

Progressive Collapse of Bridges

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ABSTRACT

Previously, history recorded several disproportionate collapse incidents around the globe like the collapse of the Tacoma bridge. The sudden collapse of bridges can have devastating consequences, resulting in significant loss of life and economic disruption. Ordinary design procedures, which are usually limited to gravity and seismic/wind loads, are inadequate for preventing progressive collapse.

This study examines progressive collapse in bridges, where localized damage triggers a chain reaction leading to catastrophic failures. It investigates the phenomenon of progressive collapse in bridge structures, emphasizing the importance of effective design strategies to prevent such failures. It also discusses the direct and indirect design methods, where the direct method focuses on enhancing the strength of key structural elements and creating alternate load paths, while the indirect method emphasizes maintaining overall structural integrity through simpler measures.

The findings highlight that while bridges can withstand the loss of a single component, their resilience decreases significantly with the simultaneous loss of multiple components, particularly in critical areas. Ordinary design events, which are usually imperfect to gravity and seismic/wind loads, are insufficient for preventing progressive collapse. Therefore, a focus on establishment and retrofitting techniques to mitigate progressive collapse is necessary.

Keywords:-*Progressive collapse, bridges, cable-stayed bridges, load-bearing elements*

INTRODUCTION

Progressive collapse, often referred to as disproportionate collapse, occurs when significant structural elements fail one after another due to an initial minor triggering event, causing the structure to collapse partially or completely.

The emergence of civilization, population growth, and urban expansion have led to an increase in the utilization of high-rise buildings and long-span bridges. The progressive collapse of these structures was caused due to disproportionate triggering events which were either accidental or intentional incidents. Regardless of the cause of initiation, the

progressive collapse resulted in loss of lives and economic losses.

The fall of the World Trade Centre (WTC) 1 and 2 towers in 2001, which was caused by the impact of hijacked planes, and the current collapse of Champlain Towers South in 2021 can be taken as a measure of how alarming the progressive collapse is.

The investigation of bridge collapses was caused by the localized failures that resulted in partial or complete collapses which caught the interest of researchers and engineers. The collapse of Tacoma Narrow Bridge, a long and narrow bridge was caused by a remarkable wind-induced oscillation that made the bridge deck twist

around its centre and ultimately collapse frame by frame. The I-35W steel deck truss bridge suddenly failed, resulting in 13 fatalities and over 100 injuries and the collapse of Tennessee Highway 69 bridge was initiated during its construction time. The progressive failure of the Hongqi Viaduct, a multi-simple-supported bridge failed unexpectedly during its planned demolition taking 9 innocent lives and imposing 16 more injuries.

Various explanations, both natural and man-made, were put out for the original cause of the failure. Landslides, flooding, scouring of bridge substructures, and vibrations caused by earthquakes are examples of natural causes. However, man-made causes of bridge collapses can also be listed, such as explosions caused by extremists, collisions between cars and ships and piers, and mistakes made in the study, design, and building of bridges.



Fig.1:-I-35W steel deck truss bridge before and after the collapse
(Source: Derseh et al., 2023)



Fig.2:-Hongqi Viaduct Bridge before and after the collapse
(Source: Derseh et al., 2023)

FACTORS INFLUENCING THE PROGRESSIVE COLLAPSE OF BRIDGES

Natural factors

Flood

One of the most frequent and dangerous natural catastrophes is floods, which can trigger other natural disasters like landslides, debris flows, and erosion. Because flow conditions vary between locations, flooding is a common reason why highway and railroad bridges sustain

damage. Floods frequently cause bridge collapses, and the main causes are recurrent flood disasters, inherent defects in foundation construction, and human-induced changes to riverbeds.

Earthquake

Due to their abrupt nature and great destructive force, earthquakes have a major effect on bridge safety, particularly when bridges are not built with the structural integrity to resist them. When

an earthquake strikes, the local bridges sustain damage that complicates post-disaster recovery and reconstruction efforts, interferes with vital transit routes, hinders rescue efforts, and increases the loss of life and property. Four main factors contribute to the seismic dangers to bridges are the superstructure collapsing, the bearings on the bridge being damaged, the piers and abutments being destroyed, and foundation failure.

Wind

Wind-induced vibrations, commonly referred to as aerodynamic vibrations, are dynamic reactions brought about by the interaction between wind and bridge constructions. Bridge aerodynamic vibrations can be categorized into multiple forms, such as flutter, vortex-induced vibration, jump, swing, and torsional vibration, based on various wind loads and interaction mechanisms. Bridge aerodynamic vibrations not only compromise a structure's longevity and safety, but they also put pedestrians and drivers in peril and can potentially cause a bridge to collapse. On November 7, 1940, the Tacoma Narrows Bridge gave way in a gale and showed remarkable oscillations before collapsing abruptly.

Scouring

The term "scour" describes the damaging impact of flowing water, which, in some situations, significantly alters the geomorphology of riverbeds, beaches, and riverbanks. Riverbed scour is caused by various events, including the construction of reservoirs, dredging, sand mining, and localized scouring of bridge piers. This phenomenon lowers the embedment depth of bridge foundations and compromises their stability. In the worst circumstances, a bridge collapse may result. Moreover, scouring caused by floods may endanger the stability of stone arch bridges.

Landslide

A landslide results in the downward and outward movement of slope-forming elements, such as rock, soil, artificial fill, or a combination of those components. It is mostly caused by water saturation, earthquakes, or eruptions. When these moving slope-forming materials strike the bridge, they will likely cause significant damage or even cause the bridge to collapse.

Manmade factors

Design errors

Errors resulting from faulty design frequently affect structural safety, and bridges may even collapse during the building phase. First of all, design engineers may rapidly apply immature methodologies and concepts in actual engineering before comprehensive design schemes are discussed if they have low awareness and do not demonstrate enough originality in their design. Second, mistakes like doing the wrong calculations or just reproducing designs that already exist might happen throughout the design phase. Thirdly, norms, studies, and theories that were not fully established at the time may have satisfied certain requirements, but they are nonetheless responsible for congenital flaws. These flaws may affect the structure's functionality and safety.

