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Capacity Of Transmission Tower Under Wind Loading Using Nonlinear Pushover and Dynamic Analysis

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الملخص العربي

انهيار ابراج نقل الطاقة الكهربائية نتيجة حدوث عواصف عاليه السرعة. و نظرا للتأثير السلبي لانهيار شبكة الكهرباء على الاقتصاد فانه من الضروري دراسة أسباب الانهيار و محاولة تقليل أثر سرعة الرياح العالية و محاولة إستيعاب تأثيرها على هذه الأبراج من خلال مراجعة الأحمال التصميمية الحالية.

كما تم دراسة إمكانية تطبيق طرق التحليل اللدن على ابرج الكهرباء كمنشأ فراغى مع دراسة تتابع تكوين المفصلات اللدنة للوصول إلى أقصى قيمة حمل يمكن للأبراج أن تتحملها اعتمادا على قطاعاتها و شكلها الفراغي . أيضا سيتم دراسة التأثير الديناميكي الحقيقي للرياح على الأبراج.

و سوف يتم حساب أقصي سعه يتحملها البرج من خلال الطريقتين و مقارنه النتائج من خلال استخدام برامج العناصر المحددة المتاحة لدراسة السلوك اللدن للأبراج و كذلك رد فعل هذه الأبراج تحت تأثير أحمال الرياح الحقيقية و هذه الطرق توفر تكاليف كثيرة للتغاضي عن التجارب العملية للأبراج .

Abstract

Failures of transmission towers are common. Predicting failure loads criteria under wind loads for transmission towers are usually done using the results from full-scale tower tests. However, the test results are only valid for a specific tower and loading conditions where static loads are normally applied.

These tests may predict approximately how the tower may behave under different static and dynamic loading conditions. As an alternative, nonlinear analysis techniques such as nonlinear static pushover analysis (NSP) Ref. [1] and incremental dynamic analysis(IDA) Ref. [2],

In this paper, the NSP and IDA procedures are used for assessing the capacity of a two example transmission tower under wind loading. A brief description of the methods along with the adopted procedure for wind loading are presented first. This is followed by the description of the tower 1 and tower 2 and calculation of the wind load on that tower.

Keywords - Transmission Tower (T.T) Failure, High Intensity Wind (H.I.W), nonlinear static Pushover (NSP)., dynamic analysis(IDA)

1-INTRODUCTION

In this paper the performance of 220kV tower and 500 KV tower with high wind intensity is observed using NSP and IDA methods. Analyses is carried out for the tower and the performance of the towers are evaluated. the modeling and analysis are discussed. The wind intensity converted into point loads and loads are applied at panel joints. Then get the comparison between base shear of capacity curve for two analysis.

2-NONLINEAR PUSHOVER ANALYSIS (NSP)

It is an approximate method based on subjecting the tower to a monotonically increasing pattern of lateral loads. This load pattern is distributed along the height of the building. The load is increased until the building achieves a pre-defined target displacement or becomes unstable. It is basically consisting of a series of sequential elastic analysis.

At the end of each step force-displacement relationship is recorded until a curve for the overall tower is achieved. At first a traditional model of the structure is built. Then the gravity loads are applied. The wind loads are then applied monotonically. The forces will be increased until achieving first yield. Then model stiffness is then updated considering the decreased value of stiffness.

Lateral forces are being increased step by step and stiffness is being reduced step by step. These steps continue until the structure achieves the predefined target displacement or becomes unstable. This method gives a force-displacement curve relationship called capacity curve which is encountered by drawing the base shear of the structure against the displacement at the tip of tower . Pushover analysis is divided into two main procedures, force-controlled and displacement-controlled. Force-controlled is a method where the applied load value is known before analysis like gravity loads.

Displacement controlled procedure is a method used when the value of the load is not known however the drifts are assessed before using any mean. The load value is being increased until the structure achieves a tower displacement predefined target or becomes unstable..

