.5 for both ends fixed and k = 0.7 for one end pinned, one end fixed.

The product of KL is often referred as the effective length for buckling.

5.2.3 Lateral Torsional Buckling

For lateral torsional buckling, the expression for elastic critical buckling moment is similar. For a uniform doubly symmetric beam like I-beam and a constant bending moment throughout its length, the elastic critical buckling moment can be expressed as,

$$M_{cr} = \frac{\pi^2 EI_z}{(kL)^2}\sqrt{(\frac{k}{k_w})\frac{I_w}{I_z} + \frac{(KL)^2 GI_T}{\pi^2 EI_z}}$$

Where,

E – is the modulus of elasticity (E = 210000 N/mm)

G – is the shear modulus (G = 80770 N/mm^2)

I_z – is the second moment of area about the minor axis

I_T – is the St Venant torsional constant

I_w – is the warping constant

L. – is the beam length between points which have lateral restraint

5.3 Erection Procedure

5.3.1 Design Error and Construction Error

Many bridges collapsed due to imperfect design and use of materials with poor quality, use of inappropriate construction method have led to bridge collapse in the construction phase. strict process control and proper supervision can effectively reduce the probability of this type of bridge failure. A functional bridge may only have a few vehicles on it when it collapses, it takes hundreds of workers to build a bridge all of whom may be in dangerous situation in case of collapse.

5.4 Proof Checking and Dual Authority

If the third-party proof-checking had been made on the design and drawings of this supporting structure, prior to construction, it would have saved the collapse and lives. Who will independently check the analysis, design and detailing of the structures. Such a procedure eliminates the percentage of failures, and any mistake made by original designer, is found and corrected at the design stage itself. Moreover, the contractor who builds the structure is also well qualified and certified, hence even if there is a constructability issues, which is missed even by the proof checker will be identified by him\her and will be rectified before construction process.

Considering the seriousness of this collapse, the structural design and quality of construction for the whole length of the flyover that has been already built, is to be thoroughly checked, even load tested as per IRC procedure, for structural safety.

5.5 CASE 1

5.5.1 Characteristic of Charus Bridge

Charus Bridge was a three span (40m+110m+40m) continuous deck type truss bridge located in Uttarakhand, India. The bridge was proposed to connect two cities namely, Srinagar on left and chauras on right bank of the river Alakhnanda.

It was 190m span length bridge, designed for 2-lanes of 7.5m wide carriageway and 1.5m wide footpaths on either side. It was a lattice truss girder bridge with subdivided top chord members. Distance between top and bottom chord members was 8.66m and c/c distance between two trusses was 7.5m. It consists of 38 panels of 5.0m length each.

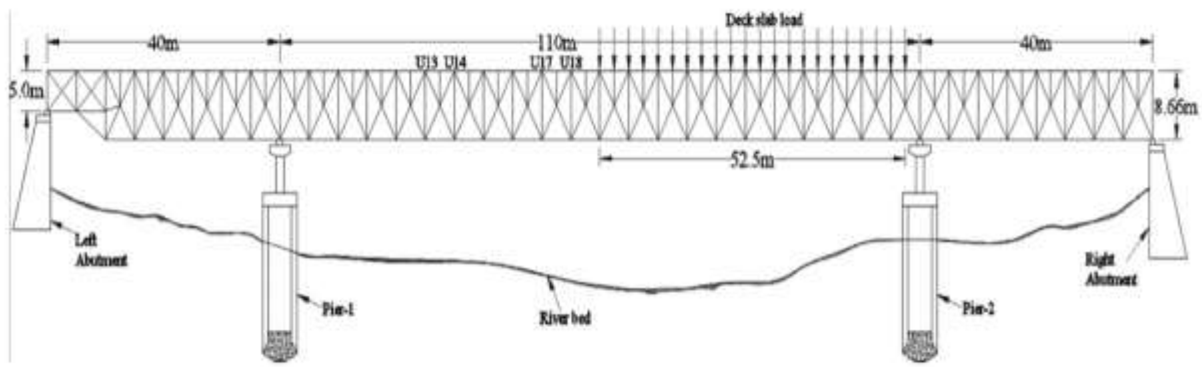

Figure 29 Chauras Bridge – Plan View (Failure of Chauras bridge Harshad, pabitra rajan, Pramod kumar, 2014)

The bridge consisted of one central span of 110m and two end spans of 40m. Top and bottom chords of the bridge consisted of buildup box sections, 500mm wide and 600mm deep, comprising four angles at four corners, and 2 x 575 mm and 2 x 390 mm wide four vertical plates.