Construction mistakes

Inappropriate building techniques and the use of improper or inferior building materials can result in subpar construction that does not meet design requirements. Furthermore, the quality of the work can be harmed by malfunctioning construction equipment. Particular attention should be paid to the overturning collapse processes and the possible safety issues in existing bridges with similar geometries and the same kinds to prevent the collapse of asymmetric box-girder bridges or single-

pier bridges. The safety of bridges is significantly influenced by the quality of the building materials. For instance, the Tuojiang Bridge in Hunan, China, fell on August 13, 2007, as a result of inadequate design and inappropriate building methods.

Lack of inspection and maintenance

Live loads and the environment constantly threaten bridges that are in use. Bridges consequently encounter deterioration can result in major issues when intensified beyond a certain point. Numerous elements, such as the mechanical, environmental, and material qualities, contribute to the degradation. Although there is always a chance that a bridge will fail, a good maintenance program that includes routine inspections and appropriate repair can stop bridge deterioration and assist in identifying any structural issues before they become major catastrophes.

Terrorist attack

There have been numerous terrorist strikes against global transportation networks in recent years. Important infrastructures are frequently attacked because of their accessibility and possible effects on both economic activity and human life. Terrorists usually target the bridge's piers and decks because their failure results in the collapse of the bridge.

Collision

Unpredictable accidents frequently occur when automobiles and bridge superstructures or between boats and bridge piers or columns collide. Extremely high lateral forces are being transmitted to the affected bridge

structures during the crash. When applied to a very tiny contact area, this enormous impact force can result in extremely high local pressure and localized damage to bridge components. Strong inertial forces and vibrations developed as a result of the bridge consuming the dynamic impact energy. Forces from collisions have the potential to bring down the bridge entirely or seriously harm its constituent parts.

TYPES OF PROGRESSIVE COLLAPSE

There are different types of progressive collapse. Each type can be characterized depending on the nature of the collapse progression through a structure.

Pancake-type collapse

The principal factor contributing to the pancake-type collapse is the reduction in vertical load-bearing capability resulting from an unusual incident, like a blast or fire. Members then begin to fail as a result, and debris begins to fall on members on lower floors. The debris frequently places a significant dynamic impact load on the storeys below, leading these levels to be loaded up to four times greater than the static loadings they were intended to withstand. This ultimately leads to the storeys collapsing. This kind of collapse primarily affects high-rise buildings. Even though many high-rise structures are extremely redundant and can create alternative load routes (ALPs) in the event of a column loss, they are not equipped to stop this.

The 1985 collapse of primary and secondary schools in southern Mexico City, and the 1999 earthquake motion-induced damage to the Marmara region of Istanbul, Turkey are some examples.

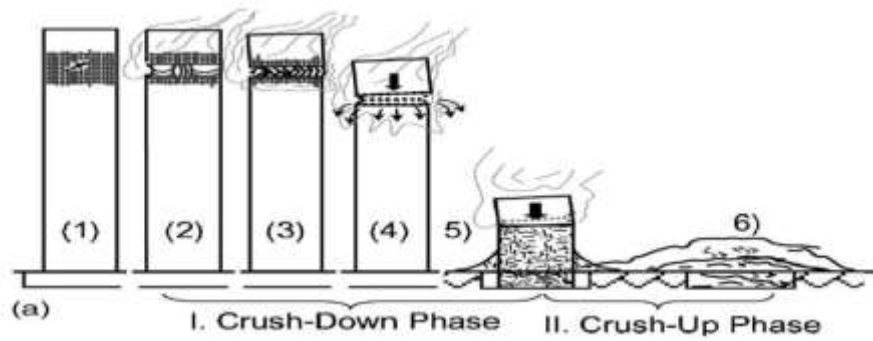


Fig.3:- Pancake collapse representation
(Source: Derseh et al., 2023)



Fig.4:- Pancake collapse at Mexico and Turkey
(Source: Derseh et al., 2023)

Zipper-Type Collapse

Unlike pancake-type collapses, zipper-type collapses are typically caused by the failure of a tension member, which causes an impulsive load to be applied to the system and redistributes the remaining weight to the additional structural components. This shift in

the balance of power results in a transverse direction is reached by an overload, instability, and failure of other components. renowned instance of this kind of failure is the Tacoma Narrows Bridge collapse, which took place in 1940 as a result of the twisted bridge girder being distorted by wind.



Fig.5:- Zipper-type collapse on Tacoma Narrows Bridge
(Source: Derseh et al., 2023)

Domino Type Collapse

When a structural element overturns due to an initial angular rigid-body-based motion, a chain reaction is set off, continuing to apply a lateral impact force on the upper edge of the overturning element on the side face of the neighbouring elements. This failure pattern resembles a row of dominoes and is a Domino-type of collapse. When an angular rigid body motion occurs, energy is transformed from gravitational potential

to kinetic energy. This energy transformation applies a horizontal force to overturn the adjacent element, with the corresponding direction of failure being the direction of the elements' overturning. The 2005 collapse of eighty-two tension and suspension of five different 110 kV overhead electrical transmission lines in Germany can be mentioned as an illustration of the Domino-type collapse of structures.

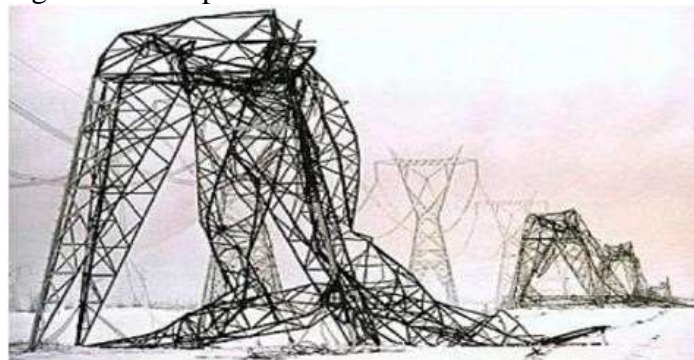


Fig.6:-*The domino-type collapse of electric transmission line in Germany*
(Source: Derseh et al., 2023)

Instability Collapse

Collapses of the instability type occur when parts that are meant to stabilize a structure fail. One element of this is bracing. For example, bracing is required for pinned steel frames to guarantee the stability of the structure under lateral loads. The structure cannot support lateral loads if the bracing fails. Depending on the function and location of the damaged piece, instability failures may result in

abrupt or progressive excessive collapses. The failure of the Tennessee Highway 69 bridge was caused by a long, unbraced girder that revealed a warping torsional instability. Bridgeport Connecticut (US) frame element collapsed due to the inadequate connection leading to instability in steel members can also be mentioned in the cluster of Instability-type of collapse.



