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P. M. Maurenbrecher
TU Delft, The Netherlands

A. Den Outer
TU Delft, The Netherlands

H. J. Luger
Delft Geotechnics, The Netherlands

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Review of Geotechnical Investigations Resulting from the Roermond April 13, 1992 Earthquake

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P.M. Maurenbrecher and A. Den Outer
Lecturer and Ph.D. Researcher, TU Delft, The Netherlands

H.J. Luger
Project Engineer Delft Geotechnics, The Netherlands

SYNOPSIS In 1987 the Engineering Geology section of the Delft University of Technology carried out a survey of the SE Netherlands to determine which areas were susceptible to liquefaction based on soil profile, groundwater levels and a Richter scale magnitude 6 earthquake along the principal rift fault through the Netherlands, the Peelrand fault system. The fault system has been active since the Triassic and forms part of the Rhine-North Sea rift system. The last major earthquake along the Peelrand fault was in 1933. Recently, in 1992 a 5.8 magnitude earthquake occurred at Roermond, near to the Dutch-German border. Though damage resulting from the earthquake was limited, remedial works to structures amounted to US$ 50 million in the Netherlands.

The paper reviews geotechnical investigations associated with the earthquake carried out in the Netherlands. Much of the damage is attributed to liquefaction; excess pore pressures resulting from the earthquake caused sand vent eruptions, river-dyke failures and slope failures. Comparisons are made between the predictions of 1987 and that which occurred in 1992. Site investigation works are recording geotechnical and building data so as to allow for correlations between extent of damage, ground geotechnical profiles and building design. Models for liquefaction are reviewed to describe the slope failure as well as the sand vent phenomena. Densification of subsoil has been inferred from CPTs taken before and after the earthquake for some sites. Pile foundation damage has been investigated for buildings in Roermond for which their susceptibility to earthquake lateral forces in terms of stiffness and pile head working load is given.

INTRODUCTION

At 03:30 hrs early Monday morning of the 13th April 1992 an earthquake of magnitude 5.8 on the Richter scale occurred in the southeastern part of the Netherlands near the border with Germany. The earthquake is the strongest ever recorded in the Netherlands and caused damage amounting to $50 million. Only one death, due to heart failure, has been attributed to earthquake, in Germany. The low human injuries resulted from the earthquake happening during the most quiet period of the week; early Monday morning when even the night life in the Netherlands is at its lowest. No occurrence of building collapse has been recorded which may be the other factor limiting human injuries. Buildings, though, were damaged. The damage required, in most cases, repairs to cracks, though buildings in Herkenbosch, Roermond and Heinsberg and villages nearby damage to many building consisted of some form of collapse: chimneys, gables, ceilings and roof tiles. Had the earthquake occurred during lunch time on what was otherwise a relatively sunny spring day many pedestrians may have fallen victim to the falling debris.

Earthquake phenomena is not new to the Netherlands, though the scale and timing of the Roermond Earthquake was unexpected. The last major earthquake was that of Uden in 1933 (5.5 on the Richter scale), the epicentre on the same Peelrand fault-line system as that for the Roermond Earthquake but 60 km to the northwest. These earthquakes have been estimated to have a recurrence period of 120 to 140 years. Research into earthquakes is not new in to the Netherlands. The number of researchers in this field is though very limited; the seismological office of the Royal Meteorological Institute in de Bilt and a number of researchers at the University of Utrecht Geological Institute and at the Delft University of Technology Engineering Geology Section. Research at Delft had been active in this field a number of years investigating the susceptibility of "foundation zone" deposits to liquefaction in the southeastern part of the Netherlands assuming an earthquake epicentre along the Peelrand fault of magnitude 6 on the Richter scale. The location map in Fig. 1 shows the susceptible areas (Lap 1987) and the significant geographical locations with respect to the Roermond Earthquake.

POST EARTHQUAKE INVESTIGATION

The Roermond Earthquake has motivated further research activity to both attempt to record and relate damage of buildings as a result of the response of these buildings and the ground they are founded on to the various forces generated by the earthquake. In addition ground disturbances attributed to liquefaction manifested itself as sand eruptions, slope failures and embankment collapse. A symposium was held nine months after the earthquake to allow presentation and discussion of initial studies and initiatives on the earthquake. The proceedings on the symposium will be published shortly, (van Eck, 1994). Various groups were formed such as the seismological, information and geotechnical to ensure subsequent to the symposium contact between the researchers is maintained. The geotechnical and information groups are sustained by TU Delft Engineering Geology Section. Activity on the information group has recently been reported at the GeoInfo V conference held in Prague in June 1994 (den Outer et al., 1994) in which present information storage and anticipated development are outlined.

