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Influence of the 1992 Roermond earthquake, the Netherlands, on shallow foundations

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Abstract

This paper considers the influence of the strong ground motion due to the Roermond earthquake on soil behaviour and shallow foundations. As no local strong ground motion acceleration record was available, a synthetic record has been used. The procedure used to obtain this synthetic acceleration record is explained. The stratigraphy of the shallow subsoil is briefly described and results of a short literature review on densification of soils are presented. Finally, an estimate is made of the settlement of shallow foundations at one site (Maasniel area) in Roermond, due to the strong ground acceleration induced by the earthquake. From this estimate we conclude that shallow foundations, designed according to Dutch building codes, will generally not suffer excessive settlement values under earthquake loading comparable to that which occurred in Roermond on April 13, 1992.

Introduction

On April 13, 1992, Roermond was hit by an earthquake, one of the largest in the history of the Netherlands. The magnitude of the earthquake was assessed to be $M_L = 5.9$. Several buildings suffered significant damage, but fortunately there were no direct casualties. One of the questions arising after the earthquake concerned the influence of the earthquake on the subsoil and the existing foundations. This is of interest in view of the current integrity and future use of these foundations. In this paper the influence on shallow foundations is examined. The influence on piled foundations is treated in a companion paper (Luger et al. 1994). The study is focused on the quarter of Maasniel, the north-eastern part of Roermond where substantial damage occurred. The studied phenomenon is earthquake-induced settlement of shallow foundations. Possible failure of shallow foundations during and shortly after the earthquake has not been considered, as this failure mode would have been noticed immediately.

Acceleration record

In order to study the influence of the earthquake it is necessary to have a representative acceleration record. In principle the following methods are available:

1. using an acceleration record measured in Roermond,
2. using an acceleration record from a station elsewhere and scaling it back to the level representative for the Roermond area,
3. using an acceleration record from an aftershock and scaling it up towards the situation at the moment of the earthquake (Gariel et al. 1994),
4. using an acceleration record from another earthquake with the same magnitude measured at a place with a comparable geology and seismicity as for Roermond,
5. generation of a synthetic acceleration record with characteristics representative for the Roermond earthquake.

Method 1 is to be preferred but, unfortunately, the magnitude of the earthquake exceeded the dynamic range of the seismographs in the vicinity. Therefore, one of the other approaches had to be used. In this study method 5 has been applied. The methods 2, 3

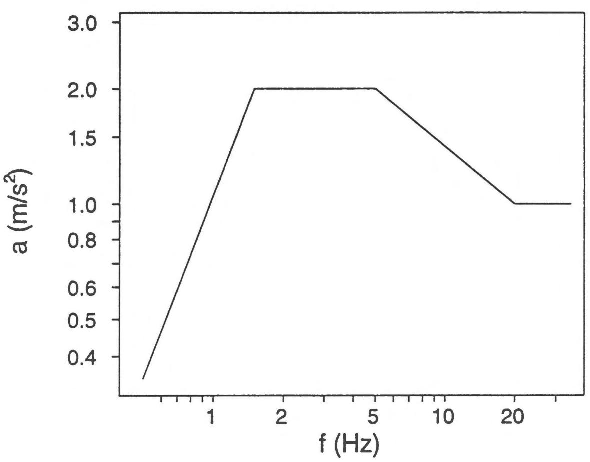

Fig. 1. Ground acceleration response spectrum for an earthquake in the Roer Valley Graben with I_0 = VII–VIII and 5% damping according to Hosser (1987). a = acceleration, f = frequency.

