A COMPARATIVE STUDY OF PROGRESSIVE COLLAPSE OF CABLE-STAYED BRIDGE USING SAP 2000

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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 modeled 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 lost.

1. INTRODUCTION

Cable-stayed and suspension bridges are the largest structure designed as a 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 static and dynamic loads causes progressive failure. Progressive collapse is a major threat in such bridges. It is dynamic event caused by localised structural injuries, disturbing the initial load equilibrium causes vibrations in the structure so it either gets new equilibrium or collapses.

2. LITERATURE REVIEW

R. Das and A. D. Pandey [1] have demonstrated modelling and analysis of a typical cable-stayed bridges through a nonlinear dynamic procedure response of the structural model for multiple types of critical cables loss is discussed. It concludes that the possibility of progressive failure is decreased when the failed cables were closer to pylon. Uwe starossek has proposed the typology and progression collapse of structures different types of collapse are pancake type, zipper type, Domino type, section type, instability type, mixed type [2]. Amir fatollahzadeh has analysed the progressive collapse of cable-stayed bridges due to cable failure during the earthquakes i.e. Tabas, Loma Prieta and Bam. The research reveals that only two elements are capable of causing consequent damage. To avoid the destruction six base isolations are used below the structure[3]. Jian- Gua CAI and Yi-Xiang Xu have done the comparison of the linear static, nonlinear static, linear dynamic and non-linear dynamic procedure for progression collapse analysis of cable-stayed bridge. It concludes that the dynamic amplification factor of 2 is good for the static analysis procedure [4].

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 with various type of pylons against the progressive collapse is done.

3.OBJECTIVE OF STUDY

- To study the effect of linear static loading on cable-stayed bridge with various cable arrangement.
- To compare the axial cable forces and the deflection of girders under the progressive collapse mechanism.
- To find out the most stable cable arrangement against the progressive collapse.

4.GEOMETRY OF CABLE-STAYED BRIDGE

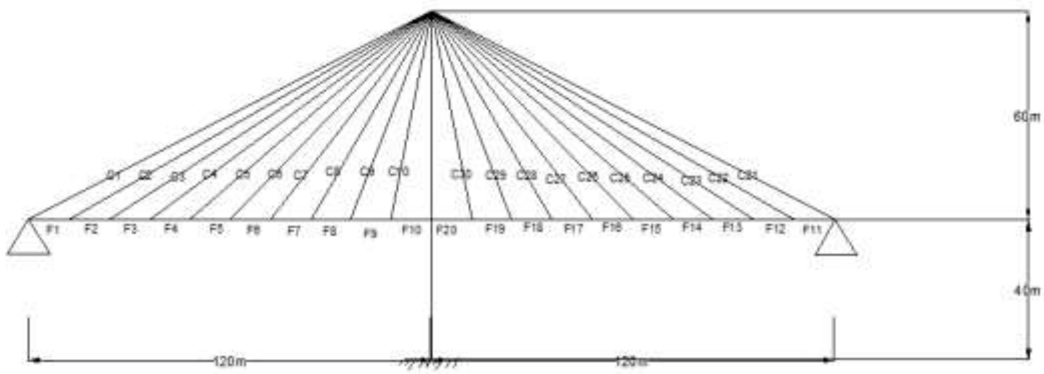

Fig.4.1 Typical Geometry of Cable-stayed Bridge

As the bridge with two pylons, three spans i.e. two end spans and one middle span is quite difficult to analyse. So here bridge with two end spans with single pylon is finalized. The schematic diagram of the cable-stayed bridge is as shown in Fig.3.1. 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 the 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 pylon is an H-type pylon, having the angle of 26.6^0. The box girder is considered with a thickness of 0.5 m, side thickness is 0.3m. The width of the girder is 26.5m which consists of 6 lanes of 3.75m each and two pedestrian tracks of 2m each.   The depth of the girder is 3m.

- **MATERIAL PROPERTIES**

Following tables shows the materials properties used for the bridge.

Table.I reinforced pylon and box girder properties

Grade of the concrete	M40
Modulus of Elasticity	3.16×10^7 N/mm^2
Possion's ratio	0.2
Weight density	24.99KN/m^3

Table. II Cable and Tendons properties

Ultimate Strength	1860 KN/m^2
Modulus of Elasticity	2.0 x 10^8 N/mm^2
Possion's ratio	0.3
Weight density	76.98N/m^3

5. MODELING BY USING SAP2000

SAP2000 is the easiest most productive solution for structural analysis and design needs. It can analyse simple 2D frames as well as the complex 3D structures. It is the most suitable finite element tool for modelling and progressive collapse analysis of cable-stayed bridges.

