STATIC ANALYSIS OF CABLE STAYED STRUCTURAL SYSTEM USED FOR COVERING STADIA

التحليل الاستاتيكي لنظام انشائي ذو كابلات مستخدم كتغطية لاستادات رياضية

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الملخص

إن الهدف الرئيسي لهذا البحث هو دراسة الاستجابة الاستاتيكية لسقف معدني كابولي ذو كابلات يغطى مدرجات استاد رياضي وذلك نتيجة للرياح في حالتها المستقرة باستخدام الكود الامريكي لسنة 2010. وقد تم اختيار أفضل هيئة أو شكل للمنشأ باستخدام برنامج فورتران تم انشاؤه من قبل المؤلف (1) للوصول الى أفضل ترتيب للكابلات والجمالونات الكابولية يولد أقل اجهادات وهبوط للمنشأ، وبعد ذلك تم اجراء تحليل استاتيكي للمنشأ أخذا في الاعتبار بعض المعايير الدراسية منها مكان تثبيت الكابل السفلي، وانحدار السقف، وتغيير عرض وطول الباكيات، والشد الابتدائي في الكابلات. وقد تم اجراء التحليلات الانشائية باستخدام برنامج فورتران تم انشاؤه من قبل المؤلف (2) مبني على أساس تصغير طاقة الوضع باستخدام طريقة الانحدارات المتبادلة وتم مراجعته باستخدام برنامج ساب2000.

Abstract

The aim of this work is to study the static response for steel cantilever roof stayed with cables used as a stadium roof due to wind steady state using ASCE 7-10 code. The best configuration for the model is generated using a FORTRAN program constructed by the author [1] to reach for the best arrangement of cables and cantilever space trusses that gives the lowest deflection and stresses in the structure. Then, the static analysis for the structure is carried out taking into account some study parameters depending upon where to attach the lower cable, the inclination of the roof, changing the panels' length and height and the initial tension in cable elements. The analysis is carried out using a FORTRAN program constructed by the author [2] based on the minimization of the total potential energy by the conjugate gradient technique and checked using SAP2000 program.

Keywords

Cable stayed Roofs, Static Response, Best Configuration, Initial tension, Wind Steady State
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1. Introduction

The most advanced investigations in structural engineering have mainly been carried out in the field of cable structures. Cable stayed bridges, cable roofs and guyed towers have a wide field of applications [2]. The static response for cantilever steel roofs suspended with cables and used as stadia roofs is investigated in this research. This type of structures is more flexible than most other forms of roof constructions. Also, the nonlinearity of the structure will add to the complexity of the analysis [3].

A FORTRAN program is created by the author [1] to generate a model for the roof having variable dimensions to cover a stand (30x31.25x17m). The best configuration for the structure is achieved by changing the arrangement of the cantilever trusses and the cables and studying its effect on the deflection, the maximum stresses and the total weight of the structure using SAP2000 program. Then, a static analysis is carried out on the structure taking effect of some study parameters and the results are checked using a FORTRAN program constructed by the author [2].

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2. The Best Configuration

The effect of changing the arrangement of the cantilever trusses and cables on the deflection, the maximum stresses and the total weight of the structure is studied in order to reach for the best configuration of the structure.

Four different study cases of the geometry arrangement of the structure are carried out. The front and side views of the structural system are shown in Figs. (1) to (8). Both of ITO and NCU are changed from 5 to 7 with the following constant parameters:

$a = 1.25m$, $ILC = 3\ (z' = 10.\ m)$
$h = 0.5m$, $\theta 1 = \theta 2 = 0°$

The geometry of the roof and the stand in the 3-Directions is shown in Fig. (9).

2.1 Properties of sections

Tower Properties: The vertical members in the tower are taken as B.F.I (340) with modulus of elasticity, $E = 2100t/cm^2$ and allowable stress, $F_{all} = 2.1t/cm^2$

Roof Properties: *All members in the structure, except for the vertical members in the tower, are taken as circular sections with an outer diameter,$D\ of\ 80mm$, modulus of elasticity, $E = 2100t/cm^2$ and allowable stress $F_{all} = 2.1t/cm^2$*

Cable Properties: All cables in the structure are spiral cables with an outer diameter$D = 116mm$, modulus of elasticity$E = 1472t/cm^2$, steel area $A_s = 0.007862m^2$, elastic weight $W = 0.066t/m$ and minimum breaking load $BF = 1048.7t$.

The initial tension is assumed after many tried cycle of solutions as 8% of BF to satisfy the following:

- To avoid compression of any cable element during any state of loading and deformation.
- To maintain the required shape during erection [4].

2.2 Loading

The following assumptions are taken into consideration to reach for the best configuration for the structure and to carry out the static analysis.

Dead Load: For cables (a self-weight for all cable groups 0.066 t/m). For Roof (a self-weight of 0.039 t/m is considered and a uniform distributed cladding 0.015 t/m^2 is taken for the lower members that carry the cladding). For Tower (a self-weight for the vertical members 0.1295 t/m is considered).

Live Load: For lower members a uniform distributed live load of 0.055t/m^2 is considered.

Wind Load: *For wind steady state,* the open structure wind loading pattern (exposure from frame object) using the code for the American Society of Civil Engineers (ASCE 7-10) [5] in SAP2000 with the properties shown below is considered.

Wind properties*:*

Basic wind speed *U(33ft)* = 18.8 m/s
Exposure type = B (For urban and suburban)
Topographical factor = 1.0 (For flat land)
Gust factor = 0.85 (For Rigid Structure)
Direction factor = 0.85 (Trussed members)
Solid/Gross area ratio = 0.7
Wind direction = 0^0 (X-Axis)

With reference to Figs. (10 to 14), case 3 had the lowest value of the vertical deflection of the roof, the horizontal sway of the tower, the normal stresses on all members and the final tension in all cables. On the other hand, the total weight of the structure in case 3 is larger than the total weight in case 1 in which *(NCU = 5)* and *(ITO = 5).* Since increasing the initial tension always decreases the deflection and increases the stresses slightly without increasing the total weight, it's recommended to choose case 1 as the best configuration.

3 Static Analysis

The static analysis for case 1 is carried out taking the following effects into consideration as study parameters [6] as shown in Fig. (15).

1) Attaching the lower cable to a point in the lower part of the tower ($ILC = 3$) at *(z' = 10.0 m)* or to the tower support ($ILC = 2$) or making the lower cable vertical ($ILC = 1$).

2) The inclined angle $\theta 1$ and $\theta 2$ of the left and right part of truss to horizontal respectively.

3) The initial tension in cable elements.

4) Removing the members inclined in 3-D direction.

5) Changing the panel length and height.

There are many investigation factors that affect the response of cable structures in static analysis. Each factor is studied considering all other parameters are kept constant. All study parameters are studied for case 1 with ($ITO = 5$) and ($NCU = 5$).

3.1 Location of attachment of the lower cable

Attaching the lower cable to a point in the lower part of the tower at ($z' = 10.0\ m$) ($ILC = 3$), to the tower support ($ILC = 2$), or making the lower cable vertical ($ILC = 1$) affects the deformations and the internal forces in the main elements of the structure. The obtained results shown in Figs. (16) to (21) illustrate that making the lower cable vertical ($ILC = 1$) gives the lowest horizontal sway for tower, the lowest vertical deflection for the roof, the lowest normal stresses on members and the lowest final tension in cables with a very small change in the total weight, *so making the lower cable vertical is more efficient than attaching it to the tower.* From these results, the lower cable is guyed vertical to the ground for the residual studies.

3.2 The inclined angle of the roof

The inclined angles, $\theta 1$ and $\theta 2$ of the left and right part of the truss respectively, to the horizontal plane have a vital architectural effect, so the structural effect of changing them from (0^0 to 20^0) with step 5^0 as shown in Fig. (22) is studied taking the initial tension in cables as 8% of the breaking force. The results shown in Figs. (23) to (26) illustrate that increasing the inclined angles affects the structure slightly. Therefore, it is possible to change the inclined angles of the roof for any architectural purpose.

3.3 The initial tension in cable elements

The effect of changing the initial tension in cables from 2% to 10% with step 2% is studied. [7] The obtained results are shown in Figs. (27) to (30). It's noticed that increasing the initial tension in cables increases the maximum negative stresses in members and the final tension in cables. On the other hand, the horizontal sway of tower and the vertical deflection of roof decrease with increasing the initial tension. So, it is recommended to choose 6% as the most suitable value for the initial tension.

3.4 Removing the members inclined in 3-D direction

The effect of removing the members inclined in 3-Directions shown in Fig.(31) is studied and the results are shown in Figs. (32) to (34). The effect on the total weight can be as follows:

Total weight for (θ1=θ2=0) with 3-D members = 541.9 ton

Total weight for (θ1=θ2=0) without 3-D members = 504.5 ton

Difference = 541.9 – 504.5 = 37.4 ton (6.9%)

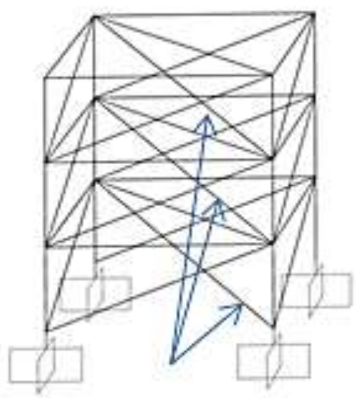

Fig. (31) The inclined members in 3-D

The obtained results illustrate that removing these members decreases the stresses in members, the deflection of tower and roof and the total weight of the structure. Therefore, it's recommended to remove these members.