Fig.7:-*Collapse of Tennessee Highway 69 bridge and Bridgeport Connecticut (US) steel frame*
(Source: Derseh et al., 2023)

Mixed- Type Collapse

In real-world failures, it is rare for a structure to undergo just one kind of collapse. As the name implies, a mixed-type collapse is the combination of the different sorts of collapses described

before. Stated differently, this kind of collapse involves the interaction and contribution of multiple types of collapse. The failure of the Alfred P. Murrah Federal Building revealed both a Pancake and domino-type collapse.



Fig.8:-Alfred P. Murrah Federal Building
(Seminar: Derseh et al., 2023)

**PROGRESSIVE COLLAPSE
ANALYSIS AND DESIGN METHODS
FOR BRIDGE STRUCTURES****Analysis Methods**

Dynamic effects can be taken into consideration in different ways depending on the analytical techniques.

Linear static method

Linear static method is considered to be the simplest method in the analysis of structures under progressive collapse. This method involves an application of magnified loads acting on the unstressed structure and post-processed analysis results including axial (P), shear (V), bending moment (M), and coupled PMM actions can easily deployed for computation of acceptance criteria (demand capacity ratio).

It is an approach where equivalent static story forces, due to wind or earthquakes, are applied to the structure. The loads in

linear static analysis are applied progressively until they reach their maximum magnitude. Once the loads reach their maximum magnitude, they stay constant (time-invariant).

In a linear static approach, structures subjected to dynamic effect, are expected to respond elastically to the effects of a sudden loss of column. In this approach, the characteristic load, consisting of dead load plus 25% of the live load, amplified by a dynamic load factor of 2 shall be used. And allows DCR greater than unity to account for inelastic deformation. However, its limitation is that it does not account for inelastic deformation, P-Delta effects, or redistribution of forces.

Non-linear static method

Applying loads incrementally is the basis of a non-linear analysis. Loads are not taken into account at one point in time throughout the computations; instead, they are increased gradually, and solutions

to consecutive equilibrium states are carried out.

To create a "pushdown curve" of the structural behaviour, the nonlinear static technique generally reflects a dynamic enhancement through a load factor and sequentially applies the gravity load reaction of the removed column. For this reason, to accurately depict the inelastic response of the system, this method needs precise representations of material behaviour.

While it's better than simple methods, it still has some drawbacks like it doesn't perfectly capture how structures move quickly (like in an earthquake) and in predicting exactly how materials will break can be difficult.

Linear dynamic method

Linear dynamic analysis incorporates the effects of time-dependent loads and inertial forces. It is used to evaluate the dynamic response of structures to sudden load changes, such as the removal of a structural element. The linear dynamic method of analysis also

known as time history is known to simulate the dynamic effects associated with the sudden loss of columns but does not represent the inelastic deformation behaviour of the system.

This method requires an extensive engineering judgment as to whether or not P-Delta effects are significant and whether or not the computed DCR values are exceeded.

Non-linear dynamic method

The nonlinear dynamic technique is known for its complex and detailed methodology, which enables it to completely capture the dynamic response of building components in the event of sudden loss of a column. This method's drawback is that it requires extensive computational time and highly skilled labour. Only structural engineers with

training and expertise in structural dynamics should utilize them.

Design Methods

There are two general ways of approaching reducing the possibilities of progressive collapse. They are direct and indirect methods.

Direct method

The direct design approach enhances resistance to progressive collapse by optimizing the strength of key structural elements and developing structures capable of spanning the local failure area. This method necessitates more complex analyses and regular design procedures compared to the indirect design approach. Two consecutive strategies are employed in the direct design. They are the specific local resistance method and the alternate load path method.

Within the specific local resistance method, critical vertical load-bearing building elements (such as columns and load-bearing walls) are explicitly designed by the engineer to withstand the specified threat level. On the other hand, the alternate load path method involves localizing the response by designing the structure to redistribute loads through an alternative path in case of primary load-bearing element failure. In contrast to the indirect method, the present approach will be applied to structures of moderate to high significance.

Indirect method

The indirect design method is recognized for its simplicity and uniformity across various projects. It focuses on ensuring that structural integrity is maintained throughout the design process, which is crucial for preventing progressive collapse. This method requires designers to incorporate general structural integrity measures. These measures include the selection of structural systems, layout of vertical load-bearing elements, member proportioning, and detailing of

connections. By doing so, the overall robustness of the structure is enhanced, making it more resilient to potential failures.

The indirect design method is typically recommended for buildings with regular layouts that have fewer transfer mechanisms and are categorized as of lower importance. This makes it suitable for structures that do not require complex design considerations. A common technique within the indirect design method is the tie-force approach. In this approach, the building is mechanically tied together, which improves continuity and ductility. This method also facilitates the development of alternate load paths, which are essential in the event of a local failure.

PREVENTION AND MITIGATION STRATEGIES

Strengthening and Retrofitting

Strengthening and retrofitting techniques refer to special measures used to enhance a structure's ability to withstand progressive collapse, which can occur due to unforeseen events leading to local failures that propagate throughout the structure.

The techniques can be categorized based on three main criteria related to progressive collapse:

1. Preventing Initial Failure: Techniques that aim to stop the first failure from occurring.
2. Controlling Collapse Propagation: Measures that help manage how a failure spreads through the structure.
3. Controlling Final Collapse Status: Strategies that influence the overall outcome of a collapse

It is not always likely to classify some consolidation and retrofitting techniques into specific categories since they have dual effects, or the performance depends on the acting threat, the load level, or the

initial failure size. Some of such techniques are as follows:

- **Adding New Alternate Load Paths:** This technique involves introducing additional structural elements, such as beams or trusses, to create alternative routes for load distribution in case of failure. Its advantage is that it provides redundancy, allowing the structure to maintain stability even if one element fails, and can significantly improve the overall load carrying capacity of the structure. However, the effectiveness can depend on the existing structural configuration and the nature of the triggering event and it may require extensive modifications to the original design, which can be costly and time-consuming.
- **Improving Existing Load Paths:** This technique focuses on enhancing the capacity of existing structural elements through methods like jacketing or adding steel plates. Strengthening a column by wrapping it with steel plates to increase its loadbearing capacity is an example. This enhances the capacity of existing structural elements without the need for complete replacement and can be less intrusive than adding new elements, preserving the original design aesthetics. If not done carefully, it can lead to unintended consequences, such as increased vulnerability to buckling or other failure modes. The effectiveness may vary based on the height of the building and the specific load conditions.
- **Use of Energy Absorbing Devices:** This technique incorporates devices that absorb energy during an impact, reducing the forces transmitted through the structure.

