

CASE STUDY OF RAILWAY EMBANKMENT STABILISATION WORKS AT MOOR PARK STATION WITH PROGRESSIVE SLOPE FAILURE

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ABSTRACT

Two railway embankments which are located next to Moor Park station, approximately 30 km northwest of London, support the southbound and northbound London Underground Metropolitan Line tracks. The embankments were identified as showing various signs of progressive slope failure and required implementation of stabilisation works. The earthworks originally constructed in the late 19th century were subject to embankment widening works in the 1940s and have had a complex history of construction and modification with evidence of past instability.

For the remedial works, there were a number of geotechnical and construction issues which included highly variable ground conditions, environmentally sensitive sites surrounding the embankments including a man-made river and existing rail structures along the embankments. This paper describes the measures taken to manage the geotechnical risks.

The adopted design criteria, the slope stability assessment and the proposed remedial works which include a bored pile wall with precast concrete capping beam, a discrete bored pile wall, a slope regrade and toe berm are presented in this paper. For the design and slope assessment, modified soil strength parameters for high plasticity index cohesive embankment fill or weathered London Clay identified from soil investigation was used.

1 INTRODUCTION

London Underground had identified two railway embankments located to the east and west of the Metropolitan Line tracks between Moor Park and Rickmansworth stations, approximately 30 km northwest of London, as showing signs of slope instability and identified for the implementation of remedial works. These works are required to reduce maintenance requirements, improve ride quality and raise the asset condition rating.

The remedial works were undertaken under a framework agreement between the London Underground and Cementation Skanska Pty Ltd. The conceptual design, temporary works and permanent works design of the railway embankments were undertaken by Mott MacDonald Metros and Civils Division, UK.

2 RAILWAY EMBANKMENTS

2.1 SITE DESCRIPTION

The remedial scheme consists of two embankments located to the east and west of the Metropolitan line tracks between Rickmansworth and Moor Park stations (see Figure 1). The embankment supporting the southbound Metropolitan Line tracks is known as EM12 and the embankment supporting the northbound tracks is known as EM13.

Embankment EM12 is an approximately 240 m long earth structure embankment which is located between chainage M062/MSB 3030 – M063/MSB 0025. The embankment is bounded to the south by Moor Park Station, to the north by bridge MR70, to the east by an old gravel or chalk pit, the excavated area now being masked by vegetation and to the west by the Metropolitan Line tracks. Embankment EM13 is an approximately 280 m long earth structure embankment (opposite embankment EM12) which is located between chainage M063/MNB 0075 – M062/MNB 0220. The embankment EM13 is bounded to the south by Moor Park station underpass and to the north by bridge MR70A. Urban housing constructed in the 1930s is situated to the west of this embankment and the Metropolitan Line tracks form the eastern boundary.

The site is adjacent to a privately owned residential estate in a semi-rural environment and surrounded by the Moor Park Conservation Area. Some of the trees along the toe of embankment EM12 and EM13 have tree preservation orders (TPO). The site within the boundary fences is moderately wooded with many mature trees.

The embankment structures are typically 3 m to 5 m high, reaching a maximum height of 7 m on the EM12 embankment adjacent to bridge MR70. The overall width of the embankments EM12 and EM13 varies between 46 m and 54 m and the average slope angle is between 18° and 22°. The embankment carries four tracks known as

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Northbound Fast, Southbound Fast (on the western half of the structure), Northbound Local and Southbound Local (on the eastern half of the structure).

There is a river present at the toe of EM13, between approximate chainage M062/MNB 0106 m and M063/MNB 0118 m. At chainage M062/MNB 0106 m this river passes diagonally beneath the embankment in culvert MR69, emerging outside the site boundary. The river channel varies between 1.5 m to 2 m deep with sheet piling and rubble gabion walls protecting the river bank on the curved culvert approach.

