Large Horizontal Displacements of Houses in Rotterdam

G. Hannink
Delft Geotechnics, The Netherlands

A. F. van Tol
Rotterdam Public Works, The Netherlands

Proceedings: Second International Conference on Case Histories in Geotechnical Engineering, June 1-5, 1988, St. Louis, Mo., Paper No. 6.68

Large Horizontal Displacements of Houses in Rotterdam

G. Hannink
Project-engineer, Delft Geotechnics, Delft, The Netherlands

A.F. van Tol
Head of Geotechnics Department, Rotterdam Public Works, Rotterdam, The Netherlands

SYNOPSIS: In 1983 it was established that six blocks of terrace houses in Rotterdam had undergone large horizontal displacements. These displacements were caused by insufficient stability of the adjacent quay and as a result one of the blocks had moved as much as 2.5 m since 1958. The foundation piles of the houses were not designed to resist any horizontal loading. As a result of these large horizontal movements the piles had deflected to such an extent that complete failure was feared. This paper describes the remedial measures that were taken to improve the stability of the quay and foundations of the houses. The present displacement behaviour is compared with the horizontal displacement predicted from creep analysis.

INTRODUCTION

In 1983 the Road Management and Surveying Department of Rotterdam Public Works discovered that six blocks of, in total, 41 terrace houses at the north side of a quay, the Zestienhovensekade, had undergone large horizontal displacements, Figure 1. Research by the Department on previous measurements indicated that the horizontal displacements, given in Table I, must have occurred since the houses were constructed. The degree of accuracy of the absolute displacements up to 1975 was estimated at within 0.1 m. The degree of accuracy of the displacements in the periods 1975 - 1983 and 1983 - 1984 was estimated at within 0.02 m. To be more certain about what appears, at first sight, to be incredible figures, the magnitude of the displacements was also established from aerial photos taken for mapping purposes. Based on these measurements, it was concluded, beyond doubt, that large displacements had occurred, and that the maximum rate of displacement was about 0.10 m/year.

TABLE I: Horizontal displacements since construction

Block	House No.	Year of construction	Horizontal displacement (m)		
			up to April 1975	up to Sept. 1983	up to July 1984
0	439	1962	0.8	0.865	0.865
0	447	1962	0.3	0.310	0.310
1	449	1957	0.5	0.525	0.530
1	473	1957	1.3	1.505	1.520
2	475	1958	1.7	2.070	2.150
2	487	1958	1.8	2.440	2.540
3	489	1938	1.2	1.525	1.560
3	499	1938	0.8	1.065	1.100
4	501	1935	0.4	0.460	0.465
4	507	1935	0.1	0.210	0.210
5	509	1938	0.5	0.630	0.630
5	519	1938	1.5	1.730	1.750

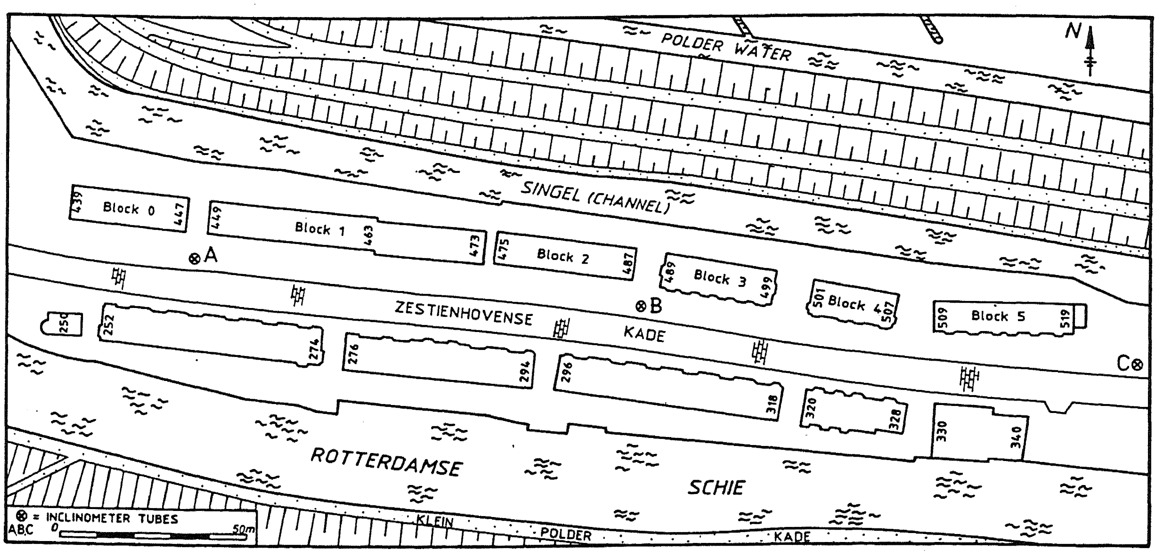


Fig. 1 Situation

Cause of the displacements

The cause of the displacements must be sought in a combination of the following circumstances, Figure 2:

- the level of the ground surface in front of the houses and behind the houses differs by 2.5 m to 3.0 m
- fill with a unit weight of 18 to 20 kN/m³ occurs at the front of the houses down to a level of between 5 m and 6 m below the ground surface and peat with a unit weight of about 10 kN/m³ occurs at the back
- the clay and peat layers under the houses are very soft and have unfavourable friction properties

The effect of displacement on the houses

The first calculations led to the following conclusions for Blocks 1, 2, 3 and 5:

- the foundation piles offer no resistance against the horizontal movements of the quay
- the stresses in the foundation piles have far exceeded the allowable values
- the remaining safety margin of the foundation is difficult to assess. Complete failure in the near future must be feared unless remedial measures are taken.