The three types of cable arrangements i.e. Harp, Fan and Radial has modelled by using SAP2000 as shown in following figures.

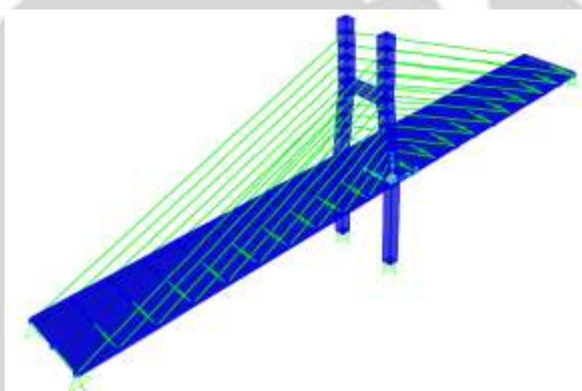

Fig. 5.1 Harp Cable arrangement

Fig.5.1.shows the FE model of harp cable arrangement. In harp cable arrangement system the cables are connected to two tower at different heights and parallel to each other. Though it seems more pleasing aesthetically, it can cause bending moments in the tower and the whole pattern tends to less stable.

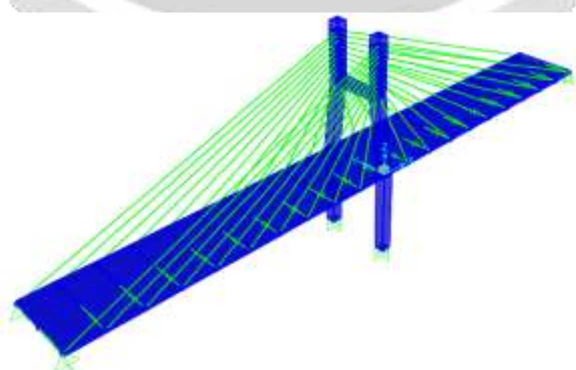

Fig. 5.2 Fan Cable arrangement

Fig.5.2. shows the FE model of the fan cable arrangement. In fan type cable arrangement, the cables are connected with a steeper slope. This system gives most effective support of vertical deke force and leads to smallest cable diameter.

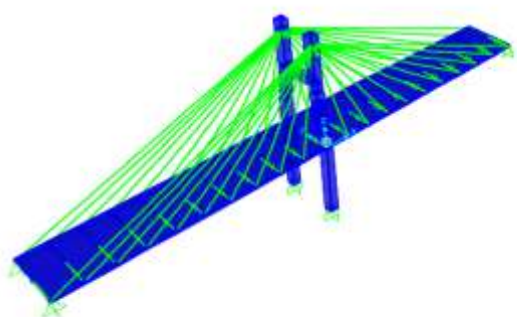

Fig. 5.3 Radial Cable arrangement

Fig.5.3 shows the FE model of radial cable arrangement. In radial cable arrangement, all the cables are anchored at the common point at the tower. In this good detail are difficult to be achieved. The concentration of anchorages can cause structural difficulties**.**Similarly the models for the various cable arrangements with A- type pylon and Y- type pylon are done in SAP2000.

6.LOADING CONDITION

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+ 1.0*CL……………. (1)

Where DL - Dead load
LL - Live load
PS - Prestressing Force
CL- Equivalent Force due to cable failure

6.1 Prestress Force Calculations for Box Girder

By considering the weight density of concrete 25kN/m^3

DL= Self Weight of Box Girder
=304 KN/m

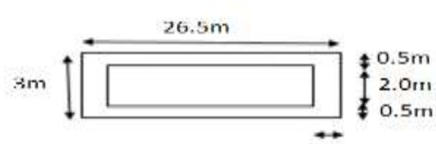

Fig.6.1.1 Box Girder

Moment of inertia of Box Girder,

$I= (BD^3/12) – (bd^3/12)$…………………..(2)
$=21.69\ m^4$

Where:

B- Outer width of box girder
D- Outer depth of box girder
b- Inner width of box girder
d- Inner depth of box girder
Section of modulus

$Z= (I/y)$ ………………………………….(3)
$=14.46\ m^3$

Where:
y- Distance of Extreme fibre from the neutral axis

For Presetting Forces,

$(P/A) \pm (P.e/z) = M/z$……………………..(4)

Where:
P- Prestressing force
A - Cross section area of Box Girder
M - Maximum bending moment
e - Eccentricity of prestressing force

