Advances in Computational Design, *Vol. 2, No. 2 (2017) 133-146*
DOI: https://doi.org/10.12989/acd.2017.2.2.133

Reliability analysis of double-layer domes with stochastic geometric imperfections

Mehrdad Gordini[*1], Mohammad Reza Habibi[1a], Mohammad Reza Sheidaii[2b] and Mehrzad Tahamouliroudsari[1c]

[1]*Department of Civil Engineering, Kermanshah Branch, Islamic Azad University, Kermanshah, Iran*
[2]*Department of Civil Engineering, College of Engineering, Urmia University, Urmia, Iran*

(Received December 1, 2016, Revised February 18, 2017, Accepted March 6, 2017)

Abstract. This study aimed to investigate the effect of initial member length an imperfection in the load carrying capacity of double-layer domes space structures. First, for the member length imperfection of each member, a random number is generated from a normal distribution. Thereupon, the amount of the imperfection randomly varies from one member to another. Afterwards, based on the Push Down analysis, the collapse behavior and the ultimate capacity of the considered structure is determined using nonlinear analysis performed by the OpenSees software and this procedure is repeated numerous times by Monte Carlo simulation method. Finally, the reliability of structures is determined. The results show that the collapse behavior of double-layer domes space structures is highly sensitive to the random distribution of initial imperfections.

Keywords: reliability; Monte Carlo simulation method; progressive collapse; imperfection; double layer grids; space structures; domes

1. Introduction

In the recent decades, the use of space structures to cover large spans with no internal columns has become increasingly prevalent due to advantages such as low weight and considerable stiffness. These structures are regularly built by connecting steel rods in single or double layer forms (Thornton and Lew 1984). The factors that affect the behavior of these structures are quite diverse and depend upon the behavior of every single member and also their connecting system. Although these structures are manufactured industrially, the members of these structures behave inconsistently because of different mechanical and geometrical properties. Carried out studies on double-layer grids have shown that these types of structures have a symmetrical behavior in the elastic state but display an asymmetrical behavior in the inelastic state and this is due to the existence of common imperfections in the structure.

∗Corresponding author, Ph.D. Student, E-mail: Mehrdad.Gordini@gmail.com or M.gordini@iauksh.ac.ir
[a]Assistant Professor, E-mail: M.R.Habibi@iauksh.ac.ir
[b]Professor, E-mail: M.Sheidaii@urmia.ac.ir
[c]Assistant Professor, E-mail: M.Tahamouliroudsari@iauksh.ac.ir

http://www.techno-press.org/?journal=acd&subpage=7 ISSN: 2283-8477 (Print), 2466-0523 (Online)

Space truss structures have been the subject of much study. Schmidt *et al.* (1980, 1982), have indicated that space trusses may fail in a brittle and unstable manner, in which the buckling of one member due to overloading can trigger a progressive collapse of the whole structure, in which successive members fail in a rapid sequence. Affan (1987) and Affan and Calladine (1989) estimated the number of redundant bars that could be removed without affecting truss stability to be 15% to 25% of the total number of truss members. It is not possible to attribute this result to all types of space structures. The failure of the stadium roof in Hartford in 1984 is a practical example of progressive collapse. Researchers know that member imperfection is one of the most important causes of this type of failure (Smith and Epstein 1980, Thornton and Lew 1984).

Space structures have a high degree of indeterminacy and typically contain hundreds of members which inevitably incorporate different types of imperfection. Initial curvature of a truss member is a common form of geometric imperfection in space structures. To estimate the safety and reliability of such structures, it is necessary to consider the effects of such uncertainty. Several investigations (Roudsari and Gordini 2015, Sheidaii and Gordini 2015, Wada and Wang 1992, Gholizadeh *et al.* 2016) have addressed one or more of these random variables. Roudsari and Gordini (2015) studied the Random imperfection effect on reliability of space structures with different supports. They showed that geometric imperfections like initial curvature have considerable influence on the load-carrying capacity of these structures. El-Sheikh (1991, 1995, 1997, and 2002) studied the sensitivity of double-layer space structures to member geometric imperfections and the overall strength, behavior, and the location of trusses to determine critical areas at which imperfect members should be avoided. El-sheikh also has investigated the effect of member length imperfections on the capacity and failure mechanism of triple-layer space structures.

Schenk and Schuëller (2005) assessed the stability of cylindrical shells with random imperfections. The employed the Monte Carlo simulation (MCS) method and assumed the stress-strain relationship to be linear. Schenk and Schuëller (2007) investigated the effect of random imperfections on the critical load of isotropic, thin-walled, cylindrical shells under axial compression by means of static nonlinear finite element analysis. They proposed the cumulative distribution function (CDF) of the limit load using the MCS. Broggi and Schuëller (2011) presented an efficient model of imperfections to analyze the buckling of composite cylindrical shells. De Paor *et al.* (2012) investigated the effects of random geometrical imperfections in shell cylinders and based on several experimental and numerical samples concluded that the random distribution of imperfections has a good concurrence with the normal distribution. Moreover, in recent years, many researches are conducted on the influence of geometrical imperfections on structural behavior. Kala (2013) analyzed the effects of a random distribution of imperfections on the lateral-torsional buckling of I-shaped rolled beams with simple supports.

Vryzidis *et al.* (2013) investigated the effect of random initial imperfection on the buckling capacity of steel pipes. Numerical analyses and experimental results have shown that initial imperfection has a noticeable effect on system buckling capacity and affects the failure mode of the pipes. Zhao *et al.* (2014) used Monte Carlo simulation to investigate the effects of random geometrical imperfections on concentrically-braced frames and showed that these imperfections have a substantial effect on design forces. Zhou *et al.* (2014) investigated the effect of member geometric imperfection on nonlinear geometrically buckling and seismic performance of a new style of space steel structure, a suspend dome, which is composed of a reticulated shell and cable-strut system. Mousavi *et al.* (2015) investigated the effects of applying different buckling modes obtained by linearized eigenvalue buckling analysis as the initial imperfection for double-domed free-form space structures.