Remedial measures

Measures which would eliminate the cause of the displacements were given high priority. Two possible solutions were presented:

1. The removal of the inciting load by excavating about 2.5 m of the heavy quay material and replacing it by lightweight material. This solution will be explained below.
2. The absorption of the load by installing an anchored sheet piling along the front side of the blocks. This solution was subsequently not found to be technically feasible and is not discussed further.

An inspection of the condition of the foundations was necessary, which included investigating the top of the foundation piles. It was planned to repair the foundations of the six blocks, if shown to be necessary, after implementing the remedial measures for the quay. The study of the measures was backed by extensive soil investigations and displacement measurements. The soil investigations included field and laboratory work. The field work consisted of cone penetration tests and borings in several cross-sections of the quay as well as the installation of piezometers, and measurements of the pore-water pressure. Cell tests and consolidation tests on undisturbed samples taken in the cross-section with the largest displacements, were carried out in the laboratory. Measurements of the horizontal displacements of the blocks were started on 13th July 1984. Initially these measurements were performed weekly, and then monthly. The vertical displacements of Block 2 were measured from 12th February 1985, and of the other five blocks from 25th March 1985. The frequency of these measurements was almost the same as for the measurements of the horizontal displacements.

IMPROVEMENT OF THE STABILITY OF THE QUAY

Figure 2 shows the composition of the subsoil in a cross-section of the quay. The subsoil consists of soft holocene clay and peat layers to a depth of 16 m below New Amsterdam Level. Below this level fine to rather coarse pleistocene sand is found. The in-situ pore-water pressure was measured in front of Block 2 and an excess pore-water pressure of about 25 kN/m² was found between 10 to 13 m below New Amsterdam Level. The soil investigations confirmed that the holocene layers have very low shear strength. The lowest values occur behind the blocks where the overburden pressure is small. A preliminary stability analysis showed that a slip circle through the soft clay layers around 11 m below New Amsterdam Level, which takes into account the excess pore-water pressure, gives the lowest factor of safety.

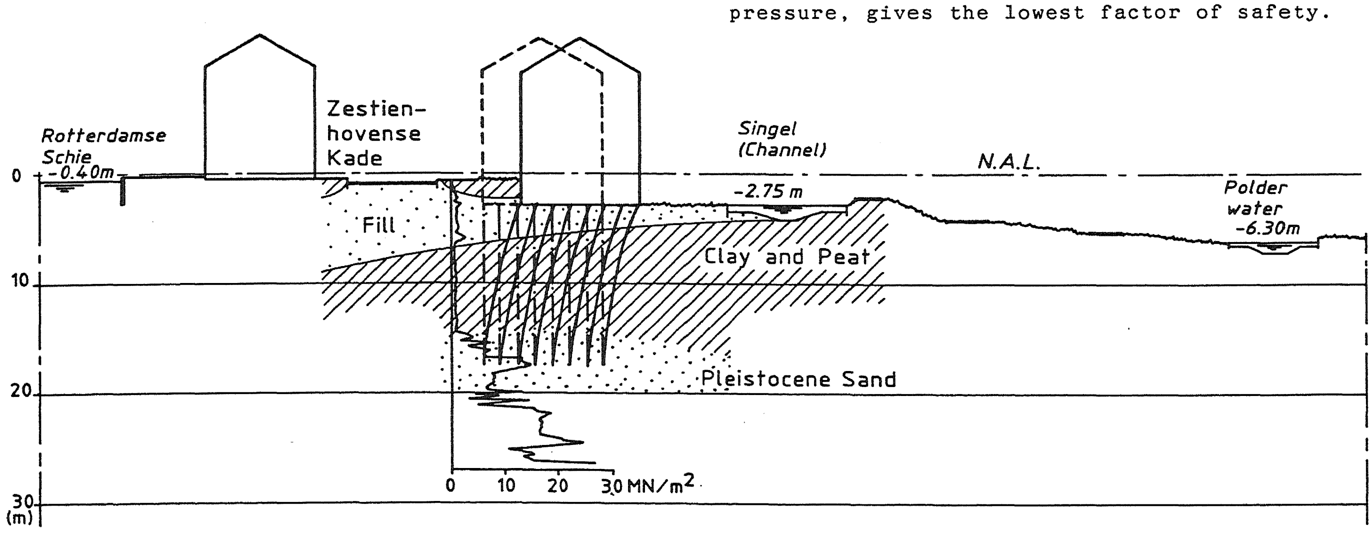


Fig. 2 Cross-section of the quay

Remedial measures

The remedial measures selected, to ensure the stability of the quay, are as follows. Over a total length of 300 m the existing fill was removed and replaced by lightweight material, Figure 3. The original fill had a unit weight of 18 to 20 kN/m³. The backfill consists of light expanded clay aggregate with a unit weight of 6 kN/m³ and polystyrene foam with a unit weight of less than 1 kN/m³. Polystyrene foam and blast furnace slag was placed underneath the road to ensure an adequate foundation. With these measures the surcharge on the subsoil was reduced by 35 kN/m² over a width of at least 12 m. To evaluate the effect of the remedial measures and to find the reasons for the large displacements of the quay, it was necessary to establish the deformation behaviour and the magnitude of the factor of safety in the present and future situation. Two types of analyses were carried out:

- stability calculations according to Bishop's method
- calculations with the Finite Element Method.

