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SEISMIC EVALUATION OF UNANCHORED CYLINDRICAL TANKS

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SUMMARY

The seismic resistance of seven existing unanchored cylindrical oil storage tanks was calculated according to Eurocode (EC) 8, part 4, Appendix A [1]. Five of these tanks, with H/R ratios (H = height of liquid, R = radius of tank) of 1.12, 1.14, 1.75, 2.16 and 3.48, were also calculated with the non-linear pushover analysis proposed by Malhotra [2]. All tanks are located in Switzerland. None of them was designed to resist earthquakes.

EC 8 limits the cyclic plastic rotation in the base plate to a maximum value of 0.2 radians (11.5 degrees). In all cases, this constraint turned out to be more stringent than elephant-footing or elastic buckling of the shell.

Very different results were obtained by the two methods of calculation. The EC 8 results show a strong correlation between H/R ratio and plastic rotation and nearly no correlation between tank volume and plastic rotation. Only a very weak H/R influence on plastic rotation can be seen in the results from the pushover analysis. However, plastic rotation increases linearly with tank volume, irrespective of the H/R ratio (with the exception of the very slender tank with H/R = 3.48).

For moderate to high H/R ratios (H/R ≥ 1. 75), EC 8 leads to plastic rotations at least twice as high as those obtained by the non-linear pushover analysis. The opposite is true for the low H/R ratios (H/R = 1.12 and 1.14). Here, pushover analysis leads to about 1.5 times the plastic rotation of the EC 8 calculation. Possible reasons for these discrepancies are discussed and tentative recommendations for practitioners are given.

INTRODUCTION

Ground-supported cylindrical tanks are used to store a variety of liquids — water for drinking and fire-fighting, crude oil, wine, liquefied natural gas (LNG), etc. Failure of tanks, following destructive earthquakes, may lead to environmental hazard, loss of valuable contents, and disruption of fire-fighting effort. Inadequately designed or detailed tanks have suffered extensive damage in past earthquakes and

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have resulted in disastrous effects (Hanson [3], USDOC [4], Gates [5], Haroun [6], Manos and Clough [7], EERI [8], Brown et al. [9], Lund [10], Cooper [11]).

Earthquake damage to steel tanks can take several forms. Large axial compressive stresses due to beam-like bending of the tank wall can cause "elephant-foot" buckling of the wall. Sloshing liquid near the free-surface can damage the roof and upper shell of tank. High stresses in the vicinity of poorly detailed base anchors can rupture the tank wall (Miles [12]). Base shears can overcome friction causing the tank to slide. Base uplifting can: (1) damage the piping connections that are incapable of accommodating vertical displacements, (2) rupture the base plate-mantle junction due to excessive joint stresses, and (3) cause uneven settlement of the foundation.

The objective of this paper is to evaluate the simplified seismic calculations of unanchored cylindrical liquid storage tanks according to Eurocode (EC) 8, part 4, Appendix A (1998) [1], henceforth referred to as "according to EC 8". The paper is addressed primarily to practitioners. In the sense of a generic study, the results of calculations according to EC 8 are compared with results of a pushover analysis by Malhotra [2]. The present paper does not contain any new development of methods of calculation of tanks, nor does it give in-depth explanation of the applied methods. The reader is referred to the aforementioned original references; only an overview is given here.

METHODS OF CALCULATION

EC 8, Part 4, Appendix A

The impulsive and convective natural periods, masses and effective heights are calculated for an anchored tank, following the simplified procedure for fixed-base cylindrical tanks by Malhotra [13] (EC 8, Appendix A.3.2.1 [1]). If applicable, the impulsive natural period and damping are increased by taking into account the inertial soil-structure interaction (SSI) effects. Appendix A.7 gives the corresponding formulas. With these increased values, the overturning moments above and below the base plate are calculated, again according to Appendix A.3.2.1. Finally, three graphs in Appendix A.8 are used to calculate the uplift width, uplift height and axial compressive stress in the tank wall as functions of normalized overturning moment (Figures A10, A11 and A12 in EC 8 [1]). These figures are based on purely static finite element analyses of Scharf [14]. Therefore, they do not account for an increase in the fundamental natural period of the impulsive mode due to uplifting, which, for most practical cases, leads to a significant decrease in the overturning moment. Figure A11 is reproduced below as Figure 1. It can be seen that the uplift height is strongly sensitive to the slenderness ratio H/R.

