PROGRESSIVE COLLAPSE VULNERABILITY OF A CABLE STAYED BRIDGE

Rajarshi Das[1], A. D. Pandey[2], Soumya[3], M. J. Mahesh[4] and S. Anvesh[5]

[1, 3, 4, 5]Department of Earthquake Engineering, Indian Institute of Technology Roorkee, India
[2]Assistant Professor, Department of Earthquake Engineering, Indian Institute of Technology Roorkee, Uttarakhand, India, 247667
e-mail: rajarshidas91@gmail.com, adpanfeq@gmail.com

ABSTRACT: Since the structural system of bridges is different from buildings, only a few of many modern codes offer adequate methods of improving resistance against progressive collapse for bridges. Although great efforts have been contributed to the progressive collapse of building structures, comparably small attention has been paid to the bridge structures, specially the cable-stayed bridges. This study demonstrates numerical modelling and analysis of a representative cable stayed bridge through the four analytical procedures, i.e., linear static, nonlinear static, linear dynamic and nonlinear dynamic to understand the collapse pattern. Furthermore, the response of the structural model is discussed for multiple types of cable loss cases. The Dynamic Amplification Factor of 2 has been approved by the results. The results also indicated a relation between the possibilities of failure progression with the location of the failed cables with respect to the pylons. Different progressive collapse patterns were also identified for nonlinear static and dynamic procedures.

KEYWORDS: Cable Stayed Bridge, Dynamic analysis, Linear analysis, Nonlinear analysis, Progressive collapse.

1 INTRODUCTION

A structural failure initiated by a localized structural injury and eventual development of a chain reaction, resulting into a breakdown of a major portion of the entire system is characterized as Progressive collapse. It is a dynamic event initiated by a release of internal energy due to the instantaneous loss of a structural affiliate disturbing the initial load equilibrium and thus, the structure vibrates until either a new equilibrium position is found or it collapses. Events like the collapse of *Haeng-Ju Grand Bridge* in Seoul in 1992 ensured the need to incorporate progressive collapse resisting procedures into the design of major bridges. Bridges being primarily horizontally aligned structures with a single main axis of extension; their possible mechanisms of collapse are different

compared to buildings. For example, the loss of cables should be measured as a possible local failure for cable stayed bridges due to their small cross sections which lead to low resistances against accidental lateral loads stemming from vehicle impact or accidental actions. The loss of cables can lead to overloading and rupture of adjacent cables.

2   A LITERATURE REVIEW

As mentioned earlier, the structural system of a bridge is relatively simple, with low redundancy, and high sensitivity to dynamic excitation. So, *Liu et al* (2011) [1] proposed a different classification than the one given by Starossek (2007) [2]. According to the authors, the reasons behind bridge progressive collapse can be divided into three aspects, (i) unexpected events, such as collision with overweigh vehicles, exploration, and earthquake, (ii) degradation of structure performance, including corrosion and creep effect, and (iii) improper design or wrong construction methods.

Two types of failure progression were stated. First was the Bearing-failure type; where the direct consequences of failure of the main bearing component included; (a) Reduction of statically indeterminacy degree, and (b) Internal force distribution. The Guangdong Jiujiang Bridge collapsed due to sailing fault as a sand filled ship collided with one of the piers, damaging the adjacent four continuous spans. Secondly, partial-failure type, where "partial" means some load-bearing components of bridge structures. Due to the failure of these members, the structural stiffness and internal forces would change, resulting in progressive collapse. In 2001, the Xiaonanmen Bridge collapsed because of stress corrosion of some ruptured hangers after being struck by an overweight truck.

Cai et al (2012) [3] studied a 2D cable-stayed bridge structural model using four modified analytical procedures, i.e., linear static, nonlinear static, linear dynamic, and nonlinear dynamic. According to the Authors' investigations, a single cable loss did not start a collapse progression, as the cable tensions remained relatively small. Although, for the simultaneous failure of two adjacent cables, cables began to yield, plastic hinges were found in the girder, and deformations increased rapidly, the bridge still remained intact. A complete collapse progression was observed when the live load was increased by two factors or more. After comparing the four improved methods the Authors' concluded that the static analysis procedure is highly conservative in comparison with the nonlinear dynamic analysis method and couldn't provide comprehensive information on structural response. Furthermore they recommended the nonlinear dynamic analysis starting with the initial state given by a nonlinear static procedure as the best solution to trace the collapse progression. The results of their research evaluation of cable loss showed that the cables near the vicinity of the ruptured cable would not reach the yield

tension and the maximum nodal vertical displacement would not increase much, when the lost cables were near the pylon.

Wolff and *Starossek* (2012) [4] examined the dynamic response of a 3D cable-stayed bridge to the loss of one and multiple cables. The Authors' conducted a quasi-static analysis using dynamic amplification factors to study the dynamic effects. The DAF were determined and the limits of the quasi-static approach for cable-stayed bridges were highlighted. Then the collapse behaviour of a cable-stayed bridge was analysed in the time domain considering geometric and material nonlinearities. From the results the authors concluded that a unique dynamic amplification factor could not be specified as it depended on the location of the ruptured cable and on the type and location of the state variable. A DAF of 2.0 was suggested for the safe design of cables. Dynamic time history analyses were recommended for the cable loss cases which would yield the highest responses. They also stated that the robustness of the bridge could be increased by increasing the stiffness of the bridge girder or by reducing the unsupported length, i.e. closer cable spacing. The Authors' recommended the former after analysing a greater chance of failure in the later case.

