FOUNDATIONS OF POWER LINE PYLONS ON GYPSUM BEARING SOILS

GRÜNDUNG VON MASTEN FÜR HOCHSPANNUNGSLEITUNGEN AUF GIPSHALTIGEM UNTERGRUND

FOUNDATIOUS SUR GYPSE ROCHE POUR DES MATS HAUTE TENSION

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SUMMARY

Buildings on salinar rock soils can require special considerations in case of the leaching of the rock. But in most cases no special constructions are necessary because the foundation is simple and solid and the calculation is done ″erring on the side of caution″. It is shown that calculations with different methods and codes for a tension footing come to nearly the same results.

ZUSAMMENFASSUNG

Wenn Bauwerke auf Salinargesteinen errichtet werden, können spezielle Überlegungen erforderlich sein, da durch die Auslaugung des Gesteins Hohlräume entstehen. Wenn jedoch einfache und robuste Konstruktionen zur Ausführung kommen, die auf der ″sicheren Seite″ bemessen sind, kann auf eine spezielle Sicherung verzichtet werden. Es wird gezeigt, dass sowohl empirische Ansätze als auch numerische Analysen zutreffende Ergebnisse liefern.

RESUMEE

En ce qui concerne des bâtiments fondés sur roche salinaire, on a besoin de quelques réflexions speciales à cause du lessivage du roche. On peut renoncer des mesures spéciales, si on a des constructions simples et robustes, calculées avec une sécurité suffissante. Les essais suivants montrent, que non seulement les méthodes empiriques, mais aussi les analyses numériques donnent des solutions correctes.

KEYWORDS: Foundations, Gypsum, Leaching

1. INTRODUCTION

Foundations on or in salinar rock formations (gypsum, anhydrite or rock salt) generally require special considerations because leaching processes can lead to caverns. These caverns can be of danger to the buildings (Fig. 1).

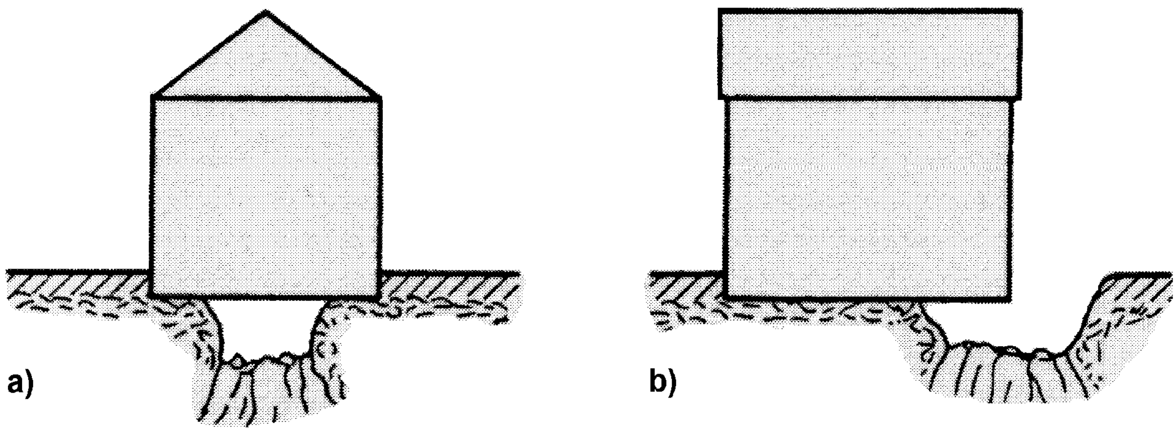
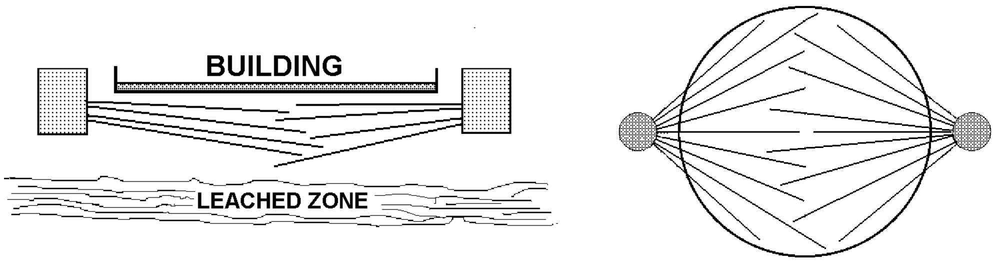


Fig. 1. *Dangers to buildings as a result of the collapse of caverns in the subsoil; internal (a) and external (b) endangerment to buildings on gypsum rock formations (Kammerer acc. to [Rogowski, 1999])*

In the main geological formation of Baden-Württemberg (B-W), the trias, the layers containing materials subject to leaching processes are the middle limestone layers (gypsum, anhydrite and rock salt) and the lower keuper layers with Gypsum and anhydrite.