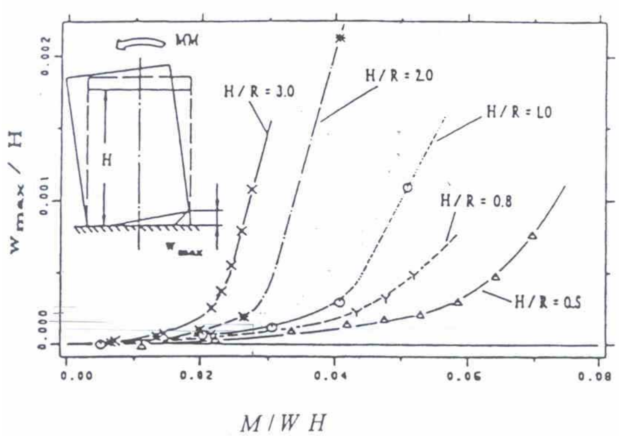

Figure 1: Maximum uplift height versus overturning moment M/WH (M: moment; W: total liquid weight; H: liquid height); Figure A11 of EC8, part 4.

The plastic rotation in the base plate is estimated from the uplift height and uplift width.

Two remarks must be made. First, the EC 8, Appendix A.8 allows an increase in the fundamental natural period (thus a decrease in the overturning moment) due to uplifting, according to Fischer et al. [15]. However, this is restricted to the range of parameter values for which design charts are available in [15], and the user has to refer to the original publication; no formulas or graphs are given in EC 8. This modification of the natural period has not been taken into account in the results “according to EC 8“ presented in this paper.

Second, EC 8, Appendix A.7, does not mention any reduction of the effective seismic load due to SSI. However, according to Wolf [16], among others, SSI can lead to a significant reduction of the effective seismic excitation. This additional “beneficial“ effect of SSI was not taken into account in the results presented here. However, the conclusions of this present paper would not have changed if this effect had been considered in the calculations “according to EC 8” (see Résonance [17]).

Nonlinear Pushover Analysis

Malhotra [2] developed a simplified nonlinear pushover analysis for tanks. The analysis is based on the concept of equivalent-linear system. It is similar to the nonlinear static procedure (NSP) for buildings (e.g., ATC [18], FEMA [19]). The stiffness of the equivalent-linear system is the secant stiffness of the nonlinear system at peak response and the viscous damping of the equivalent-linear system is such that it dissipates the same energy per cycle as the nonlinear system.

First, the nonlinear force-displacement relationship (pushover curve) is developed for the tank. This requires the computation of the uplifting resistance of the base plate. The base uplifting resistance is expressed as a relationship between the overturning base moment and the base rotation (Figure 2). The definition of this relationship is complicated by the nonlinearities arising from: (1) continuously varying contact of the base with the foundation; (2) plastic yielding in the base plate; (3) effects of membrane forces induced by large deflections in the base plate; and (4) spatial and temporal variations of the hydrodynamic base pressures.

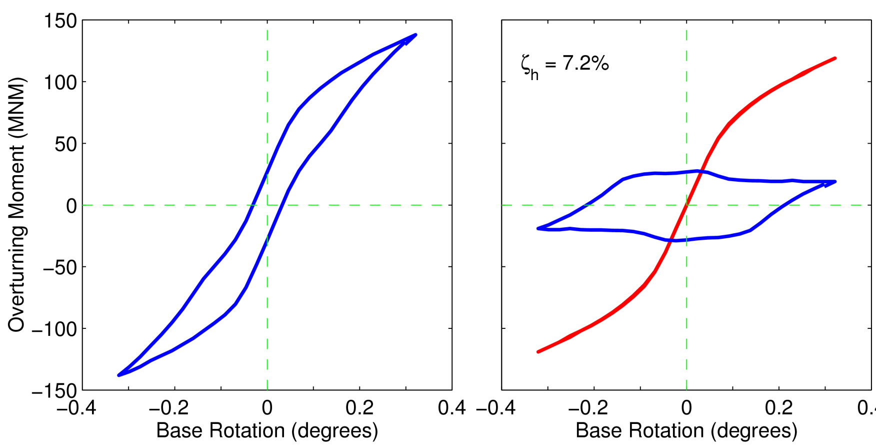

Figure 2: Overturning moment-rotation relation for partially uplifted base. The skeleton stiffness and hysteresis loop are shown on the right side.

