iJRASET
International Journal For Research in
Applied Science and Engineering Technology

# INTERNATIONAL JOURNAL FOR RESEARCH

IN APPLIED SCIENCE & ENGINEERING TECHNOLOGY

**Volume:** 7 **Issue:** X **Month of publication:** October 2019

**DOI:** http://doi.org/10.22214/ijraset.2019.10121

International Journal for Research in Applied Science & Engineering Technology (IJRASET)
*ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor: 7.177*
*Volume 7 Issue X, Oct 2019- Available at www.ijraset.com*

# Progressive Collapse of Cable Stayed Bridge under Blast Loading

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***Abstract: Progressive collapse is a major threat causes the more demolitions of structure and leads to the loss and damage of lives. The main causes of the progressive collapse are earthquake and severe wind which results in gradual and successive failure of number of elements of the structure. The present paper includes linear static analytical procedures. For linear static analysis loading is considered as per the Post Tensioning Institute (2001) recommendations and GSA (2003) progressive collapse guidelines. Alternate path (AP) method is used for progressive collapse analysis of the cable stayed bridge. The cable stayed bridges are modelled in SAP 2000 with various cable arrangements and studied the deflection of girder under static loading condition. Also studied the axial forces developed in the cables under the cable loss. The results are taken with respect to the various cable arrangement and number of cable removed.***
***Keywords: Blast loading analysis, Bridge, SAP 2000, Progressive failure***

## I. INTRODUCTION

Cable stayed and suspension bridges are the largest structure designed as platform for carrying people and vehicles. Both the bridges are held up by the cables, their modes of operations are very different. Cable stayed bridges are less expensive quicker to build and has grater stiffness. These bridges are subjected to blast loads causes progressive failure. Progressive collapse is a major threat in such bridges. It is dynamic event caused by localized structural injuries, disturbing the initial load equilibrium causes vibrations in the structure so it either gets new equilibrium or collapses. The cable stayed bridges have three types according to the cable arrangement system i.e. Harp, Fan and Radial. In this paper, the analysis of these three bridges against the progressive collapse is done.

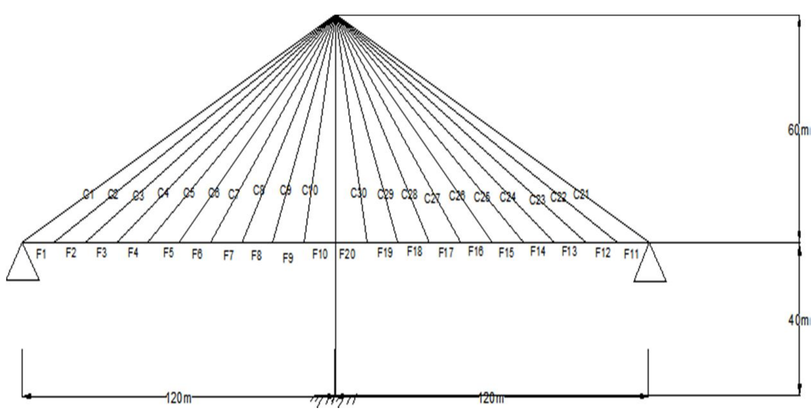

Figure -1: Typical Geometry of Cable Stayed Bridge

## II. LITERATURE REVIEW

1) *R. Das et al (2016)*[1]*:* The authors proposed 'Progressive Collapse of a Cable Stayed Bridge'. This study demonstrates modelling and analysis of a typical cable stayed bridge through a nonlinear dynamic procedure. The results indicated a decrease in the possibility of failure progression of the cable stayed model when the location of the failed cables was closer to the pylon. A definite progressive collapse pattern was also identified along this procedure. The end cables of either side of the bridge are the most vulnerable cables. Rupture in these end cables increases the probability of a failure progression throughout the whole structure. Lesser the distance of the cable from the pylon, lesser will be the chance of failure of the whole structure.
2) *Bo Sun et al (2016)*[2] *:* This paper presents 'Probabilistic aero stability capacity models and fragility estimates for cable-stayed bridge decks based on wind tunnel test data'. Wind resistance design is of vital importance for long flexible structures like

cable-stayed bridges. The developed models are constructed to give balanced estimates of the capacities of interest and properly account of the relevant uncertainties. The measured capacity values from wind tunnel tests are used to determine the subsequent statistics of model parameters through a Bayesian approach.

