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(Research Article)

Progressive collapse and soil stress analyses of Manupali steel truss bridge in Lantapan, Bukidnon under hydraulic and traffic threats

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Abstract

Bridges designed to comply with current design codes may not provide enough strength to arrest a possible local failure following an abnormal event, thus leading to a progressive collapse. One may employ different approaches that vary according to linearity and dynamicity to analyze the potential of a structure to progressive collapse. Together with a prior risk assessment, this study analyzed the susceptibility of the Manupali steel truss bridge to progressive collapse using an adapted nonlinear static approach and assessed the bridge's pile foundation capacity concerning changes in the imposed stresses caused by the dynamic effects of progressive collapse. Using P-Delta analysis in STAAD.Pro CONNECT Edition V22, this study determined that under its self-weight, the Manupali steel truss bridge is susceptible to progressive collapse, i.e., it is fracture critical. Nonetheless, being such does not mean the bridge is inherently unsafe, only that it lacks redundancy in the design. Furthermore, this study found that a collapse does not significantly detrimentally affect the foundation system except when the dynamic effect is so tremendous. With the results, this study recommends that the design of bridges should consider abnormal load cases to mitigate progressive collapse.

Keywords: Progressive Collapse; Steel Truss Bridge; Nonlinear Static; Dynamic Amplification Factor; Demand-Capacity Ratio; Hydraulic Threats

1. Introduction

Structural systems optimized and proportioned to meet member design criteria as specified in current design standards may not provide sufficient robustness to withstand a possible local failure following an extreme event [1]. Local failure of one structural element may cause the failure of another, creating a chain reaction of failures that progresses throughout the structure leading to a catastrophic collapse [2].

Progressive collapse happens to a structure when a critical member fails, leading to the propagation of collapse from the local point of failure to the neighboring or overall structural members. Members' failure may result from abnormal loadings such as vehicular impacts, bomb explosions, fires, and large hydrodynamic forces [3]. Assessing the potential of a structure to progressive collapse is critical since it may offer a chance to revise the design or introduce measures for more safety. Since progressive collapse is a rare scenario, no database is comprehensive; hence, it is instead fitting to perform a risk assessment of structures before proceeding to the analysis.

To analyze the potential of a structure to progressive collapse, one may employ the following approaches: linear static, nonlinear static, linear dynamic, and nonlinear dynamic. However, using dynamic and nonlinear methods, although more exact and accurate, is very complex and time-consuming. Thus, for practicality, the linear static approach commonly supersedes [4].

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Aside from the structural aspect of a progressive collapse, it is also fitting to assess the geotechnical perspective. In a progressive collapse, there will be changes in the load paths that might alter the subsequent loads on the foundation system of the structure. These changes in loading, in addition to an inherent change in magnitude due to the dynamic nature of the collapse, might cause stress deviations in the soil supporting the structure.

Engineers must evaluate the potential of bridges to progressive collapse as they form critical links in our infrastructure network and play an essential role in economic development. Thus, the primary purpose of this study lay primarily in conducting progressive collapse and soil stress analyses of the Manupali steel truss bridge in Lantapan, Bukidnon, under hydraulic and traffic threats.

1.1. Statement of the Problem

With the typical usage of current design guidance in bridge construction that does not directly consider alternate load paths, the potential of the Manupali Bridge in Lantapan, Bukidnon, to progressive collapse was not considered in detail during the design process. The bridge was built over a river and is susceptible to extreme hydraulic threats (i.e., flood forces, scouring, and debris impact), which may cause the failure of the bridge's pier beyond its design capacity. Thus, there will be changes in the load paths that might alter the subsequent loads on the foundation system of the structure. Considering the traffic loading, unforeseen events such as vehicular overloads may cause unwanted truss member failure, which may lead to the propagation of collapse from the failed member to the neighboring or overall structural members. The failure of the bridge under study may result in the death of users and will disrupt economic activities; therefore, analyzing the bridge's potential for progressive collapse was deemed necessary.

Objectives of the Study

This study aimed to conduct progressive collapse and soil stress analyses of the Manupali steel truss bridge in Lantapan, Bukidnon, under hydraulic and traffic threats.

Specifically, it aimed to:

a. conduct a risk assessment of the Manupali steel truss bridge in Lantapan, Bukidnon considering hydraulic and traffic threats;
b. analyze the susceptibility of the bridge to progressive collapse using a modified nonlinear static approach; and,
c. assess the bridge's pile foundation capacity concerning changes in the imposed stresses due to progressive collapse.

1.2. Significance of the Study

This study provides information regarding the progressive collapse potential of the Manupali steel truss bridge in Lantapan, Bukidnon considering independently treated abnormal loading conditions from hydraulic and traffic threats. Furthermore, this study determined the critical members of the bridge's structure, thereby providing relevant knowledge for monitoring and mitigation purposes.

1.3. Scope of the Study

This study focused on the progressive collapse and soil stress analyses of the Manupali Bridge. The procedure employed a modified nonlinear static approach patterned after the proposed procedure by Khuyen [4]. The load considered during the progressive collapse was limited to the dead load since this study did not consider the extent of loading at which the members would fracture but only how the bridge would collapse at a given load. Analyzing only an existing construction, this study did not propose a plan for a redesign [2]. Nonetheless, it offered a basic recommendation on how to mitigate progressive collapse in the design of steel bridges.

The mathematical model of the bridge was based on the technical drawings provided by the Department of Public Works and Highways – Region X. Regarding risk assessments, this study considered only flood forces, scouring, debris impact, and moving overloads.

1.4. Location of the Study

The bridge under study is located at Brgy. Balila, Lantapan, Bukidnon, Philippines (7.989841°N, 125.009681°E) as shown in Figure 1a. Figure 1b shows a photo of the Manupali bridge, a Warren steel truss through bridge having two symmetric but not continuous spans, each having a length of 60 m center-to-center of supports and a total length of 120 m.

Figure 1a Location of Manupali bridge in Lantapan, Bukidnon

Figure 1b Manupali steel truss bridge

2. Literature Review

2.1. Historical Background of Progressive Collapse

Progressive collapse first intrigued the attention of engineers when in 1968, Ronan Point in London, a 22-story apartment building, collapsed [5]. Also, the events of September 11, 2001[6], which caused the collapse of the Twin Towers in New York, US, are another milestone in the research and new design measures to resist the progressive collapse of buildings. The incidents compelled several researchers to focus on the causes of progressive collapse in building structures, seeking to establish rational methods for assessing and enhancing structural robustness under extreme events [3]. The 9/11 attack also urged the increasing enforcement of new design guidelines to prevent the progressive collapse of different types of structures. As a structural engineer, it is imperative to guarantee that sufficient measures in the design process of a structure have been made to prevent progressive collapse. An engineer should also be able to analyze a structure's progressive collapse potential using appropriate procedures and analysis software.

2.2. Definition of Progressive Collapse

ASCE [7] defines progressive collapse as the spread of an initial local failure from member to member, consequently resulting in the collapse of an entire structure or a disproportionately large part. It has recently become evident that abnormal loads need to be considered in the design of structures so that progressive collapse can be prevented [8]. Progressive collapse is a structural failure initiated by localized structural damage and subsequently develops, as a chain reaction, into a failure that involves a significant portion of the structural system. From an analytical viewpoint, a progressive collapse is a dynamic event, and the motion is initiated by a release of internal energy due to the rapid loss of a structural member. This member loss disturbs the initial load equilibrium of external loads and internal forces, and the structure then vibrates until either a new equilibrium position is found or the structure collapses [9].

Probabilistically, Miao and Ghosn [1], posited that the following equation can represent the progressive collapse process:

$$P(C) = \sum_{H} \sum_{D} P(C|D)P(D|H)P(H) \qquad (1)$$

where P(C) is the probability of system collapse, P(H) is the probability of occurrence and intensity of hazard H; P(D/H) is the probability of local structural damage scenario D, given the occurrence of the damage-initiating hazard H, and P(C/D) is the probability of structural collapse given an initial damage scenario D.

