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Liquefaction and the 1992 Roermond earthquake, the Netherlands

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Abstract

The scanty information available on liquefaction phenomena during the Roermond earthquake does not allow firm conclusions on the technical significance of these phenomena. However, speculations based on applied mechanics principles justify a few conclusions on what happened and what did not happen. First, the presence of fracture vents and sand boils agrees well with the estimated earthquake magnitude M_L = 5.8–5.9; second, the rare occurrence of sand boils and the absence of visible settlements indicate that liquefaction was not an important phenomenon during the earthquake; third, damage of farms due to liquefaction seems absent. This observation is in agreement with the theoretical analysis of responses of the farm foundations to liquefaction. If the area affected by liquefaction had exceeded some 1% of the total area, significant damage would have been detectable. The low ground water tables during the time of the earthquake have most probably prevented extensive damage.

1. Introduction

Evidence of liquefaction, in the form of fracture vents and sand boils, was found at three places near Roermond: north of Roermond, near Herkenbosch, and near Montfort, all within a radius of 10 km from the epicentre of the 1992 Roermond earthquake. Audemard & Santis (1991) reported liquefaction phenomena up to 20 km away from the epicentre caused by two moderate (and comparable) earthquakes (M_L = 5.0 and 5.7) in Venezuela. Atkinson et al. (1984) state that the magnitude of earthquakes must exceed M_L = 5.0 in order to have sufficient shear stress cycles to generate liquefaction in the subsoil. The occurrence of liquefaction during the Roermond earthquake agrees well with these observations.

Estimates of the maximum distance of liquefaction from the epicentre, or rather from the focus, rely on approximately linear relations between horizontal surface accelerations and magnitude. For shallow earthquakes (focal depth $\leq$ 10 km) the maximum distance from the epicentre for liquefaction phenomena is estimated at 0–8 km for M_S = 5.5 and at 5–20 km for M_S = 6.0 (Liao et al. 1988, Ambraseys 1988). According to Ambraseys & Bommer (1991) the near-field attenuation for earthquakes with focal depths of 15–20 km (as estimated for the Roermond earthquake) proceeds half as strong as for earthquakes with focal depths at 8–10 km (for which the previous figures on the size of liquefaction areas hold). Hence liquefaction phenomena within an area of 10 km around the epicentre as observed during the Roermond earthquake accord with the estimated magnitudes M_L = 5.8–5.9, M_S = 5.2–5.4 and focal depth 15–20 km.

The conditions for liquefaction are relatively well known (Lomnitz & Rosenblueth 1983), but the mechanisms in the field, in natural soils, are at best only partly understood. Most sensitive to liquefaction are fine uniform sands with coefficients of permeability less than 10^{-3} m/s (Lomnitz & Rosenblueth 1983). The particle size distribution near Herkenbosch is shown in Fig. 1 (Shannon & Wilson 1971); its coefficient of permeability is 10^{-4} m/s. Figure 1 demonstrates that the Herkenbosch sand is susceptible of liquefaction.

In the subsequent sections of this paper the observed lack of damage to the foundations of farm buildings in the fields surrounding the village of Herkenbosch is related to the prevailing condition of these fields at the time of the earthquake. Evidence of liquefaction of the subsoil of the fields is given by the