5.5.2 Description of the collapse

Chauras continuous deck type truss bridge failure took place during casting of the deck slab. The bridge was proposed to connect two cities is described in above section. After Placing the steel truss on two piers and two abutments, casting of deck slab was initiated from mid portion of the 110m span of the bridge towards right piers. When concrete was placed in 52.5m length from mid span of 110m towards right pier, bridge started collapse and below we will see the reason for the collapse.

Figure 30 Chauras Bridge after failure (Failure of Chauras bridge Harshad, pabitra rajan, Pramod kumar, 2014)

From the Failure of chauras bridge journals, an analysis of this bridge was carried out using STAAD Pro V8i software under the existing loads at the time of collapse. From the analysis it is found that the compressive stress in member U13U14 is 173.8 N/mm^2at the time of collapse, and maximum force in the upper chord members was 6000.1KN in member U18U19.

Casting sequence of the deck slab in chauras bridge was started from mid of 110m span, which caused lifting of 40m end span. The Ideal Procedure for casting deck slab is to start from the supports and proceeds towards mid spans. In this case casting procedure is also the major impact of chauras bridge failure.

Next major fault is dimensioning and Buckling stress calculation. Buckling stresses for the buildup section (149.8 N/mm^2) and individual plates (140.0 N/mm^2) are quit close, and these strength of these are less than the actual stress developed at failure (173.8 N/mm^2). But it is not clear whether the local buckling at the double welded plates or buckling of the entire member U13U14 initiated the collapse.

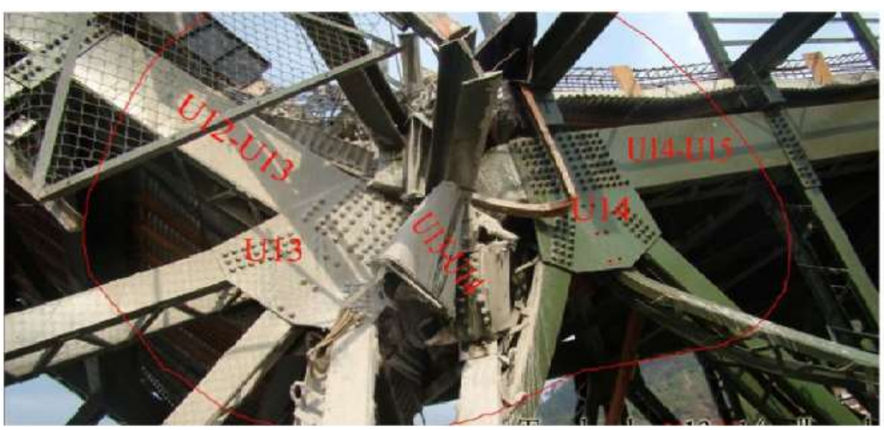

Figure 31 Joints U13 and U14 and buckled member U13U14 (Failure of Chauras bridge Harshad, pabitra rajan, Pramod kumar, 2014)

Mild steel of grade E250 used in chauras bridge had ultimate tensile strength (f_u) of 410 N/mm^2 and yield strength of (f_u) 250 N/mm^2. Permissible tensile stress for mild steel as per Indian standards is 0.6f_u = 150

N/mm^2. Slenderness ratio less than 10 maximum permissible compressive stress is also $0.6f_u$ = 150 N/mm^2, which decreases with increase in slenderness ratio. Tension and compression members of a steel truss have entirely different behavior before failure. Compression members suddenly buckle and fail without reverse strength in beyond maximum up to yield stress, while tension members have reverse strength after yielding.

Another important factor is gusset plates, the member U18U19 did not fail even at such a high compressive stress, as it was prevented against buckling by the reverts. While gusset plate at joint U13 and U14 remained intact at high stress whereas, member U13U14 buckled and failed at a lower stress. Thus, gussets plates if connected properly to the members and prevented from buckling can take compressive or tensile stress up to ultimate strength of plate.

Figure 32 Intact gusset plat at joint U13 and U14 (Failure of Chauras bridge Harshad, pabitra rajan, Pramod kumar, 2014)

5.5.3 Lesson

From the above section, it is clearly shown that Compression members buckle and suddenly fail without warning causing loss of life and consequently an additional load factor of 1.5 may be required at the limit state of strength.

