International Conference on Structural and Foundation Failures
August 2-4, 2004, Singapore

Case history of a deep multi-tied -back Excavation

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Abstract

Despite the knowledge and experiences accumulated, and the available standards on the deformation behavior of multi-tied-back excavations, a damage had occurred on a deep excavation in 1994 in southern Germany. The paper tries to reveal the source of the failure using analytical and finite element methods. It also presents the deformations and damages that had been recorded.

Keywords: deep excavation, excavation failure, finite element method, multi-tied excavation, case study, soil movement, settlement.

Introduction

It is well known that the horizontal deformation of a deep, tied-back wall may cause settlement of the ground surface behind the wall, which usually affects the nearby structures. The horizontal deformations in such a system may not be estimated accurately with the classical analytical procedures. The governing movements in such a system are the displacement and deformation of the soil wedge between the back of the wall and the mid point of the bonded anchor length. This soil wedge may be treated the same as a soil confined in a cofferdam. The movement of the soil wedge is primarily caused by: relief of stresses due to excavation, deflection of the wall, prestressing of the anchors, yielding of the anchors, shear with in the soil block, shear at the bottom of the soil block, bending of the soil block, the interaction between the soil block and the anchors, reduction of the earth pressure at rest, the water pressure with in the excavation level, and the swinging of the soil block. More information on this subject can be found in Stroh (1974), Ulrichs (1981) and EAB, EB (46) (1994).

The paper presents a case history of a damage on a deep, multi-tied-back excavation, which had been occurred in spite of the available knowledge and standards on deformations of tied-back, deep excavation systems. Both analytical and finite element analysis had been conducted to follow the magnitude of the deformation and the extent of the damage.

Description of the excavation site

The site

The excavation site was located in the southern Germany. It was intended for underground parking of the multi-storey shopping center, and completed in 1994. The site plan is shown in Fig. 1.

The excavation was about 14.5 m deep and covered an area of 90 m by 32 m. It was separated from the existing buildings by a road. The average distance between the excavation and the existing building was about 18 m.

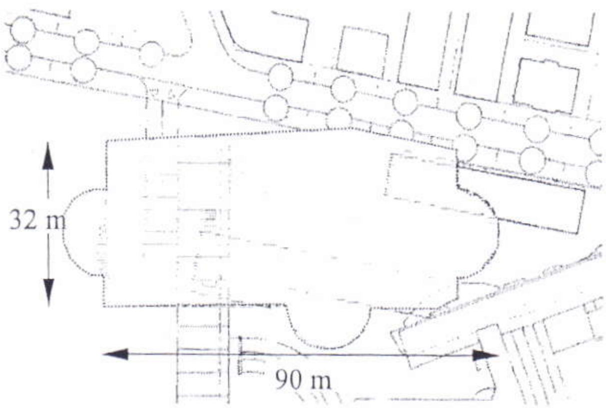

Figure 1 : The site plan.

The soil condition

The site was investigated using numerous bore holes, sounding tests, and ground water observation bore holes. The investigation revealed a ground comprising about 3 m of fill material, overlying an alluvial loam soil (haugh) of thickness about 1.5 m. Beneath is a young glacial boulder clay of thickness about 9 m, overlying densely deposited glacial boulder clay. The soil layers are shown in Fig. 2, and the corresponding soil parameters are given later in the paper in Table 2.

The ground water investigation revealed two ground water positions: one in the upper layer (on average 2.4 m below the ground surface) and the other in the lower layer (on average 7.0 m below the ground surface). The lower ground water level is believed to be the common water table found in the area, where as the upper ground water may came later in the fill layer from rain and surface water but unable to join the lower ground water because of the low permeability of the hough and boulder clay layers.

The support system

In the first design, a soldier pile with wood lagging, with a penetration depth of 6 m, a center to center distance of 1.5 m, and tied back with 7 ground anchors was suggested for an excavation depth of 15 m. However, this had been changed short before the construction had begun. A soldier pile with wood lagging, a penetration depth of 4.5 m, spacing of 2.75 m and supported with 5 ground anchors had been recommended in the final design. There was a clear difference in the magnitude of the loading considered in the two designs. In the first design active earth pressure was considered instead of the increased active earth pressure, and only the lower ground water level (-7.0 m) was included in the calculations. On the other hand, in the final design the increased active earth pressure was considered, but the ground water was taken at the level of the bottom of the excavation.