3) *Amir Fatollahzadeh et al (2016)*[3] *:* One of the causes of Progressive collapse is the failure in a number of elements during ultimate events such as earthquake or severe wind. The results show that the mentioned situation during Tabas & Loma Prieta earthquakes will lead to progressive collapse, whereas the structure can withstand two cables removal during the Bam earthquake. To avoid this destruction, six base isolations are installed below the structure.
4) *S.K. Hashemi et al (2016)*4] *:* Over the past two decades, blast loads have been recognized as one of the extreme loading events that must be considered in the design of important structures such as cable-stayed bridges. However, design provisions for blast-resistant bridges are very limited and mostly empirical owing to an inadequate understanding of the local and global dynamic response of the bridge components (piers, deck and cables) subjected to blast loading scenarios. Three different explosive sizes such as small (01W), medium (04W) and large (10W), are considered (W being the TNT equivalent explosive weight index) and placed at different locations above the deck level to determine the influence of the size and location of the blast loads on the global and local response of the bridge components. In certain, the outcomes of the computer recreations are employed to designate the type and extent of harm on the pylon and deck, and also to investigate the likely cable Removal of Cables circumstances associated with a cost of quay.
5) *M.A. Bradford et al (2016)*5] *:* This paper summarizes the analysis and design concepts of chimneys as per Indian codal provisions incorporation was also made through finite element analysis. Effect of inspection manhole on the behaviour of Cantilever steel chimney, two chimney models one with the manhole and other without manhole were taken into consideration. These models are analysed by finite element software STAAD Pro, emphasis also placed on effect of geometric limitations on the design aspects in designing chimney.
6) *Yufen Zhou et al (2015))*[6] *:* This study includes 'Numerical investigation of cable breakage events on long-span cable- stayed bridges under stochastic traffic and wind.' Cable breakage (Removal of Cables) events can be disastrous to cable-stayed bridges because of potential risks of progressive collapse following the initial failure of stay cables. The results also indicate that service traffic and wind loads as well as complex coupling effects with the bridge are important to the bridge reaction following cable-Removal of Cables trials. In the final part of the study, response envelope analysis is made and a comparative examination is also conducted between the result from the radical FE-based nonlinear dynamic approach and those from the equal static approach as suggested by the Post-Tensioning Institute (PTI).
7) *Allan Larsen et al (2015))*[7] *:* This paper proposed 'Dynamic wind things on suspension and cable-stayed bridges. Suspension and cable-stayed bridges are highly flexible and lightly damped structures whose Eigen method shapes and eigen regularities are well predicted by Finite Element methods. Structural dynamics is the key section of the aeroelasticity of these bridges and must be calculated in order to assess wind effects on the bridges. The extant paper discusses the most public phenomena related to wind things on suspension and cable-stayed bridges underlining the prominent role of the dynamic forces of the structures and emphasizing the authors' sentimentality.
8) *Kuihua Mei et al (2015))*[8] *:* In this paper the Author carried out analysis of self-supported steel chimney with effect of manhole and geometric properties. Arbitrary models of steel stacks were selected and they were analyzed using ANSYS and Mathcad. Basis of selection of geometric parameters was top to bottom diameter ratio. Limitations of codal conditions were also highlighted. No mathematical equations or correlations were established by the authors for dynamic response variance and variance in geometry.
9) *A.M.B. Martins et al (2015))*[9] *:* This paper addresses 'Optimization of cable forces on concrete cable-stayed bridges including geometrical nonlinearities.' Cable-stayed bridges are appealingly pleasing and have been generally used all over the world, oscillating from small pedestrian bridges to extended span railway and road bridges. The calculation of the cable forces is a typical aspect of a cable-stayed bridge project when related to other forms of bridges. Cable tensioning is mandatory to controller the geometry, stress spreading and to exact construction errors.
10) *P. Lonetti and A. Pascuzzo (2014))*[10] *:* The authors proposed 'Optimum design analysis of hybrid cable-stayed suspension bridges'. It includes a design methodology to predict finest post-tensioning forces and dimensioning of the cable system for hybrid cable-stayed suspension (HCS) bridges is wished-for. The structural model is built on the combination of an FE style and an iterative optimization method. The former is able to afford a polished explanation of the bridge structure, which takes into interpretation geometric nonlinearities elaborate in the bridge apparatuses. The latter is operated to augment the shape of post-tensioning forces as well.

## III.METHODOLOGY

Following methodology is adopted for this research. It includes Study of codes for progressive collapse, blast loading, modelling in SAP2000, validation and results.