They reported that in free-form double-domes the lowest buckling modes could not be considered as effective for changing the bifurcation equilibrium path into a limit equilibrium path. Reliability analysis methods have been developed over the last three decades (Schueller 1987, Fang *et al.* 2013, Hurtado and Barbat 1998, Zhao *et al.* 2013) and have stimulated the interest for the probabilistic optimum design of structures. Papadrakakis and Lagaros (2002) investigated reliability-based sizing optimization of multi-story 3-D frames. The objective function was the weight of the structure and the constraints were both deterministic and probabilistic. Randomness of loads, material properties, and member geometry were taken into consideration in reliability analysis using MCS. The probability of failure of the frame structures was determined by limit elasto-plastic analysis. Chen *et al.* (2002) assessed the probabilistic dynamic analysis of truss structures by considering the randomness of both physical parameters (elastic module and mass density) of structural materials and geometric dimension of bars consecutively and simultaneously. Koutsourelakis *et al.* (2004) investigated a reliability-based structure in high dimension and considered many random variables. They used linear sampling to investigate the failure domain and concluded that in order to correctly evaluate a structure; various factors which are effective in a structural capacity should be considered randomly. Cardoso *et al.* (2008) examined a methodology for computing the probability of structural failure by combining neural networks and MCS. Torii *et al.* (2012) investigated an approach to reliability-based shape and topology optimization of truss structures. They presented an approach to simultaneously optimize the geometry and topology of statically undetermined trusses considering the acting forces and the yielding stress of the bars as random variables. Lopez *et al.* (2014) investigated global reliability based design optimization of the size and shape of truss structures.

In this study the effect of length imperfection in the bearing capacity of double-layer domes space structures has been probabilistically investigated. Length imperfections distribution among the members has been considered probabilistically and the structure's reliability has been studied using the Monte Carlo simulation method. Considering the nonlinear behaviors and the existence of hundreds of random variables, calculating the structural reliability is very costly and time consuming. Therefore, in this paper, the structure's reliability has been obtained through a direct and simpler approach. All of the analyses have been carried out using the finite element software OpenSees (Mazzoni *et al.* 2005, McKenna *et al.* 2010).

2. Reliability and Monte Carlo simulation method

Statistical analysis techniques have become more widespread in reliability engineering. Structural reliability is the probability that a system will function desirably under predetermined circumstances for a specific time. This means that the structure must perform its intended duty without failing. In many circumstances, it is impossible to mathematically describe the response of structural systems because of random nature and the complexity of the applied loads and initial imperfections. Even after finding a mathematical model to predict the behavior of the system, there is no closed form solution for solving the equation. In such cases, simulation is one of the most applicable techniques to acquire the required information. Simulation is a special technique to approximate the quantities that are difficult to obtain analytically. Amongst many of simulation methods, the Monte Carlo Simulation (MCS) method is one of the well-known and common procedures in solving complex engineering problems (Melchers 1999, Moghadasa and Fadaeeb

2012). The base of all simulation procedures is to generate random numbers that are distributed in 0 to 1 interval. These random numbers could be generated by using a computer programs or random number generating algorithms. In the OpenSees software, standardized algorithm in C++ programming language can be used to generate random numbers. After that, many analyses are carried out and the probability of the collapse of the system is calculated. It should be noted that the calculated collapse is an estimation of the real collapse of the system. By increasing the number of simulations, the estimation will get closer to the actual state. Although this method is very time consuming, it is used in a wide range of engineering problems (Nowak and Collins 2000).

Monte Carlo Simulation is a simulation method that presents the following characteristics: it can be applied to many practical problems allowing the direct consideration of any type of probability distribution for the random variables; it is able to compute the probability of failure with the desired precision and it is easy to implement. However, despite the advantages it presents, the use of this method requires a great number of structural analyses, one for each sample of the set of random variables. The number of analyses needed to evaluate the probability of failure of a structure with a prescribed precision depends on the order of magnitude of that probability.

A reliability problem is normally formulated using a failure function, $g(X_1, X_2,\ldots, X_n)$, where $X_1, X_2,\ldots, X_n$ are random variables. Violation of the limit state is defined by the condition $g(X_1, X_2,\ldots, X_n)\leq 0$ and the probability of failure, P_f, is expressed by the following expression.

$$P_f = \mathrm{P}\left[\mathrm{g}(X_1, \mathrm{X}_2, ..., \mathrm{X}_\mathrm{n}) \leq 0\right] = \iint_{\mathrm{g}(X_1, \mathrm{X}_2, ..., \mathrm{X}_\mathrm{n})} ... \int \mathrm{f}(X_1, \mathrm{X}_2, ..., \mathrm{X}_\mathrm{n}) \mathrm{d}X_1\, \mathrm{d}X_2, ...\mathrm{d}X_n \quad (1)$$

Where $(X_1, X_2,\ldots,X_n)$ are values of the random variables and $f(X_1, X_2,\ldots,X_n)$ is the joint probability density functions. The Monte Carlo method allows the determination of an estimate of the probability of failure, given by

$$P_f = \frac{1}{N}\sum_{\mathrm{i}=1}^{\mathrm{N}} I(X_1, \mathrm{X}_2, ..., \mathrm{X}_\mathrm{n}) \quad (2)$$