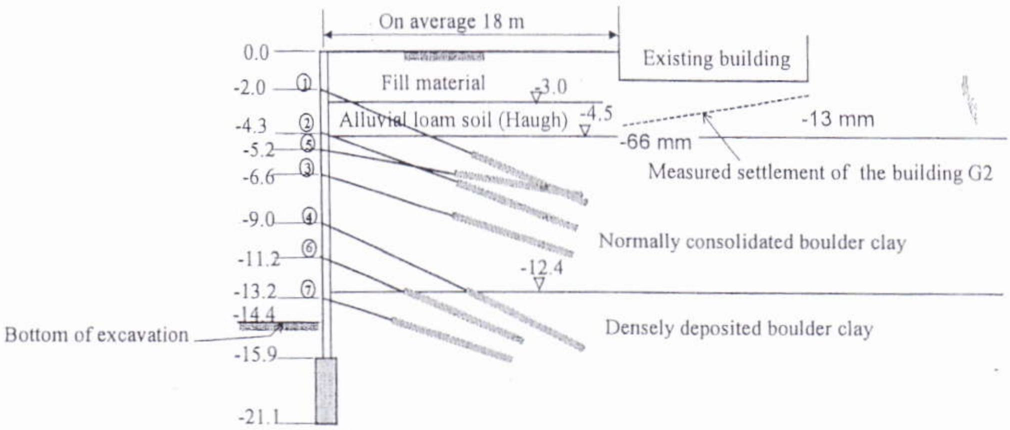

Figure 2. Soil profile, wall and anchor arrangements, and the measured settlement profile under the building.

Because a damage on the nearby structures had already been occurred during the first excavation phase, the design had been revised during the construction for the second time in order to minimize further damages. The change of the design include two additional ground anchors: one between the 2nd and 3rd anchors and instead of the 5th anchor further two anchors with a change of position and inclination of the original 5th anchor were proposed. The final arrangements of the anchors and the wall are shown in Fig. 2.

Observed damage and its course

Approximately after the first excavation stage, the first damage had been observed as a consequence of too deep excavation before the installation of the first anchor. At this time , however, the damage was relatively small; merely cracks are observed along the curb stone of the road north of the excavation. After the first anchor had been prestressed, further small settlement of the existing building north of the excavation were observed. It was at this time that the settlement and deflection measurement devices had been installed in order to follow up the course of the deformation.

After the third anchor had been installed and prestressed (excavation depth 6.8 m), further enlarged damages had been observed in the form of a clear settlement of the surface of the road and the pedestrians way. Large cracks were seen on the surface of the road, edge of the curb stone, as well as near the existing buildings. Cracks along the entrance to the stair case and the opening to basement windows of the nearby buildings were observed. Fig. 3 shows an overlook on the damage that had occurred to this time point.

After the fourth excavation stage and the 4th anchor had been installed and prestressed, the damage had increased considerably. It was at this time that additional two anchors had been suggested in order to minimize further damage. The first additional anchor, named as anchor 5 in Fig. 2, were installed at this stage between the 2nd and 3rd anchor after refilling the excavation up to the anchor level. The purpose of this anchor was mainly to relieve the already installed anchors during further excavations.

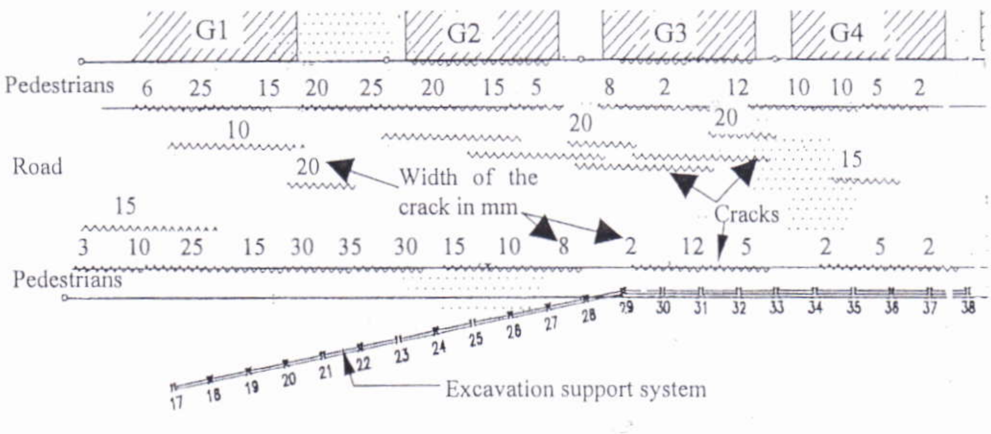

Figure 3. Observed damages and cracks after the 3rd stage of excavation (-6.8 m).