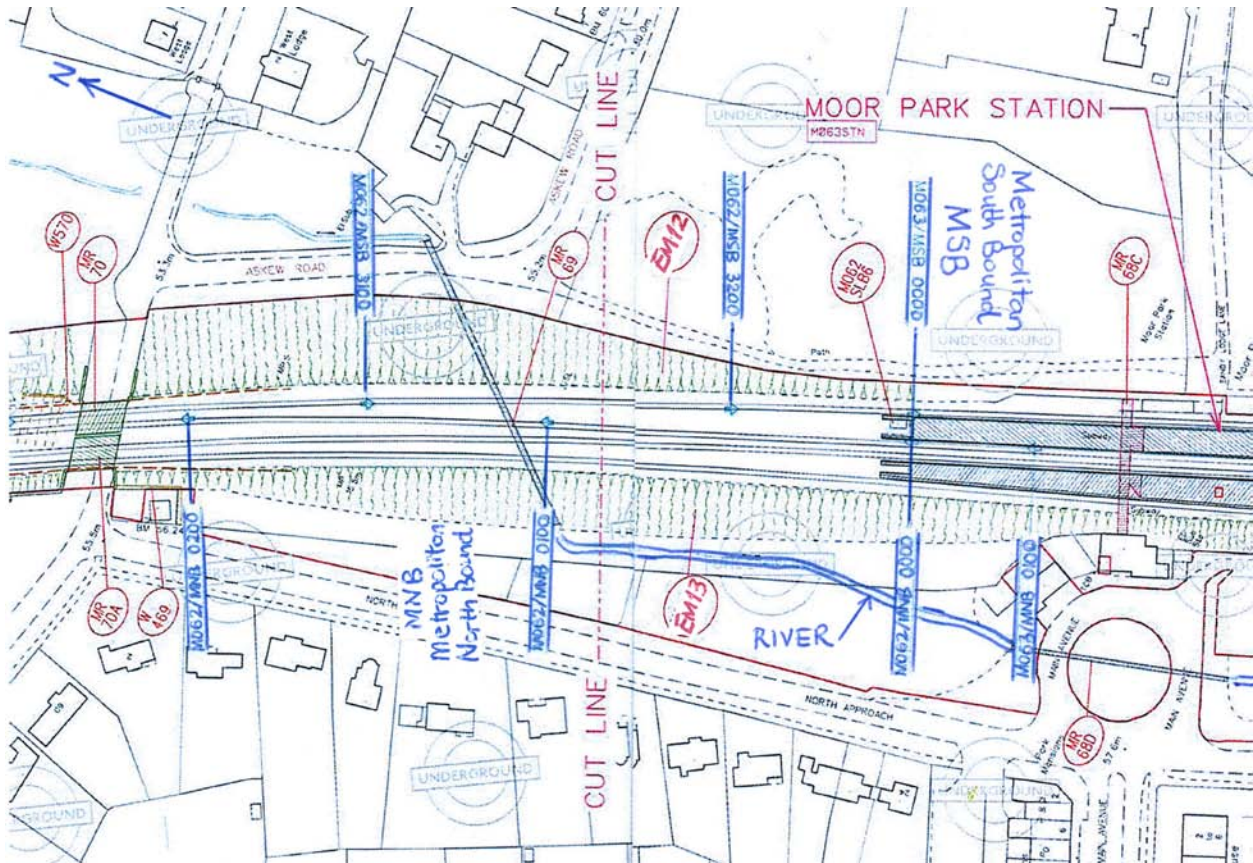


Figure 1: Asset location plan for embankments EM12 and EM13 between Moor Park and Rickmansworth station

2.2 CONSTRUCTION HISTORY

The earth structures at this site have had a complex history of construction and modification with evidence of past instability on both embankments. The embankments were originally constructed in the late 19th century from reworked London Clay and locally derived Chalk and Head Deposit materials. Over a prolonged period either side of the Second World War, widening of the embankment took place using similar fill materials, in addition to hardcore rubble from bomb-damaged sites. The embankment widening was implemented to allow two rail tracks to be added. The widening was not completed until the early 1960s.

Extensive remedial and reconstruction works have been undertaken at the site. The following highlights past remedial works which have taken place since the embankment was widened.

- i) A slip failure occurred in 1961 (at approximate chainage M062/MNB 0200 m) and was repaired with counterfort drains.
- ii) Slope failure six months after the line re-opened to traffic remediated with counterfort drains and a rubble toe retaining wall.
- iii) Problems were encountered in 1966 with the embankment cohesive fill material squeezing into the more porous structure formed by the brick rubble leading to localised settlement of the track and the formation of ash and water-filled pockets.
- iv) An extensive period of grouting of EM13 was undertaken at intervals of 10m along the full length of the embankment following the widening works in 1966. The majority of the grout take was within the shoulder and mid-slope portion of the earth structure.