Where $(X_1, X_2,\ldots, X_n)$ is a function defined as

$$I(X_1, \mathrm{X}_2, ..., \mathrm{X}_\mathrm{n}) = \left\{ \begin{matrix} 1, \text{if} \;\; \mathrm{g}(X_1, \mathrm{X}_2, ..., \mathrm{X}_\mathrm{n})\mathrm{x} \leq 0 \\ 0, \text{if} \;\; \mathrm{g}(X_1, \mathrm{X}_2, ..., \mathrm{X}_\mathrm{n})\mathrm{x} \succ 0 \end{matrix} \right\} \quad (3)$$

According to Eq. (1), N independent sets of values $X_1, X_2\ldots X_n$ are obtained based on the probability distribution for each random variable and the failure function is computed for each sample. Using MCS, an estimate of the probability of structural failure is obtained by

$$P_f = \frac{N_j}{N} \quad (4)$$

Where N_j is the total number of cases where failure has occurred.

3. Analytical model

Two Kiewitt large-span double-layer spherical lattice shells with different height-to-span ratios

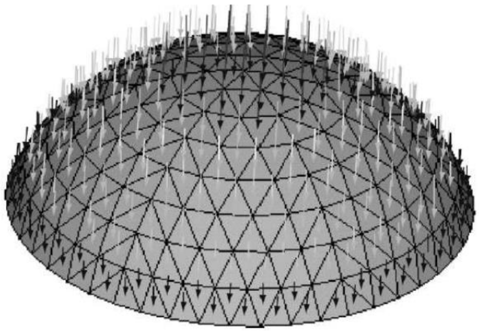
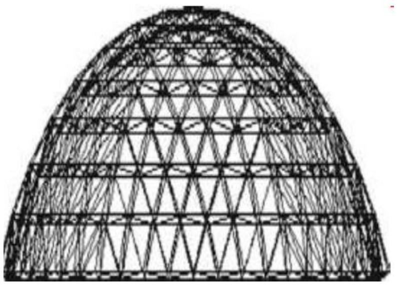
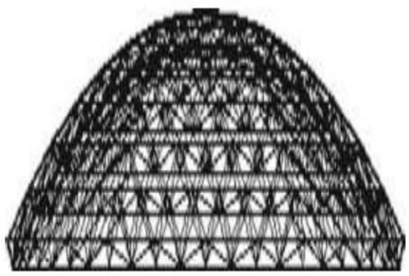


Fig. 1 Schematic of lattice dome



(a) GDA: Height-to-span ratio, 1/3



(b) GDB: Height-to-span ratio, 1/6

Fig. 2 Elevation of Kiewitt double-layer spherical lattice shell with four height-to-span ratios

Table 1 Design sections of the considered structures

Name of structures	Design Sections (mm)		
GDA	CHS 160×12	CHS 114.3×10	CHS 108×8
GDB	CHS 160×12	CHS 108×8	CHS 88.9×12

(Figs. 1 and 2) are selected to model. The 2 double-layer Kiewitt reticulated dome consisted of 8 latitudinal circles and 6 main radial ribs that divided the sphere into axial symmetrical fan-shaped segments. Diagonal members were applied to link the latitudinal and the main radial members, and similar triangular grids were thus formed all over the spherical surface. The geometry and material properties of the lattice shell are taken as follows: span length L=55 m; four curvature radiuses are selected as follows, R=27.95 m and 29.16 m respectively. The structure was designed using the AISC allowable strength design method based on weight optimization of the structural members (AISC 2010). Two different types of loads are considered: the dead load of 0.5 KN/m^2 and the snow load equal to 2.0 KN/m^2. Design sections are introduced in Table 1 (The European Standard 2006). In this model, the dead and snow loads are applied on the joints of the upper layer as concentrated loads. The diagonal members over the supports are assumed to be solid to prevent local collapse of these members and the instability of the entire structure. Steel Circular Hollow Sections (CHS) are used as the principal members of the double-layer dome with a yield stress of 235 MPa, ultimate stress of 355 MPa, and modulus of elasticity of 210 GPa. All of the boundary nodes at the outmost ring in the upper layer of the numerical model are pinned and restrained in vertical, radial and tangential directions to the dome boundary.

For the grids in Fig. 2, first letter denotes the word “grid”, the second letter denotes the word dome and the third character denotes the Height-to-span ratio. For instance, “GDA” is a dome grid with Height-to-span ratio 1/3.

4. Modelling the nonlinear behavior of compressive members

Length imperfections cause initial deformation and stress in a structure. Different methods can be used to investigate the effect of this type of imperfection on a structure. Methods such as the virtual work method, compatibility of deformations, concept of energy, and Koiter’s theory of stability necessitate the solving of complex equations; thus, employing them for complex structures such as space trusses is not possible. One suitable method to investigate the effects of these types of imperfections is nonlinear finite element analysis. The behavior of compressed members has a decisive effect on the failure behavior of double-layer space structures. Load transfer in the members of double-layer grids occurs predominantly in the form of axial forces. When a member of the structure yields under tensile force, its stiffness declines to zero; this state is consistent until the strain hardening phase. When a member buckles under compressive force and cannot sustain more loading, or if shortening of a member occurs, the axial force of the member must decrease for the member to maintain its state of balance. In other words, the compressed member will show strain softening behavior after buckling. Once the load applied to the structure exceeds the elastic limit load, buckling of the compressed member causes an abrupt decline in the bearing capacity of that member and redistribution of the structure’s internal forces ensues. If the structure is able to resist this load redistribution, it will be able to withstand additional loading. If not, other members of the structure will fail and there exists the possibility of successive failure of members, i.e., progressive collapse.