Installing dampers or foamed aluminium layers in a building to absorb seismic or blast energy is a suitable option.

This helps to reduce the forces transmitted through the structure during an impact, minimizing damage, and can be tailored to specific types of loads, such as seismic or blast loads.

The cost of materials and installation can be high, which may limit their use in some projects. Long-term durability and maintenance requirements can be a concern, especially in harsh environments.

- **Seismic Retrofitting:** This technique involves enhancing a structure's ability to withstand seismic forces through the addition of elements like shear walls or bracing systems. Adding shear walls to a building to improve its lateral stability during an earthquake. This enhances the structure's ability to withstand seismic forces, reducing the risk of progressive collapse during an earthquake. This can also improve overall structural performance and safety.

However, the interaction between seismic retrofitting and progressive collapse resistance is not well understood, which can lead to unintended weaknesses. It also requires careful design to avoid compromising the structure's performance under different loading scenarios.

- **Fire Protection Systems**
Intumescent Coatings: These coatings expand when exposed to high temperatures, providing a protective layer for steel structures. They are the most common fire protection system used for steel elements. The

effectiveness of intumescent coatings can be enhanced by incorporating fibre-reinforced materials, which also provide confinement to reinforced concrete (RC) columns, improving their axial strength.

MAINTENANCE AND INSPECTION

After the bridge is constructed, it is necessary to inspect the bridge at regular intervals of time and to maintain it in such a way that it functions properly. The defect that may develop in any one particular component of the bridge structure may extend gradually and can affect the overall load-bearing capacity. The inspection can be either detailed or routine. Each part must be properly inspected and maintained.

We can make use of the advanced technologies as well. Structural health monitoring systems for bridge structures can be used. This makes use of electronic monitoring in bridge maintenance. The system interface and integration of the actual practice are mainly based on visual inspections and combine the response of several different reliable sensors installed on the structure to monitor the progress of the damage.

EMERGENCY RESPONSE PLANNING

Emergency response plans, first responder training, and properly planned evacuation procedures are highly required in case of bridge collapses. These will help to save the lives of people and other damages.

Effective emergency response is dependent on the coordination and cooperation of emergency management, emergency communication, police, fire emergency, medical services, and other public, private, and non-profit organizations.

EXPERIMENTAL CASE STUDY: PROGRESSIVE COLLAPSE BEHAVIOUR OF A LONG-SPAN CABLE-STAYED BRIDGE INDUCED BY CABLE LOSS

About the Case Study

This case study was conducted by Q Chen, H Wang, S El-Tawil, AK Agrawal, B Bhattacharya, and W Wong. It was published in 2023 by the publisher ASCE. Cables in cable-stayed bridges are vulnerable to various forms of damage, including corrosion, fatigue, impacts, and improper designs, which can lead to progressive collapse, a situation where local damage escalates to complete structural failure. It mentioned that the main objective of this paper is to study the dynamic behaviour and potential progressive collapse behaviour of cable-stayed bridges due to single or multiple cable loss scenarios.

Methodology

Model creation: A three-dimensional (3D) explicit finite-element model (FEM) with

material nonlinearity and predetermined failure criteria is created to carry out the collapse simulations and examine collapse behavior using an existing long-span cable-stayed bridge as a prototype. A prototype cable-stayed bridge The chosen example bridge is 1004.6 meters long overall. It has a main span of 471.2 meters, two side spans of 198.1 meters, and two approach side spans of 68.6 meters on each side. With four car lanes in each direction and a single walkway on the south side, the bridge can handle two-way traffic. The main and long side spans of the bridge deck have a combined width of 42.7 meters. In the approach side spans, the width increases to 36.6 m. The diamond-shaped pylons are 175 m high and are connected to the girders by 64 stayed cables in each plane. All cables are regularly spaced at 14.3 m along the deck, except for the first and the last four back stays in the side spans.

FEM: The 3D FEM of the above-mentioned bridge was developed to run on the LSDYNA platform.

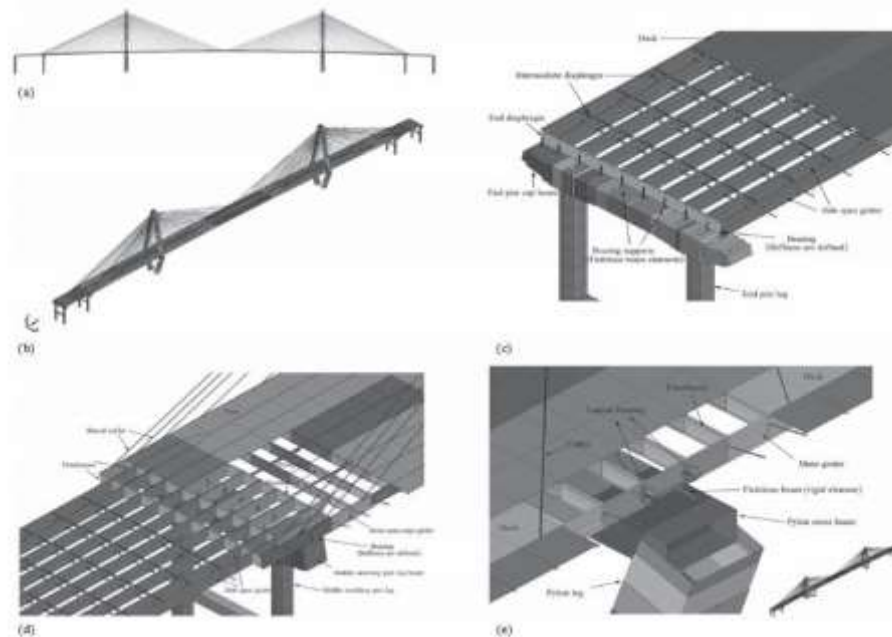


Fig.9:-FEM of the prototype bridge: a) elevation view; b) isometric view; c) components information near end pier; d) component information near the auxiliary pier; e) component information near pylon leg.