3   MODELLING OF A CABLE STAYED BRIDGE
3.1 Bridge details
The bridge considered in this present piece of work, is 822.96m long, with four 122.3m high pylons supporting the deck. These pylons rest on two 13m high piers. Fig. 1(a) and Fig. 1(b) show the 2D and the 3D geometrical configuration of the model respectively. Two portals are present between each pair of pylons to offer stability. The left and right ends of the deck are respectively hinge and roller supported on the end piers.
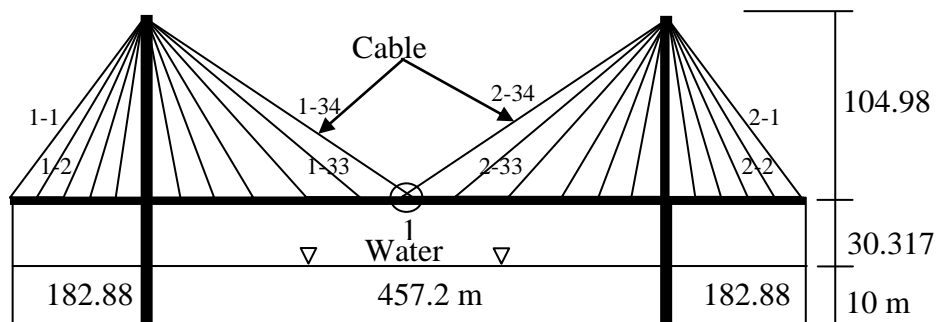

Figure 1(a). 2D Geometry of the bridge model

The deck is supported by 136 stay cables with 68 on each half of the bridge. Diameter of the cables rises linearly from 0.14m to 0.2m as either end or the centre of the bridge is approached from the pylons. These cables are prestressed

accordingly with a convergence study under gravity load. Piers are modelled as concrete cellular box type structures. The numerical investigation is done using the finite analysis software package SAP 2000. The material properties and sectional details used for the different components of the bridge model have been taken as per the corresponding Author's M.Tech. Thesis [5].
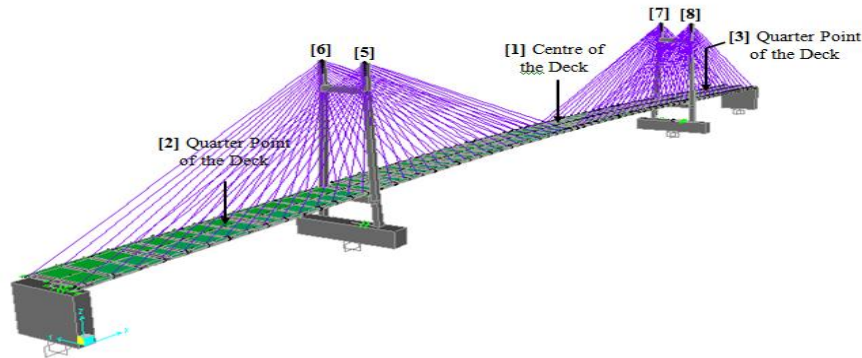

Figure 1(b). 3D Geometry of the bridge model

3.2 Nonlinearity

Practical structural response to accidental loading is always nonlinear in nature. Therefore, material and geometrical nonlinearity should be considered to obtain the genuine behaviour of the cable stayed bridges under different cable loss cases. Method of plastic hinges was used in SAP 2000 program to consider material nonlinearity for different elements. For each cable loss case two prominent failures were observed from a literature survey,

(i) Overloading of adjacent cables due to local redistribution, and

(ii) Flexural failure of the steel girders and the deck.

Therefore the material nonlinearity was considered for the cables, girders and the deck. As the cables can only be in tension, a compression limit of zero was assigned to them and an axial plastic hinge was introduced in the middle of each cable element. The yield stress was obtained from the material properties of a cable as 0.6 GPa. The strain at onset of strain hardening was taken to be 2%; then the strain at rupture was taken to be 5%. The girders are subjected to both axial forces and bending moment. Thus P-M2-M3 hinges were introduced into these elements. Hinge properties were adopted according to ASCE-365 guidelines. Instead of plastic hinges, nonlinearity was defined in its sectional properties window. Due to large deflections after the loss of different cables, the effect of geometrical nonlinearity was considered in the nonlinear static and dynamic analysis. All materials were assumed to be isotropic and homogeneous in nature.

4 PROGRESSIVE COLLAPSE ANALYTICAL TECHNIQUES

A cable stayed bridge is the only type of bridge structure routinely designed for cable loss. The loss of one or multiple number of cables can lead to overloading and rupture of adjacent cables. Furthermore, the stiffening girder could be in compression and a cable loss might reduce its bracing against flexure. One or multiple cables are assumed to get damaged primarily to trigger the collapse progression and then the following analyses are performed.

4.1 Linear gravity load analysis

The gravity load analysis is a linear static analysis of the bridge model under its own self weight and live load. This analysis procedure does not include any type of nonlinearity or dynamic effect. It only provides a basic understanding of local redistribution of stresses in the elements nearby to a failed one.

4.2 Nonlinear gravity load analysis

In this nonlinear static procedure, nonlinear behaviour of the structural components is taken into account. The axial force carried by a cable is observed to get redistributed to the supporting elements after its elimination from the structure. Stresses increased in those components and when they exceeded the yield limit, that corresponding element also failed. Hence a collapse progression is observed. Failure of different elements is indicated by hinges. Once the analysis has been performed for different cable loss cases, plastic hinges are found in the remaining cables and girders following an unique pattern. This procedure provides an important step by step scenario to understand which element is going to collapse first and which one is going to follow.

4.3 Linear dynamic analysis

For the design of cable stayed bridges, the PTI (2001) also prescribes the use of dynamic analysis in the extreme event of cable loss in terms of load applications. Once a singular or multiple numbers of cables snap, an impact load is generated on the pylon and the deck in an opposite direction, where they were previously connected. The large push generated by this sudden dynamic impact loading is identified to cause a different path of collapse progression compared to the static analysis. The damping of the structural system is taken as 2%. The instantaneously unloading method is used in this study to analyse the model for subsequent cable losses. The procedure adopted is as follows.

  i.   A nonlinear gravity load analysis is carried out on the undamaged model to determine the deflected and stressed condition of the structure.
 ii.   The initial stressed structure is considered from the nonlinear gravity load case to start the dynamic analysis.
iii.   Initial axial forces (P) in cables are noted down from the nonlinear gravity load case.

iv.   These equivalent and opposite axial forces (P) are assigned to the cables (which were primarily assumed to fail) in both the anchorage locations as shown in Fig. 2(a).
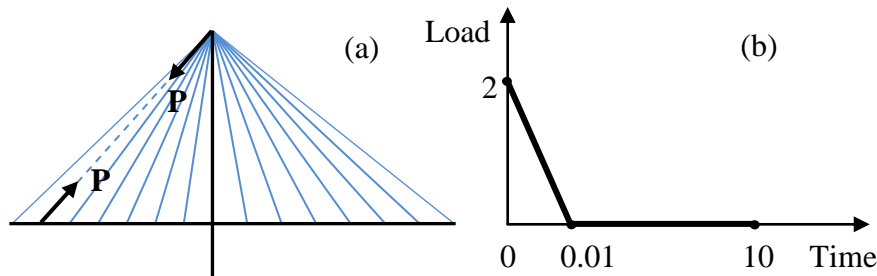

Figure 2. (a) Load direction and (b) Unload time history for the equivalent force due to cable loss

v.   An instantaneous unloading function is used as shown in Fig 2(b). When this function is applied on the model, the assigned axial forces are automatically doubled. The first P cancels out the load carrying capacity of the cable that was assumed to fail and the second P introduces the impact force to the pylon and the deck on the anchorage points. So, the corresponding cable collapses and subsequently generates an impact force on the anchorage points with the unloading function.

