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Buckling capacity of radially compressed thin-walled reinforced cementitious spheres

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ABSTRACT

High performance cementitious materials have made possible the construction of slender thin cementitious shells. For their design, the most relevant recommendations are those provided by the International Association of Spatial Structures (IASS), which, however, are inconsistent with the modern European design framework. In the case of steel shells, Eurocode 3 provides a simplified design approach based on the well-known buckling capacity curves. The aim of this paper is the determination of the capacity curve of a radially compressed spherical cementitious shell, for different fabrication quality classes and the determination of the four buckling parameters that describe the curve.

1. Introduction

The term "high performance cementitious materials" refers to materials designed to have improved properties. Usually, such properties are achieved through special chemical processes and compositions, through special reinforcing patterns and types, or by a combination of the above. Examples of the latter category are the fiber reinforced cementitious materials and ferrocement, whose reinforcing pattern consists of small diameter meshes, while a special composition is usually used for the cementitious matrix. In more detail, ferrocement, first introduced by Joseph-Louis Lambot in 1848, is a type of reinforced concrete that uses as reinforcement multiple layers of small diameter and closely spaced steel rods, distributed along the thickness of the element [1], Moreover, its reinforcement ratio is relatively high, ranging between 1% and 4%. Mainly due to its reinforcing pattern, ferrocement is characterized by higher ductility and tensile strength than conventionally reinforced concrete and almost homogeneous-isotropic properties [2]. Its mechanical properties can be further improved by combining this special reinforcing pattern with high strength and/or fiber reinforced cementitious mortar, leading to better cracking control, significantly high capacity in tension and compression, as well as in reduction of the effect of creep. An example of such structure is the recently completed huge (100 × 100 m) canopy covering the "Stavros Niarchos Foundation Cultural Center (SNFCC)" in Athens, shown in Fig. 1. Other examples of the use of high performance cementitious materials are the various projects manufactured by ultra-high-performance fiber reinforced concrete, such as Ductal® [3]. Such high-performance materials have made possible the construction of thin cementitious shells, which are prone to buckling. Moreover, the advancements in the field of high performance cementitious materials have also caused a certain revival of interest in concrete shells in the relevant literature. Recent research works include experimental, numerical and analytical investigations on the determination of the buckling capacity of cementitious shells and the study of the impact of various parameters on the ultimate resistance, such as the imperfection pattern, the amplitude of imperfections and boundary conditions, taking into account the nonlinear material behavior [4–7].

Unlike steel shell structures, for which far more research has been done, research on concrete shells is concentrated mainly around the 70 s and 80 s. As a result, even the latest European regulations, such as Eurocode 2 (EN1992-1-1) [8], contain only general provisions for the design of concrete shell structures, employing materially and geometrically non-linear analysis, which is not easily performed by practicing engineers. However, a complete design methodology is not provided. Moreover, simplified approaches that can be used as reference solutions do not exist. The most relevant recommendations for the design of reinforced concrete shells and curved plates are those provided by the International Association of Spatial Structures (IASS) [9]. These recommendations provide the calculation of the buckling capacity, p_{cr}, of a shell, taking into account the effects of creep, plasticity, cracking, reinforcement and initial imperfections with the use of reduction factors over the elastic buckling strength of the shell. The recommended formula is mainly based on the works of Dulácska and Kollár [10,11]:

$$p_{cr} = p_{cr}^{lin} \rho_{imp} \rho_{creep} \rho_{crack} \rho_{pl} / \gamma, \tag{1}$$

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Fig. 1. The SNFCC with the 100 × 100 ferrocement canopy roof.

where in the present notation p_{cr}^{lin} is the classical critical load, ρ_{imp} is the imperfection sensitivity factor, ρ_{creep} expresses the reduction of the capacity due to creep, ρ_{crack} expresses the reduction due to cracking, ρ_{pl} is the plasticity factor and γ is the safety factor to convert the nominal capacity to a design one. It should be noted, that this safety factor is not constant, but varies according to the post-buckling behavior and the slenderness of the structure.