3.5 Changing the panel length (a) and height (h)

The effect of changing the panel length and height is studied by taking a (0.625, 1.25) and h (0.5, 1.0, 1.25). These values for a, h were chosen carefully to conserve the dimensions for the structure ($38.75m$ x $31.25m$ x $25m$) constant. The obtained results shown in Figs. (35) to (37) illustrate that the best panel length and height are $a = 1.25m$ and $h = 0.5m$ as chosen before. That's because decreasing the deflection is the main aim of the analysis as long as the normal stresses are safe.

4 Conclusion

Based on the results and analyses of this study, the following conclusions are drawn:

1) The best configuration for a steel cantilever roof stayed with cables used to cover the stand is represented by a hall having main trusses at 7.5m spacing in the lateral direction and cables at 7.5m spacing in the longitudinal direction.

2) Making the lower cable in the model vertical gives the lowest horizontal sway for tower, the lowest vertical deflection for the roof, the lowest normal stresses on members and the lowest final tension in cables with a very small change in the total weight, so making the lower cable vertical is more efficient than attaching it to the tower.

3) Increasing the inclined angles of the left and right part of the roof from (0^0 to 20^0) affects the structure slightly. Therefore it is possible to change the inclined angles of the roof for any architectural purpose.

4) Increasing the initial tension in cables from 2% to 10% increases the maximum negative stresses in members and the final tension in cables as well. On the other hand, the horizontal sway of tower and the vertical deflection of roof decrease with increasing the initial tension, so it is recommended to choose 6% as the most suitable value for the initial tension.

5) Without the inclined members in 3-Directions decreases the stresses in members, the deflection of tower and roof and the total weight of the structure. Therefore, it's recommended to remove these members from the whole structure or from the tower only.

6) The optimum values of panel length and height are 1.25m and 0.5m respectively.

5 List of Symbols

a	The panel width
A_s	The steel area
BF	The breaking force of cables
D	Diameter of circular sections
E	Modulus of elasticity
F_{all}	The allowable stress
h	The panel height
ILC	*=1* The lower cable is vertical *=2* The lower cable is attached to the tower support *=3* The lower cable is attached to a point in the lower part of the tower at a height z' from the base
ITO	The number of cantilever trusses arranged parallel to each other in Y-Direction and connected with a side truss at cables' positions
NCU	The number of upper cables
$\theta 1$	The inclined angle of the left part of truss to horizontal
$\theta 2$	The inclined angle of the right part of truss to horizontal
W	The elastic weight per m'

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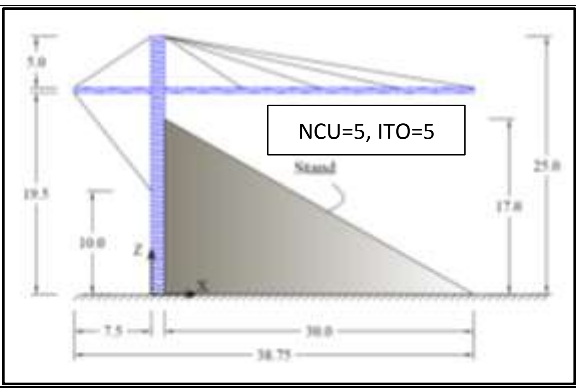