4.4 Nonlinear dynamic analysis

The nonlinear dynamic analysis is done as a simple combination of the linear dynamic and nonlinear gravity load analysis. In this procedure a dynamic analysis was performed considering the geometrical and material nonlinearity of the structural components. So, after the last step discussed in the previous linear dynamic procedure, hinges are found in relevant cables and girders after their corresponding yield points are reached. Stresses increase in the decks and cracks are expected to occur subsequently according to the results.

5   PROGRESSIVE COLLAPSE ANALYSIS RESULTS

A meticulous investigation is done on the cable stayed bridge model to study the path of failure progression in a precise manner. P-Delta effects were assigned to the frame elements to incorporate the cable stiffening behaviour due to geometric nonlinearity. The linear and nonlinear static analysis of the undamaged model provided the variation of the vertical deflection of the centre of the bridge deck. The maximum vertical displacements of the centre in linear and nonlinear static case are found to be -1.282303m and -1.268193m respectively. Then the main structural model and models with different cable

loss are analysed to visualize the load distribution, variation of different response parameters and to determine the path of collapse progression throughout the structure.

Two types of cable loss scenarios are considered.

(i) If an end cable fails first, a huge amount of stress gets redistributed to the remaining structure resulting in a significant amount of increase in the axial forces in the remaining cables and stresses in the girders and the deck. Now this redistribution ratio also depends upon the position of the elements. The remaining end cables attract more axial force than the middle cables. These end cables were already carrying high axial tensions initially, so they are more vulnerable to damage as a slight increase in the axial force can trigger their failure. Therefore, their chance of failure is much greater than the others due to the load redistribution.

(ii) Alternatively, if a middle cable fails first, again the axial loads increase in the end cables rather than the near ones, making them vulnerable to collapse. When these end cables rupture, the collapse progression occurs right through the structure as it has lost its main load carrying members.

5.1 Loss of cable 1-1 (Case-1)

The structural response of a cable stayed bridge to the sudden loss of cable 1-1 is studied in this section. The vertical deflection of the centre (1) and the quarter points (2 & 3) of the deck and the longitudinal displacement of the pylon tops (5, 6, 7 & 8) are plotted in *Fig. 3* and *Fig. 4*. The deflection values of the undamaged (main) structure are also given for a better understanding.
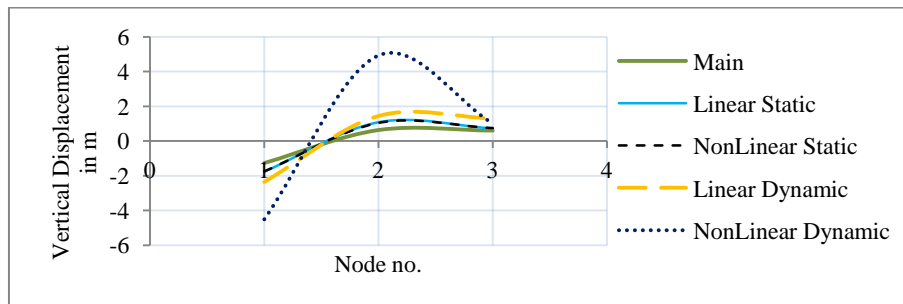

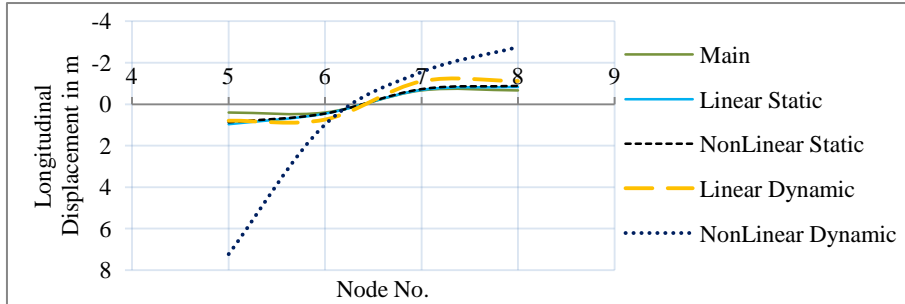*Figure 3.* Variation of vertical displacements in Case-1

Figure 4. Variation of longitudinal displacements in Case-1

In both of these above graphs it can easily be observed that there is a significant variation in the vertical and longitudinal displacements obtained from the nonlinear dynamic analysis. In *Fig. 3*, vertical displacement of the centre of the deck resulted to be more than 4.5 meters, which defines that the bridge deck has collapsed. *Fig. 4* shows that the top of the first pylon has undergone a longitudinal displacement of 7 meters due to increased flexure and vertical deflection of the longitudinal girders and the concrete deck. This reflects upon the failure of pylon 1. No such significant variation is observed for the static case. Hence, according to *Fig. 3* and *Fig. 4*, the bridge remains intact during the static procedure but collapses in the dynamic one.