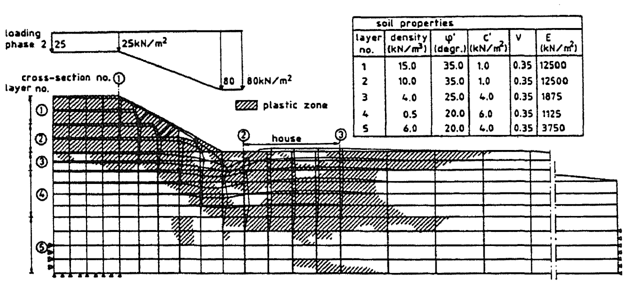
Bishop's Method

An analysis of the stability of the situation before remedial measures were taken gave factors of safety which varied from 0.82 to 1.04 depending on the assumed pore-water pressure conditions under and behind the houses. All slip circles go through the deep clay layer. The maximum depth of these slip circles is about 11 m below New Amsterdam Level. The results of stability calculations for the situation after the remedial measures show that, under fully drained conditions, the long term safety factor is about 1.5 to 1.6. If, however, it is assumed that, immediately after the measures are implemented, there is still excess pore-water pressure underneath the houses, the factor of safety falls to between 1.2 and 1.3. The required factor of safety for canal and river dykes in The Netherlands is 1.3. Therefore it can be concluded that the proposed remedial measures will be satisfactory.

Finite Element Method

An analysis with the Finite Element Method was carried out in order to obtain information about:

- the actual and future deformations
- the influence of differences in stiffness
- the factor of safety for non-circular slip surfaces.



layer no.	density (kN/m³)	φ' (degr.)	c' (kN/m²)	ν	E (kN/m²)
1	15.0	35.0	1.0	0.35	12500
2	10.0	35.0	1.0	0.35	12500
3	4.0	25.0	4.0	0.35	1875
4	0.5	20.0	6.0	0.35	1125
5	6.0	20.0	4.0	0.35	3750

Fig. 4 Phase 2: Displacements and plastic zones after loading

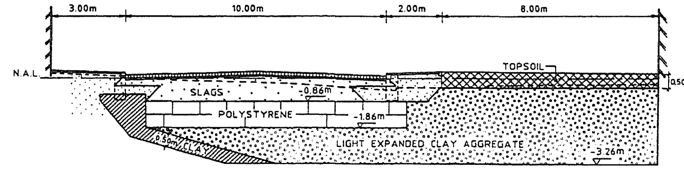


Fig. 3 Design of the improvement of the quay

The analysis was carried out with the help of the DIANA computer program (De Borst, 1984). This program is able to simulate elastoplastic soil behaviour, dilitancy, softening and hardening. In the calculations the Mohr-Coulomb criteria was used for yield; softening and hardening were not taken into account. The element mesh and soil properties are shown in Figure 4. The situation of the quay before the houses were built was simulated in Phase 1. After the construction of the houses, the quay is subsequently filled which is simulated in Phase 2 by introducing a load equal to the fill. Figure 4 also shows the load schedule and the deformations as well as the status (elastic or plastic) of the elements. As can be seen there is a continuous surface of plastic elements (the last iteration step in the Finite Element calculation did not converge).
After Phase 2 the calculated maximum horizontal displacement is 0.73 m. The actual maximum displacement, however, is 2.50 m. This discrepancy may be caused by an incorrect choice of the deformation and yielding parameters, by additional horizontal displacement caused by the dead load on the deflected piles, and by creep. The DIANA program does not take creep into account, and the magnitude of the additional horizontal displacements are unknown. In Phase 3 the load on the quay is reduced by 35 kN/m². Figure 5 shows the deformations and the status of the elements. Table II shows the maximum vertical and horizontal displacements in three cross-sections for the different phases.

TABLE II: Maximum vertical (δ_v) and horizontal (δ_h) displacements (m)

Cross-Section	1		2		3	
Phase	δ_v	δ_h	δ_v	δ_h	δ_v	δ_h
1	0.38↓	0.03→	0.05↓	0.10→	0.04↓	0.05→
2	0.63↓	0.21→	0.31↓	0.73→	0.20↑	0.37→
3	0.54↓	0.19→	0.22↓	0.71→	0.16↑	0.36→