Pushover has advantages over elastic analyses procedures. However the estimation of target displacement and the definition of used lateral loads pattern are factors strongly affect the results.

FEMA-273 [3] is recommended to enhance the accuracy of pushover analysis, utilizing two types of lateral loads from the fundamental lateral loads patterns related to inertia distribution.

3-INCREMENTAL DYNAMIC ANALYSIS (IDA)

The incremental dynamic analysis (IDA) may be employed to characterize the capacity of the transmission tower. The IDA is a powerful tool to assess the global behavior of a tower from its elastic response to global dynamic instability through yielding and nonlinear response.

The IDA procedure requires a series of linear and nonlinear dynamic analyses to be carried out for a few selected strong ground motion records that are scaled using an intensity measure. The results obtained are then employed to characterize the capacity curve in terms of an intensity measure or base shear versus lateral displacement or drift ratio.

Note that although the IDA is only used for structures under earthquake loading in the literature, it could be adopted for evaluating the structural capacity under wind loading too. In order to apply the IDA to structures under wind loading, we note that unlike the seismic loading which consists of zero mean stochastic excitations, the along wind velocity causes the drag force that can be represented as a non-zero mean loading due to the mean wind speed and a fluctuating stochastic excitations.

4-LITERATURE REVIEW

Sudhan Banik, Hanping Hong, and Gregory A. Kopp [4] studied assessment of structural capacity of an overhead power Transmission Towers (T.T) under Wind Loading using (NSP) and incremental dynamic analysis (IDA). Two nonlinear hinges assigned at the ends of each member of the modeled tower to confine the interaction of axial and flexural stresses. Wind loads from longitudinal and transverse directions considered in the analysis. It observed that the capacity curves obtained using (NSP) and (IDA) procedures showed a bilinear load-deformation relationship.

Strength Assessment of Both telecommunication tower and T.T Steel Towers was studied by (Baskaran, et al) [5] to identify the reasons for failure of towers and proposing methods to evaluate the tower strength capacities. (Shakeel Ahmad et al) [6] performed response of Transmission Tower subjected to tornado loads. A 35m high transmission tower under tornado loads analyzed and the results showed that the response of the T.T was enormously high due to the tornado wind loads. The study of dynamic response indicates that the section of tower at 27 m height affected if the tower meets the resonance conditions in modes greater than sixth mode.

Wei Zhang1*, et. al. [7] studied the Probabilistic capacity assessment of lattice transmission towers under strong wind. The lattice transmission towers built with L-shape steel members and truss, beam elements or their combinations used for modeling the structure. The material and geometric non-linearity were included by using bilinear elasto-plastic material properties and by implementing large deformation analysis, respectively. To efficiently the proposed probabilistic capacity assessment methodology, the finite element model for a 550- kV-68.6 m height of single circuit transmission tower built primary for the capacity analysis of the structure. The modal analysis performed to find the mode shapes and mode frequencies. The study had demonstrated an effective probabilistic capacity assessment approach for transmission towers considering stochastic wind loadings.

Aboshosha, H. and El Domatty, A. [8] investigate the progressive failure of two types of transmission lines, namely self-supported and guyed towers under the effect of downbursts. The downburst field based on previously computational fluid dynamics analysis. The outcomes of the study are the determination of the different modes of failure of the two systems of towers affected by the downburst loading.

N.Prasd Rao et al. [9] investigate the capacity of five previously tested transmission towers with range from 220KV to 400KV. They drown significant conclusions such as the location of failed members, the necessity of nonlinear analysis, modifying the capacity of bracing members in ASCE and IS Codes, the effect of the shape of bracing, the design of redundant members.

Behrouz Asgarian, et al [10], in their study, evaluate the progressive collapse of 400 kV transmission tower. They determined the load increase factors after the failed element removal through static analyses. In addition, the capacity to demand ratio suggested to identifying the most critical members after different removal scenarios. They compare these parameters with overload factor calculated from pushdown analysis.