At the end of the excavation, a total settlement of 14.3 cm at the road surface, a horizontal deflection of 9.9 cm, and a vertical displacement of 4 to 8 cm at the top of the soldier pile were measured. A maximum settlement of 6.6 cm in front of the building G2 (towards the excavation) and 1.3 cm behind the building G2 were recorded. The final settlement of the building G2 is shown in Fig. 2, where as the horizontal deflection of the wall at different stages are shown in Fig. 4. Because the damage on the nearby structures was considerable, the case had eventually lead to court.

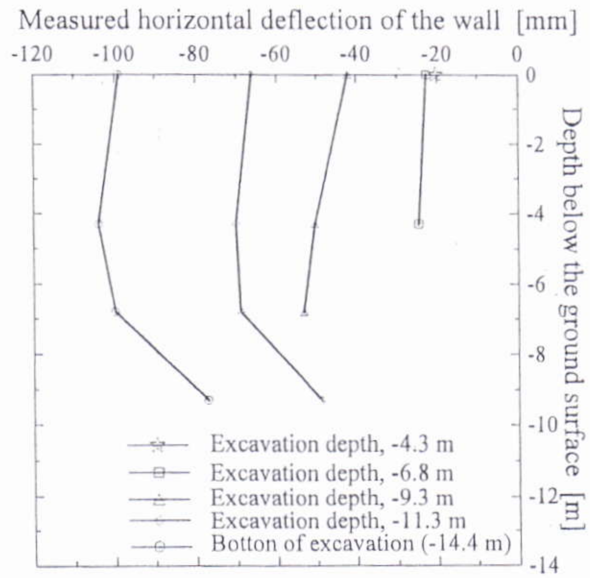

Figure 4. Measured horizontal deflection of the wall at various construction stages.

Analytical approach to approximate the deformation of the wall

The following are analytical approaches to determine the horizontal movement of the soil-anchor-wall-system. The soil block between the wall and the middle of the bonded length of the anchor is assumed to act the same as a soil confined in a cofferdam. A modified approach is applied based on the methods recommended by Nendza and Klein (1974), Stroh (1974) and Ulrichs (1981). The governing horizontal deflection of a cofferdam (the soil block) is the sum of the following deflection components: a) horizontal deflection of the cofferdam due to excavation (relief of stresses), b) horizontal deformation due to shear under the cofferdam, c) horizontal deformation due to shear with in the cofferdam, d) horizontal deformation due to bending of the cofferdam, and e) horizontal deflection due to other influences such as anchor prestressing, yielding and bending of the anchor, bending of the wall, reduction of the earth pressure at rest, and the interaction between the soil block and the anchor. The formulas used to determined the horizontal deflection of the cofferdam are given in Table 1.

Because the analytical method described above applies only either for fully submerged soil or dry soil, it was difficult to handle the two ground water tables at on time in the calculation. Hence, the calculation was divided in to two parts with two extreme positions of the ground water: one at 2.4 m below the ground surface and the other at the bottom of excavation (without GW effect). Ignoring the deflection contribution from part (e) (see above), the deflection contributions from part (a) to (d) had been calculated and are presented in Table 2. The total horizontal deflections at the top of the wall are 67.2 mm and 135.8 mm without GW and with GW respectively. Similarly, the horizontal deflection at the level of the excavation are 13.4 mm and 18.2 mm respectively.

Table1. Horizontal deformation of the cofferdam using analytical approach.

Horizontal deflection of the wall due to:	Applied formula	Horizontal deflection of the wall [mm]	
		without GW (GW at -14.4 m)	with GW (-2.4 m)
relief of stresses due to excavation	$s = 0.15\dfrac{\gamma \cdot H \cdot B}{E_E}$	8.6	8.6
Shear stresses below the cofferdam	$s = (0.4 \ \ to \ \ 1.2) \cdot \dfrac{(E_a + W_{\ddot{u}}) \cdot B}{E_E \cdot b}$	4.8	9.6
Shear stresses within the cofferdam	$s_K = \dfrac{q \cdot H^2}{6 \cdot G_E \cdot b}$	21.2	46.3
bending of the coffer dam	$s_K = \dfrac{q \cdot H^4}{30 \cdot E_{E2} \cdot I}$	32.6	71.3
Total horizontal deflection at the top of the wall		67.2	135.8
Total horizontal deflection at final excavation level)		13.4	18.2

