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Spherical Dome Buckling With Edge Ring Support

J. Michael Rotter [a,*], Greig Mackenzie [b], Martin Lee [c]

[a] Institute for Infrastructure and Environment, University of Edinburgh, UK
[b] Laing O'Rourke, Glasgow, UK
[c] Jacobs UK Ltd, Glasgow, UK

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ABSTRACT

Spherical dome roofs are widely used on large diameter tanks and silos, as they provide high strength for very limited amounts of material: buckling normally controls the design. Their design is currently based on very simple rules that were devised many decades ago. In particular, existing design is based on very simple boundary condition assumptions that cannot be realised in practice. This paper presents the first thorough study of the influence of realistic boundary conditions, in the form of a ring at the eaves, on the linear bifurcation of these domes. The outcome is a clear documentation of one of the key reference resistances required for a design description in the terminology of the Eurocode on shell buckling EN 1993-1-6. The complete study explored a very wide range of parameters, so only a sample of the findings is presented here. The complete results are expected to form the basis of future rules in the ECCS Recommendations on Buckling of Shells.

1. Introduction

Spherical dome roofs are widely used on large diameter tanks and silos, as they provide high strength using very limited amounts of material. The structural design of a spherical dome roof against buckling is an important feature of the design of such tanks. Because the dome can be very thin, it is particularly susceptible to elastic buckling failure under internal partial vacuum. The very simple rules that govern the design of these roofs in more traditional standards [1,2] are very old and predate modern computational assessments.

More recent studies [3,4] have been incorporated into the 5th Edition of the European Design Recommendations on the Buckling of Shells [5], known for short as EDR5. These now permit buckling calculations to be reliably performed for a wide range of dome geometries. They have recently been accepted as an amendment [6,7] to the Eurocode on shell buckling EN 1993-1-6 [8] using the method of Reference Resistance Design [9]. However, it is unfortunate that these rules only address the buckling problem where very simple boundary or support conditions are used at the dome edge. The only three that can potentially be applied to the practical design of a tank are the support free to slide radially; radially restrained but pinned; and fixed (Fig. 1). These three principal support conditions lead to very different stress resultant magnitudes in the dome, and may consequently produce very different buckling resistances [2,3]. The above design rules make huge distinctions in the resistance assessment according to which support condition is chosen, so there is considerable scope for improvement if the support conditions can be treated with a little more realism.

2. Background

2.1. Dome roofs under external pressure

Dome roofs on constructed tanks are almost always constructed with an eaves ring (Fig. 2). This ring has the function of resisting the radial outward thrust from the dome, and is therefore normally in tension under loading from external pressure and self-weight. The sizing of the ring is determined by the tensile stress that is believed to develop in it. Its size consequently does not relate directly to the peak membrane compressive stresses in the dome itself, and the sizing of the ring is not related directly to the buckling resistance of the resulting dome. The ring has a finite stiffness (providing radial restraint to the dome through the circumferential stretching stiffness of the ring), and it places the membrane stress pattern in the dome somewhere between those of radially sliding and pinned supports, with some rotational restraint. The peak membrane stresses in the dome depend significantly on the size of the eaves ring, and this should consequently strongly affect the buckling resistance.

* Corresponding author.
E-mail addresses: m.rotter@ed.ac.uk (J.M. Rotter), greigmackenzie@gmail.com (G. Mackenzie), marty23lee@gmail.com (M. Lee).

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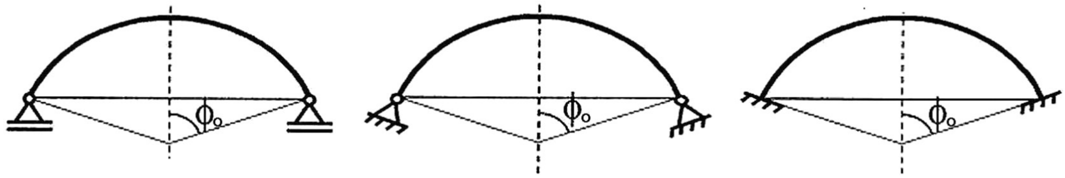

Fig. 1. Simple support conditions used to describe the buckling of spherical domes.

The design recommendations referred to above only consider three boundary conditions that have practical relevance for a dome roof: rollers, radially restrained but pinned, and fixed, as shown in Fig. 1. The same conditions and images are currently used in both the ECCS Recommendations [5] and the new Amendments to EN 1993-1-6 [6,7]. These three boundary or support conditions lead to different stress resultant patterns in the dome, and to remarkably different predicted buckling resistances [5]. Real dome shells are always constructed with an eaves ring, edge ring, curb angle or wind girder (Fig. 2), where different terminology is used by different authors [e.g. 1]. This ring has a finite stiffness against radial displacements, generally placing the stress pattern and buckling strength between that of the roller and pinned supports.