PHASE I

FINALISE THE SUBJECT

LITERATURE SURVEY

DECIDE THE SEQUENCE FOR THE ANALYSIS OF THE MODELS

PHASE II

LOADING CALCULATION

MODELLING IN SOFTWARE

CABLE STAYED BRIDGE ANALYSIS (VALIDATION)

INTERPRETATION OF THE RESULTS

COMPARISION OF CABLE ARRANGEMENT AND PYLON GEOMETRY

## IV.MODELLING AND VALIDATION OF SOFTWARE

In this chapter progressive collapse, blast loading, modelling in SAP2000 is done and the validation of the model is done and the results are compared.

### *A. SAP2000 Modelling*

As the bridge with two pylons, three spans i.e. two end spans and one middle span, is quite difficult to analyze. So here bridge with two end spans with single pylon is finalized. The schematic diagram of the cable-stayed bridge is as shown in Fig1. The bridge has one single tower of 100 m high and two equal end spans of 120 m. The girder is assumed to be hinged with the tower at a height of 40 m above from base and simply supported at both ends. It is also supported by 40 stay cables, 20 on each side. Cables have the spacing of 12m. The cross section of the tower is 5m x 5m.. The box girder is considered with thickness of 0.2 m and side thickness is 0.3m. The width of girder is 26.5m which consists of 6 lanes of 3.75m each and two pedestrian tracks of 2m each. Depth of the girder is 3m.

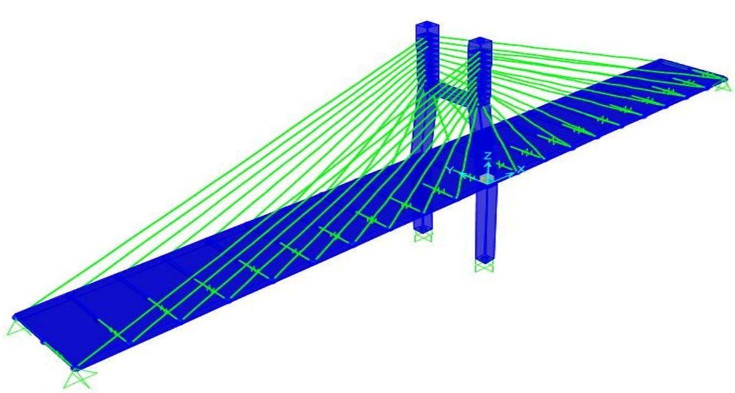

Figure -2: Fan Cable arrangement with H-type pylon

Table 1 Combinations of cable arrangement and pylon type

| Sr.No. | Cable Arrangements | Pylon |
|---|---|---|
| 1 | Harp Arrangement | A – Type |
| | | H – Type |
| 2 | Fan Arrangement | A – Type |
| | | H – Type |
| 3 | Radial Arrangement | A – Type |
| | | H – Type |

*B. Data Required For Analysis*

Table 2 Reinforced pylon and box girder properties with cable and tendons properties.

| | |
|---|---|
| Ultimate Strength | 1860 KN/m2 |
| Modulus of Elasticity | 2.0 x $10^8$ N/mm2 |
| Poisson's ratio | 0.3 |
| Weight density | 76.98 KN/m3 |
| Grade of the concrete | M40 |
| Modulus of Elasticity | 3.16 x $10^5$N/mm2 |
| Poisson's ratio | 0.2 |
| Weight density | 24.99KN/m3 |

According to the Post-Tensioning Institute (2001) recommendations and GSA (2003) progressive collapse guidelines, following loading combination is used while evaluating the progressive collapse.

Load=1.0×DL+0.75×LL+1.0×PS+1×CL

Where DL - Dead load

LL - Live load

PS - Prestressing Force.

CL- Equivalent Force due to cable failure

Dead load is calculated by the SAP2000 programme itself. Live and Prestress forces can be calculated analytically**.** The cable force depends on factors such as length of the span, number and size of panels and angles of inclination of the cables, dead weight of deck and live loads. In this case only dead load case is considered. The force in the cable stay is (Krishna Raju N., 1998),

$P_c$ = $(R/\sin\alpha)$ + Force due to self-weight of the

Where, $P_c$= force in the cable

S = spacing of cable

$W_d$ = Total load per meter of deck

$\propto$= Angle of inclination of cable with horizontal

$R$ = Vertical reaction at cable stay node = $S \times W_d$

The cable stayed bridge of 120m span is taken for the validation of the results. The vertical reaction at each node where cable connects to the deck is calculated as follows.

Weight of the deck.