Where γ is the average unit weight of the soil layers, H is the height of the wall, B is width of the excavation, E_E is the average un/reloading modulus of elasticity of the soil layers below the bottom of the excavation ($E_E \approx 4 \cdot E_s$), E_s is constrained modulus of elasticity, E_a is active earth pressure, $W_{\ddot{u}}$ is water pressure, b is width of the cofferdam (here b = 11 m), q is the value of the active earth pressure and water pressure at the position of the bottom of the excavation, G_E is modulus of shear deformation $\approx E_{E2}/(2(1+\nu))$, E_{E2} is the average un/reloading modulus of elasticity of the soil layers with in the cofferdam, and I is the moment of inertia of the cofferdam ($I = 1 \cdot b^3/12$).

Back analysis with the finite element method

The deformation behaviour of the excavation had been once more studied using the finite element method. The soil layer, the ground water, the wall, the anchors, and the construction stages were realistically simulated using the finite element computer code "PLAXIS". A section of 110 m deep and 126 m (symmetrical) wide had been taken in the analysis and the mesh was generated using 15-node triangular elements. The upper part of the soldier pile wall with wood lagging (from top of the wall up to 1.5 m below the bottom of excavation) and the lower part are simulated as two beam elements, with two different stiffness values, rigidly fixed at their common joint. The fixed part of the anchor was simulated as geotextile. Seven excavation phase had been recognized during the construction. Each excavation phase followed by the installation and prestressing of the corresponding anchor had been realistically simulated in the program. After the 4th excavation phase had been completed and the 4th anchor had been installed and prestressed, the excavation was refilled up to the level of the 5th anchor in order to provide a construction area for the machine to install and prestress the new anchor no. 5. This additional construction stage had also been included in the analysis. Part of the geometry, the mesh, the external load, and the end of excavation stage are shown in Fig. 5.

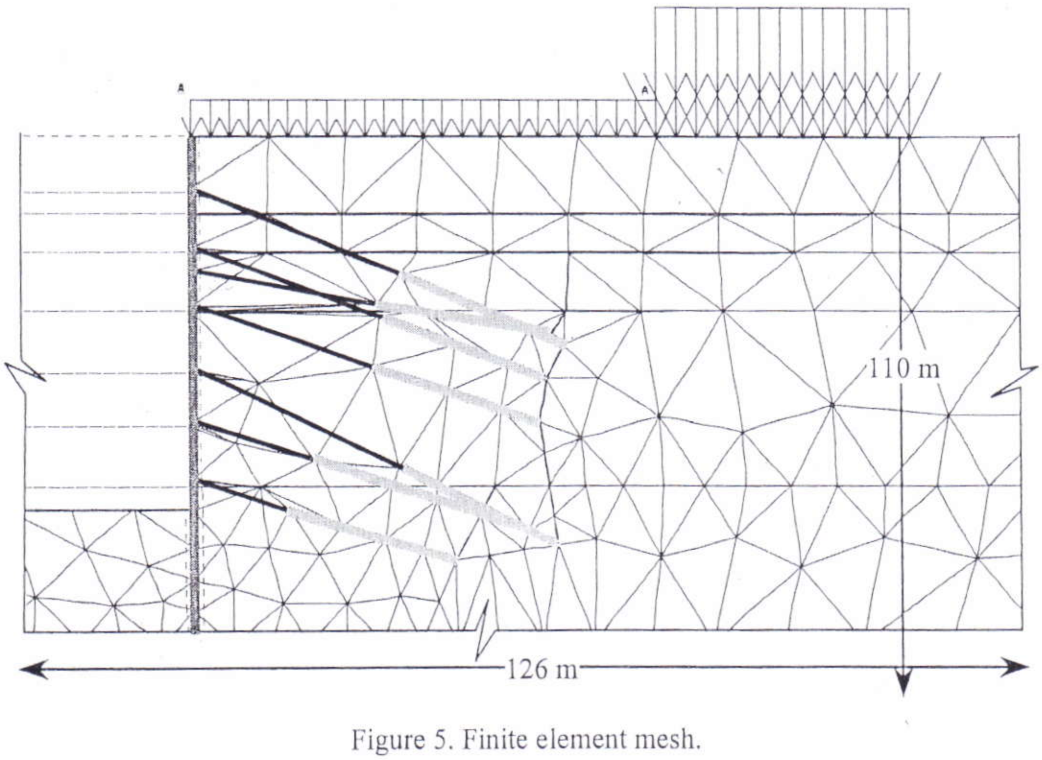

Figure 5. Finite element mesh.