2.3. Failure of Bridges

Bridge failure is defined as the incapacity of a bridge to perform as specified in the design and construction requirements [10]. In a review written by Zhang et al. [10] on the typical characteristics and causes of bridge failures based on ten former investigations in the literature, the five principal causes of bridge failure were design error, construction mistakes, hydraulic, collision, and overload, resulting in more than 70% of the bridge failures. Of the five, the hydraulic and overload threats are considered external to the bridge.

The causes of failures are closely related to the regional economy, structural type, usage, material type, and service age. The failure rate is high for steel bridges, inseparable from excessive emphasis on strength but lack of consideration of structure stability and fatigue in the early years. Extreme loads like floods, collisions, and overloads lead to many bridge failures due to the lack of extreme loads database and design theory defects. Such bridges must have sufficient redundancy and capacity protection to reduce the probability of bridge failure due to extreme loads [10].

2.4. Hydraulic Threats

Observation shows that a surprising number of bridges were destroyed by hydraulic threats [11]. Heavy precipitation usually leads to flooding, which can collapse a bridge in a few different ways, such as scour, sand missing, softened bedrock, erosion, insufficient embedment depth, river convergence, debris impact, or abrasion on bridge foundations [12, 13, 14].

The causes of bridge failures caused by hydraulic factors are mainly classified into natural factors and human factors. For natural factors, rivers in some areas suffer from rare catastrophic floods, which exceed the ultimate limit state, thus causing the bridges to fail. For human factors, the most critical factors are the lack of hydrological data upon which to base estimates of the magnitude of floods for design purposes, the lack of reliable methods for estimating scour at bridge piers, and the inability to predict the occurrence of impact and accumulation of debris against the bridge structure [10].

To protect bridges from hydraulic threats, designers should select proper bridge sites, arrange bridge spans properly, and ensure adequate foundation depth. Then, the bridge regulation and protection projects should be improved, and the direction of flood flow should be adjusted to reduce the impact and erosion of floods on bridges. Finally, bridge maintenance work should be strengthened, and foundation scouring maintenance should be included in the preventive maintenance category [10].

2.5. Moving Overloads

With the increase in traffic volume, the truckloads exceeded the limitations, resulting in bridge failures, especially for older bridges. Such bridge failures are common in developed countries such as the United States and European countries [15, 16]. Cook et al. [17] found that the average service age of failed bridges in the United States due to overload was about 64 years. Besides, due to the increasing competition in the transportation market, vehicle overload has become increasingly common and has raised serious concerns in developing countries such as China. In addition, overloads may contribute to an acceleration of fatigue damage to steel bridges [11]. Lee et al. [15] studied 135 bridge failures caused by overloads, in which steel bridge failures dominated approximately 64% while concrete bridge failures dominated approximately only 11%. That was because most steel bridges were built earlier than concrete bridges in the United States. Thus, steel bridges' bearing and overload capacities got much lower than that of concrete bridges.

We cannot improve the live load level to meet overload demand, which will cause a significant waste of resources. Overload should be treated as an extreme event. On the one hand, the laws and regulations must be observed and strictly enforced, and those who break the laws must be prosecuted. On the other hand, bridges must have sufficient redundancy and capacity protection to reduce the probability of bridge failure due to overload [10].

2.6. Linear Static Progressive Collapse Analysis Procedure

The following is the procedure for progressive collapse analysis of steel truss bridges using the linear static approach proposed by Khuyen [4].

The first step is to create the analytical model and evaluate the bridge's safety before any initial fracture. Before analysis, a bridge inspection review should be conducted to obtain data for finite element modeling, such as physical data, geometrical data, and data on corrosion, deterioration, and prior retrofits. The first step checks the performance of the intact bridge under current information on the geometry and other conditions before assumptions of any sudden breakage of members. The bridge loadings are assumed to be the combination of dead load and live load 1.0D+μL with μ as an attributed factor varying from 0 to 1.0.

The second step assumes the fractured scenarios of members. These scenarios are the candidates of fractured critical members (FCM), which are usually in tension, causing one or some remaining members to yield due to their loss. The fractured member is removed from the original analytical model created in the first step. In an actual bridge, a damage scenario can appear on more than one member; however, the probability of the presence of two or more member fractures is much lower than the probability of a single-member fracture. In addition, the case of more than one member fracture can be a combination of single-member fractures. For each target member fracture and to address the dynamic effect of a member's sudden failure, the dynamic amplification factor (DAF) is calculated as

$$DAF = 0.255\, max\left(\frac{\sigma_{is}}{\sigma_{iy}}\right) + 1.00 \qquad (2)$$

where $\frac{\sigma_{is}}{\sigma_{iy}}$ stands for the ratio of the stress in static analysis to the yield strength of the corresponding member.

The third step is to conduct a linear static analysis of the damaged bridge model by removing fractured candidates in the second step. The safety of the structure is evaluated by checking the demand-capacity ratio (DCR) by the following equations:

$$DCR_T = \frac{N}{N_p} + \frac{M_x}{M_{px}} + \frac{M_y}{M_{py}} \qquad (3)$$

$$DCR_C = \frac{P}{P_u} + \frac{1}{1-\frac{P}{P_{ex}}}\frac{M_{eqx}}{M_{px}} + \frac{1}{1-\frac{P}{P_{ey}}}\frac{M_{eqy}}{M_{py}} \qquad (4)$$

In the above equations, N and P are the axial tensile and compressive forces, respectively, and M_x and M_y are the bending moments around the strong and weak axes, respectively. N_p, M_{px}, and M_{py} are the plastic axial and full plastic moment strengths around the strong and weak axes, respectively. P_e is the Euler buckling load, and P_u is the ultimate compressive strength associated with global buckling. The equivalent uniform moments, M_{eqx} and M_{eqy}, are used to convert the linear distributed moment state into a uniform moment condition. The equations hold for tension and compression members, respectively. If at least one member fails in the demand-capacity ratio (i.e., DCR ≥ 1.0), the bridge collapses. In that case, the fractured target member is an FCM.

3. Methodology

3.1. Securement of Plan

The Manupali bridge's structural plan was duly obtained from the Office of the Department of Public Works and Highways – Region X, Philippines. The researchers submitted a communication letter to the Regional Director of DPWH Region X to request a complete copy of the Manupali Bridge's plan and the corresponding geotechnical properties of the soil. Upon compliance with the provisions of Executive Order No. 2 series of 2016, also known as the Freedom of Information Order, the Office, through the Assistant Regional Director, provided the copy of the plan and the geotechnical data of the bridge.

3.2. Risk Assessment

The concept of risk involves hazards and consequences [3]. The hazard is the triggering event (e.g., flood forces, scouring, debris impact, and overloads), while the consequences are the results caused by the hazard (e.g., collapse, personal injury, and loss of life).

Due to the limited database of progressive collapse events, it is difficult to assess the probability of occurrence of hazards; therefore, it is more reasonable to make a risk assessment. Threat or hazard identification precedes risk assessment [3]. In this study, the hazards identified are hydraulic and traffic threats, which are external causes of bridge collapse [10].

3.2.1. Flood Forces

Hydrodynamic loads result from water flowing against and around a rigid structural element or system. ASCE 7-16 Section 5.4 states that the dynamic effects of moving water shall be calculated by a detailed analysis using basic concepts of fluid mechanics. Such analysis is governed by utilizing an equivalent surcharge depth, d_h, that is derived by converting the dynamic effects of moving water into equivalent hydrostatic loads [18]. The equivalent surcharge depth shall be added to the design flood elevation as a uniformly distributed load across the structure's vertical projected area and perpendicular to the flow. The equivalent surcharge depth is expressed as:

$$d_h = \frac{aV^2}{2g} \qquad (5)$$

where a is the drag coefficient or shape factor (not less than 1.25), V is the average velocity of water, and g is the gravitational acceleration.

