TRANSPORT RESEARCH LABORATORY

A review of the durability of soil reinforcements

Prepared for Quality Services Civil Engineering, Highways Agency

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TRL REPORT 406

First Published 1999
ISSN 0968-4107

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Executive Summary

The long term stability of reinforced soil structures is dependent upon the durability of the reinforcements. At the time of the development of reinforced soil techniques in the late 1960s there was a paucity of data relating to the durability of metallic elements buried in soil. Research was therefore initiated at the Transport and Road Research Laboratory (TRRL), now the Transport Research Laboratory (TRL), to establish specifications for the detailing and construction of reinforced soil structures and also to determine the long term in situ rate of degradation of the materials used for reinforcements. The latter included planned experiments and also the opportune extraction and examination of reinforcements from in service structures.

This report reviews the findings from the field experiments undertaken by the Laboratory into the degradation of various types of reinforcement, and from this assesses where further work is required and what direction it should take. Different methods of determining the degree of corrosion of buried metallic reinforcements are discussed and data are presented for metallic elements recovered from four sites. The measured rates of corrosion are compared to current Highway Agency (HA) requirements for both reinforced soils and corrugated steel buried structures (CSBS). Data are also provided and discussed on the degradation of glass fibre reinforced plastic strips and coated polyester strips which have been used as soil reinforcements.

The data presented show that, due to their susceptibility to pitting corrosion, aluminium alloy and aluminium coated steel reinforcements are not suitable for use in reinforced soil structures. Galvanised mild steel strips, with appropriate sacrificial thicknesses of zinc and steel, and stainless steel strips are suitable for use as reinforcements but the use of the latter is limited by their relatively high cost, and perhaps also, for ferritic types, by their susceptibility to pitting corrosion in the presence of chloride and sulphate ions. In the studies described herein, the galvanised mild steel elements suffered a non-uniform loss of cross-section, but pitting was not evident. However the partial covering of zinc did not provide complete galvanic protection to the exposed steel. It would therefore seem optimistic to rely on a uniform distribution of corrosion in the long term, and for reinforced soil applications it would seem more appropriate to report the severity of corrosion in terms of loss in strength rather than the mean loss in thickness.

It is concluded that, at this stage, there are no grounds for changing the present requirements covering the durability of buried metallic components. However many more data than are currently available are required to establish the degree of non-uniformity of corrosion, and the relation between loss in tensile strength and loss of thickness, and how both of these vary with time and with the type of soil. The data for the non-metallic reinforcements suggest that current practice provides an adequate margin of safety against their rupture over the design life of a structure.

Additional reinforcements and coupons can be recovered from the various sites and two of the structures have corrosion monitoring systems from which in situ electrochemical measurements can be taken. The collection of further data from these sites could provide valuable information on the long term durability of materials buried in soil and opportunities for future work are discussed in the report.

1 Introduction

The internal stability of a reinforced soil structure is dependent upon the pullout resistance and tensile strength of the reinforcements: thus the durability of the reinforcements is an important consideration in design.

At the time of the development of modern reinforced soil techniques by Vidal (1969) there was a paucity of data relating to the durability of metallic elements buried in soil. An overly pessimistic view of durabilty would have hindered the take-up of the techniques, thereby losing their advantage of economy, whereas an optimistic assessment could have led to the premature failure of structures. (Failures of in service structures due to the corrosion of metallic reinforcements have been reported by Ramaswamy and DiMillio (1986) and also by Blight and Dane (1989)). Research was therefore initiated at the Transport and Road Research Laboratory (TRRL), now the Transport Research Laboratory (TRL)[1], to establish specifications for the construction of reinforced soil structures and also to determine the in situ rate of degradation of the materials used for reinforcements. This has included planned experiments and also the opportune extraction and examination of reinforcements from in service reinforced soil structures. However, although the performance of several in service structures was monitored during and following the end of their construction, because of problems with excavating material from them only rarely could the durability of the reinforcements be examined in a systematic manner.

The results of this research have not been published widely, but they have influenced the generation of specifications for reinforced soil construction, for example BD 70 (DMRB 2.1.5) and Murray's (1993) specification for soil nailing. Some of this work is relevant to the durability of corrugated steel buried structures (CSBS). It is necessary to review the results of the research work done to date to determine whether any further work is required and if so what direction it should take.

This report reviews the results of the field studies undertaken by the Laboratory on the degradation of reinforcing materials. Different methods of assessing the degree of corrosion are discussed and data presented for metallic elements recovered from four sites. Data are also provided and discussed on the time related changes in the mechanical properties of fibre reinforced plastic strips and coated polyester strips.

The measured rates of degradation are compared to current Highways Agency (HA) requirements for reinforced soil structures and for CSBS, and conclusions made on the reliability of these requirements.

2 Methods of assessing degradation

2.1 Visual assessment

Recovered coupons and lengths of reinforcement can be inspected to provide qualitative assessments of the degree and distribution of degradation. Highly magnified photographs (micrographs) can be taken from these to give a detailed view of the condition of any protective coatings and base metal but, as these only cover a small area, they may not be representative of the reinforcements as a whole.

2.2 Change in weight

Rates of corrosion are commonly determined from changes in weight, see for example Romanoff (1957). The mean rate of corrosion of a coupon can be calculated quite simply from its initial and final weights, its dimensions and the time of burial. However to determine the final weight any corrosion products formed on the specimen must be removed and, as discussed later, this can be problematic. The rate will be under-estimated where corrosion products are not completely removed, but over-estimated where intact coatings and base metal are removed by the cleaning process. This raises the question as to whether weight loss is the best means of defining performance, particularly when the principal concern is the residual tensile strength of the element. Weight loss is a measure of the mean loss in section whilst residual strength is a measure of the minimum section, which may be coincident with the maximum rate of degradation.

The change in the weight of a polymeric material may indicate that it has undergone some degradation, but it may be only associated with a change in moisture content which may not correlate well with a loss in strength.

2.3 Tensile strength

The deterioration of a recovered reinforcement/coupon can be assessed from its initial and residual tensile strength. The percentage loss in strength can be equated to the maximum percentage loss of section and hence be used to estimate the maximum rate of loss in section: this assumes that the original dimensions of the specimens were uniform and that the minimum strength is obtained at the minimum cross-section. It would seem that the maximum loss in strength is a more relevant measure than the mean loss in section to the reduction in stability of a reinforced soil structure. The maximum and mean losses in section can be compared to give an indication of the uniformity of the degradation.

2.4 In situ corrosion tests

The following gives details of the types of in situ test that have been undertaken as part of various corrosion experiments undertaken, or sponsored, by the Laboratory.

2.4.1 Soil resistivity

The resistivity of a soil provides a measure of its ability to act as an electrolyte in an electrochemical cell. The lower the resistivity the greater the capacity of the environment to support corrosion. The tests reported later in this report were carried out using a four-pin Wenner probe comprising 6 mm diameter stainless steel rods set at 38 mm

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[1] *For convenience, and to avoid confusion, the term Laboratory has been used to denote the TRRL and the TRL.*

centres in a block of perspex. A fixed current (I) was passed between the outer pins and the resulting potential difference (V) between the two inner pins was measured. The resistivity (ρ) of the soil between the pins is given by:

$$\rho = \frac{2\pi a V}{I}$$

where *a* is the probe spacing.

2.4.2 Redox potential

Bacterial corrosion can be generated by the presence of sulphate reducing bacteria which thrive in anaerobic conditions. The reduction-oxidation (redox) potential of a soil is a measure of the relative proportion of oxidised and reduced species and therefore may provide a quantitative indication of the ability of the soil to support microbial activity.

The potential of a platinum electrode buried in the soil (with respect to a copper/copper sulphate reference electrode) can be measured using a high input impedance voltmeter and converted to a redox potential using the Nernst equation (see for example Wranglen, 1985):

$$E = E_{pt} + E_{ref} + 59(pH - 7)$$

where
- E = redox potential (mV),
- E_{pt} = potential reading of the platinum electrode relative to a reference electrode (mV),
- E_{ref} = potential reading of the reference electrode relative to a standard hydrogen electrode (= +316 mV),

and the pH is that of the soil.

This test formed part of the assessment procedure given in the 1982 version of BD 12 (Department of Transport (DOT), 1982) for determining the aggressivity of a site to a CSBS. However, as reported by Eyre and Lewis (1987), the reproducibility of the test was poor: the results were sensitive to the degree of compaction and it was difficult to ensure an intimate contact between the probes and the particles in coarse grained soils, which are specified as backfills to CSBS. Thus the test is not included in the current version of BD 12 (DMRB 2.2.6). It does however form part of the corrosivity assessment given in the Specification for Highway Works (MCHW 1) and in BS 8006 (1995). The test is now described in BS 1377: Part 9 (1990).

2.4.3 pH measurement

The pH of a soil can be determined by various means. According to the method given in BS 1377: Part 3 (1990), a 30 g sample of the fraction of backfill passing the 2 mm sieve is made up to 100 g with distilled water. The sample is stirred, left for 24 hours and stirred again before a reading is taken with a digital pH meter.

2.4.4 Electrochemical potential

The in situ electrochemical potential of a metal coupon or reinforcement can be measured using a portable copper/copper sulphate reference electrode placed in the backfill and connected to the test piece via a high impedance voltmeter. The measured potential provides information on the corrosion activity.

2.4.5 Linear polarisation resistance

The measurements of polarisation resistance reported later in this report were carried out using a CAPCIS (ROCC®) system - this comprises a portable linear polarisation resistance (LPR) measurement unit employing an integral data logger/measurement controller. The potential of a test reinforcement is determined first with respect to a reference electrode (either a copper/copper sulphate electrode or another reinforcement). An over potential of 20 mV is then applied to the circuit via another reinforcement, acting as an auxiliary electrode, to polarise the test reinforcement. The resultant current is measured and the rate of corrosion is determined from the following relation.

$$Rate\ of\ corrosion = \frac{a\,k\,i_{corr}}{z\,D}$$

where
- a = atomic weight of the metal, which for zinc = 65.4,
- k = a constant, which for a rate of corrosion expressed in μm/year = 3.27,
- i_{corr} = corrosion current density in µA/cm^2,
- z = number of electrons in electrochemical reaction, which for zinc = 2, and
- D = density of the metal, which for zinc = 7.14 g/cm^3.

2.4.6 Electrochemical impedance

The electrochemical impedance of a reinforcing element can be measured using either a two electrode arrangement (i.e. two reinforcements) or a three electrode arrangement (two reinforcements and a copper/copper sulphate reference electrode). The response of the test reinforcement to the application of a small amplitude (20 mV) sinusoidal wave is determined by a complex plane (Nyquist) plot, of a real (resistive) component against an imaginary (capacitive) component, over a range of frequencies. The plots are used to determine the solution resistance (R_s), the film resistance (R_f), and the charge transfer resistance (R_{ct}) which is inversely proportional to the corrosion rate. The rate of corrosion is determined from the following relation.

$$Rate\ of\ corrosion = \frac{K\,B}{R_{ct}\ A}$$

where
- K = constant, which for zinc and a rate of corrosion in μm/year,
  = 11.4×10^6 cm^2. microns/amps. years,
- B = Stearn-Geary constant = 50×10^{-3} volts,
- R_{ct} = charge transfer resistance in ohms, and
- A = surface area of reinforcement in cm^2.

The rates of corrosion derived from impedance measurements and reported herein were made using a Solartron 1250 frequency response analyzer, a Thompson 251 ministat, and a laptop computer for data acquisition and analysis running under CAPCIS CORRSOFT® IMPED software.

2.4.7 Electrochemical noise

This method of assessing the rate of corrosion is based on the measurement of the spontaneous, but very small and random, changes in the current flow and potential of a corroding metal. The measurements reported herein were made using a portable version of the CAPCIS MUSYC system which monitors noise levels over a narrow band of frequencies.

3 Corrosion experiment at the laboratory

3.1 Details of experiment

3.1.1 Layout of the test site

The test site is in the Laboratory grounds at Crowthorne. The natural soils at the site are the Barton and Bracklesham Beds comprising about a 2 m thick layer of sandy clay underlain by a 4 m thick layer of sand with seams of clay. The site is reasonably level and the watertable is about 2 m below ground level.

Two 4 m x 4 m x 3 m deep pits were excavated in the natural soils at the site and backfilled with either London Clay or Bramshill Sand. A 3 m high embankment was also constructed from the same soils. The layout of the pits and embankment are shown in Figure 1. The metal coupons were placed at 0.5 m vertical spacings in a five-by-five grid as shown in Figures 2 and 3.

3.1.2 Details of the soils

London Clay and Bramshill Sand were selected on the basis that these would respectively represent aggressive and non-aggressive media to the metals. These assessments were made according to the criteria proposed by Booth et al (1967), as reproduced in Table 1: also given on this table are data for the two soils sampled at the borrow pits, and as reported by Cross (1979). The particle size distribution curves for the soils are shown in Figure 4.

Table 1 Assessment of soil aggressiveness towards buried metals (proposed by Booth et al, 1967) and data for soils (from Cross, 1979)

Classification	*Aggre-ssive*	*London Clay (mean values)*	*Non-aggre-ssive*	*Bramshill Sand (mean values)*
Soil property				
Resistivity (ohm-cm)	<2000	1156	>2000	30400
Redox potential at pH = 7 Normal hydrogen electrode (volts)	<0.43 (for clay)	0.263	>0.400 (for granular soils)	0.520
Borderline cases resolved by moisture content (%)	>20	28.5	<20	12.1

The soils were compacted in 125 mm layers using a Wacker vibro-tamper weighing 50.8 kg in the pits, and a Wacker vibro-tamper weighing 101.6 kg on the embankment. The number of passes for each layer was in accordance with the then current (5th Edition) Specification for Road and Bridge Works (DOT, 1976). The mean moisture content and the mean dry density of the compacted London Clay were 26.9 per cent and 1467 kg/m^3 respectively: the values for the compacted Bramshill Sand were 9.6 per cent and 1789 kg/m^3.

The chemical criteria for backfills to reinforced soil applications incorporating metallic reinforcements are reproduced from the, now superseded, 1991 edition of the Specification for Highway Works (SHW) (DOT, 1991) in Table 2a and from BS 8006 (1995) in Table 2b. (It should be noted that both aluminium alloy and copper have been removed from the current SHW (MCHW 1), but the criteria for galvanised steel and stainless steel are the same as given in the earlier version). Relevant data for the soils, sampled in 1990, are provided in Table 3: note that these values differ from those reported by Cross (1979). Excepting the organic content, the sand meets the criteria given in Tables 2a and 2b, and also the grading requirements for uniformly graded granular backfills to reinforced soil structures. It seems likely that the high organic content of both soils was due to contamination

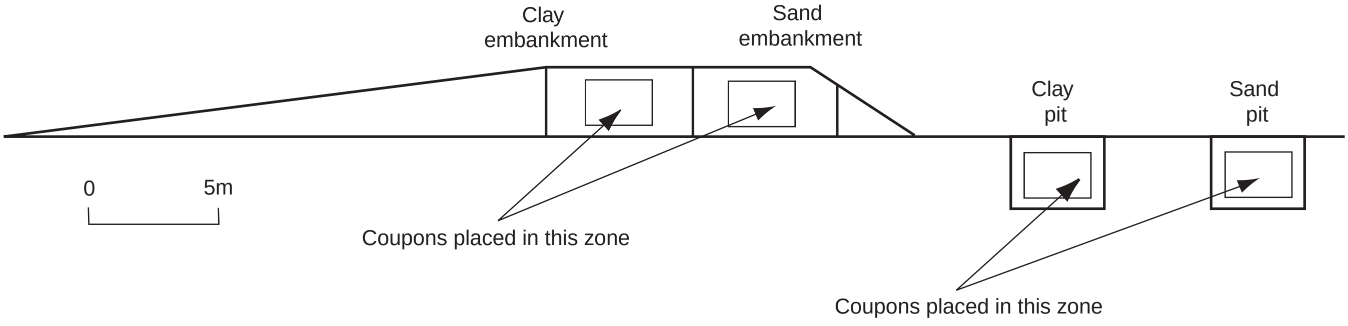
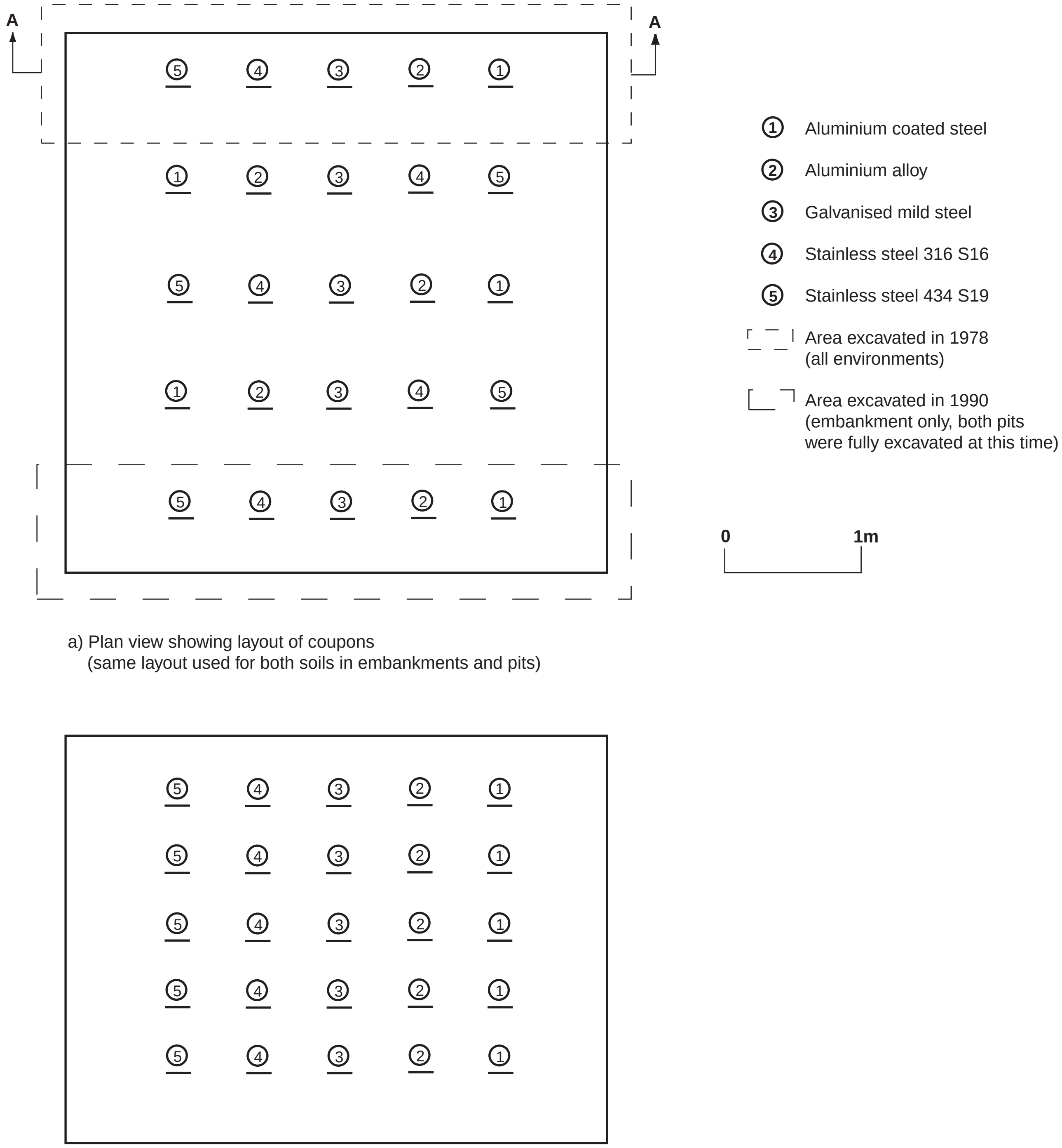


Figure 1 Cross-section through the corrosion experiment site at the Laboratory



a) Plan view showing layout of coupons
(same layout used for both soils in embankments and pits)

b) Section A-A through embankments and pits showing layout of coupons

Figure 2 Layout of the coupons at the corrosion experiment at the Laboratory

Figure 3 View of the embankment for the corrosion experiment at the Laboratory showing the arrangement of coupons and the compaction plant used

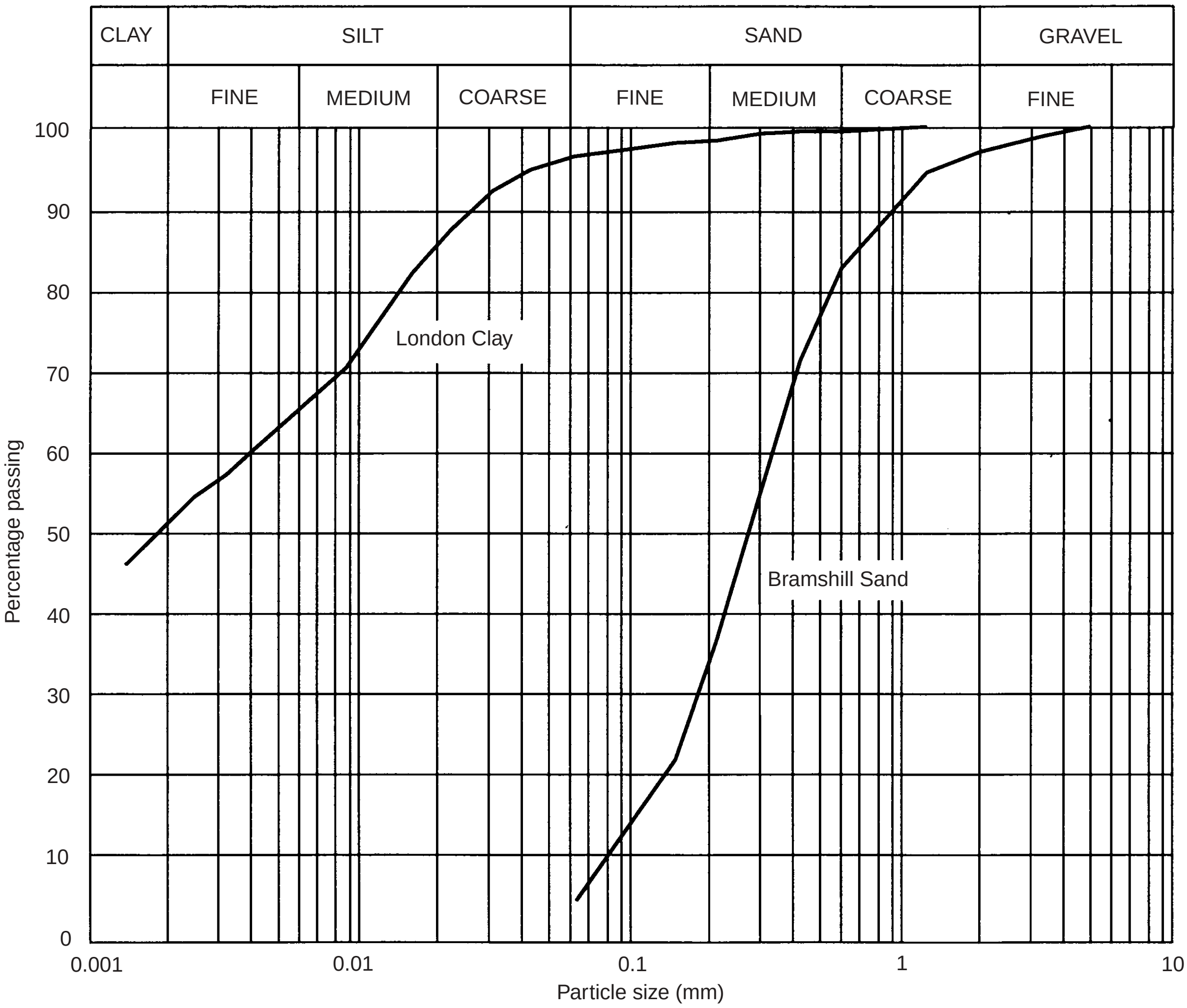


Figure 4 Particle size distribution curves for the soils used in the corrosion experiment at the Laboratory