Although these recommendations provide the most relevant methodology for the design of concrete shells, certain facts lead to inconsistencies with the modern European framework. First, the lack of clear guidance regarding the measurement of construction imperfections may lead to serious mistakes regarding the implementation of the methodology, as the length of the imperfection is as important as its amplitude. Moreover, the use of a variable "global" safety factor creates further inconsistencies with the modern design concept of the Eurocodes, which use partial safety factors for loads, materials and buckling capacity. Finally, the lack of guidelines regarding the determination of the capacity by means of numerical analysis, leads to a somehow outdated regulatory framework.

On the other hand, in the case of steel structures, which are inherently susceptible to buckling, Eurocode 3 part 1-6 (EN1993-1-6) [12] includes a methodology for the design of shell constructions of different typical geometries. The design approach is based to a great extent on the notion of the general capacity curve, which can be seen as an extension of the classical buckling curves used for the design of steel beams and columns. More specifically, the capacity curve relates the dimensionless strength χ of the shell to the slenderness of the structure and is based on four parameters ($\overline{\lambda}_0$, α, β and η) that describe the possible failure modes of such shells. Based on the above, it seems more practical to follow a design procedure similar to that of EN1993-1-6, instead of applying the IASS recommendations. In this respect, it should be investigated if similar parameters and equations as those used for the design of steel shells can also be used to describe the failure modes of cementitious cells and be used for their design.

The purpose of this paper is the identification of the failure modes of a radially compressed reinforced cementitious spherical shell and the calculation of the corresponding capacity curve. The geometry of the sphere was selected due to the fact that it constitutes one of the reference cases in the relevant literature [13–16] as well as in the IASS recommendations [9]. The capacity curve is numerically obtained following the methodology of EN1993-1-6, while the material properties are based on the provisions of EN1992-1-1. The investigation initially focuses on determining a group of initial imperfection patterns which yield a safe estimate of the capacity in the elastic range. In the sequel, capacity curves for three different levels of initial imperfections are obtained by geometrically and materially nonlinear analyses of the imperfect structure (GMNIA), the computer aided design methodology found in EN1993-1-6 [12]. Finally, an attempt is made for the capacity curves to be approximated using the four aforementioned parameters ($\overline{\lambda}_0$, α, β and η). For the needs of this approximation, simplified equations for the determination of these parameters are proposed, based on the amplitude of the initial imperfections.

In addition, for the formulation of a complete design methodology, an on-site tolerance measurement method of the imperfections needs to be defined. The IASS recommendations fail to clearly define such a technique, while the amplitude of the accidental imperfections is derived by equations in the relevant literature, as well as by acceptable deviations agreed with the contractor. However, the measurement of initial imperfections needs to follow a certain methodology and to be related to a certain gauge length [12,17]. Notice that the length of the imperfection may be as critical as its amplitude. A large amplitude over a wide area may have less impact than a smaller one, over an area whose geometry is similar to natural buckling bulges calculated analytically and observed experimentally. This is also evident in the investigation presented later in this paper, regarding the impact of the imperfection length on the elastic buckling load of the imperfect sphere. Finally, the use of a measurement technique facilitates the categorization of the structures into fabrication quality classes, by setting certain limits to the allowable deviations.

In the present paper, a measurement method of initial imperfections for radially compressed cementitious spheres is proposed, in accordance to the methodology described in EN1993-1-6, through the use of the most adverse initial imperfections pattern. The structures are also categorized into fabrication tolerance quality classes, similar to those of EN1993-1-6, and a capacity curve is provided for each class.

2. Background to the design of steel shells according to EN1993-1-6

As the purpose of this paper is to provide a "proof of concept" that the design of a cementitious spherical shell under radial compression can be treated under the capacity curve methodology of EN1993-1-6, it seems essential to provide the basic background of the corresponding framework. As aforementioned, the design approach proposed by Eurocode in the case of steel shells is based on a capacity curve that describes the relationship between the dimensionless strength χ of the shell to its slenderness $\overline{\lambda}$ and in its simplest form is defined by four parameters, as shown in the following. The nonlinear characteristic buckling resistance R_k can be obtained as

$$R_k = \chi R_{pl}, \tag{2}$$

where R_{pl} is the plastic resistance of the shell and χ is given as:

$$\chi = 1 \quad \text{when} \quad \overline{\lambda} \leqslant \overline{\lambda}_0 \tag{3}$$