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RENOVATING FUSEE CERAMIQUE VAULTS

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KEYWORDS

Fusée Céramique Vaults, Strengthening, Stiffening, Buckling Risk

ABSTRACT

Just before WW2 the French architect Jacques Couëlle invented a system to build structures, using ceramic tubes embedded in concrete, known as Fusée Céramique. During WW2 this system was used in France and Germany to construct barracks and shelters. After the war this system was applied in France, North Africa and The Netherlands to construct low-rise cylindrical vaults and domes for workshops, swimming halls, stations, schools and churches. Thanks to the ceramic infill the need of material and the dead weight of the vaults was minimal. For example in the city of Woerden, The Netherlands, a cylindrical low-rise vault was built with a span of 19,8 m, a thickness of 135 mm and a ratio thickness-span of 1:147. In 2012 this workshop was pulled down. During the sixties the costs of labour were rising and this system could not compete with other systems. Fifty years later most of these structures are pulled down. The remaining buildings do not meet the demands of the present concerning comfort, safety, insulation and need of energy. Nevertheless these structures are a fine example of engineering, so at least some of these vaults have to be preserved for the coming generations. In the past the low-rise vaults were designed for the permanent load and a modest live load. Unfortunately due to temperature variations and time dependent deformations the Fusée Céramique roofs are subjected to internal forces, which can cause cracks and reduce the stiffness and load bearing capacity substantially. Consequently some vaults are not safe and have to be strengthened.

This paper describes for cylindrical Fusée Céramique vaults, composed of concrete and ceramics, the effect of the time dependent deformations concerning strength, stiffness, buckling risk and bearing capacity. To strengthen these vaults a cost-effective method is described, using the potentiality of the hangers, connecting the ties with the vaults, to reduce deformations and buckling length. Possibly this method can be used to strengthen and stiffen arches and vaults of concrete, masonry, steel and timber too.

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INTRODUCTION

Just before WW2 the French architect Jacques Couëlle invented a system to build structures, using ceramic tubes embedded in concrete vaults, known as Fusée Céramique, to reduce the dead weight and save materials. In 1954 this system was introduced in the Netherlands [1] to construct low-rise vaults and domes. During the sixties the costs of labour were rising and this system could not compete with other systems. Fifty years later most of these structures are pulled down. The remaining buildings do not meet the demands of the present concerning comfort, safety, insulation and need of energy. Nevertheless these structures are a fine example of engineering, so at least some of these vaults have to be preserved for the coming generations. Due to temperature variations and time dependent deformations Fusée Céramique structures are subjected to internal forces, which can cause cracks and reduce the stiffness and load bearing capacity substantially [2]. Probably some remaining Fusée Céramique vaults are not safe and have to be strengthened. This paper describes for these vaults a cost-effective method to increase the buckling resistance and load bearing capacity.

Figure 1: Low-rise Fusée Céramique vault, Woerden, The Netherlands [2].

STRUCTURAL DESIGN

A Fusée Céramique element is a tube with a conical top, which can be shoven into the open rear of the next element. The elements have a length of 350 mm, an outer diameter of 80 mm and a thickness of 10 mm, see figure 2.

Figure 2: Fusée Céramique element used in Woerden, The Netherlands.

The roofs were made on the site. A thin layer of concrete of 25 mm was poured on the mould, The fusées were pushed into the concrete and a second layer was poured on top of the first layer. In a section with a width of 1,0 m eleven elements were placed with a spacing of 10 mm and a centre to centre distance of 90 mm, see figure 3.

Generally the low rise cylindrical vaults were designed with a ratio rise to the span of 1:8 [3]. Low-rise vaults with a span smaller than 15 m were constructed with one layer of fusées and a thickness of 110 mm. For vaults with a span larger than 15 m the thickness had to be increased. So was for a vault in Woerden, with a span of 19,8 m, the thickness increased to 135 mm [2].