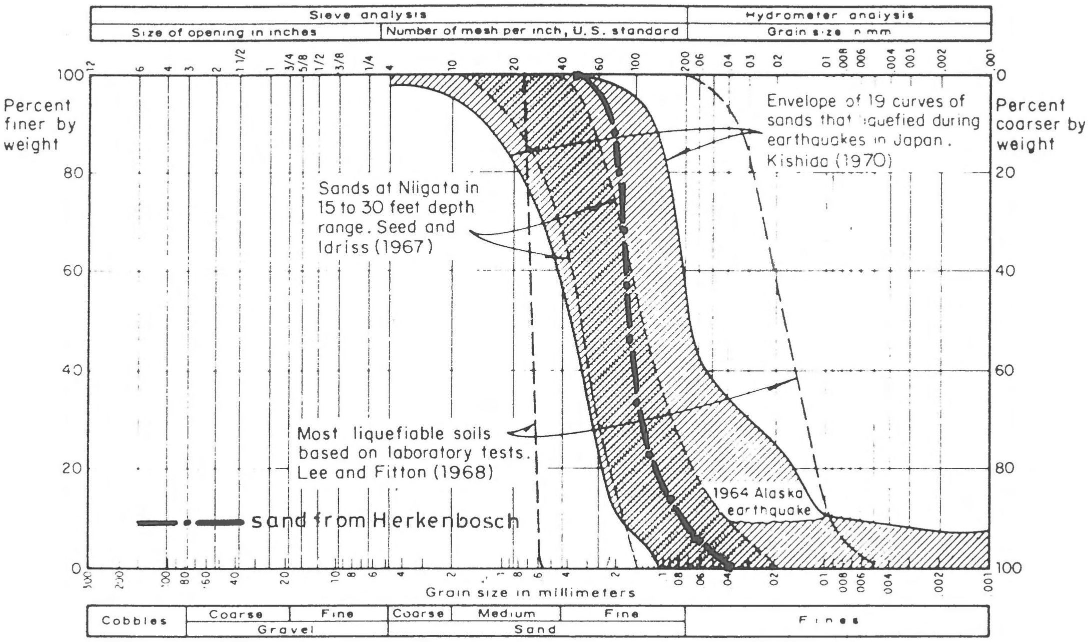

Fig. 1. Relation of grain size distribution and susceptibility of liquefaction showing grading of sand at Herkenbosch (Shannon & Wilson 1971).

presence of sand boils at the surface (Fig. 2). These sand boils occur locally, many in a few areas. The areas covered by sand boils comprise less than 1% of the total area which could have been affected by liquefaction.

It is assumed on the basis of the field observations that the subsoil of the fields was only locally liquefied, and the farm foundations in these fields are therefore considered as subjected to partial liquefaction of the supporting subsoil. On the relation between field observations, partial liquefaction and damage to structures little has been published in the earthquake engineering literature. To gain insight in this relation an analytical approach is chosen in this paper. Due to lack of supporting observations or measurements the approach remains speculative in some aspects.

2. Deductions from the presence of fracture vents and sand boils

The depth of the ground water level at the time of the Roermond earthquake was at least 3.5 m below the surface of the fields near Herkenbosch. Together with the amount of sand spread over the surface (0.4 m^3 in the sand boil of Fig. 2), this suggests an effective

Fig. 2. Evidence of liquefaction: sand boil and vent fracture near Herkenbosch, east of Roermond, April 1992.

upward hydraulic transport of suspended sand. Under what conditions could such a flow occur?

A saturated sand body, such as the loamy sand subsoil near Herkenbosch below the groundwater table, may react in two ways to the shaking motion generated by an earthquake. If densely packed, the sand will increase in volume and the pore pressure will drop. Densely packed sands will not liquefy. If loosely packed, the sand will decrease in volume and the pore pressure will rise. The effective stress will consequently even drop to zero values. This effect is called liquefaction. In strong earthquakes nearly all sands behave as being loosely packed (Castro 1987), but in moderate earthquakes such as the Roermond earthquake the distinction between loosely and densely packed sands is relevant. If loosely packed sands do not liquefy during the earthquake, the dissipation of the generated excess pore pressures proceeds by ordinary groundwater flow.

If liquefaction occurs, the sand particles may even loose contact during the shaking and the sand body may contain local water lenses (Housner 1958). Such lenses can only exist locally and adjacent to impermeable and cohesive layers. Locally, because water bodies cannot support the overburden and should therefore be small and adjacent to impermeable and cohesive layers as the water body would otherwise be filled with setling sand. Clay and silt seams are present in the subsoil near Herkenbosch. Therefore water lenses or pockets may have existed during and briefly after the earthquake.

A liquefied sand body or a water pocket presents a failing support for the overburden. Cracks in the overburden, such as fracture vents, originate easily in these conditions. The upward transport of the suspended sand through a fracture vent could in principle consist of individual sand particles suspended in water or of a very dense suspension with a similar water content as the liquefied sand mass. The amount of sand spread over the surface in a sand boil (Figs 2, 4), in the order of 0.1 m^3 of sand per m of fracture vent, would require quite a lot of transport fluid if individual sand particles were transported. In the case of a dense suspension not only the amount of sand per volume of sand-water mixture is large, but the flow velocity of the suspension could be much less than for individual particles. In the last case, the flow velocity must at least exceed the setling velocity of the particles (0.1 m/s or more).