F.Albermani,. et al [11] present a nonlinear technique for the failure analysis of transmission towers. They use this technique to verify a new tower design. In addition, the authors suggest using this technique to save the costs of full test.

This paper investigates the nonlinear inelastic transmission towers response under HIW loading and provides a comparison of the tower capacities (i.e., yield and maximum) for wind loading. The modeling of actual and real lattice transmission tower presented herein. Commercial software SAP 2000 used in the modeling due to its ease handling of nonlinear material properties and3-D numerical simulation capability. The analysis considers both material and geometric nonlinearity and it shown that an adequate approximation of the capacity curve of the tower obtained using the NSP method. Capacity curve for wind loading condition in transverse direction to the tower obtained.

5-Analytical Models For Transmission towers

For tracing of failure mechanisms and determination of its capacity under the effect of High Intensity Wind loads, two types of towers studied. The structural design of the tower based on the wind loads acting on the conductor/tower body as well as self-weight of the conductor /tower

Three-dimensional nonlinear models developed using SAP 2000-V17.3. The static pushover analysis procedure, which is well known technique in performance-based design for seismic and wind loading is adopted. All members modeled by introducing the material nonlinearity at discrete, user defined plastic hinges at terminals.

The hinge properties created with pushover analysis and are provided based on FEMA-356 [3]criteria.

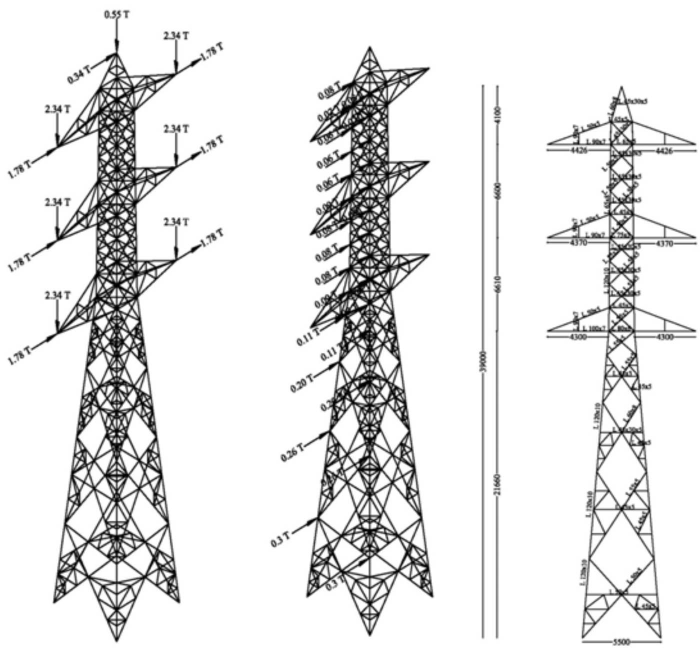
The used modulus of Elasticity equals to 2100 t/cm2, basic wind speed value of 35m/s, the main legs formed from high tensile steel with proof strength 3.60 t/cm2 while the web members made of mild steel of yield strength 2.4 t/cm2. The applied loads considered as per Egyptian Ministry of Electricity Code (12).

The first studied tower is T.T 220 KV that has a 39 ms height, and 5.5 m × 5.5 m square base distance. It has a total of 885 members and 186 joints and the structural system is self-supported cantilever type. All members of the tower are equal legged angle sections and modeled as three dimensional frame elements. Bracing members, which used to decrease the slenderness ratio of the main members thereby increasing their bucking capacity.