Fig. (1) Front view of the roof for Case 1

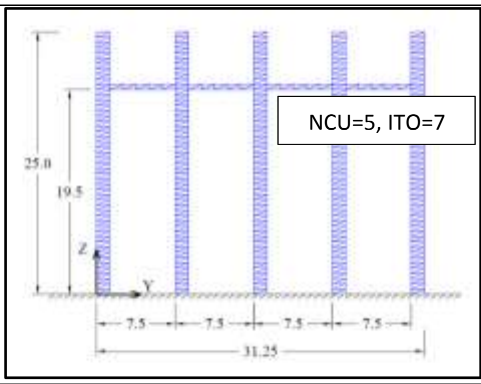


Fig. (2) Side view of the roof for Case 1

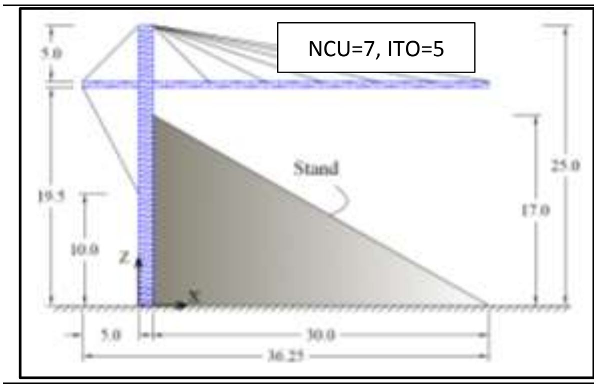


Fig. (3) Front view of the roof for Case 2

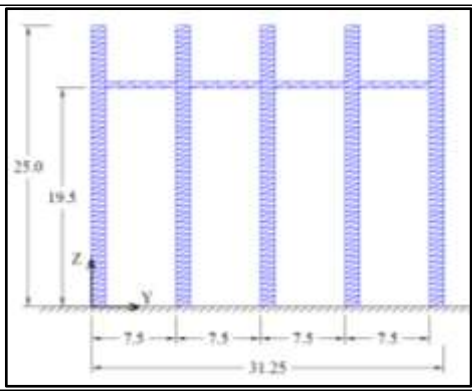


Fig. (4) Side view of the roof for Case 2

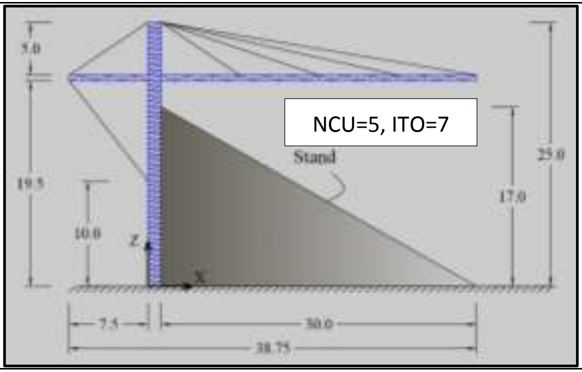


Fig. (5) Front view of the roof for Case 3

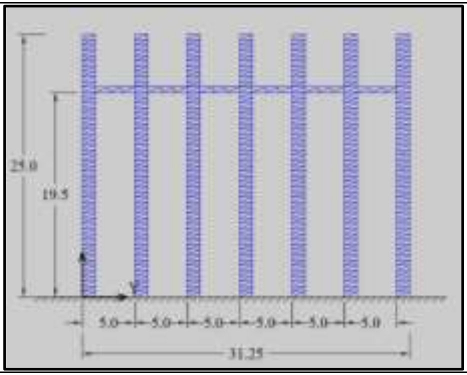


Fig. (6) Side view of the roof for Case 3

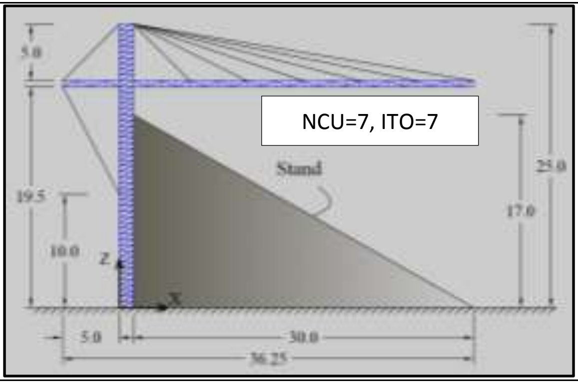


Fig. (7) Front view of the roof for Case 4

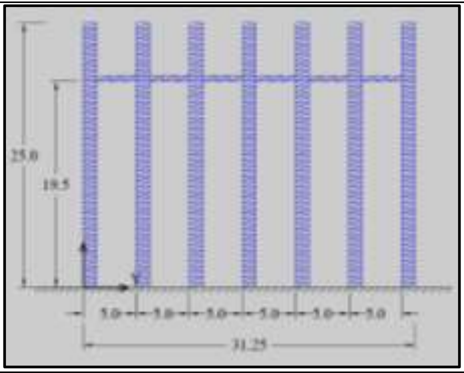


Fig. (8) Side view of the roof for Case 4

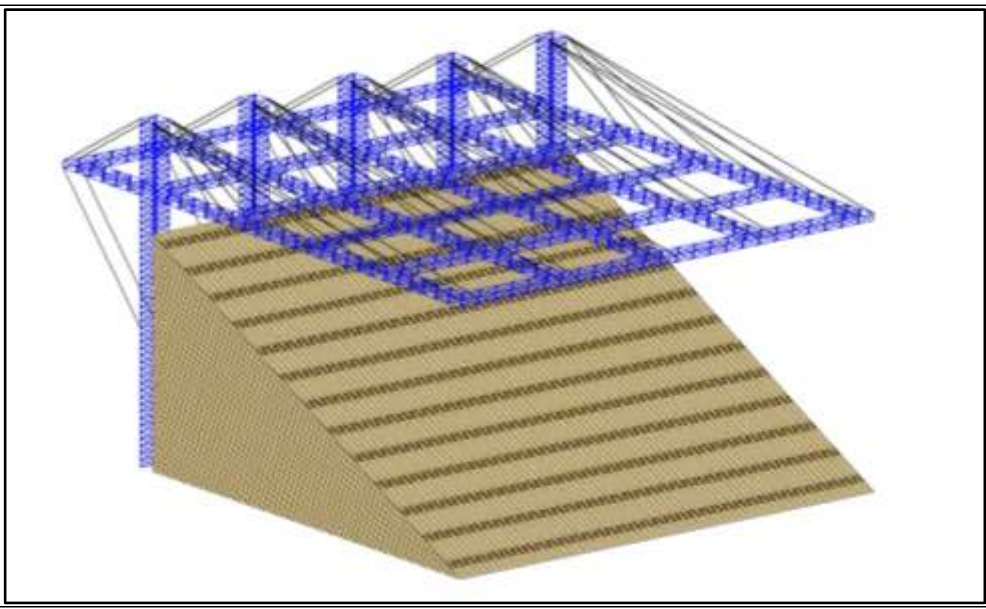


Fig.(9) 3-D view for the stand and the roof for Case 1

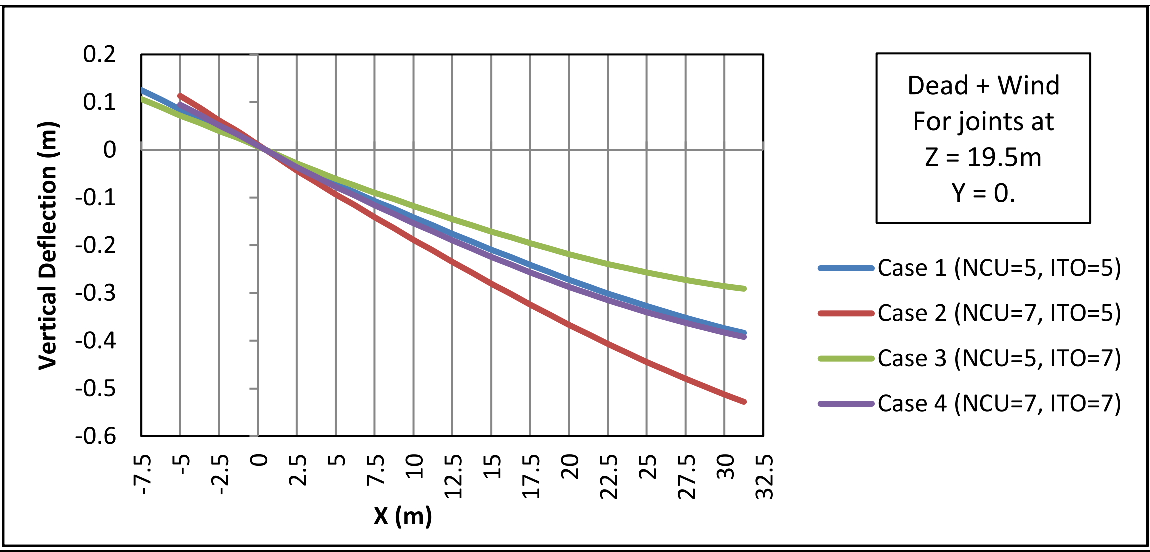


Fig. (10) The vertical deflection for joints at (Z=19.5m, Y=0.) in case 1 to 4

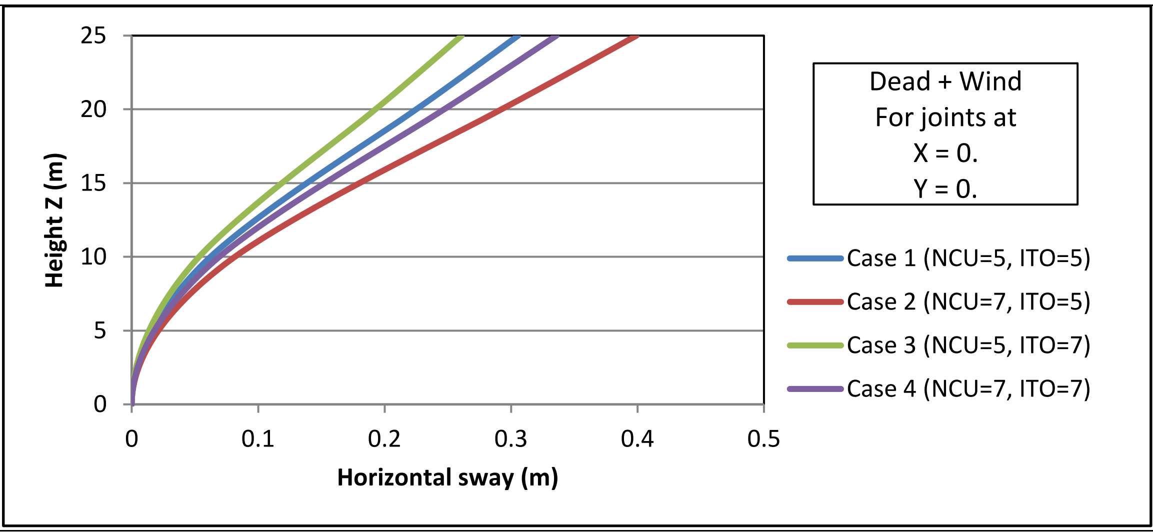


Fig. (11) The horizontal sway for joints at (X=0., Y=0.) in case 1 to 4

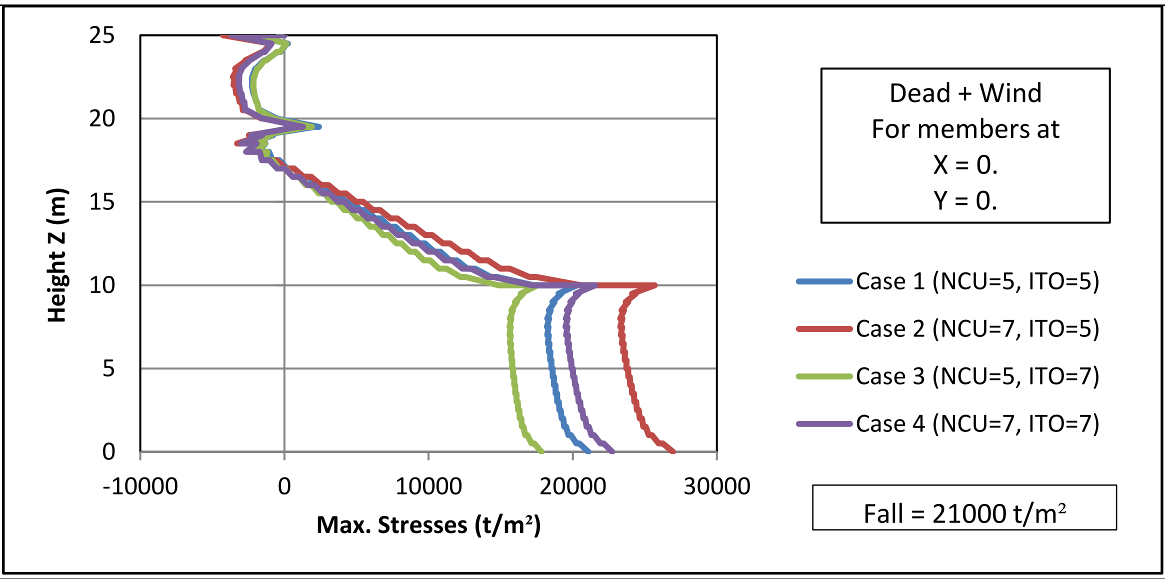


Fig. (12) The normal stresses for vertical members at (X=0., Y=0.) in case 1 to 4

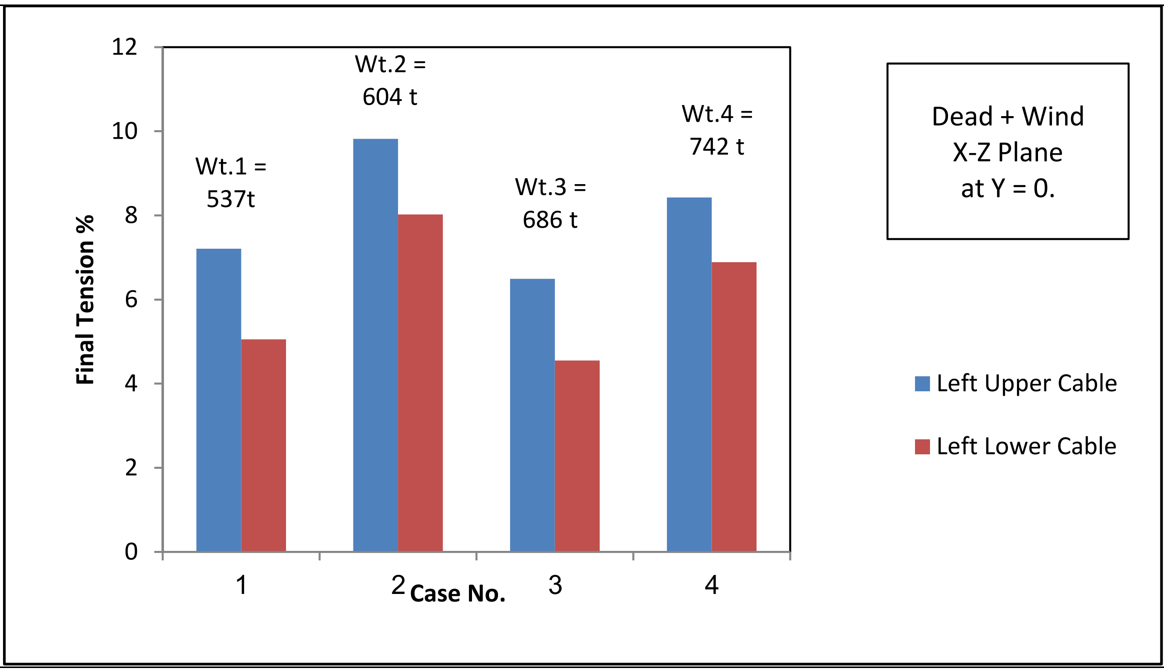


Fig.(13) The final tension for some cables in X-Z plane and the total weight in case 1 to 4