Fig. 5 shows the cable forces in four most vulnerable cables after the mentioned cable failure, using the nonlinear static procedure. Cable 1-1 is the leftmost end cable. As discussed earlier, it can be seen that, considerable increase in demand occurs in one of the end cables (Cable 2-1) at the opposite end of the bridge. However, in this case, as an important load carrying member fails, forces also increase significantly in the cables near the vicinity of the ruptured cable. In the nonlinear static case for this particular cable loss as shown in Fig. 5, the increase in the axial force is observed to stay within the limit, thus failure haven't occurred.
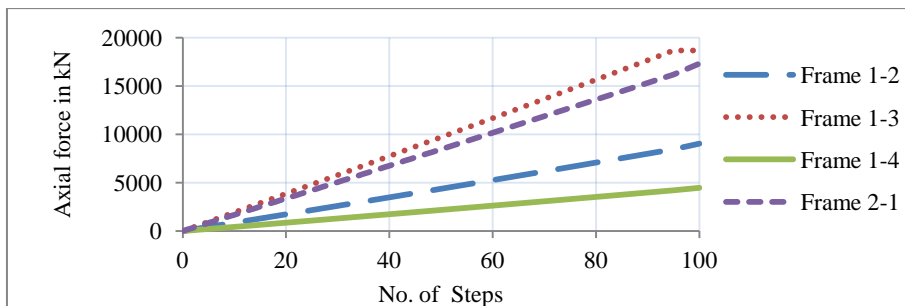

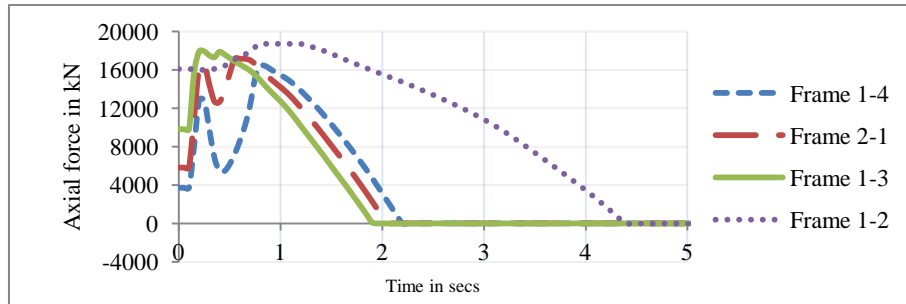*Figure 5.* Variation of axial forces in nonlinear static analysis of Case-1

Figure 6. Variation of axial forces in nonlinear dynamic analysis of Case-1

Fig. 6 shows the cable forces in four most vulnerable cables after the mentioned cable failure, using the nonlinear dynamic procedure. From this figure, the dynamic behavior of cables can easily be explained with increasing time steps for the nonlinear dynamic analysis. As Cable 1-1 fails, it generates an instantaneous push on the anchorage points on the pylon and the deck. These components get deflected in the opposite direction, increasing a significant amount of axial tension in the neighboring cables of the ruptured one. The axial tension in the corresponding cables is noticed to reach its maximum value after a certain time interval and as it exceeds the yield limit the relevant cable fails. Failure of an element means its zero contribution to the structure. So, a cable carrying zero axial tension is considered as a ruptured cable. In *Fig. 6*, it is seen that Cable 1-2 fails after 1.9 secs and Cable 1-3 fails after 2.1 secs. Then Cable 1-4 and Cable 2-1 follows.

The final stressed state before the collapse of the entire structure, determined in the nonlinear static and dynamic case are shown in *Fig. 7* and *Fig. 8* correspondingly.
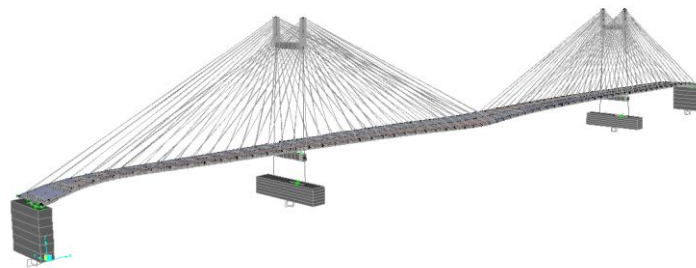

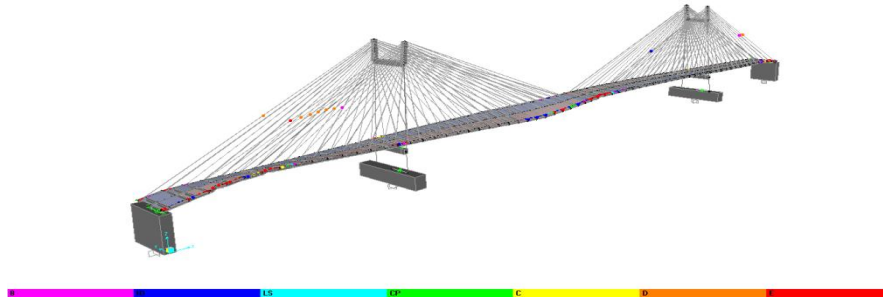*Figure 7.* Final state of nonlinear static analysis in Case-1

Figure 8. Final state of nonlinear dynamic analysis in Case-1

5.2 Loss of cable 1-1 and cable 1-2 (Case-2)

The investigation of the cable stayed bridge due to sudden loss of cable 1-1 and 1-2 is described in this sector. *Fig. 9* and *Fig. 10* describe the variation in the vertical deflection of the centre (1) and quarter points (2 &3) of the bridge deck and the longitudinal displacement of the pylon tops (5, 6, 7 & 8). The deflections of the undamaged structure are also shown for comparison study.
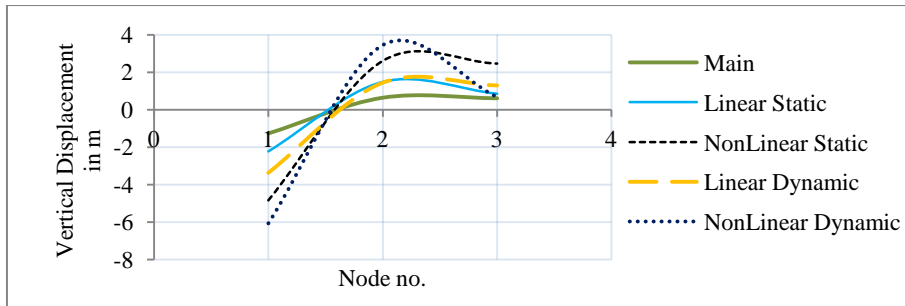

Figure 9. Variation of vertical displacements in Case-2
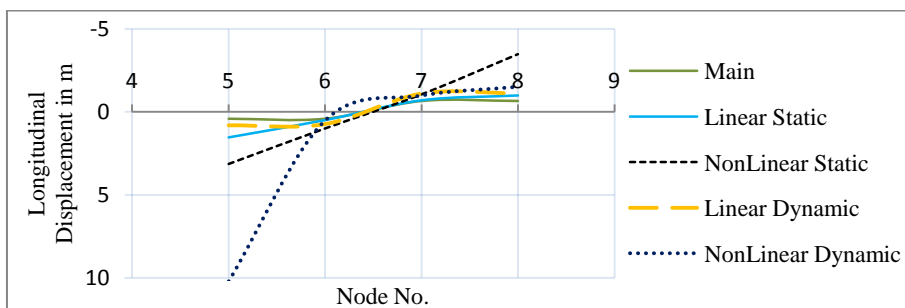
Figure 10. Variation of longitudinal displacements in Case-2