3.2.2. Foundation Scouring

Scour refers to the erosion of a streambed or bank material from bridge foundations due to flowing water, usually considered as long-term bed degradation, contraction, and local scour [15, 19]. The effect of scouring on bridges is considered here through the estimated maximum depth of scour holes formed around bridge piers during a flood event. It is assumed that bridge abutments are well-protected against scour. Three types of scouring affect bridge piers: contraction, degradation, and local scour. Contraction results from faster flow velocities where river width gets narrower due to a natural contraction of the stream channel or a bridge. Degradation is caused by the long-term erosion of the riverbed upstream and downstream of a bridge. Local scour at piers develops due to the formation of vortices at pier bases. Within the scope of this study, sole concentration is given to the local pier scour induced by flood events, and contraction and degradation types of scours are ignored. For this reason, hereafter, the term 'scour' only refers to local pier scour.

In the present research, scour depths (y_s) at bridge foundations are estimated using the following equation suggested by the HEC-18 [20]:

$$y_s = 2.0 y_1 K_1 K_2 K_3 (\frac{a}{y_1})^{0.65} Fr_1^{0.43} \qquad (6)$$

where y_1 is the flow depth directly upstream to the bridge pier; a is the pier width; K_1, K_2, and K_3 are correction factors for pier nose shape, angle of attack of flow, and bed condition, respectively. Fr_1 is the Froude number defined by $V/\sqrt{(gy_1)}$, where V and g represent the mean velocity of the upstream flow and gravitational acceleration, respectively.

3.2.3. Debris Impact

Impact loads result from debris and any object transported by floodwaters striking against structures or parts thereof. Impact loads shall be rationally determined as concentrated loads acting horizontally at the critical location at or below the design flood elevation.

To calculate the impact force, one can use the impulse-momentum theorem, whereby the following equation is derived [18]:

$$F = \frac{Wv}{t} \qquad (7)$$

where W is debris mass (recommended as 450 kg), v is the object's velocity assumed equal to water velocity, and t is the time the object takes to stop.

3.2.4. Traffic Threat

The bridge's potential risk to traffic threat was also evaluated. The traffic threat considered in the assessment was moving overloads, i.e., the AASHTO design truck HS-20. A repeated sequence of this truck loaded the bridge until the bridge was found to fail.

The live load on the bridge is a moving load since it includes trucks moving along the bridge. Therefore, some positions of the trucks are the worst cases for the bridge, which can be exacerbated if the trucks are more than the limitations.

3.3. Modelling of Bridge

The structural framing system of the Manupali Bridge, as detailed in the structural plan, was modeled using Structural Analysis and Design (STAAD) Pro. CONNECT V22 for the simulation of progressive collapse.

Note that truss joints are conventionally idealized as pinned. However, large gusset plates, fasteners, rivets, and bolts allow the transfer of moments through these joints [4]. Hence, in this study's models, truss members and lateral bracings were assumed as rigidly connected to the truss joints. This modeling approach was validated by [21].

3.4. Determination of Loads

The Manupali bridge's performance before the assumptions of any member breakage was checked. The bridge members' loadings were a combination of dead load and live load (1.0D + μL), wherein μ varies from 0 to 1.0 [4]. In this study, μ was considered equal to 0 for a more simplified analysis since progressive collapse analysis is not strictly concerned with the extent of loading at which the members would fracture but only with how the bridge would collapse at a given load.

3.5. Determination of Dynamic Amplification Factor

The dynamic amplification factor (DAF) for each target member removal case was calculated using Khuyen's [4] formula to approximately capture the dynamic effect on the structure of a member's sudden failure:

$$DAF = 0.255\, max\left(\frac{\sigma_{is}}{\sigma_{iy}}\right) + 1.00 \qquad (2)$$

where $\frac{\sigma_{is}}{\sigma_{iy}}$ stands for the ratio of the stress in static analysis to the yield strength of the corresponding member.

3.6. Progressive Collapse Analysis using STAAD.Pro

The progressive collapse analysis of the bridge covers scenarios of truss member removals. The bridge's critical members were identified through simulation of the removal of the identified critical truss members one at a time. The member that, when removed, caused a progressive collapse was considered a critical member.

3.7. Checking of DCR Values

The robustness of the structure against progressive collapse was evaluated by checking each member's demand-capacity ratio by dividing the combined stresses by the corresponding yield stresses of the members. When the DCR exceeded 1.0, the member was considered a failure and thus notionally removed from the model.

3.8. Foundation Capacity Analysis at Abutment and Pier

Changes in the soil stresses of the bridge's foundation due to changes in load path were equivalently assessed by doing a foundation capacity analysis. In this step, pile capacity was checked first. Then, it was compared against the imposed reactions. If the pile capacity is lesser than the reaction, the pile is considered to fail. This approach is rationalized by noting that the pile capacity is merely a function of soil stresses, i.e., shear stress from the skin friction and normal stress from the end bearing [22].

4. Results and Discussion

This study analyzed the potential for progressive collapse of the Manupali steel truss bridge in Lantapan, Bukidnon. Using the technical drawings provided by DPWH Region X, the structural model of the bridge was analyzed in STAAD Pro. CONNECT Edition V22 using a modified nonlinear static method. By notionally removing a member from the model, the response of the bridge was determined whether there would be a sufficient alternate load path to arrest the failure. By incorporating P-delta analysis and dynamic amplification factors, the nonlinearity and dynamicity of the bridge's respective geometry and collapse were duly accounted.

Results showed that among the notionally removed members, only three were not critical to the bridge's collapse, irrespective of whether it was tensile or compressive. This observation is mainly due to the magnitude of stress the member was supposed to carry in a normal-load condition. If the member is a primary force-carrying member, then removal of the same would cause the bridge to find a way to redistribute the load. If the alternate load is insufficient, the bridge collapses.

Furthermore, a pile capacity analysis was performed on the bridge's pier and abutment. During the collapse, there would be changes in the reactions of the supports, thereby affecting the soil stresses. Equivalently, soil stress can be evaluated by comparing the reactions against the pile capacity. Whenever the reactions exceed the pile capacity, it denotes a high stress on the soil, thus causing the pile to fail. In other words, the pile capacity speaks for the soil stress.

This study also carried out a risk assessment on the bridge that is independent of the mentioned analyses. Using appropriate guidelines and methods, the hydraulic and traffic threats were assessed. The flood forces, scour, and debris impact were studied for the hydraulic part. On the other hand, the traffic threat considered was only moving overloads.

4.1. Hydraulic Threat: Flood Forces

Figure 2a shows the loads-and-moments interaction diagram of the bridge's pier considering flood forces. For the bridge to be considered safe, the action effects of the factored load moment (M_u) and axial load combination (P_u) must be less than the combination of flexural (ϕM_n) and axial (ϕP_n) design strengths. The primary safety criteria are per the provisions of the ACI 318-19 [23]: $\phi P_n \geq P_u$ and $\phi M_n \geq M_u$, as aided by the strength interaction diagram.

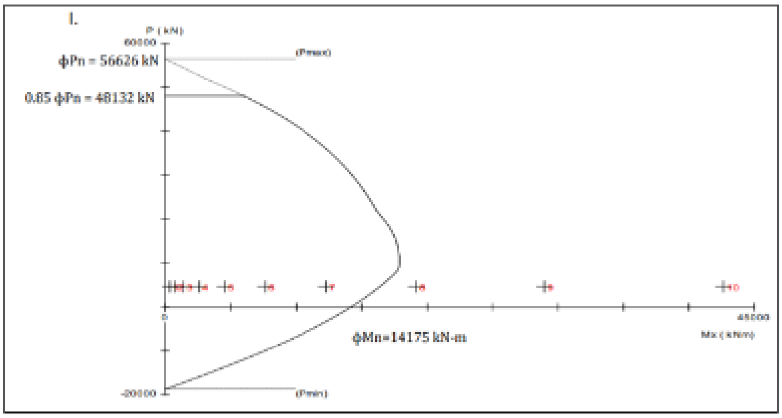

Figure 2a Strength interaction diagram of the bridge's pier with plotted values of load effects from axial and flood forces (generated using spColumn v6.00 Software)

Figure 2a further shows that the combination of the factored moment and axial loads indicated by the points 8 to 10 lies outside the design strength curve, deemed as not meeting the design criteria. The maximum axial design strength is equal to 56626 kN and is limited to 85 percent for spiral transverse reinforcement to account for accidental eccentricity, thus yielding an axial design strength at the pier equal to 48132 kN. Moreover, the maximum bending moment at the pier equals 14175 kN, which is considered a point of maximum flexural stress for the structure to be safe.