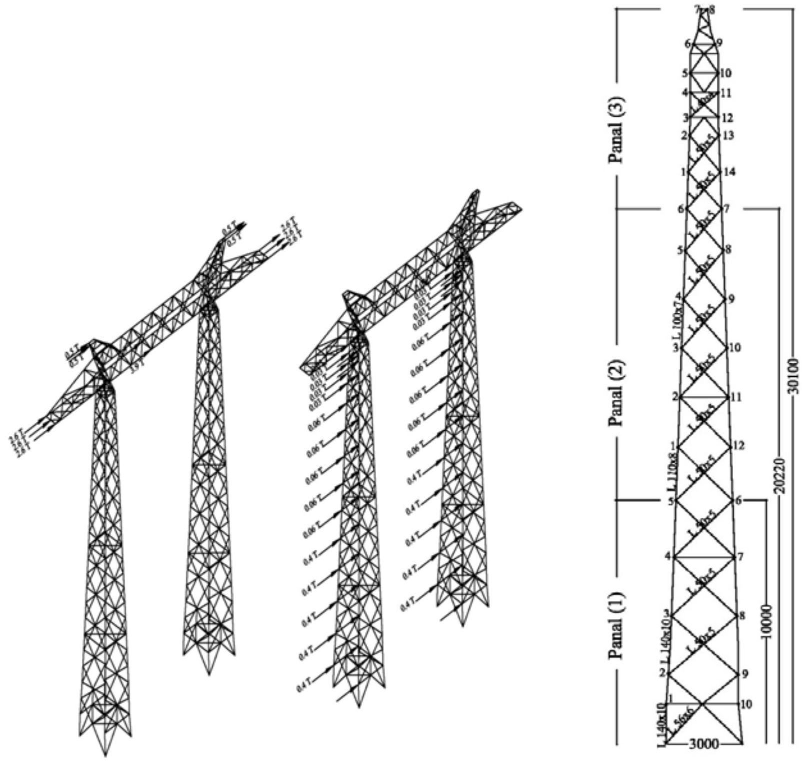
The diagonal triangulation systems in this type of transmission towers are X-bracing. The tower configuration, dimensions and members cross sections are shown in Figure (1).

The second analyzed tower is T.T 500 KV. The tower has a framed configuration with horizontal truss as the frame girder and two vertical trusses as the vertical columns. This tower has a height of 30 m and 3 m × 3 m as a square base dimensions for each vertical truss. The tower has a total of 1618 members and 698 joints.

All members of the tower are equal legged angle sections with different sizes as detailed in Figure (2) and modeled as three dimensional frame elements.



Fig(1) 220KV Tower dimensions and members cross sections



Fig(2) 500KV Tower dimensions and members cross sections

6-Analysis and Results

The NSP and IDA analysis done for each tower under the normal operating load condition (self-weight and wind pressure). Under this condition, the dead load case was applied at first and then the incremental application of transverse wind load until reaching the ultimate capacity. The results of the analysis are the capacity curve that correlate the relation between the base shear (summation of the horizontal reaction in wind direction) and the horizontal displacement at the top of the tower in dominate direction. Figures (3) and (4) shows the capacity curves for the 220KV and 500 KV towers respectively. From the analysis.