(Source: Chen et al., 2023)

Table 1 and Table 2 contain the element types and material models used to model the different structural components of the bridge and the material properties of each structural component respectively.

Table 1:-Element and material information for the FEM of the prototype bridge
(Source: Chen et al., 2023)

Structural members	Element types	Material information
Stay cables	Multiple truss elements	MAT_PLASTIC_KINEMATIC
Substructure (bridge pylons, piers)	Belytscho-Schwer resultant beam	MAT_MOMENT_CURVATURE_BEAM
Structural steel members (girder, floor beam, etc)	Hughes-Liu beam with cross-section integration	MAT_PLASTIC_KINEMATIC
Concrete deck	Fully integrated shell element	MAT_PLASTICITY_COMPRESSION_TENSION
Bearings	Discrete beam element	MAT_NONLINEAR_ELASTIC_DISCRETE_BEAM
Miscellaneous connection	N/A	CONSTRAINED_NODAL_RIGID_BODY
Non-structural elements	Mass element	N/A
Support on substructure	Belytscho-Schwer resultant beam	MAT_RIGID

Table 2:-Material properties of structural components in the prototype bridge
(Source: Chen et al.,2023)

Structural components	Materials	Young's modulus (GPa)	Compressive strength (MPa)	Yield stress (MPa)	Ultimate strength (MPa)	Failure strain
Pylons and piers	Concrete	32.88	48.26	–	–	–
Deck	Concrete	35.15	55.16	–	–	–
Reinforcing steel	ASTM A615 Grade 60	200	–	413.7	620.5	0.2
Post-tensing strands	AASHTO M203 Grade270	196.5	–	1,689.9	1,861.6	0.06
Stay cables	ASTM A416 Grade 270	196.5	–	1,689.9	1,861.6	0.06
Girder, floor beams, and others	AASHTO M270 Grade 50	200	–	344.7	448.2	0.2

Structural steel members: This includes girder members, floor beams, stringers, steel diaphragms, and secondary bracing members. The nonlinear behaviour of structural steel members is modelled using a cost-effective elastoplastic material model, which allows for the automatic deletion of elements upon reaching a defined ultimate strain (20%), ensuring a realistic simulation of material failure.

Bridge pylon: To ensure accuracy in predicting structural behaviour, the reinforced concrete bridge pylons are modelled using a simplified technique that incorporates the interaction between axial forces and bending moments as well as the non-linearity of the reinforced concrete. The model is then evaluated against experimental data.

Stay cable: There are 128 cables with different cross-section areas and pre-tensioning forces in the bridge. The stay cables are modelled with truss elements to accurately represent their behaviour, including material nonlinearity and thermal expansion effects, with failure criteria based on typical strain limits for the materials used.

The stay-cables consist of 15.7 mm diameter uncoated, seven-wire, weldless, low relaxation strands complying with the requirements of ASTM A416, Grade 270.

(fu) for the cables. Each cable was modelled with 10 truss elements to accurately represent the sag effect.

The bridge deck and post-tensioning strands: The bridge deck is modelled using fully integrated four-node isotropic shell elements. This approach allows for a comprehensive representation of the deck's behaviour under various loading conditions. The deck is connected to underlying steel girder members and floor beams through rigid links, ensuring a stable structural connection.

To simplify the modelling process, detailed reinforcement is not explicitly modelled. Instead, a simplified model is used to account for material nonlinearity. Post-tensioning strands are strategically placed at the centre of the main span and near auxiliary piers to

prevent cracking in the concrete deck. In the finite element model, these strands are represented by truss elements that share common nodes with the adjacent shell elements of the concrete deck. This integration ensures that the effects of post-tensioning are effectively transmitted to the deck. The model also considers thermal expansion effects, which are crucial for accurately simulating the behaviour of the bridge under varying temperature conditions.

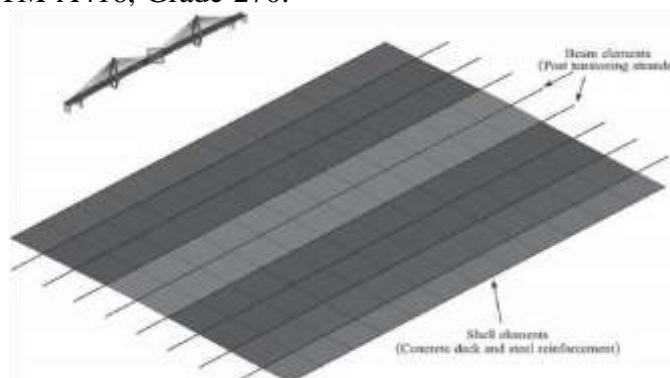


Fig.10:-Bridge deck model with post-tensioning strands
(Source: Chen et al., 2023)

Elastomeric bearings: These are used to connect the main girder members of the bridge to the pylon legs. They allow for

vertical and horizontal movements while providing support to the structure. The design of these bearings is crucial for

accommodating thermal expansion, vibrations, and other dynamic loads that

the bridge may experience during its lifespan.

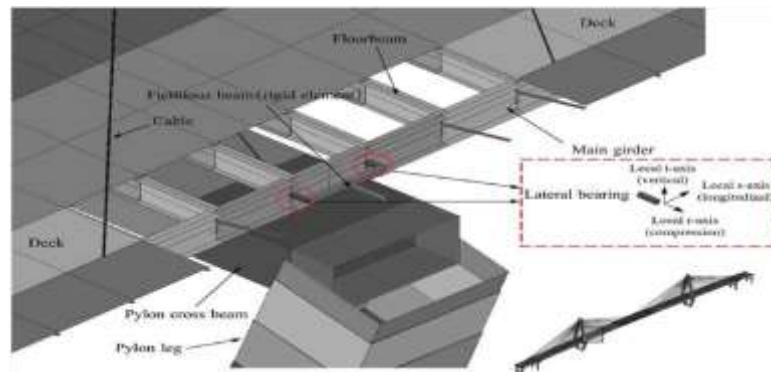


Fig.11:-Lateral elastomeric bearings connecting girder members to pylons.
(Source: Chen et al., 2023)

Dead and Live Load Analysis in the Bridge: Dead and live load analysis is a crucial aspect of bridge design, particularly for long-span cable-stayed bridges. The analysis is conducted in multiple stages to reflect the actual construction sequence of the bridge. This approach is necessary because achieving the same sequence in Finite Element Modelling (FEM) can be challenging and time-consuming. The dead load (DL) analysis was simplified in the FEM to facilitate this process.