Figure 2b shows the different values of floodwater velocities with corresponding bending moments at the pier structure. The combination of the effect of axial load and bending moment was analyzed in spColumn software with the specified structure's section properties. The Pu-Mu values from points 1 to 7 lie inside the design strength curve, which indicates that a maximum of 13 meters per second floodwater velocity is considered safe. On the other hand, the combination of effects from points 8 to 10 lies outside the design strength curve region and is thus deemed unsafe. A floodwater velocity greater than 13 meters per second can exceed the ultimate limit state of the pier structure. However, a 13 m/s floodwater velocity is unrealistic and nearly impossible. Thus, the bridge was designed to have an above-average performance to withstand extreme flood events with minimal or no damage.

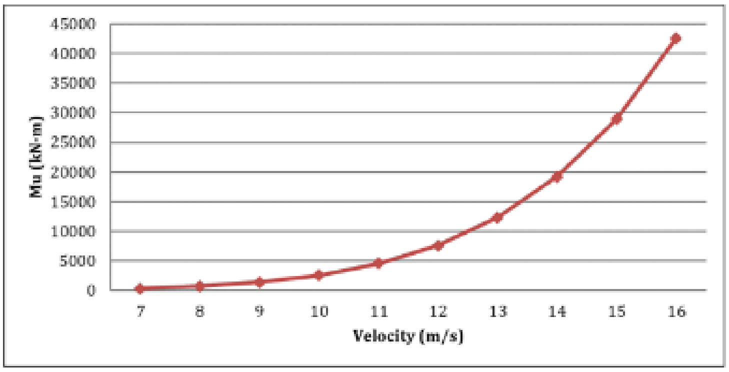

Figure 2b Floodwater velocities and the corresponding maximum bending moments at the pier structure

4.2. Hydraulic Threat: Scour

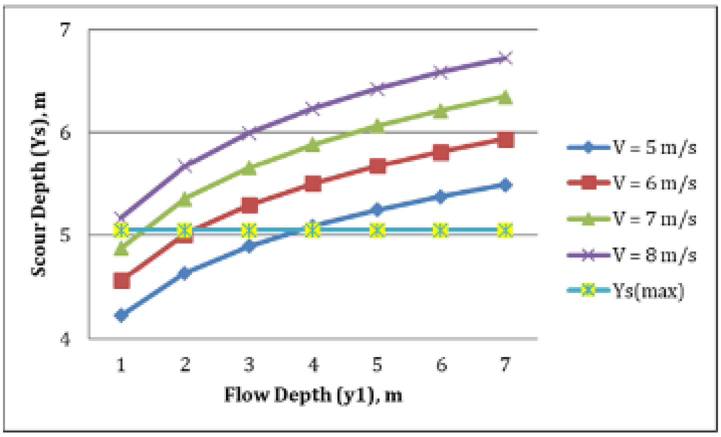

Figure 3 Scouring depths of various water flow heights and flow velocities

Scouring can lead to the development of vortices around the pier, thus causing the bridge to be prone to failure. The effect of scouring on bridges was considered through the estimated maximum depth of scour holes formed around bridge piers during a flood event. Scour depths at the bridge pier were estimated using the equation suggested by the HEC-18 document.

Figure 3 shows the estimated scour depths of assumed water flow velocity and flow height values. The design velocity of 5 m/s and design flood elevation of 2.5 m determined from the bridge's technical plan was used to estimate the maximum scour depth.

The correction factors used for pier nose shape (K_1), angle of attack of flow (K_2), and bed condition (K_2) are 1.0, 1.0, and 1.1, respectively. Based on the design data, the maximum scour depth at the bridge's pier is equal to 5.054 m.

The water flow velocity and flow depth directly affect the scour depth at the pier. The greater the velocity of water flow and flow depth, the higher the value of scour depth. Moreover, the scour depth also depends on the width of the pier nose shape, angle of attack of flow, and bed condition. With an increase in scour depth, the lateral resistance of the soil supporting the foundation is significantly reduced, thus increasing the lateral deflection of the foundation head. In addition, when the critical scour depth is reached, bending buckling of the foundation may occur under the combined effect of a dead load of bridge superstructures and traffic load [24].

Furthermore, there is a high probability that scouring is affected by whether the flow is subcritical or supercritical. Froude number with less than one is classified as subcritical, while if it is equal to 1.0, it is considered critical, and if it is greater than one, it is supercritical. Thus, scour almost certainly depends on whether the flow is subcritical or supercritical.

As seen in figure 3, water flow velocities of 5, 6, 7, and 8 m/s with flow depths of 4, 3, 2, and 1 m, respectively, start to exceed the maximum estimated scour depth. Thus, these values could cause the occurrence of critical scour depths.

4.3. Hydraulic Threat: Debris Impact

The pier was subjected to combined axial loads and bending moments from its self-weight, reactions at the pier, and different impact loads from an assumed 450 kg of debris. The object's velocity was assumed to be equal to water velocity, and the time the wood debris stop is assumed equal to 1.0 s.

From figure 4a, the combined factored axial loads and moments from points 8 to 10 exceed the ultimate design strength of the pier. The values lie outside the structure's strength interaction curve, which is considered unsafe for the structure and may have the potential risk of bridge failure. On the other hand, the combined axial loads and moments of points 1 to 7 are considered safe and do not exceed the structure's design strength.

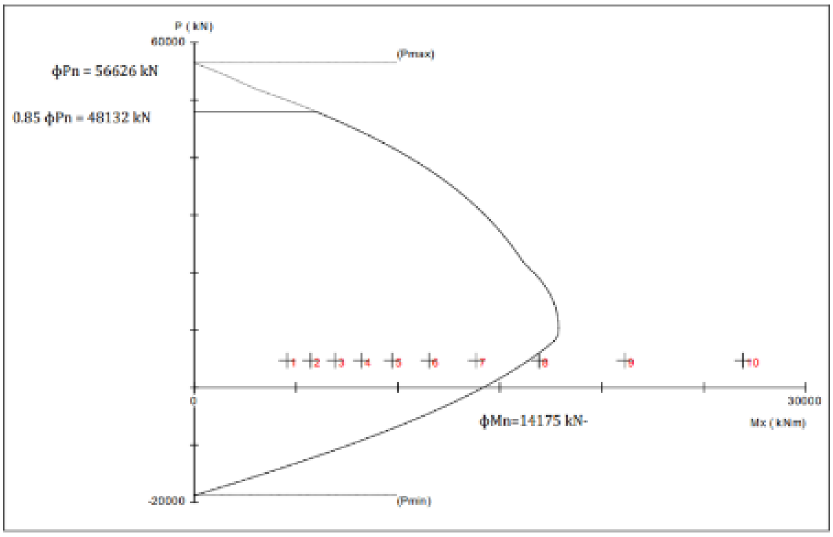

Figure 4a Strength interaction diagram of the bridge's pier with plotted values of load effects from axial, flood, and debris impact forces (generated using spColumn v6.00 Software)

Figure 4b shows the different values of the object's velocities assumed as equal to floodwater velocity with corresponding bending moments at the pier structure. The combined effect of the axial load and bending moment on the bridge's pier was plotted in figure 4a and labeled as points 1 to 10. As shown, an impact force with an object and floodwater velocity greater than 10 meters per second can be considered unsafe and may lead to bridge failure.