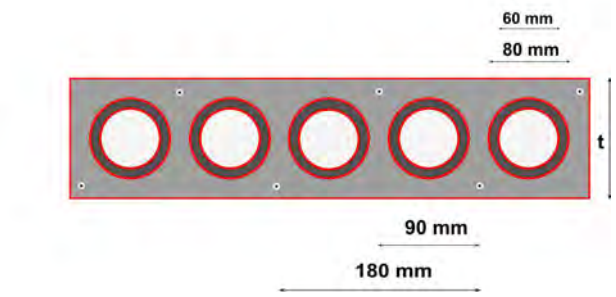

Figure 3: section of a fusée vault perpendicular to the span.

In the past the low rise vaults were designed according to the Theory of Elasticity [3]. The effect of cracks was neglected. The vaults were assumed to be subjected to an equally distributed permanent load q_g and live load q_e acting symmetrically or asymmetrically at one side, see figure 4.

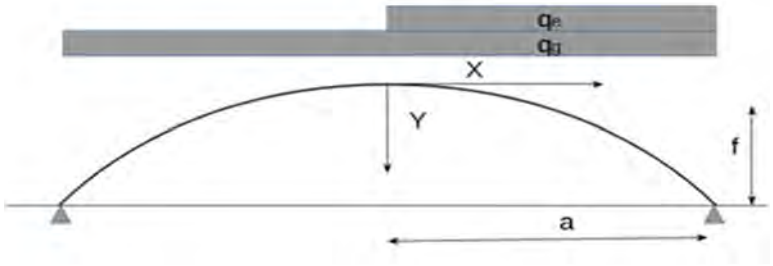

Figure 4: Vault subjected to symmetrical permanent load and asymmetrical live load.

The effect of the stiffness will be shown for a cylindrical vault, following a parabola, with a span of 14,4 m, a thickness of 110 mm and a rise of 1,8 m. The centre of the coordinates is positioned at the crown. To resist the bending moments due to asymmetrical live load this vault was reinforced with bars Ø6 – 180 in the top and bottom. In practice these vaults were not reinforced with distribution bars. Table 1 shows the area and second moment of the area of the Fusées, concrete and steel reinforcement.

Area of the fusées:	$A_f = 11 \times ¼\ \pi \times (80^2 - 60^2) =$	$24{,}2 \times 10^3$ mm^2
Area of the concrete:	$A_c = 1000 \times 110 - 11 \times ¼\ \pi \times 80^2 =$	$54{,}7 \times 10^3$ mm^2
Area of the re-bars 2Ø6-180:	$A_s = 2 \times ¼\ \pi \times Ø6^2 \times 1000/180 =$	314 mm^2
Second moment of the area, fusees:	$I_f = 11 \times \pi \times (80^4 - 60^4)\ /64 =$	$15{,}1 \times 10^6$ mm^4
Second moment of the area, concrete;	$I_c = 1000 \times 110^3/12 - 11 \times \pi \times 80^4\ /64 =$	$88{,}8 \times 10^6$ mm^4
Second moment of the area, steel:	$I_s = 2 \times 157 \times (½ \times 110 - 15 - ½ \times 6)^2 =$	$0{,}43 \times 10^6$ mm^4

Table 1: Area and second moment of the area of the Fusées, concrete and steel for a width of 1,0 m.

The stiffness of the fusées, concrete and reinforcement was assumed to be respectively: E_f = 17000 MPa, E_c = 21000 MPa and $E_s = 2{,}1\times10^5$ MPa. The stiffness of the vault with a with of 1,0 m was calculated with $EA = \Sigma E_i A_i = 1{,}63\times10^9$ N and $EI = \Sigma E_i I_i = 2{,}2\times10^{12}$ Nmm^2.