Dead Load Initialization: Initialization of the dead load condition occurs in two stages.

Stage 1 ($t = 0-8$ s): During this stage, pretension forces in the cables and the self-weight of the main structures are

applied. A global damping of 80% of critical damping is used to prevent excessive vibrations during this phase.

Stage 2 ($t = 8-20$ s): The stiffness of the post-tensioning strands is increased to its normal value, and post-tensioning forces are applied. This stage also employs large global damping to mitigate numerical issues related to vibrations.

Live Load Application: Stage 3 ($t = 20-30$ s): After the completion of the first two stages, live load is applied. Similar to previous stages, a high level of global damping is maintained to avoid spurious failure modes due to vibrations when the live loads are introduced. This analysis is essential for understanding how the bridge will behave under real-world loading conditions.

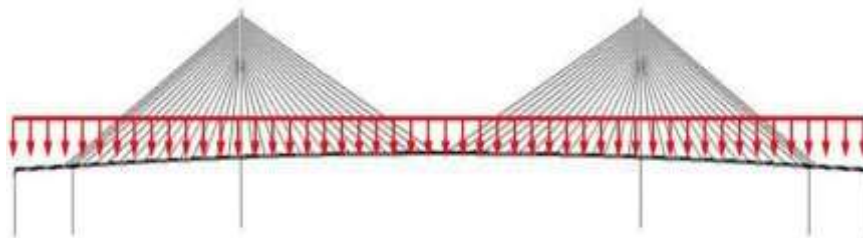


Fig.12:-Live load case
(Source: Chen et.al)

Member naming scheme: The member naming scheme is essential for identifying and categorizing the various cables in a cable-stayed bridge. The cables in the

bridge are divided into four distinct zones based on their location and connection to the pylons.

Zone 1: Cables N.01 to N.32 are connected to the west pylon in the north plane. These cables are not in the cable removal plane but are connected to the pylon from which the cables were removed.

Zone 2: Cables N.33 to N.64 are connected to the east pylon in the north plane. Similar to Zone 1, these cables are not in the cable removal plane and are not connected to the pylon from where the cables were removed.

Zone 3: Cables S.01 to S.32 are connected to the west pylon in the south plane. These cables are in the cable removal plane and are directly connected to the pylon from which the cables were removed.

Zone 4: Cables S.33 to S.64 are connected to the east pylon in the south plane. These cables are also in the cable removal plane but are not connected to the pylon from where the cables were removed.

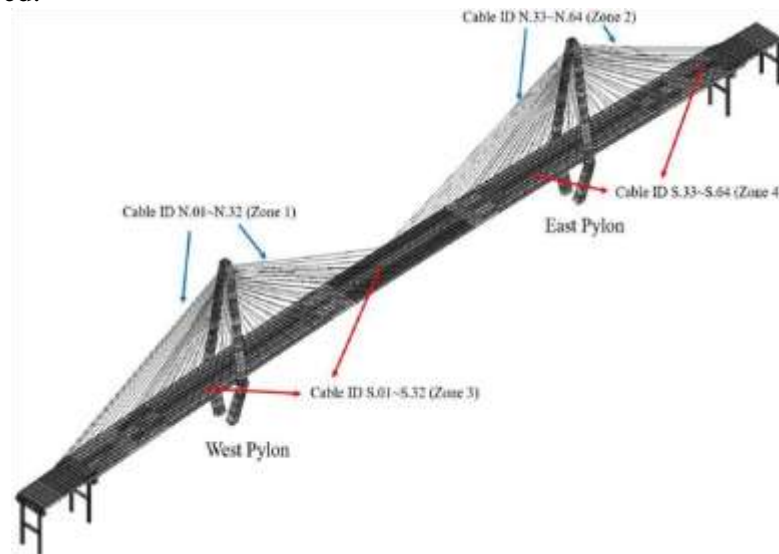


Fig.13:-Cable ID designation
(Source: Chen et.al, 2023)

Simulation Stages and Damping Effects in Cable-Stayed Bridge Analysis:

The simulation of the progressive collapse behaviour of a long-span cable-stayed bridge involves several critical stages, each with specific damping effects.

Stage 1 ($t = 0-8$ s): The simulation begins with the application of pretension forces in the cables and the self-weight of the main structures. This stage sets the initial conditions for the bridge under dead load (DL).

Stage 2 ($t = 8-20$ s): During this stage, the stiffness of the post-tensioning strands is increased to its normal value, and post-tensioning forces are applied. A high global damping of 80% of critical damping is maintained to prevent

numerical issues due to excessive vibrations.

Stage 3 ($t = 20-30$ s): Live load (LL) is applied after the bridge reaches equilibrium under the dead load. The high damping continues to mitigate any spurious failure modes that could arise from the sudden application of live loads.

Stage 4 ($t = 30-35$ s): The bridge is subjected to normal damping (2% of critical damping) after the vibrations from the load application subside. This stage prepares the bridge for the next critical event.

Stage 5 ($t = 35-45$ s): A single cable is suddenly removed at $t = 35$ s, causing the bridge to vibrate. This stage lasts for 10 seconds, allowing for the capture of the peak response of the bridge.

Stage 6 ($t = 45\text{--}65$ s): After the initial vibrations, the global damping is increased again to a high value (80% of critical damping) to quickly damp out the vibrations and allow the bridge to reach a new steady state.

Response Phases: The bridge's response to cable loss can be categorized into three phases:

Steady-State Response (S_{intact}): The initial response of the intact bridge.

Peak Damage Response ($S_{\text{damage peak}}$): The maximum response following cable loss.