In *Fig. 11* and *Fig. 12*, the increase in axial force and resulting failure of the first four vulnerable cables are explained by the nonlinear static and dynamic

procedure respectively. Cable 1-1 and 1-2 are the two leftmost end cables. So, significant changes were observed in the axial forces in the end cable at the opposite end of the bridge and also the nearby cables.
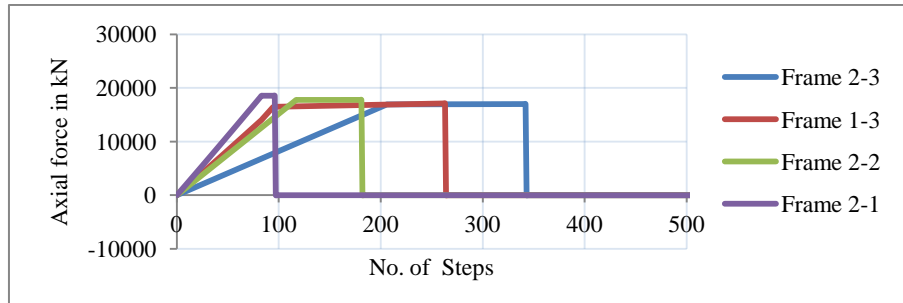

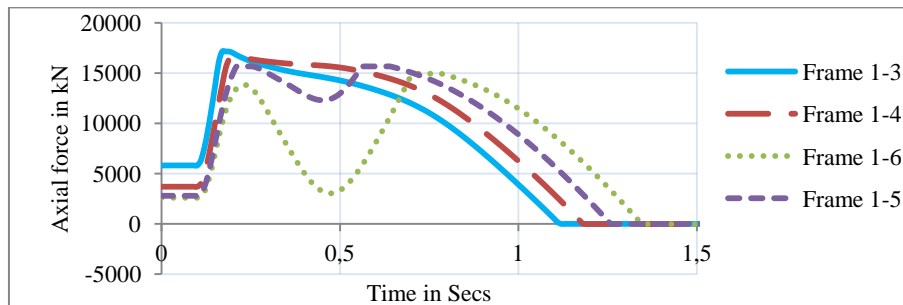*Figure 11.* Variation of axial forces in nonlinear static analysis of Case-2

Figure 12. Variation of axial forces in nonlinear dynamic analysis of Case-2

In the linear gravity load case, rise in axial forces in cable 2-1 was observed. With the help of GSA guidelines, it can be shown that the DCR value for cable 2-1 was more than 1. Therefore, it would get damaged. In the nonlinear static case, as expected, plastic hinge is found initially in cable 2-1 as shown in *Fig. 11*. Thus, cable 2-1 got damaged and this was noted as the beginning of a collapse progression. The increase in the axial forces and flexural stresses exceeded the yield limit for multiple members and so, more hinges were noticed. In the linear dynamic analysis the exceedance of axial force in a different cable was noticed. The DCR value showed that the maximum axial force in cable 1-3 exceeded the yield tension of cables. Plastic hinge is also found in cable 1-3 in the nonlinear dynamic procedure. A new path of collapse progression is identified. The final state of the structure before total collapse, obtained from the static procedure, is shown in *Fig. 13*.
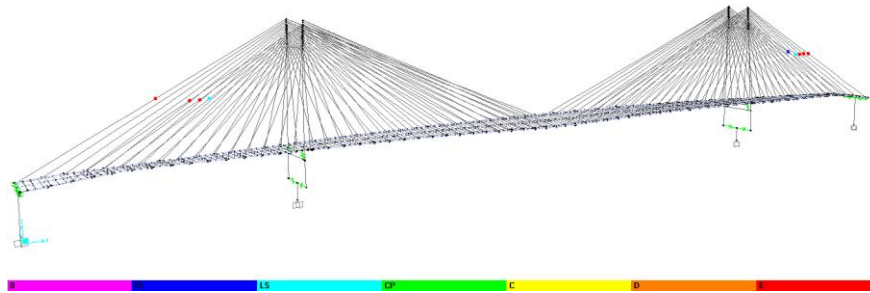
Figure 13. Final state of nonlinear static analysis in Case-2

In the nonlinear dynamic analysis the exceedance of axial force in a different cable is noticed due to the inherited impact force generated during the instantaneous failure of two cables. The final state of the structure before overall collapse is displayed in *Fig. 14*.
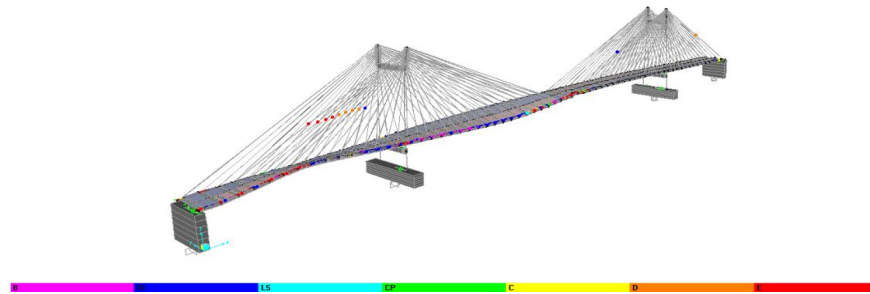