Table 2 Chemical properties of backfills specified for metallic reinforcements

a Chemical properties of backfills for use with metal components in reinforced soil structures, reproduced from SHW (DOT, 1991)

	Properties of the fill								
Reinforcing element material	*pH value*		*Max chloride ion content (per cent)*	*Max organic content (per cent)*	*Max water soluble sulphate content (gms/litre) (1gm/litre ≡0.1%)*	*Minimum resistivity (ohm-cm)*	*Minimum redox potential (volts)*	*Micro-bial activity index*
	Min	*Max*						
Aluminium alloy	6	8	0.050	0.2	0.5	3000	0.43	Less than 5
Copper	5	9	0.050	0.2	0.5	2000	0.25	
Hot dipped galvanised steel	6	9	0.025	0.2	0.25	5000	0.43	
Stainless steel	5	10	0.025	0.2	0.5	3000	0.35	

b Electrochemical properties of backfills for use with plain steel, galvanised steel and stainless steel materials, after BS 8006 (1995)

		Properties of fill										
		Tests required for all fills					*Additional test required in some cases*					
Reinforcing element material	*Location: OUT or IN water (not sea)*	*pH Min*	*pH Max*	*Max chloride content (per cent)*	*Max water soluble sulphate content (per cent)*	*Min resistivity (saturated) (ohm-cm)*	*Max organic content (per cent)*	*Min redox potential (volts)*	*Max microbial activity index*	*Min resistivity (in situ) (ohm-cm)*	*Max sulphide content (per cent)*
Galvanised or ungalvanised steel	OUT	5	10	0.02	0.10	1000	0.2	0.4	5	5000	0.03
Stainless steel		5	10	0.025	0.10	1000	0.2	0.35	5	3000	0.03
Galvanised or ungalvanised steel	IN	5	10	0.01	0.05	3000	0.2	0.4	5	5000	0.01
Stainless steel		5	10	0.01	0.05	3000	0.2	0.35	5	3000	0.01

Table 3 Results of tests undertaken on soil specimens recovered from the corrosion experiment at the Laboratory in 1990

	Properties of fill, specimens taken from pits and embankment							
Soil	*Moisture content (per cent)*	*pH value*	*Chloride ion content (per cent)*	*Organic content (per cent)*	*Water soluble sulphate content (gms/litre)*	*Resistivity (ohm-cm)*	*Redox potential (volts)*	*Microbial activity index*
London Clay	35.0	7.9	<0.01	1.95	0.058	946	0.39	> 5
Bramshill Sand	9.7	8.6	<0.01	0.85	0.002	14954	0.57	< 5

during sampling. The clay does not meet the chemical nor the grading criteria for backfills to reinforced soil structures.

The current system for classifying the aggressivity of a site for the installation of a CSBS is reproduced from BD 12 (DMRB 2.2.6) in Table 4: it should be noted that it is the aggressivity of the site and not just the soil that is assessed. On this basis the sand environments would be classified as non-aggressive whereas the clay environments would be very aggressive. Were the organic content to be overlooked, the clay sites would be classified as aggressive. However buried structures are required to be surrounded by selected backfills, and although the sand would meet the requirements for such backfills the clay would not.

Table 4 Corrosivity classification, reproduced from BD 12 (DMRB 2.2.6)

Property	*Measured value*		*Points*
Soil type	Fraction passing 63μm sieve	≤ 10%	
	Plasticity Index (PI) of fraction passing 425μm sieve	≤ 6	+1
	Fraction passing 63μm sieve	> 10%	
	PI of fraction passing 425μm sieve	≤ 6	0
	Any grading PI of fraction passing 425μm sieve	> 6 but < 15	-1
	Any grading PI of fraction passing 425μm sieve	≥ 15	-2
	Organic content, or material containing peat, cinder or coke	> 1.0%	-3
Resistivity (ohm - cm)	≥ 10,000		+2
	< 10,000 but ≥ 3,000		+1
	< 3,000 but ≥ 1,000		-1
	< 1,000 but ≥ 100		-3
	< 100		-4
pH of soil	6 ≤ pH ≤ 9		0
	5 ≤ pH < 6		-2
	Less than 5 or more than 9		-5
Soluble sulphates (ppm)	≤ 200		0
	> 200 but ≤ 500		-1
	> 500 but ≤ 1,000		-2
	> 1,000		-4
Chloride ion (ppm)	≤ 50		0
	> 50 but ≤ 250		-1
	> 250 but ≤ 500		-2
	> 500		-4
Sulphide and hydrogen sulphide	No discolouration	of lead acetate paper	0
	Slight to moderate darkening	of lead acetate paper	-2
	Rapid blackening	of lead acetate paper	-3

Points total	*Corrosivity classification*
0 or more	*Non-aggressive*
-1 to -4	*Aggressive*
-5 or less	*Very aggressive*

3.1.3 Details of the metal coupons

The metals were selected on the basis of their actual or potential usage as reinforcing elements for soils. Details of the metal coupons are given in Table 5. A total of 100 coupons of each type of metal were buried at the test site in 1976 and a further 20 mild steel coupons were buried in 1979. Prior to burial the dimensions of each specimen were measured to the nearest 0.1 mm and their weight determined to the nearest 0.01 gm.

Table 5 Details of the metals and coupons used in the corrosion experiment at the Laboratory

Metal	*Description*	*Proof stress (N/mm^2)*	*Nominal dimensions of coupons (mm)*
Aluminium coated steel	Grade CR4 steel coated with 99.7% pure aluminium BS 1449 : Pt 1 : 1972 (now withdrawn)	0.5% > 140	200 x 75 x 1.99
Aluminium alloy	Grade NS8 in H2 condition BS 1470 : 1972 (now withdrawn)	0.2% > 235	200 x 60 x 3.27
Stainless steel	Austenitic type 316 S16, CR BS 1449 : Pt 2 : 1975 (now withdrawn)	0.2% > 400	200 x 60 x 2.77
Stainless steel	Ferritic type 434 S19 BS 1449 : Pt 2 : 1975 (now withdrawn)	0.2% > 250	200 x 60 x 0.50, and 200 x 60 x 1.66
Galvanised steel	Hot dipped galvanised grade 43/25 steel (BS 729 : 1971)	0.2% > 245	200 x 60 x 3.10
Mild steel	Grade 43/25 BS 1449 : Pt 1 : 1972 (now withdrawn)		200 x 60 x 2.55

Before weighing, two number 2 mm diameter holes were drilled at specific positions in the coupons to provide a means of identification. The mild steel coupons were galvanised subsequent to the drilling of the holes, but because the aluminium coated steel coupons were cut from a pre-coated sheet they were not drilled but identified by attaching tags.

A minimum coating weight of 100 gm/m^2, equivalent to a thickness of about 37 μm, was specified for the aluminium coated mild steel coupons. The coupons were cut from pre-coated sheets, i.e. their edges were uncoated.

Prior to burial, the mean measured coating weight on ten of the galvanised mild steel coupons was 328 gm/m^2, equivalent to a thickness of about 46 μm. This was much lower than the minimum requirement of 1000 gm/m^2, proposed in the draft BE 3 (which was issued by the DOT in 1978) - but it was anticipated that this coating would be corroded away within the lifetime of the experiment and so corrosion rates for the galvanising and underlying steel substrate could be determined. However to supplement the database uncoated mild steel coupons were buried at the test site in 1979. These additional coupons were installed at locations where corrosion coupons had already been removed.

BE 3 has now been superseded by BD 70 (DMRB 2.1.5) - the Highways Agency's (HA) implementation standard for BS 8006 (1995). Note that Amendment 1 for this British Standard (BSI, 1998) corrects a typographic error by stating that the minimum coating weight for galvanised mild steel reinforcements is 1000 gm/m^2.

3.2 Details of the 1978 survey

Coupons were removed from the test pits and the embankment in the spring of 1978, i.e. after two years burial. The locations of the excavated coupons are shown on Figure 2a. The extent of the corrosion was not severe and the corrosion products could be removed from the coupons by cleaning them with a soft brush under running water.

The results of the survey are presented in Table 6. The time of burial was too short for other than general conclusions to be drawn from the data: nevertheless the stainless steel coupons did not show any deterioration; there was some loss of zinc and aluminium from the coated mild steel coupons; and pitting was already prevalent on the aluminium alloy coupons.

Table 6 Results of the 1978 survey of the corrosion experiment at the Laboratory

Type of coupon	Number of coupons removed from each environment	Mean rate of corrosion (mean loss in thickness (μm) per face per year)				Pitting corrosion
		Sand embank -ment	*Sand pit*	*Clay embank -ment*	*Clay pit*	*All sites*
Aluminium coated steel	5	1.3	1.5	0.4	0.7	slight
Aluminium alloy	5	0.2	0.3	0.8	0.2	extensive
Stainless steel 316 S16	5	0	0	0	0	Nil
Stainless steel 434 S19	5	0	0	0	0	Nil
Galvanised steel	5	3.4	2.0	5.0	3.9	Nil

As none of the stainless steel coupons showed any signs of deterioration, they were replaced in their original positions when the trenches were backfilled. To provide data on the corrosion of uncoated mild steel, coupons of grade 43/25 steel (BS 1449: Part 1: 1972) were installed during backfilling of the trenches at the positions of the coupons that were excavated and retained. The trenches were backfilled with the excavated material using the same compaction procedure and plant as previously described in 3.1.2.

3.3 Details of the 1990 survey

In December 1990, all the coupons were removed from the test pits and from a section through the embankment as defined in Figure 2a. An excavator was used to remove the bulk of the soil from above the coupons, which were then located using a metal detector and removed by hand: all but a few of the coupons were extracted without damage.

Samples of the two soils were taken during the excavation of the coupons, and the results of the tests performed on them are given in Table 3. The data show that since the study began the moisture content of the London Clay had increased by about 7 per cent whereas it had remained sensibly constant within the Bramshill Sand.

All the recovered coupons were at least partly coated with soil and in most cases by corrosion products. A combination of mechanical and chemical treatments was used to remove the adherent soil particles and corrosion products, but some of the products formed a dense adherent layer on the coupons, which proved difficult to remove without some loss of the uncorroded metal or coatings. An intimate mixture of soil and corrosion products formed a particularly tenacious deposit on the galvanised mild steel coupons. Strength tests were therefore undertaken on selected cleaned coupons.

3.3.1 Cleaning of coupons

Most of the soil and some of the weakly bonded corrosion products were removed by scrubbing the coupons with a nylon brush under running water. A wire brush was sparingly used to remove the thicker deposits of soil. The remaining soil particles were almost all removed by immersing the specimens in water and applying ultrasonic vibration, a surfactant being added to aid the cleaning process: this was a time-consuming process.

Following mechanical cleaning, the coupons were immersed in a solution of an oxide remover, trade name 'Biox': this removes oxides from metals by the action of enzymes. The period of immersion was varied according to the severity of the corrosion, but the cleaning process could be accelerated by warming the solution. It was found by experiment that the rate of cleaning could be substantially increased by applying ultrasonic vibration to the bath: the vibration also warmed the solution. The combined action of 'Biox' and ultrasonic vibration was effective in cleaning the various types of coupon, but on the galvanised coupons it also tended to strip away any remaining zinc. Experiments showed that the rate of loss of galvanising due to the combined action of the 'Biox' solution and ultrasonic vibration was about 6 gm/m^2/hour: the original coating weight was about 328 gm/m^2.

Other cleaning methods as described by Romanoff (1957) and as recommended in BS 7545 (1991) were therefore tried on the galvanised coupons. Following the method described by Romanoff (1957), the specimens were immersed for 30 minutes in a 10 to 15 per cent strength solution of ammonium chloride at between 75 and 80^o C. Following removal from the solution the specimens were then scrubbed under running water and dried. However this method was not particularly effective even when alternate cycles of immersion and scrubbing were used. (Experiments showed that when ultrasonic vibration was applied to the bath the galvanising was removed at a rate of about 33 gm/m^2/hour). The method given in BS 7545

(1991) is based on immersion of the specimens in an aqueous solution of ammonium peroxodisulphate at between 20 and 25° C. The solution was made up of 100 gm of the salt per litre, and the suggested time of immersion was 5 minutes. This was effective in cleaning areas of bare steel but prolonged immersion led to a loss of galvanising. Therefore following a 5 minute immersion in the solution the coupons were scrubbed using nylon or wire brushes under running water and then dried. Cycles of immersion, scrubbing and drying were continued until the change in the dry weight of the coupon between successive cycles was less than 0.05 gm.

3.3.2 Tensile strength tests

Tensile tests were undertaken on a number of cleaned coupons to establish correlations between weight loss and strength. The tensile strength of a coupon was determined, in stress units, using the original dimensions of the coupon, but where necessary taking account of any drilled holes. The tests were undertaken at a rate of strain of 10 per cent per minute using a Dennison T.42.C.2 tensile testing machine housed at Middlesex University. The lightly corroded coupons failed at the position of the holes drilled for identification purposes, nonetheless some useful conclusions can be drawn from the test data.

3.3.3 Results

Rates of corrosion have been calculated from the percentage loss in weight and the original dimensions of the coupons, and reported herein in terms of the mean loss in thickness per face per year.

The rate of loss at the minimum cross-section, calculated from the strength tests, was compared to the mean loss for the coupon as a whole. The minimum cross-section could of course occur over part of the coupon that was clamped in the tests (and so would be prevented from rupturing) and the variation in the strength of pristine coupons also introduces some element of uncertainty into the analysis. Moreover this approach could only be used rigorously for coupons that did not fail at a cross-section containing an identification hole. Nevertheless the ratio of the rate of loss at the minimum cross-section and the mean loss for the coupon as a whole provides a measure of the uniformity of corrosion, i.e. a pitting index (K). Such an approach was used by Bulow (1948) and more recently by Darbin et al (1988): the latter defined the coefficient of corrosion heterogeneity (K) as the ratio of the loss in tensile strength to the mean loss in thickness. The values of the pitting index and the coefficient of corrosion heterogeneity are synonymous: the value is a function of the dimensions of the test coupon. Given the problems associated with determining the losses in tensile strength and thickness, the scatter in the values of K could be expected to be fairly wide: furthermore the values of K could increase or decrease with time. Nonetheless, unless the sizes of the test coupons and prototype components are similar, the application to design of a mean value of K derived from a batch of coupons would be unconservative.

Aluminium coated steel

These coupons were cut from pre-coated sheets and so the steel along the perimeter did not have a covering layer of aluminium. Corrosion of the coupons occurred predominantly along these uncoated edges, but pitting and blistering of the upper and lower surfaces were also common. Typical corrosion features are shown on Figure 5.

Figure 5 Typical corrosion features on the aluminium coated steel coupons recovered from the corrosion experiment at the Laboratory in 1990, from the sand pit (number 85) and the clay embankment (24)

The condition of the edges of the recovered coupons varied from a general roughening to a loss of several millimetres of metal: the degradation being much more severe in the clayey soil than the sandy soil. The pits on the upper and lower surfaces were up to 0.5 mm deep and up to 5 mm in diameter, again the frequency and the size of the pits were greater on coupons recovered from the clayey soil. The pits seemed to form along rather ill-defined lines running parallel to the length of the coupons, and it seems likely that this pattern was due to damage or contamination of the surface by roller feeds during manufacture of either the sheets or the coupons.

The losses of aluminium and of steel could not be determined separately and the corrosion rates given in Table 7a have been calculated from the percentage change in weight and the original dimensions of the coupons. These data show that the rate of corrosion in the clayey soil was about 3 times that in the sandy soil. The rates determined for the coupons recovered from the embankment fell within the ranges determined for the coupons recovered from the pits, and therefore little can be concluded regarding the relative corrosivity of the environments.

Table 7 Results for aluminium coated mild steel coupons recovered from the corrosion experiment at the Laboratory in 1990

a Mean rates of corrosion based on weight loss

Environment	*Number of coupons*	*Range of rates of corrosion (mean loss in thickness (μm) per face per year)*	*Mean rate of corrosion (mean loss in thickness (μm) per face per year)*
Sand embankment	4	0.79 - 1.19	0.97
Sand pit	18	0.40 - 2.54	1.00
Clay embankment	4	2.81 - 3.62	3.25
Clay pit	14	1.40 - 4.89	2.46

b Tensile strength and weight loss

Coupon reference	*Loss in weight (per cent)*	*Tensile strength (N/mm^2)*	*Strain at rupture (per cent)*	*Equivalent loss in thick -ness (μm) per face per year at minimum cross-section**	*Pitting index** (K)*
88 SP	0.91	379	22	1.05	1.8
71 SE	1.07	370	20	2.63	3.7
86 SP	2.25	369	18	2.80	1.9
43 CP	3.74	364	17	3.68	1.5
50 CP	4.93	359	16	4.56	1.4
37 CP	7.46	310	7	13.14	2.7

* *based on a pristine strength of 385N/mm² and assuming % loss of strength ≡ % loss of section*

** *ratio of rate of loss at minimum cross-section and mean rate for coupon as a whole*

SE: Sand embankment SP: Sand pit CP: Clay pit

The data presented in Table 7b show, not unexpectedly, that the higher the loss in metal the lower the retained strength of the coupons: the reduction in strength was accompanied by a reduction in the strain to rupture.

Aluminium alloy

All the coupons suffered from pitting attack, with typically less than 5 per cent of the surface area of the coupons being affected. Typical corrosion features are shown in Figure 6, and a profile through a particularly badly corroded section of a coupon is shown in Figure 7. The local rates of corrosion were therefore much higher than the mean values given in Table 8a. (The profile given in Figure 7 shows a maximum depth of metal loss of about 2.2 mm equivalent to a rate of corrosion of about 150 μm per year).

The data given in Table 8a show that the rate of corrosion in the clayey soil was much higher than that in the sandy soil and that the clay embankment was a more aggressive environment than the clay pit. The data given in Table 8b show that a loss of weight of between 2 and 3 per cent was equivalent to about a 30 per cent reduction in strength.

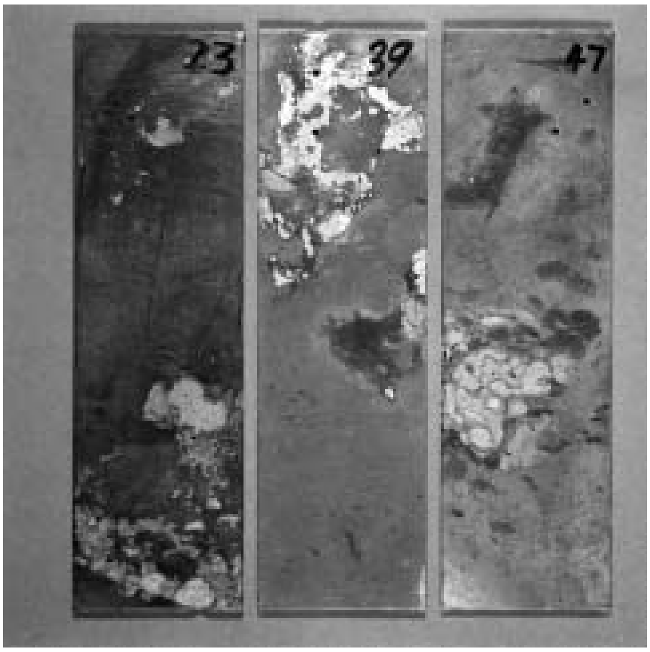


Figure 6 Typical corrosion features on aluminium alloy coupons recovered from the corrosion experiment at the Laboratory in 1990, from the clay embankment (number 23) and the clay pit (39 and 47)

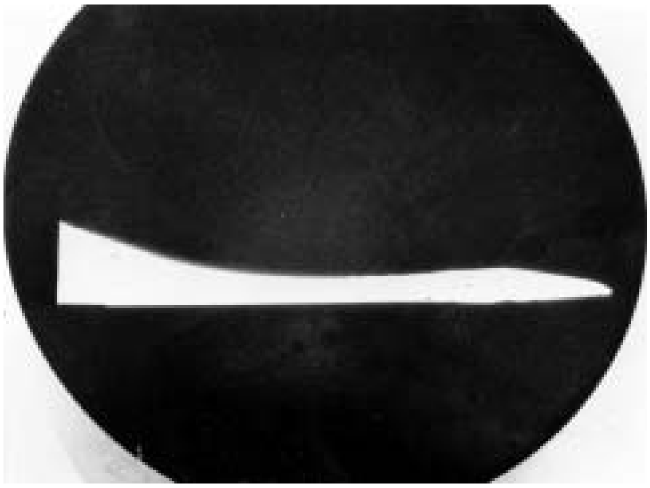


Figure 7 Profile of a particularly heavily corroded section of an aluminium alloy coupon recovered from the corrosion experiment at the Laboratory in 1990

Stainless steels

Corrosion of the stainless steel coupons was restricted to minor surface blemishes: these were removed by immersion in the 'Biox' solution, and after cleaning the coupons showed negligible loss in weight.