The danger to buildings as a result of swelling due to clay minerals and the change of anhydrite to gypsum are more common than the damage due to the collapse of underground caverns. Even though the danger as a result of the collapse of underground caverns looks to be very dramatic. It does not occur often because the leaching process is very slow compared with the life expectancy of the buildings. Therefore only in very special cases are buildings on the surface at danger.

The development of ground injection methods such as Soil-Frac-Method allow large and sensitive structures to be erected on geological formations which may be subject to leaching. The schematic diagram in Fig. 2 shows the arrangement of the injection pipes of the Soil-Frac-Method used to keep a cooling tower at a power station, founded in the middle limestone layer in a stable position [Cartus, 1999].



SECTION PLAN

Fig. 2. *Improvement of the subsoil with the Soil-Frac-Method: injection pipes are driven under the building from shafts near the building*

A relevant endangerment due to leaching is generally only possible when, due to technical processes, a considerable ground water flow occurs. For example due to the lowering of the ground water in quarries, underground structures, locks and dams. This experience with gypsum bearing formations in B-W can be transferred to other regions of the earth. A consultant for instance made the suggestion that the foundation for a power line mast founded on a gypsum bearing formation in Syria should be protected by a roof construction (Fig. 3).

In the following the case of gypsum leaching will be considered with respect to the possible endangerment of overhead power line masts. It will be shown that roof constructions are not necessary.