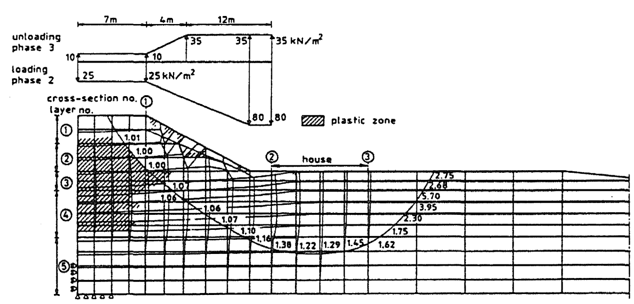


Fig. 5 Phase 3: Displacements and plastic zones after unloading

For the elements which form the imaginary slip surface, the factor of safety is estimated by comparing the mobilized shear strength with the ultimate shear strength. This is expressed by the following equation:

$$\text{factor of safety} = \frac{\sin \phi'}{\sin \phi'_{mob}} \qquad (1)$$

where: ϕ' = angle of internal friction at failure
ϕ'_{mob} = mobilized angle of internal friction

It appears that the elements yielding in Phase 2, which are located underneath the houses, have a factor of safety varying from 1.2 to 1.4 in Phase 3. The calculated displacements in Phase 3 are shown in Table II. Compared with Phase 2 there is only a small decrease in horizontal displacements.

Creep

Singh and Mitchell (1969) presented the following general function for soils that expresses the strain rate, at any time t, after application of sustained deviator stress, Figure 6.

$$\dot{\varepsilon} = A\, e^{\bar{\alpha}\bar{D}} \left(\frac{1}{t}\right)^m \qquad (2)$$

where: $\dot{\varepsilon}$ = strain rate
A = projected value of strain rate at zero deviator stress on logarithmic strain rate versus deviator-stress plot for unit time
$\bar{\alpha}$ = dimensionless parameter defined as the value of the slope of the mid-range linear portion of the logarithmic strain rate versus stress level curve, all points corresponding to the same time after load application
$\bar{D}$ = normalized stress level, defined as the ratio of the deviator stress to the deviator stress at failure
t = elapsed time divided by unit time
m = slope of a logarithmic strain rate versus logarithmic time straight line

A general relationship between the creep strain ε and time is obtained by integration of Equation 2.

$$\varepsilon = Ae^{\bar{\alpha}\bar{D}} \left(\frac{1}{1-m}\right) t^{1-m} \qquad (m \neq 1) \qquad (3)$$

The effect of unloading on the relationship between strain and time is illustrated in Figure 7. When, at time t_1, a soil sample is unloaded from deviator stress $\bar{D}_1$ to $\bar{D}_2$, it is assumed that the sample will follow the strain rate according to curve $\bar{D}_2$ at time t_2. This is shown by the dotted curve. The principle of the equivalent time concept according to Hanrahan (1973) is used here. The effect of unloading on the strain rate can be calculated as follows. The imaginary point of time t_2 is found by putting the strain at time t_1, according to $\bar{D}_1$, equal to the strain at time t_2, according to $\bar{D}_2$, in Equation 3. It follows that:

$$\frac{t_2}{t_1} = e^{\bar{\alpha}(\bar{D}_1 - \bar{D}_2)/(1-m)} \qquad (4)$$

The ratio between the strain rates follows from Equation 2:

$$\frac{\dot{\varepsilon}(t_1)}{\dot{\varepsilon}(t_2)} = e^{\bar{\alpha}(\bar{D}_1 - \bar{D}_2)} \left(\frac{t_1}{t_2}\right)^{-m} \qquad (5)$$

Combining (4) and (5) leads to:

$$\ln \frac{\dot{\varepsilon}_1}{\dot{\varepsilon}_2} = \bar{\alpha}\, \frac{\bar{D}_1 - \bar{D}_2}{1-m} \qquad (6)$$

The reduction in strain rate can therefore be calculated from Equation 6 if the parameters $\bar{\alpha}$ and m are known, and if the factors $\bar{D}_1$ and $\bar{D}_2$ have been calculated by a stability analysis. Future strains, after time t_2 can be calculated with Equation 3. It follows that:

$$\frac{\varepsilon(t_3)}{\varepsilon(t_2)} = \left(\frac{t_3}{t_2}\right)^{1-m} \qquad (7)$$

The imaginary time t_2 follows from Equation 4. The strain at a selected number of years after t_2 at time t_3 follows from Equation 7 if the strain at time t_2 is known.