Galvanised mild steel

It proved difficult to remove all the corrosion products from the coupons without stripping away some of the remaining galvanising. Thus the rates of corrosion for some coupons may have been over-estimated by up to about 10 per cent. The rates, given in Table 9a, are similar to those reported by Darbin et al (1988) from a number of

Table 8 Results for aluminium alloy coupons recovered from the corrosion experiment at the Laboratory in 1990

a Mean rates of corrosion based on weight loss

Environment	*Number of coupons*	*Range of rates of corrosion (mean loss in thickness (μm) per face per year)*	*Mean rate of corrosion (mean loss in thickness (μm) per face per year)*
Sand embankment	5	0.00 - 0.03	0.01
Sand pit	19	0.00 - 0.14	0.04
Clay embankment	3	0.83 - 3.10	2.25
Clay pit	13	0.03 - 2.36	1.02

b Tensile strength and weight loss

Coupon reference	*Loss in weight (per cent)*	*Tensile strength (N/mm^2)*	*Strain at rupture (per cent)*	*Equivalent loss in thick -ness (μm) per face per year at minimum cross-section**	*Pitting index** (K)*
50 CP+	0.05	333	5	n/a	n/a
44 CP+	0.04	330	5	n/a	n/a
21 CE	0.80	290	3.5	16.3	19.6
25 CE	3.00	241	2	32.3	10.4
22 CE	2.29	237	1.5	33.6	14.2
37 CP	2.21	222	1.5	38.5	16.9

n/a: not applicable

+ failure through cross-section containing hole drilled for identification purposes

** based on pristine strength of 340N/mm² and assuming % loss of strength ≡ % loss of section*

*** ratio of rate of loss at minimum cross-section and mean rate for coupon as a whole*

CE: Clay embankment CP: Clay pit

Table 9 Results for galvanised mild steel coupons recovered from the corrosion experiment at the Laboratory in 1990

a Mean rates of corrosion based on weight loss

Environment	*Number of coupons*	*Range of rates of corrosion (mean loss in thickness (μm) per face per year)*	*Mean rate of corrosion (mean loss in thickness (μm) per face per year)*
Sand embankment	3	1.64 - 1.88	1.74
Sand pit	5	1.23 - 2.49	1.91
Clay embankment	4	2.80 - 8.42	5.18
Clay pit	12	4.78 - 7.66	5.67

b Tensile strength and weight loss

Coupon reference	*Loss in weight (per cent)*	*Tensile strength (N/mm^2)*	*Strain at rupture (per cent)*	*Equivalent loss in thick -ness (μm) per face per year at minimum cross-section**	*Pitting index** (K)*
25 CE+	2.54	373	20	n/a	n/a
74 SE+	1.54	371	20	n/a	n/a
39 CP+	4.41	365	20	n/a	n/a
71 SE+	1.51	355	16	n/a	n/a
36 CP+	6.09	350	22	n/a	n/a
23 CE	7.58	229	8	42.8	5.1

n/a: not applicable

+ failure through cross-section containing hole drilled for identification purposes

* based on pristine strength of 373N/mm² and assuming % loss of strength ≡ % loss of section

** ratio of rate of loss at minimum cross-section and mean rate for coupon as a whole

SE: Sand embankment CE: Clay embankment CP: Clay pit

in service reinforced soil structures that incorporated 3 mm thick mild steel strips having a zinc coating of 30 μm (equivalent to a coating weight of 215 gm/m²). Data for those specimens where the remaining galvanising had been substantially lost due to the cleaning process have not been included in the table.

As shown in Figure 8 corrosion was not particularly uniformly distributed over the surface of the coupons, but even the more highly corroded specimens did not suffer from intense pitting. The majority of the coupons recovered from the sandy soil retained at least a partial covering of zinc, and corrosion of the exposed steel was not severe. The coupons recovered from the clayey soil were more severely corroded but most retained a thin patchwork cover of zinc. Cross-sections through the coupons, as shown for example in Figure 9, show that there was no substantial pitting of the exposed steel.

The data given in Table 9a show that the clayey soil was a much more corrosive medium than the sandy soil, but little can be concluded about the relative corrosivity of the embankment and the pits.

The data from the tensile tests are given in Table 9b. Five of the six coupons ruptured at a cross-section containing one of the holes drilled for identification purposes, the other coupon (23CE) had a retained strength of 229 N/mm² and ruptured at a strain of about 8 per cent: thus a loss in weight of about 7.6 per cent was accompanied by a reduction in strength of about 38 per cent and a substantial reduction in the ductility of the material.

Mild steel

The coupons were corroded over most of their surfaces, but pitting was not prevalent. The rates of corrosion given in Table 10a confirm that the clayey soil was more corrosive than the sandy soil.

Data from the tensile tests are provided in Table 10b. All the coupons ruptured at a cross-section that included one of the holes drilled for identification purposes and there is no consistent relation between the loss in weight and the reduction in tensile strength.

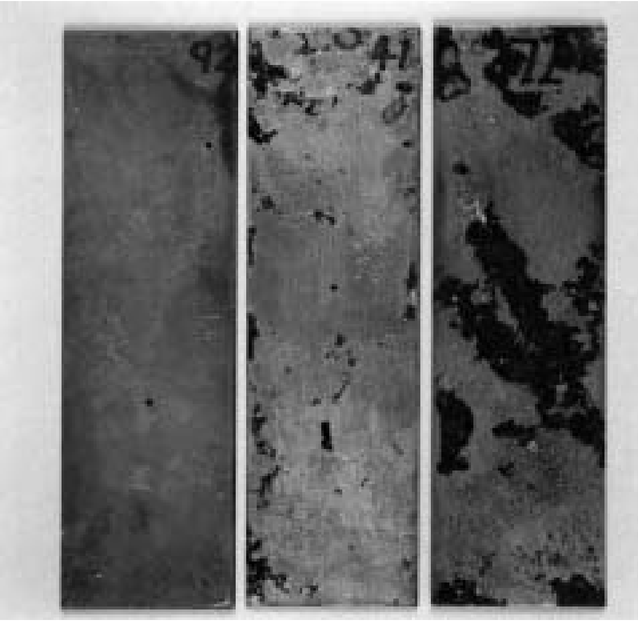


Figure 8 Typical corrosion features on galvanised mild steel coupons recovered from the corrosion experiment at the Laboratory in 1990, from the sand pit (number 92), the clay pit (41) and the sand embankment (72)

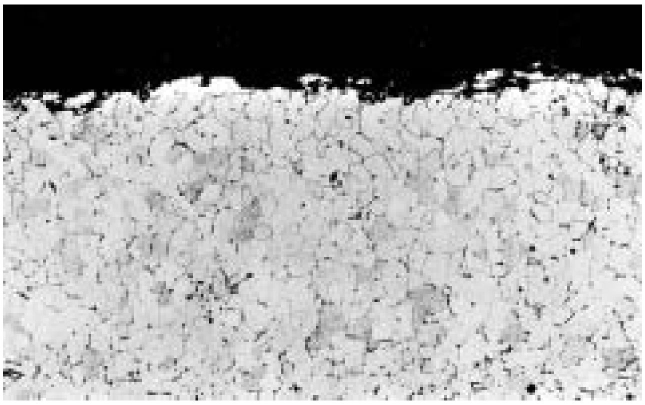


Figure 9 Cross-section through a galvanised mild steel coupon recovered from the corrosion experiment at the Laboratory in 1990, showing no substatntial pitting of the exposed steel surface

3.4 Discussion

It proved difficult to remove adherent soil particles and corrosion products from some of the coupons without some loss of metal, but such deposits must be removed to determine the loss in weight and hence the reduction in thickness per year. However it may be inappropriate to express the rate of corrosion for coated materials in terms of the mean reduction in thickness per year, particularly, as is the case with aluminium coated steel, when the densities of the coating and the substrate are quite different. Similarly it may be misleading to express the rate of corrosion in terms of the mean reduction in thickness when pitting is prevalent, as was the case with the aluminium alloy. Thus the loss in strength may be a more convenient and a better measure of the severity of corrosion than the mean loss of thickness. The ratio of the equivalent rate of loss of thickness at the minimum cross-section (calculated from the proportionate strength loss) and the mean loss for a coupon as a whole provides a means of assessing the uniformity of corrosion.

Table 10 Results for mild steel coupons recovered from the corrosion experiment at the Laboratory in 1990

a Mean rates of corrosion based on weight loss

Environment	*Number of coupons*	*Range of rates of corrosion (mean loss in thickness (μm) per face per year)*	*Mean rate of corrosion (mean loss in thickness (μm) per face per year)*
Sand pit	3	4.16 - 7.45	5.59
Clay pit	5	5.65 - 14.02	8.24

b Tensile strength and weight loss

Coupon reference	*Loss in weight (per cent)*	*Tensile strength (N/mm^2)*	*Strain at rupture (per cent)*
16 CP$^+$	5.93	324	14
18 CP$^+$	6.74	321	14
3 SP$^+$	5.40	317	20
20 CP$^+$	14.71	283	14
19 CP$^+$	8.84	268	10

$^+$ *failure through cross-section containing hole drilled for identification purposes*

Mean pristine strength = 383N/mm^2

SP: Sand pit CP: Clay pit

3.4.1 Aluminium coated steel

The aluminium coated steel coupons were heavily corroded along their uncoated edges and pitting was evident on their upper and lower faces. Such strips should not be used for reinforced soil applications and indeed BD 70 (DMRB 2.1.5) does not permit their use.

In 1990, the then current Design Standard for corrugated steel buried structures, BD 12 (DOT, 1988), specified rates of corrosion for the buried surfaces of aluminium coated steels of 1 μm per face per year for non-aggressive environments and 4 μm per face per year for aggressive environments. The sand environments would be classified as non-aggressive, and the mean rates of corrosion in these environments were, up to 1990, close to 1 μm per face per year. The maximum mean rates of corrosion of the coupons in the clay were also close to the design value of 4 μm per face per year for aggressive environments. However in some cases the loss in steel may have outweighed the loss in aluminium, and where pitting had occurred the localised rates of corrosion were substantially higher than the mean rates. The relatively low values of the pitting index were probably due to the bias introduced by the loss of steel along the uncoated edges of the coupons. Aluminium coatings are prone to pitting attack and so are not well suited for steel buried structures where aggressive species, such as chloride ions, could promote pitting. Such coatings may be damaged during construction and so not provide much protection to the exposed steel substrate. On the basis of data from this and other studies, the extant version of BD 12 (DMRB 2.2.6), permits only the use of hot-dip galvanised coatings for CSBS.

A comparison of data given in Tables 6 and 7a shows that the rate of corrosion in the sandy soil had remained reasonably constant from the start of the study. The data for the clayey soil show an acceleration in the mean rate of corrosion, and therefore importantly indicate no reduction in the rate of pitting attack. However the rates of loss of aluminium and of steel were not determined separately.

3.4.2 Aluminium alloy

BE 3 (DOT, 1978) stipulated that, for frictional fills and a design life of 120 years, aluminium alloy reinforcing strips should have a 0.15 mm thick layer of sacrificial metal per face: this corresponds to a loss in thickness of 1.25 μm per face per year. The mean rates of corrosion for the coupons in the Bramshill Sand ranged up to 0.14 μm per face per year but local rates of corrosion were substantially higher than the mean values. For cohesive frictional fills and a design life of 120 years, such strips were required to have a 0.3 mm thick layer of sacrificial metal per face: this corresponds to a loss in thickness of 2.5 μm per face per year. The London Clay fill could be expected to be much more corrosive than soils that meet the requirements for cohesive frictional backfills: although the maximum mean rates of corrosion for the coupons recovered from the clay were close to this value, again the predominant form of corrosion was pitting.

On the basis of the results from this and other studies, for example as reported by Murray (1992), aluminium alloys were removed from the list of approved materials for use as soil reinforcements.

A comparison of the data given in Tables 6 and 8a shows that since the start of the study the mean rates of corrosion substantially decreased in the sandy soil but increased in the clayey soil. This difference in behaviour highlights the problem of extrapolating data from short-term corrosion trials to long lifetimes, particularly with materials that are prone to pitting.

3.4.3 Stainless steels

None of the stainless steel coupons were corroded. However ferritic stainless steels, such as 434 S19, are known to be susceptible to pitting attack particularly in chloride contaminated soils - evidence of this phenomenon has been presented by Murray (1992) - and so they are not currently permitted in the UK for reinforced soil applications. The effect of chloride contamination on performance was not covered in this study.

3.4.4 Zinc coatings

The zinc coatings had been penetrated over areas of the coupons buried in the sandy soil and corrosion of the underlying steel substrate had occurred. Thus, contrary to the findings of Darbin et al (1988), the partial covering of zinc did not provide complete galvanic protection to the exposed steel. The mean rates of corrosion may reasonably be compared to current requirements for zinc coatings, but it should be recalled from above that the mean rates of some of the coupons may have been overestimated by up to about 10 per cent.

According to BD 70 (DMRB 2.1.5), galvanised mild steel reinforcements are required to have a coating weight of 1000 gm/m^2, which is equivalent to a thickness of about 140 μm. The mean rates of corrosion of the coupons buried in the sand ranged up to 2.5 μm per face per year, and so for this rate the lifetime of a 140 μm thick coating would be 56 years, i.e. about 30 per cent longer than anticipated by King (1977).

BD 12 (DMRB 2.2.6) gives assumed rates of corrosion of galvanising of 4 μm and 14 μm per year for non-aggressive and aggressive environments respectively. The data given in Table 9a suggest that these rates are reasonably conservative, but the sand site may of course not represent the most severe environment that could be classed as ‘non-aggressive’.

Assessments based on the mean rates of corrosion suggest that the current requirements for galvanised steel reinforcements and for CSBS are reasonably conservative. However, although pitting was not pronounced, corrosion was not particularly uniformly distributed and some level of conservatism is required when using mean rates of corrosion. For the sand environments, the design rates of corrosion given in BD 12 (DMRB 2.2.6) were about 1.5 to 3 times the mean rates for the coupons. Darbin et al (1988) reported mean values of K of 1.2 and 1.5 for mild steel coupons having 25 μm and 50 μm thicknesses of galvanising respectively, but the individual values of K were widely scattered particularly for the lightly corroded specimens. And, as shown by the data in Table 9b, the pitting index for coupon 23CE was about 5, suggesting that the level of conservatism built into BD 12 (DMRB 2.2.6) may be inadequate. The value of the pitting index, and any other measure of the uniformity of corrosion, is a function of the size of the coupons, and statistical analysis would have to be used to determine how non-uniform corrosion affects the retained strength of larger galvanised steel components. Engineering judgement is required to assess how the loss in strength of a component affects the overall performance of a structure. The lack of data prevents any sensible application of statistical analyses here, but the use in design of mean rates of corrosion, however expressed, seems unsatisfactory.

A comparison of the data given in Tables 6 and 9a suggests that the mean rate of corrosion in the sand embankment may have reduced during the course of the study. The data are however limited in extent, and those for the other environments suggest little if any change in the mean rates of corrosion. This is not in agreement with the findings of Darbin et al (1988) that the rate of corrosion reduces with time: their data generally showed that the rate of corrosion reduced with the logarithm of exposure time.

3.4.5 Mild steel

BS 8006 (1995) specifies a sacrificial thickness of 0.75 mm per face for steel reinforcements in fills of class 6I, 6J, 7C and 7D as defined in Table 6/1 of the SHW (MCHW 1). The data given in Table 10a are limited in extent, but they show that the mean rates of corrosion in the sandy and the clayey soils ranged up to 7.45 μm and 14.02 μm per face per year respectively. The Bramshill Sand met the grading requirements for 6I and 6J materials but not the uniformity coefficient limits. Nonetheless the data suggest that the mean lifetime of a 0.75 mm thick layer in Bramshill Sand would be about 100 years. Values for the pitting index could not be properly determined, but for the coupons listed in Table 10b the ratio of the loss in strength to the mean loss in weight ranged from 1.8 to 3.4, which suggests that the sacrificial layer may be lost at some cross-section of a reinforcement buried in the sand in about 30 years. The London Clay does not satisfy the requirements for any of the above classes but the data suggest that the mean lifetime of a 0.75 mm thick layer in this soil would be in excess of 50 years.

In BD 12 (DMRB 2.2.6) the required thickness of sacrificial steel for external buried surfaces is determined from the following equations:

$T = 22.5t^{0.67}$ for non-aggressive environments,
$T = 40.0t^{0.80}$ for aggressive environments.

where T is the thickness in μm, and

t is the life of sacrificial thickness of steel in years.

For the time of burial in this study, i.e. about 11 years, the losses in steel according to the above equation would be 112 μm and 272 μm for the non-aggressive and aggressive environments respectively. The total mean losses determined in this study ranged up to 82 μm in the Bramshill Sand and 154 μm in the London Clay. On this basis there seems to be a degree of conservatism in the assumed rates but again the sand site may not represent the most severe environment that could be classified as 'non-aggressive'. Many more data than provided from this site are required to confirm the reliability of the above equations.

A comparison of the data given in Tables 9 and 10 shows that the mean rates of corrosion for the mild steel coupons were substantially higher than recorded on the zinc coated ones.

3.4.6 Lifetimes of galvanised mild steel reinforcements

Typically, six metre long, 60 mm wide, 5 mm thick galvanised mild steel strips are used for reinforced soil retaining walls. The combined surface area of the eight 200 mm long, 60 mm wide coupons recovered from the sand environments was therefore about a quarter of the face area of a typical reinforcement. The data given in Table 9a show that the maximum mean rate of corrosion for these coupons was about 2.5 μm per face per year. Assuming a pitting index of 5, then the maximum loss at any cross-section of any of the coupons would be about 12.5 μm per face per year. A coating weight of 1000 gm/m^2 is equivalent to a thickness of about 140 μm and so its minimum lifetime would be about 11 years. Similarly, as discussed above, the minimum lifetime of a 0.75 mm thick layer of sacrificial steel would appear to be about 30 years. Thus the minimum lifetime of the sacrificial coating and metal on a reinforcement could be as low as 40 years. This is probably an underestimate, but a 120 year life can only be attained, using the above maximum mean rates of corrosion, with a pitting index of less than 1.3 and this appears to be rather optimistic. The critical assumptions in the above calculation are that the rate of corrosion and the distribution of corrosion do not change with time. Many more data than available from this site are required to properly test these assumptions.

Although the foregoing may seem to be cause for concern, it should be recognised that reinforced soil structures have a built-in factor of safety, over and above the provision of sacrificial metal, of at least two. Moreover the maximum reduction in cross-section may not occur at the section carrying the highest tensile force, and the rupture of any one reinforcement is unlikely to result in the collapse of a structure.

3.5 Summary

The aluminium coated steel coupons were cut from pre-coated sheets and the uncoated edges of the coupons were heavily corroded, particularly within the clayey soil. Strips manufactured from such pre-coated sheets should not be used for reinforced soil applications or for CSBS. Similarly the coupons of aluminium alloy suffered from intensive pitting and such alloys should not be used for these applications.

The stainless steel coupons showed only surface staining, but the use of austenitic stainless steel is limited by its relatively high cost and ferritic stainless steel by its proneness to pitting corrosion in the presence of chloride and sulphate ions.

The galvanised mild steel coupons suffered from a non-uniform loss of cross-section, but intensive pitting was not evident. The partial covering of zinc left on some of the coupons did not provide complete galvanic protection to the exposed steel. Moreover some of the data did not show a substantial decrease in the rate of corrosion with the time of burial: these findings are not in accord with those of Darbin et al (1988).

The rate of corrosion of the mild steel coupons was higher than for both the aluminium coated and the zinc coated coupons: but, although corrosion was not particularly uniform, intensive pitting was not evident.

As could be expected the London Clay proved a more aggressive medium than the Bramshill Sand. However, with the exception of the aluminium alloy coupons buried in the clayey soil, there was no discernable difference in the rates of corrosion for the pit and embankment environments.

3.6 Future work

Although all the coupons have been removed from the pits, in the embankment there remains 15 of each coupon type in each of the soils. These could be removed in one or more operations. Removal of the coupons would require the use of formwork to support the sides of the trench and allow safe access.

The greater the number of coupons removed at any one time the more reliable the statistical analysis and interpretation of the test data, but the less the number of visits that can be made. A compromise must therefore be struck between the accuracy of the analysis of the data and the number of data points that can be plotted between for example, the loss in strength and elapsed time. A minimum of five tests on each material would seem to be necessary to form any reliable conclusions, and the arrangement of the coupons would make it difficult to extract them in anything other than groups of five. However it has to be admitted that five test results are unlikely to provide particularly tight or reliable statistical bounds to the data - on the other hand such bounds have not been established from the earlier data sets.

The coupons could be cleaned, weighed and their tensile strength determined. Cleaning the coupons would be quite time consuming, but because they were pre-weighed the coupons provide a good opportunity for comparing corrosion rates calculated from loss of weight and from loss of tensile strength. The scale of the exercise could be reduced by omitting tests on the aluminium coated steel and aluminium alloy: the data from these coupons should not influence future editions of BD 70 (DMRB 2.1.5) or BD 12 (DMRB 2.2.6) and therefore may be of only academic interest.

4 Experimental retaining structure at the Laboratory

4.1 Details of experiment

An experimental reinforced soil structure was built in the grounds of the Laboratory during 1977 and 1978. Although the primary objective of the experiment was to investigate the constructability of the systems and to assess the adequacy of design methods, following completion of this work the structure has acted as a useful source of ageing reinforcements - both metallic and non-metallic. Full details of the construction and performance of the structure have been reported by Brady (1992).

The structure was built close to the corrosion experiment site described above. A plan of the structure is shown in Figure 10 and the layout of the reinforcements through a section of the structure is shown in Figure 11.

The 6 m high structure was built using three soils of approximately equal thickness, sheets of geosynthetic were used to separate the different backfills. The bottom layer of fill is a sandy clay, known locally as Bray Clay; the middle layer is formed of a gravelly sand; and the upper layer of fill is a silty clay, known as Harlington Brickearth. Some details of the fill materials are given in Table 11.