The skeleton stiffness (backbone curve) and equivalent damping are obtained from the cyclic force-displacement relationship. The site response spectrum is adjusted for the equivalent damping of the system. The adjusted spectrum is plotted in an acceleration-displacement format. The backbone force-displacement curve is converted to the acceleration-displacement curve by dividing the force by the impulsive mass. The intersection of the backbone curve with the acceleration-displacement spectrum provides the response acceleration (Figure 3, top). The overturning moment and base shear are calculated from the response acceleration. From the overturning moment, responses associated with base uplifting are computed. These are: (1) maximum uplift, (2) radial separation, (3) plastic rotation, (4) axial compressive stress in tank wall, and (5) hoop stress in tank wall (Figures 3 and 4).

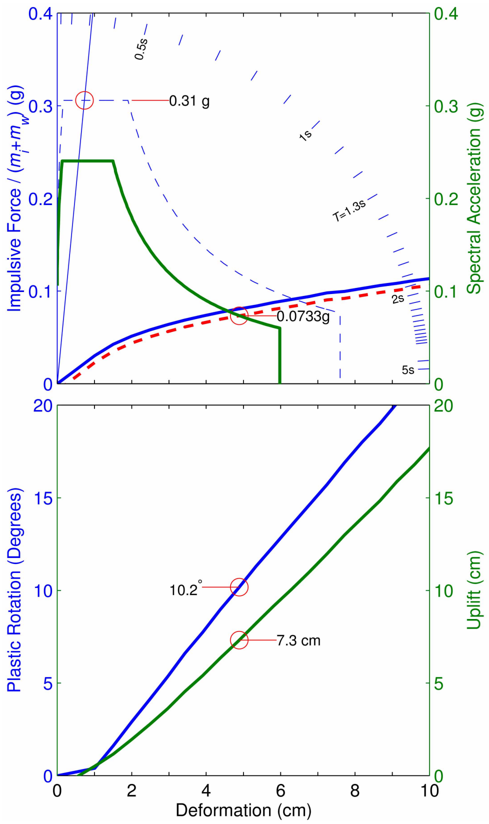

Figure 3. Static pushover analysis from skeleton stiffness and response spectrum (top). Plastic rotation and base uplift (bottom).

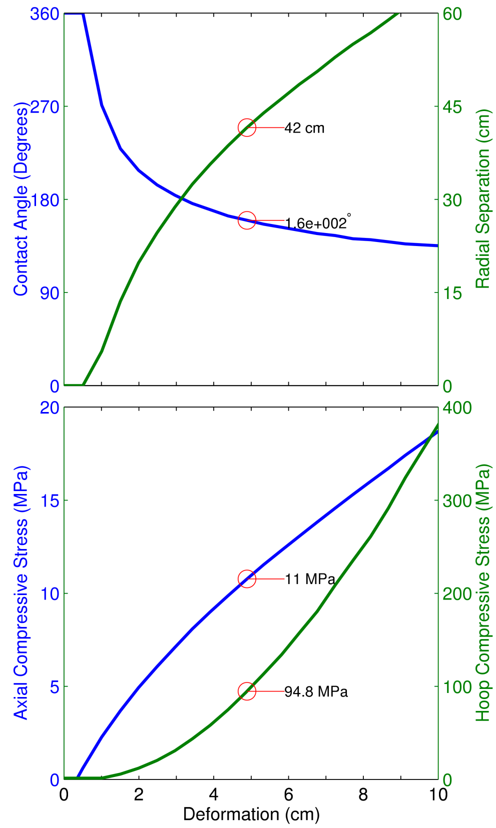

Figure 4. Contact angle and radial separation (top). Axial and hoop compressive stresses in tank wall (bottom).