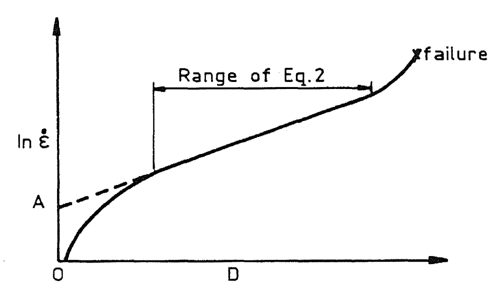


Fig. 6 Relationship between strain rate and deviator stress at given time

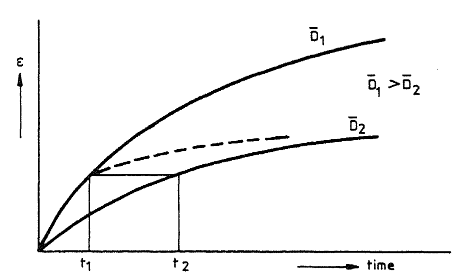


Fig. 7 Effect of unloading on the relationship between strain and time

Prediction of future strain rate and strain

In order to make the predictions extensive laboratory investigations are required to determine the parameters $\bar{\alpha}$ and m. Since only strain rates and strains before and after the remedial measures have to be compared, it was decided that indicative calculations would be sufficient. Therefore values given by Singh and Mitchell (1968, 1969) were used, complemented by data from tests on Oesterdam clay carried out by Delft Geotechnics. These data indicate that the parameter $\bar{\alpha}$ varies between 1.4 and 6.7. It appears that the value of $\bar{\alpha}$ is higher for soft clays than for stiff clays. The parameter m varies between 0.70 for soft soils and 1.30 for stiff. The worst case approach was followed for predicting. Therefore $\bar{\alpha}$ = 3 and m = 0.7 were taken. The factor $\bar{D}_1 - \bar{D}_2$ is derived from the stability calculations. An increase of the actual safety factor from 1.0 to 1.3 immediately after the remedial measures, indicates $\bar{D}_1 - \bar{D}_2 = 0.23$. If the long term factor of safety is taken as 1.5, it follows that $\bar{D}_1 - \bar{D}_2 = 0.33$. Substituting these values in Equation 6 it appears that the ratio between the strain rates before and just after the remedial measures, is at least 10. This ratio increases to at least 30 in the long term.
To predict the future horizontal displacements of Block 2 the strain was considered for a period of 50 years after the remedial measures. The magnitude of the horizontal displacement was about 2.5 m in the period 1958 - 1984, that is, t_1 = 26 years and, from Equation 4, it follows that t_2 = 260 years for $\bar{D}_1 - \bar{D}_2 = 0.23$. Taking t_3 = 310 years and substituting the values in Equation 7 it appears that the ratio between the strains just after the remedial measures, and 50 years later, is 1.05. The horizontal displacement of Block 2 will therefore increase by 5%, being 0.13 m. Taking $\bar{D}_1 - \bar{D}_2 = 0.33$ leads to a ratio of 1.02 and an expected horizontal displacement of Block 2 of 0.05 m. A horizontal displacement of 0.10 m has been taken into account in the design of the new foundations for the blocks.
The observed strain rate is about 100 mm/year for Block 2. Immediately after the remedial measures the strain rate should reduce to 10 mm/year, and in the long term to 3 mm/year. The analysis of the future strain based on the actual displacement, indicates a strain rate of 2.5 mm/year immediately after the remedial measures and 1 mm/year in the long term. The discrepancy can be explained by the fact that the observed strain rate of 100 mm/year, is not in agreement with Equation 2. This can be explained as follows:

- Equation 2 is only valid for values of $\bar{D}$ between 0.3 and 0.9 and not for the near failure conditions which occur before the remedial measures are applied, Figure 6.
- Additional fills to compensate settlements are not in agreement with the assumption of sustained deviator stress.
- The large horizontal displacements cause a secondary horizontal load on the pile foundations which cannot be neglected and therefore an increase in the strain rate.

Equation 2 is valid for conditions after the improvement of the quay and the foundations, and the prediction of the future strain rate based on the observed horizontal displacement is reliable.

IMPROVEMENT OF THE FOUNDATIONS OF THE HOUSES

Due to the extreme horizontal displacements the stability of at least 4 of the 6 blocks of terrace houses was in danger, the more so since the dead load of the houses on the deflected piles increases the displacements. Data from archives and inspection of some foundation piles showed the following:

- Block 1, consisting of 13 houses, is founded on 55 precast concrete piles, 0.35 m square, and with a length of 19 m. Six pile heads have been inspected. The upper part of these piles have inclinations varying from 6 : 1 till 30 : 1. Several pile heads show serious cracks, Figure 8. Sonic integrity tests on two piles demonstrated that these piles had no cracks on deeper levels.

Fig. 8 A cracked concrete pile

- Block 2, consisting of 7 houses, is founded on tapered timber piles with a precast concrete upper section above water level. Four out of five upper sections inspected were almost vertical; one had an inclination of 6 : 1. The upper part of the timber piles have inclinations of 3 : 1 to 6 : 1.
- Block 3, consisting of 6 houses, is founded on 103 tapered timber piles. The upper part of the piles have inclinations varying from 4 : 1 till 6 : 1.
- Block 5, also consisting of 6 houses, is founded on 93 tapered timber piles. The upper part of the timber piles have inclinations varying from 2 : 1 till 3 : 1.

The following sequence of improvements was chosen as a result of these inspections: Block 2 and then Blocks 5, 3 and 1. Measurements showed that the houses had also undergone vertical displacements. Absolute displacements could not be established, but the differential displacements since construction varied from 0.2 m for Block 3 to 0.7 m for Block 5. It was striking that despite the large horizontal and vertical displacements, the blocks showed so little cracks. This was caused by the relatively stiff cellar floor that was present under all the blocks.

Demands on the foundations

Measures were necessary to guarantee the stability of the terrace houses for the next fifty years. The following aspects had to be taken into account in the design of a new foundation or in the re-use of the existing one:

- there must be an equilibrium of forces and moments without large deformations
- a horizontal displacement of 0.1 m is to be expected in 50 years after the remedial measures for the quay are carried out
- all the parts of the construction must have sufficient strength
- the design must be practicable.