$(W_d)$ =27.7 x 25 = 692.5 KN/m

Vertical reaction R1 = R2 = R3=692.5 x 10= 6925 KN

$\sin\propto=28/60$

$\propto = 25.02^0$

Cable stay force in C1 = 819.5/sin ($25.02^0$) = 1937.65

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Self-weight of cable = Density of material ×Area of cable × Length of cable

$= 78.5 \times 0.785 \times 0.1^2 \times 66.21$

$= 40.80$kN

$P_C = 16375.02 + 40.80$

$= 16415.8$kN

Table 1: Calculation of Cable Forces

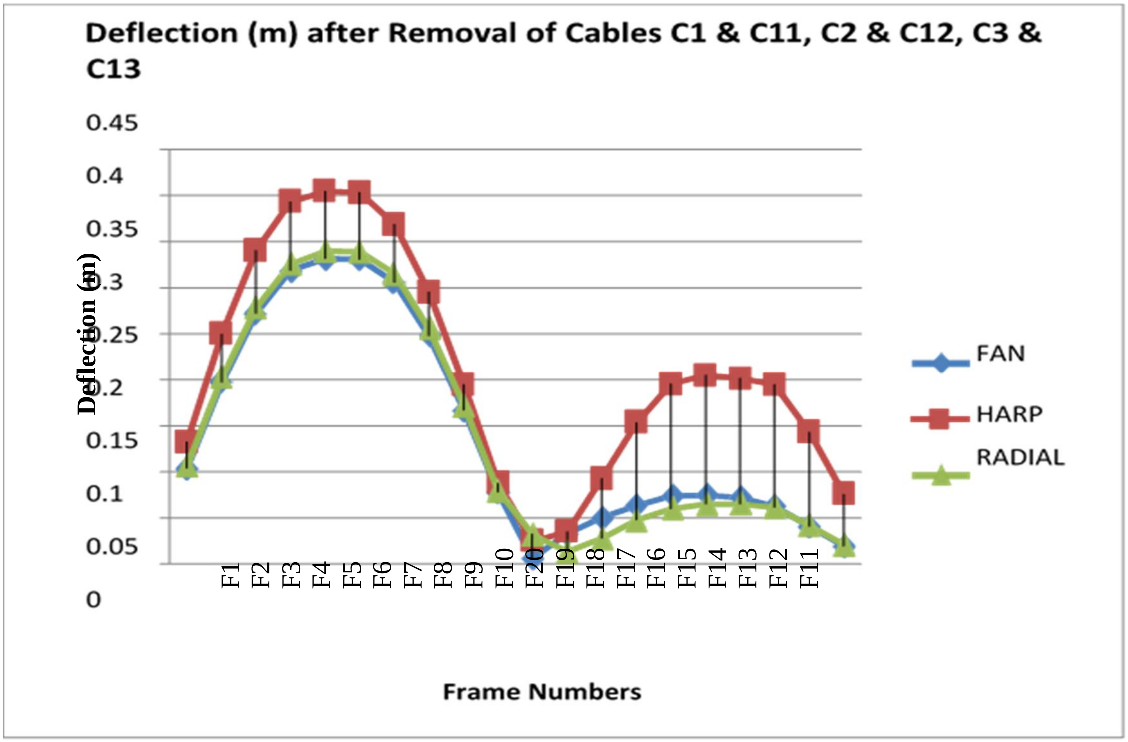
| Cable No. | Vertical reaction (kN) | Angle of cable (˚) | Cable Stay force (kN) | Length of cable (m) | Self-weight of cable (kN) | Total Cable Forces (kN) |
|---|---|---|---|---|---|---|
| 17 | 6925 | 25.02 | 16375.02 | 66.21 | 40.80 | 16415 |
| 18 | 6925 | 28.48 | 14522.32 | 57.30 | 35.30 | 14557. |
| 19 | 6925 | 32.63 | 12842.81 | 48.82 | 29.04 | 12871. |