The pressure p(z) (Fig. 3) exerted by the (dry) overburden along the interface $z = z_1$ equals roughly:

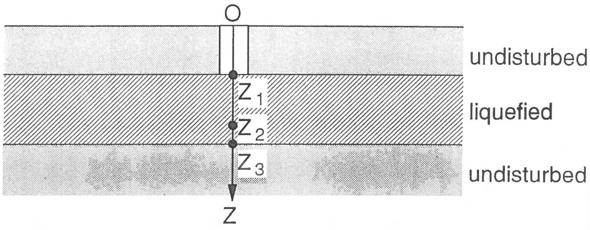

Fig. 3. Liquefied layer sandwiched between undisturbed layers; Z indicates depth.

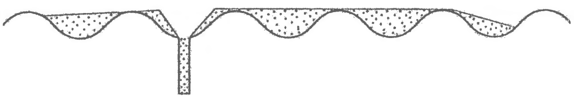

Fig. 4. Schematic cross section of a sand boil over plow furrows.

$$p(z_1) > \rho_g g z_1 \qquad (2.1)$$

with ρ_g = bulk weight of the (dry) top soil and g = acceleration of gravity. In the fracture (f) the soil-water mixture, with bulk weight = ρ_s, will roughly flow according to Bernoulli (Batchelor 1967):

$$p_f(z) > \rho_s g z \qquad (2.2)$$

The soil – water mixture will consequently flow upward as long as along the interface $z = z_1$ the condition:

$$\rho_s < \rho_g \qquad (2.3)$$

Since the soil – water mixture in the fracture will loose water laterally into the dry overburden both ρ_s and ρ_g tend to increase as the discharge through the fracture proceeds.

A rough idea of the venting process through a fracture may be obtained by introducing values in Eq. 2.3. If only quartz-particles, specific gravity 2650 kg/m^3, are considered and a usual *in situ* bulk weight ρ_g of 1700 kg/m^3 for the dry overburden layer is taken then, initially, the suspended sediment in the flowing mixture must be smaller than 42% by volume. That is much smaller than the solid matter content of *in situ* sand. Hence, extra water should be available, for example in the form of water pockets, to initiate the venting process. If the dry overburden soil adjacent to a fracture vent gets saturated, the suspended sand content

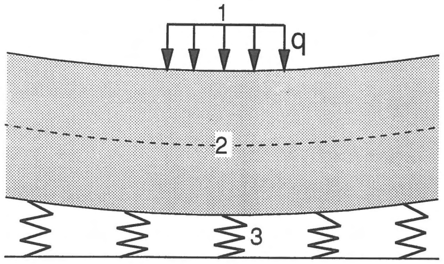

Fig. 5. Computation model for a uniformly loaded 'sand beam'. 1. farm building replaced by a uniform vertical load q. 2. 'sand beam' in bending. 3. subsoil described by springs.

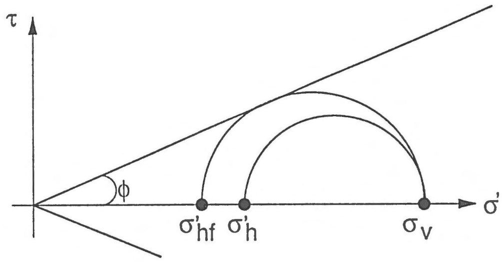

Fig. 6. Mohr diagram illustrating failure stresses in sand. τ – shear stress, σ' – normal stress, σ'_h – horizontal stress during bending, σ'_{hf} – horizontal failure stress, σ'_v – . . . . ., ϕ – angle of internal friction.

may rise to the solid matter content of *in situ* sand, implying that the upward flow may consist of liquefied subsoil without added water. If finally the shaken and liquefied subsoil reconsolidates, the solid matter content in the upward flowing mixture will drop implying that ρ_s in Eq. 2.3 will drop. In that case the flow rate in the fracture may even increase, but the amount of transported sand decreases of course. Eventually, the upward flow, consisting of clear water only, will wane away.

Although comprehensive eye-witness descriptions of the process of sand boil formation are lacking (Housner 1958), the essentials of the analytical description presented above – initially water with a sediment load, followed by a 'steady' flow of a dense suspension and concluded by clear water flow – may be found in recorded observations.