The geotechnical group activity subsequent to the symposium has been limited to compiling and submitting research proposals in cooperation with many institutes in Germany, the Netherlands, Belgium and England for European Commission finance. Though unsuccessful the initiatives maintain contact with researches in the Roermond Earthquake.

Geotechnical work on the Roermond earthquake in the Netherlands has been limited to a few investigators within two Delft institutes: Delft Geotechnics which has carried out investigations on behest of the municipality of Roermond and the Engineering Geology Section of the Technical University Delft as a continuation of earlier research and within university funded engineering geology mapping project for the province of Limburg (south east Netherlands) in which the earthquake occurred.

The work from Delft Geotechnics investigated possible unseen damage to foundations of houses both on shallow and on piled foundations. The area that was particularly of concern by the municipality of Roermond is along an old tributary of the River Roer to the east of the city and corresponds to the high liquefaction potential (shaded) area shown in Fig. 1.

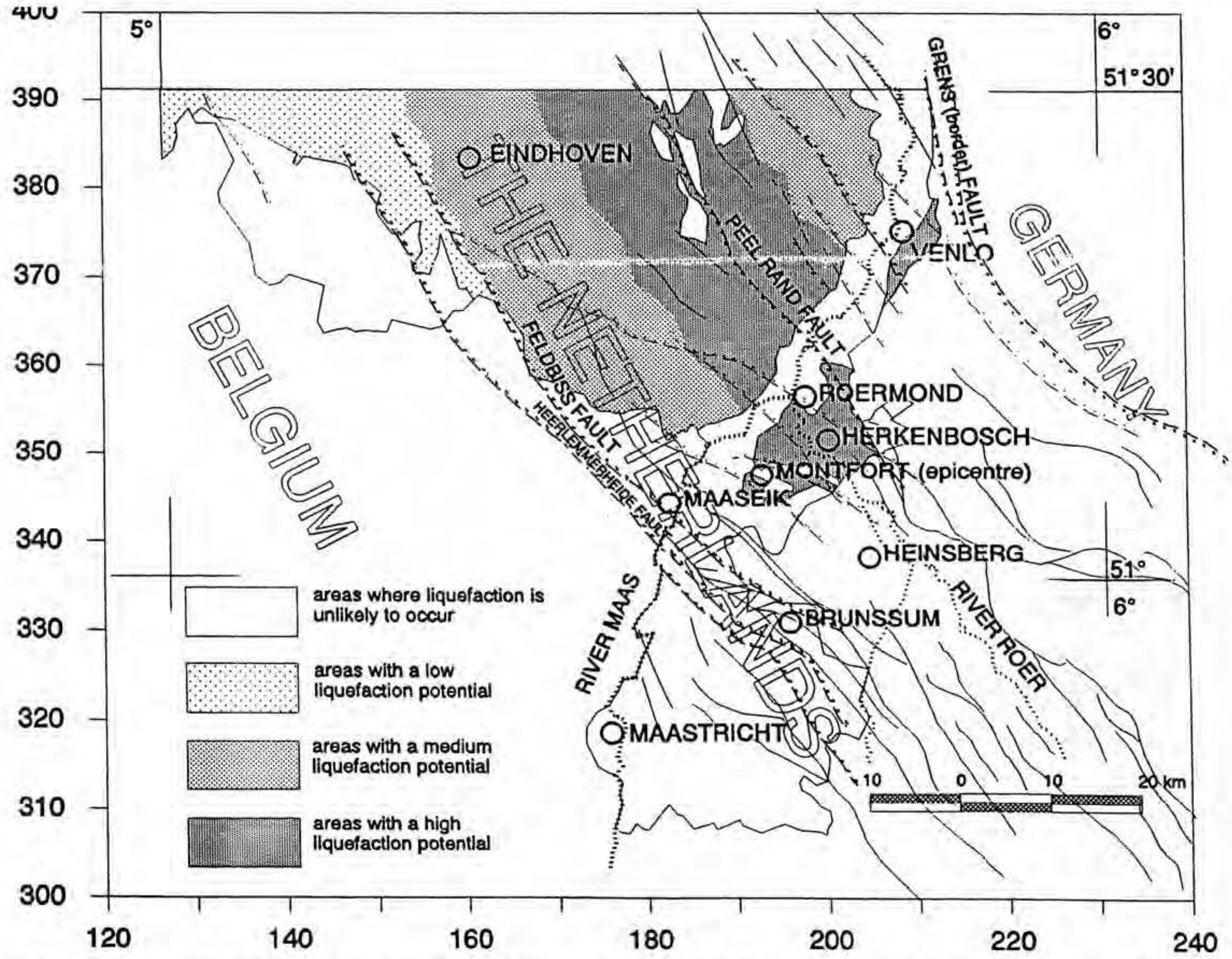

Fig. 1. Liquefaction potential map of southeast Netherlands for Richter Scale magnitude 6.0 along the Peelrand Fault (Lap, 1987).

