SECOND ORDER PLASTIC ANALYSIS AND DESIGN OF A STEEL SCHWEDLER DOME BY MEANS OF SPECIAL SOFTWARE

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1. ABSTRACT

A system approach is employed for the collapse analysis and design of a conical steel Schwedler dome, acted upon by dead, snow and wind loads. The features of the incremental – iterative 2nd order plastic analysis of the NIDA Pro 8 software are utilized, which (a) allows for member and structure initial imperfections as well as the change of structural geometry after loads, (b) accounts for the 2nd order non-linear effects due to axial force and the effect of member slenderness on the axial force capacity, (c) designs a structure by section capacity check without the need of the effective length, (d) checks the member as well as the global structural instability and (e) captures all stages of progressive yielding - failure during load increase till total collapse. After initially choosing the cross – sectional sizing of the dome's tubular members via linear elastic analysis, the plastic analysis leads to the final choice of cross-sections and to the most unfavorable load combination. The collapse load is found 2.65 greater than the factored elastic one, while all intermediate local and global sway – buckling phenomena are comprehensively revealed.

2. INTRODUCTION

Domes are one of the oldest and well-established structural forms and have been used in architecture since the earliest times. They are of special interest to engineers as they enclose a maximum amount of space with a minimum surface and have proved to be very economical in terms of consumption of constructional materials. Focusing on single-layer braced domes, frequently referred to also as single layer reticulated shells, and according Makowski's classification [1, 2], some commonly used configurations are shown in Fig.1.