3. Explanations for conditions of minimal liquefaction damage to farms

3.1. Introduction

The fields near Herkenbosch that show evidence of liquefaction are surrounded by a number of farms. None of these farms show structural damage that can be attributed to the earthquake. The foundations which are shallow strip footings on loamy sand can be schematized as a uniformly loaded 'sand beam' on an elastic foundation consisting of springs (Fig. 5). These beam elements are assumed to bend under vertical loads by compression in the upper parts and extension in the lower parts (Den Hartog 1949). By further assuming that plane sections remain plane in bending – they only rotate with respect to each other – all stresses and deformations in a beam can be determined. It is somewhat unusual to consider a sand body as a beam, however, it has been done before (De Josselin de Jong 1962). The shallow strip footings, certainly not deeper than some 60 cm, embedded in a stiff loamy sand layer that is unaffected by liquefaction down to the ground water level at some 3.5 m of depth, can be described as a uniformly loaded sand beam. The similarly sandy subsoil, below the ground water level, must have been liquefied in places during the earthquake. A model of this subsoil with supporting springs may simulate roughly some of the liquefaction effects by reducing the spring constants or by taking some of the springs out. Computations with this model will be presented in the next section.

3.2. Computation of stresses and deflections

Unlike concrete or steel, pure sand, characterized by zero cohesion and angle of internal friction ϕ, cannot carry tensile stresses. During bending the existing horizontal stress σ'_h in the lower part of the sand beam (Fig. 7b, and Mohr's plane Fig. 6) will decrease to σ_{hf}. σ_{hf} (failure) is the horizontal stress for which Mohr's circle touches the failure envelope (Fig. 6) implying a state of failure for the sand. The interval σ'_h-σ'_{hf} is available for induced tensile stresses; the total effective horizontal stress remains of course compressive.

It is not uncommon to describe the stress-strain behaviour of sand under small deformations by a modulus of elasticity, E, which is similar for loading and unloading. In that case the stress distribution in bending is similar to the one of a common structural beam as depicted in Fig. 7a. The horizontal stress distribution is linear with depth and symmetrical with respect to an unstressed centre fibre, the neutral line.

The maximal induced tensile stress in the sand beam obeys (Fig. 6):

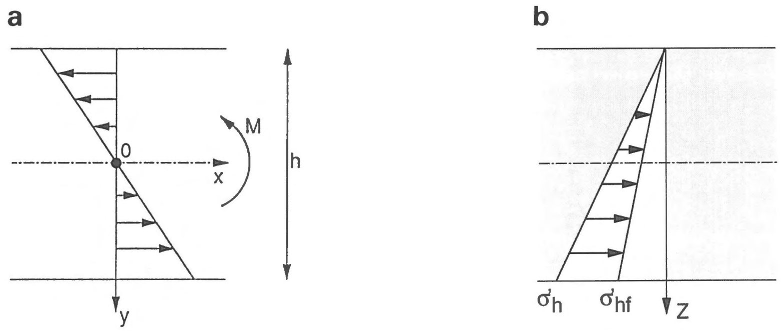

Fig. 7. Comparison between stresses in a structural beam and a 'sand beam' as depicted in Fig. 5. a: Stresses in a structural beam with length 1 due to moment M. b: Available induced horizontal tensile stresses (σ'_h-σ'_{hf}) in a 'sand beam'. See text for explanation.

$$\sigma_t^{max} = \sigma'_h - \sigma'_{hf} = \left[k_o - \frac{1-\sin\phi}{1+\sin\phi}\right](\rho_g - \rho_w)\, gz \quad (3.1)$$

with k_o the coefficient of *in situ* horizontal pressure, ρ_g bulk weight of soil, ρ_w bulk weight of water and z depth of the sand beam with respect to its top. With k_o often approximated by $k_o = 1 - \sin\phi$, Eq. 3.1 becomes:

$$\sigma_t^{max} = \frac{\sin\phi(1-\sin\phi)}{1+\sin\phi}(\rho_g - \rho_w)\, g\, z \quad (3.2)$$