The radial restraint provided by the eaves ring must surely make for transitions between the widely different outcomes of the current simplified boundary conditions.

There is an additional important aspect of this study, which is of more general interest in the field of shell buckling. The reason why the boundary conditions are so important is that they strongly influence the local membrane stresses in the region where buckling may occur. One of the major debates in shell buckling is the question of which local stress state should be used to assess buckling when the membrane stresses are not uniform, and this study contributes usefully to that discussion.

The key geometrical parameters are the spherical radius to thickness ratio R/t and the subtended angle from the apex to the support ϕ_o.

In line with the design recommendations noted above, this study is confined to spherical dome shells under uniform external pressure. This load case is directly relevant to the key condition of an induced partial internal vacuum, but is also closely related to self-weight, snow and wind loading. However, there appear to be no documented cases of failures of spherical dome roofs, so the practical criteria for design do not appear to have been put to the test in practice.

2.2. The current design approach of EN 1993-1-6 and ECCS EDR5

The best design rules for the buckling of shells at the present time are those given in the Eurocode on shell structures EN 1993-1-6 [8], which are reproduced and explained in detail in the European Recommendations [5]. For computational assessments of general conditions in shells, these documents set out two alternative procedures. The comprehensive procedure of a geometrically and materially nonlinear analysis with explicitly included modelling of imperfections (GMNIA) is very onerous and requires very extensive computation, particularly because many shell buckling strengths are acutely sensitive to small imperfections. The second, and much more design friendly procedure, termed LBA–MNA, requires a computational assessment only of the linear elastic eigenvalue (LBA) together with a determination of the plastic limit load (MNA).

According to the EN 1993-1-6 [8] rules, any practical application of GMNIA must include an LBA and an MNA analysis to ensure that the interpretation of the nonlinear analysis is correct. A fuller description and detailed discussion of the LBA–MNA design process may be found in [10]. All buckling resistances are now related to these two reference resistances. Linear bifurcation analyses were conducted here because this is the base case used for design by the Eurocode EN 1993-1-6 [8], and any new problem must be characterised in this way before the effects of geometric nonlinearity and geometric imperfections are introduced. It is consequently very important to establish the linear bifurcation strength (LBA) for dome shells, as is done in this paper. The second reference condition (MNA) is less significant here because these thin shells lie in the range where elastic buckling controls.

The design procedure used in EDR5 [1,2] for the buckling of spherical domes relates the elastic buckling resistance to the classical external pressure to cause buckling of a complete sphere, determined by the original study of Zoelly [11] in 1915. This classical pressure is given by

$$p_{cl} = \frac{2}{\sqrt{3(1-\nu^2)}} \cdot E \cdot \left(\frac{t}{R}\right)^2 \tag{1}$$

in which p_{cl} is the classical elastic critical external pressure at buckling, E and ν are Young's modulus and Poisson's ratio, R is the radius of the sphere and t is the thickness. When other boundary conditions were explored [3,4], it was found that this equation could closely model most situations with the introduction of an addition coefficient C_c, which varies according to the particular boundary condition. The resulting

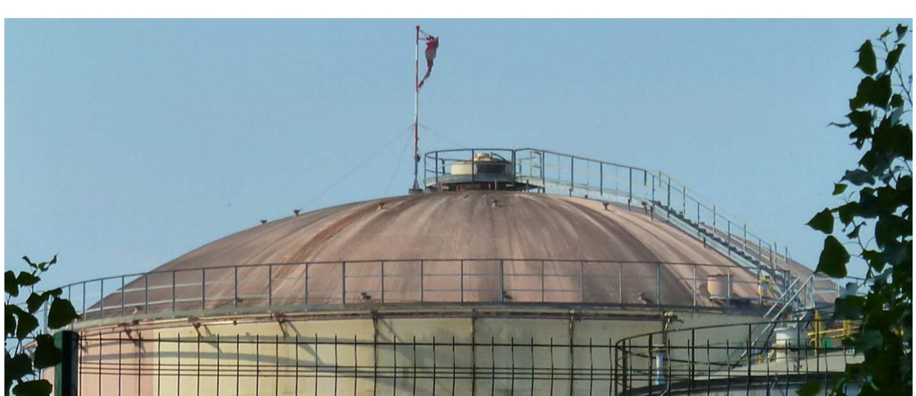

Fig. 2. Typical spherical dome roof on a storage tank.