Table 2: Comparison of Results

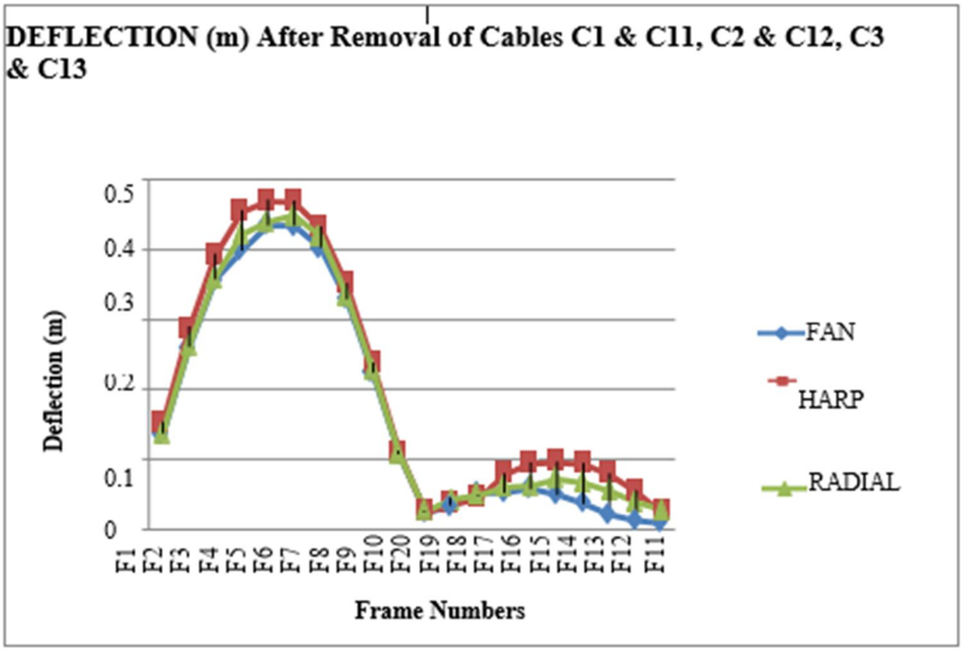
| Cable No. | Total Cable Forces (Analytically)(kN) | Total Cable Forces (by using SAP2000) (kN) | % Error |
|---|---|---|---|
| 17 | 16415.8 | 16095.32 | 1.952 |
| 18 | 14557.62 | 14377.62 | 1.236 |
| 19 | 12871.85 | 12621.85 | 1.942 |

## V. RESULT AND DISCUSSION

In this chapter, all the results that were obtained in comparison of Deflection of girder for various cable arrangements with A-type Pylon and H-type Pylon.

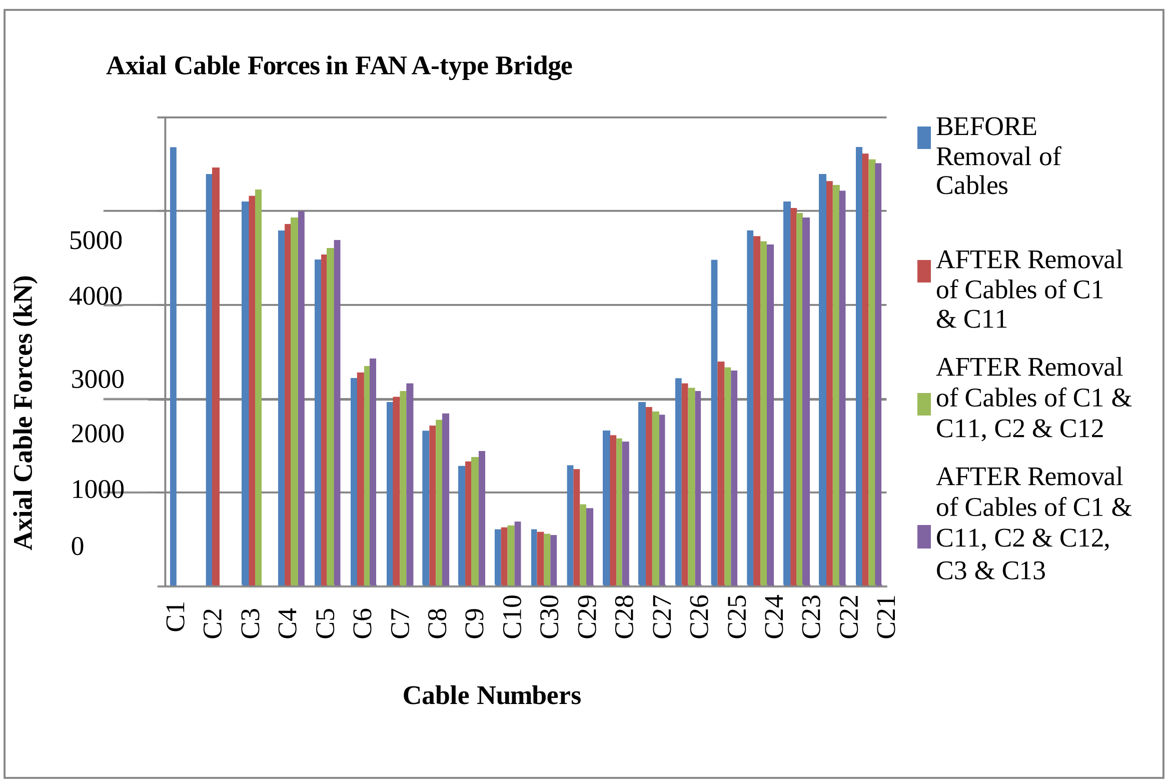



Graph -1: Comparison of Deflection of girder for various cable arrangements with cable Removal of Cables of C1 and C11, C2 and C12, C3 and C13 A-type pylon




Graph -2: Comparison of Deflection of girder for various cable arrangements with cable Removal of Cables of C1 and C11, C2 and C12, C3 and C13 H-type pylon

From the obtained graph it is observed that FAN arrangement has the least deflection of girder i.e. 0.4345m whereas HARP arrangement has maximum deflection of girder i.e. 0.4714m .