Most of the structure was constructed using 450×450×80 mm (thick) concrete units; cast in dowels and sockets provide an interlock between these units. A cast in steel bracket and vertical pin secure most types of reinforcement, except for the geosynthetic strip which was wrapped around a horizontally aligned bar, and the concrete planks which, because of their higher load carrying capacity, were connected to two adjacent horizontal units.

The other types of facing were:

i 2.3 m high by 1 m wide interlocking reinforced concrete units;

ii a post-and-panel system comprising precast reinforced concrete posts and paving slabs;

iii a lightweight metal facing unit manufactured from aluminium coated mild steel.

The connection between these facing units and the reinforcements was the same as for the 450 mm concrete units described above.

The following materials were used as reinforcements in the structure:

i Stainless steel.

ii Galvanised mild steel.

iii PVC coated mild steel.

iv Aluminium coated (calorised) mild steel.

v ‘Paraweb’ (continuous aligned polyester filaments in a polyethylene sheath).

vi Glass fibre reinforced plastic (GRP).

vii Pre-stressed concrete planks.

Details of dimensions, grades and strengths of the reinforcements are given in Table 12.

4.2 Recovery of reinforcements

Paraweb straps were recovered from the structure in 1984, 1990 and 1994 and GRP reinforcements were recovered in 1984, 1988 and 1993. Specimens of the other reinforcement types were also excavated in 1984. All the specimens of Paraweb have been recovered from the upper silty clay layer, while the other types of reinforcement have been excavated from all three fill materials. On each occasion the bulk of the overburden was removed either by a face shovel or a back actor, with the remainder being removed by hand tools. The reinforcements were labelled to define the vertical column (number) and horizontal row (letter) that they were recovered from, as shown in Figures 10 and 11.

4.2.1 Inspection

All the reinforcements excavated from the structure in 1984 were inspected. Most of them were not subject to tensile tests or measurements of coating thickness because either the material was not used commercially for soil reinforcements, or if it was, there were insufficient data, regarding for example initial strength or coating thicknesses, to make other than a qualitative assessment of their condition. Nevertheless the condition of some of the materials could sensibly be compared to coupons of the same material recovered from the adjacent corrosion experiment site.

Loosely bonded particles of soil were removed by cleaning with a soft brush under running water. The non-metallic reinforcements were then inspected for any visible signs of degradation and the metallic ones for evidence of corrosion.

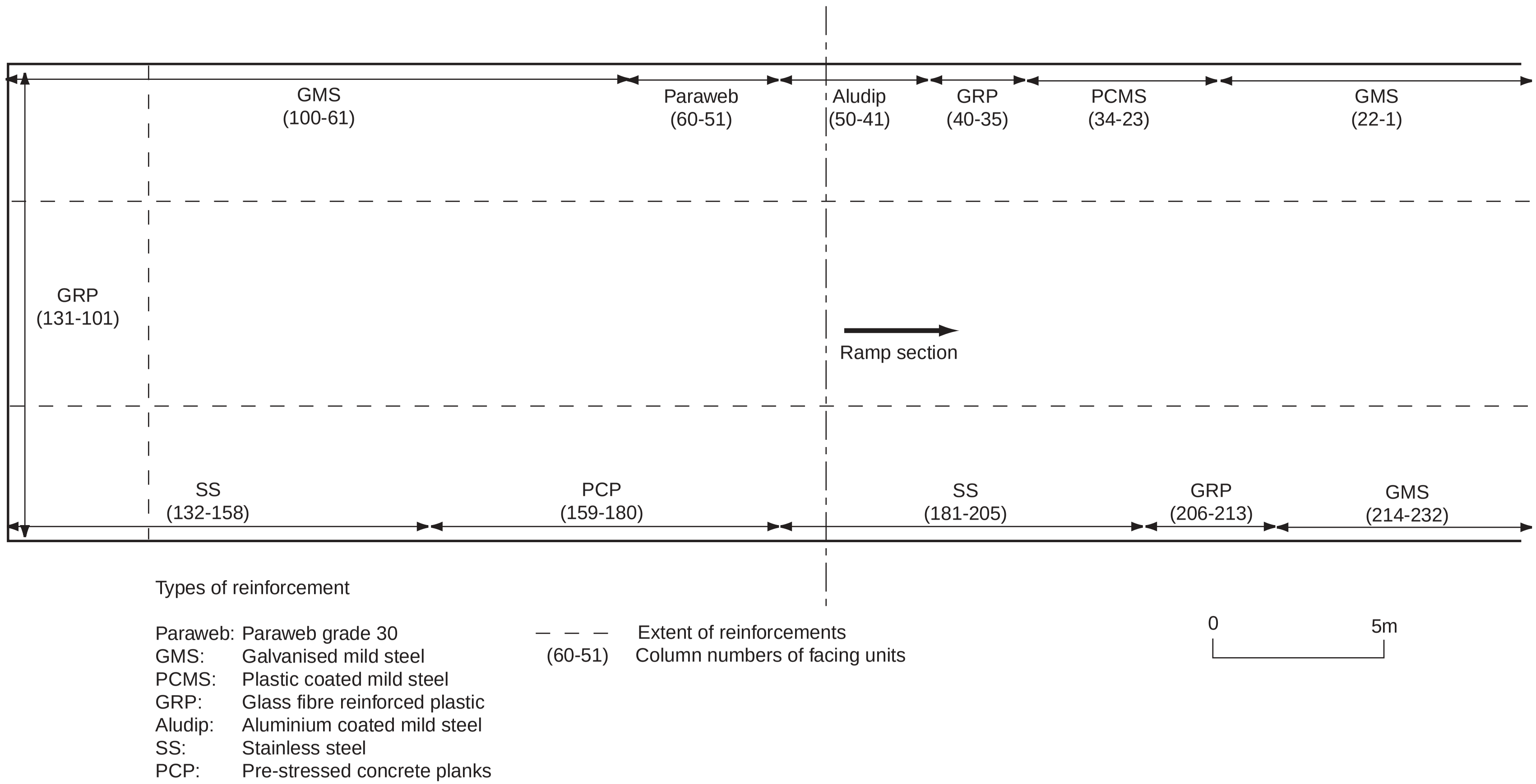


Figure 10 Plan view of the experimental reinforced soil structure at the Laboratory

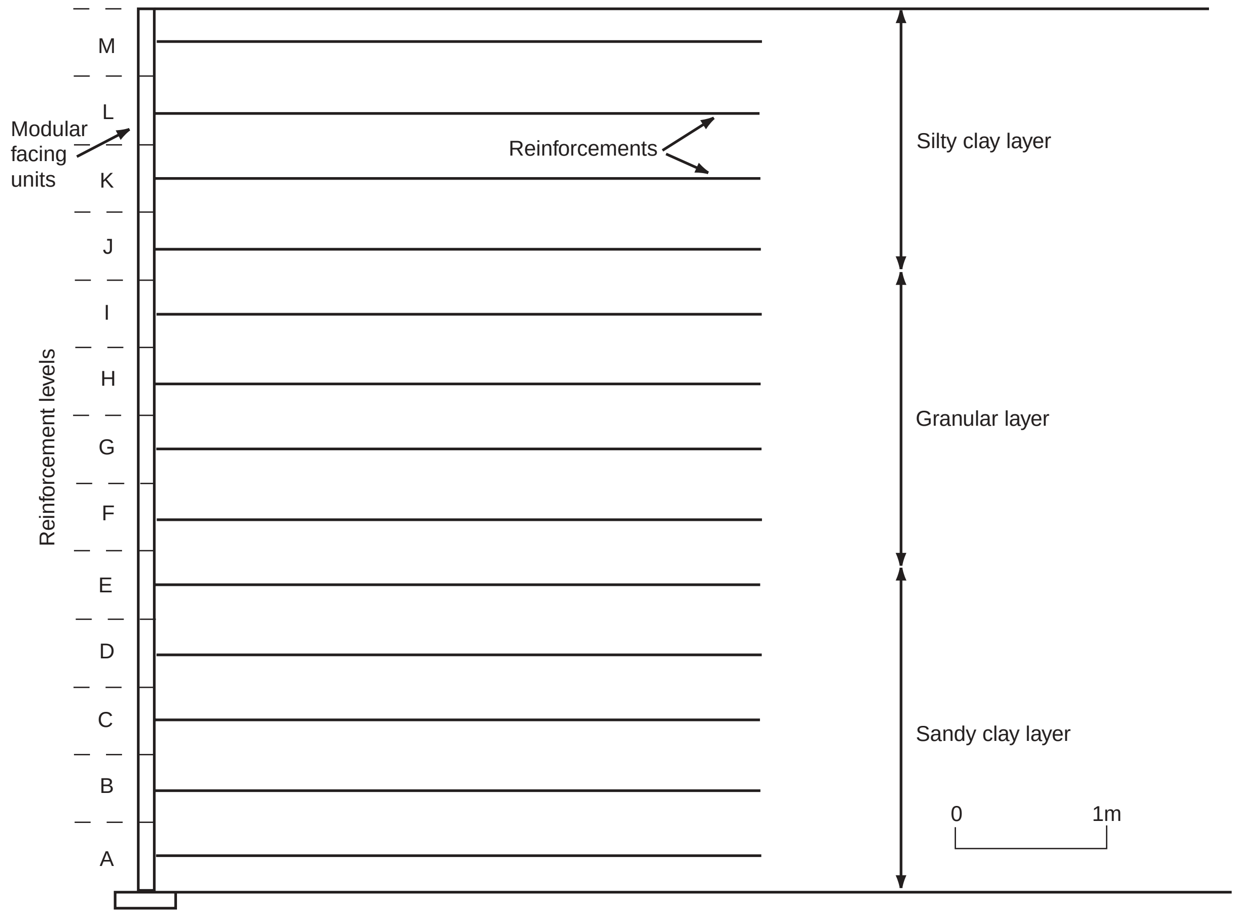


Figure 11 Sectional view showing levels of reinforcements at the experimental reinforced soil structure at the Laboratory

Table 11 Geotechnical parameters of the fill materials used in the experimental reinforced soil structure at the Laboratory

	Plasticity and grading characteristics							
	Plasticity limits						*ϕ_{pk} (degrees)*	
	Liquid limit (per cent)	*Plastic limit (per cent)*	*Grading analysis, % of various fractions*				*From shear -box tests*	*From triaxial tests**
			Clay	*Silt*	*Sand*	*Gravel*		
Lower clay layer	30	17	10	39	51	0	37	28 to 33.5
Middle granular layer	Non plastic		0	5	65	30	~40	
Upper clay layer	42	21	28	65	7	0	37	

* Envelope to data was slightly curved, highest values of ϕ obtained at lowest cell pressures

Table 12 Details of reinforcements used in the experimental soil structure at the Laboratory

Type of reinforcement	*Dimensions of reinforcements width × thickness (mm)*	*Minimum tensile strength (N/mm^2)*
Stainless steel (Grade 316, S16 cold rolled)	68 × 1.5 68 × 1 48 × 1	400
Galvanised mild steel (Grade 43/25, minimum coating weight 610gm/m^2)	65 × 3 60 × 3 50 × 3	200
PVC coated mild steel (Grade 43/25, 0.4mm thick plastic cover)	65 × 3 60 × 3 50 × 3	200
Aluminium coated mild steel (Grade CR4) ‘Aludip’	65 × 2	290
Polyester filaments in polyethylene sheath ‘Paraweb’	88 × 2	170 (short term value)
Glass fibre reinforced plastic (GRP) (Type 96/1, hairpin connection)	80 × 2.5	200 (short term value)
Pre-stressed concrete planks	120 × 55	42kN (load per plank)

4.2.2 Tensile strength tests

Tensile tests were undertaken on 1 to 1.5 m long specimens of Paraweb cut from different parts of a number of the recovered straps: this was done to investigate whether there was any substantial variation in tensile properties with position. The test specimens were clamped using roller grips and the tests were carried out using a rate of strain of 10 per cent per minute. The strains were determined from the gauge length of the specimen and the relative movement of the clamps. In the most recent set of tests strain was also determined using a non-contacting laser extensometer: the strains measured by both methods were in good agreement.

The tensile tests on the GRP specimens were undertaken by Rapra Technology Ltd and followed the procedure used by Pilkington Bros, the manufacturer of the strips, and as reproduced in Appendix A. It is difficult to grip the relatively smooth and brittle GRP test specimens and there is a tendency for the specimens to fail adjacent to the grips: this may lead to the strength of the specimen being underestimated. Because the dimensions of the strips vary and the initial cross-sectional area of the ruptured specimen is not known, strength is not usually reported in stress units.

4.3 Results

4.3.1 Paraweb

The recovered straps were generally in good condition but the polyethylene sheath had deformed around the 25 mm diameter connection bar at the face of the wall, and in some cases the sheath had fractured on the outside radius thereby exposing the polyester fibres. The surface of these fibres had become discoloured over time by the ingress of clay particles.

The results of the tests are provided in Table 13 and the mean relations are plotted in Figure 12 along with that given by the supplier in 1977.

4.3.2 Glass fibre reinforced plastic

The results of tensile tests undertaken on sections of the strips recovered from the structure at the Laboratory are provided in Table 14; also given are the results of QA tests undertaken at the time of their manufacture.

No substantial or consistent differences could be found in the results for strips recovered from the three different backfills, but the data show that the degradation of the end connections was more severe than that suffered by the straight leg sections.

Table 13 Details of tensile tests on Paraweb straps recovered from the experimental reinforced soil structure at the Laboratory

Strap reference	*Distance of specimen from connection point (m)*	*Load at rupture (kN)*	*Strain at rupture (per cent)*
A: Time of burial 2250 days (6.2 years)			
51J	0.6 - 1.8	29.5	12.5
51J	0.6 - 1.8	30.5	12.1
51J	2.7 - 3.9	30.5	12.5
51K	0.7 - 2.0	32.5	13.0
51K	0.7 - 2.0	32.0	12.5
51L	2.5 - 3.5	32.5	12.5
53K	2.0 - 3.0	27.5	10.5
60M	1.5 - 2.75	29.5	11.3
Mean and range of values		30.6	12.1
		(27.5 - 32.5)	(10.5 - 13.0)
B: Time of burial 4330 days (11.9 years)			
56K	0 ± 0.5	28.9	11.8
56K	-	32.0	12.7
56K	-	32.4	13.3
58K	0 ± 0.5	29.4	12.9
58K	-	32.3	13.2
58K	-	31.4	12.4
59L	0 ± 0.5	31.2	13.7
59L	-	32.3	12.7
59L	-	32.4	12.7
Mean and range of values		31.4	12.8
		(28.9 - 32.4)	(11.8 - 13.7)
C: Time of burial 5700 days (15.6 years)			
52J(i)	0 ± 0.5	30.3	12.5
52J(i)	1.0 - 2.0	31.6	11.8
52J(i)	2.3 - 3.3	31.6	12.3
52J(ii)	0 ± 0.5	29.9	11.3
52J(ii)	1.0 - 2.0	31.5	12.9
53J(i)	0 ± 0.5	27.5	10.5
53J(i)	1.0 - 2.0	31.9	12.6
53J(i)	2.3 - 3.3	31.6	12.1
53J(ii)	0 ± 0.5	30.3	12.2
53J(ii)	1.0 - 2.0	32.0	12.3
53J(ii)	2.3 - 3.3	32.0	12.1
Mean and range of values		30.9	12.1
		(27.5 - 32.0)	(10.5 - 12.9)

(i), (ii) *differentiates between the two straps connected to the same facing unit*

\- *data unavailable*

Table 14 Tensile strength of GRP straps removed from the experimental reinforced soil structure at the Laboratory

	Mean tensile strength (in kN and expressed as a percentage of initial strength)							
	Initial		*7 years*		*11 years*		*16 years*	
Specimen type	*kN*	*per cent*	*kN*	*per cent*	*kN*	*per cent*	*kN*	*per cent*
Straight sections taken from legs	87.9+	100	50.4 (20)	57	52.4 (20)	60	55.4 (10)	63
‘Hairpin’ connection	85.0+	100	39.4 (24)	46	30.9 (10)	36	40.7 (10)	48

+ *based on QA results from batches manufactured around the time of those supplied*

(20) number of specimens tested

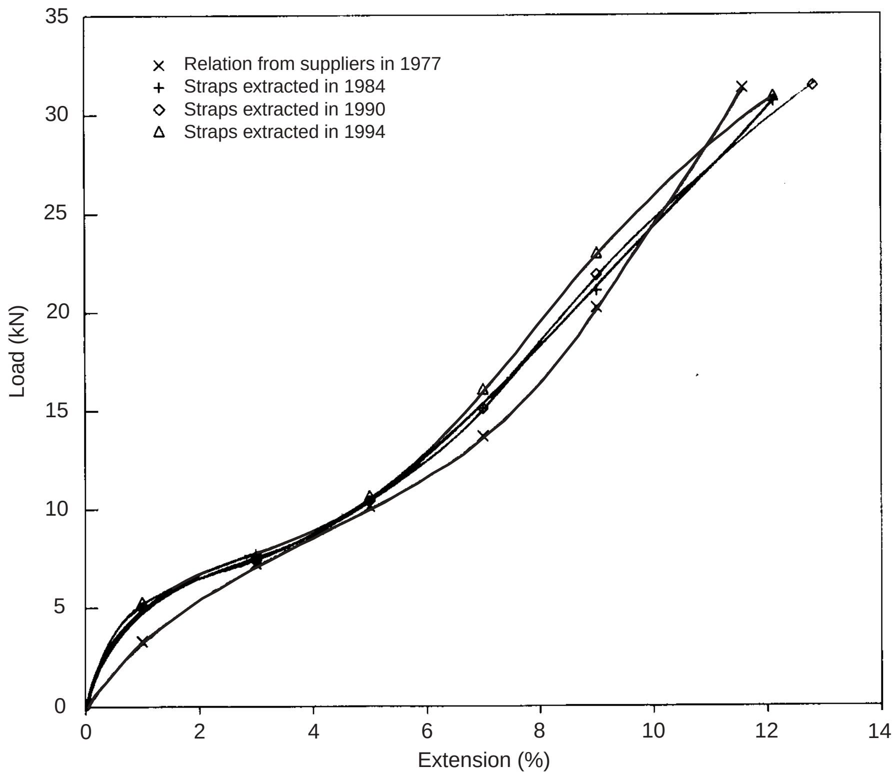


Figure 12 Mean load-extension relations for 'Paraweb'

Figure 13 presents graphically the data from the strips extracted from the experimental structure. Also plotted are the lower bound curves obtained from a series of stress-rupture tests undertaken by Pilkington Bros. Full details of this work and other tests on GRP reinforcements have been reported by Greene and Brady (1994).

4.3.3 Other types of reinforcement

Stainless steel

These reinforcements showed only minor surface staining.

Galvanised mild steel

The zinc coating on the recovered reinforcements appeared to be intact, but some white corrosion products, probably zinc oxides, were apparent. There was no sign of intensive pitting.

PVC coated mild steel

The PVC coating was intact except at the connection detail where it had become cracked and here corrosion of the underlying steel was evident. (This damage to the coating may have occurred during installation). Corrosion of the steel was also noted at a pin hole towards the end of the reinforcement. This Achilles heel was formed in the coating procedure - each strip was dipped in molten PVC and so there was no coating where the strip had been held between grips. The effect of the corrosion on the stability of a reinforced soil structure could be serious at the connection of the strip with the facing, but would be of little consequence at the end remote from the facing.

Aluminium coated mild steel

As with the coupons recovered from the corrosion experiment there was substantial corrosion of these strips particularly along the uncoated edges.

Prestressed concrete planks

These reinforcements were in good condition but there was some corrosion of the steel connection detail.

4.4 Discussion

4.4.1 Paraweb

The mean load extension relations show that there has been no substantial change in the properties of the Paraweb over the first 15 years or so of burial in the silty clay. Differences in the relations up to about 10 kN load, representing a typical maximum design value, are negligible. At higher loads the relations seem to show a stiffer response with increasing time, but this trend needs to be substantiated by further tests.

As shown in Table 13, in a number of tests the load at rupture was marginally lower than the initial minimum short-term breaking load for the reinforcement, i.e. 30 kN. These lower-than-anticipated strengths may be due to damage inflicted during installation or extraction of the specimen, or in service degradation, or from all these sources. Whilst it is not possible to quantify these individual effects, it would appear that the cracking of the polyethylene sheath around the connection bar may have resulted in some small loss in strength; but any degradation of the load carrying polyester fibres must have been small. It should be noted that the properties of the polyethylene sheath have been improved since the production of the material used in the structure. The results of installation damage trials undertaken on specimens of Paraweb manufactured with this new sheathing have shown the material to be resilient to mechanical damage (Watts et al, 1996).

The in service loads carried by the reinforcements were low compared to their original strength, thus the data do not show conclusively that the load-extension properties of Paraweb are unaffected by the load carried. Nonetheless, except perhaps for the tests undertaken on the specimens recovered from around the anchorage point, there is no clear relation between strength and position so the load-extension properties cannot be particularly sensitive to the sustained load.

4.4.2 Glass fibre reinforced plastic

Greene and Brady (1994) concluded that under normal service conditions the long term strength of a GRP reinforcement would be governed more by the sustained load than the ambient temperature. Due to the inherent conservatism in the design of reinforced soil structures, it could be anticipated that the loads carried by the recovered straps would have been low relative to their recommended maximum working loads: as such the data from the straps recovered from the Laboratory structure after 7 and 11 years, plotted in Figure 13, are uncomfortably close to the lower curve obtained from the stress-rupture tests, particularly those from the end connections. However the data do not show any deterioration between 7 and 16 years burial thus the poorer than anticipated performance could be partly due to damage inflicted by the installation and excavation processes and any long term out-of-plane bending stress due to differential movements between the facing units and the backfill. As the recommended maximum design load was derived by applying a factor of 2.5 to the lower-bound curve from the stress-rupture tests (as shown in Figure 13) there should be a reasonable margin of safety against rupture of the GRP reinforcements remaining in the structure.

4.5 Future work

There are numerous metallic and non-metallic reinforcements remaining within the structure and their removal would be a straightforward task. Reinforcements of each material type could be excavated, cleaned, visually inspected and photographed for the record. Sections of the Paraweb and GRP strips could also be tested for tensile strength. (Although GRP is not currently used in reinforced soil applications it is used in other underground applications such as pipelines and buried tanks).