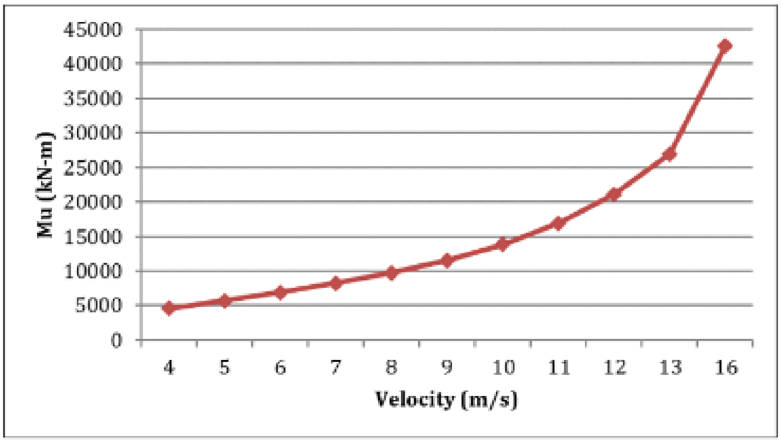

Figure 4b Maximum bending moment on the pier against the corresponding floodwater and object velocities

4.4. Traffic Threat: Moving Overloads

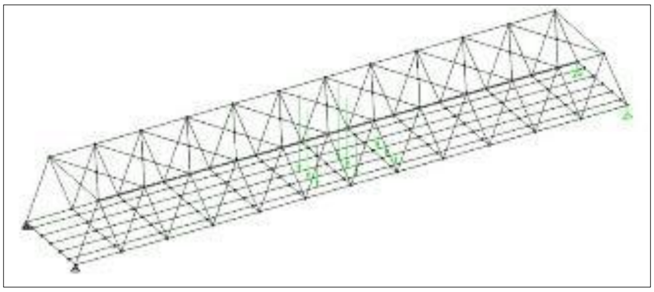

Figure 5a One HS20 truck moving loads applied on the bridge

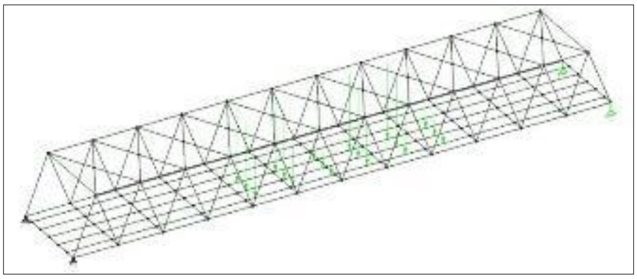

Figure 5b Two HS20 truck moving loads applied on the bridge

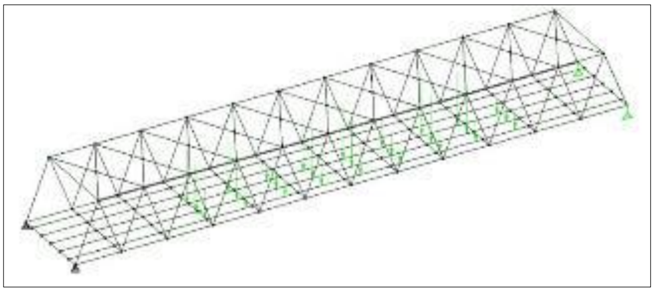

Figure 5c Three HS20 truck moving loads applied on the bridge

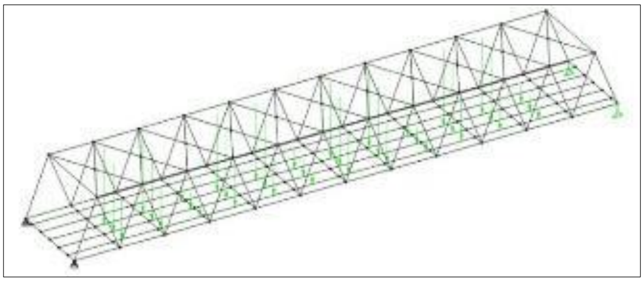

Figure 5d Four HS20 truck moving loads applied on the bridge

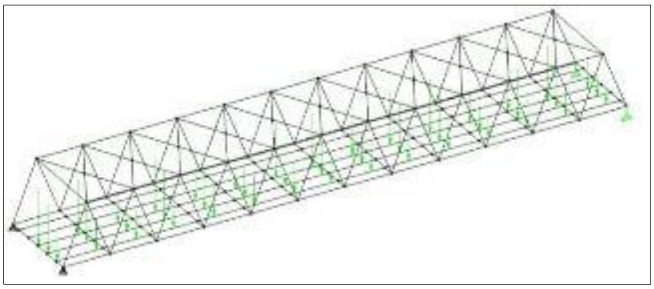

Figure 5e Five HS20 truck moving loads applied on the bridge

Moving overloads are extreme loading conditions caused by trucks moving along the bridge. Some positions of the trucks are hence the worst cases for the bridge, which can be exacerbated if the trucks exceed the bridge design specifications. Overloaded vehicles may damage road surfaces and cause bridge fractures, significantly shortening the bridges' service life.

The analysis for the moving loads was conducted using STAAD.Pro CONNECT, wherein the moving loads were applied at a three-meter interval along the bridge span. The loadings were gradually increased by adding another HS20 loading until a member of the bridge yields, in which case its DCR value would exceed 1.0. This increase in the number of trucks simulated truck loadings moving in tandem that led to a bridge collapse.

Figures 5a to 5e show an increasing number of trucks moving along the bridge. The respective DCR values are 0.7201, 0.8228, 0.9202, 0.9844, and 1.004. This finding suggests that only after five trucks will the bridge members begins to yield. Thus, this kind of loading scenario should best be avoided to prevent bridge damage.Results further connotes that the bridge members generally have low potential against yielding when subjected to a moving overload scenario such as multiple HS20 trucks moving loads. The results, however, only hold for the bridge members and do not necessarily apply to the bridge connections, which are beyond the scope of this study.

4.5. Preliminary Assessment of the Members

Figures 6a to 6d show the bridge members' designation. Shown in figures 6a and 6b is the numbering of members in the bridge's right and left side elevations, consisting of diagonals, bottom chords, top chords, and end posts. In figure

6c, the designated numbers for each member of the top portion of the bridge are shown, comprising top lateral bracings, struts, and portal struts. Moreover, in figure 6d, the stringers and floor members are numbered.

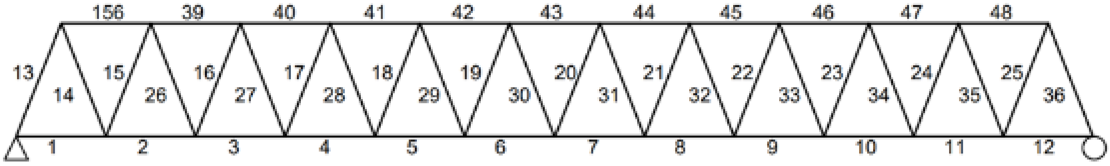

Figure 6a Member designation (right side elevation)

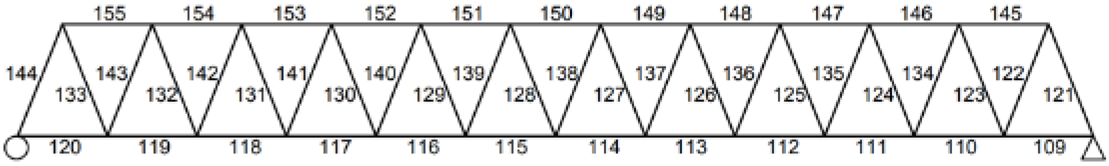

Figure 6b Member designation (left side elevation)

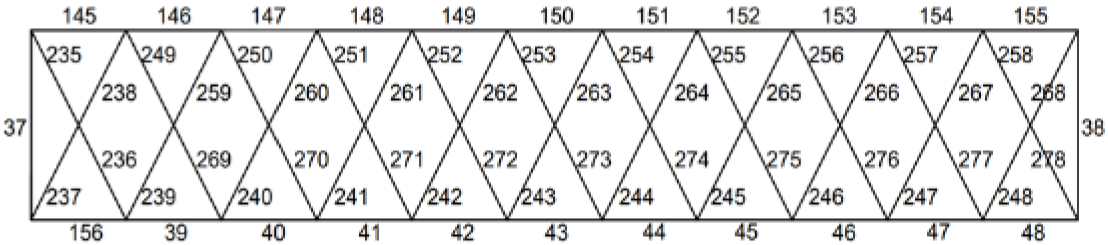

Figure 6c Member designation (top members)