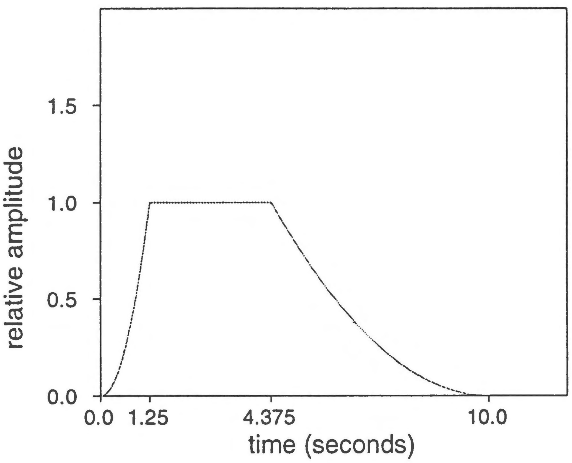

Fig. 2. Assumed acceleration amplitude envelope constraining the synthetic ground acceleration.

and 4, using measured acceleration records, were disregarded, partly because of the uncertainty introduced by assumptions in scaling the measured records to the event which occurred in Roermond, and partly because relevant data were not available at the time of this study (second half of 1992).

The procedure used for generating the synthetic acceleration record consists of the following steps:

- assessment of the intensity in Roermond of the earthquake and the peak ground acceleration (PGA),
- selection of the representative ground response spectrum,
- assessment of the duration of the earthquake,
- generating a synthetic acceleration record.

TNO-Building Research Delft, the Netherlands has investigated some of the damage which has occurred (Staalduinen 1992). From the damage the intensity of the earthquake was assessed. In a number of cases the PGA has been assessed by back-calculation of the acceleration needed to cause the observed damage. The conclusion of the survey was (Van Staalduinen 1992) that the intensity (I_0) of the earthquake was VII, both on the Modified Mercalli (MM) scale and on the Medvedev-Sponheuer-Karnik (MSK) scale. The PGA in the city of Roermond was assessed to be 0.9 m/s^2 (0.09 g)

The design ground response spectrum for an equivalent earthquake in the Roer Valley Graben with soft soil has been taken from Hosser (1987). This spectrum is shown in Fig. 1 and is valid for an earthquake with I_0 = VII–VIII and 5% damping. The duration of the strong ground acceleration at the site was estimated to be about 10 seconds. The acceleration amplitude envelope, which has been used is shown in Fig. 2. A synthetic acceleration record a(t) has been generated by a summation of sine-terms with different angular-velocities (ω_i), phase-shifts (ϕ_i) and amplitudes (a_i):

$$a(t) = \Sigma\, a_i.\sin(\omega_i.t + \phi_i)$$

multiplied the depicted envelope (Fig. 2). ϕ_i was selected randomly, while a_i depends on ω_i, constrained by the design ground response spectrum as shown in Fig. 1. Relatively high amplitudes are found for angular velocities in the order of 20–30 rad/s (3–5 Hz). For lower and higher angular velocities the amplitudes will decrease. An iterative procedure was adopted to obtain a record yielding the given ground response spectrum. Initially an arbitrary relation between amplitude and angular velocity has been assumed to generate an acceleration record. From the synthetic acceleration record the response spectrum is determined and compared with the design ground response spectrum as shown in Fig. 1. Based on this comparison the initially selected relation between amplitude and angular velocity is adjusted and the procedure is repeated. After three or four iterations an acceptable agreement between the response spectrum of the synthetic acceleration record and the design ground response spectrum is generally achieved. Figure 3 shows the obtained synthetic acceleration record and Fig. 4 its response spectrum compared with the design ground response spectrum.

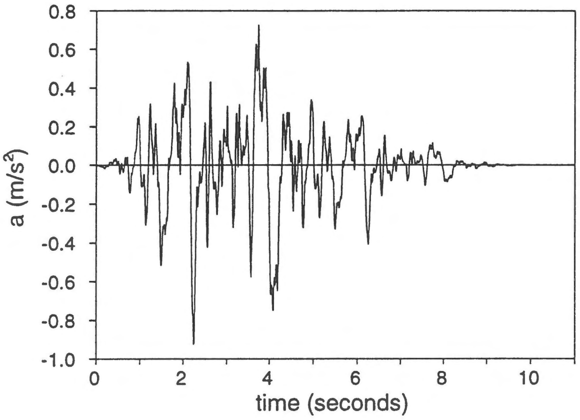

Fig. 3. Synthetic ground acceleration (a) record.