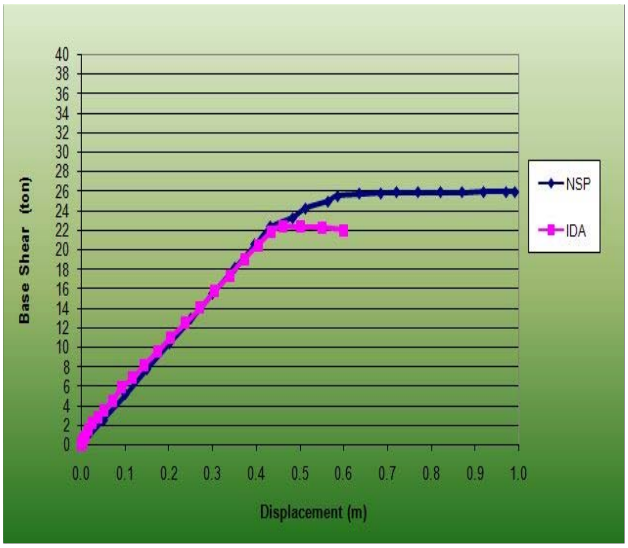


Figure (3) 220 KV tower capacity curve

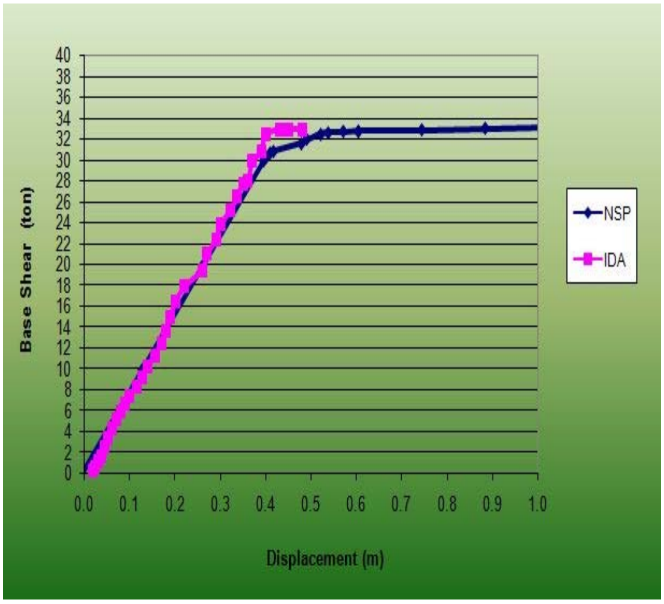


Figure (4) 500 KV tower capacity curve

The maximum horizontal displacement at failure for the 220KV tower is 969 mm at maximum base shear of the value 25.4 tons. For 500 KV tower, the maximum horizontal displacement is 613 mm at maximum base shear equals to 33.4 tons.

Referring to the design loads at normal operating condition, the maximum horizontal displacement in the dominant direction is 270 mm at base shear equals to

15.22 tons for 220 KV tower while the corresponding values for the 500 KV tower are 383 mm and 25.70 tons respectively.

Table (1) summarize the obtained results. From that table, the actual wind load factors for the normal operating condition (λ = Maximum Base shear at Failure/Design Base shear) for both studied towers are determined.

Table (1) Actual wind load factor at NSP

T.T. Type	Design Base Shear (Ton)	Design Max. Horizontal Displacement (mm)	Failure Base Shear (Ton)	Failure Max. Horizontal Displacement (mm)	Wind Load Factor λ
220 KV	15.22	270	25.4	969	1.67
500 KV	25.78	383	33.4	613	1.29

The maximum horizontal displacement at failure for the 220KV tower is 488 mm at maximum base shear of the value 22.39 tons. For 500 KV tower, the maximum horizontal displacement is 475 mm at maximum base shear equals to 32.9 tons. Table (2) summarize the obtained results for the IDA analysis.

Table (2) Actual wind load factor at IDA

T.T. Type	Design Base Shear (Ton)	Design Max. Horizontal Displacement (mm)	Failure Base Shear (Ton)	Failure Max. Horizontal Displacement (mm)	Wind Load Factor λ
220 KV	15.22	270	22.39	488	1.47
500 KV	25.78	383	32.9	475	1.27

The mode of failure of the 220 KV tower resulted from the theoretical analysis indicates that the first two plastic hinges formed at the upper part of first diagonal member at the first panel and the failure occurred in the second panel started 5.6 meters from the base of the tower. Figures (5) illustrate the both obtained theoretical mode of failure and the real collapsed 220 KV tower at two cases.

For the 500 KV tower, the mode of failure obtained from two analysis demonstrate that the first two plastic hinges occurred at the lower part of the first diagonal and the failure take place at a height of 8 meters from the base. The comparison between the theoretical mode of failure and the real collapsed tower shown in the figures (6) .