In the case of member U13U14 the slenderness far excess, the width to the thickness ratio of individual 575mm wide, 8mm thick plate was 72 while it is limited to 30.

Designing of dimension is required at the same time Buckling of compression members check too.

Proper deck casting procedure might save the bridge during casting, loss of life and property.

5.5.4 Summary

Staad pro analysis shows that the compressive stress in members U13U14 of Chauras bridge at the time of collapse was 173.8 N/mm^2 while the permissible stress is 149.8 N/mm^2. Failure took place due to buckling of members U13U14. Buckling of the member was also facilitated because 8mm thick plate is placed instead of 16mm plates. Buildup section must be carefully dimensioned.

Compression members buckle and suddenly fail without warning causing loss of life and consequently an additional load factor of 1.5 may be required at the limit state of strength for DL+LL case.

5.6 CASE 2

5.6.1 Characteristic of Kolkata Flyover Bridge

Kolkata flyover is also known as Vivekananda Road flyover which is consists of 2.5km stretch in central Kolkata. The flyover is in Burra bazar area which is one of the largest wholesale markets in Asia. It was constructed to reduce the traffic flow.

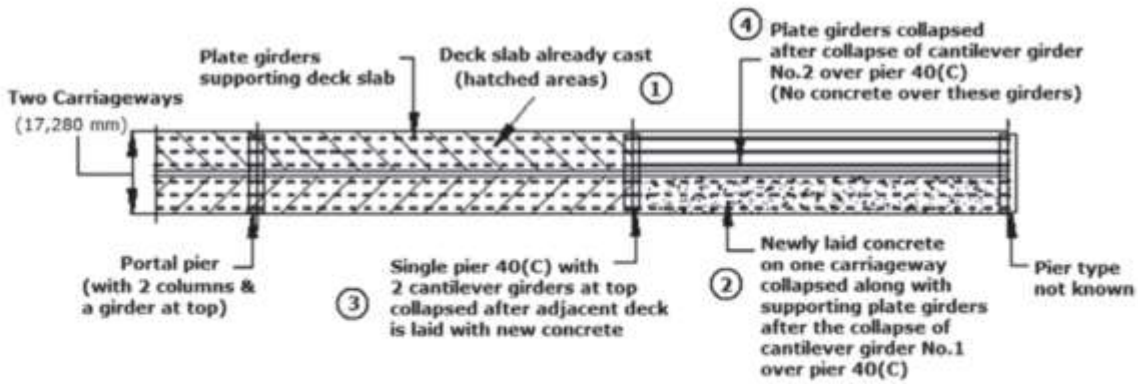

Figure 33 Part plan of Kolkata Flyover at the point of collapse (Collapse of Kolkata flyover N.Prabhakar & N.Subramanian, 2017)

It was a 2.5km stretch composite construction flyover having 2 serviceable roads with provision for expansion of 4 lanes. Steel structural chosen for superstructure to complete the project in tight time schedule. Box girders were chosen for columns and beams forming rigid portal frames on RCC foundations supported on bored cast in pipes-driven to 45m below the r level. It is a composite construction i.e., reinforced concrete deck slab over steel plate girders which are supported on steel piers at an interval along the length of the flyover. On the top of the steel super structure a cast in situ RCC slab 200mm thick was designed topped by a 100mm PC wearing course and finally 50mm thick mastic asphalt surfacing was laid.

5.6.2 Description of the collapse

The site visit is restricted so most of the journals and papers have used photo of the damaged bridge to analysis for the cause of failure and in this case also photo used as the main source to describe the cause of failures. The main cause of failure is the joint details adopted at the cantilevered beam at pier 40(C). The twisting of steel plate girders placed on top of the cantilever girders, which indicated that the failure could have been due to lateral torsional buckling of the girders, as there may be inadequate bracing to their top flanges. From the below fg, it is clearly shown the box section of the cantilever girders was not connected to the vertical face of the pier 40(C) by either bolting or welding. From the sketch it is clearly seen that 4 no's beam over pier cap is the only structural strength for the cantilever girders.

Figure 34 Close view of Pier Cap (Collapse of Kolkata flyover N.Prabhakar & N.Subramanian, 2017)

The pier 40(C) which had collapsed on 31st Mar 16 was supporting two simply spans. One side of the pier the deck slab on the carriageways were already cast. While on the other side, concreting for deck slab was done for one of the carriageways while the other carriageway slab was not cast prior to the collapse. From this it is clearly seen that cantilever Girder no.1 carried the full dead load from the deck slab on the both sides, whereas cantilever Girder no.2 carried the dead load from the deck slab on one side only. This is clearly seen from the pic source SEFI Website. It must be noted from below fig, that the concrete debris had fallen on one side whereas on the other side the bare plate girders have fallen, without having any concrete over on it.