2.3 SITE WALKOVER AND TOPOGRAPHIC SURVEY

Following the commission of work by London Underground, a topographic survey was undertaken which covers the full length of both embankments EM12 and EM13. The slope geometry of EM12 varies, with an irregular generally concave profile comprising a steep upper slope (25° to 33°) giving way to a shallower middle and lower slope (14° to 20°). Locally the slope steepens towards the toe and a toe drainage ditch exists with overgrown vegetation. The slope geometry of EM13 has a generally convex profile with a flat to gently dipping shoulder giving way to a steeper middle and lower slope (23° to 39°).

From initial slope geometry and slope distress/movement observations, EM12 appears to be in a deteriorated state and exhibits significantly more instability indicators than EM13. On EM13 the slope is generally regular and even.

A site walkover was also undertaken at both embankments and shallow slope instability features were observed (see Figure 2). These features include ravelling of the ash upper slope at certain stretches of EM12 and EM13, exposing footing posts for the lineside services cable run and resulting in some ash build up against trees. Other instability features observed include creep terracing and exposure of downslope roots of many of the mature trees.



Figure 2: Signs of instability at crest: vertical displacement of services and partially buried cable (left photo) and leaning cable posts carrying cable runs (right photo).

Evidence of past remediation works were observed during the site walkover. These include presence of grouting at the northern end of EM12 and extensive grouting along much of the length of EM13. Other visual evidence includes a rubble wall at the toe of a short section of embankment EM13 and construction debris including bricks and concrete.

3 GEOTECHNICAL CONDITIONS

3.1 UNDERLYING GEOLOGY

The embankments were constructed close to the course of a river, which was diverted to pass beneath the embankment in a culvert. The area is situated on the southern flank of the River Colne and River Gade river valleys, of which the river is a tributary and the surrounding land dips gently to the north-east.

Fluvial flows across the area have locally removed the Tertiary beds on site such that London Clay and Reading Beds occupy the higher ground to the south-east and south-west of the site whilst Upper Chalk may be found sub-cropping immediately below superficial deposits beneath the embankment. Gravel pits are located adjacent to the site. Chalk is expected at an average depth of around 3 m below the embankment toe.

Brickearth and River Terrace Gravel are also present beneath the embankment. There is a thin layer of alluvium overlying the River Terrace Gravel at certain sections of EM12. The original ground level at the site is approximately 152 to 154 mTD.

3.2 GROUND CONDITIONS

Geotechnical investigations were carried out for the earth structures to confirm the geological profile beneath embankments EM12 and EM13 and the embankment fill materials.

Embankment EM12 comprises predominantly cohesive fill material, probably derived from London Clay. Ballast is present at the track, and beneath the ballast a layer of Ash of variable thickness exists. Ash is also present on the slope surface at the crest and shoulder of the slope. It reaches a thickness of nearly 3 m at the northern end of the embankment and locally forms a large proportion of the embankment volume. The thickness of embankment cohesive fill (EFC) is approximately 6 m throughout the length of EM12. Some small patches of laterally non-persistent granular fill up to 1.3 m deep exist within EM12. The EFC material overlies brickearth at the southern end of the asset and Terrace Gravel at the northern end. Chalk exists beneath the Brickearth and Terrace Gravels at a level of approximately 148 mTD throughout the length of asset EM12. This chalk is highly weathered to a depth of 5 m to 7 m below the base of the embankment.

Embankment EM13 comprises cohesive fill (EFC) overlying Brickearth of maximum thickness 4 m, which overlies a generally 3 m thick layer of river terrace deposits. An Ash layer, up to 1.5 m thick, exists at the surface of the slope at the crest. At the northern end of the asset there has been significant ravelling of the Ash shoulder down the slope. Brick rubble and grouted brick rubble were encountered at a number of sections and was considered not likely to be confined to these locations.

Table 1 summarises the ground conditions encountered during geotechnical investigations within the site.