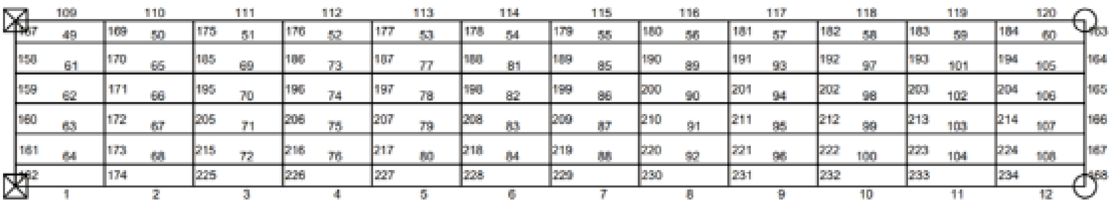

Figure 6d Member designation (bottom members)

After creating an analytical model, the bridge's safety was evaluated before any initial fracture. The bridge was analyzed considering service load conditions and nominal strength using STAAD.Pro CONNECT V22. The bridge loadings were assumed to be the combination of dead load and live load 1.0D+μL with μ as an attributed factor varying from 0 to 1.0. To simplify the analysis, this study sets the attributed factor μ to 0, which means that the live load is not added. This study did not consider the extent of loading the members would fracture but how they would collapse at a given load. Thus, only the dead load was considered, which comprises the weight of the girders, slab, and superimposed dead loads such as guardrails and walkways.

The analysis assumed a stick model of the bridge, utilizing the truss geometric diagram indicated in the technical drawings forwarded by DPWH Region X. In the conventional truss bridge design, the deck system does not improve the trusses' strength, stiffness, and load sharing [4]. Therefore, the deck slab was only considered in calculating the dead loads and not as a contributor to the total strength of the truss.

4.6. Progressive Collapse Analysis

Progressive collapse occurs in a structure when a critical member fails, leading to the propagation of collapse from the local point of failure to the neighboring or overall structural members. Members' failure results from abnormal loadings such as vehicular impacts and overloads, bomb explosions, fires, and large hydrodynamic forces [3]. Assessing the potential of a structure to progressive collapse is critical since it might offer a chance to revise the design or introduce measures for more safety.

Table 1 Maximum DCR and DAF of each removal case

CASE	MEMBER REMOVED	MAXIMUM DCR ($^{\sigma_s}/_{\sigma_y}$)	MAXIMUM DAF
1	13*	3.405	1
2	14	2.624	1.669
3	15*	2.102	1
4	26	1.847	1.471
5	16*	2.068	1
6	27	1.661	1.424
7	17*	1.566	1
8	28	1.210	1.309
9	18*	1.004	1
10	29	0.758	1.193
11	19*	0.623	1
12	30	0.618	1.158
13	1	1.629	1.415
14	2	1.480	1.377
15	3	1.862	1.475
16	4	1.743	1.444
17	5	1.625	1.414
18	6	1.484	1.378
19	156*	29.41	1
20	39*	59.91	1
21	40*	203.8	1
22	41*	236.2	1
23	42*	35.23	1
24	43*	33.51	1

Note: Members with (*) sign means compression members; otherwise, tension.

This study employed the nonlinear static approach in analyzing the steel truss bridge in terms of its potential to progressive collapse. Specifically, it applied the P-Delta analysis in calculating the stresses of the members. The P-Delta analysis is a second-order analysis brought by the geometric nonlinearity that considers both the effects of the P-Large delta (P-Δ) and P-small delta (P-δ) deformations. Conventionally, the equilibrium in first-order structural analysis is expressed in the undeformed structure's geometry. In the case of the linearly elastic structure, the relationship between external force and displacement is proportional. Thus, the unknown deformations can be obtained directly, whereas second-order analysis requires an iterative process to arrive at the solutions. The latter is due to the deformed geometry of the structure that is unknown during the formation of the kinematic and equilibrium relationship. Thus, the analysis follows a step-by-step incremental way, utilizing the deformed geometry of the structure derived from iterative cycles of the calculation.

In the analysis, any member is considered to have failed if the value of the (DCR) at the end connection of the member or along the span crosses the permissible limit based upon the combined stresses. In the study of Khuyen [4], it was stipulated that such permissible limit is DCR=1.0. If any value exceeds this limit, the member shall be treated as a failed member because the demand (combined stresses that the member carries) exceeds the capacity (yield strength of the same member). Once the initial damage occurs in the member, this study assumes that the member loses its intended function; thus, it shall be removed from the analytical model.

In this study, the bridge members removed were diagonals, top chords, and bottom chords. For the diagonals, the members removed were members 13, 14, 15, 26, 16, 27, 17, 28, 18, 29, 19, and 30. For the top chords, only members 156, 39, 40, 41, 42, and 43 were removed, while members 1, 2, 3, 4, 5, and 6 were removed for the bottom chords.

Table 1 presents the maximum demand-capacity ratio (DCR) for each case of member removal. The table shows that the maximum and minimum DCRs are 236.2 and 0.618, respectively. During the pre-removal analysis, all the DCR values did not exceed the limit, which implies that the members are safe under service dead load. However, during the progressive collapse analysis, each case removal resulted in the escalation of the DCR values wherein such values exceeded the permissible limit, except for the removal of members 29, 19, and 30 with low DCR values of 0.758, 0.623, and 0.618, respectively.

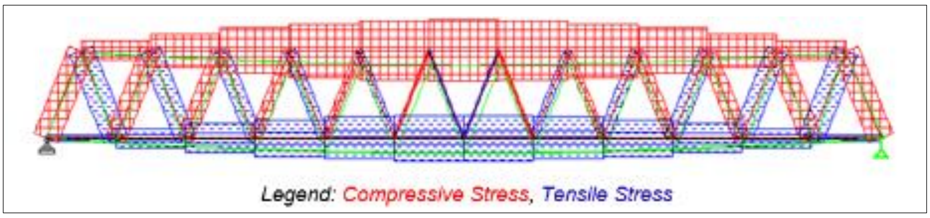

Figure 7 Stress distribution in pre-removal

The foregoing result can be explained by the observation that members 29, 19, and 30 are diagonal members located close to the mid-span, wherein they experience the least stresses as indicated in figure 7. It can be deduced that these members are not critical, and even if they fail, such will not significantly affect the bridge's progressive collapse potential.

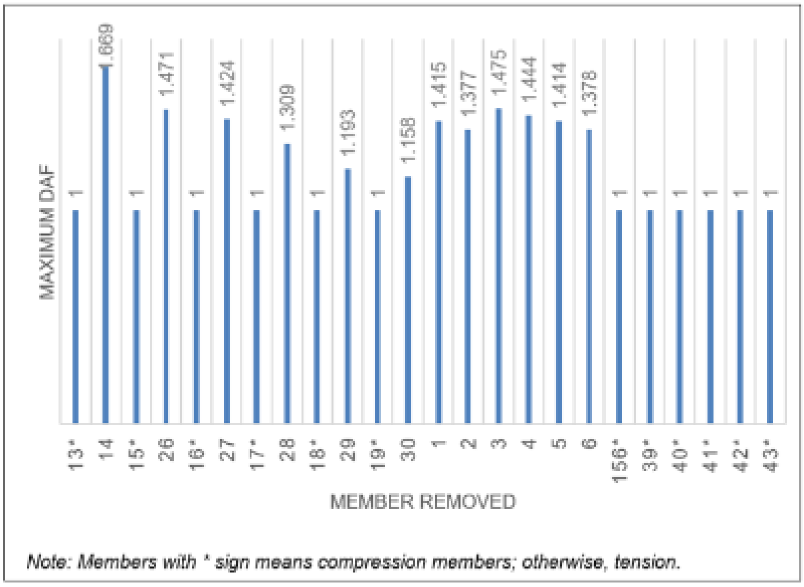

Figure 8 Maximum DAF of each case of member removal

This study used a linear static approach to analyze the bridge instead of a complex dynamic analysis. Nevertheless, a dynamic amplification factor (DAF) was used to mimic the dynamic effects of progressive collapse. DAF is the factor to express how many times the results in static analysis shall be multiplied to get the same results when the dynamic analysis is performed. It accounts for the effects of release forces and vibrations during the fractured case scenario. The fracturing event is dynamic and results in dynamic structural motion due to the sudden change in geometry. In addition, the calculation of DAF considers the multiple degrees of freedom and the location of the fractured member.