The thickness of the residual zinc coating on the GMS strips could be measured using, for example, an ‘Elcometer’ magnetic thickness gauge. Although the initial coating thickness is not known, these measurements would give an indication of the uniformity of corrosion that has occurred. Tensile tests could also be undertaken - the initial strength of the strips being estimated from tests on specially prepared specimens where the cross-section had been reduced to remove all traces of galvanising and corrosion. Alternatively the tensile strength of the recovered strips could simply be recorded and act as a reference to strips excavated at later dates.

5 Reinforced soil wall at Whitley Bridge

5.1 Details of the site

A reinforced soil wall was constructed at the M62/A19 interchange near Whitley Bridge in Yorkshire in 1974. It

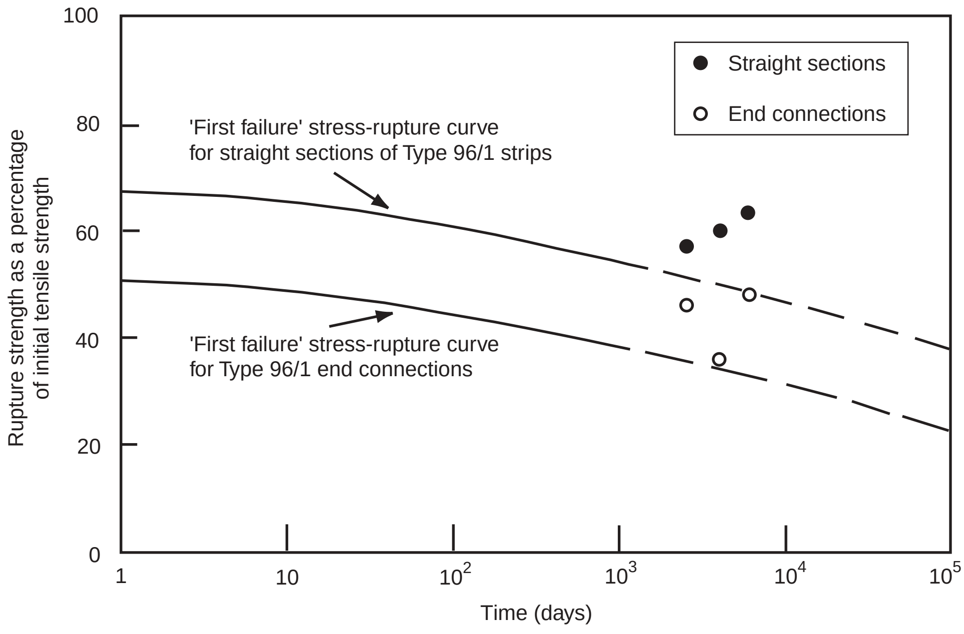


Figure 13 Results of tensile tests on sections of GRP reinforcements recovered from the experimental reinforced soil structure at the Laboratory

was one of the first reinforced soil structures to be built on the highway network in the UK. The wall was about 100 m long, had a maximum height of 6 m and it supported a slip road to the motorway; a view of the completed structure is shown in Figure 14.

Figure 14 View of the completed reinforced soil wall at Whitley Bridge

5.2 Details of the reinforced soil wall

Details of the design and construction of the wall have been provided by Barton et al (1976); for clarity and completeness the following summarises that information.

The structure was built using the 'York' technique (Jones, 1978), the main features of which are shown in Figure 15. In this system the reinforcing strips and interlocking hexagonal facing units were threaded over vertical poles. The facing units were not intended to sustain lateral earth pressures but only to provide protection against weathering. A gasket was placed between the facing units to accommodate any irregularities in fit and also to prevent seepage of water through the joints.

The vertical poles comprised 16 mm diameter mild steel bars grouted into 35 mm internal diameter polyvinyl chloride (PVC) tubes. The facing units were made from glass fibre reinforced cement (GRC) and, as shown in Figure 15, consisted of hexagon based pyramids, 225 mm deep and measuring 600 mm across the flats.

The galvanised mild steel (GMS) reinforcements were 5 m long and had a 75 x 3 mm cross-section. A 20 mm diameter hole was provided 40 mm from one end so that the strip could be threaded over the vertical poles. For design purposes the strength of the steel was taken to be

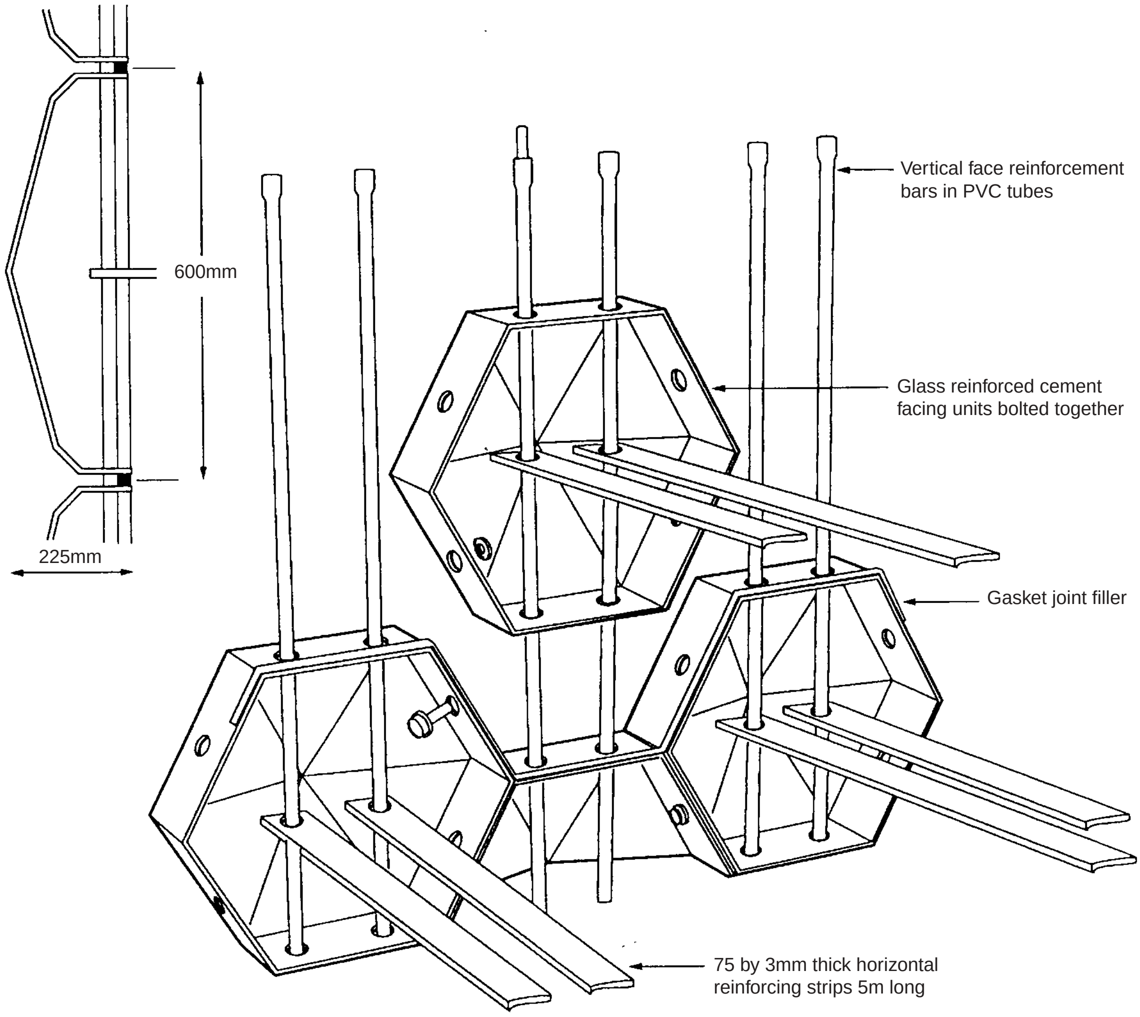


Figure 15 Components of the York system

250 N/mm²; thus the design strength of the cross-section at the connection point was about 41 kN. A sacrificial thickness of steel of 0.25 mm per face was provided in addition to the galvanising. The minimum specified weight, the actual coating weight on the strips, and the strength of the strips have not been reported in any of the references that have come to hand.

Two types of glass fibre reinforced plastic (GRP) strip were also installed in a 3.5 m long test section in the structure; details of these strips have been provided by Greene and Brady (1994). Most of the strips used were of the Mark I type which were provided with a holed stainless steel end plate; the Mark II type had an integral looped end formed from the GRP fibres. Both types of strip were 5 m long, 75 mm wide and 2.5 mm thick. The short term tensile strengths of the Mark I and Mark II strips were 136 kN and 176 kN respectively (Greene and Brady, 1994).

The backfill to the structure was a weakly cemented Bunter Sandstone that occurred across the site: it was also used as a general fill for the works. The particle size distribution of the fill is shown in Figure 16 and data from chemical analyses of the backfill are provided in Table 15. The peak angle of friction of the sand, at a dry density close to its optimum, was reported by Barton et al (1976) to be 32°.

Table 15 Chemical analysis of the Bunter Sand used in the reinforced soil wall at Whitley Bridge

Mineral/Property	*Soil composition (per cent)*
SiO_2	90.4
Al_2O_3	4.7
Fe_2O_3	1.1
TiO_2	Trace
CaO	Trace
MgO	0.04
Na_2O	0.01
K_2O	0.25
Loss on ignition	1.2
Moisture content	2.2
pH value	7.0 - 7.2

5.2.1 Construction

The construction of the wall commenced with the casting of a small no fines concrete footing into which the vertical poles and the first row of facing units were cast: details of this arrangement are shown in Figure 17. The top of the wall was finished off with half units and in situ concrete to the arrangement shown in Figure 18.

Initially the strips were installed towards the bottom of the facing units but this was later rectified so that they lay at

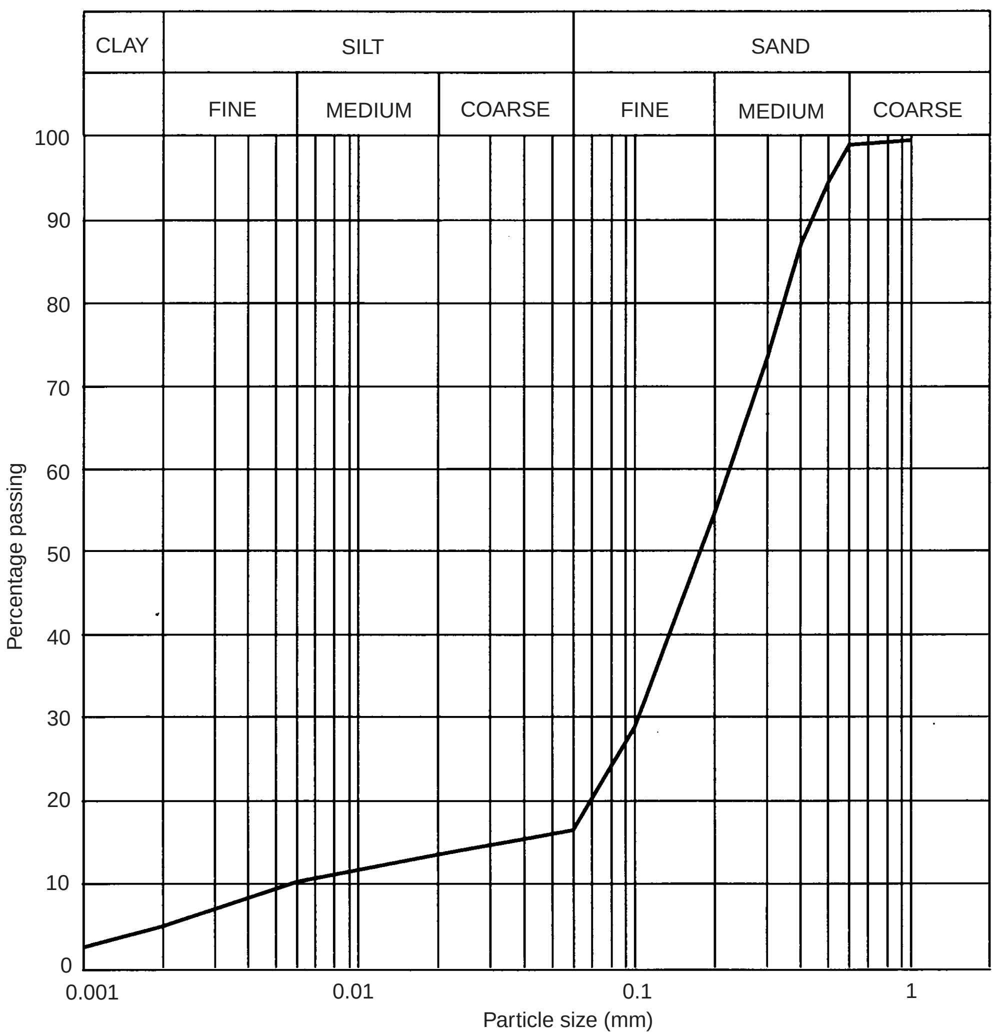
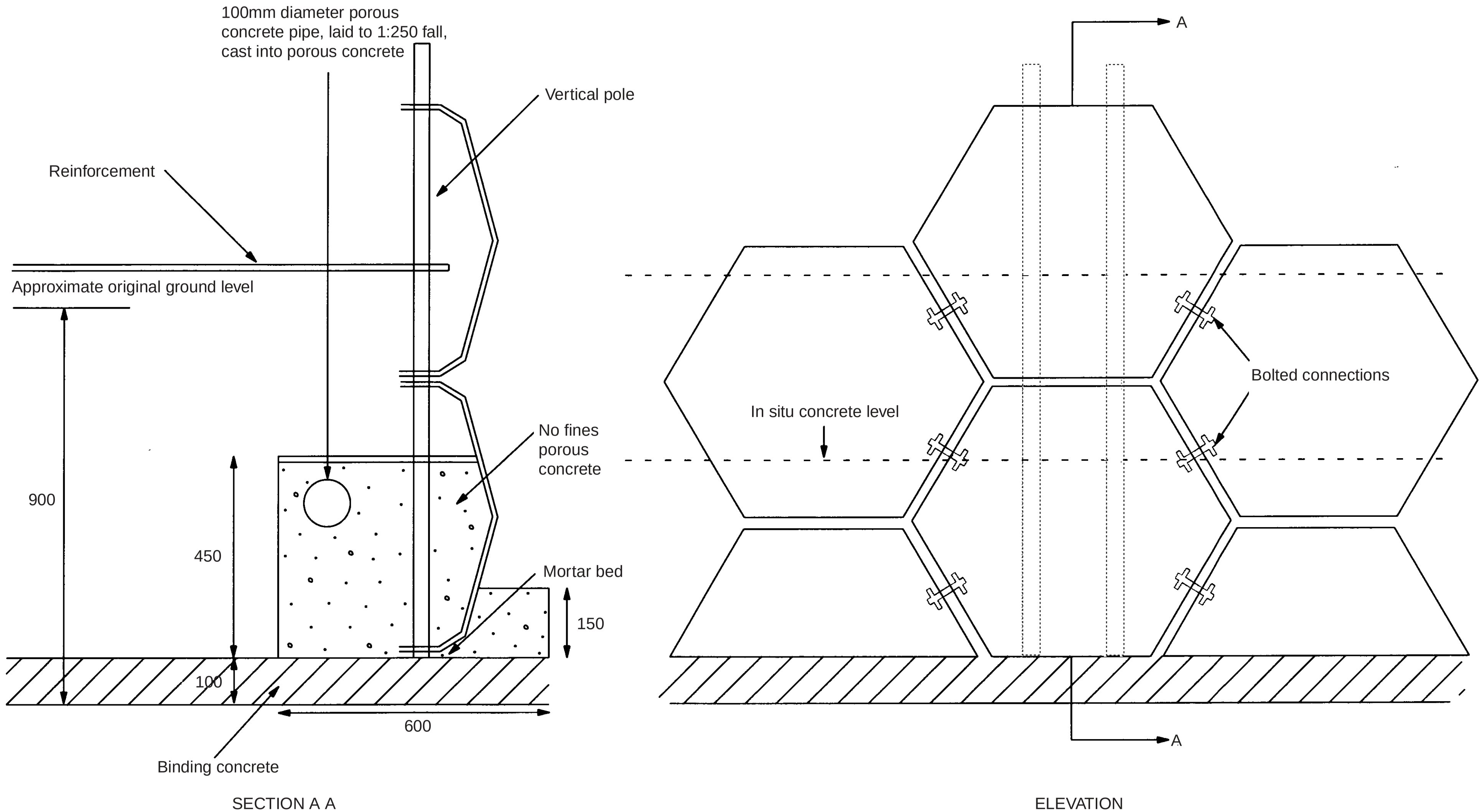


Figure 16 Particle size distribution curve for the Bunter Sand at the Whitley Bridge site



All dimensions in millmetres (not to scale)

Figure 17 Details of construction arrangement at the foundation of the wall at Whitley Bridge

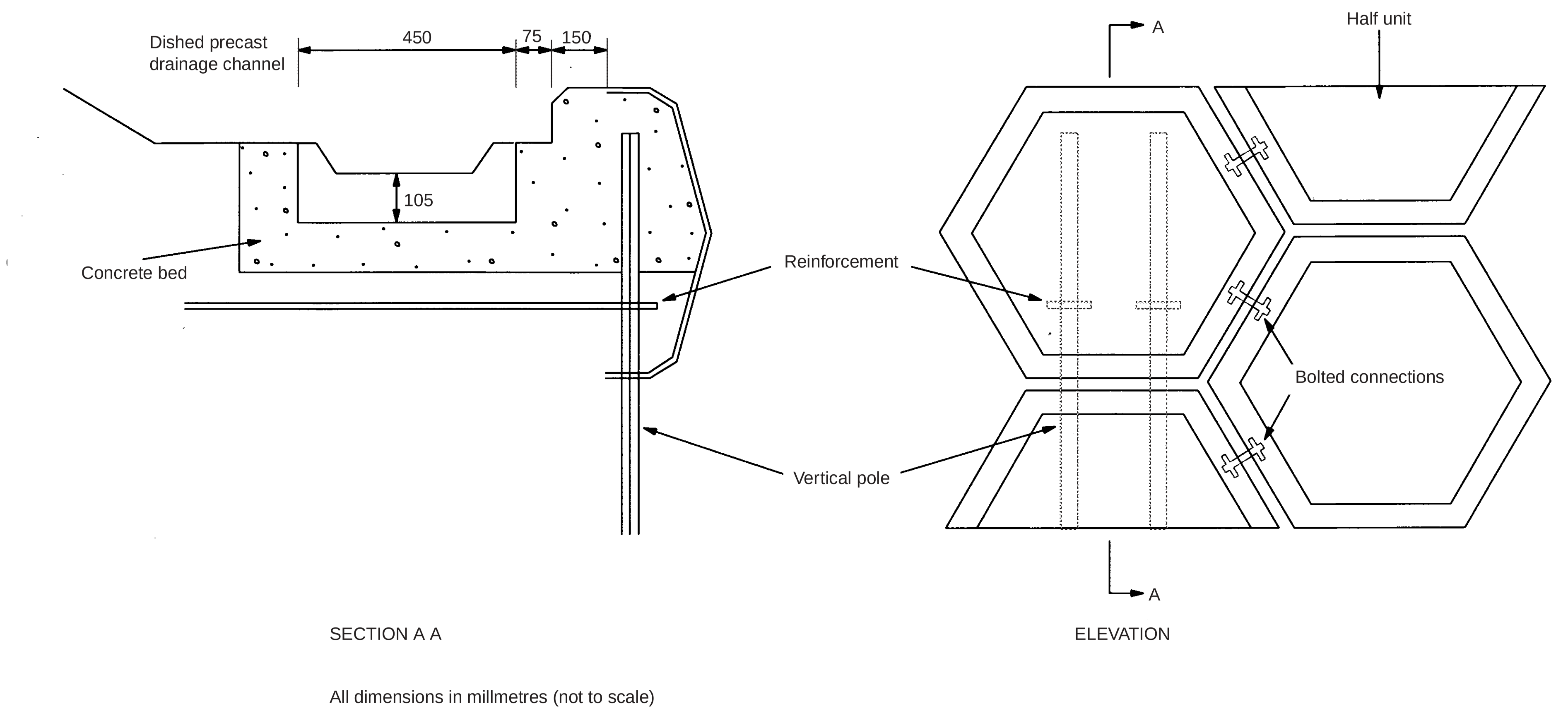


Figure 18 Details of the top of the reinforced soil wall at Whitley Bridge

about the mid-height. For ease of construction the vertical poles were installed in two metre lengths. The PVC tubes were extended using a spigot and socket connection and the mild steel bars were extended by abutting adjacent lengths - the joints being staggered along the wall.

An advantage of this system was that none of the light-weight components required on-site cranage. However heavy compaction plant could not operate within a metre of the light-weight facing units as this would have led to localised bulging of the wall face. Shortly after construction commenced the GRC units began to exhibit shrinkage cracks and a number of the more heavily cracked units had to be replaced. The problem of short-term cracking was overcome by increasing the proportion of glass in the mix and by introducing sand into the cement mortar but by this time the wall was largely complete.

The structure was completed in July 1974, the road pavement being placed during August and the highway opened to traffic in September 1974.

5.3 Details of investigation

By the late 1980s the condition of the GRC facing units had deteriorated considerably. Most of the units showed extensive cracking while others had been vandalised or were missing altogether. A technique of clipping new units onto the vertical poles had been devised during construction, but in 1991 the Yorkshire and Humberside Regional Office of the DOT decided that the best means of ensuring structural integrity was to place a berm of fill in front of the wall.

The construction of the berm rendered the wall redundant and the opportunity was taken to recover some of the reinforcements for assessment. In addition, CAPCIS Ltd were contracted by the Laboratory to undertake electrochemical tests to assess the in situ rate of corrosion of the GMS strips. Because the construction of the berm would restrict access to the reinforcements, a monitoring system was installed to measure the long term rate of in situ corrosion.

Figure 19 shows a schematic plan of the site identifying the areas from which the strips were excavated and where electrochemical tests were undertaken.

5.3.1 Recovery of strips

In August 1991 a 5 m deep, 2.5 m wide trench was excavated over the full height of the wall. As shown in Figure 19 the centreline of the trench was located on the intersection of the GRP and GMS strips so that both types could be recovered. A total of 36 GMS and 34 GRP strips were removed from the excavation.

The trench was supported by steel sheet piles and an hydraulic shoring system. An excavator was used to remove the bulk of the soil from above the strips but the final few centimetres were removed by hand: all but a few of the strips were recovered without damage.