Table 1: Summary of Strata Encountered

Strata	Thickness (m)	Reduced level of top of stratum (mTD)	Reduced level of bottom of stratum (mTD)	Location
	Max/Min	Max/Min	Max/Min	
Railway Ballast	0.8/0.1	159.8/159.5	159.4/159.0	Track area surrounding sleeper
Ash	3.5/0.1	160.0/157.2	159.9/156.2	Beneath track bed and locally forming parts of the embankment shoulder
Cohesive Embankment Fill (EFC)	6.1/0.05	159.7/152.4	159.3/151.7	Present as fill throughout the structure
Granular Embankment Fill (EFG)	1.3/0.05	159.7/154.8	159.5/153.9	Locally present throughout the structure
Grouted Granular Embankment Fill	1.2/0.2	159.1/157.3	159.0/157.0	Northbound slope and on southbound slope adjacent to MR70.
Alluvium	0.5/0.05	153.3/152.45	153.25/151.95	Locally present above the Terrace Gravels
River Terrace Gravels	2.8/0.3	153.4/149.5	153.1/149.2	Present beneath the embankment towards the north end of the site.
Head Deposits/Brick Earth	5.5/0.2	155.3/149.7	155.0/147.0	Present beneath the embankment throughout the site.
Chalk	-	152.31/147.03	-	Present beneath the whole of the embankment. Base of stratum not penetrated.

4 DESIGN CRITERIA

The design criteria for the remedial works have been set out in London Underground Engineering Standards and are summarised as follows;

- i) To increase the Factor of Safety, FOS, using a limit equilibrium approach to greater than 1.3 against failure of the embankment slopes from deep seated slope failure (slips affecting track or lineside services or structures which support track or lineside services).
- ii) To increase the Factor of Safety, FOS, using a limit equilibrium approach to greater than 1.15 against failure of the embankment slopes from deep seated slope failure over lengths which use reduced soil strengths.

- iii) To increase the FOS using limit equilibrium approach to greater than 1.15 against shallow slips passing through the sloping part of the embankment. Shallow slips are defined as those which only pass through the sloping part of the earth structure outside of the lineside services.
- iv) To improve the shoulder stability of the embankments.
- v) The design of the stabilisation works shall also ensure that the slopes do not reach the serviceability limit state. The serviceability limit state is defined as being when movements affect the function of the slopes which support the track, lineside services and/or other adjacent structures and services, including those assets owned by third parties.
- vi) To design the remedial work such that it will have no detrimental effect on the stability of existing structures.
- vii) A 0.7-1 m wide walkway to be provided at the crest and at the toe of both embankments where space permits.
- viii) To restrict differential settlement of the outside rail to 1:500 along the track over any 10 m (+/-20 mm) interval and 1:300 (+/-5 mm) across the rails.

5 GEOTECHNICAL PARAMETERS

5.1 SOIL PARAMETERS

The soil parameters used in the design of the embankment stabilisation are presented in Table 2 and are based on previous geotechnical investigations. In general, the parameter values are in agreement with the values provided by London Underground Standard except for the embankment fill cohesive (EFC).

Table 2: Geotechnical parameters

Material type	Bulk density(kN/m ³)	c' (kN/m ²)	Friction angle (°)
Embankment Fill Cohesive (EFC)	19.0	See Table 3	See Table 3
EFC, Residual Strength	19.0	1	16
Embankment Fill Granular (EFG)	18.0	0	35
Brick Earth	19.3	1	27
Terrace Gravel	19.0	0	35
Ash	11.0	0	35
Imported Class 1A Granular Fill	21.0	0	40
Rail Ballast	18.0	0	40
Chalk	17.0	1	28

It has been known for many years that cuttings and embankments in stiff high plasticity clays are prone to progressive failure, with average mobilised strength for first time failures between the peak bulk strength and residual strength (Chandler and Skempton, 1974; Chandler, 1984). Progressive failure can develop when strains within an earth structure are mobilised in a non-uniform manner and the material in the earth structure exhibits a post-peak loss of strength. Back analyses carried out by Skempton (1977) of several failures of cuttings in weathered London Clay indicated strength parameters of effective cohesion (c') of approximately 1 to 2 kPa and friction angle of approximately 20 to 21°. It should be noted that these were all delayed failures which took place several decades after construction. The parameters given in the London Underground standard have been derived from these back analyses, and it should be noted that these failures were within medium to large cuttings, typically between 5 m and 8 m depth.