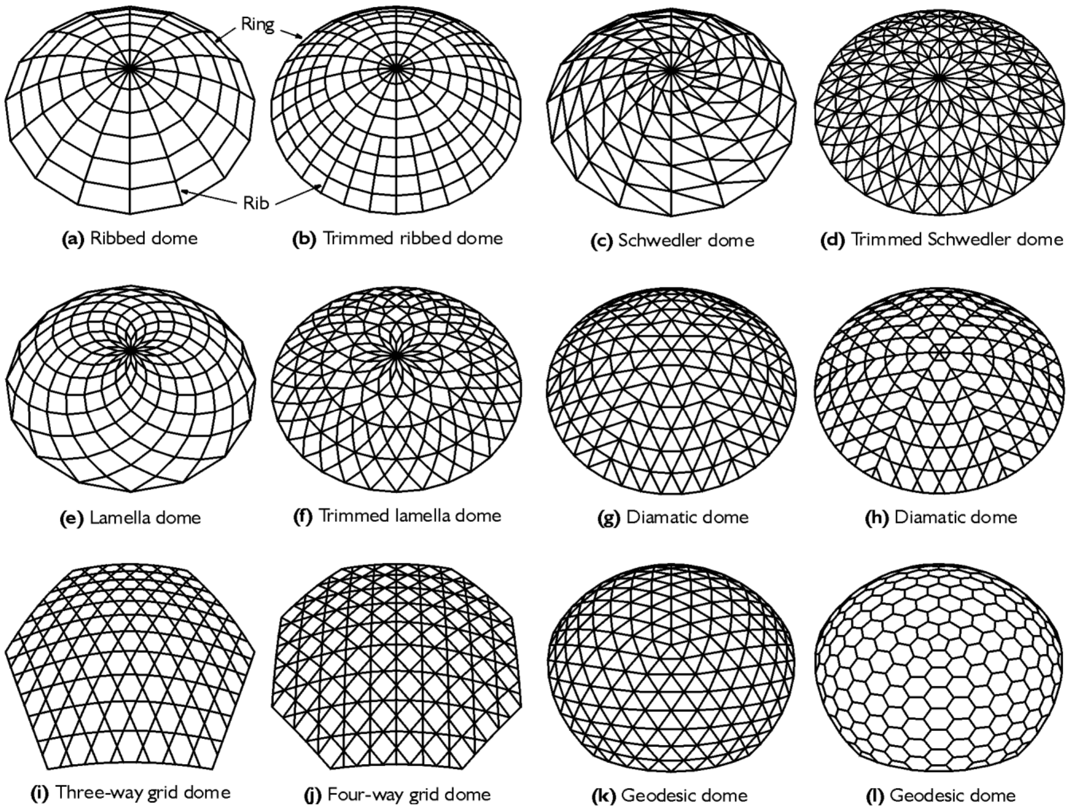


Fig. 1 Configuration of basic single layer domes

In particular, a Schwedler dome (after the name of the German engineer J.W. Schwedler, who introduced this type of dome in 1863) consists of meridional ribs connected together to a number of horizontal polygonal rings. To stiffen the resulting structure so that it will be able to resist non-symmetric loads, each trapezium formed in the above manner is subdivided into two triangles by introducing a diagonal member. Its design may also involve trimming to avoid overcrowding of the elements at the upper part of the dome (Fig.1c, d), while the popularity of this dome type is due to the fact that, on the assumption of pin-connected joints, the structure can be analyzed as statically determinate. Evidently, as also valid for all types of braced domes, steel is the material of choice and tubular members combined with various types of well-recognized joints are used worldwide. An important point that should be borne in mind is that one should be careful in using single layer domes unless the jointing system provides sufficient rigidity for the connections and that the elements are designed for resisting bending and shear in addition to the axial forces. Otherwise, the structures will be prone to snap through buckling.

Depending on their dimensions and member connectivity, in conjunction with material geometrical nonlinearities (due to imperfections), these structures under combined conditions – mainly dead, snow and wind loads – may exhibit numerous different of progressive collapse and various local and global instability phenomena [3-7]. Hence, the software tool that should be used for their analysis and design must be capable of capturing all intermediate response stages till total failure in a comprehensive manner, including elasto-plastic material behavior. This is the case of the program NIDA (Nonlinear (Nonlinear Integrated Analysis and Design) Pro 8 [8], developed by Prof. S.L. Chan in the Hong Kong Polytechnic University, and more specifically of the 2nd order plastic incremental – iterative analysis embedded therein, which provides versatile nonlinear buckling solution methods (Conventional and Modified Newton-Raphson, Displacement Control, Constant Work, Arc-Length, Minimum Residual Displacement, more details in the the relevant web site) and possesses enhanced features and capabilities, described in follows: (a) When a member fails, a hinge is inserted to the member end closer to the position and