The work from the TU Delft Engineering Geology Section as a consequence of the earthquake investigated two locations: the relatively heavily damaged town of Herkenbosch, sand eruptions in fields located south of the town and slope failures at Brunssum (see fig. 1 for the locations).

Traditionally all subsoil investigation for foundation zone in the Netherlands is carried out by the (static or "Dutch") cone penetration test (CPT). For more information on this type of test and its comparison with other penetration tests such as the standard penetration tests (see de Ruiter, 1988). In all the instances of the above investigations use is made of CPT data to determine the response of the ground to earthquake accelerations.

FOUNDATION SUSCEPTIBILITY TO DYNAMIC LOADS

The most common application of the cone penetration test (CPT) is to determine the soil profile and its load bearing properties for a foundation. The point resistance together with the friction resistance allows an accurate determination of the soil type[1]. In the Netherlands this is generally done for every pile location below a foundation to find the end-bearing layer of the pile. It is this standard procedure that allows for estimating foundation susceptibility to dynamic loading [*Luger et al., to be published; Meijers et al., to be published*].

In the city of Roermond (the Maasniel district) CPT analysis, verified with borings, shows a 6 to 12 m thick layer of varying soil types, eg. predominantly fluvial deposits of low density or strength varying in grain size from sands to silty clays. Below these deposits further over-consolidated fluvial deposits consisting of dense sands and gravels are encountered causing a CPT resistance of 30 MPa. The upper part of the loose/ weak layers has been further disturbed by clay exploitation for brick making. The pits have been generally infilled with building debris and refuse.

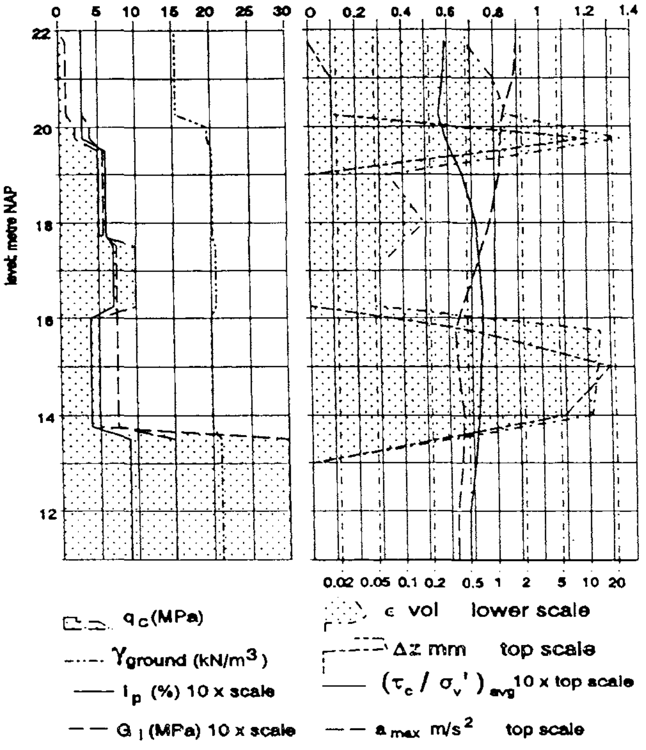

Fig. 2. Soil profile data and calculated values

Fig. 2 shows the various profiles of the CPT end bearing, q_c,

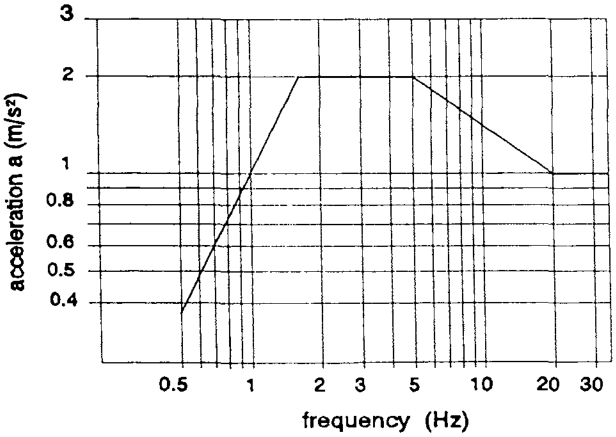

Fig. 3. Ground acceleration response spectrum (Hosser 1987)