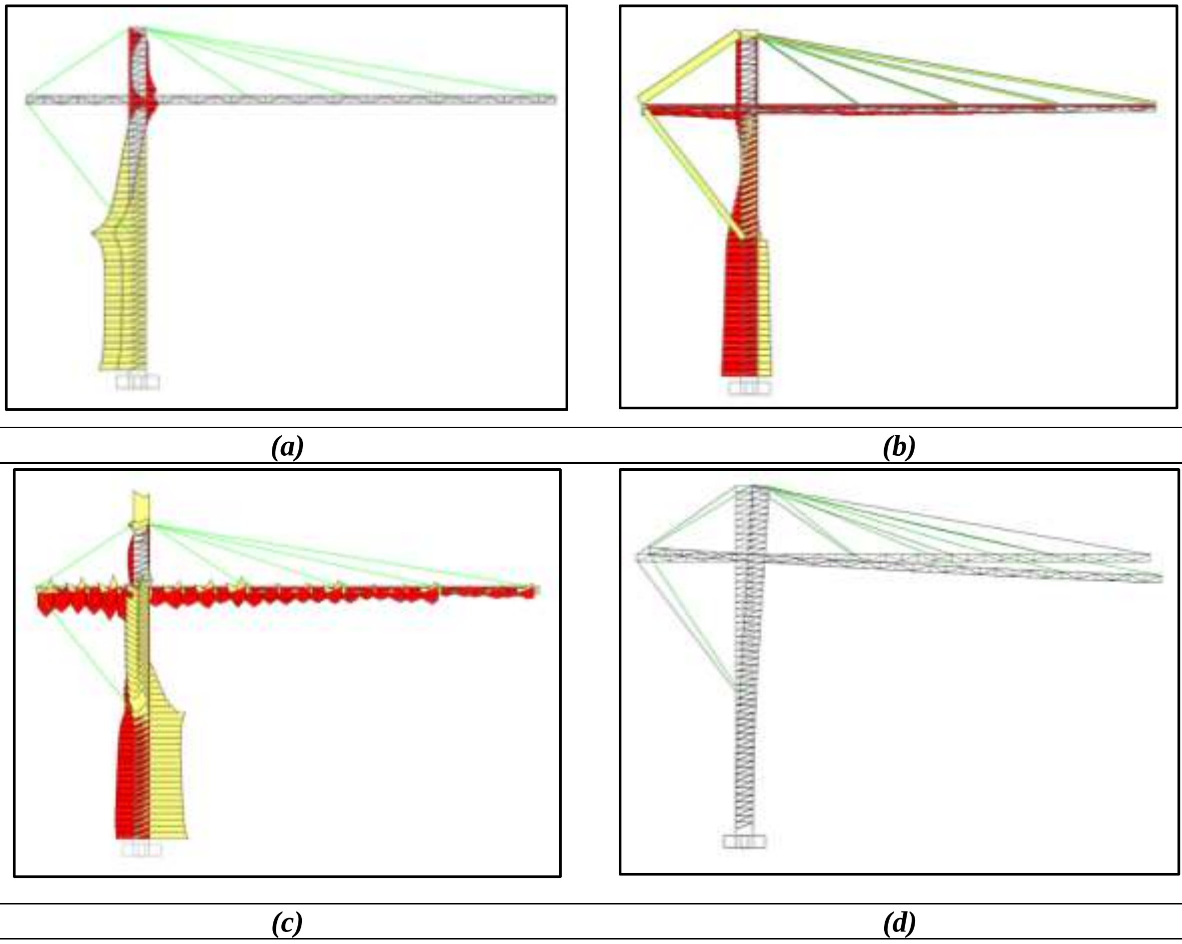


Fig. (14) Result from case 1 for X-Z plane at Y= 0 (a) Bending moment (b) Normal force (c) Max Min stresses (d) Deflection

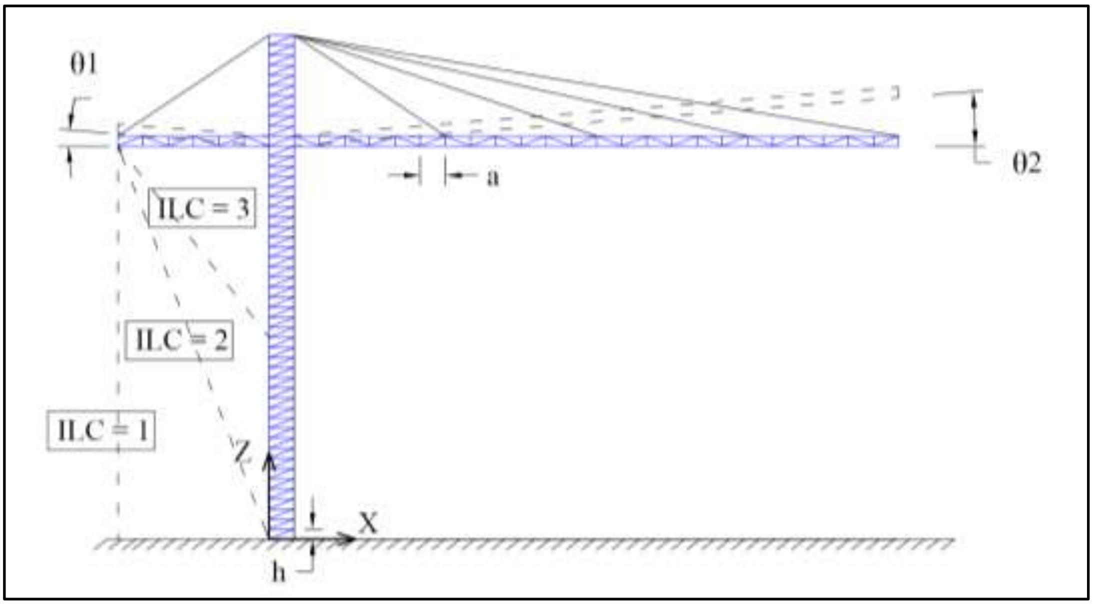


Fig.(15) Study parameters used in the study

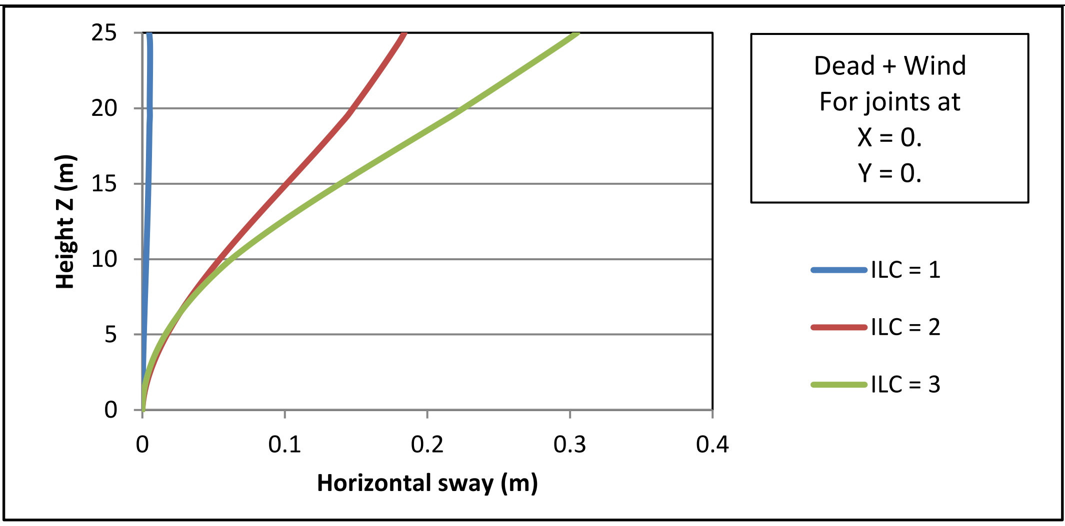


Fig. (16) The horizontal sway for joints at (X=0, Y=0.)

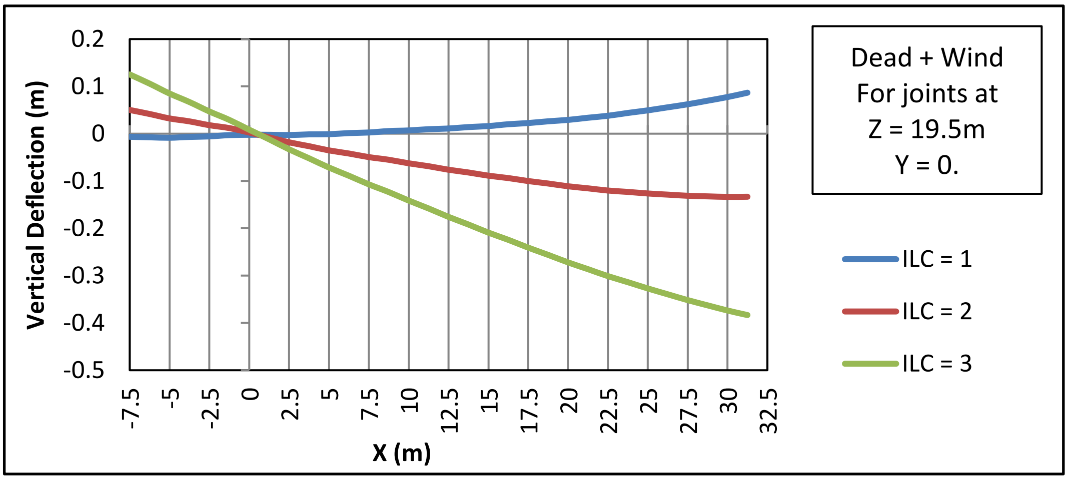


Fig. (17) The vertical deflection for joints at (Z=19.5m, Y=0.)