URS Corporation [25] theorized that the conventional DAF is 1.854 for bridges with a single degree of freedom; however, such an approach is conservative as the bridge system acts as multiple degrees of freedom. The DAF varies between bridges and the fractured members' locations. In this paper, the DAF of every fractured case scenario was determined. It was calculated by determining the DCR of each member; then, the maximum DCR was used for the calculation of the DAF using the formula DAF = 0.255 (max. DCR) + 1.0.

Figure 8 present the maximum DAF for each case of member removal. As shown in the table and figure, removing all tension members resulted in DAF values greater than 1. The maximum DAF among the removal of tension members occurred in case 2 or the removal of member 14 with a value of 1.669, while the minimum occurred in case 12 (member 30) with a value of 1.158. Tension members are usually the fracture-critical members (FCMs) whose failure would cause a portion or the entire bridge to collapse. The sudden failure of a bridge member causes an immediate geometric change in the structure, resulting in the release of energy confined to the damage's immediate vicinity. In steel bridges, if tension members fracture, they release the load-carrying function of the members immediately. Hence, the dynamic effects must always be considered in the calculation by incorporating the DAF.

On the other hand, for the compression members, the DAFs are 1.0, which implies that there is no addition of the dynamic effects. The release load factor was increased only until DAF=1.0 because the failure appears without the dynamic effect of sudden fracture. A compression member is fractured because of buckling. However, buckling is not a sudden failure, and the sudden loss of capacity does not occur. Hence, the dynamic amplification factor was not incorporated.

4.6.1. Member Removal Scenarios

Redundancy and progressive collapse analysis is the analysis that is performed to assess whether the initial fracture on a member would propagate throughout the structure causing partial or entire bridge collapse and the subsequent loss of service. The analysis also evaluates redundancy level, predicts progressive collapse potential, and identifies fracture-critical members on the bridges [4].

Fracture critical member is a component in tension whose failure is expected to result in the bridge's collapse or inability to perform its function. FCMs are essential parts of a non-redundant bridge system. In in-service steel truss bridges, if analysis demonstrates that a bridge has sufficient strength and stability to avoid partial or total collapse and carry traffic in the presence of a fractured member, the fractured member is not qualified as FCM [26].

This study centers on predicting the progressive collapse potential and identifying critical members of the Manupali bridge following initial member failures, wherein treated as complete removal of the member. The probability of two or more member fractures is much lower than the probability of a member fracture. In addition, the case of more than one member fracture can be a combination of single-member fractures. Understanding the case of a one-member fracture can provide grounds for further study of fractures beyond single members and connection failures [4]. Hence, in this study, only one candidate is assumed to be damaged per case scenario.

A sudden member failure is a dynamic event in which the structural motion is initiated by the energy released by the sudden loss of a load-carrying member. Addressing this dynamic event requires dynamic or static analysis with an alternate load path that can account for the dynamic structural motion after the damage. The dynamic analysis, however, is complex and requires much time and a heavy workload. Static analysis with an alternate load path, on the other hand, is an excellent approach to avoid using the more time-consuming dynamic analysis. This study also employs a new method to calculate the dynamic amplification factor (DAF) proposed by Khuyen [4], which is required for the alternate load path analysis.

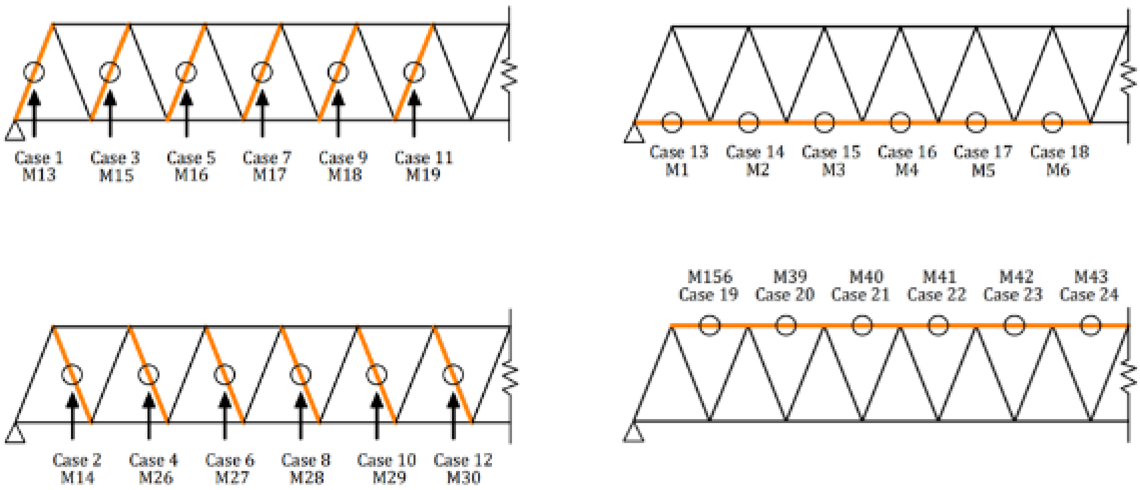

Figure 9 Removal cases

Diagonals

In this study, twelve diagonal members are removed, namely members 13, 14, 15, 26, 16, 27, 17, 28, 18, 29, 19, and 30. Six members involved are in compression, namely members 13, 15, 16, 17, 18, and 19, while members 14, 26, 27, 28, 29, and 30 are in tension. Tension members are usually the fracture-critical members (FCMs) whose failure would cause a portion or the entire bridge to collapse. If tension members fracture, they release the load-carrying function of the members immediately. Hence, the dynamic effects must always be considered by incorporating the DAF in the calculation. Nonetheless, the compression members can also be regarded as critical members since their failure may also lead to a collapse of the entire bridge, only that they do not abruptly fracture.

Among the listed compression members above, members 13, 15, 16, 17, and 18 are considered critical members, implying that any removal of the said members will propagate damage to other members and components, leading to a progressive collapse of the bridge. However, only member 19 is considered not critical, thereby indicating no damage propagation to the neighboring members once it fails. This is because such a member is located close to the midspan, wherein it experiences lesser stress.

Meanwhile, for the listed tension members, members 14, 26, 27, and 28 are all fracture-critical (FCMs), which implies that removing any of the listed members will propagate damage to other components, leading to a disproportionate collapse of the bridge. Moreover, only two of the diagonal tension members (members 29 and 30) are considered non-FCMs, meaning the bridge will not collapse even if any of the two members is removed. Such members also situate closer to the midspan and experience less stress which explains why they are non-FCMs.

Although some members are not critical, it is also important to note that the analysis only employed a one-member removal at a time rather than a simultaneous removal of two or more members. One member removal may not propagate damage to other components. However, the simultaneous removal of two or more members is still possible and may lead to damage propagation to other members and components, thus detrimental to the bridge [4].

Bottom Chords

For the bottom chords, six members were removed namely, members 1, 2, 3, 4, 5, and 6 which are all in tension. Analogous to a simply-supported beam with the self-weight directed downward due to the action of gravity, the top layer is compressed while the bottom layer is stretched which explains why the bottom chords are in tension. Thus, a DAF was integrated into the analysis. The results of the analysis disclosed that all the members are fracture critical. It can be therefore deduced that any removal of the listed members will propagate destruction to other members and components, thereby causing a catastrophic collapse of the entire bridge.