Verifications

The stresses and plastic rotations resulting from both methods of calculation were compared with admissible values given in EC 8, Appendices A.8 and A.9. The following verifications were undertaken:

1. Plastic rotation in the base plate,
2. Elastic buckling of the shell (mantle), and
3. Elephant footing (elastic-plastic collapse).

Assuming a maximum allowable steel strain of 5% and a length of the plastic hinge of 2 times thickness of the base plate, the maximum allowable rotation according to EC 8, Appendix A.8, is 0.20 radian (11.5 degrees). It turned out that plastic rotation was the controlling parameter in all cases. Some engineers think that a maximum admissible cyclic strain of 5% may be too conservative. Others, however, think that 5% may be optimistic for the cyclic deformation of the welded joint between the mantle and the base plate.

CHARACTERISTICS OF INVESTIGATED TANKS

Seven existing unanchored cylindrical oil storage steel tanks in Switzerland were investigated. It is believed that these tanks are representative of the tank population in Switzerland. The tanks' main characteristics are given in Table 1. None of them was designed to resist earthquakes. All tanks were calculated according to EC 8 [1], and five of them were also calculated according to Malhotra [2].

Table 1: Main characteristics of the investigated tanks: height H, radius R, volume V, thickness of the lowest course t_{lc}, equivalent thickness of the mantle t_{eq}, yield stress of the mantle f_{ym}, thickness of the base plate t_{bp}, yield stress of the base plate f_{yb}. The tanks designed with * were only calculated according to EC 8.

Name of tank	Year of construction	H/R [–]	H [m]	R [m]	V [m^3]	t_{lc} [mm]	t_{eq} [mm]	f_{ym} [MPa]	t_{bp} [mm]	f_{yb} [MPa]
St-Triphon	1951	1.12	16.2	14.5	10‘700	24	17.7	235	10	235
Mellingen	1967	1.14	25.0	22.0	38‘000	27	20.1	295	27	235
*Niederhasli 43	1976	1.52	20.0	13.13	10‘800	12	9.0	355	8.0	355
*Niederhasli 3	1958	1.67	20.0	12.0	9‘050	23	16.1	235	10	235
Rümlang	1975	1.75	26.3	15.0	18‘600	16	11.8	355	7.0	355
Birsfelden 4	1955	2.16	19.4	9.0	5‘000	16	11.9	235	10	235
Vernier	1995	3.48	20.0	5.75	2‘080	7.0	6.9	235	7.0	235

For the generic study presented here, all tanks were calculated for identical soil conditions. A deep alluvial deposit with a shear-wave velocity of 400 m/s in the uppermost 30 m was assumed for the calculation of the SSI. This relatively stiff soil condition ensured that the effects of SSI remained moderate. For the calculations according to Malhotra [2], a local foundation stiffness (Winkler coefficient) of 4×10^7 N/m^3 was assumed. For a sensitivity study carried out for the tank “Rümlang”, the values of 1×10^7 N/m^3 and 4×10^8 N/m^3 were also used. Note that the foundation stiffness used in [14] for the calculation of the diagram shown in Figure 1 was 4×10^9 N/m^3, which was judged to be unrealistically high, but on the safe side.

CHARACTERISTICS OF SEISMIC INPUT

For all calculations, the EC 8 response spectrum of type 1 for ground class B, with peak ground acceleration of 1.0 m/s^2 was used. It is shown in Figure 5. It corresponds to the zone 2 response spectrum according to the Swiss Building Code SIA 261 [20]. An importance factor of 1.0 was adopted for this generic study, although tanks, in general, will have to be checked and designed for higher importance factors.

RESULTS

In all cases, the plastic rotation of the base plate was the relevant parameter. Only in the case of the tank “Vernier”, elephant-footing was nearly as critical as the plastic rotation. Therefore, only the plastic rotation is shown in the following. This quantity is presented versus the H/R ratio (Figure 6) and the fluid volume (Figure 7), respectively. It is recalled that the admissible maximum plastic rotation is 0.2 radian (11.5 degrees) according to EC 8.