5.3.2 Laboratory tests on recovered strips

The recovered strips were visually inspected and the thickness of the residual zinc coating on the GMS strips was measured using an ‘Elcometer’ magnetic thickness gauge. The tensile strength of sections of a number of both the GMS and GRP strips was also determined.

Nine of the 36 recovered GMS strips were closely examined to assess the distribution and intensity of corrosion. Specimens, approximately 20 mm square, were cut from apparently uncorroded and more highly corroded areas of these nine strips. The cut surfaces were polished and mounted in black powder; highly magnified photographs (micrographs) were then taken so that the condition of the residual galvanising could be assessed.

The end of each GMS test strip was cut off 300 mm from the centre of the connection hole and 300 mm long specimens were cut from the front, middle and back of the remaining length of strip. The tensile strengths of 20 such specimens were measured using a Dennison T.42.C.2 loading frame housed at Middlesex University. The rate of strain applied in these tests was 10 per cent per minute.

To estimate the initial strength of the reinforcements, tests were also undertaken on five specially prepared specimens. These were formed by reducing the cross-section of a recovered specimen, by successively finer abrasion, to remove all traces of galvanising and corrosion. The cross-sectional area of these polished specimens was measured prior to tensile testing, which was undertaken using the same equipment and rate of strain as above.

Tensile tests were undertaken on both the end connection and straight sections of some of the recovered GRP strips. The tests followed the procedure used by Pilkington Bros and reported earlier and were again carried out by Rapra Technology Ltd acting under contract to the Laboratory.

5.3.3 On site corrosion tests

The ends of many of the reinforcements were exposed where the GRC facing units were damaged or missing. To the west of the pedestrian underpass, the ends of twelve GMS strips were exposed in close proximity to one another: their locations are shown in Figures 19 and 20. In August 1991 a series of electrochemical tests were undertaken on these exposed strips and also on the surrounding fill. These included measurements of soil resistivity, redox potential, pH, electrochemical potential, LPR, electrochemical impedance and electrochemical noise: see Section 2.

5.3.4 Long term corrosion monitoring system

To monitor the long term rate of corrosion it was necessary to measure the potential between a reinforcement and a reference electrode, or between reinforcements. A monitoring system was installed at the end of August 1991 to monitor six pairs of reinforcements. Three facing units were removed from each side of the pedestrian underpass and a reference electrode (Ag/AgCl/KCl) was buried in the backfill between each pair of strips. Wires were run from each of the strips and the electrode, through plastic ducting, to a waterproofed stainless steel junction box placed centrally above the underpass. The locations of the strips are shown in Figure 19. Measurements of corrosion potential, LPR, and electrochemical impedance were made on these strips in February 1992.

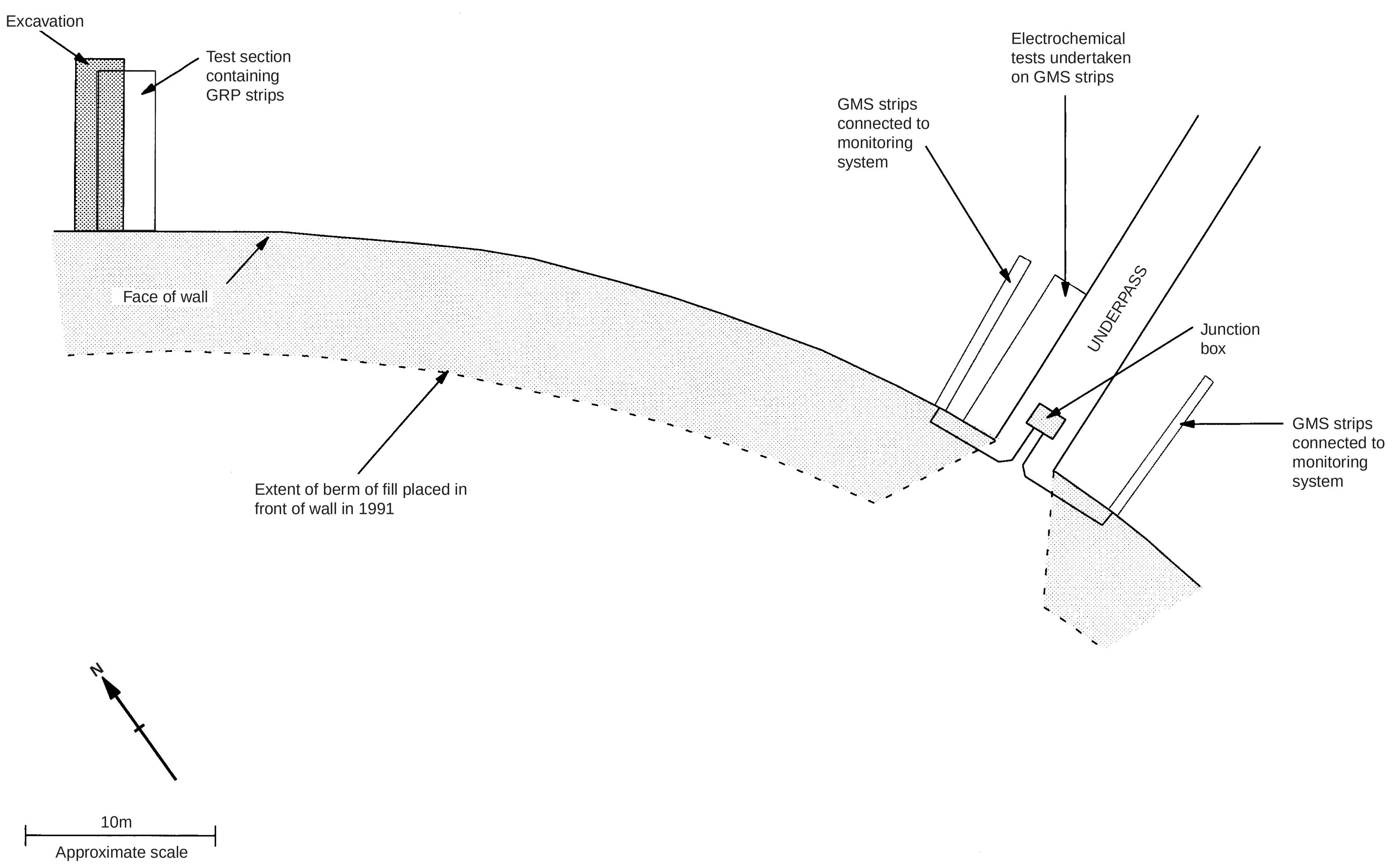


Figure 19 Schematic plan of the site at Whitley Bridge

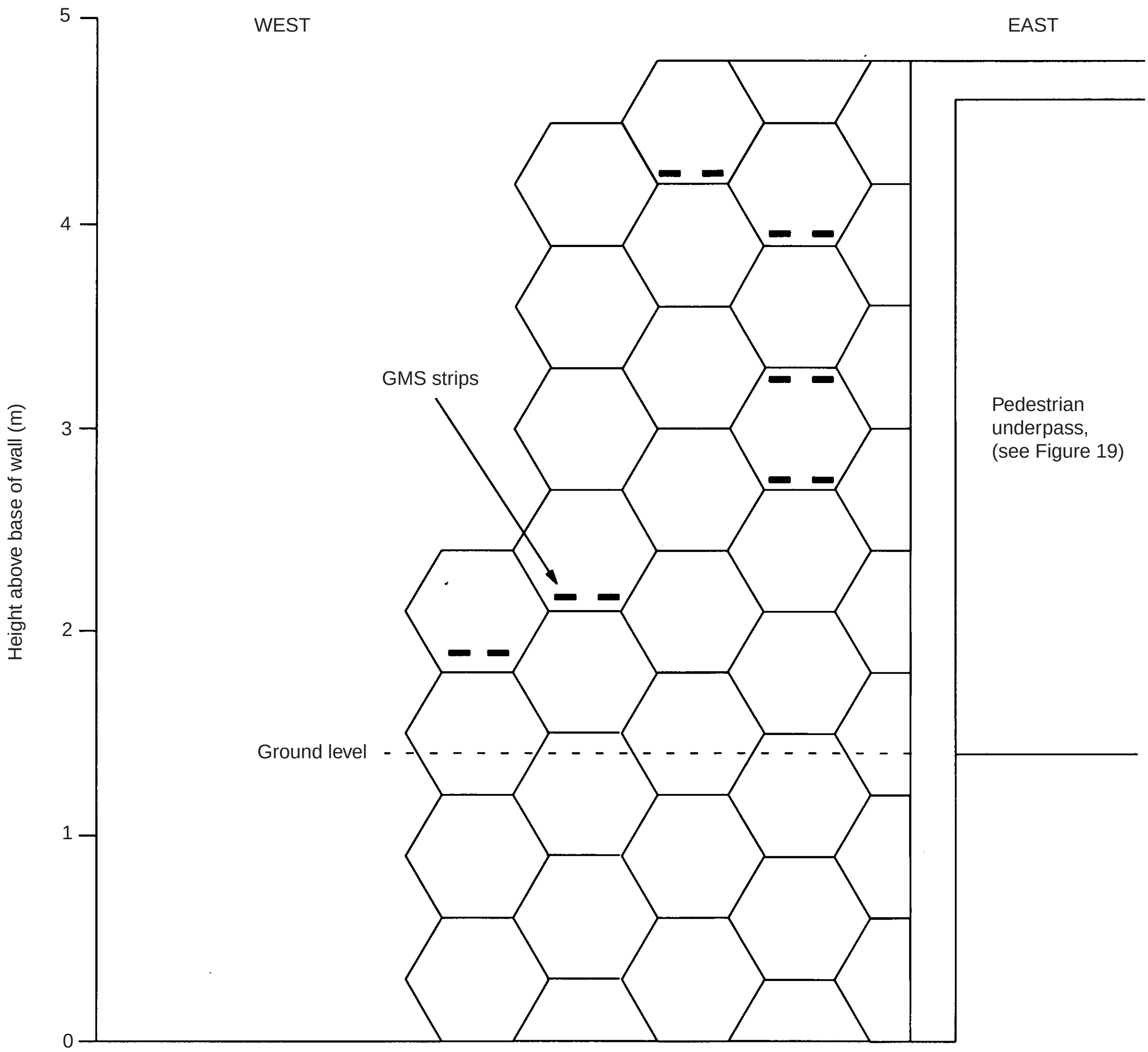


Figure 20 Location of strips used for electrochemical tests at the Whitley Bridge site

5.4 Results

5.4.1 Visual inspection and metallographic tests

The visual examinations showed that the cover of zinc on the strips was largely intact but all of the strips exhibited some loss of galvanising, with those at the bottom of the wall appearing to be slightly more corroded. However there was no obvious relation between the degree of corrosion and position along the strip. The measured maximum and minimum thicknesses of residual zinc on the test specimens are given in Table 16 and presented graphically in Figure 21. The data show that the zinc layer had been lost in part from all of the strips and also that the maximum residual thickness was greater towards the top of the wall. Rates of corrosion based on the loss of thickness cannot provide a complete picture of the extent of corrosion but they may provide a general basis for comparison.

At the time of the construction of the wall in 1974, it was usual for a nominal coating weight of between 610 gm/m^2 and 765 gm/m^2 of zinc to be specified for most galvanised products for civil engineering purposes (Constructional Steel Research and Development Organisation, 1983). These coating weights are equivalent to zinc thicknesses of about 85 μm and 107 μm respectively. The measured maximum thickness ranged up to 272 μm, but the device used to measure the thickness of the coating did not distinguish between unreacted zinc and corrosion products formed from the reaction of the galvanising. Nonetheless the wide range in the thicknesses indicates that neither the initial thickness of the zinc coating nor the intensity of corrosion was particularly uniform.

Typical micrographs from strips located at the top and bottom of the wall are reproduced in Figures 22 and 23 respectively. For the upper strip the micrograph shows that there are small patches where the zinc had been partially removed, while on the lower one there are areas where it had been lost altogether and some corrosion of the steel had occurred. The original thickness of the galvanising is not known and so the rate of loss at a particular cross section cannot be determined particularly well. However

Table 16 Thickness of residual coating on GMS strips, recovered from the reinforced soil wall at Whitley Bridge, measured with 'Elcometer' magnetic thickness gauge

Height above base of wall (m)	Strip designation	Residual thickness of galvanising (μm)	
		Maximum	Minimum
5.7	A (top)	213	0
	A (bottom)	173	0
5.1	B (top)	173	0
	B (bottom)	124	0
4.5	C (top)	203	0
	C (bottom)	272	0
3.9	D (top)	89	0
	D (bottom)	94	0
3.3	E (top)	59	0
	E (bottom)	94	0
2.7	F (top)	173	0
	F (bottom)	99	0
2.1	G (top)	148	0
	G (bottom)	124	0
1.5	H (top)	84	0
	H (bottom)	99	0
0.9	I (top)	158	0
	I (bottom)	40	0

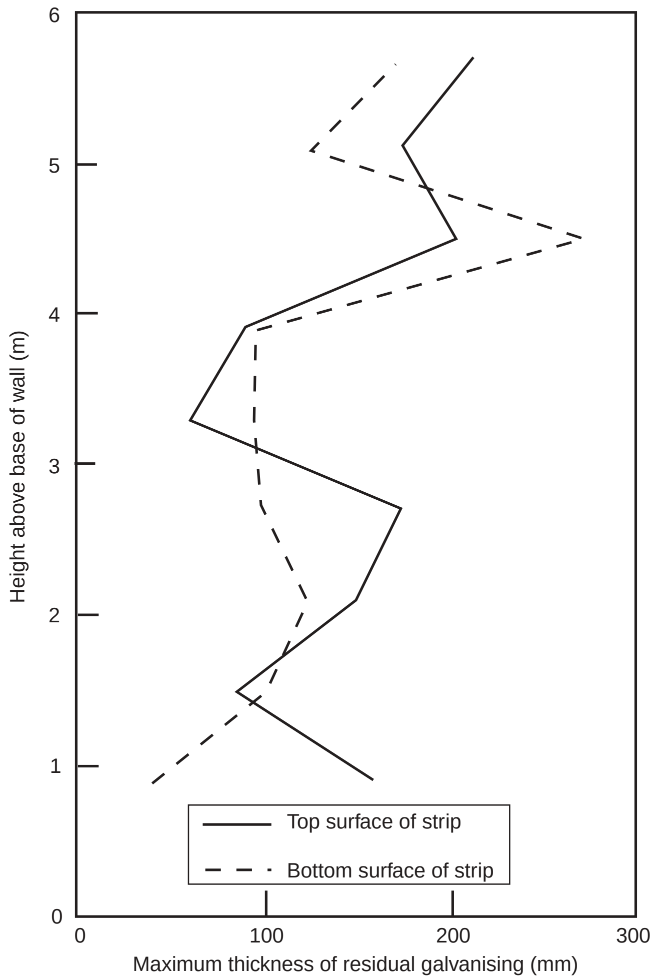


Figure 21 Variation in maximum thickness of galvanising with height above base of wall at the Whitley Bridge site

the thickness of the residual zinc layer shown in Figures 22 and 23 ranged from 35 μm to 50 μm and so if these thicknesses had originally been applied over the full area of the strips, the corrosion rates at the points where the coating had been perforated would have been 1.9 μm per year and 2.8 μm per year respectively. The micrographs were taken from apparently uncorroded areas and the more corroded parts of the strips and so the rates are not representative of the entire strip.

5.4.2 Tensile strength tests

Data from tensile tests undertaken on the untreated and the specially prepared specimens are given in Table 17. The data show that there was a large variation in the strength of the strips. Indeed it appears that the strips recovered from layers D, E, H and I were formed from a steel having an ultimate tensile strength of about 350 N/mm^2 whereas those in layers C and G were formed from one having an ultimate tensile strength of about 470 N/mm^2. These correspond more or less to Grade 43 and Grade 50 steels respectively (Constructional Steel Research and Development Organisation, 1983). However the results of tests on the strips recovered from layers B and F do not fit either grade particularly well.

Due to this variation in the strength of the strips, corrosion rates could only be determined for those specimens taken from a strip where a specially prepared specimen was available. The rate was calculated by assuming the percentage strength loss could be attributed to a uniform pro rata loss of metal from each face over the time of burial. These rates, given in Table 18, ranged up to about 8 μm per face per year with an overall mean value of about 5 μm per face per year. They are likely to represent maximum rather than mean values since the specimens would have ruptured at the minimum cross-section. Furthermore the calculated strength loss of an untreated specimen will be overestimated because the measured cross-section will include the zinc layer, which has a lower tensile strength than steel.

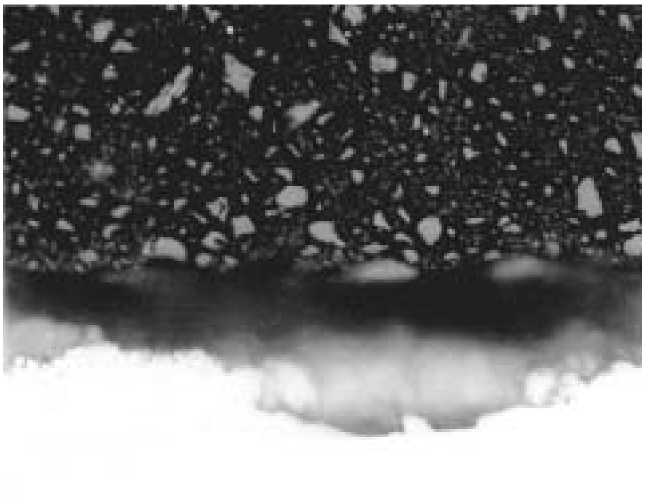
5.4.3 In situ corrosion measurements

Data from the electrical resistivity, pH and redox potential tests are given in Table 19. Also reproduced, for comparison, are some of the criteria given in the SHW (MCHW 1) for backfills to reinforced soil. The data suggest that the backfill would be deemed to be non-aggressive according to current specifications.

The results of the corrosion potential tests are provided in Table 20. The potentials are consistent with that for zinc, indicating that the galvanised layer was reasonably competent. The potential of bare steel is usually higher than -600 mV with respect to a copper/copper sulphate electrode. The corrosion potential did not vary with the height of the strips above the base of the wall.

The rates of corrosion calculated from the LPR

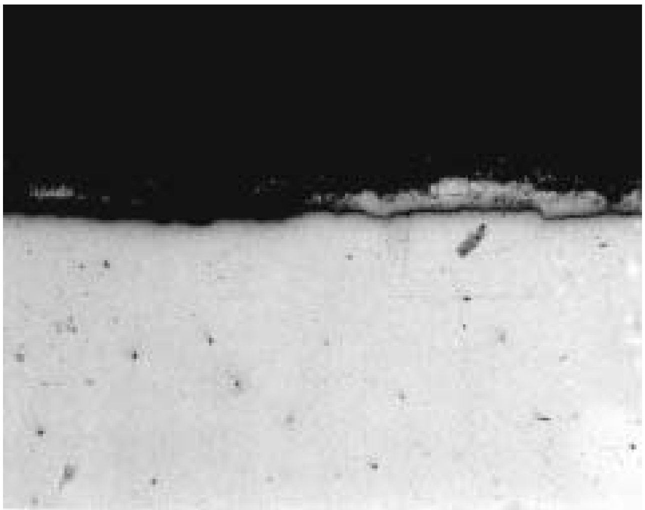
a Micrograph showing partial loss of galvanising (x100)



b Micrograph showing partial loss of galvanising and the presence of corrosion products (x200)

Figure 22 Micrographs from strip recovered from the top of the wall (layer A) at Whitley Bridge

a Micrograph showing loss of galvanising (x20)



b Micrograph showing loss of galvanising and corrosion of underlying steel (x100)

Figure 23 Micrographs from strip recovered from the bottom of the wall (layer I) at Whitley Bridge

Table 17 Details of tensile tests on specimens taken from GMS strips recovered from the reinforced soil wall at Whitley Bridge

a Untreated specimens

Specimen	*Maximum load (kN)*	*Strain at maximum load (per cent)*	*Ultimate tensile strength (N/mm²)*
B 1	97.5	32	358
B 9	106	33	387
C 1	129	26	463
C 5	130	26	465
C 9	129	29	464
D 1	82	30	327
D 9	82	33	329
E 1	86	33	331
E 6	87	33	335
E 9	88	35	332
F 1	103	32	398
F 9	102	31	419
G 1	131	27	475
G 5	132	30	476
G 9	129	25	485
H 1	93	33	349
H 9	92	30	343
I 1*	82	33	338
I 5	82	29	333
I 8	80	25	344

* *Failed near grips*

b Specially prepared specimens

Specimen	*Maximum load (kN)*	*Strain at maximum load (per cent)*	*Ultimate tensile strength (N/mm²)*
D 5	21.5	32	357
F 4	27.4	30	427
F 5	27.1	28	424
H 5	23.6	31	349
I 4	22.9	32	366

Table 18 Corrosion rates determined from tensile tests on GMS strips recovered from the reinforced earth wall at Whitley Bridge

Specially prepared specimens		*Untreated specimens*			
Strip	*Strength (N/mm²)*	*Strip*	*Strength (N/mm²)*	*Difference in strength (per cent)*	*Estimated corrosion rate (µm/face/year)*
D 5	357	D 1	327	8.4	7.7
		D 9	329	7.8	7.0
H 5	349	H 1	349	0	0
		H 9	343	1.7	1.7
I 4	366	I 1	338	7.7	6.8
		I 5	333	9.0	8.1
		I 8	344	6.0	5.5
F 4	Mean =	F 1	398	6.5	6.1
F 5	425.5	F 9	419	1.5	1.3

Table 19 Electrochemical properties of the backfill at Whitley Bridge and criteria for backfills to reinforced soil structures

a Results of electochemical tests on backfill at Whitley Bridge site

		Properties of the fill		
Location	*Height above base of wall (m)*	*Resistivity (ohm-cm)*	*Redox potential (mV)*	*pH value*
Between pair 2	4.0	33600		
Between pair 3	3.2	19900		7.6
Between pair 4	2.8	16600	588	7.6
Between pair 5	2.2	13200	674	

b Criteria for backfills for use with hot-dipped galvanised steel components in reinforced soil structures, reproduced from SHW (MCHW 1)

Max chloride ion content (per cent)	*Max organic content (per cent)*	*Max water soluble sulphate content (gm/litre)*	*Minimum resistivity (ohm-cm)*	*Minimum redox potential (mV)*	*pH value Min*	*pH value Max*
0.025	0.2	0.25	5000	430	6	9

Table 20 Corrosion potential measurements taken at the reinforced soil wall at Whitley Bridge

		Corrosion potential relative to $Cu/CuSO_4$ reference electrode (mV)	
Pair of strips	Height above base of wall (m)	West strip	East strip
1	4.3	-	-
2	4.0	-898	-890
3	3.2	-931	-935
4	2.8	-927	-894
5	2.2	-882	-941
6	1.9	-	-899

measurements are presented in Table 21a. The rates varied from 0.25 μm per year to 0.5 μm per year but there was no clear relation between the rate and the location of the strips. The rates of corrosion determined from the measurements of electrochemical impedance, presented in Table 21b, varied from 0.52 μm per year to 1.12 μm per year. The measurements of film resistance (R_f) at the strip/soil interface indicate the presence of zinc corrosion products.