analysis continues until the collapse load is reached. The collapse load is taken as the load level which does not allow further load increase indicated as a curve descended or stagnated in the load vs. deflection plot. In design, this collapse load should be greater than or equal to the factored design load in all load cases, (b) The program calculates the displacements and rotations at all the nodes or junctions of elements or members, allowing for the change of structural geometry (P-Δ and P-δ effects) after (c) Calculates the bending moments about the element axes, torsional moment about the longitudinal length of the element and axial force in the member, allowing for the second-order non-linear effects due to axial force. Shear will also be determined in the computer output, (d) Designs structure by section capacity check that effective length is not to be assumed. No explicit assumption or evaluation of effective length is required. Thus, the classification of sway and non-sway for determination of effective length ratio as required by some ultimate limit state design codes is not needed. Note that effective can hardly be assessed accurately in many real cases, (e) Performs checking of members well as global structural instability via a rigorous incremental-iterative process, (f) Designs the structure by a system approach, in contrast to the "member –based" design method, The design and buckling capacity of a structure can be computed by incrementing the until member failure, sensed by material yield or sectional failure, occurs and (h) Assumes elastic – plastic material behavior, including strain-hardening.

According to all the above, the purpose of this work is to demonstrate the application of the aforementioned methodology on the collapse analysis and design of a 60m in base diameter, and 30m in center height conical steel Schwedler dome, acted upon by dead, wind and snow loads as prescribed in the ultimate limit state combination requirements of Eurocode 1 [9,10]. The initial choice of tubular members' cross-sections and sizing was the product of linear elastic analysis, with a significant capacity margin for seismic design, while, after several attempts and alternations of cross-sectional properties, the final design lead to the collapse load being equal to 2.65 times the factored elastic design one. All local and global instability phenomena were captured during the process, the most unfavorable load combination was identified and the sway-buckling collapse mechanism was captured.