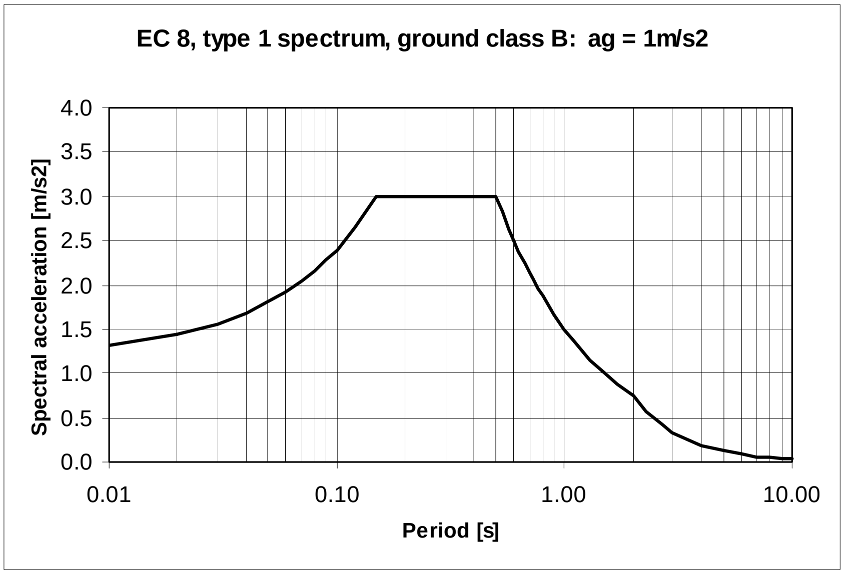

Figure 5. Acceleration response spectrum used for the seismic calculation of tanks.

The plastic rotation for the tank with H/R = 2.48 could not be elaborated according to EC 8 since the overturning moment was far outside the range covered by the graphs in EC 8. However, it can be concluded from this fact that the plastic rotation would be far above the upper limit (0.2 radian) of the range shown in Figure 6. This confirms the strong trend of increasing plastic rotation with increasing H/R ratio, visible in Figure 6, for the EC 8 results. This trend appears very clearly although the results for tanks with very different volumes are drawn in Figure 6. Any possible influence of the absolute volume onto the plastic rotation must be small. This is indeed confirmed by Figure 7, where no clear trend can be seen, the low values of plastic rotation corresponding to the two very squat tanks – irrespective of their volumes.

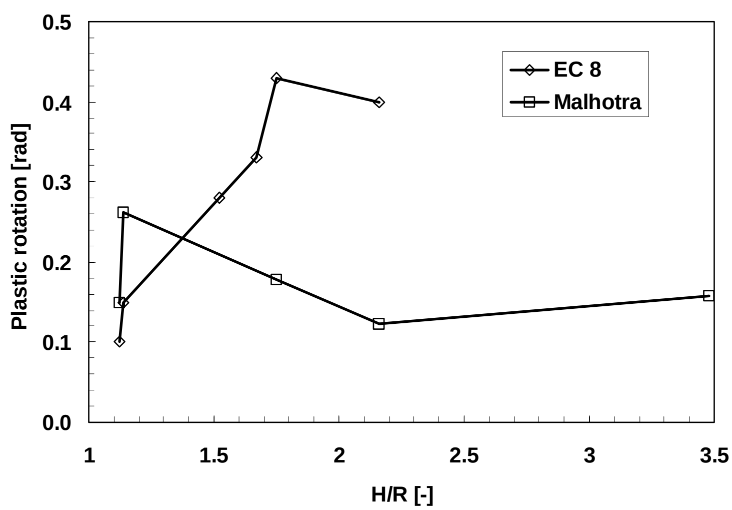

Figure 6. Plastic rotation versus H/R; the result for the tank with H/R = 3.48 ("Vernier") would be far above the uppermost value (0.5) of the graph.