Also:

$$\sigma_t^{max} = \frac{M.^h/_2}{I},\ I = \frac{bh^3}{12} \quad (3.3)$$

where the maximum moment (M) which the sand beam can support can be established. $^h/_2$ is the y coordinate of the maximum horizontal stress (Fig. 7) and I is the moment of inertia of a prismatical beam with width b and height h (Den Hartog 1949). The equilibrium of forces in a uniformly loaded small element of a beam supported by springs is depicted in Fig. 8. The vertical equilibrium satisfies:

$$\frac{dQ}{dx} = ky - q \quad (3.4)$$

with k the 'spring constant' of the subsoil and y the deflection of the beam. Q is the shear force across the beam, q is the uniformly distributed load on top of the beam (see also Fig. 8). And with the well-known relations from beam theory:

$$\frac{dQ}{dx} = \frac{d^2M}{dx^2}$$

and

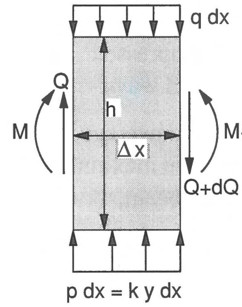

Fig. 8. The equilibrium of forces in a uniformly loaded small element of a beam on elastic foundation. For explanation see text.

$$M = -EI\frac{d^2y}{dx^2}$$

from Eq. 3.4 we obtain:

$$EI\frac{d^4y}{dx^4} = -ky + q \quad (3.5)$$

This is the general equation for beams on an elastic foundation (Hetenyi 1971). With the aid of the Eqs 3.2, 3.3 and 3.5 some cases of loading of the schematized farm foundations can be considered.

By introducing common values for loamy sand, E = 10^5 kPa, ϕ = 25, 35, 40°, ρ_d (dry soil) = 1700 kg/m^3, k = 10^4 kN/m^3 (Heijnen 1971, 1972), and by introducing common dimensions for the farm foundation and its loading, b = 10 m, l = 20 m, h (thickness of the beam) = 3.5 m and q = 40 kN/m^2, some computation results may be presented. The maximum moment to be supported by the sand beam is 190 kN m/m' (ϕ = 35°), the moment supported by the beam in integral (not liquefied) condition is 46 kN m/m' (m' stands for per meter of width b). Hence, the farms are safely founded in static (normal) conditions.

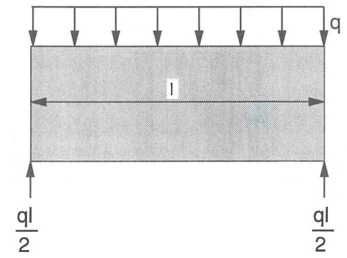

Fig. 9. Sand beam with interrupted supports.

During partial liquefaction the spring constant k (coefficient of subgrade reaction) will be reduced. One could imagine that part of the support is lost and that the remaining supports carry the full load and are, consequently, further compressed. This condition can easily be considered in two ways:

- through a reduced average value of k,
- by considering the maximum free (unsupported) span of the sand beam.

Failure of the foundation (or the building) may also occur in two ways:

- the building cannot withstand the deflections of the foundation
- the sand beam cannot withstand the free spans.

According to the Dutch specifications (1990, 1991) no damage to buildings in masonry, such as the farms, is expected as long as the tilt of the foundation is less than 0.33%. Very heavy damage occurs if the tilt exceeds 1.25%. In the case of the Herkenbosch farms even a reduction of the k-value with a factor 10, implying that only 10% of the original supports of the sand beam are still functioning, does not result in a tilt larger than 0.3%. This situation seems such an extreme reduction in support that the flexibilities of the buildings themselves must be large enough to cope with major foundation damage.

In contrast, the beam of loamy sand can only allow limited free spans. By considering two supports separated by a free span (Fig. 9) and adapting the figures for q to some location below the farm foundation, the allowable free span (for $\phi = 35°$) appears to be circa 1.40 m. In the next section this value is compared with estimates of the relative extent of liquefied areas below the foundation. The influence of the angle of internal friction on the allowable moment in the sand beam and on the allowable free span of the beam is minimal.

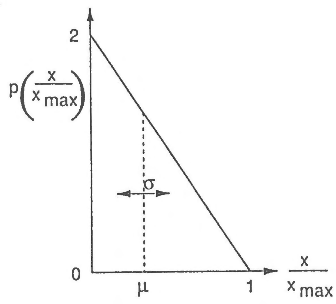

Fig. 10. Triangular probability distribution for liquefied areas x, with mean (μ) and standard deviation (σ).