The two ground water locations, revealed from the soil investigation, namely at 2.4 and 7.0 m below the ground surface, are realistically represented in the model. It was assumed that the water pressure due to the upper ground water will cease to zero at the middle of the normally consolidated boulder clay layer above the lower ground water. Where as the normally consolidated clay below the lower ground water table and the densely deposited boulder clay are subjected to a water pressure that starts from zero at the lower ground water table. For the purpose of comparison, a ground water at the level of the bottom of excavation (this was indicated in the analytical analysis as "without water" condition) was also considered in separate analysis.

The properties of the soils were represented by elasto-plastic-cap constitutive soil model called Hard Soil Model (HSM) (PLAXIS, 1998). The soil parameters required to completely define the constitutive behaviour of the soil according to the HSM are given in Table 2, where E_{oed}^{ref} and E_{50}^{ref} are the oedometer modulus and triaxial secant modulus at 50% of the maximum deviatoric stress respectively for a reference pressure p^{ref} = 100 kN/m², E_{ur}^{ref} is modulus of elasticity for un/reloading for p^{ref} = 100 kN/m², ν_{ur} is the Poisson's ratio for un/reloading/reloading, m is the power, K_0 is the coefficient of earth pressure at rest, R_{int} is the interface property, and R_f is the ratio of the deviatoric stress at failure and the ultimate deviatoric stress.

Table 2. Soil parameters used in back analysis using the finite element method

Soil layer	γ [kN/m³]	φ' [°]	c' [kN/m²]	E_{oed}^{ref} [MN/m²]	E_{50}^{ref} [MN/m²]	E_{ur}^{ref} [MN/m²]	ν_{ur} [-]	K_0 [-]	m [-]	R_{int} [-]	R_f [-]
Fill material	20	25	0	7	5.3	21	0.25	0.58	0.8	0.67	0.9
Haugh (alluvial loam soil)	22	25	5	7	5.3	21	0.25	0.58	0.9	0.67	0.9
Normally consolidated Boulder clay	22	27.5	5	8	8.0	32	0.25	0.54	0.9	0.67	0.9
Densely deposited boulder clay	23	27.5	30	40	30	120	0.25	0.54	0.8	0.67	0.9

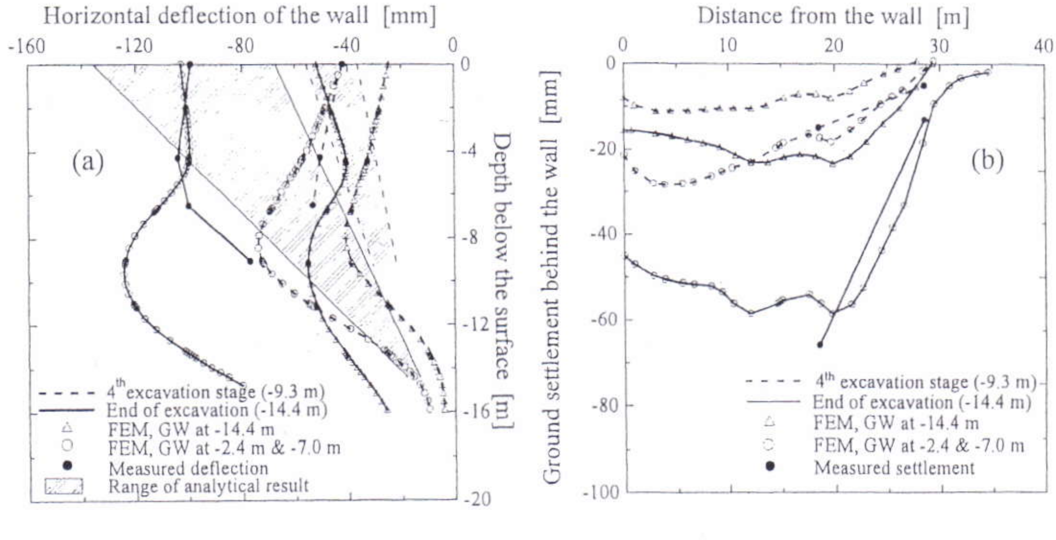

Figure 6. a) Horizontal deflection of the wall, and b) settlement of the ground behind the wall after the 4th excavation (-9.3 m) and full excavation (-14.4 m).