Injection piles

Already in an early stage of the investigation it was decided that if it was impossible to re-use the existing piles injection piles would be installed as replacements. An injection pile is a steel tube pile filled with a hardened grout. During installation the soil is pushed aside, which improves the bearing capacity of the pile. The steel tube is brought to the right depth by hammering, the grout being injected at the same time. The grout is forced down the tube and out of the bottom up the outside. The grout functions as a lubricant during the installation, and in this way, reduces the resistance. After hardening the grout contributes to the strength and the stiffness of the pile, transfers a part of the load to the soil, and protects the steel tube against corrosion. In this project a coupled injection pile was used with an external diameter of the tube of 114.3 mm and a wall thickness of 5.4 mm. The total external diameter of the pile, inclusive the grout will finally amount to about 150 mm.

Possible solutions

Two possible solutions for the design of a new foundation were considered. In the first solution the installation of new raking and vertical piles was studied, Figure 9. In principle an equilibrium of forces is reached if the magnitude of the active earth pressure H_1 and the passive earth pressure H_2 can be determined with sufficient accuracy. However, this solution has the disadvantage that it contains the idea of fixing the position of the house, while some horizontal deformation has to be taken into account.

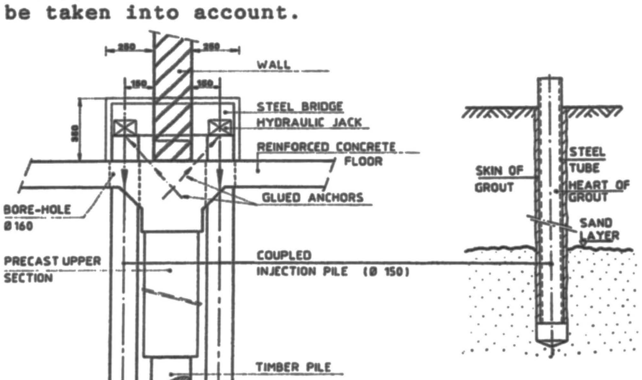


Fig. 11 Design of the foundation improvement and principle of the injection pile (dimensions in mm)

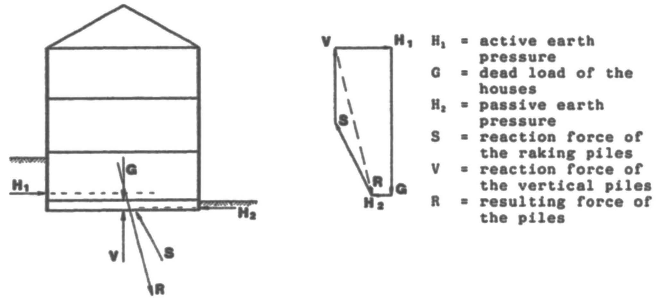


Fig. 9 Solution with raking and vertical piles

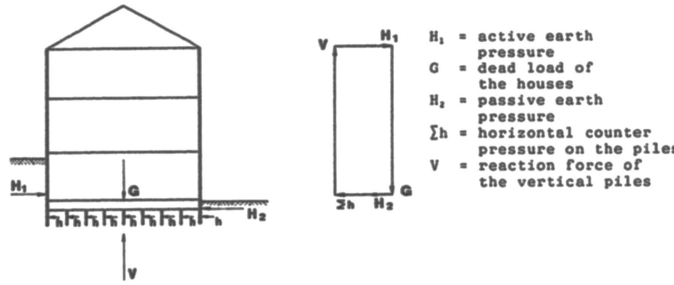


Fig. 10 Solution with only vertical piles

When deformation of the quay takes place, the soil tends to displace more than the house, and H_1 will increase and H_2 decrease. This solution requires relatively many raking piles and therefore extra piles, and, as a result, is less attractive. However, in the case of a new building this solution, with relatively massive piles, would probably be selected. In the second solution only the installation of vertical piles was studied, Figure 10. An equilibrium of forces is reached if also the soil behind the upper part of the piles delivers some counter pressure. To be able to deliver extra counter pressure, the pile and therefore the house has to be displaced horizontally a little bit more than the soil. The advantage of this solution is that H_1 is smaller than in the first solution, and H_2 is larger. The horizontal loads and the moments and shear forces on the piles will remain small. This solution is practicable because it requires not very many piles and no massive piles. In addition it is a flexible solution. One condition, however, is of great importance; the total horizontal displacement, after the replacement of the foundation piles, must not be excessive. Based on the calculations discussed above this solution was selected for the replacement of the foundation piles.

Design of foundation improvements

The injection piles were installed from the cellar floor at both sides of the bearing walls and then connected to the walls by glued anchors and a steel beam, Figure 11. The designs for the foundation improvements for the various blocks were based on the various requirements discussed above. The result was a complete replacement of the bearing function of the foundation piles under Blocks 2, 3 and 5. The design for Block 1 will probably consist of partial replacement.

Execution of foundation improvements

The injection piles were hammered to a deeper level than the existing piles. However, no serious problems were encountered during the execution of the works. One aspect is interesting. The blocks of terrace houses have not been displaced horizontally the same amount. Block 5, in fact, shows a difference in displacement of about 1 m between the ends. As a result new piles could be hammered accidentally on to the existing piles, Figure 12. Although this was taken into account in the design as much as possible, it did occur a few times, however without consequences. The step-wise transfer of loads from the existing piles to the new piles went as planned.