Recent numerical modelling by Potts *et al.* (1997) and Ellis and O'Brien (2007) have provided insight into the relevant factors that affect progressive failure in the EFC material and showed that the size of earth structure and strain to mobilise peak strength are important considerations. As the size of earth structure increases progressive failure becomes more significant, the average mobilised strength over a failure surface will be higher for smaller embankments/cuttings and the mobilised strength would be higher for embankments than cuttings (Ellis and O'Brien, 2004). Internal applied research carried out for the EFC material concluded that the strength parameters for this material given in the London Underground Engineering Standard was overly conservative for small embankments and cuttings (less than 4 m to 5 m high) and higher strengths which are more realistic have been proposed in the earth structure assessment and detailed design.

Based on the geotechnical investigation results, the EFC material found at assets EM12 and EM13 had an average plasticity index of 40% which is considered high plasticity clay and therefore at risk of suffering progressive failure. Also, the height of the EFC within the embankments is generally less than 5 m.

Although various parts of the embankment have been subject to cement grout treatment and use of brick and granular material have been reported in the ground investigation results, all of which would improve the strength of the EFC layer, the modified EFC parameters have been adopted in the embankment design. The modified EFC parameters used in the design for these chainages utilise the guidance given in Table 3 which has been developed based on the internal applied research and validated from monitoring results obtained from instrumented earth structures for recently completed stabilisation works. Higher strengths have been proposed for small embankments, which is consistent with the results of the applied research on progressive failure.

Table 3: Modified parameters based on embankment height

Embankment Height (m)	c' (kN/m ²)	Friction angle (°)
≤ 2	4.5	20
4	3.5	20
6	2.0	20
≥ 7	1.0	20

Residual strength EFC has been used at certain chainages where there are local instability indicators such as thick layer of Ash (up to 3m) at the slope crest, localised terracing and toe bulges. Although no previous slope failure had been reported for EM12, taking into account the 120 year design life, it was considered prudent where such instability indicators are present to adopt a residual strength EFC, which would lead to a robust design in these areas.

Along the southern section of EM13, the EFC material encountered from the ground investigation encountered a number of window sampling refusals due to presence of grout, brick and cement along the midslope and crest of the embankment. The soil friction angle in this area was increased to 24°.

5.2 GROUNDWATER CONDITIONS

The existing river and counterfort drains at embankment EM13 would allow water to drain away and the piezometric line is likely to be close to the river surface level. The embankment is also expected to be underdrained due to the presence of Brick Earth, Terrace Gravel, and Chalk layer which exist partially or wholly beneath the embankments. There was also significant variation in the groundwater monitoring results, which was interpreted as a result of the complex distribution of different material types within the structure.

Based on the monitoring results and the guidelines provided in the Engineering Standard, the groundwater profiles were taken between 1 m to 2 m below the Ash-EFC interface, or 1 m below ground level at the toe.

5.3 LIVE LOADS

In accordance with the London Underground Standard, a minimum uniform vertical live load of 50 kPa over the track area and uniform vertical live load of 5 kPa across the walkway along the crest of the embankment were applied in the embankment design.

6 METHOD OF ANALYSIS

Analysis of the stability of the embankment has been carried out using the slope stability program Slope/W 2007 (GEO-SLOPE International Ltd).

The analysis technique comprises carrying out a search routine using the Morgenstern-Price method to identify the geometry of a critical slip circle with the minimum factor of safety. Then, for the critical analyses, the out of balance force, F_x , which is required to achieve a factor of safety of 1.3 is calculated. The force, F_x , is then input into a bored pile wall design spreadsheet, which carries out the calculations in accordance with Eurocode EC7 (2004). The spreadsheet calculation is consistent with the requirements of EC7 in that the dimensions of the piles are derived from factored soil strengths and unfactored loads, and the structural requirements (i.e. reinforcement and concrete strength) are derived from factored loads and unfactored soil strengths. For simplicity, pore water pressures are unfactored. In this instance because there is no head difference across the piled retaining structures, the impact on design is believed to be negligible.