Figure 35 After failure, Concrete Deck Slab had been Laid on one Cantilever Side only (Collapse of Kolkata flyover N.Prabhakar & N.Subramanian, 2017)

The collapse was not initiated when the concrete had been laid on one side of the girders only. When the new concrete was laid on the other side which was supported by cantilever Girder no.1, this girder-initiated collapse due to flexural and shear failure and collapse of Cantilever Girder no.2 took place following it because of a common beam supporting them. It implies that the joint at the cantilever girder was not designed for full dead load condition of supporting concrete deck slab on either side of the girders, apart from the deficiency they had with deflection. When the loads were applied, the bottom edge of the cantilever beam was pushed into the hollow steel column and making it to dent. Because of this there was heavy tension in the top plate of the hollow cantilever, which teared off, when it exceeded the ultimate tensile force.

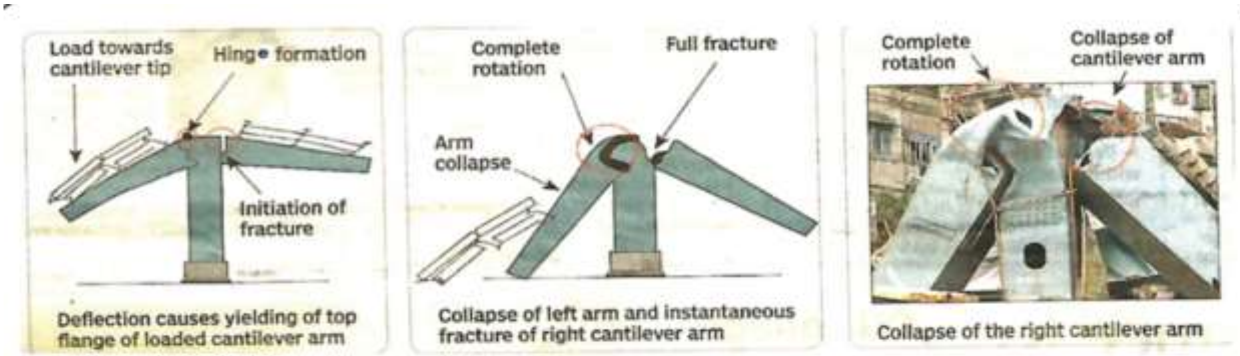

Figure 36 (Kolkata flyover collapse, Nirmalendu Bandyopadhyay, April 2016)

This initiated the shear failure of the side plates of the hollow cantilever beam, thus resulting in total collapse. From the Below fig, it is seen that the cantilever girder 1, had a large opening near the supporting end and a spliced connection. Opening near the support is to reduce the shear capacity of the girder while here affected the shear capacity of the beam. And another major defect was the inadequate number of bolts were used in many important locations.

Figure 37 Collapse of the two cantilever girder (Collapse of Kolkata flyover N.Prabhakar & N.Subramanian, 2014)

From the fig we can clearly see that other spans of the flyover did not fail. Because in other spans, there are two piers supporting the hollow beams, in which case it will be in simply supported condition, and hence there will not be any problem in carrying the load. While in this section 40(C), only one column is provided, the cantilevered beam resulting in a failure, as described above.

5.6.3 Lesson

That the cantilever girders were not at all designed to carry any super imposed loads that would be there on the flyover when it will be put into service.

Considering the seriousness of this collapse, the structural design and quality of construction for the whole length of the flyover that has been already built, is to be thoroughly checked, even load tested as per IRC procedure, for structural safety.

If the third-party proof-checking had been made on the design and drawings of this supporting structure, prior to construction, it would have saved the collapse and lives.

5.6.4 Summary

It is to be noted that the failure has occurred when the bridge is not subjected to any live load. Therefore, it can be concluded that there was some basic design deficiency.

The combination of failure mode is listed below.