Top Chords

For the top chords, six cases of member removal were involved in the analysis. Such members are the following: 156, 39, 40, 41, 42, and 43. Opposite to the bottom chords, these members are all in compression. It still follows the beam analogy wherein the top layer is compressed which supports the idea as to why all the listed members are in compression. Being compressive, such members do not abruptly fracture. However, it does not necessarily mean that they are not critical. Following the analysis, it was found out that all of them are critical members, thereby implying that the failure of any member is detrimental to the bridge since it will generate damage to the other components, leading to a progressive collapse of the bridge.

4.7. Pile Capacity Analysis

Expecting large dead and live loads to be passed on the bridge pier and abutments, a bored pile structure was chosen for the bridge. In figures 10 and 11 shown below, presented are the imposed loads on the pile foundation upon the removal of truss members. The figures compare the evolution of the reactions during the progressive collapse against the supports' allowable pile capacity. The formula used for calculating the value of the pile capacity is:

$$\sum \pi\phi\,(SF_n * L_n) + \frac{\pi\phi^2}{4} * EB \qquad (8)$$

where ϕ is the pile diameter, SF_n is the skin friction at a soil layer, L_n is the length, and E.B. is the end-bearing capacity. By substituting the values determined from the geotechnical investigation and the technical drawing of the pile foundation to the formula, the pile capacity is 14844 kN per pile. Using a factor of safety of 2.0, the allowable capacity of the pile becomes 7422 kN.

Since there are two piles in the abutments, the allowable pile capacity of the abutments is 14844 kN. Meanwhile, there are five piles in the pier, so the total allowable pile capacity is 37110 kN. Any load more than that value may result in the failure of the piles in the abutment or pier.

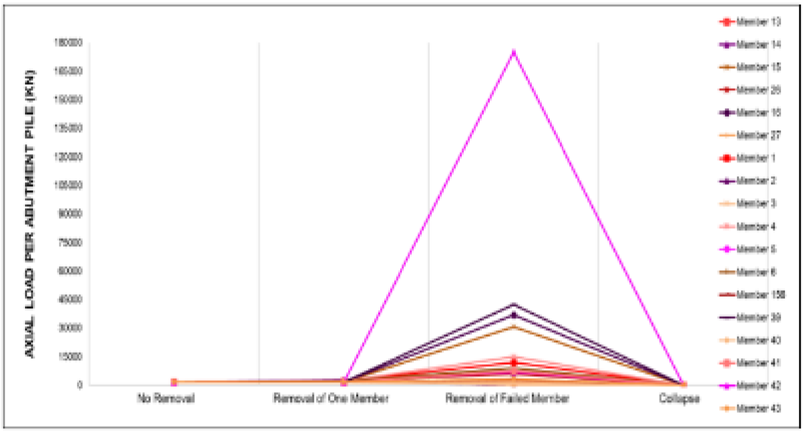

Figure 10 Variation of loads per abutment pile during progressive collapse

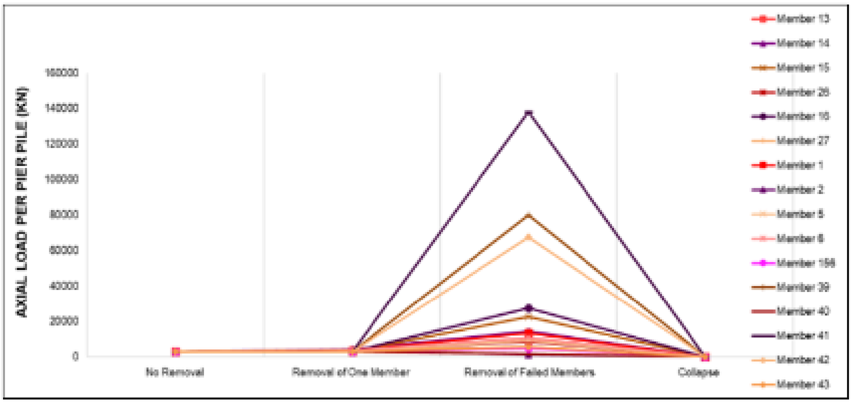

Figure 11 Variation of loads on pier piles during progressive collapse

From the results of the STAAD analysis, the values of the reactions at the support can be determined when each truss member is removed. The reactions can then be compared to the pile capacity to check if any possibilities exist for the piles to fail due to exceeding the allowable capacity limit.

In figures 10 and 11, as seen below, the removals of members 42, 39, 16, 15, 17, 28, and 4 have led the reactions to exceed the allowable pile capacity of the abutment. To wit, the removal of member 42 had the greatest value of 174944 kN, which means that failure of the same member would cause the greatest impact on the bridge's abutment.

Meanwhile, in the pier's case, the removals of members 41, 39, and 42 have led the loads to exceed the allowable pile capacity. Specifically, the removal of member 41 has the greatest value for the reaction, which is 137902 kN, which means that removing such a member would cause the greatest chance of making the pier fail.

The foregoing observations and statements are anchored on the theory of dynamic amplification. It means that the progressive collapse of the structure, which is dynamic in nature, imposes large impact loads on the bridge's supports. In other words, the consequential increase in the reactions could overcome the allowable pile capacity. This observation should be supported by a more thorough analysis and verification using a more sophisticated method.

The study of Cai et al. [9] found that static procedures are more conservative than the dynamic approach. Furthermore, they also found that assuming the materials to be in the elastic range during collapse would cause deviations from the geometrically and materially nonlinear dynamic analysis. According to Khuyen & Iwasaki [27], the dynamic method is a direct solution to account for the impact loading and dissipation of the energy caused by the initial member fracture.

It is recommended that a geometrically and materially nonlinear dynamic analysis be conducted in future studies to compare with the findings of this research. This would also clarify the results of the pile capacity analysis.

5. Conclusion

The following conclusive statements attempt to answer the respective objectives of this study:

- From the conducted risk assessment of the Manupali steel truss bridge in Lantapan, Bukidnon considering hydraulic threats, it was found that the bridge was designed to have above-average performance against floods making it withstand extreme flood events with minimal or no damage. Regarding the bridge's risk of scouring, water flow velocities of 5, 6, 7, and 8 m/s with flow depths of 4, 3, 2, and 1m, respectively, were found to cause the occurrence of critical scour depths. Meanwhile, for risk against debris impact, an impact force with an object and floodwater velocity greater than 10 m/s was found unsafe and may lead to bridge failure. On the other hand, results from the risk assessment considering traffic threats showed that the onset of bridge damage occurs when five successive HS20 trucks move along the bridge. Results further indicate that the bridge members generally have low potential against yielding when subjected to a moving overload scenario.
- The progressive collapse analysis of the Manupali steel truss bridge revealed that the bridge has a very high susceptibility to progressive collapse. Out of all the notionally removed members, only three were not critical to the bridge's collapse. Among the twelve diagonals, five were critical members, four were fracture-critical members (FCM), all the bottom chords were FCMs, and all the top chords were critical members.
- From the assessment of the bridge's pile foundation capacity concerning changes in the imposed stresses due to progressive collapse, it was found that the removal of member 42 has the greatest impact on the bridge's pier and abutment piles which can lead to pile capacity failure.

Recommendations

Based on the findings of the study and the drawn conclusion, the following are recommended:

- Use a nonlinear dynamic approach in assessing the bridge's progressive collapse potential. Although the nonlinear static approach with the integration of dynamic amplification factor (DAF) mimics the effect of a dynamic event, the nonlinear dynamic approach with time-history transient analysis may provide a more accurate result. It may also further validate the result of nonlinear static.
- Since the progressive collapse analysis of the Manupali Bridge in this study is a post-construction analysis, this study can only recommend mitigation measures for the bridge. Thus, this study recommends redundancy of the fracture critical tension members of the bridge identified in the study. Such redundancy approaches include but are not limited to the following:

    - splitting the tension members' H sections into two T-sections individually connected to the gusset plates, creating two independent tension members capable of resisting the load; and,
    - providing an internally redundant and not fracture-critical posttensioned concrete member to the main bottom chords tension members that takes all the tension force.
- Perform progressive collapse analysis with the live loads and incorporate results in designing a steel truss bridge for future constructions.

Compliance with ethical standards

Acknowledgments

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Disclosure of conflict of interest

There are no conflicts of interest.

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