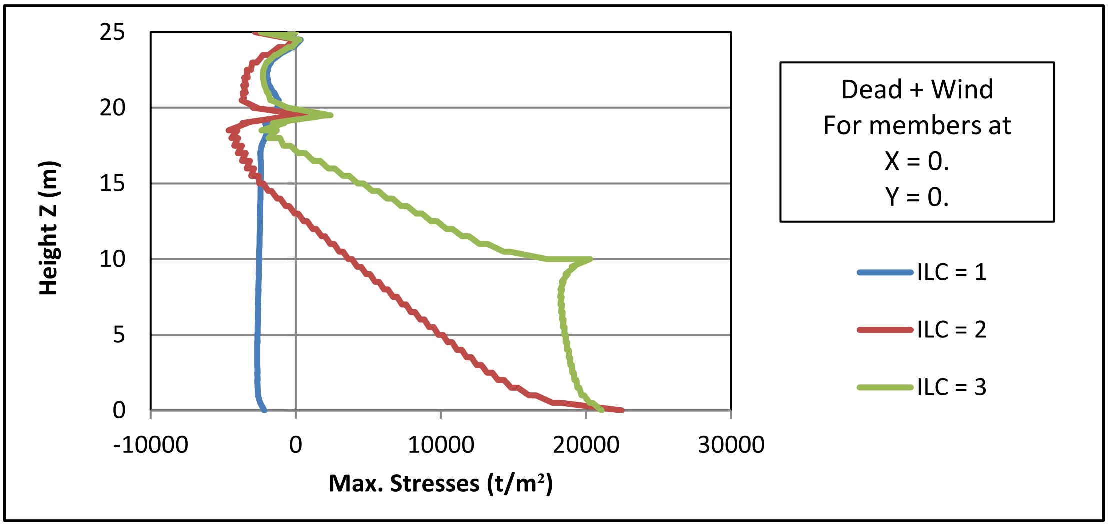


Fig. (18) The normal stresses for vertical members at (X=0., Y=0.)

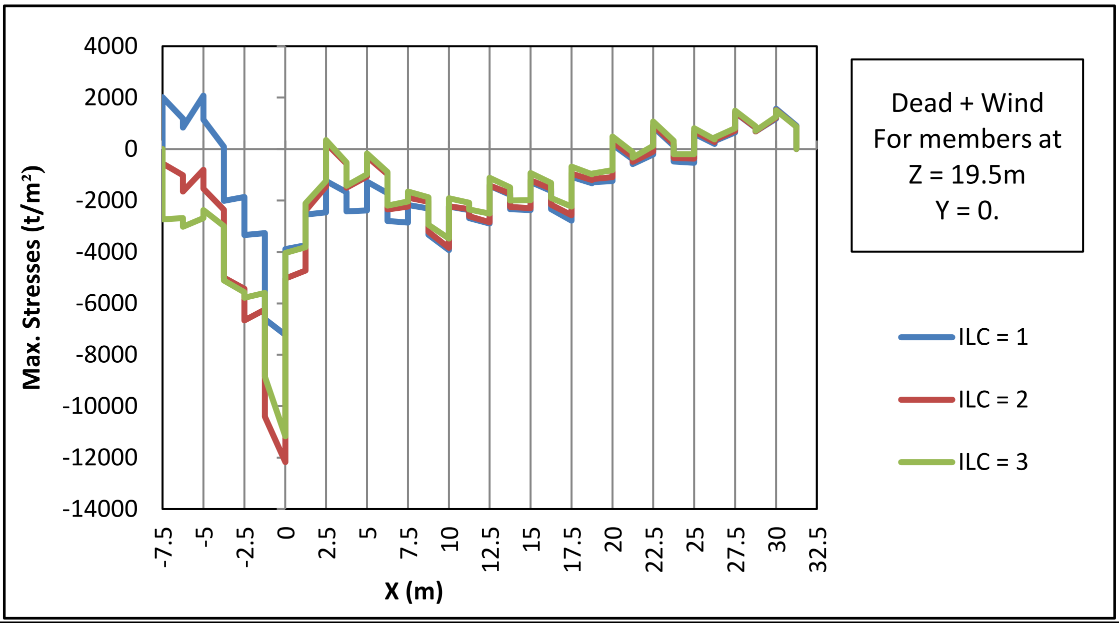


Fig. (19) The normal stresses for horizontal members at (Z=19.5m, Y=0.)

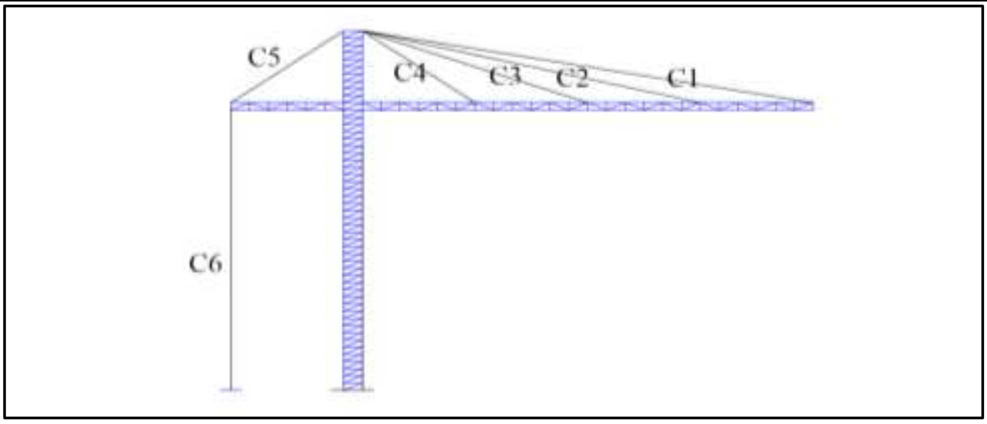


Fig.(20) Numbering of cable members in X-Z plane at Y=0.

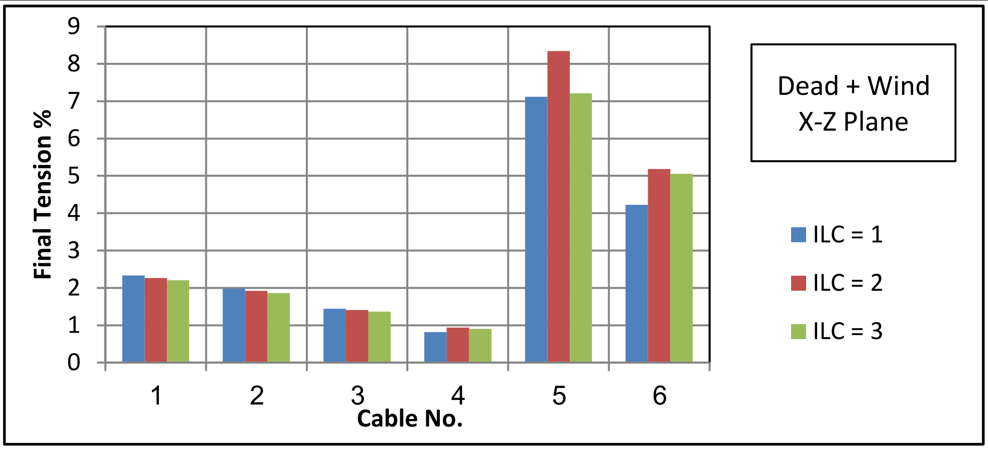
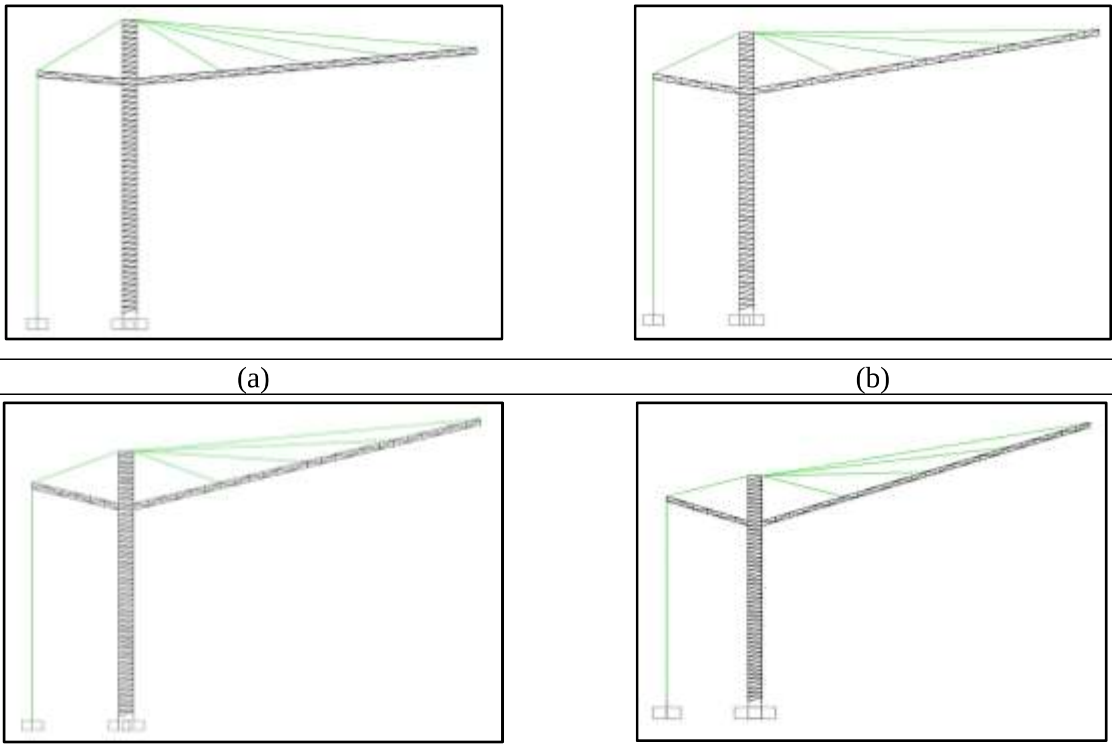


Fig. (21) The final tension for cables in X-Z plane at Y=0.