The external forces acting on the retaining structure above the critical slip surface are represented by F_x values. The forces acting on the retaining structure below the critical slip surface are represented by active and passive pressure profiles. The capping beam develops effective fixity at the head of the vertical piles. Maximum bending moments acting on the vertical piles are calculated from the above limit equilibrium calculations. The vertical pile lengths are also calculated to ensure sufficient vertical and horizontal capacity.

Calculations are then carried out to assess the required moment capacity of the piled retaining structure to stabilise the slipping mass and to achieve the required factor of safety. The required embedment of the piles below the potential slip surface, and structural capacity of the pile section to provide this resistance are checked by an automated calculation using a design spreadsheet.

Slope stability analysis is then carried out to confirm the required depth of the retaining structure to give a factor of safety of 1.3, or 1.15 in the case of residual strength analyses.

Design of individual structural members is then carried out, including the capping beam, and vertical pile reinforcement. The capacity of the pile for selected reinforcement quantities is calculated in accordance with the British Standard BS 8110 (1997).

An analysis of the downslope side of the bored pile retaining structure is undertaken using the slope stability program Slope/W. Further remedial works, such as re-grading are designed if a factor of safety below 1.15 is calculated.

7 ASSESSMENT OF EMBANKMENTS

Slope stability analyses for the existing slope profile were carried out based on the topographic survey information. In locations where no evidence of previous instability is reported, or it is reported that previous failures have been remediated with grout, the preliminary analyses are based on a first time slip occurring. In locations where it is believed that the slope has suffered previous instability, analyses are based on a residual-strength slip surface. These locations are indicated by a large thickness of ash downslope of the embankment shoulder discovered during ground investigation and by terracing of the slope.

A summary of the analyses for both deep seated and shallow surface slips for the existing slope profile is presented in Table 4 below. In accordance with the London Underground Standard deep seated slips are required to have a factor of safety of 1.3 (or 1.15 where the EFC is assumed to mobilise residual strength only) and shallow slips a factor of safety of 1.15.

Table 4: Summary of the existing slope profile stability analysis

Earth Structure	Section	Chainage	FOS for deep seated slip (affecting track or lineside services)	FOS for shallow slip	Comments
EM12	A1	M062/MSB 3038	0.86	1.01	EFC residual strength modelled
	A	M062/MSB 3060	0.93	1.05	EFC residual strength modelled
	B	M062/MSB 3110	1.21	1.16	EFC residual strength modelled
	C	M062/MSB 3145	1.54	1.72	EFC residual strength modelled
	D	M062/MSB 3216	1.52	2.22	EFC modelled using modified parameters
EM13	E	M062/MNB 0180	1.20	1.63	EFC modelled using modified parameters
	F	M062/MNB 0142	1.16	1.17	EFC modelled using modified parameters
	G	M062/MNB 0106	1.08	1.18	EFC modelled using modified parameters
	H	M062/MNB 0080	1.22	1.16	Friction angle of EFC increased due to grouting
	J	M062/MNB 0036	1.22	1.13	Friction angle of EFC increased due to grouting
	K	M063/MNB 0075	1.18	1.27	EFC modelled using modified parameters

8 PROPOSED REMEDIAL DESIGN

Various types of remedial works were selected to address the slope instability along embankments EM12 and EM13. Descriptions of the remedial options and the extent used for the earth structures are presented in the following sections.

8.1 BORED PILE WALL AND CAPPING BEAM

This type of remedial work consists of a 310 mm bored pile wall tied to a capping beam structure with regrading. The primary role of this structure is to provide the necessary crest support and allow a 0.7 m to 1 m wide walkway to be constructed at the crest of the embankment. The vertical piles are designed to pass beneath the slope failure plane and be embedded in the hard stratum below the embankment. The capping beam and piles run approximately parallel to the lineside services.

8.2 DISCRETE BORED PILE WALL

This form of slope stabilisation measure requires a combination of 450 mm diameter discrete bored piles positioned at mid-slope with re-grading above the piles to support the Ash shoulder. This option will allow a 0.7 m to 1 m wide walkway at the toe and crest of the embankment. The vertical piles are designed to pass beneath the slope failure plane and be embedded in the stable ground beneath.