New Steady-State Response ($S_{\text{damage steady}}$): The response after vibrations are damped out.

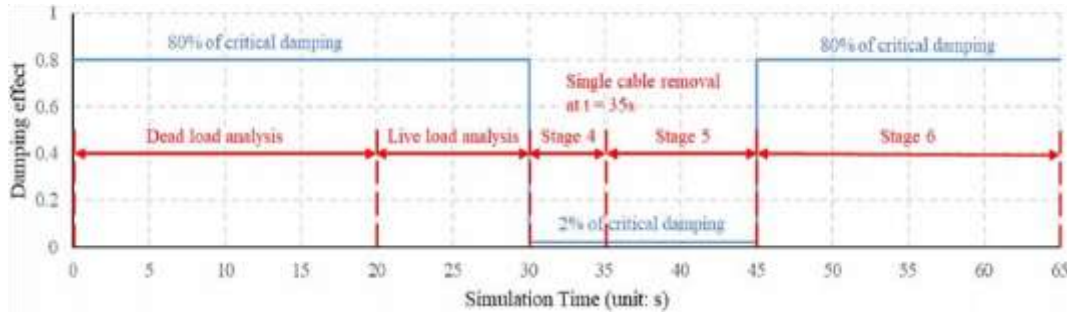


Fig.14:-Simulation stages and damping curve for single cable removal analysis
(Source: Chen et al., 2023)

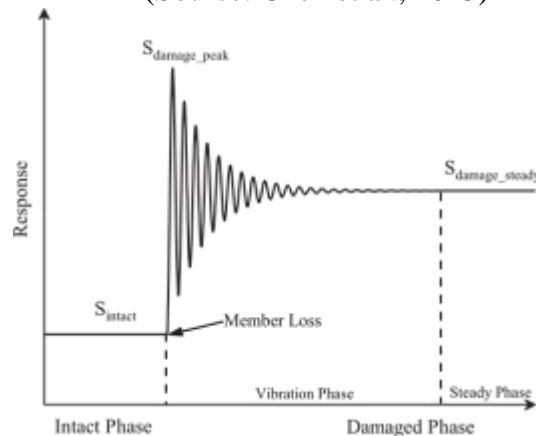


Fig.15:-Typical structure response time history under single-member loss
(Source: Chen et al., 2023)

Indices

6.3.1 Demand to Capacity Ratio (DCR): DCR is defined as the ratio of the demand (stress) on a structural member to its capacity (yield stress). It is calculated using three specific conditions: DCR intact, DCR damage peak, and DCR damage steady.

The study found that most cables in the intact bridge had a DCR of approximately 0.45, with some cables near the pylons showing lower values of around 0.35. This indicates that the bridge was operating well within its capacity before any cable loss.

After the sudden removal of cables, the $DCR_{\text{damage peak}}$ for certain cables increased significantly which indicates a temporary increase in stress. For example, cable S.18's $DCR_{\text{damage peak}}$ rose from 0.31 to 0.58, demonstrating the bridge's reserve capacity after a cable loss event.

Once the vibrations from the cable loss dissipated, the bridge reached a new steady-state condition. The $DCR_{\text{damage steady}}$ showed that most cables returned to stress levels similar to the intact state, indicating limited effects on cables in other zones after a single cable loss.

Notably, the maximum $DCR_{\text{damage steady}}$ was observed in cable S.22, with a value

of 0.60, suggesting that the bridge maintains a robust performance even after the loss of a cable. Overall, the DCR analysis indicates that the bridge is quite robust against the sudden removal of a single cable, as all cables remained within their elastic range, confirming the structure's resilience to such scenarios.

6.3.2 Dynamic Amplification Factor (DAF): DAF is defined as the ratio between the dynamic response of a structure and its static response. It helps in understanding how much the dynamic effects amplify the structural response compared to static conditions.

Various equations have been proposed to calculate DAF, especially for cable-supported structures experiencing sudden cable loss. However, some methods may yield unrealistic values, particularly when localized damage occurs, leading to small denominators in the calculations

All four zones showed a similar trend for DAF overall. The envelope of DAF in the four zones varied from 1.05 to 1.25, 1.03 to 1.15, 1.06 to 1.23, and 1.03 to 1.19, based on 32 typical cable loss events. Since the cables in Zones 1 and 3 are connected to the same pylon from which cables were disconnected, the DAF values in those zones were marginally higher than those in Zones 2 and 4. Furthermore, in each of the four zones, the DAF was somewhat bigger in the cables close to the bridge pylons. Analysis of the DAF results along with the low DCRs observed earlier confirm that the bridge is relatively insensitive to single cable removal.

Behaviour of the bridge under multiple cable loss events

The bridge's response to multiple cable loss scenarios was analyzed under dead load (DL) and live load (LL) conditions. The global damping constant was initially set to 2% of critical damping before the first cable was removed, simulating realistic conditions during the event.

After the first cable was removed, the damping constant was increased to 80% of critical damping for a duration of 10 seconds to capture the peak response. Once the bridge reached a new steady state, the damping was reduced back to 2% before the next cable was removed. This process was repeated for subsequent cable loss scenarios.

Two multiple cable loss scenarios were analysed, termed Cable Loss Scenario 1 (CLS 1) and Cable Loss Scenario 2 (CLS 2). Each scenario represented different failure sequences in various parts of the bridge, allowing for a comprehensive evaluation of the bridge's behaviour under stress. Three limit states were established to evaluate damage levels:

Functionality Limit State: Defined by tension cracks in 10% of the deck area or additional deflections reaching $L/400$.

Member Failure Limit State: Occurs when a main structural member reaches its yield point.

Ultimate Limit State: Characterized by a fracture in a main structural member or complete bridge collapse.