(a) (b)

(c) (d)

Fig.(22) The inclined angle of the roof (a) θ1=θ2=5 (b) θ1=θ2=10 (c) θ1=θ2=15 (d) θ1=¥2=20

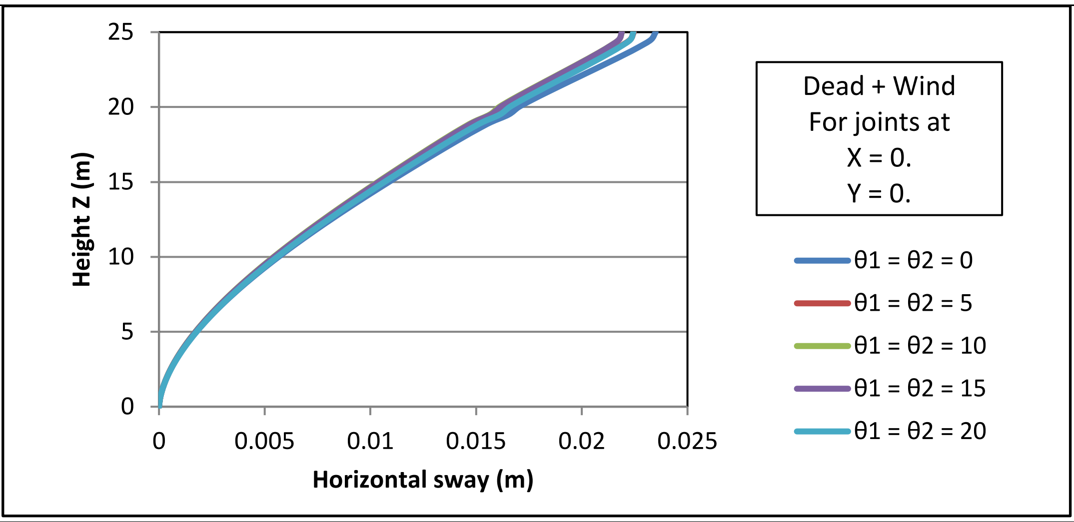


Fig. (23) The horizontal sway for joints at (X=0., Y=0.)

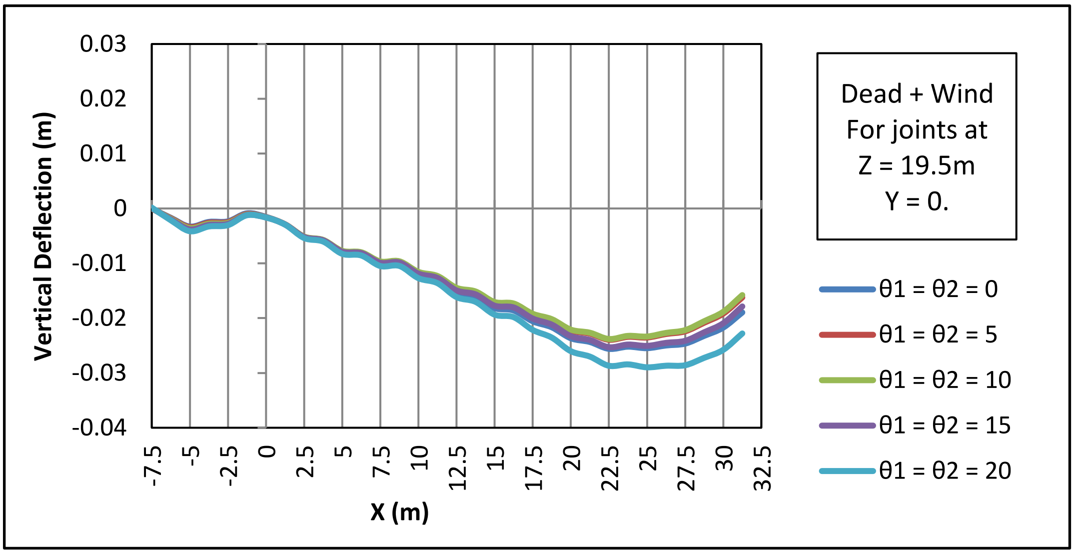


Fig. (24) The vertical deflection for joints at (Z=19.5m, Y=0.)

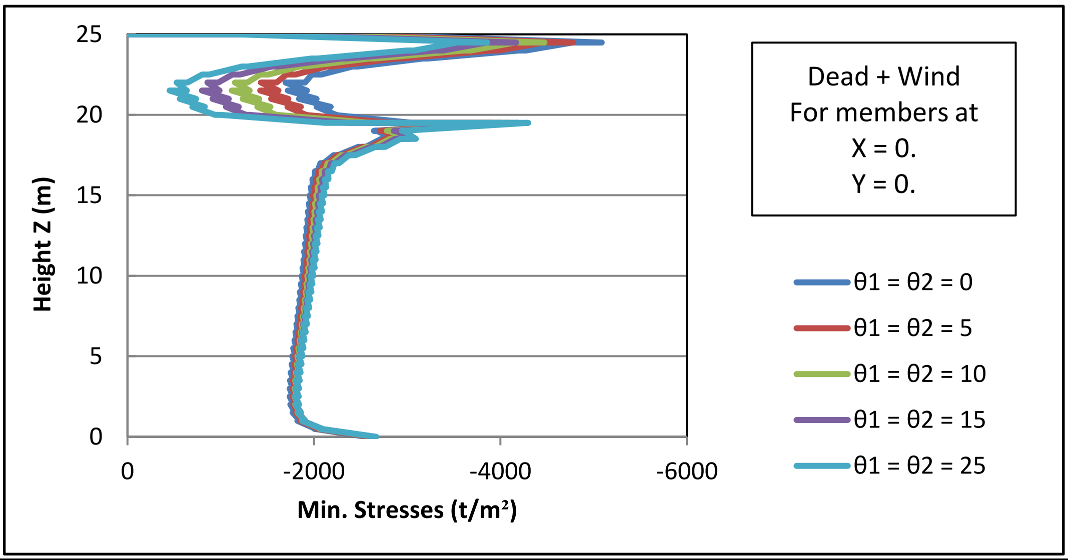


Fig. (25) The normal stresses for vertical members at (X=0., Y=0.)

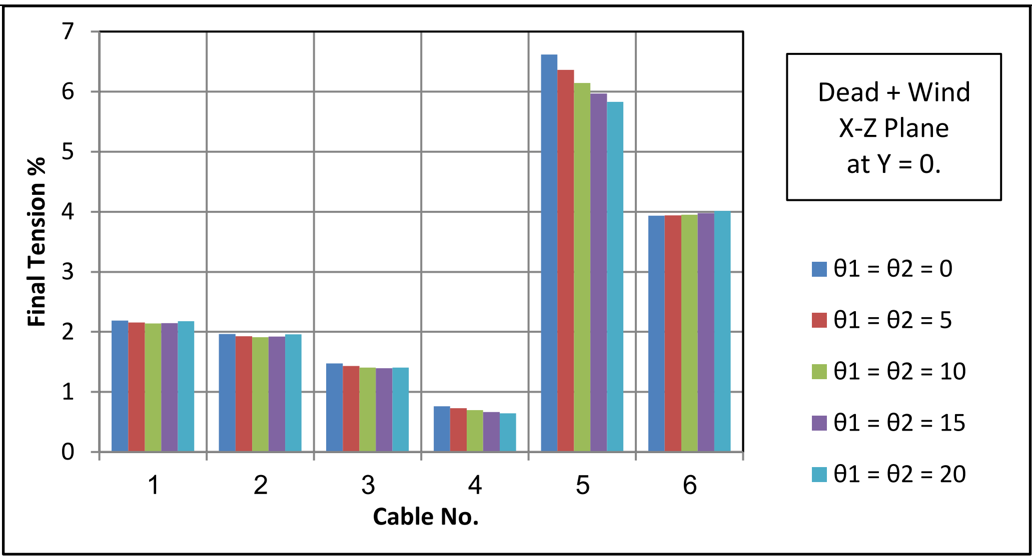


Fig. (26) The final tension for cables in X-Z plane at Y=0.