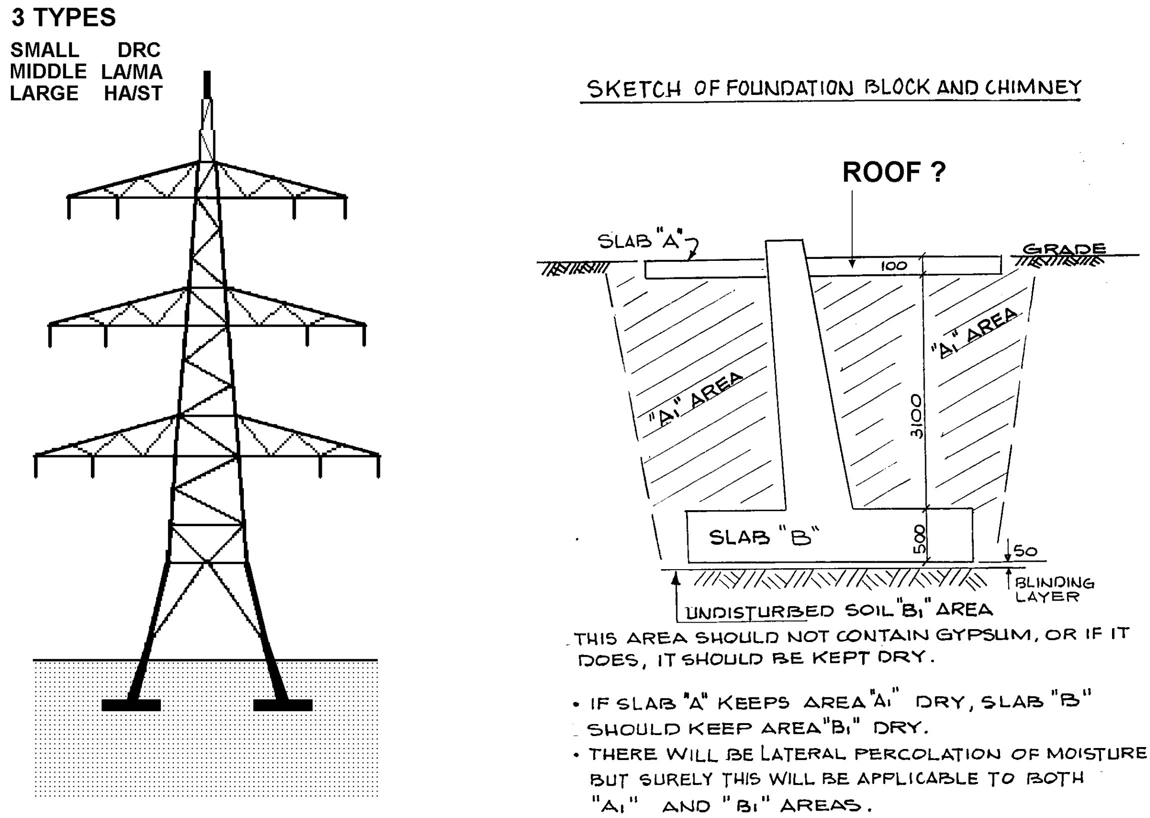


Fig. 3. *Power line mast and foundation protected by a roof*

2. FOUNDATION TYPE AND GYPSIFERIOUS SUBGRADE

For conventional foundations on gypsiferous subgrade no particular precautionary measures are required since the process of gypsum leaching will only lead to phenomena such as sink holes and depressions etc. over geological time scales (many thousands of years). During the service time of normal technical installations (100 years) gypsum leaching will have nearly no effect on conventional foundations provided these are dimensioned safely. Special measures would only be necessary in the case of pre-stressed grout anchors or similar.

Setting the towers for overhead power lines on deep embedded anchors is a type of foundation which is simple to construct yet not susceptible to trouble. It is an individual footing which may be classified as a shallow foundation. Since the loads to which the structure is exposed are predominantly horizontal forces (wind, wire tension), it is not necessary to examine the foundations for ultimate bearing capacity or settlement. In view of the considerable anchoring depth it is not necessary to provide evidence of the horizontal forces.

Crucial for the sizing and safety of the foundation is the uplift resistance of the footing. It is therefore necessary to test whether the calculation for the uplift is regarded as ″erring on the side of caution″.

3. VERIFICATION OF THE FOUNDATION CALCULATION

There are to compare the calculations according the *Safetity Rules For Overhead Lines* (SROL) with

- FE-analyses (rotationally symmetric, Mohr-Coulomb's elastic-plastic material law) and
- calculations according [Vermeer et al., 1985]

The calculation with respect to uplift resistance are based on the following assumptions:

- unit weight of soil (backfill): 15 kN/m^3;
- angle of friction (φ): 15^o;
- cohesion (c): 0.

These assumptions should be regarded as being conservative. Since the angle of dilatancy (ψ) must be taken into consideration for the comparative calculations, the simple and safe assumption ψ = 0 was selected. For friable moderately compacted backfill material the angle of dilatancy lies between 5^o and 10^o.

For cohesive material the assumption $\psi = 0$ does not incorporate any safety margin; for this reason, cohesion should be set at between 5 kN/m^2 to 20 kN/m^2. These assumptions for the soil mechanic characteristic values take into consideration the fact that the soil is exposed to a substantial alternation between drying out and high moisture levels and that the effects of binding or dilatancy are not permanent. The assumption c = 0 is based on the premise that the cohesive effect of a binding agent is not present.

The formula is [Vermeer et al., 1985]:

$$\frac{P_{\lim}}{\gamma \cdot B \cdot L \cdot H} = 1 + \left[\left(\frac{H}{B} + \frac{H}{L} \right) \cdot \tan\varphi + \frac{2 \cdot c}{\gamma \cdot B} + \frac{2 \cdot c}{\gamma \cdot L} \right] \cdot \cos\varphi_{cv}$$

For the investigation of foundations for which

$$B = L; \varphi = \varphi_{cv}; c = 0$$

the formula [Vermeer et al., 1985] can be transformed to

$$\frac{P_{\lim}}{\gamma \cdot H \cdot B^2} = 1 + \frac{2 \cdot H}{B} \sin\varphi.$$

The limit-loads of the FE-calculations are the maxima of the load-displacement-curves of Fig. 4. For the FE-analyses the Plaxis-Code [Plaxis] with the FE-meshes of Fig. 5 was used. The results of the calculations according to [Vermeer et al., 1985] and the FE method are summarised in Table 1.

The limit loads of the FE calculation derive from the load-displacement curves shown in Fig. 4. For the elastic-plastic calculation a shear modulus of 20,000 kN/m^2 and a Poisson's ratio of 0.25 were selected. The square foundations were converted to circular foundations of the same area.

So a rotationally symmetrical analysis was performed. The FE grids are presented in Fig.5.