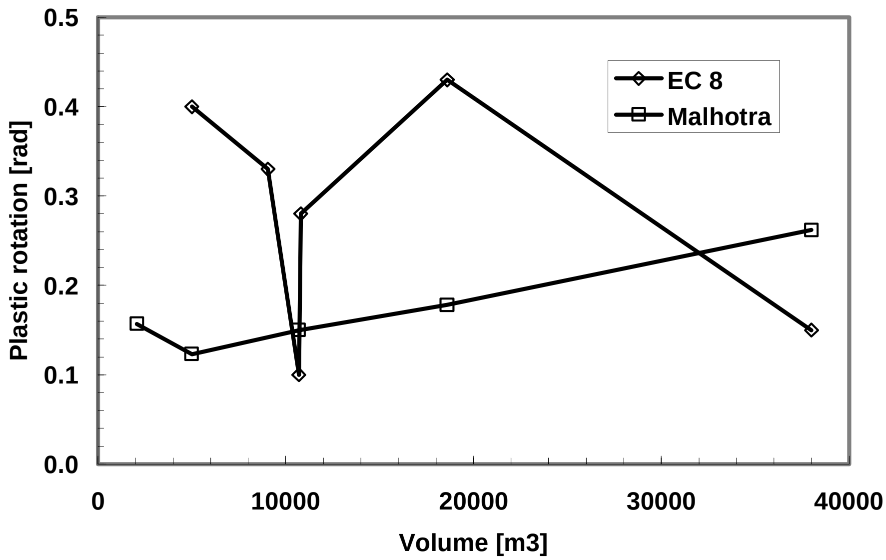

Figure 7. Plastic rotation versus fluid volume.

The situation is very different, however, for the results according to Malhotra [2]. Here, no clear trend as a function of H/R is visible in Figure 6. On the contrary, two tanks ("St-Triphon" and "Mellingen") with nearly identical H/R ratios, but very different volumes, show significantly different plastic rotations. In Figure 7, a trend of increasing plastic rotation with increasing fluid volume can be seen. Except for the particularly slender tank "Vernier" ($V = 2100\ m^3$), this trend is nearly perfectly linear. It can be concluded from these observations that the influence of the volume onto the plastic rotation is much stronger than the influence of the slenderness ratio H/R.

Sensitivity with respect to foundation stiffness

The calculations according to Malhotra [2] are sensitive with respect to the "local" foundation stiffness. This means that it makes a difference whether the tank is supported by a "rigid" concrete mat, a concrete ring or simply compacted soil. Therefore, a sensitivity study was carried out for the tank "Rümlang", for three values of the foundation stiffness: $1\times10^7\ N/m^3$ ("soft"), $4\times10^7\ N/m^3$ ("moderately stiff", value used otherwise throughout this study), and $4\times10^8\ N/m^3$ ("very stiff"). Table 2 presents the results that were obtained.

It can be seen from Table 1 that an increase of the foundation stiffness beyond the generic value used in this study seems to have little effect on plastic rotation (~10% increase in plastic rotation for a 10-fold increase in foundation stiffness). Furthermore, it seems that an overestimation of the foundation stiffness leads to results on the safe side. It has to be kept in mind, though, that for a soft foundation (with low shear wave velocity of the underlying soil), the seismic excitation might be significantly stronger due to local site effects.

The local foundation stiffness cannot be varied easily for the calculations according to EC 8; a rigid foundation mat is always assumed. Therefore, no corresponding sensitivity study was performed for the calculations according to EC 8. Note, however, that the rigid mat is assumed to lay on a viscoelastic soil, which gives rise to SSI effects.

Table 2: Results of calculations according to Malhotra [2] for different values of foundation stiffness for the tank "Rümlang".