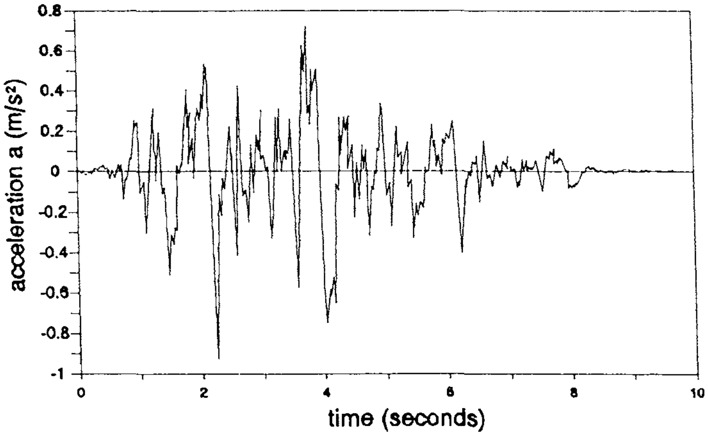

Fig. 4. Generated acceleration record

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[1] [In fact several researchers have determined such correlations (Schmertmann, 1977, Searle, 1979 and Olsen & Malone, 1988). Searle's chart, the most ambitious of the three, allows, just about, every soil parameter to be derived from the point resistance/ friction ratio relation and is shown as part of fig. 6. The more recent chart from Olsen and Malone allows a more limited selection of more descriptive soil parameters, allowance is made for insitu stresses, and hence can be considered the more

and the interpreted soil parameter values: in-situ relative density (I_D) and the dynamic shear modulus (G) of the layers. These values are obtained from a knowledge of q_c and the types of soil from borehole logs using correlations based on Lunne et al.,1983 and Richart et al.,1970. In this way the relevant parameters are obtained to allow estimates to be made of densification and the strain of each layer under dynamic loading.

The program SHAKE (Schnabel et al. 1972) is used to predict the ground motion due to dynamic loading. In addition to the geotechnical profiles the program requires some characteristics of the signature of the earthquake. The length of period of strong motion has been estimated from the earthquake magnitude at 10 seconds, with a maximum acceleration of 0.9 m/s^2. For the area under investigation the ground acceleration response spectrum, with predominant frequencies between 3 and 5 Hz, was obtained from Hosser,(1987), see fig. 3. The inverse Fourier transform of this response spectrum, adapted for 5% damping and random phase behaviour in the time domain, will give the earthquake signature that is needed for the program SHAKE.

Using the output of the program SHAKE (displacement and stresses of the subsoil) the shear strain and therefore the volumetric strain (see soil response in fig 2) is obtained, which result is an estimate for the settlement of the layers (5mm). It was found that there is no excessive settlement expected at the considered location, however, differential settlement between back-filled pits and their surroundings should be watched for. The maximum ground acceleration might have been higher than 0.9 m/s^2, which can give problems for soils with a lower relative density than 40%.

The resulting calculated data from SHAKE was input into a programme TILLY (DUT 1992) for estimating lateral pile displacement and soil reaction force based on P-Y analyses in accordance with API (1989 edition). The key input data and main results of the analyses are given in fig. 5. for a bored pile 11 m long and 0.3 m diameter and having a de-sign load of 30 MN. The effect of eccentric loading causes a reduction of the maximum allowable bending moment. The calculation show that the lateral loading bending moment exceed those of wind loading by a factor of two. There is a possibility that critical overloading could have occurred. Further investigation, in the field, is required to verify any damage.

SLOPE STABILITY AND SUSCEPTIBILITY TO DYNAMIC LOADS

A very important use of the relative density obtained from the CPT test is the determination of possible liquefying layers in the subsoil. The methods often used to obtain susceptibility to liquefaction are Tsuchida (or more popular Seed and Idris. Both methods use the SPT-N value to express the influence of the relative density on the process of liquefaction. The dynamic character of the SPT-N is seen as an advantage to express the dynamic behaviour of the subsoil. However, the dynamic energy is small and of a specific energy content (mass of falling driving weight). The discontinuous character of the test is a real disadvantage while the transitions of the individual layers might lie within one SPT-N value determination.

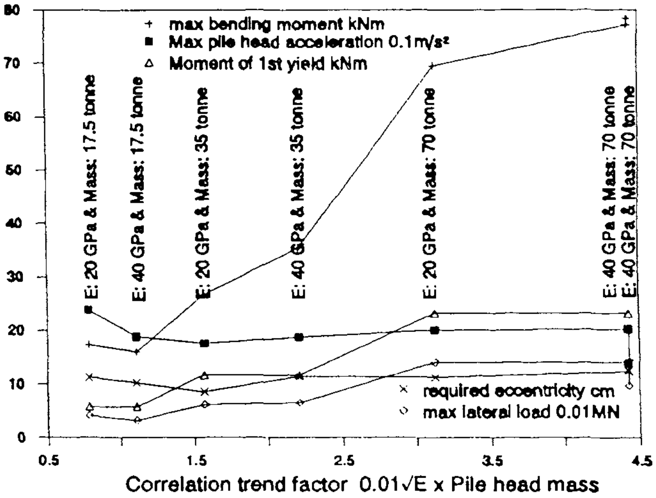

Fig. 5. Pile calculated values

There are very good correlations of the CPT values with the SPT-N values (Burland et al.,1985) and the continuous character of the CPT test is an advantage over the SPT-N.