For an equally distributed load q the thrust follows from: $H = ½\ q \times a^2/f$. For an asymmetrical live load q_e the thrust follows from: $H = ¼\ q_e \times a^2/f$. At a distance x from the top the normal force follows from: $N_x = (H^2 + V_x^2)^{0.5}$. The permanent load due to the dead load and finishing is equal to p_g = 2,2 kN/m^2. According to the TGB 1955 the live load is assumed to be p_e = 0,5 kN/m^2. Table 2 shows the forces due to the permanent load, a symmetrical and an asymmetrical live load at a distance x = ½ a from the top.

		Sym.			Asym.	
		perm. load	live load	perm. + live load	live load	perm. + live load
shear force:	V	7,9 kN	1,8 kN	9,7 kN	0,9 kN	8,8 kN
thrust:	H	31,7 kN	7,2 kN	38,9 kN	3,6 kN	35,3 kN
normal force:	N	32,7 kN	7,4 kN	40,1 kN	3,7 kN	36,4 kN

Table 2: Resulting forces conform the Theory of Elasticity, for t = 0, x = ½ a

The critical buckling load was calculated with the well known expression given by Euler: $N_{cr} = \pi^2\ EI/(\psi\ s)^2$. With s the length of the vault from the top to the support. For asymmetrical buckling ψ = 1. For a parabolic vault with a ratio f/*l* = 1:8 the length of the vault from the crown to support s = 1,04×a. Substituting the stiffness and length gives: N_{cr} = 387,3 kN. The ratio buckling force with respect to the normal forcen follows from: $n = N_{cr}/N_d$. For the permanent and asymmetrical live load: $n = N_{cr}/N_d = 387{,}3/36{,}4 = 10{,}6$, thus the effect of the second order is small.

The vault is subjected to bending moments in case the live load acts asymmetrically at one side. The bending moment due to this load q_e is equal to: $M_o = q_e \times a^2/16 = 1{,}62$ kNm. For x = ½ a the concrete stresses due to the permanent and asymmetrical live load, including second order, are quite small: σ_c = - 0,48 +/- 0,85 × 10,6/(10,6 - 1) MPa.

Actually the stresses are effected due to the time dependent deformations. Possibly the vault is cracked and these cracks can reduce the stiffness substantially.

INSTANTANEOUS AND TIME DEPENDENT DEFORMATIONS

The normal load acting at a section of the vault is resisted by the concrete, fusées and reinforcement. Due to a compressive load N the instantaneous specific deformation of the concrete, fusées and reinforcement is $\varepsilon_0 = N/EA$. The forces in the concrete, fusées and reinforcement follow from: $N_c = \varepsilon_0\ E_cA_c$; $N_f = \varepsilon_0\ E_fA_f$; $N_s = \varepsilon_0\ E_sA_s$. For the permanent load q_g = 2,2 kN/m the average value of the normal force: N = 32,7 kN. For t = 0 the specific deformation: $\varepsilon_0 = N/AE = 32700/1{,}63\times10^9 = 2{,}0\times10^{-5}$. Table 3 shows for the fusées, concrete and reinforcement the distribution of the permanent load for time t = 0.

$\sigma_c = 0.020\times10^{-3}\times 2.1\times10^4 =$	0.42 MPa	$N_c = 0.42\times54{,}7\times10^3 =$	$23{,}0\times10^3$ N
$\sigma_f = 0.020\times10^{-3}\times 1.7\times10^4 =$	0.34 MPa	$N_f = 0.34\times24{,}2\times10^3 =$	$8{,}2\times10^3$ N
$\sigma_s = 0.020\times10^{-3}\times 2.1\times10^5 =$	4.20 MPa	$N_s = 4{,}20\times 314 =$	$1{,}3\times10^3$ N

Table 3: Stresses and forces due to the permanent load for time t = 0

The distribution of the loads will change due to the creep and shrinkage of the concrete. Due to shrinkage and creep the specific deformation of the concrete will rise during

time t with $\varepsilon_{sc} + \phi\, \varepsilon_0$. The total time dependent specific deformation during time t is equal to: $\varepsilon_{sc} + \varepsilon_0 \times (1+ \phi\, \varepsilon_0)$. The concrete is firmly attached to the fusées and reinforcement. The fusées and reinforcement will prevent partly the time dependent deformation of the concrete. The concrete, fusées and reinforcement are subjected to inner forces F_c , F_f and F_s. These forces are in balance, thus: $F_c = F_f + F_s$.