2. DOME GEOMETRY, DESIGN CHALLENGES AND LOADING

2.1 Geometry and design challenges

The structure dealt with was a single-directional Schwedler Dome of conical shape, with base diameter equal to 60m and height at the apex 30m, with pin-jointed steel tubular members made of S275 steel. In order to successfully address problems concerning constructability, medium slenderness of members and standard connection angles in the cone was divided into 30 levels, with the diameter of each circle from intersection decreasing by a step of 2.00m, while 90 meridian divisions were adopted. No trimming took place before the 19th level, and the arrangement of members approaching the apex was performed on 1:2 density bases for all joints. The final dome structure consisted of 2035 meridian, 2120 horizontal and 2030 diagonal members, illustrated throughout 2 and 3.

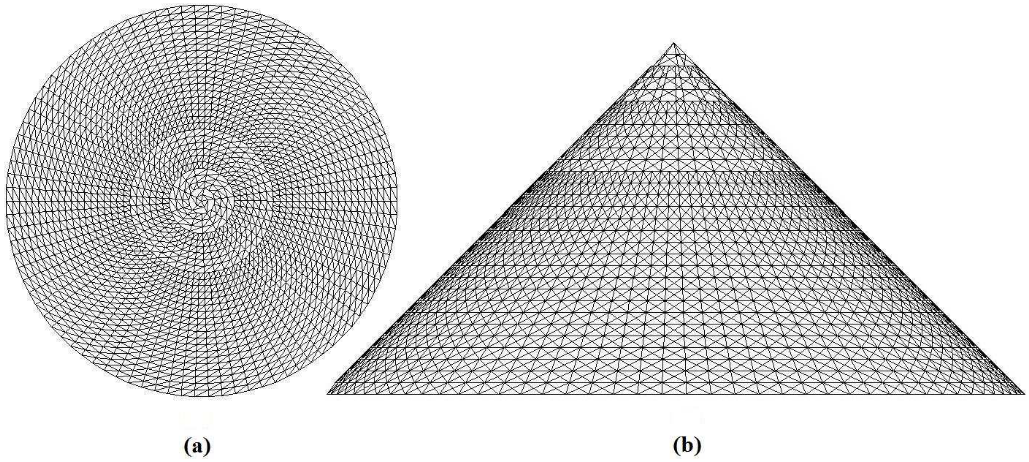


Fig. 2 Top view (a) and front view (b) of the dome

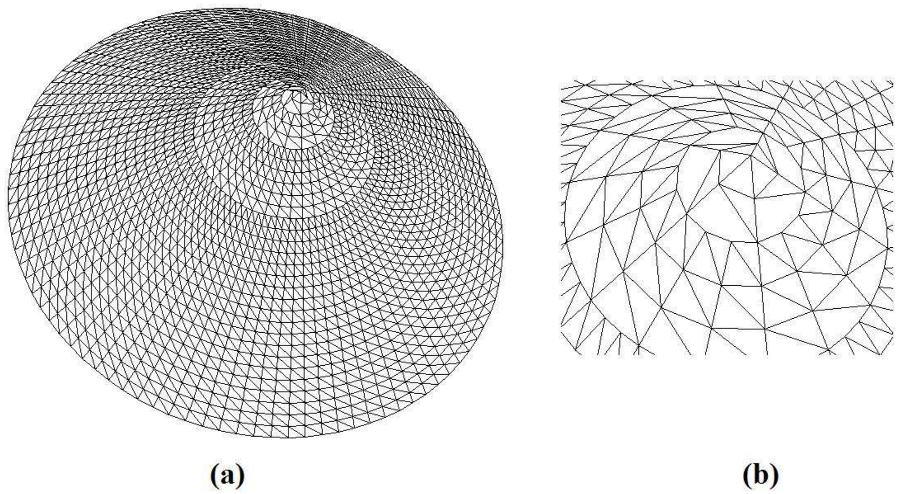


Fig. 3 3D view (a) and apex detail (b) of the dome

2.2 Loading of the dome structure

As mentioned earlier, the loads considered in the analysis and design of the dome were snow, wind and self-weight. Given the geometry of the structure, each of these loads was calculated according to Eurocode 1 and inserted as concentrated on the joints, with the prescribed direction. The three major ultimate state combinations, e.g. 1.35G + 1.50Q, 1.35G + 1.50W, 1.35 (G+Q+W), were considered in the analysis outlined below, while as far as self-weight was concerned (automatically calculated by the software) a 2.5 % increase was inserted in order to account for the weight of cover and connection.

3. ANALYSIS AND DESIGN

3.1 Linear elastic analysis and preliminary choice of cross-sections

At first, a uniform CHS cross-section, namely 139.7x4, was chosen for all members. After performing linear elastic analysis of the dome via NIDA (for the aforementioned three load combinations), it was found that all members were severely understressed and that a significant variation of capacity performance was revealed; meridian members showed a 75% redundancy, while this percentage was narrowed down to 17% and 12% for the horizontal and diagonal members. Hence, it was decided that three different cross-sections should be adopted for the members grouped as above, i.e. one for the meridians, a weaker one for the horizontals and a weakest for the diagonals. In doing this, CHS 219x5, 193.7 x5 and 139.7x4 were adopted, and all of them exhibited an 1/2.7 = 37% elastic redundancy.

Consequently, since seismic effects were not accounted, the deformed configuration of the linear elastic analysis was taken as the initial design stage of the 2nd order plastic analysis that followed.

3.2 2nd order plastic analysis, results and discussion

Thereafter, the 2nd order advanced analysis of NIDA was performed on the structure, for all three loading combinations, in order to capture (a) the most unfavorable loading pattern and (b) all related local and global instability phenomena till total failure – collapse. Due to the dense connectivity of members to a quite significant height, snapping was never revealed, while the mechanism of sway-buckling was the one that lead to collapse, due to the 1.35(G+Q+W) combination after 53 increments of 0.05 times the elastic load. Hence, the collapse load was found equal to 2.65 times the factored elastic one, and the progressive yielding for all incremental stages was comprehensively identified for each and every member, while the global buckling response was captured till total failure. Evidently, this may perceived in the side and top views of the dome structure at some characteristic stages, were the sway-buckling phenomenon is pronounced, given in Figures 4 and 5.