Recent research (Mwingira,1994) has shown that the CPT can be converted well for the standard CPT-cone of 60° depending on the D_{50} of the soil grain size. Additionally a small correction for the overburden stress has to be done to make the conversion complete.

After obtaining the SPT-N values from the CPT test the SPT-N values for the Tsuchida method or the relative density for the Seed and Idris method can be fully implemented to get the final estimate of the liquefaction susceptibility. The kinematic processes and geotechnical profile are shown in fig. 6 from (Maurenbrecher et al 1994).

Using the cone end bearing values, q_c of the CPT the soil profile values are determined for input into the susceptibility analysis based on that of Seed et al. (1983). The analysis showed that the slope needed only very small acceleration of 0.06g to become unstable. This would correspond to the likely acceleration from the earthquake. Brunssum was not classified by Lap (1988) as an area susceptible to liquefaction (see fig 1). The area has been prone to liquefaction slides due to high pore water pressure in a loose sand spoil derived from lignite mining. Further more the area is the source of the Roodebeek stream with the springs and associated quicksands within the vicinity of the failed slopes.

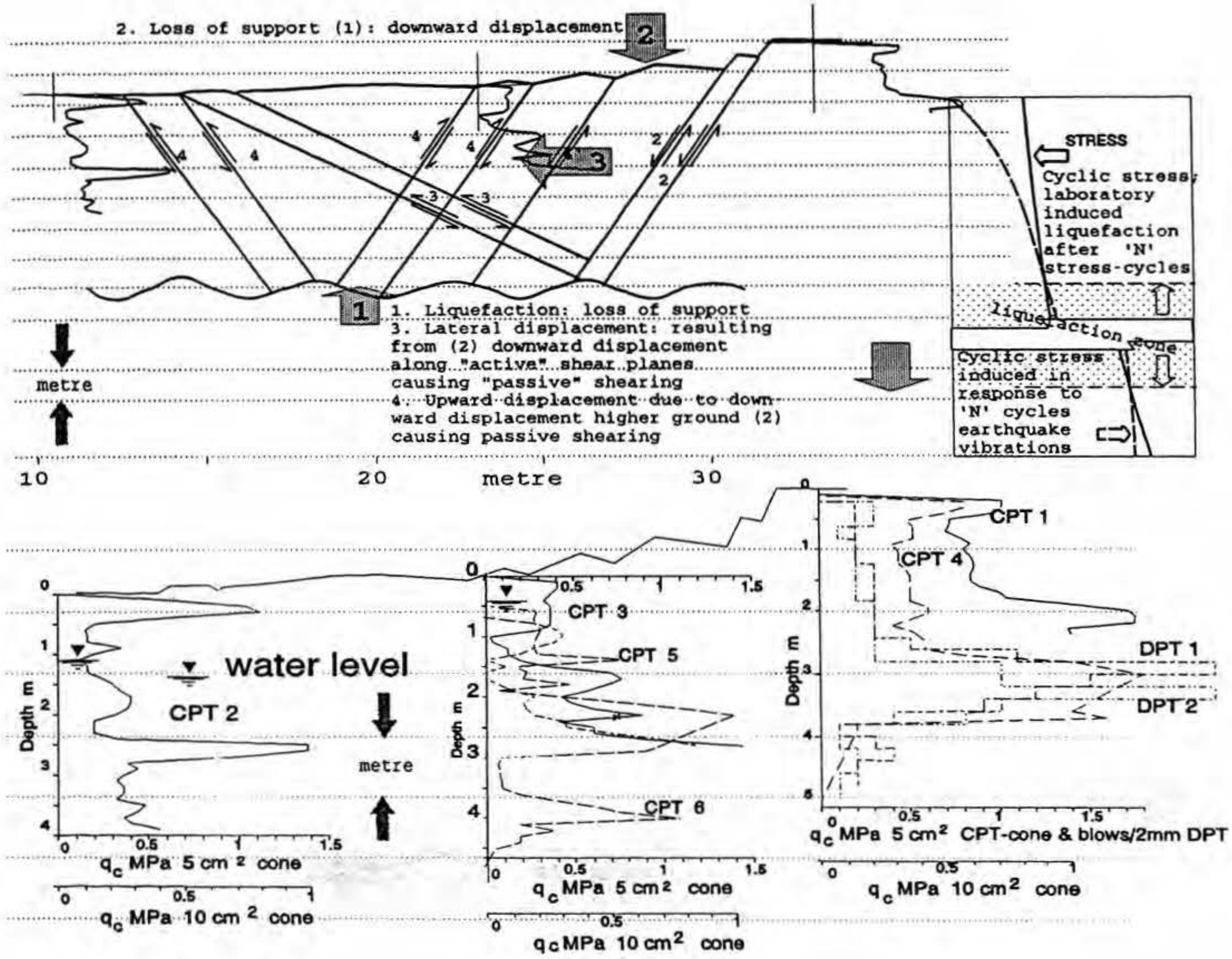

Fig. 6. Brunssum slope liquefaction kinematics & geotechnical profile

HERKENBOSCH SAND-WATER VENTS

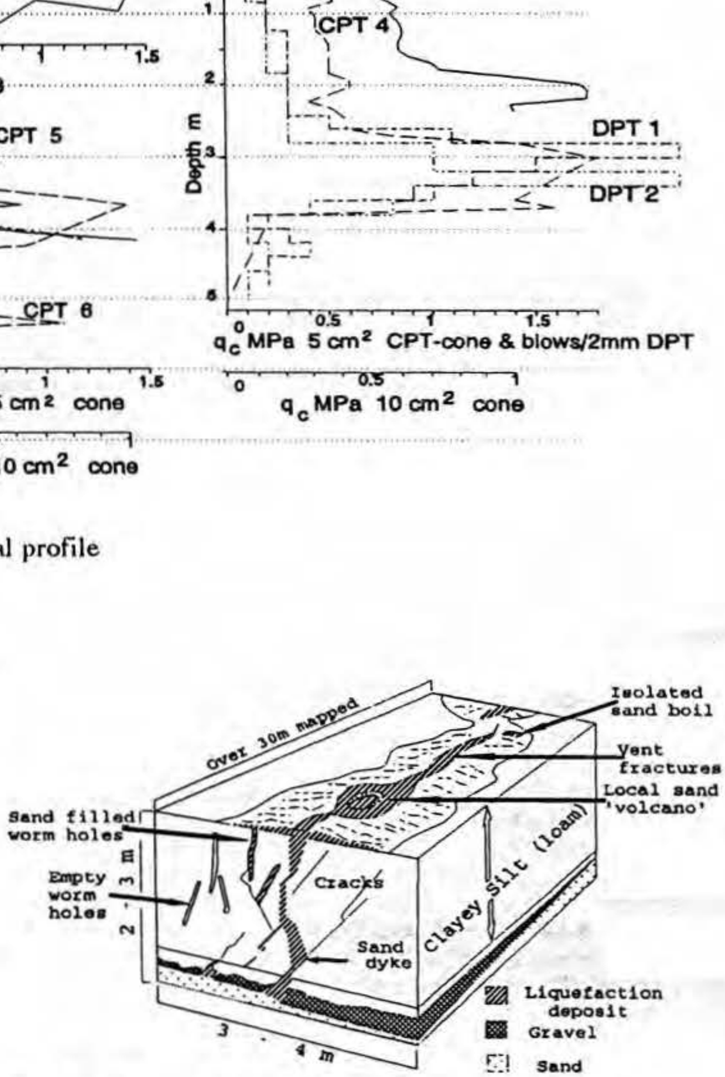

Fig. 7. Composite block diagram of liquefaction structures, Herkenbosch

Substantial water-sand outflow occurred from linear fissures in the ploughed fields between the town of Herkenbosch and the Rive Roer. Excavation revealed cracks infilled with sand extending to a sand source layer two to three metres underneath the top layer consisting of slightly clayey silt (fig. 7). The latter is quite impervious as the excavations were relatively dry despite a preceding very wet autumn period. Davenport et al. (1994) attribute the fissures to follow a brittle type failure and have the same pattern as subjecting a cake of clay to tension and are characteristic of surface strains associated with flow-type failure at depth. At least two metres of excess pore water pressure was required to cause the water to reach the surface for a piezometric head estimated at two meter below ground level. This is probably caused by densification of the sand and sandy gravel layers beneath (see fig 8) resulting in a volume decrease forcing water to escape. The fissuring process providing escape routes for the water are believed to result from horizontal displacements of the higher Roer river flood plane terraces moving towards the lower lying flood plane levels. Densification is suggested by CPTs at a location in Herkenbosch taken before and after the earthquake (fig 9). This however must be checked and verified by other tests carried out with a large time span between the