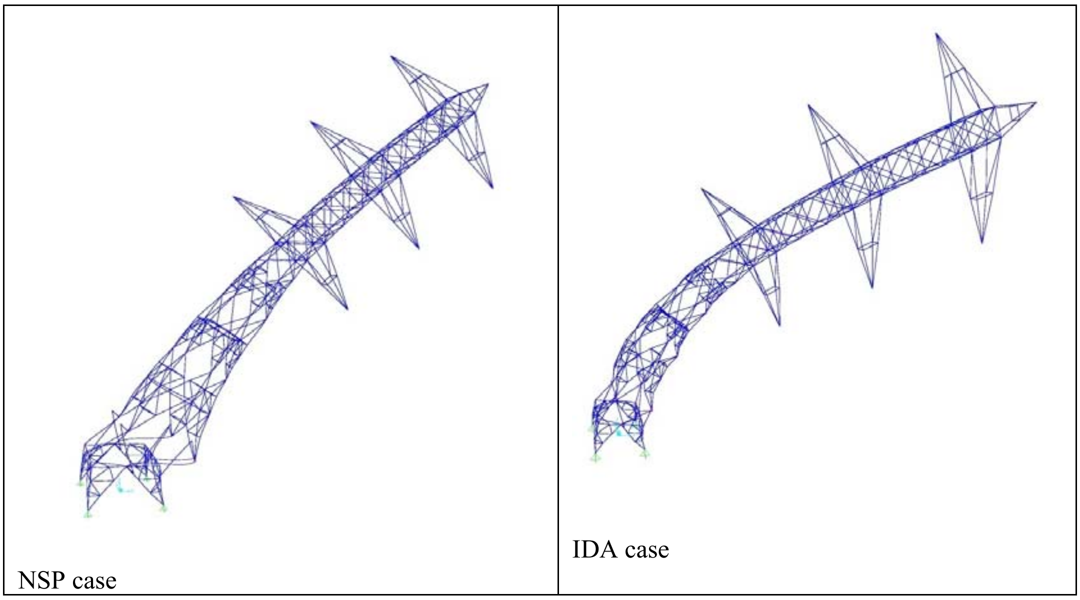


Figure (5) 220 KV tower deformed shape at collapse

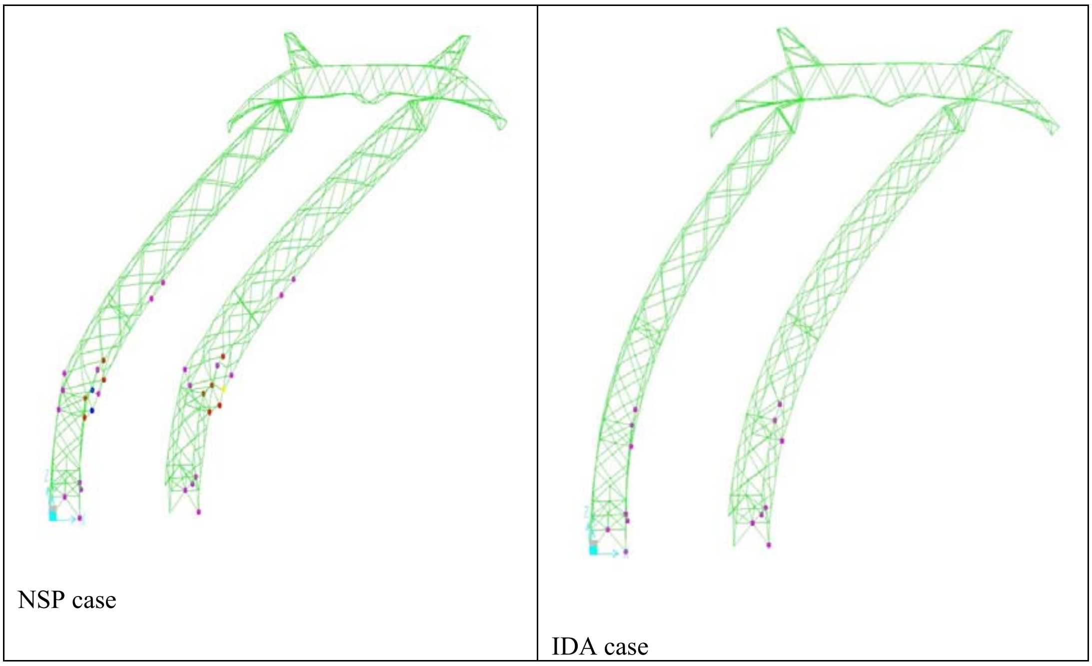


Figure (6) 500 KV tower deformed shape at collapse

The comparison between the theoretical base shear and the real collapsed tower at NSP and IDA shown in the figures (7) and figures (8).