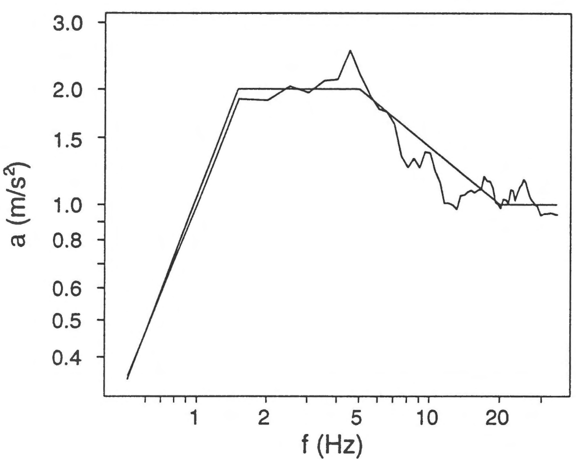

Fig. 4. Acceleration response spectrum (5% damping) of the synthetic ground acceleration record in Fig. 3. a = acceleration, f = frequency.

The thus obtained acceleration record was used in subsequent analyses.

General soil profile and local features

For the geotechnical analysis only the upper part of the soil is of interest. From cone penetration tests and borings performed in the Maasniel area for building projects, it was concluded that the upper 6–12 m of the soil consists of layers of sand, loam, clay and silty, loamy or clayey sand. Below this depth a stiff sand layer is present with a cone penetration resistance in the order of 30 MPa. Due to their alluvial sedimentary environment the different layers are not uniformly distributed over the area. Furthermore, a number of clay pits for brick-works are present near Roermond. Since the end of exploitation these pits were filled with very loose material. For the analysis a soil profile at the Sportlaan in Maasniel has been chosen (Table 1). This profile is considered representative for the studied area. In the past a clay pit existed at this location, which apparently was filled with loose material. The relevant soil parameters are assessed from cone penetration resistance, using empirical correlations (Richart et al. 1970, Lunne et al. 1983). The ground level is NAP + 22 m. The water table is taken at NAP + 20 m.

Table 1. Soil profile at Maasniel used in the analysis.

Description	depth (m + NAP)	q_c (MPa)	I_D (%)	γ_{ground} (kN/m^3)	G_i (MPa)
back-fill	22–20	1	30	15.5	30
back-fill	20–19.5	2	30	19.6	40
silty sand	19.5–17.5	5	60	20.2	56
silty sand	17.5–16	10	70	20.6	75
clayey sand	16–13.5	5	40	19.9	75
sand/gravel	13.5–	30	90	21.0	150

- q_c: cone penetration resistance
- I_D: relative density
- γ_{ground}: wet volumetric weight of the soil
- G_i: dynamic shear modulus at low shear strains (0.0001%)

I_D is defined as follows:

$$I_D = \frac{e_{max} - e}{e_{max} - e_{min}}$$

where:

- e: the ratio between volume of voids and volume of solids,
- e_{min}: void ratio corresponding to maximum density obtainable for given soil (minimum volume of voids),
- e_{max}: void ratio corresponding to minimum density obtainable for given soil (maximum volume of voids).

Densification of the soil

An earthquake can cause foundation damage both due to liquefaction of the subsoil (resulting in complete loss of strength) and to differential settlements. Bearing capacity failure in non-liquefied conditions may occur, but is relatively rare in an earthquake of magnitude 5.9. All these phenomena may occur during and shortly after the earthquake. Differential settlements can cause cracking of the foundation and the structure. Even when no immediate cracking occurs it is possible that the structure is stressed to such an extent that a future additional settlement due to an increase of the foundation load will lead to damage.