The results of the electrochemical noise tests indicated a low rate of reasonably uniform corrosion.

Table 21 Corrosion rates determined from electrochemical tests undertaken at the reinforced soil wall at Whitley Bridge

a Linear polarisation measurements

Height above base of wall (m)	*Strip reference*	*Rate of corrosion (μm/face/year)* West strip	*East strip*	*Mean*
4.0	Pair 2	0.40 - 0.42	0.45 - 0.50	0.44
3.2	Pair 3	0.43 - 0.47	0.45 - 0.49	0.46
2.8	Pair 4	0.33 - 0.37	0.39 - 0.43	0.38
2.2	Pair 5	-	0.33 - 0.35	0.34
1.9	Pair 6	-	0.25 - 0.28	0.27

b Electrochemical impedance tests

Height above base of wall (m)	*Strip reference*	R_s *(kΩcm²)*	R_f *(kΩcm²)*	R_{ct} *(kΩcm²)*	*Corro -sion rate (μm/face/ year)*
4.0	Pair 2	64.5	184.4	581.0	0.98
3.2	Pair 3	92.2	73.8	507.2	1.12
2.8	Pair 4	784.0	157.0	737.8	0.77
2.2	Pair 5	69.8	350.4	1106.7	0.52

5.4.4 Long term monitoring system

The measured potentials of the 6 pairs of strips installed further from the underpass are given in Table 22. The potentials of the strips in the upper and middle levels are indicative of the presence of a reasonably competent layer of zinc. The strips at the bottom of the wall had higher potentials consistent with a zinc/steel couple.

The rates of corrosion determined from linear polarisation resistance measurements and electrochemical impedance tests on these strips are given in Table 23. The rate ranged from 0.2 μm per year to 1.02 μm per year for the LPR measurements and from 0.4 μm per year to 0.76 μm per year for the impedance measurements. The data indicate that the rate of corrosion was higher towards the bottom of the wall; the rates are not grossly different to those derived from the tests on exposed strips installed adjacent to the underpass.

5.4.5 Tensile strength tests on glass fibre reinforced plastic strips

All the tensile tests were undertaken on the Mark I type of strip. The results for both straight sections and the end connections are given in Table 24.

Table 22 Potentials obtained from the long term corrosion monitoring system installed at the reinforced soil wall at Whitley Bridge

a West side of underpass

Height above base of wall (m)	*Corrosion potential relative to Ag/AgCl/KCl reference electrode (mV)* West strip	*East strip*
3.5	-889	-903
2.3	-948	-928
1.0	-669	-687

b East side of underpass

Height above base of wall (m)	*Corrosion potential relative to Ag/AgCl/KCl reference electrode (mV)* West strip	*East strip*
2.8	-827	-860
1.0	-895	-898
0.6	-630	-713

Table 23 Corrosion rates obtained from the long term corrosion monitoring system installed at the reinforced soil wall at Whitley Bridge

a Linear polarisation measurements (west of underpass)

Height above base of wall (m)	*Rate of corrosion (μm/face/year)* West strip	*East strip*	*Mean*
3.5	0.30 - 0.33	0.27 - 0.29	0.30
2.3	0.26 - 0.28	0.25 - 0.27	0.27
1.0	0.35 - 0.40	0.39 - 0.45	0.40

b Linear polarisation measurements (east of underpass)

Height above base of wall (m)	*Rate of corrosion (μm/face/year)* West strip	*East strip*	*Mean*
2.8	0.20 - 0.23	0.22 - 0.25	0.23
1.0	0.23 - 0.25	0.20 - 0.23	0.23
0.6	0.28 - 0.34	0.91 - 1.02	0.64

c Electrochemical impedance tests (west side of underpass)

Height above base of wall (m)	R_s *(kΩcm²)*	R_f *(kΩcm²)*	R_{ct} *(kΩcm²)*	*Corrosion rate (μm/face/year)*
3.5	97.5	323.3	1016.8	0.56
2.3	72.2	-	1423.0	0.40
1.0	35.7	-	913.0	0.62

d Electrochemical impedance tests (east side of underpass)

Height above base of wall (m)	R_s *(kΩcm²)*	R_f *(kΩcm²)*	R_{ct} *(kΩcm²)*	*Corrosion rate (μm/face/year)*
2.8	73.5	362.2	1227.2	0.46
1.0	72.2	371.4	1361.2	0.42
0.6	59.9	55.2	750.7	0.76

Table 24 Results of tensile strength tests undertaken on Mark I strips and end connections recovered from the reinforced soil wall at Whitley Bridge

		Tensile strength after 17 years burial	
Specimen type	*Initial tensile strength (kN)*	*kN*	*Expressed as a percentage of initial strength*
Straight lengths from Mark I straps	136+	105.3 (10)	77
Mark I end connection	-	27.5 (10)	-

+ based on results for ½-width strips
- no data available
(10) number of specimens tested

Table 25 Summary of corrosion rates determined from electrochemical tests undertaken at the reinforced soil wall at Whitley Bridge

		Rate of corrosion (μm/face/year)	
Electrochemical tests		*Range*	*Mean*
On site tests	LPR	0.25 - 0.5	0.4
	Electrochemical impedance	0.52 - 1.12	0.85
Long term monitoring system	LPR	0.2 - 1.02	0.34
	Electrochemical impedance	0.4 - 0.76	0.54

5.5 Discussion

The measured pH, redox potential and resistivity of the backfill are indicative of a non-aggressive soil. The decrease in the resistivity towards the base of the wall was probably associated with an increase in moisture content of the backfill with depth.

The visual and metallographic examinations showed that the galvanising on the strips was largely intact but the zinc layer had been penetrated in places. The loss of the zinc was more pronounced for strips located towards the base of the wall, and here some corrosion of the steel had occurred. Thus the zinc layer did not provide complete galvanic protection to the exposed steel.

The corrosion rates determined from the micrographs ranged from about 1.9 to 2.8 μm per face per year. But the micrographs represent only a very small section of the specimen and so the rates determined from them are not necessarily representative of the strip as a whole.

The variability of the results of the tensile tests highlight the problem of undertaking corrosion assessments on structures that were not constructed specifically for the purpose. Because of the large variation in the original strength of the strips their loss in strength could not be determined particularly well. However the estimates of the maximum rate of corrosion ranged up to about 8 μm per face per year and this is consistent with the rate required to remove the specified minimum coating of zinc and some small thickness of the underlying steel substrate.

The data from the electrochemical tests indicated that the galvanising was in generally good condition but there were areas where the zinc coating was partially lost. The data from the long term monitoring system indicated that the loss of galvanising was more pronounced towards the base of the wall. A summary of the corrosion rates determined from all the electrochemical tests is shown in Table 25. The mean rate was about 0.5 μm per face per year. (The higher rates obtained from the electrochemical impedance tests may have been due to the measurement of mean corrosion rates for a zinc/steel combination where there was perforation of the galvanising.) The results of these tests are a function of the electrochemical reactions on the surface of a strip and therefore provide mean values of the (present) corrosion rate. The relatively low values are probably associated with the formation of a layer of corrosion products.

5.5.1 Pitting index

The investigation showed that corrosion activity was not uniformly distributed. As discussed earlier, the ratio of the corrosion rate at the minimum cross-section and the mean corrosion rate for the strips as a whole provides a measure of the uniformity of corrosion, i.e. a pitting index (K). The rate of loss at the minimum cross-section ranged up to about 8 μm per face per year. The electrochemical tests provide a measure of the mean rate of corrosion but the formation of corrosion products could have reduced the rate of corrosion over the time of burial. Thus the current measured value of 0.5 μm per face per year probably underestimates the mean rate for the 18 year burial period. Given that the rates determined from the micrographs ranged from about 2 to 3 μm per face per year the value of K for this site would be about 4 - in reasonable agreement with the values from the corrosion experiment at the Laboratory.

5.5.2 Lifetime of galvanised coating

The reinforcements at the Whitley Bridge site were provided with a 250 μm thick layer of sacrificial steel in addition to the galvanising. Assuming a mean coating thickness of 100 μm and a corrosion rate of 8 μm per face per year, the lifetime of the sacrificial layers at any particular point would be about 43 years. Bearing in mind that the results of the electrochemical tests suggest that the corrosion rate reduced with time, the actual lifetime could be considerably longer. Although the above calculation is perhaps simplistic in that it assumes a uniform thickness of galvanising and a constant rate of maximum corrosion, it does provide some assurance that the durability of the reinforcements at this site should not have given cause for concern.

5.5.3 Glass fibre reinforced plastic strips

The initial strength of the end connection of the GRP strips extracted from the structure was not determined, but the data could be taken to show that degradation of the end connection was more severe than for the straight sections, i.e. in agreement with the findings obtained from the experimental retaining structure at the Laboratory. The retained strength (in percentage terms) of the straight sections was higher than for the strips recovered from the structure at the Laboratory and so it could be *inferred* that there would have been a reasonable margin of safety against rupture of the GRP reinforcements had the structure remained in service.

5.6 Summary

The examinations of the recovered GMS strips showed that the galvanising was largely intact but there were some small areas where it had been perforated. Corrosion rates determined from micrographs ranged from 2 to 3 μm per face per year. The micrographs also showed that the residual zinc layer did not provide complete galvanic protection to the exposed steel.

Because of the large variation in the initial strength of the strips incorporated in the structure, the strength loss of the recovered strips could not be determined particularly well. However the best estimates of the maximum rate of corrosion ranged up to 8 μm per face per year.

The results of the electrochemical tests indicated that the backfill to the structure would be deemed to be non-aggressive according to current specifications. The data also showed that the current mean rate of corrosion was about 0.5 μm per face per year. This relatively low value may reflect a reduction in the rate of corrosion with time and this may be associated with the formation of a layer of corrosion products.

The evidence indicates that the sacrificial thicknesses of zinc and steel were likely to have been sufficient for the design life of this structure. And the data from the GRP reinforcements suggest that there would have been an adequate margin of safety against rupture.

5.7 Future work

The reinforced soil wall at Whitley Bridge is now redundant and so further reinforcements could be extracted without compromising the integrity of the structure. However the recovery of further reinforcements would be costly because of safety requirements including traffic management works. Also the vertical poles to which the reinforcements are attached will hinder the removal of the strips.

A long term corrosion monitoring system was installed and measurements taken in 1991. It would seem timely to take a further set of in situ measurements in 2001 to determine if there has been a change in the rate of corrosion. A decision on whether to extract a further batch of steel reinforcements could be based on the results of these tests. To extract any useful information from the recovered strips it will be necessary to undertake tensile tests on specially prepared specimens to determine their likely initial strength.

6 Anchored earth wall at Otley

6.1 Introduction

The concept of the TRL ‘Anchored Earth’ system, first described by Murray and Irwin (1981), followed as a logical development of reinforced earth à la Vidal (1969). The use of end-bearing anchors in the former has technical and economic benefits over frictional anchors as used in the latter.

The first anchored earth wall to be constructed on the trunk road network was on a section of the A660 Otley bypass in 1984. Aspects of the design, the construction sequence and the early performance of the wall have been provided by Snowdon et al (1986). Its performance was monitored by pressure cells and load cells and photogrammetry was used to determine movements of the face of the wall. Pre-weighed corrosion coupons were installed near the top of the structure. Electrical connections were also made between a number of pairs of anchors so that the variation in the rate of corrosion with depth could be assessed from electrochemical measurements.

The site data collected in the ten years subsequent to the completion of construction, i.e. from 1984 to 1994 have been reported by Brady et al (1995).

6.2 Details of the structure

In the area of the retaining wall, the bypass follows the course of an abandoned railway line. The wall is located on the south side of the new carriageway at the base of a relatively steep cut through weathered friable mudstone. There is a history of landsliding in the area with rotational slips being common in the upper part of the hillside and shallow solifluction movements in the lower part extending into the railway cutting. A cross-section through the wall is shown in Figure 24.

The wall is 86 m long, has a maximum height of 6 m and supports an access road and bankseat to a footbridge. A view of the structure, taken in October 1998, is shown in Figure 25.

6.2.1 Design

The design was based on the method subsequently given in the 1987 version of BE 3 (DOT, 1978) and the approach was verified by a series of centrifuge models - details of these have been provided by Craig et al (1991). Prior to construction the pullout capacity of the type of anchor used was investigated by a series of full-scale tests undertaken at TRL. The analyses used to derive the design pullout resistance of the anchors were described by Snowdon et al (1986). The design was confirmed by the Geotechnical Consulting Group (GCG) acting as Category III checkers: their checks included the use of finite element analyses the results of which were reported by Jones et al (1985).

6.2.2 Materials and components

Seventy two prestressed concrete units were used to form the structural face: the units were 1194 mm wide and 150 mm thick and ranged in height from 2.3 to 6.7 m. Slots were

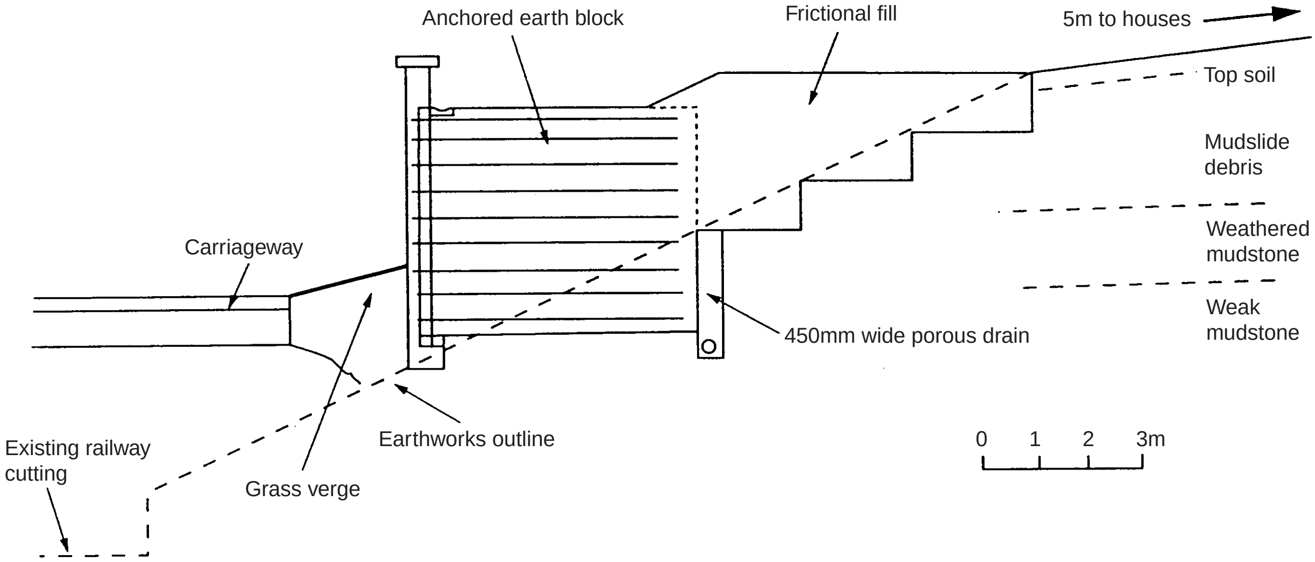


Figure 24 Ground profile section of the site at Otley

Figure 25 View of the anchored earth wall at Otley taken in October 1998

pre-formed into these units for fixing the anchors and also for tying-in the masonry facing.

A total of 588 triangular anchors was used to support the wall. Details of the anchor specification are provided in Figure 26. The welded joint at the anchor head was wrapped with Densopol 60 to provide additional protection against corrosion. The anchors were generally spaced 1.2 m horizontally and 0.5 m vertically, but two additional rows were installed beneath the bankseat to the footbridge.

A well graded crushed limestone aggregate having a maximum particle size of 40 mm was used as backfill to the wall.

6.2.3 Construction

Construction started with the installation of the 2.5 m deep porous (no fines) concrete drain at the back of the anchored earth block, followed by excavation of the existing slope. A concrete footing was then cast in situ to form the foundation to the facing units.

In February 1984, 26 of the 72 facing units were erected in a line at the highest part of the wall. Layers of backfill and anchors were installed up to the top of the units at the end of the wall leaving a ramp into the unbuilt length of wall. The facing units were individually propped from the carriageway using soldiers to give an inward batter of 1 in 40. The units were bedded on a 150 mm thick layer of dry sand and cement mortar mix laid on a 150 mm wide strip of roofing felt. Two coats of a pitch epoxy waterproofing compound were applied to the back of the erected units.

A 150 mm diameter porous pipe was laid at the base of the facing units and 225 by 450 mm drainage pillars, formed from porous blocks, were built up along the line of the vertical joint between adjacent units.

The backfill material was placed in 150 mm thick layers and most areas were compacted with six passes of a Bomag BW 605 twin vibrating roller but a Robin EY20D vibrating plate was used close to the back of the facing units.

Pieces of compressible foam rubber were placed beneath the anchor shafts where they passed through the slots in the facing units. These inserts would allow some differential movement between unit and anchor which was generated by the placement and compaction of subsequent layers of backfill. Galvanised steel washers and nuts were then attached to the protruding anchor shafts and tightened to remove free play from the anchoring system.

The remaining facing units were erected during March and April 1984, and a view of the construction works immediately following their erection is given in Figure 27.

Following completion of backfilling operations, the soldiers supporting the facing units were removed during

Anchor schedule			
No. req'd	Nominal width (mm)	Dim^n 'B' (mm)	Dim^n 'C' (mm)
495	650	4370	830
76	900	4080	1120
2	900	5080	1120
2	900	4580	1120
1	900	5580	1120
8	1200	3760	1440
2	1200	5260	1440
1	1200	4760	1440
1	1200	4260	1440

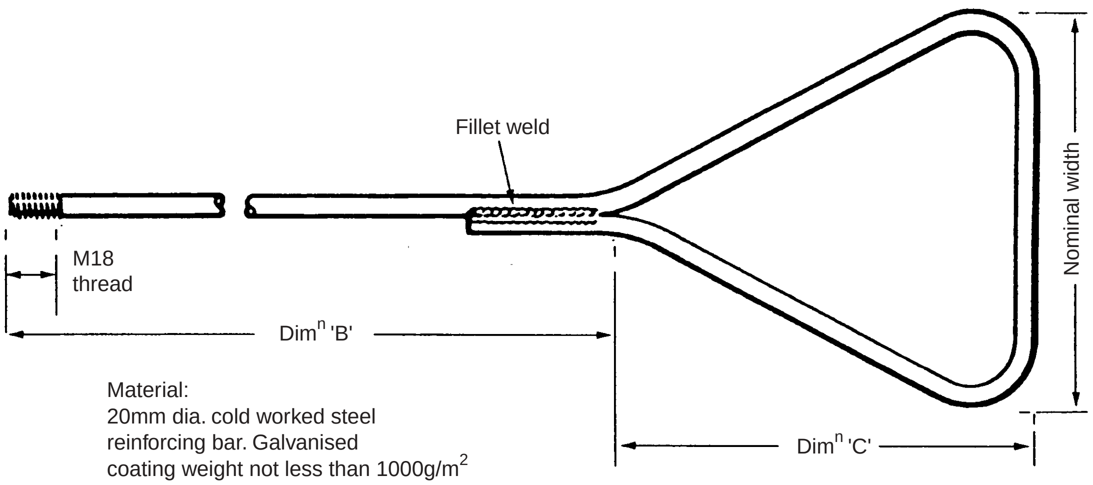


Figure 26 Anchor specification

Figure 27 View of the construction works at the Otley site

May and June 1984. The bankseat for the footbridge was then cast in situ on top of the backfill. A rubber bearing pad was glued with an epoxy adhesive to the topside of the cast slab. The 29 tonne pre-cast concrete footbridge deck was installed on 5 July 1984.

Compressible caps were placed over the nuts and washers to the anchors prior to the construction of the masonry block facing to the wall. The masonry blocks were tied to the facing units with dovetails installed in the cast-in slots. The gap between the masonry facing and the structural facing was filled with well-rammed cement mortar. The masonry blockwork was continued above the top of the facing units to provide a 300 mm thick, 1 metre high parapet wall.

6.2.4 Corrosion monitoring

A selection of ungalvanised and galvanised straight lengths and welded sections of reinforcing bar was buried in the backfill at the same level as the top row of anchors. Forty five, one metre long, weighed coupons were installed: their positions are shown in Figure 28. To provide an indication of the likely change in the rate of corrosion with the depth of cover, pairs of anchors were electrically connected to enable electrochemical measurements to be made. The eight pairs of monitored anchors are identified on Figure 29.

6.3 Corrosion measurements

6.3.1 Data from corrosion coupons

Figure 30 shows the appearance of the set of coupons following their recovery from site in May 1994. As shown by this figure, the coupons were partly coated with soil particles: these and any loose corrosion products were removed by scrubbing with a wire brush. The problem of cleaning corrosion coupons and the uncertainty that this places on the measured weight loss and hence on the calculated rate of corrosion has been discussed earlier. Nevertheless,

- i the changes in weight of the galvanised specimens were very small, and
- ii although the ungalvanised specimens were not particularly uniformly corroded there was no severe pitting.

The data obtained from the weight loss coupons are provided in Table 26. The measured increase in the weight of some of the recovered galvanised specimens was probably due to the formation of strongly adherent corrosion products. The data show that the rate of corrosion towards the top of the structure has been very low in the ten years or so following the end of construction.