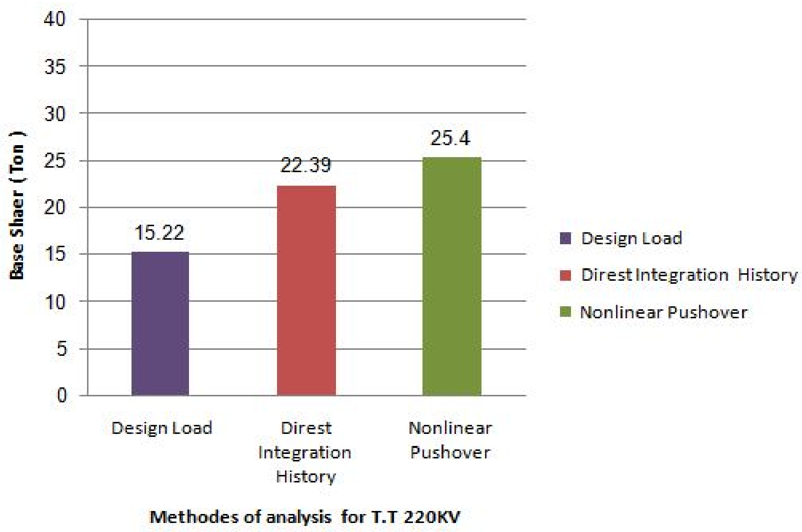


Figure (7) 220 KV tower base shear at collapse

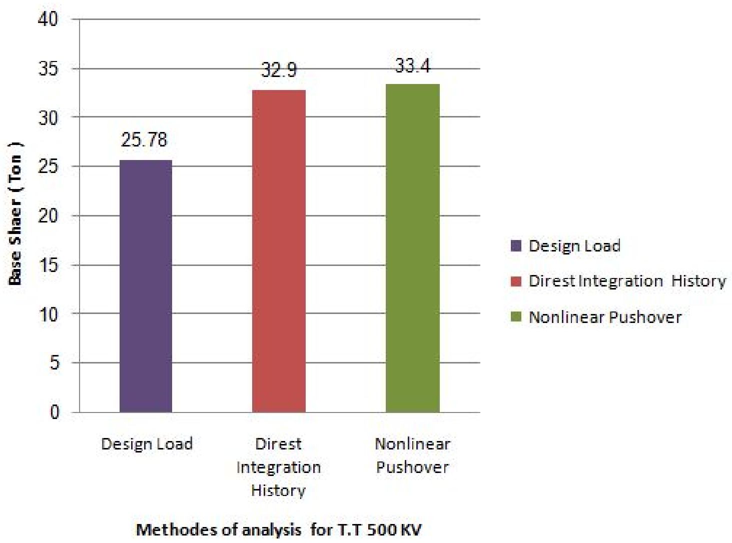


Figure (8) 500 KV tower base shear at collapse

7- Conclusion

From the preliminary results, it is observed that the capacity curves obtained using NSP and IDA procedures show a bilinear load-deformation relationship for the transmission tower under wind loads. It is also observed that there is no significant difference between the capacity curves obtained from these procedures.

No considerable difference was observed for IDA capacity curves resulting from using wind histories of different durations . This presented that the capacity curves based on a 1-minute fluctuating time history provide a suitable description of the force-deformation relation of the structure.

If estimated results for the capacity curve of the tower are sought, it is as a result reasonable to use the capacity curve obtained using the NSP analysis. This greatly reduces the computing effort for analysis.

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