The existence of length imperfections causes initial deformations and stresses in the structure. To investigate the effects of this type of imperfection in the structure, the nonlinear finite element method can be employed. To carry out the static nonlinear analysis for assessing the collapse behavior of space structures, first the nonlinear behavior of each member will be determined. In this paper, the axial load-axial displacement behavior of the members in tension has been considered to be elastic perfectly plastic.

To determine the axial load-axial displacement response of compression members, the nonlinear static analysis has been utilized. To obtain the governing buckling mode of the members, an initial curvature with the maximum lateral deflection of 0.001 L is considered. The initial geometrical imperfection has been considered as a sinusoidal half-wave in such a way that the maximum deviance will be in the mid-span of the member (Fig. 3). This member was created in OpenSees with twenty Elastic-Perfectly Plastic nonlinear displacement-based beam-column elements with equal length, integrated at 4 points along the element. The integration is based on the Gauss-Legendre quadrature rule which enforces Bernoulli beam assumptions. All the subjected profiles in this study have been modelled using the “Fiber section” model in the OpenSees software. The cross section of the member along its radius and circumference has been divided to 4 and 16 equal parts respectively (Sheidaii and Gordini 2015).

Afterwards, for each of the design sections, a nonlinear analysis has been carried out and the load or the maximum force that the member can bear is calculated. In Fig. 4, the load-displacement diagrams for each one of member are depicted. This figure has been drawn based on the corresponding load and displacement.

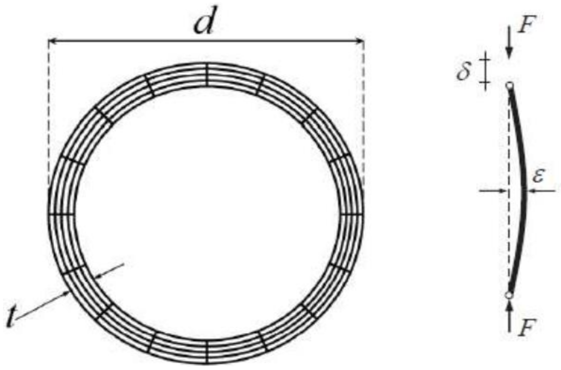


Fig. 3 Geometry and mesh properties of a compression member

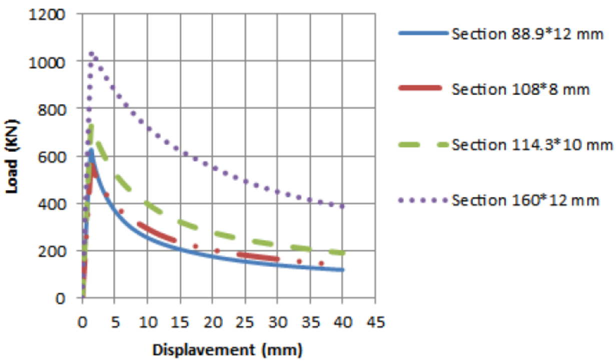


Fig. 4 Axial load-axial displacement behavior of the compression members

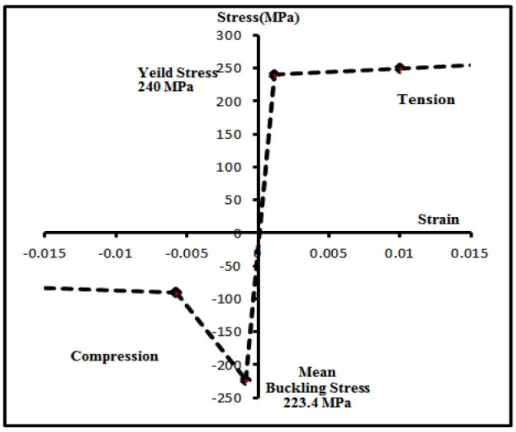


Fig. 5 The axial stress-strain diagram of imperfect member (CHS 114.3×10 section)

The axial stress-strain relationship of structural members is assumed to be elastic perfectly plastic in tension. However, the performances of compressive members are considered to be a function of three fundamental factors namely: the slenderness ratio, the yield stress and the initial imperfection. Based on the variation of these three parameters, the compression members may have a ductile or brittle behavior. The axial force-displacement relationship of an imperfect

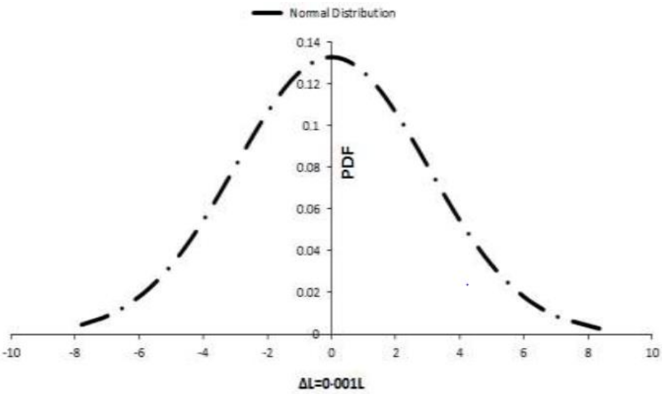


Fig. 6 Probability density function of the members' length imperfections in compression members with normal distribution