Progressive collapse behaviour and failure modes during CLS 1

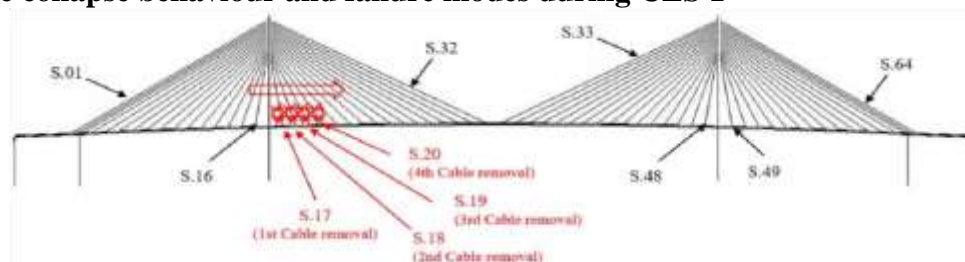


Fig.16:-Cable removal pattern of CLS 1
(Source: Chen et al., 2023)

In CLS 1, the first member failure limit state was reached after the loss of the third cable. This indicates that the bridge began to experience significant structural distress at this point, highlighting its vulnerability to cable loss.

The bridge suffered a partial collapse after the loss of the fourth cable. This suggests that the removal of just a few cables can lead to severe consequences, emphasizing

the importance of cable integrity in maintaining overall bridge stability.

The failure modes observed during CLS 1 included instability-type partial collapse, which occurred as the bridge could not redistribute loads effectively after the loss of critical cables. This type of failure is characterized by sudden and significant deformations, leading to a rapid decline in structural integrity.

Progressive collapse behaviour and failure modes during CLS 2

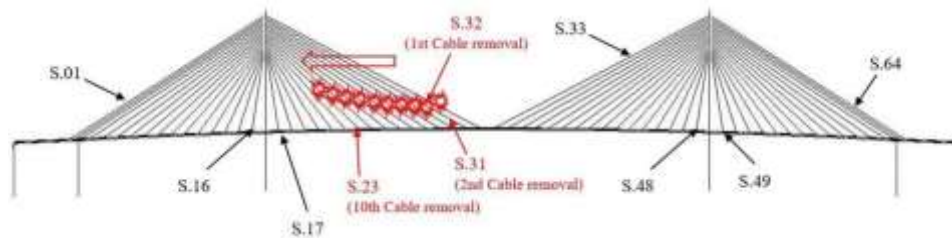


Fig.17:-Cable removal pattern of CLS 2
(Source: Chen et al.,2023)

In CLS 2, the functionality limit state was reached after the loss of the second cable. This indicates that the bridge began to show signs of damage relatively early in the cable removal process, suggesting a lower tolerance for cable loss compared to CLS 1. The first member failure was recorded after the loss of the third cable. Despite this, the bridge exhibited relatively small deformations and maintained robust behaviour until the loss of the fifth cable, at which point severe damage was noted.

The analysis revealed that the bridge was more tolerant to cable loss near its centre span, where it could withstand the loss of up to ten cables, compared to only four cables near the pylon before triggering a progressive collapse. This highlights the importance of cable location in assessing structural resilience. The findings from CLS 2 suggest that the bridge's design must account for the critical nature of cable loss, particularly near the pylons.

RESULT

Two types of indices (DCR and DAF) were utilized to assess the impact of cable loss on system performance, and a variety of single cable loss scenarios were modelled. The simulation findings demonstrated that the dynamic impacts associated with a single cable removal scenario largely affected structural elements in the region of the removed cable, particularly the adjacent cables in the same cable plane. After removing a single cable, the DCRs of the cables in the damaged bridges demonstrated that all of the cables and other major steel structural elements were still within their elastic range. The bridge was found to be fairly resilient to this kind of situation.

It also looked into how the bridge behaved under various scenarios of cable loss. In particular, the method used for removing a single cable was applied to the removal of each cable one at a time. The bridge was closely observed at each stage of the cable removal process, with particular attention paid to the way the important

structural parameters such as the bridge's vertical displacement, the stress in the main girder, and the stress in the neighbouring stay cables reacted.

Following the loss of the third cable, CLS 1 hit its first member failure limit state. Following the loss of the fourth cable, part of the bridge collapsed. Following the loss of the second connection, CLS 2 reached its operational limit condition, and the first member failure was noted following the loss of the third cable. Deformations were minimal, though, and the bridge behaved robustly up until the loss of the fifth cable, at which point significant damage was discovered. The bridge would fall by an unzipping form of progressive collapse, which involved progressive rupture of the neighbouring cables, once five more cables, or up to the tenth cable, were lost.

CONCLUSION

The study on the progressive collapse behaviour of a long-span cable-stayed bridge revealed that the structure demonstrated considerable robustness against the sudden loss of a single-stay cable. The simulations indicated that the DCR for the remaining cables and structural components remained within safe limits, suggesting that the bridge could endure such an event without immediate failure. However, the analysis also highlighted that the bridge's resilience significantly decreased when subjected to the simultaneous

loss of multiple cables, particularly near the supporting pylons. This area exhibited a critical vulnerability, leading to a partial collapse after the removal of the fourth cable, which underscores the importance of understanding the implications of cable loss in bridge design and safety assessments.

Furthermore, the study identified that the failure mode of the bridge, following multiple cable losses, was characterized by an unzipping type of progressive

collapse. This failure mechanism was more pronounced near the pylons compared to the midspan, where the bridge showed greater resistance to cable loss. The findings emphasized the necessity for engineers and designers to consider critical cable loss locations and their effects on overall structural performance. By understanding these dynamics, future designs can be improved to enhance the safety and resilience of cable-stayed bridges against potential progressive collapse scenarios.

INTERPRETATION BY AUTHOR

Progressive collapse of bridges can lead to significant loss of life and economic damages. The collapse can be due to various factors. So it is necessary to highlight the importance of preventive measures and robust design strategies and to create awareness regarding progressive collapse in high-rise buildings and long-span bridges.

The investigation into bridge collapses and the factors influencing them reveals that localized failures can trigger partial or complete structural failures. This has gathered significant interest from researchers and engineers, indicating a need for improved understanding and mitigation strategies for such failures.

In summary, this study underscores the importance of understanding the mechanisms behind structural failures, the need for rigorous monitoring and evaluation, and the implementation of effective design strategies to prevent progressive collapses in critical infrastructure. It also discusses in detail the case of progressive collapse due to cable loss in long-span cable-stayed bridges. The case study highlights that certain areas of the bridge, particularly near the supporting pylons, exhibit critical vulnerabilities when multiple cables are lost. This insight emphasizes the need for targeted design strategies to reinforce

these weak points to prevent progressive collapse.

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