Due to the internal forces the composite structure is subjected to a specific deformation $\Delta\varepsilon$. Due to the internal force F_c the specific deformation of the concrete is decreased by F_c/A_cE_c. During the time t this specific deformation increases by creep with F_c k ϕ/A_cE_c. The force F_c is not constant but is increasing during the time t, the factor k compensates for the time dependency of this force. Scherpbier [4] showed that this factor k = ½. The specific deformation due to the internal force F_c including the creep is: $F_c\,(1+ k\, \phi)/AE_c$.

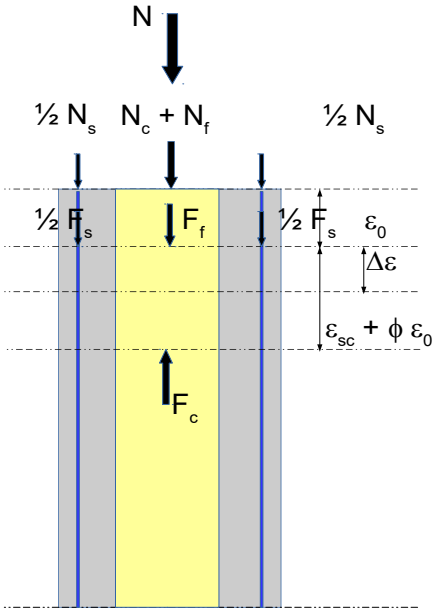

Figure 5: Deformations due to shrinkage and creep and the compensating forces for the vault composed of concrete, steel and Fusées.

The specific deformation $\Delta\varepsilon_\xi$ and the internal force F_x for the concrete, fusées and reinforcement follows from:

$$\Delta_c = \frac{\varepsilon_{sc} + \phi\, \varepsilon_0 - F_c\,(1+ k\, \phi)}{A_c\, E_c\, A_c} \qquad F_c = \frac{E_c\, A_c}{(1+ k\, \phi)} \times [\, \varepsilon_{sc} + \phi\, \varepsilon_0 - F_s/E_sA_s\,]$$

$$\Delta_f = F_f\,/E_f\,A_f\,; \qquad F_f = F_s\, E_f\, A_f / E_sA_s$$

$$\Delta_s = F_s\,/E_s\,A_s \qquad F_s = \frac{[\varepsilon_{sc} + \phi\, \varepsilon_0\,] \times E_c\, A_c\,/(1+ k\, \phi)}{1 + [A_f E_f + E_c\, A_c\,/(1+ k\, \phi)]\,/E_s\, A_s}$$

Table 4 shows the stresses and forces due to the time dependent deformations for the permanent load, calculated for a specific deformation of the concrete due to shrinkage $\varepsilon_{sc} = 0.4 \times 10^{-3}$ and a creep coefficient $\phi = 3$. The concrete is tensioned, probably the

concrete will be cracked. Due to the cracks the stiffness of the vault will be much smaller than assumed before.

	Normal force	Stress: $\sigma_x = N_x / A_x$
concrete	$-N_c + F_c = -23{,}0 + 107{,}9 = +\ 84{,}9$ kN	$\sigma_c = +\ 84900/54700 = +\ 1{,}55$ MPa
fusées	$-N_f - F_f = -\ 8{,}2 - 93{,}0 = -\ 101{,}2$ kN	$\sigma_f = -\ 101200/24200 = -\ 4{,}2$ MPa
reinforcement	$-N_s - F_s = -\ 1{,}3 - 14{,}9 = -\ 16{,}2$ kN	$\sigma_s = -\ 16200 / 314 = -\ 51{,}6$ MPa

Table 4: Stresses and normal forces due to the permanent load for time t.