There was some loss of weight of the ungalvanised coupons, but the data do not show any clear trend. Corrosion was not particularly uniformly distributed on the coupons but severe pitting was not evident. The mean loss in weight of the two specimens recovered in May 1994 corresponds to a mean loss in thickness of about 14 μm, i.e. a mean rate of corrosion of 1.4 μm/year.

6.3.2 Electrochemical potential

The corrosion potentials measured in May 1994 are reproduced in Table 27. These data indicate that the potential decreases from the top to about the mid-height of the structure and then remains reasonably constant to the bottom of the wall. The high negative values suggest that the potentials are associated with zinc. The corrosion potential of bare steel is usually greater, i.e. more positive, than -600 mV relative to a $Cu/CuSO_4$ reference electrode.

6.3.3 Linear polarisation resistance

The rates of corrosion determined from the linear polarisation resistance measurements taken in May 1994 are also presented in Table 27. These data indicate that the rate of corrosion in the upper half of the structure was less than in the lower half, but the corrosion rates are low and the difference is not significant.

6.3.4 Electrochemical impedance

The rates of corrosion determined from the measurements of electrochemical impedance taken in May 1994 are also presented in Table 27. These data show the same trend in the rate of corrosion with the depth of the anchor as the LPR measurements. The values of the solution resistance (R_s) and the film resistance (R_f) determined from the data taken in May 1994 are given in Table 27. The trend in the solution resistance may be due to variation in the moisture content of the backfill with depth. The measurements of film resistance show that all the anchors had a high resistivity layer of zinc corrosion products.

6.3.5 Variation in the rate of corrosion with time

A summary of the rates of corrosion determined from the weight loss coupons is given in Table 28. Because the measured changes in weight are not at least an order of magnitude greater than the accuracy of the weighings the early data are not reliable.

A comparable summary of the rates of corrosion determined from the electrochemical measurements is given in Table 29. The rates determined from the linear polarisation resistance and impedance measurements were in good agreement and so only their mean value is provided in the table.

There are insufficient data to complete a thorough comparison of the rates determined from the corrosion coupons and the electrochemical measurements. The data from the weight loss coupons could be expected to be an underestimate of the rate of corrosion as some of the loss in weight would be offset by the formation of adherent corrosion products that would not be completely removed by brushing. Nevertheless the later data sets are in good agreement and indicate a rate of corrosion of about 0.2 μm/year in the upper half of the structure. The electrochemical measurements indicate a rate of corrosion of about 0.3 μm/year in the lower half of the structure.

The electrochemical measurements indicate that the rate of corrosion increased marginally between June 1988 and May 1994 but further work is required to establish whether this change is significant.

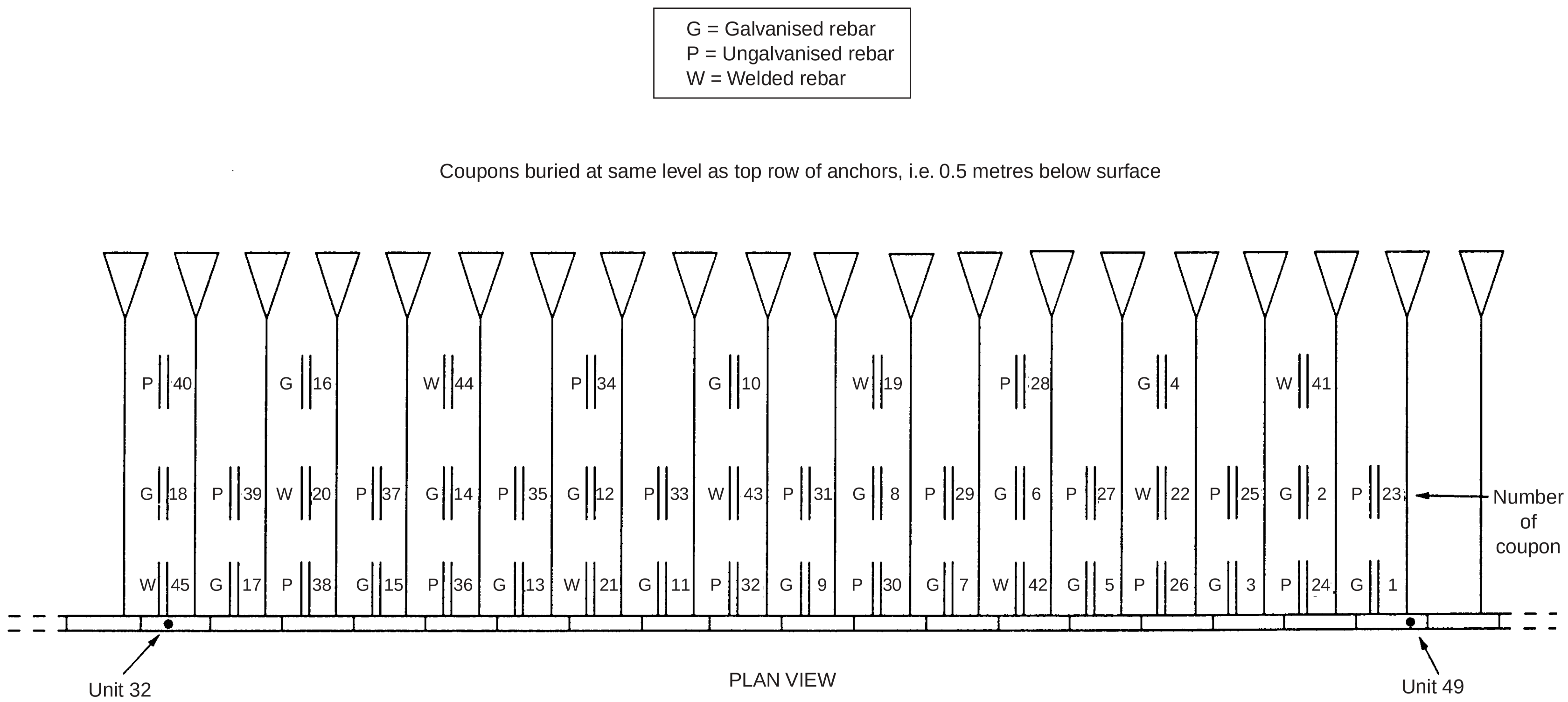
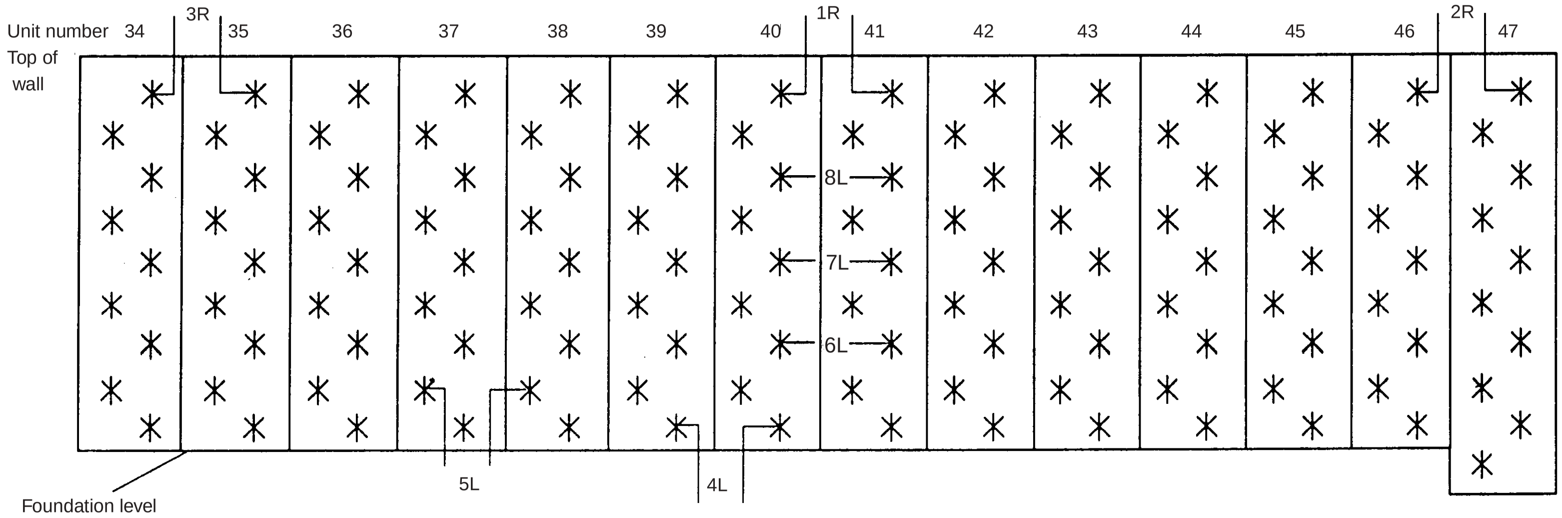


Figure 28 Layout of the corrosion coupons at the Otley site



(L and R refer to the connection arrangement in the junction box)

Figure 29 Connected pairs of anchors for electrochemical measurements at the Otley site

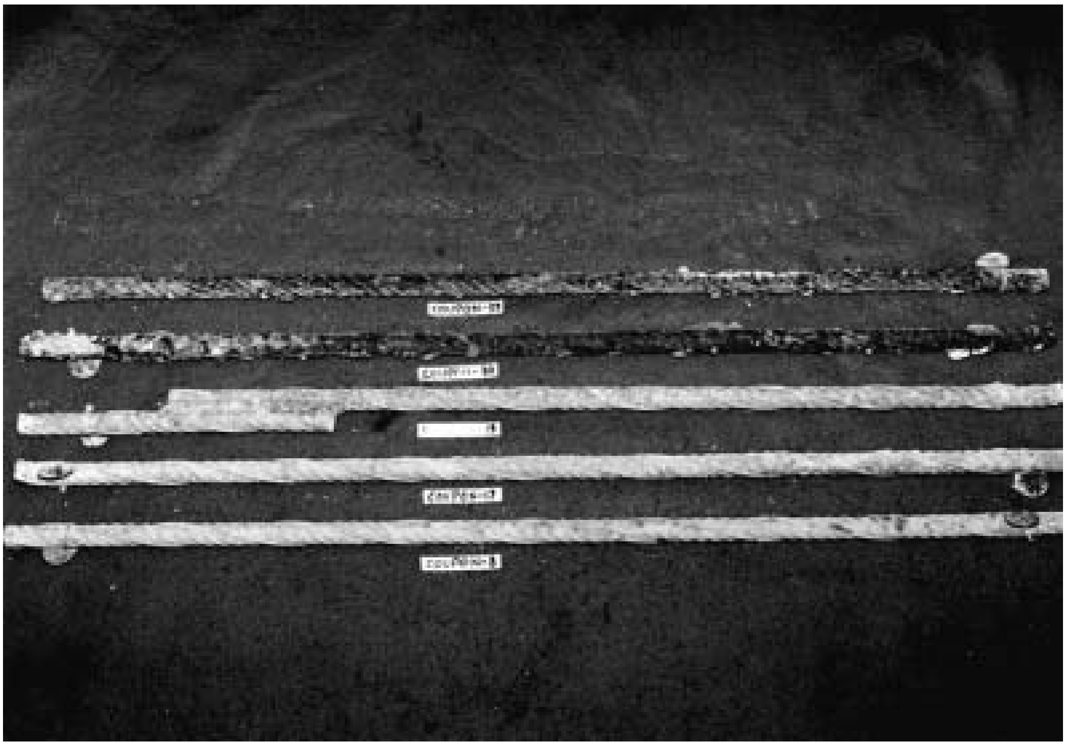


Figure 30 Appearance of coupons following their recovery from the Otley site in 1994

Table 26 Data from corrosion coupons recovered from top of the anchored earth wall at Otley

Date of recovery (approximate time of burial)	*Galvanised specimens*		*Welded specimens (both galvanised and ungalvanised)*		*Ungalvanised specimens*	
	*Coupon number**	*Change in weight (gm)*	*Coupon number**	*Change in weight (gm)*	*Coupon number**	*Change in weight (gm)*
June 1985 (1.2 years)	6 15	0 +2	42 ungalvanised	-1	28 37	-2 -1
April 1986 (2 years)	4 11	+1 +1	22 galvanised	+1	26 33	-1 0
April 1987 (3 years)	9 18	0 +2	45 ungalvanised	-6	31 40	-4 -7
June 1988 (4.2 years)	1 12	0 +2	21 galvanised	0	23 34	-3 -5
May 1994 (10.1 years)	8 17	0 -1	19 galvanised	-1	30 39	-8 -6

** Position of coupons shown in Figure 28*
The initial weights of the coupons were:
1) ungalvanised about 2.4kg
2) galvanised about 2.5kg
3) welded about 3.0kg

Table 27 In situ electrochemical measurements taken at the anchored earth wall at Otley in May 1994

	Corrosion potentials		*LPR measurements*		*Impedance measurements*		
	Individual (mV)	*Mean (mV)*	*Corrosion rate (μm/year)*	*Mean value (μm/year)*	*Mean corrosion rate (μm/year)*	*Solution resistance (R_s) (kΩcm²)*	*Film resistance (R_f) (kΩcm²)*
Uppermost row of anchors (Pair Nos 1, 2, 3)*	-864, -844, -899, -923, -857, -859	-881	0.17, 0.18, 0.13, 0.15, 0.16, 0.16	0.16	0.21 (range 0.15 to 0.33)	1197 (range 1027 to 1334)	612 (range 550 to 671)
Third row of anchors (Pair No 8)*	-889, -893	-891	0.24, 0.22	0.23	0.26	1246	249
Fifth row of anchors (Pair No 7)*	-929, -918	-924	0.31, 0.30	0.31	0.28	920	172
Seventh row of anchors (Pair No 6)*	-943, -931	-937	0.30, 0.32	0.31	0.30	958	182
Eighth row of anchors (Pair No 5)*	-933, -938	-936	0.28, 0.36	0.32	0.29	882	144
Ninth row of anchors (Pair No 4)*	-925, -941	-933	0.35, 0.42	0.39	0.31	830	192

* *Position of anchors shown in Figure 29*

Table 28 Rates of corrosion determined from weight loss coupons recovered from the anchored earth wall at Otley

Date of recovery (approximate time of burial)	*Type of coupon*	*Mean rate of corrosion from recovered coupons (μm/year)*
June 1985 (1.2 years)	Galvanised Ungalvanised	0 2.2*
April 1986 (2.0 years)	Galvanised Ungalvanised	0 0.5*
April 1987 (3.0 years)	Galvanised Ungalvanised	0 3.8
June 1988 (4.2 years)	Galvanised Ungalvanised	0.3 1.9
May 1994 (10.1 years)	Galvanised Ungalvanised	0.14 1.4

* *Not deemed particularly reliable, values close to sensitivity of measurements*

Table 29 Mean rates of corrosion from electrochemical measurements taken at the anchored earth wall at Otley

Date of recovery (approximate time of burial)	*Mean rate of corrosion (μm/year)*					
	Top level	*3rd level*	*5th level*	*7th level*	*8th level*	*Bottom level*
June 1985 (1.2 years)				*		
April 1986 (2.0 years)				*		
April 1987 (3.0 years)	<0.3 for all levels of anchors**					
June 1988 (4.2 years)	0.10	0.13	0.19	0.18	0.20	0.23
May 1994 (10.1 years)	0.18	0.24	0.30	0.31	0.31	0.36

* Data not deemed reliable
** Values close to sensitivity of measurements

6.4 Summary

The latest data sets indicate a loss of thickness of galvanising of between 0.14 and 0.36 μm/year. Given that the specified minimum thickness of the galvanising was about 140 μm, this loss is undoubtedly due to the reaction of zinc: this is confirmed by the high recorded negative potentials and the appearance of the coupons. If these rates of loss were maintained the mean lifetime of the coating would be in excess of 400 years. The time to extensive perforation of the coating would be much less than this, say about 80 years, but this is more than adequate. This relatively low rate of corrosion is probably associated with the formation of a protective layer of zinc products, such as complex oxides and carbonates formed from the reaction of the galvanised coating with the surrounding limestone backfill.

The latest set of data from the bare steel weight loss coupons indicated mean rates of corrosion of about 1.4 μm/year. The anchors were designed with a specified sacrificial steel thickness of 0.75 mm and so this could provide a mean lifetime of about 500 years. The effect that a patchy coating of zinc has on the rate of corrosion of the exposed metal surface cannot be estimated with any certainty. Nonetheless the durability of the anchors is not a cause for concern.

6.5 Future work

Twenty five of the forty five pre-weighed sections of reinforcing bar have been removed. It would be relatively straightforward to remove further specimens and the cost of excavation and determining the weight loss of the coupons would not be prohibitive. But from previous experience it may prove difficult to remove all the corrosion products from the specimens and it might be worthwhile considering the determination of the tensile strength of the specimens.

There are also eight pairs of anchors connected electrically to enable electrochemical measurements to be made. Further readings can be taken at any time, but at the very least it would seem sensible to take in situ measurements at the same time as any further specimens are recovered.

7 Conclusions

This report reviews the field studies undertaken by the Laboratory over the past 20 years to investigate the durability of reinforcements buried in soil.

The data presented show that aluminium alloy, aluminium coated steel, and ferritic stainless steel reinforcements are not suitable for use in reinforced soil structures due to their susceptibility to pitting corrosion. Galvanised mild steel elements, with appropriate thicknesses of zinc coating and sacrificial steel, and austenitic stainless steel reinforcements are suitable but the use of the latter is limited by the relatively high cost.

The galvanised mild steel elements in the studies suffered from a non-uniform loss of cross-section, but intensive pitting was not evident. The values of the pitting index (K), the ratio of the rate of loss at the minimum cross-section and the mean loss for a specimen as a whole, were about 4. However the partial covering of zinc on some of the elements did not provide complete galvanic protection to the exposed steel as suggested by Darbin et al (1988). It would therefore seem optimistic to rely on a uniform distribution of corrosion in the long term: for reinforced soil applications it would seem more appropriate to report the severity of corrosion in terms of loss in strength rather than the mean loss in thickness.

The data from the corrosion experiment at the Laboratory showed that the minimum lifetime of the thicknesses of zinc coating and sacrificial steel, given in BD 70 (DMRB 2.1.5), may be as low as 40 years at a given cross-section, but the data from the other sites indicate that the sacrificial thicknesses are more than adequate for a design life of 120 years with mean rates of corrosion varying between 0.2 and 3 μm per face per year. Many more data than available from the studies reported are required to establish the degree of non-uniformity in the corrosion of buried metallic strips, the relation between loss in tensile strength and loss of thickness, and how both of these vary with time and with the type of soil. But at present there are no grounds for changing the current requirements for corrosion protection.

The majority of the data suggest that the sacrificial thicknesses given in BD 12 (DMRB 2.2.6), for CSBS are reasonable. However again many more data are required to establish accurately the rates and methods of degradation of such structures.

The data for the GRP and Paraweb reinforcements suggest that current design rules afford an adequate margin of safety against rupture over the design life of a structure incorporating such reinforcements.

Additional reinforcements and coupons can be recovered from the various sites and two of the structures have corrosion monitoring systems from which in situ electrochemical measurements can be taken. The collection of further data from these sites would provide valuable information on the longer term durability of materials buried in soil.

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Appendix A: Testing procedure for glass fibre reinforced plastic strips

The method is based on BS 2782: Part 3 (1970). Method 301L: Tensile strength of reinforced plastics with rectangular test specimen.

A1 Sample preparation

Immediately after taking both straps and anchor samples, each individual specimen should be identified by marking on the resin code number (batch no./drum no.) from which the samples were made.

a *Anchor samples*

These should be 300 mm long overall and are prepared by first degreasing the two strap parts of the anchor and then sandwiching these between two chopped strand mat/resin laminates. The laminates used should consist of 3 layers of 450 g/m^2 chopped strand mat, each the full width of the specimen and 125 mm long. The resin/catalyst used are identical to that from which the straps are made.

The laminated sample is then wrapped in cellophane and sandwiched between glass plates under light pressure ready for curing. The prepared sample is then placed in an oven at 100° ± 5°C for 10 minutes to cure the laminates and the edges are trimmed ready for insertion into the test grips.

b *Strap samples*

These are 300 mm long and are prepared by abrading and degreasing the ends of the strip so that resin / glass laminates can be applied to the faces of the ends. In this case, the laminates are made up of 3 interleaved layers of glass cloth and a film adhesive (Ciba-Geigy Redux adhesive 308). The size of the laminates should be 75 mm long with a width 10 mm greater than that of the specimen. These are cured by inserting one end of the test piece (with a laminate either side) into a hot press at 170°C and pressing the sample at a minimum of 80 psi (552 kN/m^2) (allowing for the film adhesive to soften on heating) for 1 hour.

A2 Testing

The sample should be tested on an Instron or similar testing machine, applying load by constant displacement. The time to failure of the sample should be no less than 30 seconds, with the final jaw separation speeds being 5 mm/min for the strap samples and 2 mm/min for the anchors. Standard wedge type grips are used for the strap samples and the appropriate pin jig is used for the anchors.

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BS 2782: Part 3. (1970). *Methods of testing plastics - mechanical properties*.

Abstract

This report reviews the findings of the field studies undertaken by the Transport and Road Research Laboratory (TRRL), now the Transport Research Laboratory (TRL), on the degradation of metallic and non-metallic strips and anchors used for reinforcing soils. Different methods of assessing the degree of corrosion of metallic reinforcements are discussed and the measured rates of corrosion are compared to the current requirements of the Department of the Environment, Transport and the Regions (DETR) for both reinforced soils and corrugated steel buried structures (CSBS). Data are also provided on the loss in strength of non-metallic reinforcements. Opportunities for future work in the light of this review are also discussed.

It is concluded that, at this stage, there are no grounds for changing the present requirements covering the durability of buried metallic components. However many more data than are currently available are required to establish the degree of non-uniformity of corrosion, and the relation between loss in tensile strength and loss of thickness, and how both of these vary with time and with the type of soil. The data for the non-metallic reinforcements suggest that current practice provides an adequate margin of safety against their rupture over the design life of a structure.

Related publications

TRL132 *Long term performance of an anchored earth wall at Otley: 1984 to 1994* by K C Brady and M J Greene. 1995 (price £50, code L)

CR288 *Assessment of steel for reinforced and anchored earth structures* by K A Murray. 1992 (price £25, code F)

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SR457 *Reinforced earth and other composite soil techniques*. Proceedings of a symposium sponsored jointly by the Transport and Road Research Laboratory and the Heriot-Watt University (price £20)

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