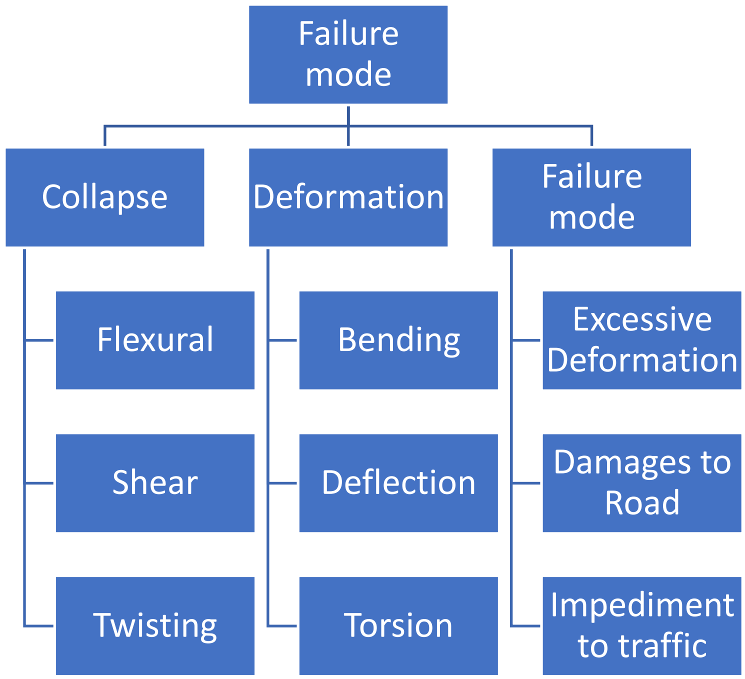

Figure 38 Combination of failure mode

The longitudinal beams spanning between the cantilever girder had no bracings on the compression flanges to prevent lateral buckling. Such buckling imposed additional horizontal loads on the portal frame box girders. At the cantilever girder the horizontal box girder beams should have extra depth at the knee joint to withstand additional moments resulting from moment redistribution according to stiffness of each member at the joint. The box girder should have internal ribs to withstand torsion forces resulting from torsion and buckling of the girders.

CHAPTER 7: LESSONS FOR THE PRACTICE

7.1 General observation

Many authors have already done this, some of them failures, and some in general terms and some depend regions…etc. There is nothing new about learning from accidents and recommending that others make use of the insights gained.

The following topic describes the lessons learned from this thesis, in the stages of design, structural calculations, detailing design, construction managements as well as maintenance.

7.2 Design

During in design stage the designer never thinks about failures of his design, it will increase the risk of future failures. A large proportion of defects in structures, not only those leading to failures, are a result of mistakes made at the design stage. Here we will see about structural safety verifications, coordination, various aspects of calculations and precautionary measures.

7.2.1 Organization: Coordination, delegation, exchange of information

The contractor carrying out the scaffolding work on the structures must notify the supervising body of the name of the person responsible for the technical coordination of the work. The person must confirm the coordination which has been affected on the construction documents.

Many failures are due to improper coordination or deficiency in the organization of the construction process. I mentioned this type of failures in the section buckling failures case study. Section 5.5.3 Proper deck casting procedure might save the bridge during casting, loss of life and property. Construction process should maintain the sequences to avoid this kind of failures.

7.2.2 Verification of Structural Safety

The verification of structural safety by calculation, for various reasons, become extremely extensive. Some of the reasons are unavoidable, such as complicated limit conditions resulting from new type of construction and greater exploitation of components.

7.2.2.1 Scope, Summary, Form

Here we are going to the see the above failure case as summary with related to structural safety. In the section 3.4.4 Scouring By analyzing this failure in the bridges, the future planning can be better. Therefore, the aim should be to design the bridges for all times and for all occasions. Foundation of new bridges, bridges to be widened, replaced shall be designed to resist the scour for 100-year flood criteria, which may create the deepest scour at foundations. And in the section buckling case study it is clearly shown that Compression members buckle and suddenly fail without warning causing loss of life and consequently an additional load factor of 1.5 may be required at the limit state of strength.

Staad pro analysis shows that the compressive stress in members U13U14 of Chauras bridge at the time of collapse was 173.8 N/mm^2 while the permissible stress is 149.8 N/mm^2. Failure took place due to buckling of members U13U14. Buckling of the member was also facilitated because 8mm thick plate is placed instead of 16mm plates. Buildup section must be carefully dimensioned.

Compression members buckle and suddenly fail without warning causing loss of life and consequently an additional load factor of 1.5 may be required at the limit state of strength for DL+LL case.