Figure 14. Final state of nonlinear dynamic analysis in Case-2

5.3 Loss of cable 1-2 and cable 1-3 (Case-3)

The sudden removal of cable 1-2 and 1-3 is studied in this section. The vertical deflection of the centre (1) and quarter points (2 & 3) of the bridge deck and the longitudinal displacement of the pylon tops (5, 6, 7 & 8) are plotted in *Fig. 15* and *Fig. 16*. The displacements at the same points of the undamaged structure are also shown to study the variation.
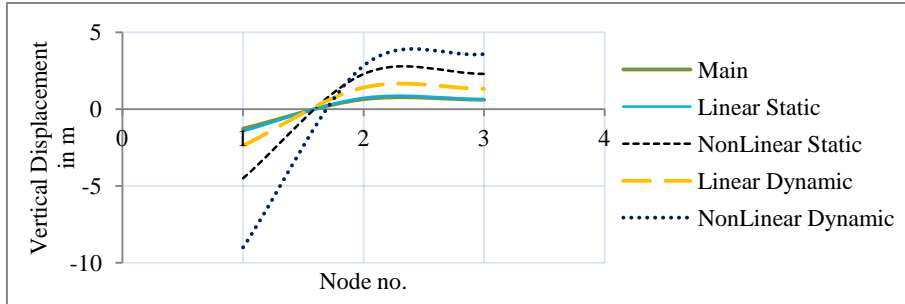
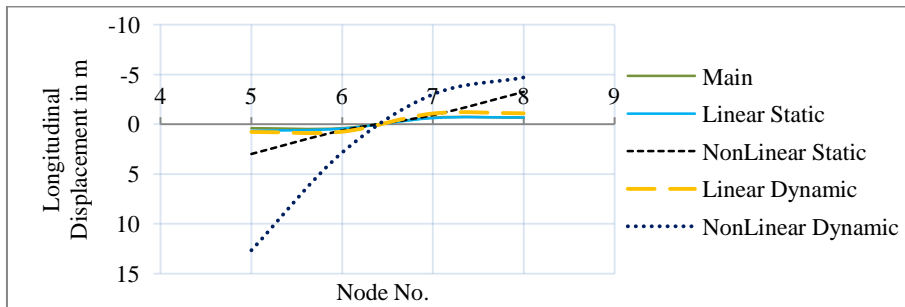*Figure 15*. Variation of vertical displacements in Case-3

Figure 16. Variation of longitudinal displacements in Case-3

Cable 1-2 and cable 1-3 are not end cables. So, significant changes are observed in the axial forces in cable 1-1. *Fig. 17* and *Fig. 18* shows the variation in cable forces for the first four vulnerable cables after the elimination of these two cables using the nonlinear static and dynamic procedures.
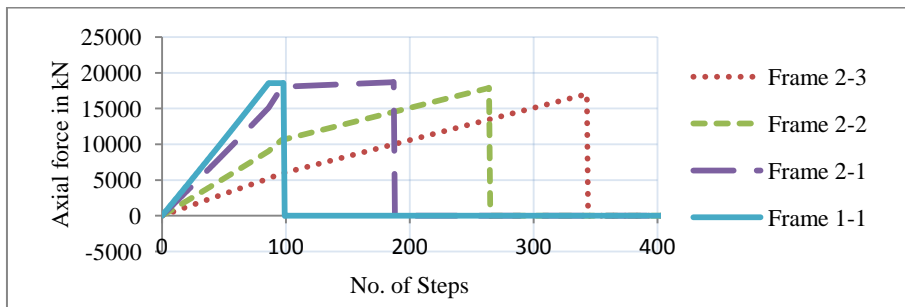

Figure 17. Variation of axial forces in nonlinear static and dynamic analysis of Case-3
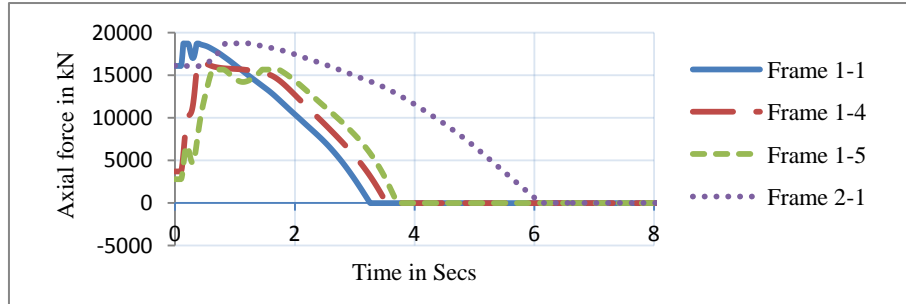
Figure 18. Variation of axial forces in nonlinear static and dynamic analysis of Case-3

The linear gravity load case has shown the rise in axial forces in cable 1-1. As per the GSA guidelines, it could have been easily concluded that cable 1-1 got damaged as the DCR value for cable 1-1 was more than 1. In the nonlinear static case, plastic hinges are found in cable 1-1. After the failure of cable 1-1, an important load carrying end cable, failure progression is found to occur. More hinges are found in different members.

In the linear dynamic analysis exceedance of the axial force in the same cable is noticed. The DCR value showed the maximum axial force in cable 1-1 exceeding the yield tension of cables. Although, the primary plastic hinge is found in cable 1-1 in the nonlinear dynamic procedure, a new progression path was noticed as shown in *Fig. 18.*

Fig. 19 and *Fig. 20* show the final state of the model obtained from the nonlinear static and dynamic procedure respectively, before total collapse of the whole structure.
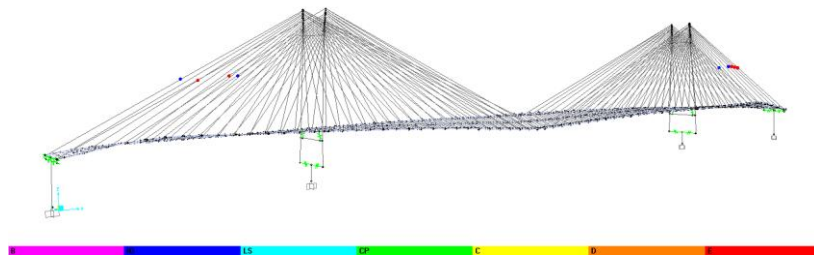

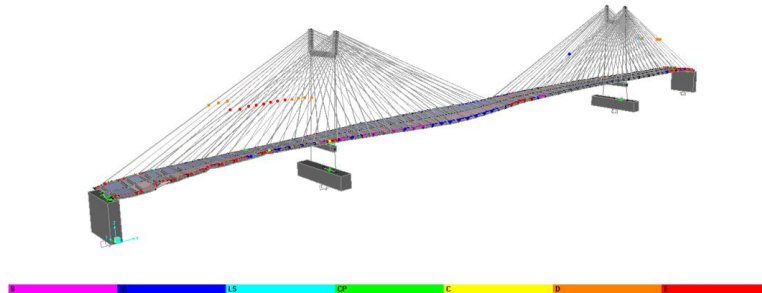*Figure 19.* Final state of nonlinear static analysis in Case-3