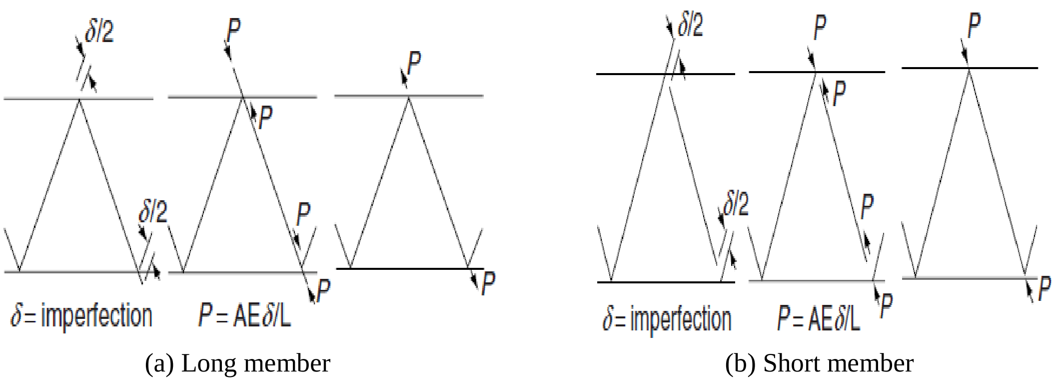


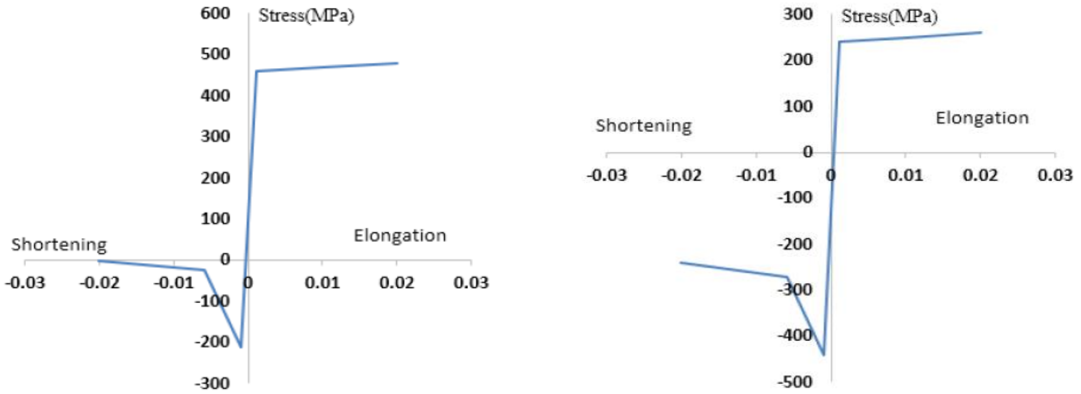
Fig. 7 The modelling approach of the member with the length imperfection (EL-Sheikh 1997)

member is obtained by the nonlinear static analysis using the Newton-Raphson algorithm and considered as Fig. 5.

5. Random modeling of the length imperfection the members

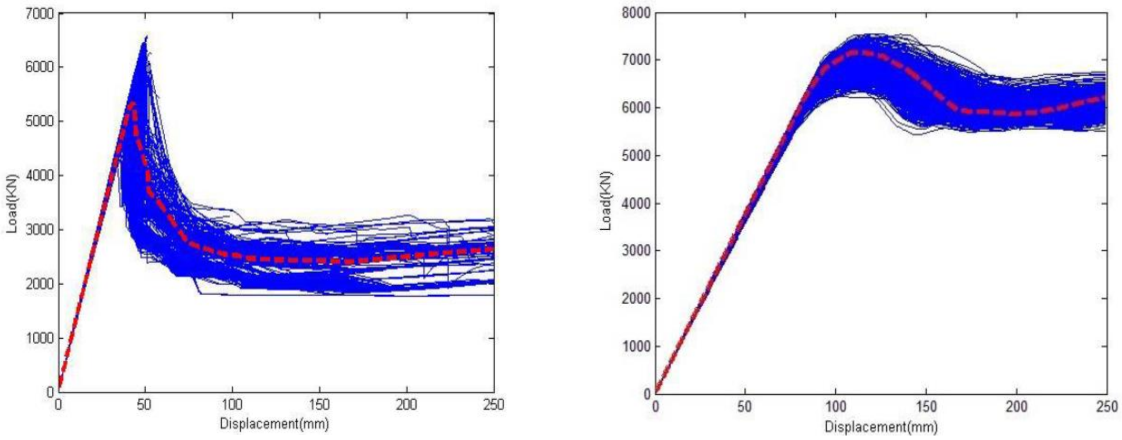
In this study, Member length imperfections were also considered using normal distribution. The parameters of the normal distribution had an average of zero and a standard deviation of 0.0001 L (*L*=member length). The maximum imperfection was limited to 1% of the member length because application of members with large imperfections can easily be avoided. Member length imperfections for both long and short members were randomly taken into account according to the randomly generated numbers. These random numbers were generated using a normal distribution and the aforementioned specifications to obtain the PDF of the imperfections shown in Fig. 6.

The method proposed by El-Sheikh was employed to model length imperfections of the members. Fig. 8 shows that, for a member to be in its ideal place, it must be subjected to tension or compression. These forces change the behavior of the member and also change the stress-strain diagram of the imperfect member. The length imperfections shown in Fig. 7 have been exerted as



(a) a long member with ΔL=+2.3 (b) a short member with ΔL=-1.7

Fig. 8 Idealized stress-strain relationship of imperfect members under tension and compression for the 88.9×12(mm) sample with 3 meter length



(a) GDA (b) GDB

Fig. 9 Load-displacement diagrams of the double-layer domes space structures

force couples on the ends of the members to place them in position by lengthening or shortening them. The force exerted on the member to cause a change in length is

$$P = \frac{EA\Delta L}{L} \tag{5}$$

Where P, E, A and ΔL are the force applied to the member, the modulus of elasticity, the member's cross sectional area and the member's length imperfection, respectively.