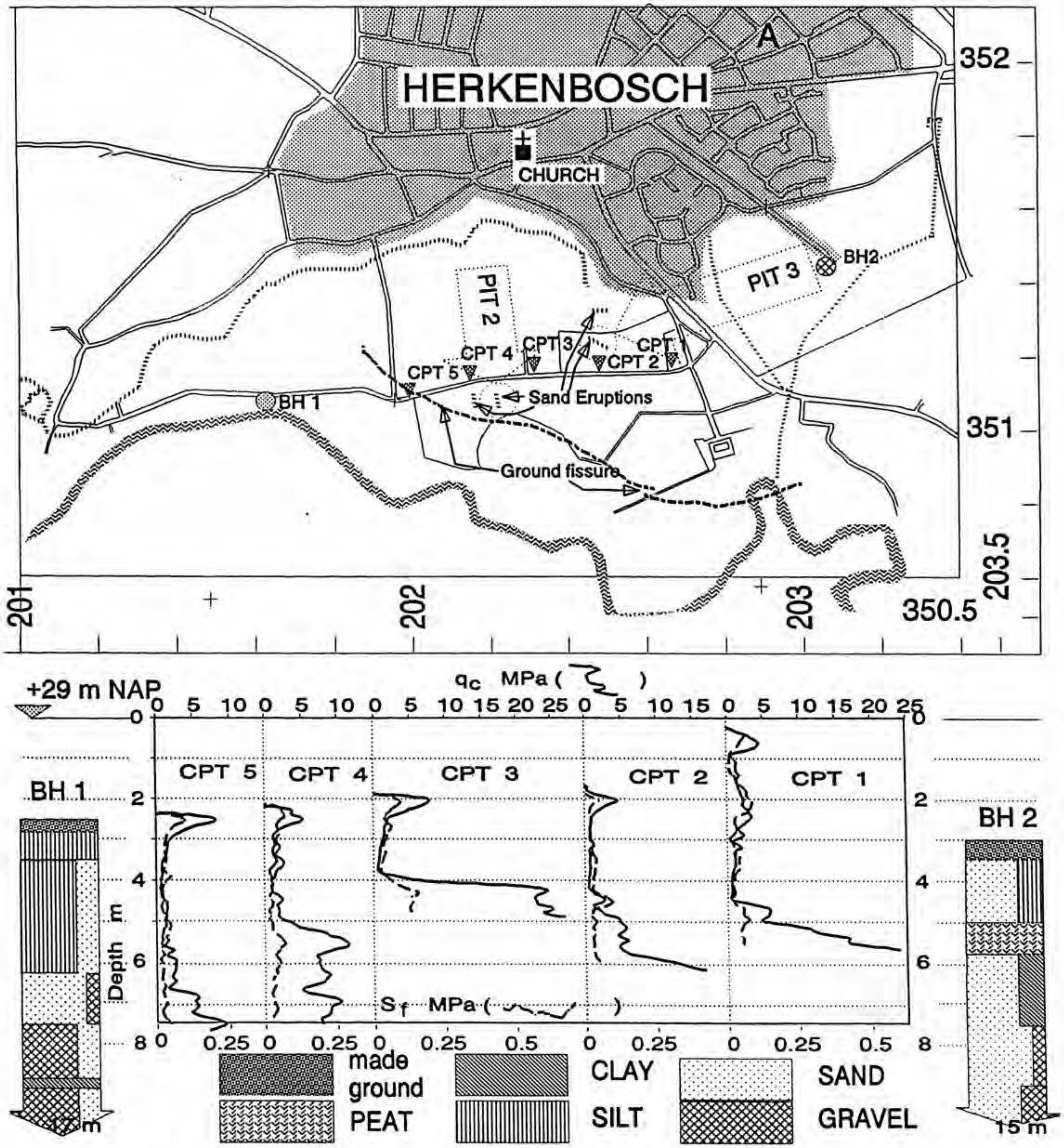

Fig. 8. Herkenbosch: Soil profiles and ground fissuring extent at Herkenbosch

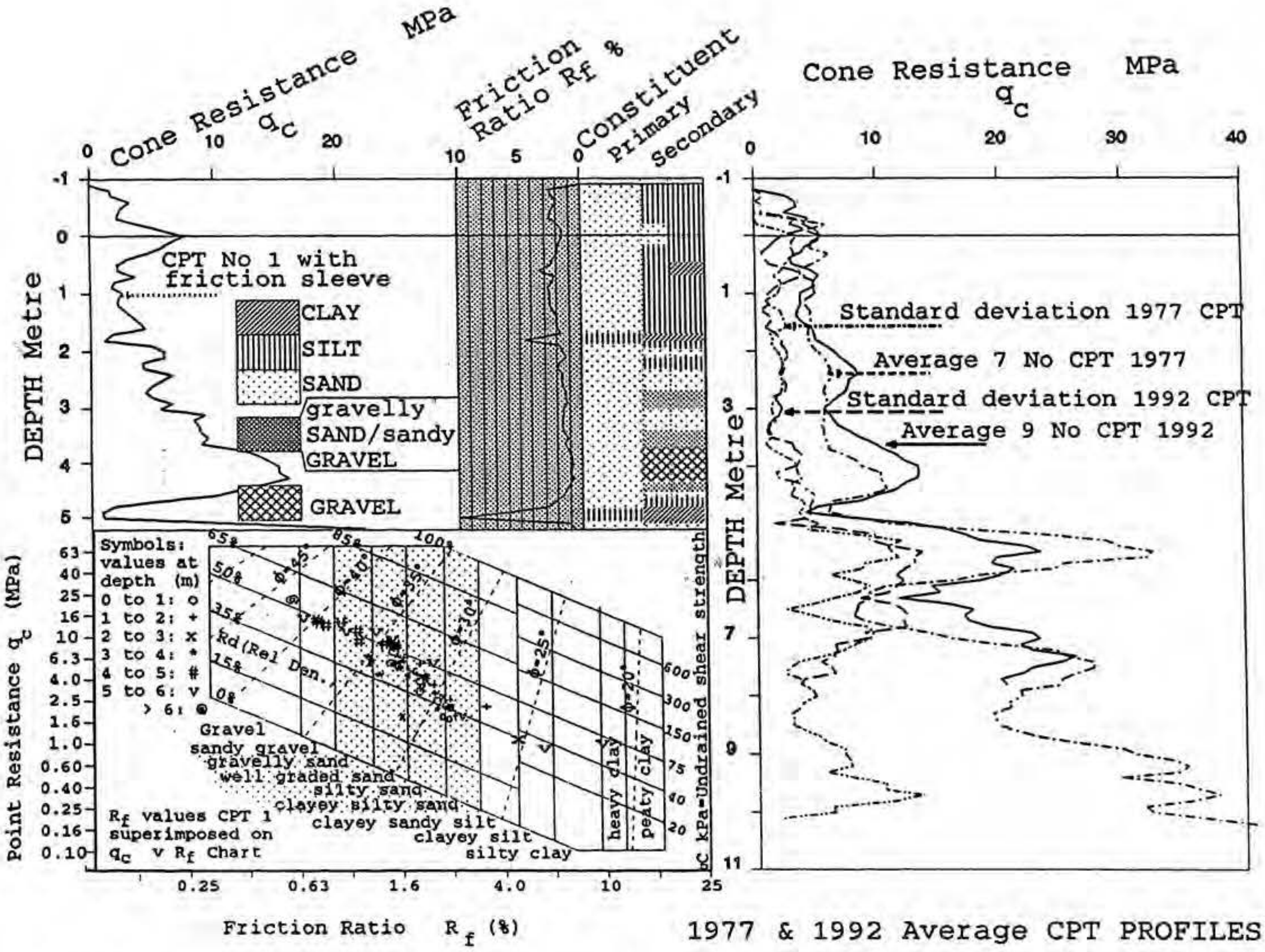

Fig. 9. CPT profiles location A (fig. 7) before & after earthquake

tests (25 years) without an earthquake event to determine other possible causes of densification such as the history of effective stress fluctuations due to foundation loading and ground water level changes.

Through a questionnaire damage survey and the compilation of geotechnical maps a better understanding of the relationship between soil profile/ foundation and structure behaviour is intended for the town of Herkenbosch (Outer and Maurenbrecher, 1994) and (de Vries, 1994).

FURTHER RESEARCH

The Netherlands counts few researchers in the field of earthquake geotechnics as major earthquake events seldom occur. Hence the approach to analyzing the geotechnical consequences of such an earthquake may seem to full-time earthquake geotechnical researchers from countries where earthquakes are a continual hazard as unusual. Despite this, tremors do occur regularly in the vicinity of the Roer River graben (tectonic) and also due to extraction of hydrocarbons elsewhere in the Netherlands. Hence the need, despite relatively high quality construction, exists to be able to determine the potential damage that earthquakes can induce on structures, not only for insurance accessors but also for safety reasons. The largest obstacle to research is finance. Injuries and loss of life often spur contributions. The Roermond Earthquake, despite being a 1 in 130 year event (Crouzen, 1993), seemed to choose a time of the day when such injury would be kept at a minimum. Only six months later a comparable earthquake both in geotechnical environment and strength occurred in Cairo causing 200 deaths. The timing was in the afternoon and the probably inferior structural integrity of the buildings caused the higher loss of life. A great advantage over Egypt is the lesser damage to buildings does not obscure as much the initial weakness of buildings and as there was no loss of life so that information from residents is more complete. Furthermore site investigation frequency for buildings and contamination studies are done relatively frequently with generally accessible archives of the information allowing detailed studies of soil profile distribution. Such arguments seem to be lost on funding organisations such as the European Union research programmes, which are more likely to provide funds to more vulnerable areas which in the longer term will not necessarily produce the right solutions.

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