The phenomenon of liquefaction has received much interest in literature, whereas only a limited number of publications exists about the process of densification (e.g. Pyke et al. 1975, Tokimatsu & Seed 1987, Stamatopoulos et al. 1991). In fact both phenomena are related. During cyclic loading, as in an earthquake, the soil has a tendency to densify. When this process occurs within a relatively short period of time, drainage can be neglected and the net volume change will be close to zero. The tendency to densify is then counteracted by an increase in pore pressure, eventually leading to a complete loss of inter-particle stresses. After the earthquake, drainage will occur, resulting in densification of the soil and thus causing settlements.

In Roermond, sand, loamy sand and clay are present in the subsoil. For assessing the possible densification due to the earthquake use has been made of the graphs presented by Tokimatsu & Seed (1987). From these graphs the volumetric strain (ε_{vol}) can be assessed when the shear stress ratio ($\Delta\tau/\sigma'_v$) and the SPT-value (from the Standard Penetration Test) are known. The graphs are based on results of cyclic tests on different types of sand and have been proven to give satisfactory results. The original graphs are valid for an earthquake of $M_L = 7.5$ and correction factors for other earthquakes are given by the authors. In the graphs presented here for the sand above and below the groundwater table (Figs 5, 6) the correction factors for an earthquake of $M_L = 5.9$ have already been incorporated. Furthermore, a relative density instead of an SPT-value is used for the horizontal axis in Fig. 6.

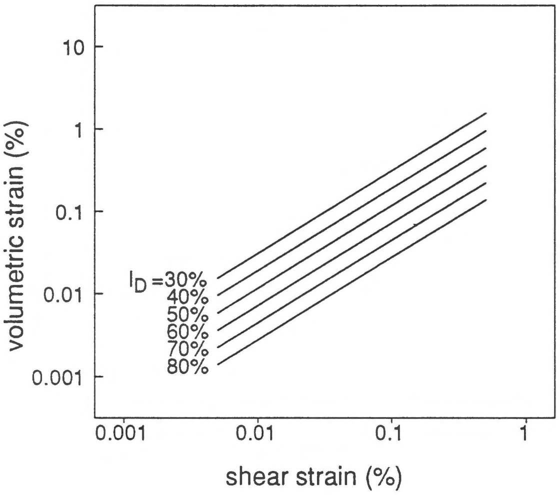

Fig. 5. Volumetric strain of sand above the groundwater table as a function of shear strain after Tokimatsu & Seed 1987), (modified for $M_L = 5.9$; I_D = relative density.

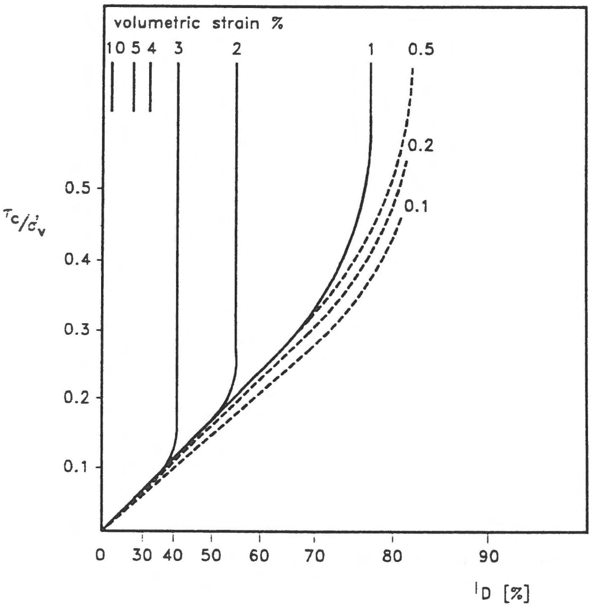

Fig. 6. Volumetric strain of sand below the groundwater table after Tokimatsu & Seed 1987), (modified for $M_L = 5.9$; τ_c = amplitude of the shear stress, σ'_v = initial vertical effective stress, I_D = relative density.