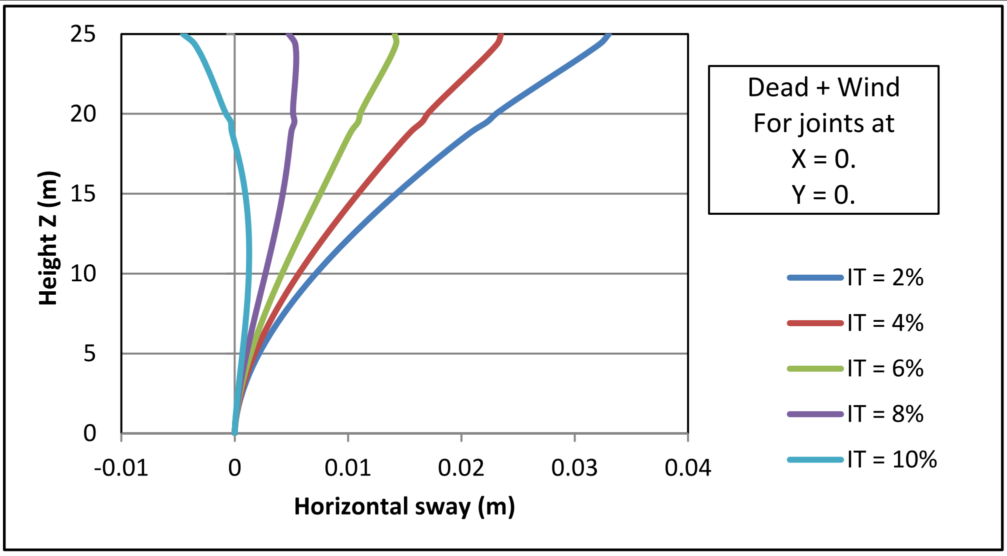


Fig. (27) The horizontal sway for joints at (X=0., Y=0.)

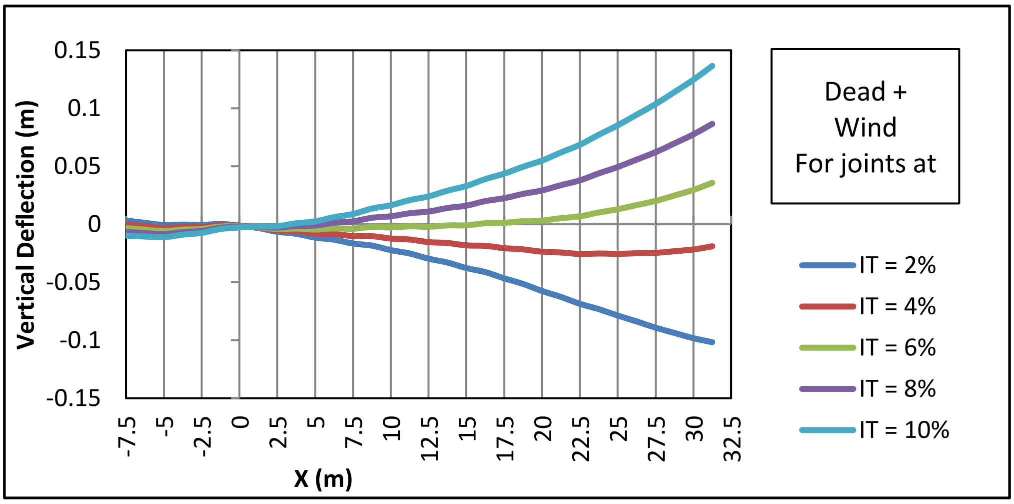


Fig. (28) The vertical deflection for joints at (Z=19.5m, Y=0.)

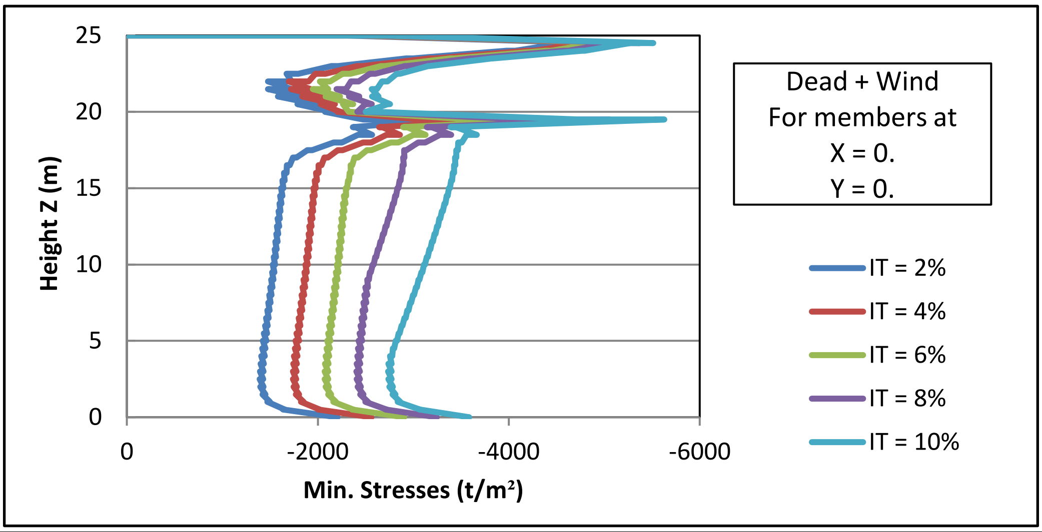


Fig. (29) The normal stresses for vertical members at (X=0. Y=0.)

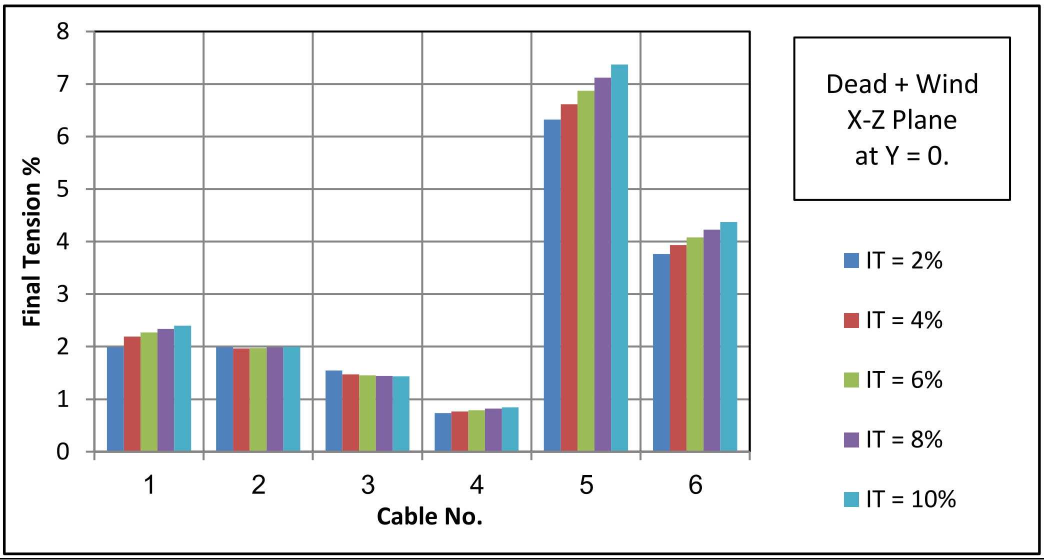


Fig. (30) The final tension for cables in X-Z plane at Y=0.

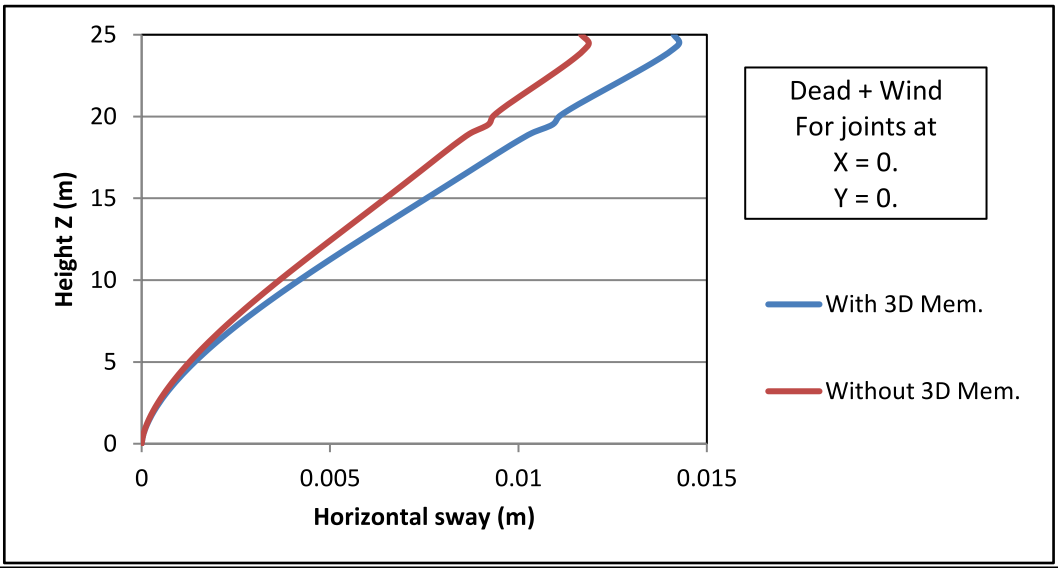


Fig. (32) The horizontal sway for joints at (X=0, Y=0.)