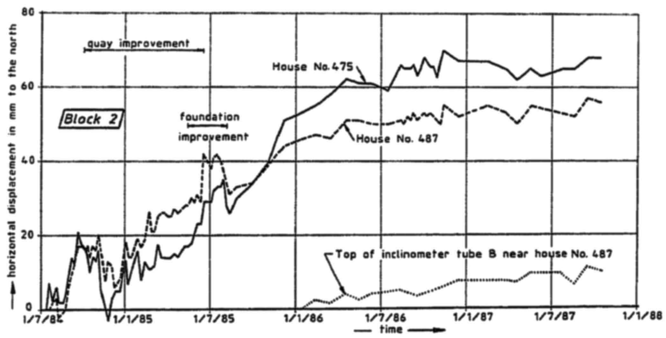


Fig. 13 Horizontal displacements of Block 2

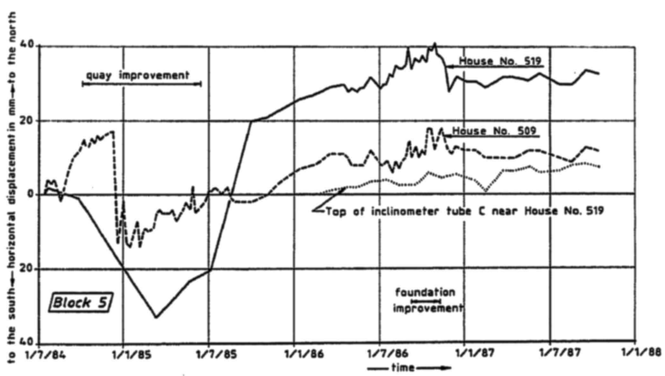


Fig. 15 Horizontal displacements of Block 5

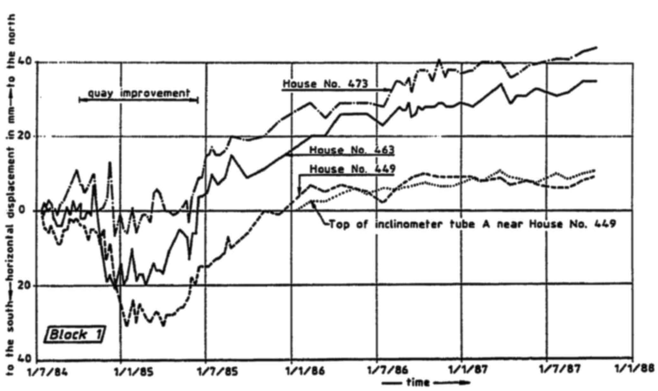


Fig. 16 Horizontal Displacements of Block 1

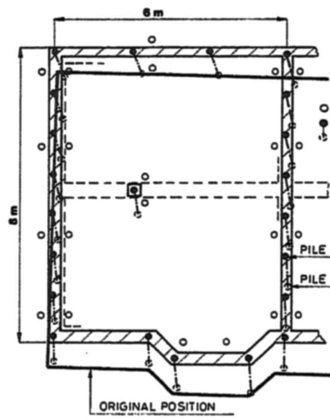


Fig. 12 Location of the new piles relative to old piles

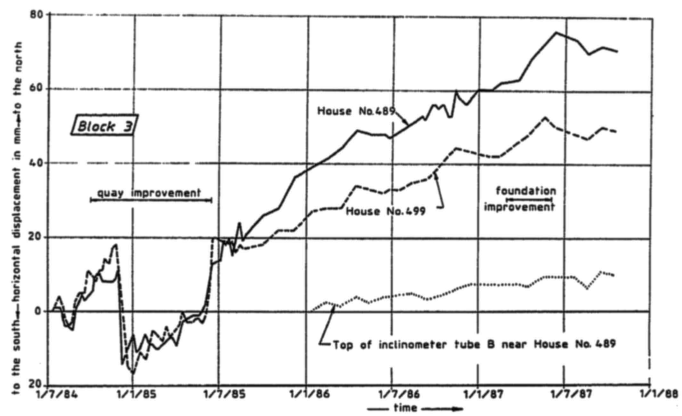


Fig. 14 Horizontal displacements of Block 3

DISPLACEMENT BEHAVIOUR SINCE 1984

Initially the measurements of the horizontal displacements were made relative to a straight line. Both ends of this line were situated outside the area of influence of the soil movements. Since 13th January 1986 another system has been used. The new measuring line is defined as the line between the top of three inclinometer tubes installed with the bottoms in the deep Pleistocene sand layer. The results of the measurements of the horizontal displacements of the blocks of terrace houses are given in the Figures 13 to 16. The horizontal displacements of the top of the inclinometer tubes are given in the same figures. The vertical displacements of Blocks 1, 2, 3 and 5 are given in Figure 17. The following periods are indicated in the figures:

- reconstruction of the quay : 03-10-1984 to 14-06-1985
- new foundation for Block 2 : 14-05-1985 to 05-08-1985
- new foundation for Block 5 : 10-09-1986 to 10-11-1986
- new foundation for Block 3 : 02-03-1987 to 04-06-1987