Graph -3: Axial Cable Forces in FAN various cable arrangements with cable Removal of Cables of C1 and C11, C2 and C12, C3 and C13 A-type Bridge

Table 6.9 Axial Forces after Removal of Cables in FAN A-type Bridge

| Cable No. | CABLE AXIAL FORCES (KN) | | | | | | |
|---|---|---|---|---|---|---|---|
| | BEFORE Removal of Cables | AFTER Removal of C1 & C11 | % CHANGE IN FORCE | AFTER Removal Of Cables of C1 & C11, C2 & C12 | % CHANGE IN FORCES | AFTER Removal Of Cables of C1 & C11, C2 & C12, C3 & C13 | % CHANGE IN FORCE |
| C1 | 4680 | 0 | | 0 | 0 | 0 | 0. |
| C2 | 4393.2 | 4460.24 | 1.5253 | 0 | 100.0 | 0 | 0. |
| C3 | 4100.53 | 4166.06 | 1.5981 | 4233.8 | 1.6282 | 0 | 100 |
| C4 | 3797.65 | 3861.28 | 1.6755 | 39305 | 1.7932 | 4004.73 | 8880 |
| C5 | 3478.94 | 3540.26 | 1.7626 | 3609.7 | 1.9617 | 3687.21 | .1470 |
| C6 | 2226.59 | 2285.66 | 2.6529 | 2355.7 | 3.043 | 2437.62 | .4775 |
| C7 | 1969.13 | 2024.64 | 2.8190 | 2091.5 | 3.3048 | 2171.17 | 3.8067 |
| C8 | 1665.41 | 1715.57 | 3.0119 | 1775.8 | 3.5137 | 1847.62 | 4.0414 |
| C9 | 1295.63 | 1336.45 | 3.1506 | 1384.0 | 3.5587 | 1439.25 | 3.9913 |
| C10 | 613.45 | 638.04 | 4.0085 | 665.28 | 4.2693 | 695.45 | 4.5349 |
| C30 | 613.45 | 591.09 | 3.6450 | 572.45 | -3.1535 | 556.83 | -2.7286 |
| C29 | 1295.63 | 1257.68 | 2.9291 | 872.87 | -30.5968 | 847.36 | -2.9225 |
| C28 | 1665.41 | 1618.56 | 2.8131 | 1579.1 | -2.4343 | 1546.02 | -2.0986 |
| C27 | 1969.13 | 1916.98 | 2.6484 | 1873.1 | -2.2890 | 1836.19 | -1.9705 |
| C26 | 2226.59 | 2170.85 | 2.5034 | 2123.8 | -2.1641 | 2084.36 | -1.8603 |
| C25 | 3478.94 | 2395.76 | -31.135 | 2346.35 | -2.0624 | 2304.79 | -1.7713 |
| C24 | 3797.65 | 3736.24 | 1.617 | 3683.64 | -1.4078 | 3638.89 | -1.2148 |
| C23 | 4100.53 | 4037.15 | -1.545 | 3982.8 | -1.3450 | 3936.67 | -1.1595 |
| C22 | 4393.23 | 4328.3 | 1.478 | 4272.66 | -1.2855 | 4225.35 | -1.1073 |
| C21 | 4680 | 4614.01 | 1.410 | 4558.33 | -1.2068 | 4512.01 | -1.0162 |

It is observed that in FAN A-type bridge the maximum increase in axial cable forces as 4.0085%, 4.2693% and 4.5349% after the Removal of Cables of two, four and six critical cables.