A discrete bored pile solution will be sympathetic to the surrounding environment as vegetation clearance is only required upslope of the piles to allow for slope regrading. It is envisaged that vegetation downslope of the piles can remain largely in place along with some larger trees at the crest. Where there are large trees at the toe of the embankment some protection measures may be installed during the works and regrade placed locally around them. This solution would lessen any visual impact compared to erecting a physical slope retaining structure such as the bored pile wall and capping beam solution.

8.3 SLOPE REGRADE WITH TOE BERM

For design cases where the height of the embankment reduces and there is sufficient space, a simple regrade solution of the slope to a shallower angle is proposed. A toe berm with slope regrade has also been included in the permanent works design for slope sections where there is sufficient space at the toe of the embankment. The remedial option of a slope regrade would avoid the need for a bored piled wall solution and represents a considerable cost saving.

8.4 CAPPING BEAM

The reinforced capping beam will be formed from pre-cast CemRailBeam® units. The pre-cast unit comprises the front and back faces of the capping beam tied together by reinforcement bars. The pre-cast units are then placed over the top of the exposed vertical reinforcement extending from the pre-installed piles. Longitudinal reinforcement bars are placed along the beam to tie the individual units together. The void between the front and back face is subsequently filled with concrete to form the complete capping beam.

The capping beam will be fitted with a two bar handrail throughout and positioned at the crest of the slope. The walkway is formed by placing backfill behind the capping beam.

8.5 SUMMARY OF DESIGN

A summary of the remedial design is presented in the table below.

Table 5: Summary of remedial works for Embankments EM12 and EM13

Site/Zone	Track	Approximate Chainage		Section Length (m)	Representative Design Cross Section	Description of Remedial Works
		Start	Finish			
EM12	MSB	3035	3045	10	A1	Bored pile wall and capping beam with end return Vertical Piles: 310 mm diameter, 9 m long at 620 mm c/c spacing.
	MSB	3045	3054	9	-	Bored pile wall and capping beam plus stop end & discrete bored piles. Vertical Piles: 310 mm diameter, 9 m long at 620 mm c/c spacing. Discrete Vertical Piles: 450 mm diameter, maximum 9 m long at 900 mm c/c spacing.
	MSB	3054	3155	101	A, B, C	Discrete Bored Piles and regrade Discrete Vertical Piles: 450 mm diameter, maximum 9 m long at 900 mm spacing.
	MSB	3155	0025	120	D	Slope regrade .
EM13	MNB	0220	0200	20	-	Minor slope regrade only.
	MNB	0200	0110	90	E, F	Slope regrade with toe berm.
	MNB	0110	0090	20	-	Minor slope regrade only.
	MNB	0090	0096	129	H, J	Slope regrade with toe berm.
	MNB	0096	0075	21	K	Bored pile wall and capping beam and regrading. Including installation of stop end and staircase connecting crest and toe walkway. Vertical Piles: 310 mm diameter, 5 m long at 620 mm c/c spacing.

9 DESIGN IMPACTS ON EXISTING STRUCTURES

There are a number of existing structures which would be affected by the proposed remedial works. The following presents the design checks, mitigation measures and precautions that were proposed for the stabilisation works.

9.1 UNDERLINE BRIDGE MR70/MR70A

Underbridge MR70/MR70A forms the northern boundary of the site. The bored pile wall and capping beam would return back against bridge MR70 and the end return would allow a tapered crest walkway to match the existing ballast level.

Pile installation is largely free from vibration, and hence it is considered the installation process is unlikely to have any influence on the performance of existing structures. Design checks confirmed that the construction of piles close to the underline bridge MR70 would not increase loading on the bridge structure. The capping beam return would not be tied to the bridge wing walls, so there is no structural connection between the two structures. This means that the loads cannot be shed from the capping beam return onto the bridge structure and the installation of the piles or the performance of the remedial structure would not have a detrimental effect on the existing bridge abutments.

During the pile installation process, it was recommended that an inspection and monitoring programme for the wing walls and retaining wall should be undertaken by the contractor to validate that the pile installation process had not impacted on the functional performance of the bridge wing walls.