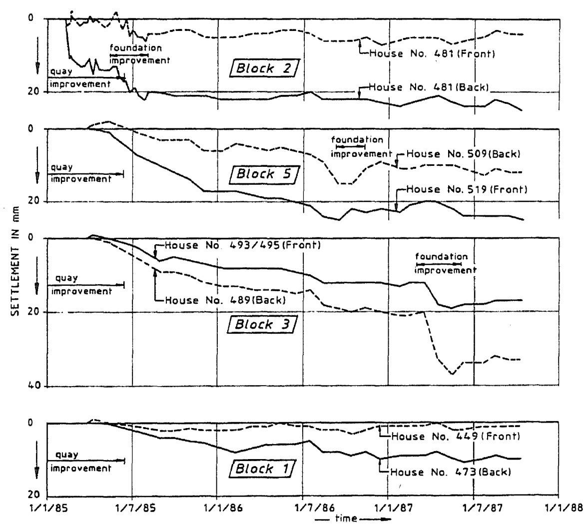


Fig. 17 Vertical displacements of Blocks 2, 5, 3 and 1

Reduction of rate of horizontal displacement

Data given in Table I and those measured since July 1984 indicate the following rates of horizontal displacements, Table III. The data since July 1984 are given in three periods. The division in periods has been made in such a way that temporary influences like foundation improvements occur in only one period.

TABLE III: Rates of horizontal displacement to the north (mm/year)

Block	House No.	date of constr. to 1975	1975 - 1983	9/1983 - 7/1984	13/7/84 - 13/1/86¹)	13/1/86 - 26/1/87	26/1/87 - 14/10/87
0	439	62	8	0	3	12	-1
0	447	23	1	0	-5	7	-3
1	449	28	3	6	2	6	0
1	473	72	24	19	18	11	8
2	475	100	44	101	35²)	14	1
2	487	106	75	126	31²)	8	3
3	489	32	38	44	26	21	16²)
3	499	22	31	44	18	15	10²)
4	501	10	7	6	2	0	1
4	507	3	13	0	-7	-2	-3
5	509	14	15	0	5	5²)	0
5	519	41	27	25	17	5²)	3

¹) = period includes quay improvement
²) = period includes foundation improvement

The period 1975 - 1983 is taken as the reference period for the horizontal displacements before improvement of the quay for the analysis of the reduction of the rates of horizontal displacement. Table III shows, for Block 2, a reduction of this rate by a factor of 3 to 10, one year after the improvement of the quay and the foundation. One year later the reduction has increased to a factor of 25 to 40, the rate of horizontal displacement amounting to 1 to 3 mm/year. The reduction factor for Block 5 amounts to 3 to 5, one year after the improvement of the quay. One year later, that is also one year after the improvement of the foundation, the reduction has increased to a factor of at least 9, the rate of horizontal displacement amounting to 0 to 3 mm/year.

The reduction factor for Block 3 amounts to about 2, one year and about 3, two years after the improvement of the quay. Improvement of the foundations has only been implemented recently, and future measurements will indicate the effect on the magnitude of the horizontal displacements. House No. 473 of Block 1 shows a reduction factor of about 2 one year and of about 3 two years after the improvement of the quay. Improvement of the foundations is still in the design stage. The inclinometer tubes also show decreasing rates of horizontal displacement. Data before the improvement of the quay are not available. Comparing 1987 with 1986, however, shows a reduction factor of 2 to 3 for the tops of Tubes A and B. Data of 1987 show a rate of horizontal displacement of about 3 mm/year. Tube C shows a variation of measuring results between the two periods, and is therefore not considered here.

CONCLUSIONS

The remedial measures have resulted in a considerable reduction of the rate of the horizontal displacement of the houses. The combined effect of the improvements to the quay and the foundations is a reduction of this rate by a factor of 9 to 40 for Blocks 2 and 5 in a period of two years. The predicted reduction amounted to a factor of at least 10 initially, subsequently increasing to at least 30. Prediction and observation therefore agree rather well. The new foundations were designed on a horizontal displacement of 100 mm to be expected in 50 years, that is an average rate of 2 mm/year. Although the rate immediately after the implementation of the remedial measures turned out to be rather large, this rate has, in the mean time, reduced to 0 to 3 mm/year for Blocks 2 and 5. The present behaviour therefore is satisfactory. After improving the foundations of Blocks 2, 3 and 5 the settlement process, of these blocks, stopped. The inclinometer tubes show some movement of the new quay, but the rate of horizontal displacement is reducing from about 8 mm/year, one year after the improvement of the quay, to about 3 mm/year one year later. The measurements will be continued for the time being.

REFERENCES

Borst, R. de, and P.A. Vermeer, (1984), "Possibilities and Limitations of Finite Elements for Limit Analysis", Geotechnique, vol. 34, 199-210

Hanrahan, E.T., and M. Shahrour, (1973), "Prediction of Strain Rates using Eg, Ek parameters", Proc., 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, vol.1, 171-175

Singh, A., and J.K. Mitchell, (1968), "General Stress-Strain-Time Function for Soils", Journal of the Soil Mechanics and Foundations Division, Proc. ASCE, vol. 94, no. SM1, 21-46

Singh, A., and J.K. Mitchell, (1969), "Creep Potencial and Creep Rupture of Soils", Proc., 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico, vol. 1, 379-384