Physical quantity	Foundation stiffness		
	$\kappa = 1x10^7$ N/m^3 ("soft")	$\kappa = 4\times10^7$ N/m^3 ("moderat. stiff")	$\kappa = 4\times10^8$ N/m^3 ("very stiff")
Hysteretic damping	10 %	7.2 %	6.2 %
Plastic rotation	0.125 rad	0.178 rad	0.202 rad
Uplifting height	5.7 cm	7.3 cm	8.3 cm
Contact angle	230°	160°	96°
"Length" of uplifted part	22 cm	42 cm	55 cm
Axial compressive stress	6.4 MPa	11 MPa	20 Mpa
Hoop compressive stress	49 MPa	95 MPa	112 MPa

DISCUSSION

For the squat tanks (H/R = 1.12 and 1.14), whether the volume is moderate or large (V = 10,700 m^3 and 38,000 m^3), the calculation according to EC 8 leads to a significantly smaller plastic rotation than the calculation according to Malhotra [2], by a factor of 1.5 to 1.7. Two concurrent aspects can qualitatively explain this discrepancy: On one hand, partial uplifting increases the fundamental natural period less for squat than for slender tanks. This means that the EC 8 calculation, neglecting this effect, is less penalized for squat than for slender tanks. On the other hand, the influence of (global) SSI, taken into account by the EC 8 calculation, but neglected in the calculation according to Malhotra [2], is more important for squat than for slender tanks. For squat tanks, the translational horizontal motion with respect to the surrounding soil is dominating the SSI, and this motion is highly damped, whereas for slender tanks, SSI is dominated by rocking, with a much lower damping. SSI is therefore much more "beneficial" in the case of squat tanks.

For H/R > ~1.5, the EC 8 leads to larger plastic rotations, with an increasing factor of discrepancy for increasing slenderness ratio H/R, this factor being greater than 2 for H/R = 1.75 (tank "Rümlang"). Again, the same aspects as before can qualitatively explain this trend. Firstly, for slender tanks, as indicated above, the importance of the SSI, neglected by Malhotra [2], is less pronounced than for squat tanks. Secondly, neglecting the lengthening of the fundamental natural period by the EC 8 calculation, as was done in the present study, as well as neglecting the hysteretic damping due to cyclic plastic deformations in the base plate, strongly penalizes the results according to EC 8. In fact, as can be seen from Figure 1, which is based on purely static considerations, the partial uplift as a function of the overturning moment is extremely sensitive with respect to the slenderness ratio H/R. It is therefore very important to take into account the lengthening of the fundamental natural period, since this decreases the overturning moment.

CONCLUSIONS

The objective of the present paper was to evaluate the appropriateness of the simplified seismic calculations of unanchored cylindrical liquid storage tanks according to EC 8, part 4, Appendix A (1998) [1]. To this aim, the results of calculations according to EC 8 were compared with results of a more sophisticated method, i.e. a pushover analysis by Malhotra [2].

Since both methods neglect “beneficial” physical effects (EC 8: lengthening of the fundamental natural period and the damping due to cyclic plastic rotation in the base plate; Malhotra [2] the SSI effects), a small Swiss expert team assumed that both methods would “probably be conservative”. They concluded that it would be acceptable to consider a tank as earthquake safe if it was safe either according to EC 8 or according to Malhotra [2] (with the judgment of plastic rotation and stresses still according to EC 8). This opinion was strongly influenced by the fact that the results found in the present study seem to be on the safe side in the light of the statistical investigation on damaged tanks published by O'Rourke and So [21].

For practitioners, it is interesting to know that a calculation according to EC 8, as presented in this paper, seems to be overly conservative for tanks with slenderness ratios H/R > ~1.5. This might be of little importance for the design of new tanks, as the additional cost of an overdesign may remain small. However, this aspect can become very important for the re-evaluation of existing tanks, where unnecessary margins of conservatism might lead to significant, but unnecessary expenses.

For tanks with a slenderness ratio H/R > ~1.5, therefore, it is strongly recommended to take into account at least the lengthening of the fundamental natural period of the “impulsive” motion – by any appropriate method – in order to eliminate unnecessary margins of conservatism.