Figure 20. Final state of nonlinear dynamic analysis in Case-3

5.4 Loss of cable 1-30, 1-31, 1-32, 1-33 and cable 1-34 (Case-4)

The change in force and variation of displacements due to the sudden removal of cable 1-30, 1-31, 1-32, 1-33 and 1-34 is studied in this section. The vertical deflection of the centre (1) and quarter points (2 & 3) of the bridge deck are given in *Fig. 21* and the lateral displacement of the pylon tops (5, 6, 7 & 8) are plotted in *Fig. 22*. To highlight the variation, the displacements at the same points of the undamaged structure are also shown.
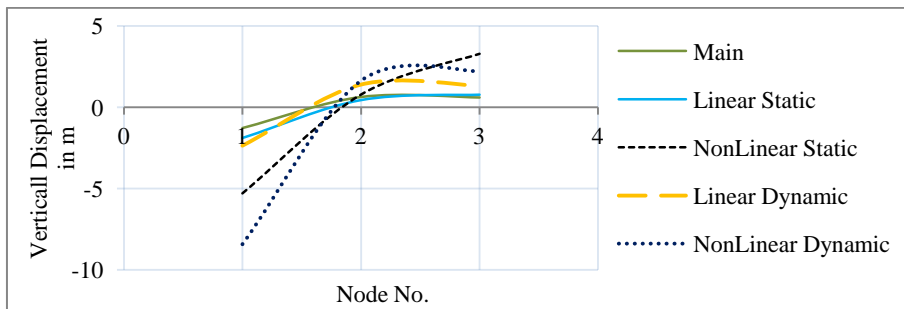

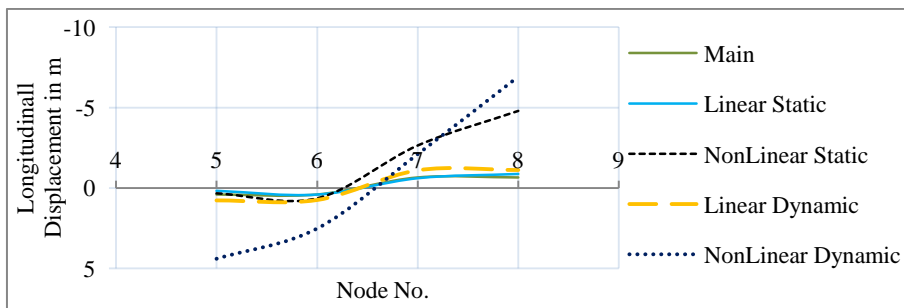*Figure 21.* Variation of vertical displacements in Case-4

Figure 22. Variation of longitudinal displacements in Case-4

Since, Cables 1-30, 1-31, 1-32, 1-33 and 1-34 are the cables holding the middle portion of the deck, significant changes are observed in the axial forces in cable 2-1 (right end cable) in the static and dynamic analysis as shown in *Fig. 23* and *Fig. 24*. *Fig. 23* and *Fig. 24* shows the variation in cable forces of the first four damage prone cables after the elimination of the mentioned cables, using the nonlinear static and dynamic procedures respectively.
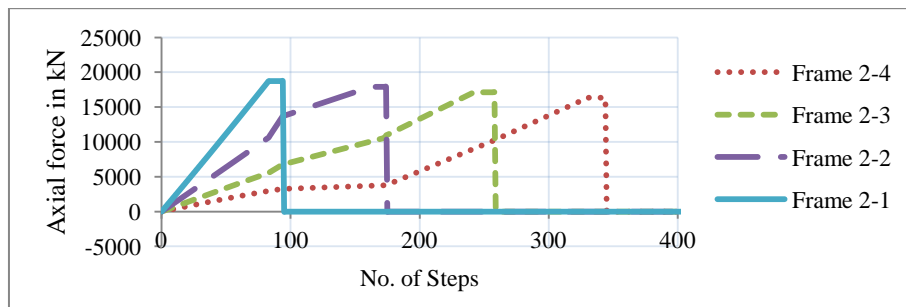

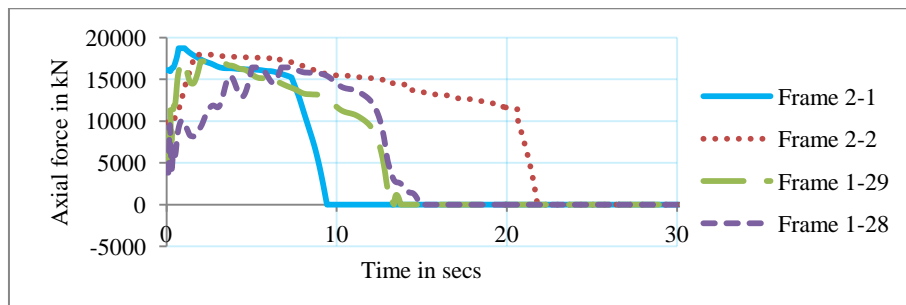*Figure 23.* Variation of axial forces in nonlinear static analysis in Case-4

Figure 24. Variation of axial forces in nonlinear dynamic analysis in Case-4

Fig. 23 shows an increase in axial force in cable 2-1 as observed in the linear gravity load case. As per the GSA guidelines, it can be concluded that cable 2-1 gets damaged since the DCR value for this cable is more than 1. In the nonlinear static case, as expected, plastic hinge is found in cable 2-1. Progressive failure is seen to occur after the failure of cable 2-1 which is an important load carrying end cable. More hinges are found in different members.

The value of the axial force overshoots in the same cable as observed in the linear dynamic analysis, shown in *Fig. 24*. The DCR value shows that the maximum axial force in Cable 2-1 has exceeded the yield tension values of cables. Plastic hinge is also found in Cable 2-1 in the nonlinear dynamic procedure. Thereafter a new path of collapse progression is again noticed.

5.5 Loss of cable 1-11, 1-12, 1-13, 1-14 and cable 1-15 (Case-5)

In this section the behaviour of the bridge model is studied for initial failure of cables 1-11, 1-12, 1-13, 1-14 and 1-15. It is clearly noticed from all the four analysis procedures that the model remains in a safe situation after these five cables rupture initially due to some accidental cause. All the axial forces in the remaining cables are observed to stay within the yield limit. The girders are also seen to be safe and no cracks have occurred in the bridge deck. So, further investigation was done by failing more nearby cables and it was observed that after an initial rupture of two more cables, cable 1-16 and 1-17, the model was still safe in all the four analysis. When cables 1-9 and 1-8 were ruptured, a collapse progression was observed to originate in the nonlinear dynamic analysis. So, at least seven cables on the same side of the pylon are required to fail to start a failure sequence. This is considered to be a very rare case and thus ignored in this present study.