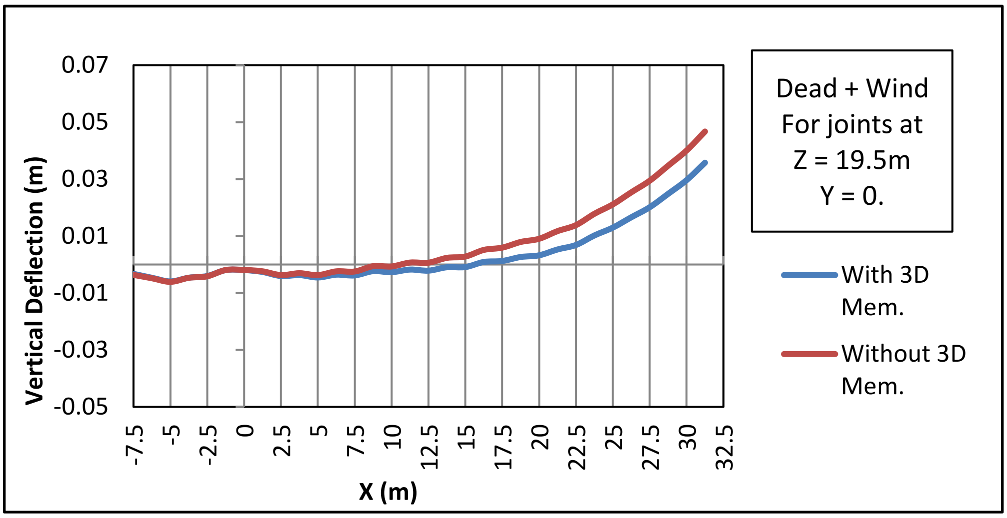


Fig. (33) The vertical deflection for joints at (Z=19.5m, Y=0.)

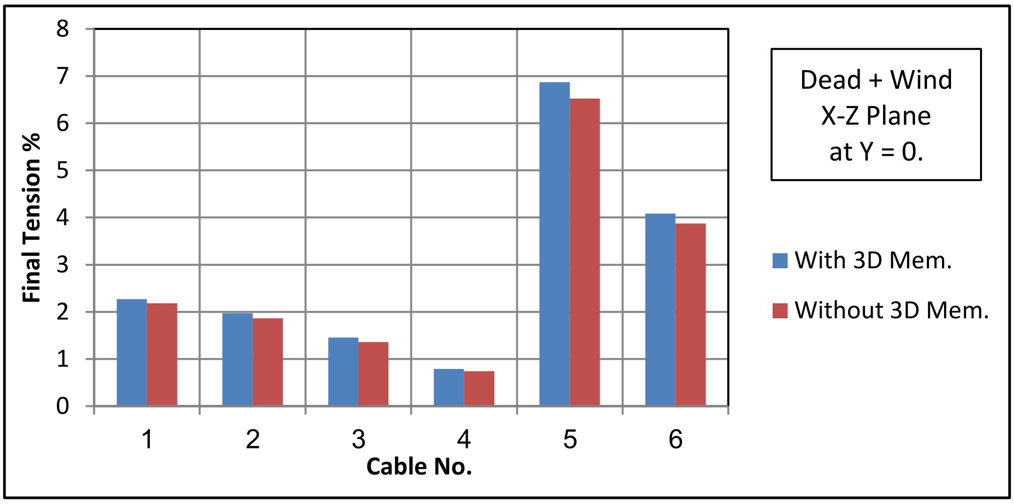


Fig.(34) The final tension for cables in X-Z plane at Y=

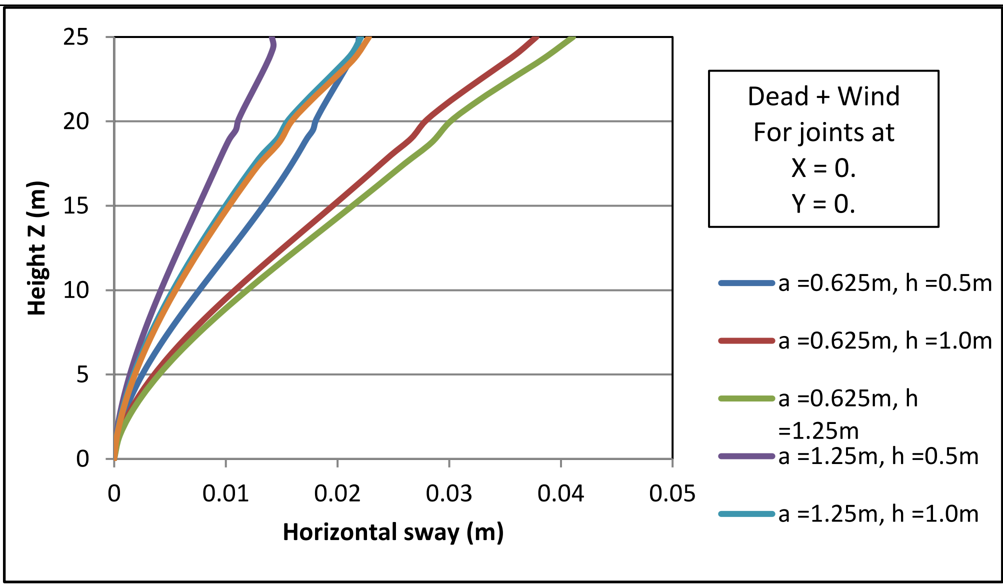


Fig. (35) The horizontal sway for joints at (X=0. Y=0.)

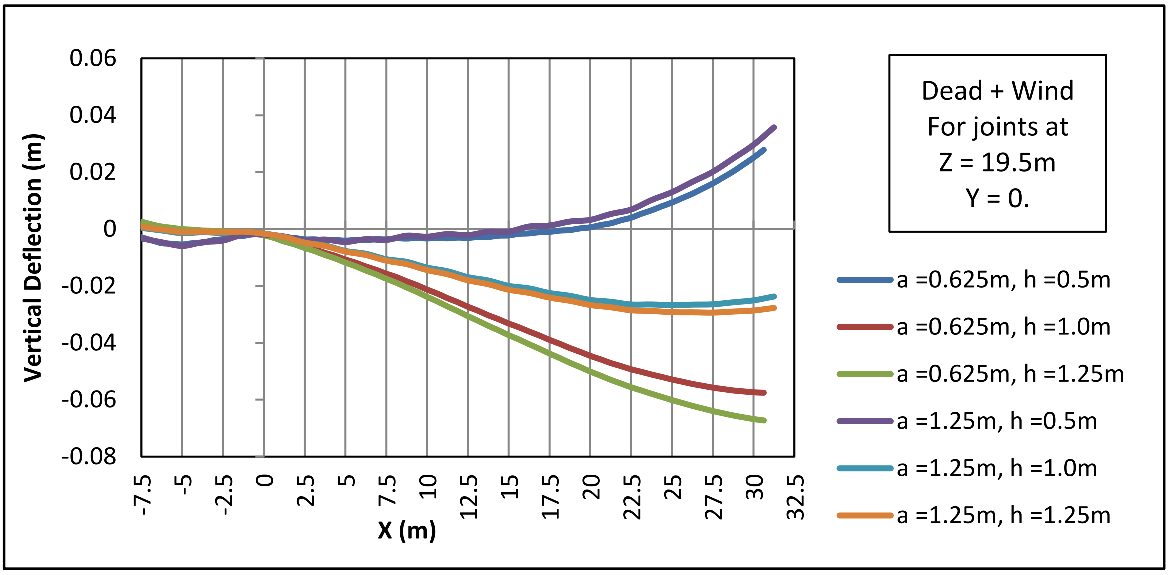


Fig. (36) The vertical deflection for joints at (Z=19.5m, Y=0.)

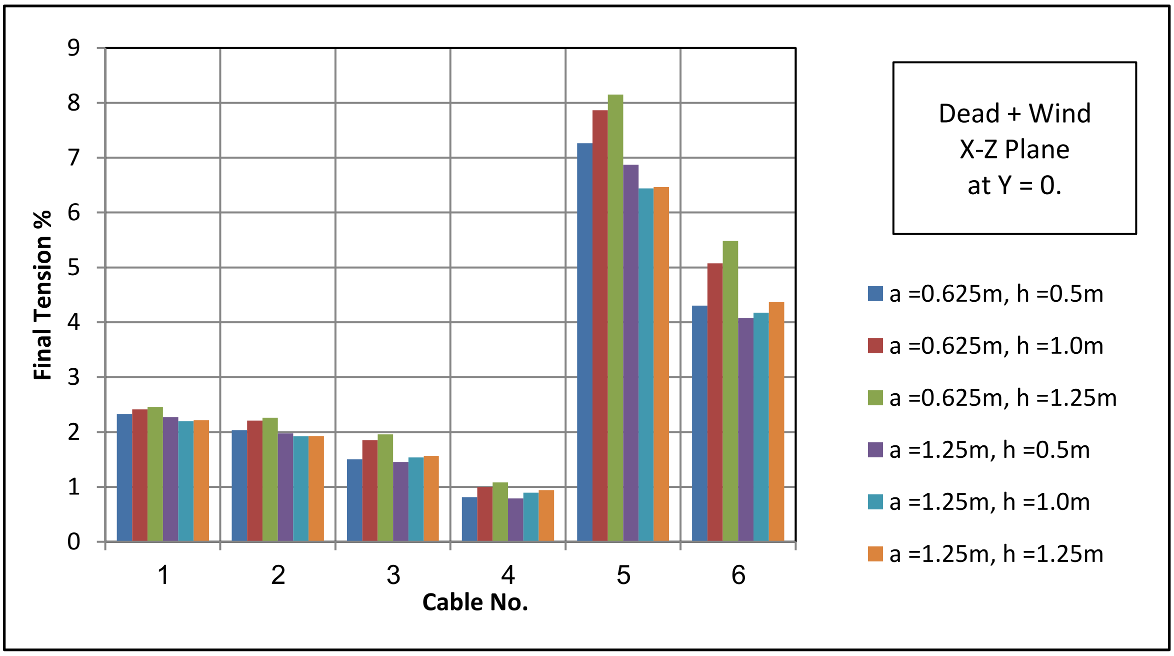


Fig. (37) The final tension for cables in X-Z plane at Y=0.