7.2.2.2 Various aspects of Calculation

In this section we are going to discuss failures in various stages and aspects. Modeling is an essential for the assessment of the dynamics of a load-bearing structures. From the section scouring by analyzing this failure in the bridges, the future planning can be better. Therefore, the aim should be to design the bridges for all times and for all occasions. Foundation of new bridges, bridges to be widened, replaced shall be designed to resist the scour for 100-year flood criteria, which may create the deepest scour at foundations.

If the results of this study are born in mind when modeling, they may serve to prevent engineers from placing their faith in complexity of the model in the belief that this will get them closer to reality, and that the computer can cope with any calculation.

7.3 Construction Management

There is a great inherent risk in the divergence between the increasing complexity of load bearing structures and the reduction in the quality of staff employed to produce the structures in workshops and on the construction site. In the section scouring case Poor communication between the various design professionals involved and supervision. Poor communication between fabrications and erectors. This type of bridge requires proper repair and maintenance, so if there is any crack then it has to apply grouting operation. But there should not be such operation was made. And in the section corrosion This accident is classified as the failure of railway staff and others. Because the responsibility of the maintenance and repair work of this bridge was of the Western Railways."

7.3.1 Precautionary measures

It is extremely difficult to build a structure in accordance with seemingly unimportant details in the construction plans if members of stasis are not aware of the consequences of deviation from these plans. For this reason, thoroughly training of staff is vital.

CHAPTER 8: MY OWN SUMMARY

Here I have drawn the following lessons from the cases described in this thesis work and also considering the other authors particularly in the case study.

8.1 Sequence of Construction process

Do not take any decision in the design on your own if the following cases met

- you don't have enough time

- you don't have necessary experiences designs

- you don't have suitable staff

- you don't have adequate funding

When working with rules and regulations, always bear in mind their area of application. Train staff at all levels when delegating new tasks.

8.2 Design

Don't allow yourself to be led by the vision of others such as clients, architectures, etc.,.When designing a structures, think of all aspects that could influence the results. Designer never think about failure or collapse of his design. You will not lose face if you ask the advices of colleges from other disciplines to avoid mistakes.

Provide complete, unambiguous and clearly arranged instructions for the people doing the job.

Work in close collaboration with those colleagues responsible for design and structural calculation and in the other side with those responsible for manufacturing.

8.3 Verification of Structural safety

Always check whether the first assumed permanent load has changed during the design process. Remember that several bridges have collapsed because, the actual load was greater than the load for which the bridge was build.

When you have completed your work, don't fail to provide brief documentation of how your structures functions at all stages of its constructions and its later service life.

Go through the construction process repeatedly in your mind's eye until you are certain that no intermediate stage has been overlooked in your structural calculations.

8.4 Construction Management

If any construction components are missing never assume to simply substitute them with others.

If the information available to you is inadequate for this, never take it in your own hands to solve the problem.

Be aware that the load-bearing capacity of many components depends in its combination with others, such as the right nuts for threads in threaded connections or the correct tube diameter for tube couplings.

CHAPTER 9: CONCLUSIONS

The above-mentioned principle cause of failures is the primary cause of a Bridge failures, but there are the results of the two subcategories namely enabling and triggering causes. concurrently. Enabling and triggering causes are the means by which a bridge can fail, where enabling causes are generally internal to the bridge structure and triggering causes are external to the bridge; for an example enabling cause and a triggering cause could be inspection errors and tornado damage respectively. These causes can create a situation, where a total collapse or a partial collapse is probable; a total collapse is the situation where the traffic is no longer serviceable. Bridge Scour is one of the major failures of about 60 percent of total failure. Most of the bridge scour failure due to inadequate maintenances and complied database is listed in table. Second most failures occur during construction process due to improper management and in construction sequence and poor communication. In this work we discussed about this kind of failures in the section buckling failures.

In this thesis work the summary of the most common types of failures are presented. Each bridge failures due to different types of causes and some are major, and some are minor. From the case study it is clearly known that the failures are due to structural deficiency, improper management, Design error, Construction Sequence issues and careless. It is highly recommended to create a group of members responsible for the bridges, one authority and one responsible.

From the search, it is noted that insufficient in the field of case study for both major and minor bridges failures. It is also one of the reasons for inadequate knowledge in this filed. Engineers are not only responsible for design, construction…etc. but also for creating and shaping our society and improve the way we work and live. They should aware the responsibilities connected to the society and people's life. The minimum qualification for doing practicing should be master's degree.

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