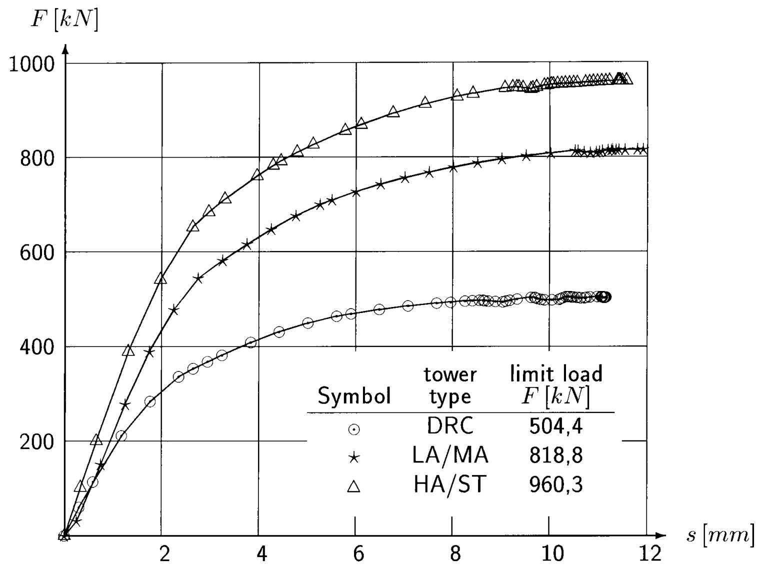


Fig. 4. *Load-Displacement-Curves of FE-analyses*

Table 1. *Results of the calculations*

		Tower type		
		DRC	LA/MA	HA/ST
Factored load: $N_{z,max}$ Tension	$[kN]$	433	768	926
Depth (H)	$[m]$	3.0	3.6	3.8
Width (B)	$[m]$	2.4	2.8	3.0
Area (A)	$[m^2]$	5.76	7.84	9.0
Volume (V_{block})	$[m^3]$	16.8	28.2	34.2
$G_{block} = \gamma_{soil} \cdot V_{block}$	$[kN]$	252	423	513
H/B	[1]	1.25	1.286	1.267
$2 \cdot \frac{H}{B} \cdot \sin\varphi$	[1]	0.647	0.666	0.656
$F_{lim,soil}$	$[kN]$	415	704	850
$V_{concrete,plate}$	$[m^3]$	2.016	3.92	4.5
$V_{concrete,column}$	$[m^3]$	1.94	3.19	3.40
$\Delta G = V_{concrete} \cdot 10$	$[kN]$	39.56	71.1	79
$F_{lim,tot} = F_{lim,soil} + \Delta G$	$[kN]$	454.56	775.1	929
$F_{lim,tot,vermeer}$/factored load	[1]	1.05	1.01	1.003
FE analysis: $F_{lim,tot}$	$[kN]$	504.4	818.8	960.3
$F_{lim,tot,FEM}$/factored load	[1]	1.16	1.07	1.04