Variations of ϕ between 25° and 40° make little difference. A reduction of the thickness of the sand beam, implying a higher ground water level and a shallower location of liquefied areas, has large influence on both allowable moments and allowable free spans. By reducing the thickness with a factor 2, the moments and free spans are reduced by a factor 2^3 (= 8).

3.3. Relative extent of liquefied areas

The geometry and the lithology of the layers essentially control the propagation of liquefaction in a liquefaction prone soil body. Since these data are not available and since lithological simulation on a scale as large as a farm foundation is still in its infancy, it is unavoidable to tackle the problem by speculation. Results are obtained by assuming that:

- the ratio of maximum sizes of liquefied patches and 'undisturbed patches' below a farmhouse is similar to the ratio of liquefied and undisturbed areas in the vicinity,
- the types of size distributions of liquefied and undisturbed patches are similar,
- the largest liquefied area is similar to the foundation area of a farmhouse (larger areas are not worth considering).

As stated before, the distribution of liquefied areas in a plane is unknown. Some distribution has to be chosen. The normal distribution is unrealistic (too many small patches), the binominal distribution is only valid for finite samples and a Poisson distribution is probably well-suited, but should be combined with a simulation method. A homogeneous distribution is unrealistic (too many large patches). Since only a first-order estimate is required, the triangular distribution is used (Fig. 10). This distribution is realistic at first sight and simplifies computations. The areas of liquefied or undisturbed patches are normalized by division with the maximum area x_{max} (200 m^2).

It can easily be verified that the average μ of the triangular distribution satisfies $\mu = 1/3$ and that the standard deviation σ equals $\sigma = 0.2357$. To avoid underestimating the sizes of liquefied patches, x_c, the 5% level of exceeding a certain value, is chosen as:

$$x_c = (\mu + 2\sigma)x_{max} \quad (3.6)$$

If the ratio of liquefied and undisturbed areas in the region equals r then:

$$\mu_l = rx_{max}, \mu_u = (1 - r)x_{max} \quad (3.7)$$

with $_l$ = liquefied, $_u$ = undisturbed.

The maximum allowable free span (d) of the partly-supported loamy sand beam must equal at least the crossection of a liquefied area. d is of the order of $\sqrt{rx_c}$ and therefore d satisfies:

$$d = \sqrt{r(\mu + 2\sigma)x_{max}} \quad (3.8)$$

Using d = 1.40 m and $\phi = 35°$ (previous section), the relative liquefied area r can be determined from Eq. 3.8 i.e. r = 0.013 (1.3%).

This result neither depends on the type of sand (variation of ϕ) nor the characteristic or average values in Eq. 3.6 (μ in stead of $\mu + 2\sigma$). For average values, the allowable ratio of the liquefied area becomes 3% instead of 1.3%. The result should depend on the selection of x_{max}, an essentially unknown figure. But again this influence is acceptable. If 100 m^2 were chosen in stead of 200 m^2, r would become 1.6% and if 300 m^2 were chosen r = 0.9%. These estimates, which fortunately enough are not very sensitive to variations in soil properties and statistical assumptions, demonstrate the degree of vulnerability of foundations to liquefaction. Liquefaction is in fact known to be a destructive agent for foundations (Ambraseys 1988).

4. Conclusions

- The presence of liquefaction phenomena within an area of 10 km radius around the epicentre appears to be consistent with the estimated magnitude and focal depth of the Roermond earthquake.
- Theoretical scenarios are used to explain scanty observations; the conclusions are therefore of a tentative nature: the suspended sand content in the discharge of a venting fracture through a dry top soil must be small initially. This implies the presence of water pockets in the layer from which the soil – water mixture originates.
- The duration of flow through the fracture is largely determined by the settling ability of the top soil.
- Damage to foundations of farm buildings, assuming a relatively thick (3.5 m) and unliquefied loamy top sand layer (as is the case in the area near Herkenbosch), is determined by the strength of the top layer rather than by the flexibility of the foundation.
- The 'strength' of the top layer is strongly determined by its thickness and much less by the actual strength of the soil itself.
- Farm foundations may only withstand relatively small areas of liquefied subsoil. If more than 1 or 2% of the subsoil area below the top layer is liquefied, the foundations may fail.
- It is likely that a greater amount of damage to shallowly founded structures in the Roermond area would have occurred, if the ground water level had been higher.

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