The result of the finite element analysis together with the result of analytical analysis and field measured values of the horizontal deflection of the wall as well as the settlement of the near by building (G2) after the 4th excavation phase (-9.2 m) and end of excavation (-14.4 m) are shown in Fig. 6. The shaded part in Fig. 6a shows the region where the analytical result lies for the two extreme position of the ground water table. From Fig. 6, it would appear that the horizontal defection of the wall as well as the settlement of the ground behind the wall would had been predicted realistically with finite element and would have avoided the damage that had been occurred. The average horizontal deflection at the top of the wall could have also been reasonable predicted using the analytical approach.

How could have been the damage reduced?

A lot of analytical conventional calculation had been conducted to design the excavation in the first as well as in the revised design phases. However, non of them had predicted the deformations that had been recorded. On the other hand, by applying the finite element method and the above described analytical approach, the magnitude of the deformation would have been predicted.

A simple parametric study on variation of the anchor length (Table 3) show that the horizontal deflection at the top of the wall would have been reduced by 60%, if the anchor length had been increased by 9 m for the unfavourable GW position (i.e., GW at -2.4 m below the ground surface). Thus, the damage that had occurred would had almost been avoided.

Position along the wall	Horizontal deflection [mm] at the top of the wall			
	The anchor length increased by an amount of:			
	$\Delta l = 0$ m	$\Delta l = 3$ m	$\Delta l = 6$ m	$\Delta l = 9$ m
Top of the wall	135	89	70	55
Foot of the wall	18	16	15	14

Summary and Conclusion

It is well known that the horizontal deformation of a deep, tied-back wall, causes a settlement of the ground surface behind the wall, which affects the nearby structures. These horizontal deformations may not be estimated accurately with classical analytical procedures, which are based on the elastic deformation of the wall only. The governing movements in such a system are the displacement and deformation of the soil block between the back of the wall and the middle point of the bonded length of the ground anchors. Thus, the soil block may be treated as a soil confined in a cofferdam.

The horizontal deflection of the wall coupled with the movement of the assumed back of the cofferdam (assumed sliding surface) may lead to damages on the near by buildings, in particular when the existing building lies on the active sliding surface.

Specially, if the length of the ground anchors in a muti-tied-back system is almost the same, it will lead to a rapid development of settlement behind the wall (EAB, 1994). To avoid such damages, it is recommended to straddle and to draw up the anchors in an echelon as much as possible. Moreover, if there is an existing building near by an excavation, it would be recommended to design the length of the anchor so that the foundation of the building will lie fully on the cofferdam (before the bonded part of the anchor) instead of immediately behind it.

The governing displacement and deformation of the soil block in the cofferdam may reasonably be approximated using the analytical approach or the finite element method. Therefore, it is possible to predicted the extent of the damage that would occur as a result of the horizontal deflection of the wall during the design phase and take the appropriate measures instead of trying to correct the damage once it had occurred.

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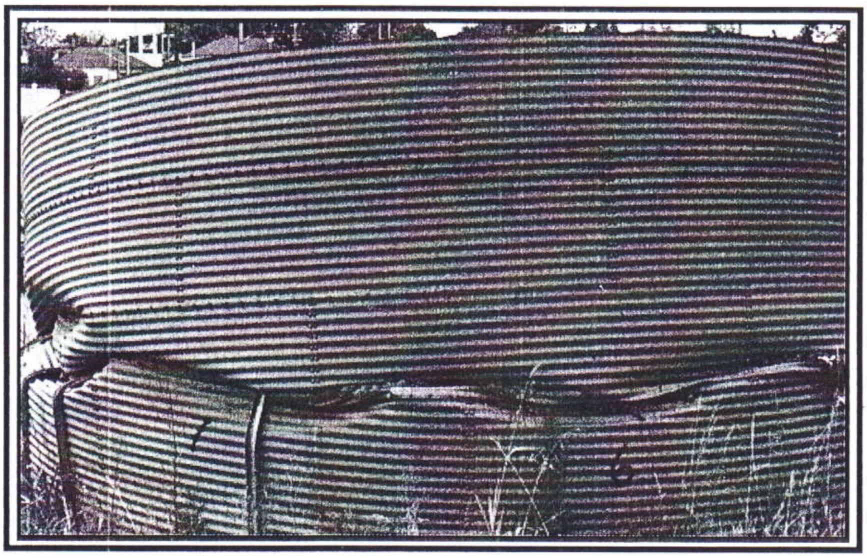

Edited by

C.M. Wang and K. Murugappan