These forces affect the stress-strain specifications of the member and to shorten or elongate the member by $\pm\Delta L$, a force couple is needed to produce an axial stress-equal to $\sigma=E.\ \varepsilon=\mathrm{E}.\Delta L/\mathrm{L}$. So, each one of the generated ΔLs corresponds to the every forces that alter the behavior of the imperfect member. Ultimately, these changes only appear in the ideal stress-strain diagram of the imperfect member. Fig. 8 shows the stress-strain diagram for a three meter pipe member with the cross section of 88.9×12 millimeters, having taken into account the length imperfection of the

Table 2 Statistical specifications of the collapse load

Grids Name	Failure Load				
	Mean (x)	Deviation (S)	PMAX	PMIN	RSD (s/x)
GDA	5330	150	6651	4400	0.028
GDB	7021	230	7500	6100	0.032

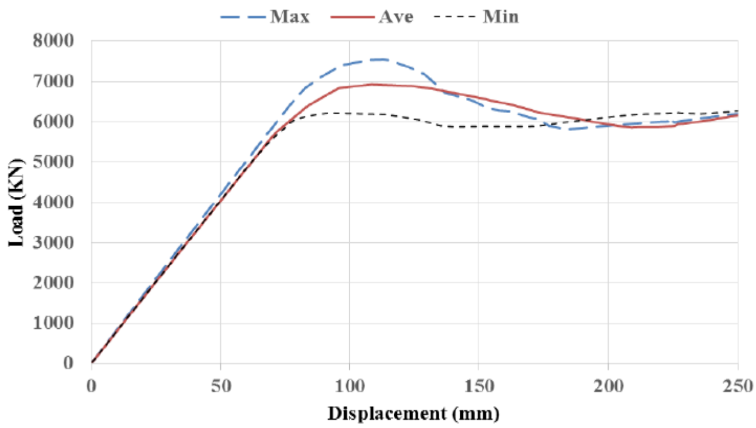


Fig. 10 The maximum, average, and minimum load bearing capacity of the double-layer dome space structures (GDB)

member. The length of imperfection of a member is random and follows the probability density function (PDF). Fig. 8, for example, the values -1.7 mm and +2.3 mm have been considered as the imperfections that cause a member to be short or long, respectively. It is worth mentioning that in this study, for each length imperfection, such calculation has been carried out and the stress-strain specifications of the imperfect member, as well as the behavioral model for both long and short members have been obtained and illustrated in Fig. 8. Each stress-strain specification that corresponds to a certain imperfection was modeled as a random variable in OpenSees. For each imperfect member of the grid, a stress-strain diagram was generated using random numbers.

6. Results of the random collapse analysis

To calculate the bearing capacity of the imperfect grid, first the amount of the length imperfection of the members has been considered as a random variable with normal distribution. By allocating a random length imperfection to each member, their idealized stress strain diagrams change and the effect of the length imperfection of the members will have been taken into account by modifying these diagrams.

After allotting the stress-strain diagram of the imperfect material to each member, the structure was analyzed with the Push Down analysis along the vertical direction. The random allocation of imperfections and the nonlinear Push Down analysis were repeated thousands of times. At first 100 analyses are done, then average failure load are computed, in the next step 200, 300, … and 1000 analyses are done and average failure load are calculated again. It is observed that, there is no remarkable change in amount of average failure load after 500 analyses. Therefore to obtain more

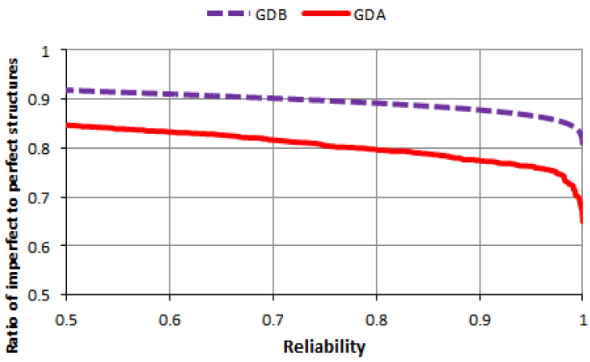


Fig. 11 Reliability of selected structures

Table 3 Reliability of the GDA and GDB double-layer domes space structures

Reliability	Capacity ratio of imperfect to perfect structure		Coefficient Load factor	
	GDA	GDB	GDA	GDB
1.00	0.653	0.800	1.35	1.20
0.99	0.670	0.811	1.33	1.19
0.98	0.700	0.822	1.30	1.18
0.97	0.702	0.836	1.30	1.17
0.96	0.704	0.840	1.30	1.16
0.95	0.708	0.852	1.29	1.15
0.94	0.721	0.858	1.28	1.14
0.93	0.734	0.861	1.27	1.14
0.92	0.735	0.863	1.27	1.14
0.91	0.741	0.866	1.26	1.13
0.90	0.742	0.868	1.26	1.13
0.85	0.745	0.877	1.26	1.12
0.80	0.748	0.885	1.25	1.12
0.75	0.750	0.891	1.25	1.11
0.70	0.760	0.910	1.24	1.09

certainty and valid results about reliability, 1000 analyses are conducted. According to above descriptions, we would come to the same result even with more than 1000 analyses, since there is no significant modification in average failure load after 500 analyses. In these analyzes, both the material and geometrical nonlinearities were accounted for and the diagram of vertical load applied to the structure against the vertical displacement of the mid joint of the lower layer was derived. The obtained results from 1000 simulations are presented as load-displacement diagrams in Fig. 9. In these diagrams, the vertical and the horizontal axes represent force and displacement in terms of KN and millimeters, respectively.