9.2 RETAINING WALL W469

W469 is located next to bridge MR70A on embankment EM13 and retains the slope for a length of approximately 20 m. The retaining wall protects a derelict building located at the slope toe. Works behind this wall comprise minor regrade to form a crest walkway. The remainder of the regrade on the slope has been designed to match the existing profile so that W469 does not experience any increase in loading as a result of the proposed remedial works. A design check has also been carried out and has confirmed that W469 wall is not affected by the works carried out further upslope.

During the regrade process an inspection and monitoring programme for this structure would be undertaken during construction to validate that the structure has not been adversely impacted upon.

9.3 CULVERT MR69

Culvert MR69, which carries the river flow, runs beneath the embankment EM13 and EM12, before discharging outside the London Underground site boundary. Within the vicinity of the MR69 culvert headwall, the permanent works comprised local regrading of the slope, creating a crest walkway for shoulder stability and installing a protective handrail above the headwall and returning to both ends.

Additional analysis was undertaken and confirmed that the applied plant load surcharge at the working platform level (approximately 4.5 m above the culvert soffit level) would not significantly increase the existing applied pressure on the culvert. Furthermore, it was considered that any plant loading above the culvert would be transient and for short term only, which is determined to be much less than the existing overburden pressure and 50 kPa rail surcharge below the rail section.

To avoid damaging culvert MR69, a 3 m wide offset from the outside edge of the culvert walls has been specified for the permanent works, resulting in a 7.7 m wide pile exclusion zone. Utilising the soil face resistance on both sides of the slope and the base stiffening effect of the buried culvert structure, the calculation check has shown that the slope section unsupported by discrete piles meets the minimum required slope factor of safety.

It was recommended that a condition survey using CCTV be undertaken for the culvert prior to the start of construction and at the end of construction in order to validate that the culvert has not been damaged by the planned works.

10 ENVIRONMENTAL CONSIDERATIONS

The embankments are located in an environmentally sensitive area and the environmental risks arising from the proposed remedial works required careful considerations.

Implementation of the proposed remedial solutions would require vegetation removal from the crest and slopes of embankments EM13 and EM12. On embankment EM12, where the remedial solution is discrete piles with upslope regrade, it was envisaged that some vegetation downslope of the piles could be retained. It was highlighted to the contractor that where possible, large trees located at the toe of the embankments should remain in place and regrade be placed locally around them.

A river is present along the southern half of embankment EM13. Pollution measures will be put in place following guidance given in the Environment Agency's Pollution Prevention Guidelines. A scaffold and ply temporary hoarding was provided at least 1.5 m from the river bank to prevent plant from entering the exclusion zone. A shallow trench is excavated next to the barrier on the works side, which is backfilled with filter material and lined with geotextile filter sheet tied to the plywood boards. The system is intended to trap sediments from surface run off and prevents spoil from entering the river.

A landscaping scheme was designed and undertaken by the contractor to provide an even water demand from vegetation along the embankment. A programme of replanting will be carried out once the remedial works are complete, in accordance with London Underground Standards. The replanting included wildflower seeding, shrubs and low water demand trees.

11 CONCLUSIONS AND RECOMMENDATIONS

The case study of a railway embankment stabilisation works on two earth structures which were showing various signs of progressive slope failure has been presented. The following highlights the conclusions and lessons learnt from the remedial design work undertaken for the embankments:-

- i) It is critical that adequate geotechnical investigation should be undertaken prior to the conceptual and detailed design of an embankment/cutting stabilisation works, particularly for earth structures which have experienced previous remediation and modifications.
- ii) Site walkover along the length of the earth structure is recommended to gain an appreciation of the site conditions and site constraints such as existing rail structures and plant access route. Observations should be made for any slope distress signs such as significant Ash thickness, terracing or bulging at the toe of earth structures. Considerations should be made for adopting residual strength parameters where there are instability indicators.
- iii) Earth structures with high plasticity index clays are prone to progressive failure and research undertaken by others has shown that the average mobilised strength over a failure surface is higher for smaller earth structures (less than 5 m high).
- iv) The selection of remedial options for earth structure stabilisation works should consider the economic impact, impact on existing structures and environmental risks on site.
- v) In the author's experience, early contractor and client involvement can provide significant benefits to the selection and optimisation of the remedial solutions. Collaboration between stakeholders and the designers can improve the understanding of the project risks and facilitate better management of these risks.

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