ULTIMATE STATE

Nowadays structures are designed in the ultimate state to resist design loads, including the safety factors. For the given vault, described previously, the maximal design load: $q_d = 1{,}2 \times 2{,}2 + 1{,}5 \times 0{,}5 = 3{,}4$ kN/m. Due to the permanent load and asymmetrical live load the vault is subjected to a bending moment $M_d = 1{,}5 \times 1{,}62 = 2{,}43$ kNm and normal force $N_d = 1{,}2 \times 32{,}7 + 1{,}5 \times 3{,}7 = 44{,}8$ kN. For the ultimate state the stiffness is defined with a MN-κ-diagram [5] for the ultimate design load: $N_d = 44{,}8$ kN, see figure 6. For $M_d = 2{,}43$ kNm the stiffness $EI = 0{,}32 \times 10^{12}$ Nmm^2.

Figure 6: MN-κ diagram, ultimate state for a normal force $N_d = 44{,}8$ kN.

The buckling force: $N_{cr} = \pi^2 \times 0{,}32 \times 10^{12/} / (1{,}04 \times 7200)^2 = 56{,}3 \times 10^3$ N. The ratio buckling force with respect to the normal force is very small: $n = N_{cr}/N_d = 56{,}3/44{,}8 = 1{,}3$. Due to the second order the bending moment increases much: $M_d = 2{,}43 \times 1{,}3/(1{,}3\text{-}1) = 10{,}5$ kNm. This bending moment, including the second order, is larger than the ultimate bending moment $M_u = 4{,}9$ kNm, so the structure is unsafe and has to be strengthened. Possibly hangers between the ties and vault can reduce the effect of the second order. Especially for renovations this can be very cost-effective. The potentiality of the hangers to reduce the effect of the second order will be studied for the described vault.

TIES AND HANGERS

Three or more hangers, connecting the tie with the vault, can reduce the buckling length substantially, provided the slender hangers do not buckle. Palkowski [6] researched for bridges the increase of critical buckling load in case all hangers are tensioned continuously due to the load acting on the deck. For a structure with three hangers the buckling length follows from: $l_{buc} = \psi\ s = [1\text{-}\cos\ \phi]^{1/2} \times s \geq ½\ s$. For an arch or vault with

f/a = ¼ and cos ϕ = 0,8944: ψ = (1-0.8944)$^{1/2}$ = 0,32. Due to the hangers the factor ψ is smaller than ½, then the buckling length l_{buc} = ½ s is decisive.

The hangers of a bridge with the deck hanging on the arches, are tensioned contimuously.However the hangers of a vault, supporting the ties, are not tensioned continuously. Possibly some hangers are compressed if the vault is loaded asymmetrically. Slender hangers cannot resist compressive forces, so the buckling of the vault is only restricted by the tensioned hangers. Consequently the critical buckling length is not reduced much [2]. Curving the ties upward, by shortening the hangers, will tension all hangers and reduce the buckling length of a vault substantially. However a convex tie will decrease the lever arm amd increase the thrust, so the camber must be small, just enough to tension the hangers continuously. For the described vault the effect of the convex tie will be shown.

Assume the coordinates of the tie are halfway the span and at a quarter of the span respectively c×f and ¾ c×f. Due to the curvature of the tie, the rise of the vault decreases with f×(1-c). For a vault subjected to an equally distributed load q the thrust increases with a factor 1/(1-c): H = ½ q a × (a/f)/(1-c). The force S, acting at the hangers, follows from the equilibrium of the bending moments for the tie halfway the span and at a quarter of the span: S = H×c×f/a. For the vault the bending moments are for x = 0 and x = ½ a equal to zero. So the forces S do not increase the bending moment.