Considering the parabolic prestressing and eccentricity is taken as 10%

e = 10% of 3m= 0.3m

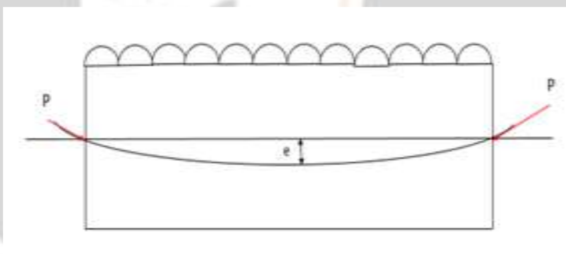

Fig.6.1.2 Typical Diagram of Prestress Force

Maximum Bending Moment,

$M = (wl^2)/8$…………………………..(5)

Where:
W-Dead load+ pedestrian load
l- Length of segment of Box Girder

For Live Load,

Pedestrian Live Load
For effective span > 30m

p = [(13.3) + (400/L)] * [(17-b) / (142.6)] KPa……..…(6)

Where,

b = Width of footpath

L = Length of effective span

p = 1.3625 KN/m^2

= 1.3625 * 26.5

p = 36.1065 KN/m

As per IRC specifications,

For Heavy Loading, a load of 1.93 tons/m Length of each traffic lane is considered.

1.93 * 9.81 = 18.933 KN/m^2

Total Heavy Loading for 6 Lanes

= 6 * 18.933

= 113.599 KN/m

Total Live Load = Pedestrian Load + Traffic Load

= 149.706 KN/m

Total Load, w = 453.706 KN/m

Max. Bending Moment,

= $(wl^2) / 8$………………From Eq. 5

= 8166.708 KN-m

Prestressing force P,

(P/A) ± (P.e/z) = M/z………………(7)

i) (P/A) + (P.e/z) = M/z,
P = 5467.37 KN.

ii) (P/A) - (P.e/z) = M/z
P = 9231.62 KN

7.CONCLUSION

From the above results and discussion, it is concluded that

- The deflection of girder at the other side of the pylon cannot be considered as negligible under the loss of outside cables. The vertical deflection at the other side of the pylon decreases as the location of the lost cable approaches the pylon.
- The cables adjacent to the ruptured cable do not reach the tension yield and the maximum nodal vertical displacement decreases when the lost cables are near the pylon.
- When only Cable arrangement is considered the maximum deflection is obtained in HARP cable arrangement whereas the FAN cable arrangement gives least deflection.
- 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.

- The axial forces in the cables start increasing in the adjacent of the lost cables up to the location of the pylon, after the pylon, the axial forces in cables starts decreasing.
- The cables which are in the vicinity of the pylon have very less axial forces, so they have very less possibility of cable loss.

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

8. REFERENCES

[1] R. Das, A. D. Pandey, "Progrssive Collapse of a Cable-stayed Bridge", 12th international conference on vibration problems, Elsevier ICOVP 2015.

[2] Uwe Starossek,"Typology of progressive collapse", Elsevier Engineering Structures 29 (2007) 2302–2307

[3] A. Fatollahzadeh, M. Naghipour, "Analysis of Progressive Collapse in Cable-Stayed Bridges due to Cable Failure during Earthquake", International Journal of Bridge Engineering (IJBE), Vol. 4, No. 2, (2016), pp. 63-72

[4] J.G. Cai., Y.X. Xu, L.P. Zhuang, "Comparison of various procedures for progressive collapse analysis of cable-stayed bridges", Journal of Zhejiang University-Science A, 2012.

[5] Y. Liu, B. Han, Ma Xiao, "Advances in Progressive Collapse of Bridge Structures", Pacific Science Review, 2011.

[6] J Cheng, J Jiang,"Aerostatic Stability of Long Span Cable-Stayed Bridges:Parametric Study", Tsingua Science and technology, ISSN 1007-0214, 16/21 pp201-205 Volume *, Number 2, April 2003.

[7] P. Zhang, B. Chen, "Progressive Collapse Analysis of Reinforced Concrete Frame Structures in Linear Static Analysis Based on GSA", Third International Conference on Intelligent System Design and Engineering Applications, IEEE 2013.

[8] General Service Administration, (2003), "Progressive Collapse Analysis and Design Guidelines".

[9] Wolff, M. and U. Starossek, "Cable-loss analyses and collapse behavior of cable-stayed bridges", in Proceedings of the 5th International Conference on Bridge Maintenance, Safety and Management 2010.

[10] Post Tensioning Institute, (2001). Recommendations for Stay Cable Design, Testing and Installation. Cable- Stayed Bridges Committee, USA.

[11] Zoli, T.P., Steinhouse J., "Some Considerations in the Design of Long Span Bridges against Progressive Collapse." June 2010