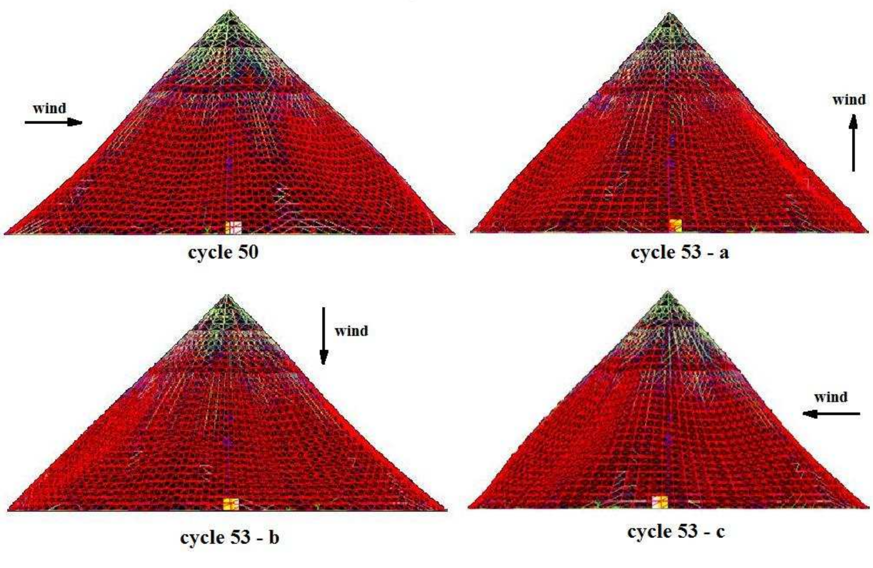
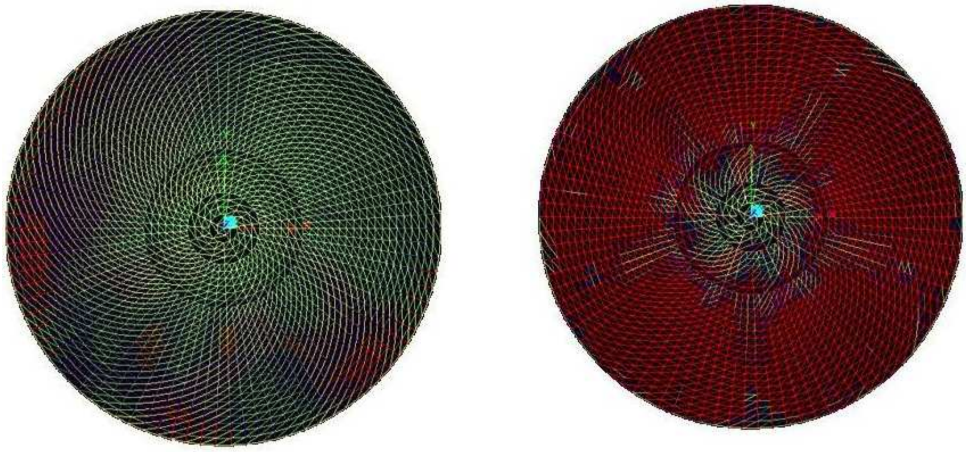


Fig. 4 Side views of the dome near and at collapse



cycle 49 **cycle 50**

Fig. 5 Top views of the dome near collapse

Interestingly enough, it also was found that the member ends displacement vs. load factor behavior was not typical for all three member types - horizontal, meridian and diagonal - but to a quite significant extent the response of the diagonals was different, exhibiting a reversal at some late stage, as shown in Figures 6 and 7. It is postulated that this phenomenon is ought to (a) their smaller cross-section, (b) their stiffening effect on the overall structure and (c) abrupt change of deformation pattern due to yielding and/or local buckling of neighboring members.

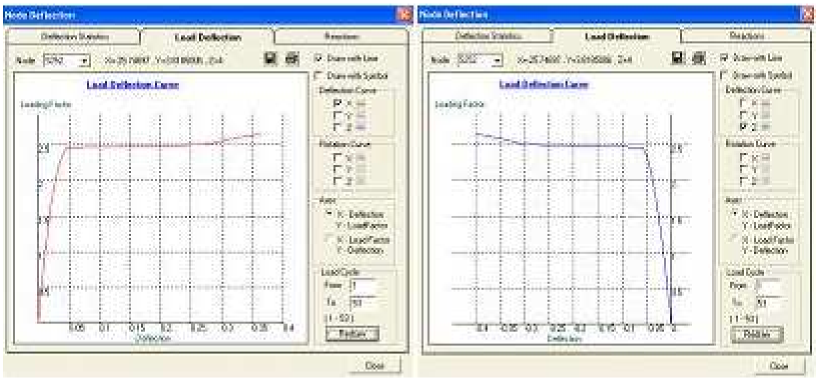


Fig. 6 Typical end displacement - load factor curves for horizontal and meridian members