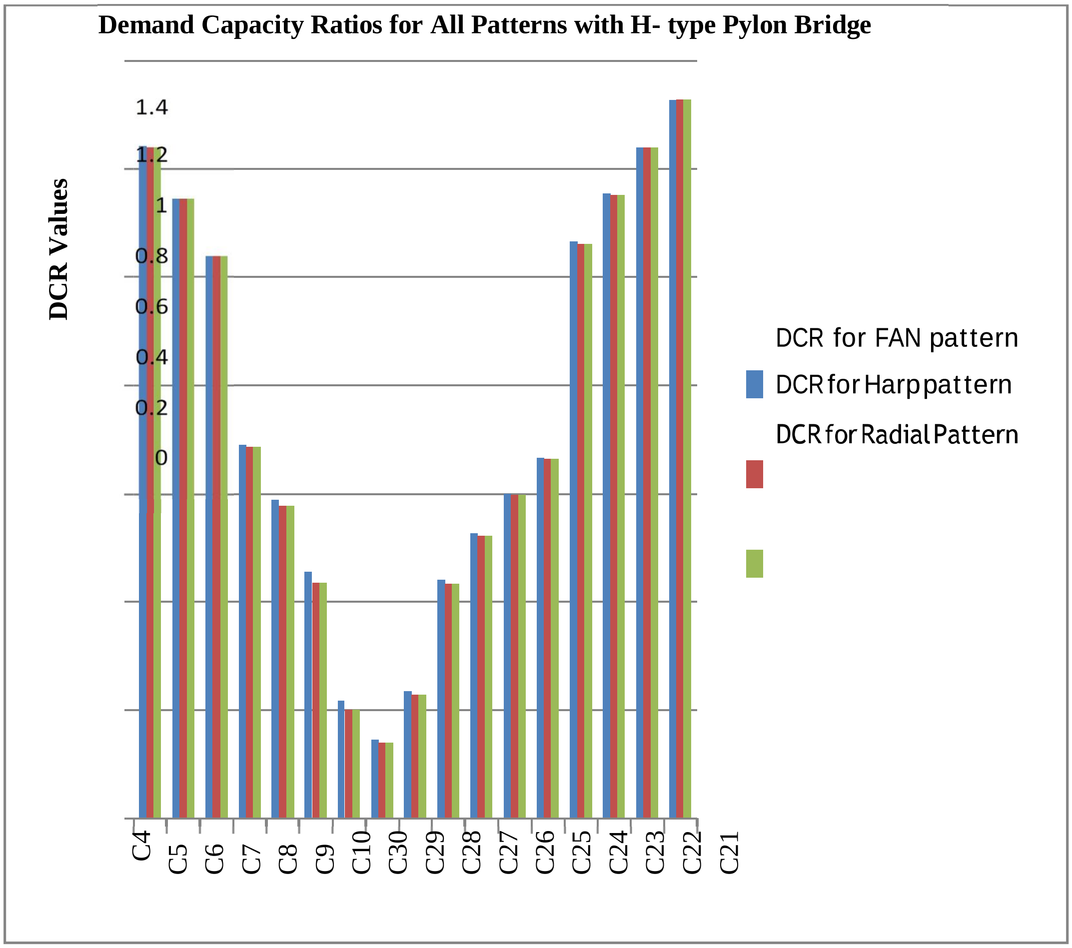
Similarly, It was observed that in HARP A-type bridge the maximum increase in axial cable forces as 2.0414% , 2.272% and 2.4119% after the Removal of Cables of two , four and six critical load cables respectively.

It is observed that in RADIAL A-type bridge the maximum increase in axial cable forces as 2.9167% , 3.4674% and 4.0942% after the Removal of Cables of two , four and six critical load cables respectively.

It is observed that in FAN H-type bridge the maximum increase in axial cable forces as 7.6323%, 8.3883% and **9.0497%** after the Removal of Cables of two , four and six critical load cables respectively.

It is observed that in HARP H-type bridge the maximum increase in axial cable forces as 5.5040%, 6.4715% and 7.2696% after the Removal of Cables of two , four and six critical load cables respectively.

It is observed that in RADIAL H-type bridge the maximum increase in axial cable forces as 6.2146%, 6.8910% and 7.6178% after the Removal of Cables of two , four and six critical load cables respectively.




Cable Numbers

Graph -4: Demand Capacity Ratios for All Patterns H- type Pylon Bridge

Graph -4 shows very little variation in the DCR values. So it can be concluded that the variation in cable forces due to pylon geometry has less significance. They are just used to calculate DCR values to avoid the progressive collapse. Here, The cables C7, C8, C9, C10, C30, C29, C28, C27, C26 and C25 have less than one. So they can redistribute the stresses and they are not susceptible to progressive collapse.

## VI. CONCLUSION

*A.* In case of cable arrangement with pylon geometry, the FAN cable arrangement with A-type pylon gives best results against progressive collapse. HARP cable arrangement with H-type pylon gives worst results progressive collapse.

*B.* When only Cable arrangement is considered the maximum deflection obtained is 0.4714m in HARP cable arrangement with H-type pylon whereas the FAN cable arrangement with A-type pylon gives least deflection0.3317m.