5.6 Discussion of results

The mid part or the centre of the bridge deck undergoes maximum vertical displacement and the top of the pylon are subjected to maximum transverse and longitudinal displacements resulting due to high rotation developed due to its self weight.

Axial stresses in the cables connecting the end of the deck with the top of the pylons are very high under the self weight of the structural model. These are the most important load carrying members in the model. Thus, failure of these end cables increases the possibility to initiate the collapse progression throughout the structure.

As the pylons are approached from either end of the model, a decrease in the axial forces in cables is observed. Again a force increment is noticed as the centre of the deck is reached, although it is lesser than the previous variation. Also in the last failure case (Case-5) discussed in Section 5, it is noticed that, if failure occurs in a cable near to the pylon, the possibility of progressive collapse becomes less as the small amount of axial force incorporated with that cable(s) before failure, easily gets redistributed among the other nearby cables.

The collapse progression determined from the nonlinear static and dynamic cases follows a different pattern in each cable loss case. In the nonlinear static case the sudden impact force due to a cable snap and the dynamic unloading of a cable is ignored. The elimination of a cable leads to redistribution of axial force carried by that particular cable initially, throughout the whole structure. Whereas, in the dynamic analysis, the sudden push due to the dynamic unloading of that cable also contributes, creating a whole new different pattern of failure.

In the nonlinear static analysis a similar progressive collapse pattern for all the cable loss cases is found. When a cable is eliminated from the bridge model

and analysed, the bridge tries to get back to its symmetric configuration. For example, when cable 1-1 and cable 1-2 are eliminated, the symmetrically opposite cables 2-1 and 2-2 rupture in the first step. As the symmetry is reached, cable 1-3 fails due to the stress reformation. Then again cable 2-3 fails. This continues and after a certain step, cable 1-35 ruptures with cable 2-35 losing its load carrying capacity in the next step.

In the nonlinear dynamic analysis a different failure behaviour is noticed for all the cable loss cases. When a sudden push is generated on the anchorage joints of the pylons and the deck due to the snapping of a cable, it creates an extra deflection of both the elements, giving rise to an additional axial force in the neighbouring cables. So, the bridge model does not get a chance to attain its symmetric configuration. The nearby cables fail first and then one of the end cables fail. For example, when cable 1-1 and cable 1-2 are assumed to fail at first, cable 1-3 and cable 1-4 follows. Then cable 2-1 fails as axial tension increases in the opposite side of the model. Again cables 1-5, 1-6, and 1-7 get damaged due to the inertia effect of the pylons and the deck. Then Cable 2-2 fails due to stress redistribution. Thus, a new path of failure progression has occurred.

In the proposed structure, the loss of one cable does not lead to a progressive collapse in the nonlinear static case. However in the nonlinear dynamic case, the whole structure fails as the dynamic amplification factors and sudden impact forces participate with an important role. Thus, the conservative nature of a static analysis procedure is concluded, as it cannot provide complete information on structural response.

A definite concept behind the total collapse may also be derived. Due to the sudden loss of the mentioned cables for each case, upright deformations start to develop in the longitudinal girder of the damaged cable plane. The anchorage joints of the initially ruptured cable(s) on the pylon and deck are pushed in the opposite direction due to the unloading impact force and thus generate an extra axial force in the nearby cables. Normal forces acting on this bridge girder gets transferred to the longitudinal girder of the undamaged cable plane. These girders can't resist the additional normal forces and commence to buckle in the vertical direction. So, vertical deformations grow strongly and can't be stopped since the bridge deck is not restrained by fix supports in the longitudinal direction and thus, ultimate stresses exceeds in the bridge girder. The inertia effect of the pylon and deck and the downward deflection of the longitudinal girder of the intact cable plane finally caused the rupture of the nearby cable. This continues in the vicinity till the structure tries to regain its symmetric configuration and hence, an end cable fails in the other half of the bridge. The whole phenomenon discussed is repeated and the girders in the middle of the decks fail due to flexure. Forces get transferred to the pylons and they are pulled towards the centre of the deck, causing the final break down of the entire bridge structure.

6 CONCLUSIONS

The centre of the bridge deck and the top of the pylons are the most critical points of a cable stayed bridge. The end cables on either side of the bridge are the most vulnerable. Failure in these end cables increases the probability of a failure progression throughout the structure. Lesser the distance of the cable from the pylon, lesser will be the chance of collapse of the whole structure. So, strengthening of the end cables might offer better resistance against progressive collapse. The dynamic procedure provides a completely different trend of progressive collapse, compared to the static method. In the static procedure the structure tries to reach for its symmetrical configuration, whereas, in the dynamic study the impact force does not offer that much time. A nonlinear static analysis alone is conservative in nature and thus to trace the failure progression due to an initial failure of one or more cables, the nonlinear dynamic analysis starting with the initial stressed state given by the nonlinear static procedure is the best solution. A DAF of 2.0 is approximately justified by comparing the deflection values obtained from the static and dynamic analysis

REFERRENCES

[1] Starossek, U, "Progressive Collapse of Bridges – Aspects of Analysis and Design", *International Symposium on Sea-Crossing Long-Span Bridges*, Mokpo, Korea, 2006.
[2] Liu, Y, Han, B, Xiao, Ma, "Advances in Progressive Collapse of Bridge Structures", Pacific Science Review, Vol. 13, No. 3, pp. 173-181, 2011.
[3] Cai, JG, Xu, YX, Zhuang, LP, Feng, J, Zhang, J, "Comparison of various procedures for progressive collapse analysis of cable-stayed bridges", Journal of Zhejiang University-Science A, Vol. 13, No. 5, pp. 323-334, 2012.
[4] Wolff, M, Starossek, U, "Cable-loss analysis and collapse behaviour of cable-stayed bridges", *Proc. 5th International, Conference on Bridge Maintenance, Safety and Management, (IABMAS2010)*, Philadelphia, USA, 2010.
[5] Das. R, "Progressive Collapse of a Cable Stayed Bridge", M. Tech. Thesis, Department of Earthquake Engineering, Indian Institute of Technology, Roorkee, India, 2015.