ACKNOWLEDGMENTS

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REFERENCES

1. Eurocode (EC) 8, part 4: “Silos, Tanks and Pipelines.” ENV 1998-4, 1998; European Committee for Standardization, Brussels
2. Malhotra P. “Practical Nonlinear Seismic Analyses of Tanks.” Earthquake Spectra, 16(2), 473-492, 2000.
3. Hanson, R. D., “Behavior of liquid-storage tanks, the Great Alaska Earthquake of 1964.” National Academy of Science, Washington, D. C., **7**, 331-339, 1973.
4. USDOC. “Earthquake damage to water and swage facilities, San Fernando Earthquake of February 9, 1971.” U. S. Department of Commerce, National Oceanic and Atmospheric Administration, Washington, DC, 2, 135-138, 1973.
5. Gates, W. E. “Elevated and ground-supported steel storage tanks, reconnaissance report, Imperial County, California Earthquake of October 15, 1979.” Earthquake Engineering Research Institute, Oakland, California, 65-73, 1980.
6. Haroun, M. A. “Behavior of unanchored oil storage tanks: Imperial Valley Earthquake.” J. of Technical Topics in Civil Engineering, ASCE, 109, 1, 23-40, 1983.
7. Manos, G. C. and Clough, R. W. “Tank damage during the May 1983 Coalinga Earthquake.” J. Earthquake Engrg. Struct. Dyn., 13, 4, 449-466, 1985.
8. EERI. “Northridge Earthquake of January 17, 1994, reconnaissance report.” 1. 95-03, J. F. Hall, Editor, Earthquake Engineering Research Institute, Oakland, California, 162-179, 186-188, 259-267, 1995.
9. Brown, K. J., Rugar, P. J., Davis, C. A., Rulla, T. A. “Seismic performance of Los Angeles water tanks.” Proc. Fourth U.S. Conf. On Lifeline Earthquake Engrg., M. J. O'Rourke, Editor, ASCE, New York, NY, 668-675, 1995.

10. Lund, L. V. “Lifeline utilities lessons, Northridge Earthquake.” Proc. Fourth U.S. Conf. on Lifeline Earthquake Engrg., M. J. O'Rourke, Editor, ASCE, New York, NY, 676-683, 1995.
11. Cooper, T. W. “A study of the performance of petroleum storage tanks during earthquakes, 1933-1995.” NIST GCR 97-720, US Dept. of Commerce, National Institute of Standards and Technology, Gaithersburg, MD, 1997.
12. Miles, R. W. “Practical design of earthquake-resistant steel reservoirs.” Proc. Tech. Council on Lifeline Earthquake Engineering, Specialty Conf., ASCE, Reston, VA, 168-182, 1977.
13. Malhotra P. K., Wenk, T. and Wieland, M. “Simple procedure for seismic analysis of –storage tanks.” Structural Engineering International, IABSE, 10(3), 197-201, 2000.
14. Scharf K. “Contribution to the Behaviour of Earthquake Excited Above-Ground Liquid Storage Tanks.” Doctoral Thesis. Institute of Light Weight Structures, University of Vienna, 1989.
15. Fischer F. D., Rammersdorfer F.G. and Scharf K. “Earthquake Resistant Design of Anchored and Unanchored Liquid Storage Tanks under 3D Earthquake Excitation.” Schneller G.L. Editor. Structural Dynamics Recent Advances, Springer Verlag, 1991.
16. Wolf J. P. “Foundation Vibration Analysis Using Simple Physical Models.” Chapter 7. Englewood Cliffs: PTR Prentice Hall, 1994.
17. Résonance Ingénieurs-Conseils SA. “Erdbebensicherheit bestehender unverankerter Stehtanks.”, report TB 245-02/MK, 2003.
18. ATC. “Seismic Evaluation and Retrofit of Concrete Buildings.” Report ATC-40, November, 1996.
19. FEMA. “NEHRP Guidelines for the Seismic Rehabilitation of Buildings.” FEMA 273; and NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings, FEMA 274, October, Federal Emergency Management Agency, Washington, D.C, 1997.
20. SIA 261: 2003. “Einwirkungen auf Tragwerke.” Swiss Code SN 505 261, 2003, Schweizerischer Ingenieur- und Architektenverein, Zürich
21. O'Rourke M. J. and So P. “Seismic Fragility Curves for On-Grade Steel Tanks.” Earthquake Spectra 2000; 16(4), 801-815.