*C.* The axial cable forces are generated more in the Fan pattern. And least axial forces generated in the Harp pattern. So as per the demand capacity ratio the Fan pattern cables are more susceptible to the cable removal of cables.

*D.* In case of axial cable forces, maximum percentage increase occurred as 9.049% in FAN cable arrangement with H- type Pylon Bridge.

*E.* As per the DCR values it can be concluded that the pylon geometry has less significance in the variation of cable forces. These values are very close.

*F.* FAN cable arrangement with A-type Pylon can be considered as the best possible combination against the progressive collapse.

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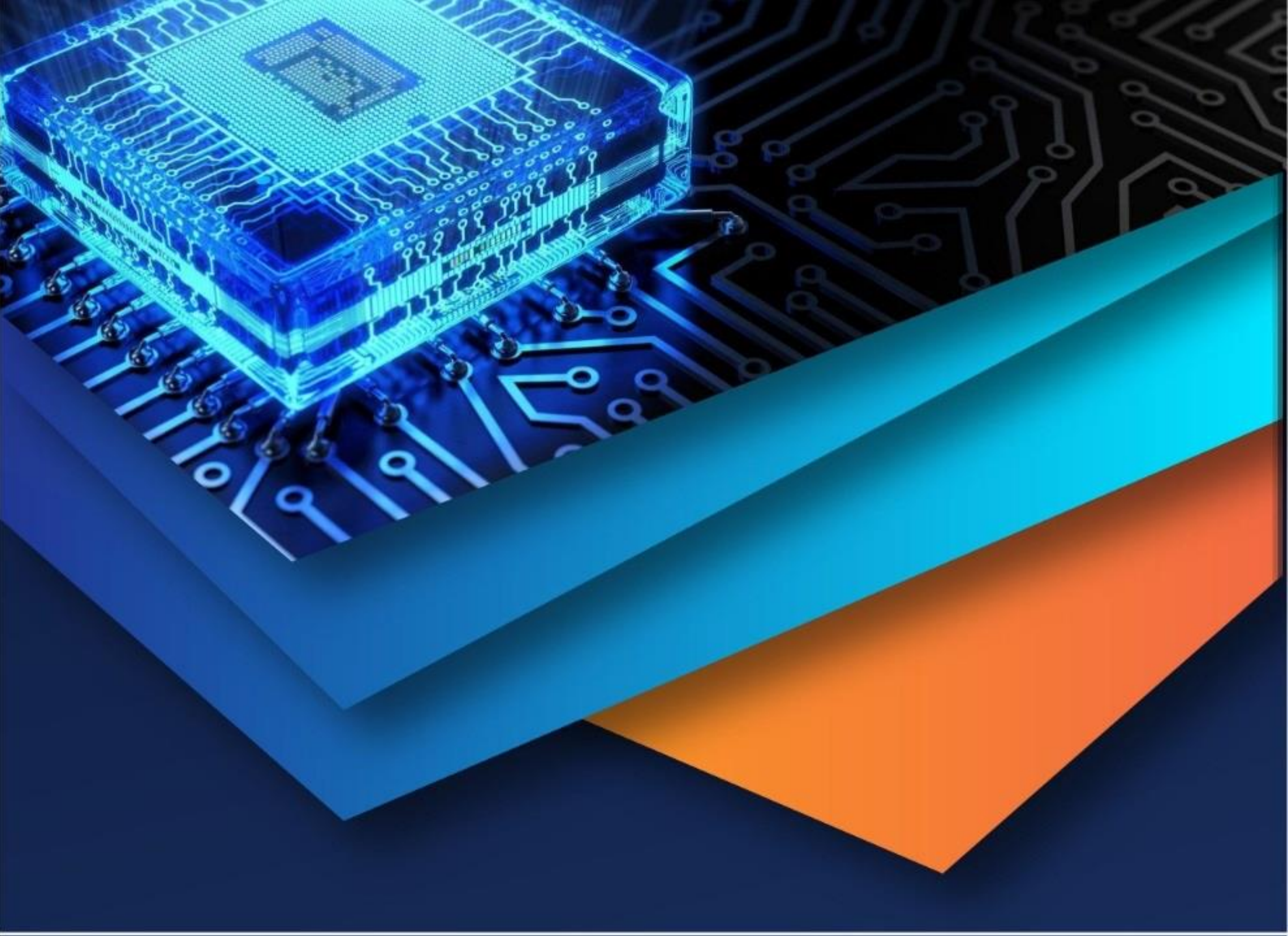



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