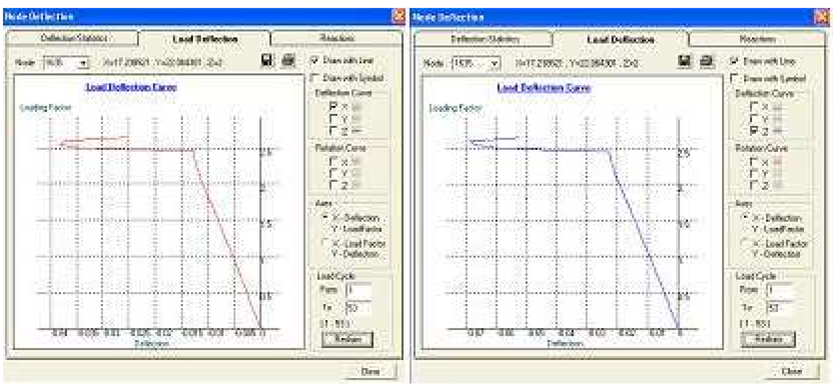


Fig. 7 Typical end displacement - load factor curves for diagonal members

4. CONLUDING REMARKS

The second order plastic analysis employed herein, via NIDA software, was found adequate and successful in capturing all local and global phenomena related to the collapse of the structure dealt with. More specifically, under the combined factored loading of self-weight, wind and snow, the dome exhibited a sway-buckling mode, which leads to collapse, after moderate choice of cross-sections based on linear elastic analysis.

It is suggested that the proposed scheme should be the starting point for preliminary plastic design of steel space structures, the outcome of which would serve as background material for more sophisticated FEM analyses, combined with wind-tunnel scaled results.

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ΠΛΑΣΤΙΚΗ ΑΝΑΛΥΣΗ 2ΗΣ ΤΑΞΗΣ ΚΑΙ ΣΧΕΔΙΑΣΜΟΣ ΧΑΛΥΒΔΙΝΟΥ ΘΟΛΟΥ ΤΥΠΟΥ SCHWEDLER ΜΕΣΩ ΕΞΕΙΔΙΚΕΥΜΕΝΟΥ ΛΟΓΙΣΜΙΚΟΥ

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ΠΕΡΙΛΗΨΗ

Χρησιμοποιείται μια συστημική προσέγγιση για την ανάλυση κατάρρευσης και το σχεδιασμό ενός χαλύβδινου θόλου τύπου Schwedler κωνικού σχήματος, υπό φορτία ιδίου βάρους, χιονιού και ανέμου. Τούτο πραγματοποιείται με εφαρμογή της επαυξητικής – επαναληπτικής πλαστικής ανάλυσης 2ης τάξης του λογισμικού NIDA, που (α) επιτρέπει αρχικές ατέλειες μελών και την αλλαγή της γεωμετρίας του φορέα λόγω της φόρτισης, (β) λαμβάνει υπόψη μη γραμμικές επιρροές 2ης τάξης λόγω αξονικής έντασης καθώς και την επιρροή της λυγηρότητας μελών στην αντοχή τους σε αξονική δύναμη, (γ) σχεδιάζει μια κατασκευή μέσω ελέγχου διατομών χωρίς να απαιτείται υπολογισμός του ισοδύναμου μήκους λυγισμού θλιβόμενων στοιχείων, (δ) ελέγχει έναντι αστάθειας τόσο τα μέλη όσο και την κατασκευή συνολικά και (ε) «συλλαμβάνει» όλα τα στάδια προοδευτικής διαρροής – αστοχίας μέχρι την κατάρρευση. Μετά την αρχική επιλογή των ΚΚΔ των μελών μέσω γραμμικής ελαστικής ανάλυσης, η πλαστική ανάλυση οδηγεί στην τελική επιλογή διατομών και στην αναγνώριση του δυσμενέστερου συνδυασμού φόρτισης. Το φορτίο κατάρρευσης βρέθηκε ίσο με 2.65 φορές το αντίστοιχο ελαστικό, ενώ αποκαλύφθηκαν και όλα τα ενδιάμεσα φαινόμενα τοπικής και καθολικής αστάθειας τύπου μετάθεσης.