No relevant publications are known to us about densification of loamy sand. It is expected that the degree of densification in this type of sand lies between that of clay and clean sand. The volumetric strain in clean sand is therefore considered to be an upper bound value for the volumetric strain in loamy sand. Evidence from the Roermond earthquake confirmed this. Cone penetration tests performed in the Maasniel area indicate a rather low relative density in some of the loamy sand layers. In the case of clean sand with such a low density, large volumetric strains and subsequent settlement would be expected. As this has not been reported, it can be concluded that the densification will be less in loamy sand than in clean sand. The volumetric strain in clay is very small compared to the volumetric strain in sand and can therefore be neglected (Jagau & Gudehus 1991, Ohara & Matsuda 1988).

Table 2. Calculated peak accelerations and settlements in Maasniel due to the 1992 Roermond earthquake

depth (m + NAP)	a_{max} (m/s^2)	$(\tau_c;\sigma'_v)_{av}$ (–)	ε_{vol} (10^{-4})	Δz (mm)
21.75	0.9	0.059	0.5	0
21.0	0.89	0.058	1.0	0.1
20.25	0.84	0.056	1.4	0.1
19.75	0.83	0.059	24	1.2
19.0	0.82	0.066	0.07	0
18.0	0.78	0.072	0.17	0
17.0	0.72	0.074	0.05	0
16.25	0.66	0.075	0.05	0
15.75	0.63	0.075	14	0.7
15.0	0.64	0.074	13	1.3
14.0	0.66	0.073	11	1.1
13.0	0.65	0.071	0.00	0
12.0	0.63	0.068	0.00	0
11.0	0.63	0.068	0.00	0

– NAP: Normaal Amsterdams Peil
– a_{max}: peak ground acceleration at depth
– $(\tau_c/\sigma'_v)_{av}$: average shear stress level (amplitude of shear stress divided by initial vertical stress)
– ε_{vol}: volumetric strain
– Δz: settlement of the considered layer.

Ground acceleration and volumetric strains at depth

The response of the subsoil has been determined using the computercode SHAKE (Schnabel et al. 1972). This is a one-dimensional program for the propagation of shear-stress waves in the soil. The program uses strain-dependent soil parameters. In the calculation the acceleration record as shown in Fig. 3 has been used as input. The program calculates the response (displacements and stresses) in the subsoil, taking into account the shear strain in each layer. The results of the calculation are shown in Table 2. Summation of the settlement (Δz) for all layers reveals that the maximum expected settlement at the Sportlaan in Roermond is 5 mm. This settlement is acceptable according to the Dutch regulations for foundations. As pointed out before, it is expected that in loamy sand the settlement will be less than in clean sand. The assessed settlement is therefore considered as an upper bound value. It is interesting to notice that most of the densification occurs in loosely packed sand below the groundwater table. Figure 6 shows that for certain combinations of relative density and shear stress ratio (which is directly related to the PGA) the amount of settlement is very sensitive to small changes in these values. At locations with very loose sand (relative densities $\leq 20\%$) or a larger PGA than used in this study, significantly higher settlements are to be expected. Locations with such loosely packed sand are, however, not expected to be present at Roermond

Conclusions

The analyses described in this paper show that no excessive settlement is to be expected at the location considered. However, attention should be paid to buildings at the edges of former clay pits, where the subsoil below one part of the building may differ significantly from that below another part. In this case differential settlements are to be expected. The same holds for buildings at other places with strongly varying subsoil conditions, e.g. the edge of a former stream or an old road.

In our study a PGA of 0.09 g, as assessed from the observed damage, has been used. According to Gariel et al. (1994) the PGA may have been in the order of 0.15 g. At locations with this PGA and clean sand with a relatively low density, significant settlement is to be expected. On the other hand, for a PGA of 0.09 g, no significant settlement is to be expected in soils with a relative density above 40%.

As loose loamy sand is a very common soil type in the south-eastern Netherlands, which is also the country's most earthquake-prone area, an investigation into the densification characteristics of this type of soil would be of much interest.

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