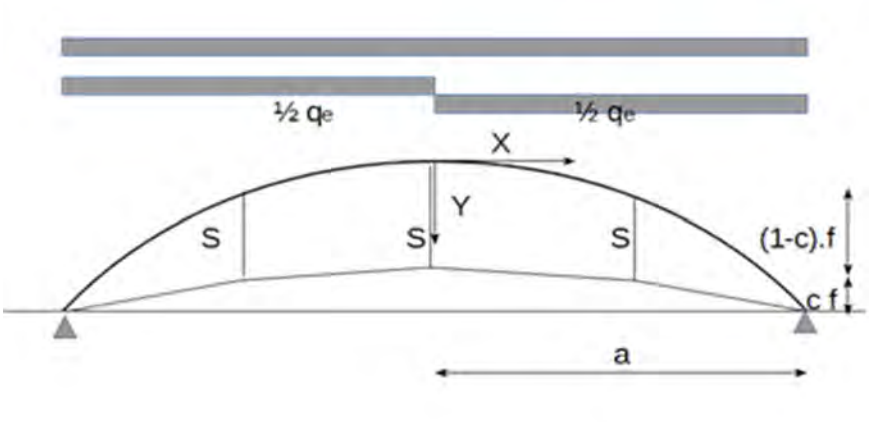

Figure 7: Vault with convex tie subjected to a symmetrical and anti-metrical load

An asymmetrical load q_e acting at half of the vault can be splitted into a symmetrical equally distributed load ½ q_e and an anti-metrical load ½ q_e , see figure 7. The thrust H due to the anti-metrical load ½ q_e is zero, so the force S is not effected by anti-metrical load. For x = ½ a the bending moment is still equal to: $M_{x = a/2}$ = $q_e \times a^2/16$.

		Sym.			Asym.	
		perm. load	live load	perm. + live load	live load	perm. + live load
shear force, x = ½ a	V	8,8 kN	1,9 kN	10,7 kN	1,0 kN	9,8 kN
thrust:	H	33,4 kN	7,6 kN	41,0 kN	3,8 kN	37,2 kN
hanger	S	0,4 kN	0,1 kN	0,5 kN	0,1 kN	0,5 kN
normal force:	N	34,5 kN	7,8 kN	42,3 kN	3,9 kN	38,4 kN

Table 5: Resulting forces for the vault with convex tie with c = 0,05.

Table 5 shows the results for a vault with convex ties and c = 0,05 for x = ½ a. Comparing the results of table 2 and table 5 shows that the normal loads do not increase much due to the convex tie.

Due to the permanent load and asymmetrical live load the vault is subjected to a normal force and bending moment of respectively N_d = 1,2×34,5 + 1,5×3,9 = 47,3 kN and M_d = 1,5 × 1,62 = 2,43 kNm.

For a buckling length l_c = ½ × 1,04 × 7,2 m the buckling force N_{cr} = 225,3 × 10^3 N. The ratio buckling force with respect to the normal force n = N_{cr}/N_d = 225,3/47,3 = 4,8. Due to the second order the bending moment is increased slightly: M_d = 2,43×4,8/(4,8 -1) = 3,1 kNm. This moment is smaller than the ultimate bending moment: M_u = 4,2 kNm. This vault with convex tie can transfer the ultimate design loads safely.

CONCLUSIONS

The time dependent deformations can reduce the stiffness of Fusée Céramique vaults composed of fusées, reinforcement and concrete much. Possibly slender vaults are not safe concerning the buckling risk and have to be strengthened. Shortening the hangers will curve the ties upward. The convex ties will tension the hangers continuously. Tensioned hangers reduce the buckling length of the vault substantially. However shortening the hangers will increase the normal forces, so it is advisable to curve the ties slightly, just enough to tension the hangers continously.

Possibly the described method to strengthen the Fusée Céramique vaults can be helpfully to preserve these slender vaults for the coming generations. Furthermore this method can be also used to design arches and cylindrical vaults efficiently, to save materials, to reduce the embodied energy and to decrease the emissions of greenhouse gasses.

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