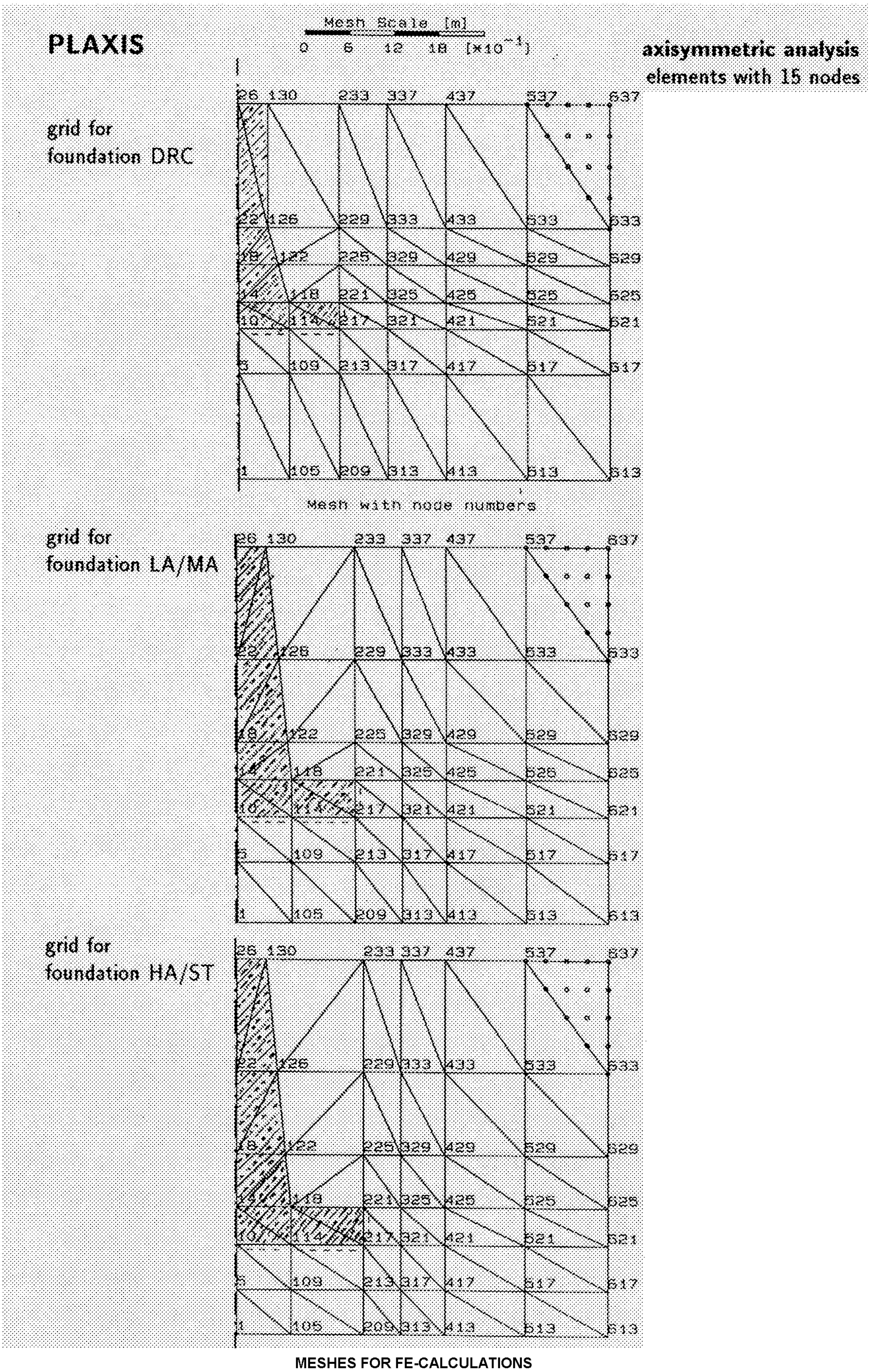


Fig. 5. *FE-grids for the analyses*

6 EVALUATION OF THE CALCULATION

Both the calculation according to the formula by [Vermeer et al., 1985] and the calculation according to the elastic-plastic FE analysis give slightly higher limit loads than those calculated according SROL. This demonstrates that both the assumptions made and the SROL calculation incorporate a margin of safety.

Whereas the SROL calculation was made using factored loads, applicable German safety standards require that calculations are made using loads in the state of serviceability and that testing is carried out to establish whether the safety requirements of German Subgrade Standard DIN 1054 [DIN 1054, 1976] are fulfilled.

In the ANSI standard C2, on which the SROL calculation is based, the following Overload Capacity Factors (table 261-4) are defined:

- Vertical loads: Grade B: 1.5; Grade C: 1.5 (when vertical loading significantly reduces the loading on a structure member, a vertical overload factor of 1.0)
- Transverse loads: Wind: Grade B: 2.5; Grade C: 2.2
  Wire tension: Grade B: 1.65; Grade C: 1.1
- Longitudinal loads: In general Grade B: 1.1;
  Grade C: no requirements;
  at deadends: Grade B: 1.65; Grade C: 1.1

According to DIN 1054 shallow embedded anchors may be classified either as foundation elements with uplift (taking into consideration lateral soil reaction) or as anchor piles with alternating loads. The required safety coefficients are as follows:

- Foundation elements with uplift: Loading case 1: 1.4;
  Loading case 2: 1.4; Loading case 3: 1.2.
- Piles with alternating loads: Loading case 1: 2.0;
  Loading case 2: 2.0; Loading case 3: 1.75.

Since the main load for linear power lines from wind and wire tension is only relevant at angles or corners it is clear that the safety coefficients required by ANSI C2 are higher than those required by [DIN 1054, 1976].

6. CONCLUSION

It was shown that calculations with different standards and methods come to similar results. Because the tension footings are a simple and solid construction Gypsum leaching does not present any danger in the duration of serviceability. The installation of a ″roof″ (slab A of Fig. 3) is not necessary. Moreover, the construction method selected and the assumptions on which the calculations are based can be classified as being safe with respect to the extreme variation between drying out and high moisture levels.

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