If the first maximum points in Fig. 9 are considered as the collapse and capacity points of the structures, the statistical distribution of the collapse load and the respective statistical parameters can be achieved. The statistical parameters of the systems, such as the average and the maximum, minimum as well as the standard deviation of the collapse load, are summarized in Table 2. The red dashed lines represent the average of the load bearing capacity of the considered structures. For more clarification, the maximum, average, and minimum of GDB grid is demonstrated in Fig. 10.

According to Fig. 9 and Table 2, it is obvious that the GDB grid can carry more loads in comparison with the GDA grid. Furthermore, the average load that is carried by the GDB grid is 31 percent more than the GDA grid. As it can be seen, by decreasing the height to span ratio, increasing the load bearing capacity of the structures occurred. GDB have a much higher capacity compared to the GDA and when a member of the structure fails, the drop in the capacity of the system is quite low. Structural reliability is the probability that a system can desirably resist under a predetermined conditions. Reliability of a structure is commonly shown by "*R*" and is defined as *R*=1-*Pf*, in which *Pf* is the probability of the failure of the structure. The reliability of the structure, *R*, at a specific applied load, *Fs*, is the probability that the system collapse load, *F*, is greater than the specific load, *Fs*. This can be expressed as

$$R(\mathrm{Fs}) = \mathrm{p}\,(\mathrm{F} \succ \mathrm{Fs}) \tag{6}$$

By using this definition, the Reliability of selected structures is calculated and its results are introduced in Fig. 11 and Table 3. The second to fourth columns state the amount of strength reduction in each support condition because of the imperfections. It is worth noting that the perfect structure is a structure without any imperfection, but in reality, construction of such an ideal structure is impossible. Therefore, in the present work, the structure with the maximum load carrying capacity was considered as a perfect structure. The fifth to seventh columns shows the drop in capacity of the system caused by the existence of geometrical imperfections. A column labeled "coefficient load factor" has been added. In order to achieve the intended safety, the designer of the structure must multiply the required ultimate capacity of the system by the coefficient load factor from the last columns of the tables.

According to Fig. 11, the GDB system is more reliable than the GDA system. It is worth noting that the perfect structure is a structure without any imperfection, but in reality, construction of such an ideal structure is impossible. Therefore, in the present work, the structure with the maximum load carrying capacity was considered as a perfect structure. It can be seen from Fig. 11 and Table 3 that, for example, for designing a GDA grid and reliability of 0.99, the capacity of the imperfect structure is 33 percent less than the capacity of the perfect structure. Similarly, for reliability of 0.95, the capacity of the imperfect structure is about 75 percent of the capacity of the perfect structure. In general, existence of initial imperfections in the selected space structure reduced the capacity of the system from 12 percent through 35 percent. Therefore, this nonlinearity on capacity reduction should be considered in designing of these kinds of structures.

In addition to this, by knowing the maximum load bearing capacity of the structure, the designer can ensure the safety of the structure under any probable overload and if necessary, adopt the appropriate measures to increase the safety of the structure.

7. Conclusions

The aim of the present work is to study reliability of domes space structures under a random member's length imperfections. In this study, effects of random initial length imperfections on double layer domes grids have been investigated. The carried out investigations based on nonlinear finite element analyses in the OpenSees software and the Monte Carlo simulation method suggest the sensitivity and significant capacity drops of these structures due to random initial geometric imperfections. By deriving the reliability diagrams and by employing them in the design of the structure, the necessary design load to achieve the required safety can be easily determined. As a matter of fact, possessing such diagrams helps the designer to conduct his designs with the intended level of safety and without the need to perform detailed reliability analyzes for every specific design. It is worth noting that the results of this study are only applicable for double-layer flat space structures and using these results for other types of space structures is not recommended. Determining the reliability of other types of space structures requires further investigations in the same way.

Acknowledgments

This work is financially supported by Kermanshah Branch, Islamic Azad University. This support is gratefully acknowledged.

References

Affan, A. (1987), “Collapse of double layer space grid structures”, Ph.D. Dissertation, University of Cambridge, U.K.

Affan, A. and Calladine, C.R. (1989), “Initial bar tensions in pin jointed assemblies”, *J. Space Struct.*, **4**(1), 1-16.

AISC/ANSI 360-10 (2010), *Specification for Structural Steel Buildings*, Chicago, U.S.A.

Broggi, M. and Schuëller, G.I. (2011), “Efficient modeling of imperfections for buckling analysis of composite cylindrical shells”, *Eng. Struct.*, **33**(5), 1796-1806.

Cardoso, J.B., De Almeida, J.R., Dias, J.M. and Coelho, P.G. (2008), “Structural reliability analysis using Monte Carlo simulation and neural networks”, *Adv. Eng. Soft.*, **39**(6), 505-513.

Chen, J.J., Che, J.W., Sun, H.A., Ma, H.B. and Cui, M.T. (2002), “Probabilistic dynamic analysis of truss structures”, *Struct. Eng. Mech.*, **13**(2), 231-239.

De Paor, C., Cronin, K., James, P. and Kelliher, D. (2012), “Statistical characterization and modeling of random geometric imperfections in cylindrical shells”, *Thin-Wall. Struct.*, **58**, 9-17.

El-Sheikh, A.L. (1991), “The effect of composite action on the behavior of space structures”, Ph.D. Dissertation, University of Cambridge, U.K.

El-Sheikh, A.L. (1995), “Sensitivity of space trusses to member geometric imperfections”, *J. Space Struct.*, **10**(2), 89-98.

El-Sheikh, A.L. (1997), “Effect of member length imperfections on triple-layer space trusses”, *J. Eng. Struct.*, **19**(7), 540-550.

El-Sheikh, A.L. (2002), “Effect of geometric imperfections on single-layer barrel vaults”, *J. Space Struct.*, **17**(4), 271-283.

Fang, Y., Chen, J. and Tee, K.F. (2013), “Analysis of structural dynamic reliability based on the probability density evolution method”, *Struct. Eng. Mech.*, **45**(2), 201-209.

Gholizadeh, S., Gheyratmand, C. and Davoudi, H. (2016), “Optimal design of double layer barrel vaults considering nonlinear behavior”, *Struct. Eng. Mech.*, **58**(6), 1109-1126.

Hurtado, J.E. and Barbat, A.H. (1998), “Monte Carlo techniques in computational stochastic mechanics”, *Arch. Comput. Meth. Eng.*, **5**(1), 3-29.

Kala, Z. (2013), “Elastic lateral-torsional buckling of simply supported hot-rolled steel I-beams with random imperfections”, *Proc. Eng. J.*, **57**, 504-514.

Koutsourelakis, P.S., Pradlwarter, H.J. and Schuëller, G.I. (2004), “Reliability of structures in high dimensions, part I: Algorithms and applications”, *Prob. Eng. Mech.*, **19**(4), 409-417.

Lopez, R.H., Torii, A.J., Miguel, L.F.F. and Cursi, J.S. (2014), "An approach for the global reliability based design optimization of truss structures", *Proceedings of the 2nd International Symposium on Uncertainty Quantification and Stochastic Modeling*, Rouen, France.

Mazzoni, S., McKenna, F. and Fenves, G.L. (2005), "The opensees command language manual", Department of Civil and Environmental Engineering, University of California, Berkeley, U.S.A.

McKenna, F., Fenves, G.L. and Scott, M.H. (2010), "Open system for earthquake engineering simulation", Pacific Earthquake Engineering Research Center, University of California, Berkeley, U.S.A.

Melchers, R.E. (1999), *Structural Reliability Analysis and Prediction*, 2nd Edition, John Wiley & Sons.

Moghadasa, R.K. and Fadaeeb, M.J. (2012), "Reliability assessment of structures by Monte Carlo simulation and neural networks", *Asian J. Civil Eng.*, **13**(1), 79-88.

Mousavi, M.A., Abedi, K. and Chenaghlou, M. (2015), "Imperfection sensitivity analysis of double domes free form space structures", *J. Struct. Stab. Dyn.*, **15**(4), 1450067.

Nowak, A.S. and Collins, K.R. (2000), *Reliability of Structures*, McGraw Hill.

Papadrakakis, M. and Lagaros, N.D. (2002), "Reliability-based structural optimization using neural network and Monte Carlo simulation", *Comput. Meth. Appl. Mech. Eng.*, **191**(32), 3491-3507.

Roudsari, M.T. and Gordini, M. (2015), "Random imperfection effect on reliability of space structures with different supports", *Struct. Eng. Mech.*, **55**(3), 461-472.

Schenk, C.A. and Schuëller, G.I. (2005), "12 stability analysis of cylindrical shells with random imperfections", *Uncert. Assess. Large Fin. Elem. Syst.*, **24**, 81-109.

Schenk, C.A. and Schuëller, G.I. (2007), "Buckling analysis of cylindrical shells with cutouts including random boundary and geometric imperfections", *Comput. Meth. Appl. Mech. Eng.*, **196**(35), 3424-3434.

Schmidt, L.C. and Morgan, P.R. and Hanaor, A. (1982), "Ultimate load testing of space trusses", *J. Struct. Div.*, **180**(6), 1324-1335.

Schmidt, L.C., Morgan, P.R. and Hanaor, A. (1980), "Ultimate load behavior of a full scale space truss", *Proceedings of the Institution of Civil Engineering*, **69**(2), 97-109.

Schuëller, G.I. (1987), "A prospective study of materials based on stochastic methods", *Mater. Struct.*, **20**(4), 243-248.

Sheidaii, M.R. and Gordini, M. (2015), "Effect of random distribution of member length imperfection on collapse behavior and reliability of flat double-layer grid space structures", *Adv. Struct. Eng.*, **18**(9), 1475-1485.

The European Standard EN 10210-2 (2006), *Hot Finished Structural Hollow Sections of Non-Alloy and Fine Grain Steels*, U.K.

Thornton, C.H. and Lew, P. (1984), *Investigation of the Causes of Hartford Coliseum Collapse*, Elsevier Applied Science Publishers.

Torii, A.J., Lopez, R.H. and Biondini, F. (2012), "An approach to reliability-based shape and topology optimization of truss structures", *Eng. Optim.*, **44**(1), 37-53.

Vryzidis, I., Stefanou, G. and Papadopoulos, V. (2013), "Stochastic stability analysis of steel tubes with random initial imperfections", *Fin. Elem. Anal. Des. J.*, **77**, 31-39.

Wada, A. and Wang, Z. (1992), "Influence of uncertainties on mechanical behavior of a double-layer space truss", *J. Space Struct.*, **7**(3), 223-235.

Zhao, J., Zhang, Y. and Lin, Y. (2014), "Study on mid-height horizontal bracing forces considering random initial geometric imperfections", *J. Constr. Steel Res.*, **92**, 55-66.

Zhao, W., Qiu, Z. and Yang, Y. (2013), "An efficient response surface method considering the nonlinear trend of the actual limit state", *Struct. Eng. Mech.*, **47**(1), 45-58.

Zhou, Z., Wu, J. and Meng, S. (2014), "Influence of member geometric imperfection on geometrically nonlinear buckling and seismic performance of suspen-dome structures", *J. Struct. Stab. Dyn.*, **14**(3), 1350070.

TK